Section Shapes for Short-Span UHPC Bridges

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FOREWORD

The recent release of the AASHTO *Guide Specifications for Structural Design with Ultra-High Performance Concrete* (UHPC) enables the optimization of UHPC structural components by leveraging UHPC's enhanced material properties. One of the anticipated early entry points for owners and engineers to start designing and implementing UHPC structural components is in short-span bridges, where many advantages can be realized.

The information presented in this report provides background, context, and foundational knowledge to bridge owners, designers, and researchers interested in using UHPC structural components for short-span bridges. The report aims to help facilitate the implementation of UHPC structural components for short-span bridges by providing owners and designers information on section development and design considerations, suggested sections shapes for short-span bridges, and different circumstances where it is beneficial for UHPC structural components to be considered.

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Short-span bridge superstructures are commonly made of precast concrete elements, and those elements are proportioned according to the properties of conventional concrete. The emergence of ultra-high performance concrete (UHPC), and the release of the AASHTO Guide Specification for Structural Design with UHPC, creates new possibilities for the redesign of short-span bridge superstructures. This report provides a discussion on section development and design considerations for UHPC structural components, including background on some of the UHPC structural components used in the U.S. to date, some general limitations to a UHPC section geometry, typical controlling aspects of UHPC structural component design, and the effect of UHPC material properties on design. Three possible UHPC section shapes (multi-stem T-beam, optimized box beam, and optimized NEXT D beam) are presented for span lengths up to 125 feet. Design tables, suggested span lengths, and general design considerations are provided for each section. Several different hypothetical case studies are presented to demonstrate some of the advantages of using UHPC structural components in short-span bridges.</td></tr>
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NOTATION

ADT	average daily traffic
A_g	gross area (inch^2)
A_p	total area of prestressing strands (inch^2)
$A_{p,0.6in}$	area of 0.6-inch diameter prestressing strand (inch^2)
$A_{p,0.7in}$	area of 0.7-inch diameter prestressing strand (inch^2)
A_{ps}	total area of prestressing on flexural tension side of member (inch^2)
b_0	critical shear perimeter for two-way shear (inch)
b_s	support width (inch)
b_w	width of member's web (inch)
c_c	clear cover (inch)
d	distance from extreme compression face to centroid of non-prestressed reinforcement (inch)
d_e	depth of center of gravity of steel; effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (inch)
d_v	effective shear depth (inch)
E	modulus of elasticity (ksi)
E_c	modulus of elasticity of concrete or UHPC (ksi)
$E_{c,req'd}$	required modulus of elasticity of concrete or UHPC to meet optional live load deflection criteria (ksi)
E_s	modulus of elasticity of steel reinforcement (ksi)
E_p	modulus of elasticity of prestressing strands (ksi)
f_c	stress in extreme compression fiber; stress in concrete (ksi)
$f_{c,a}$	allowable compression stress (ksi)
$f_{c,SeI,m}$	stress in extreme compression face at midspan due to Service I load combination (ksi)
f'_c	compressive strength of concrete for use in design (ksi)
f'_{ci}	compressive strength of concrete at release for use in design (ksi)
f_{pe}	effective stress in prestressing (ksi)
f_{pu}	ultimate strength of prestressing strands (ksi)

f_{py} yield strength of prestressing strands (ksi)

f_r modulus of rupture of concrete (ksi)

f_s stress in steel (ksi)

f_{sy} yield strength of non-prestressed reinforcement (ksi)

f_t stress in extreme tension face (ksi)

$f_{t,a}$ allowable tensile stress (ksi)

$f_{t,i}$ stress in extreme tension face at release (ksi)

$f_{t,SeI}$ stress in extreme tension face due to Service I load combination (ksi)

$f_{t,SeI,m}$ stress in extreme tension face at midspan due to Service I load combination (ksi)

$f_{t,cr}$ effective cracking strength for UHPC (ksi)

$f_{t,cri}$ effective cracking strength for UHPC at time of transfer (ksi)

$f_{t,loc}$ crack localization strength for UHPC (ksi)

f_y yield strength of transverse reinforcement (ksi)

H average annual ambient relative humidity (percent)

h section height (inch)

h_c composite section height (inch)

h_{beam} beam height (inch)

I_g gross moment of inertia (inch^4)

K_1 correction factor for modulus of elasticity to be taken as 1.0 unless determined by a physical test, and as approved by the owner

K_3 correction factor for creep to be taken as 1.0 unless determined by physical tests, and as approved by the owner

K_4 correction factor for shrinkage to be taken as 1.0 unless determined by physical tests, and as approved by the owner

L span length (ft)

L_{beam} total beam length (ft)

L_{max} maximum possible span length (ft)

L_{span} span length (ft)

L_T total beam length (ft)

M	moment in member (kip-ft)
M_{cr}	cracking moment (kip-ft)
M_n	nominal flexural resistance (kip-ft)
$M_{n,m}$	nominal flexural resistance at midspan (kip-ft)
M_r	factored nominal flexural resistance (kip-ft)
$M_{r,m}$	factored nominal flexural resistance at midspan (kip-ft)
M_u	factored flexural demand (kip-ft)
n_{beams}	number of beams in superstructure
n_{spans}	number of spans
P_{wheel}	single wheel load for HS-20 truck (kips)
t_d	age of concrete or UHPC at time of deck placement or installation of beams (days)
t_f	flange thickness (inch), age of concrete or UHPC at final time (days)
t_i	age of concrete or UHPC at time of load application (days)
V_n	nominal shear resistance (kips)
V_r	factored nominal shear resistance (kips)
V_u	factored shear demand (kips)
v_c	nominal shear resistance provided by concrete for two-way members (kips)
v_n	nominal shear resistance for two-way members (kips)
v_u	shear stress due to factored shear force (kips)
W_{beam}	total beam weight (kips)
W_{bridge}	total weight of superstructure (including barrier weight) (kips)
W_{slab}	total slab weight per beam (kips)
$W_{superstructure}$	total weight of superstructure (kips)
w_{beam}	width of beam (inch)
w_c	unit weight of concrete or UHPC (kcf)
w_g	unit weight of beam (kip/ft)
w_s	unit weight of composite deck (kip/ft)
y_b	distance from bottom to centroid of gross section (inch)

α_u — reduction factor to account for the non-linearity of the UHPC compressive stress-strain response

γ_u — reduction factor to account for the variability in the UHPC tensile stress parameters

Δ — deflection (inch)

Δ_L — maximum live load deflection used for optional live load deflection check (inch)

Δ_{LL} — live load deflection; deflection due to lane load (inch)

Δ_{LT} — deflection due to design truck (inch)

Δ_{limit} — live load deflection limit (inch)

$\Delta_{sus,LT}$ — long-term deflection due to sustained loads (inch)

Δf_{pES} — prestress losses due to elastic shortening (ksi)

Δf_{pLT} — long-term prestress losses due to time dependent effects (ksi)

ε — general strain (inch/inch)

ε_c — strain in UHPC; net compressive strain in extreme compression fiber of the UHPC section (inch/inch)

ε_{cp} — elastic compressive strain limit (inch/inch)

ε_{cu} — ultimate compressive strain of conventional concrete or UHPC for use in design (inch/inch)

ε_{sh} — shrinkage strain (inch/inch)

ε_{ps} — strain in prestressed reinforcement (inch/inch)

ε_{pu} — ultimate strain of prestressed reinforcement, typically referred to as total elongation (inch/inch)

ε_{py} — strain in prestressed reinforcement at yield (inch/inch)

ε_s — net tensile strain in the extreme tension steel (inch/inch)

ε_{su} — ultimate strain of non-prestressed reinforcement, typically referred to as total elongation (inch/inch)

ε_t — net tensile strain in extreme tension fiber of the UHPC section (inch/inch)

$\varepsilon_{t,cr}$ — elastic tensile strain limit of UHPC corresponding to a tensile stress of $\gamma f_{t,cr}$ (inch/inch)

$\varepsilon_{t,loc}$ — crack localization strain of UHPC for use in design (inch/inch)

ε_y — yield strain for the reinforcement (inch/inch)

λ — concrete density modification factor for conventional concrete

λ_s size effect factor

ϕ resistance factor

ϕ_w hollow column reduction factor as defined in Article 5.6.4.7.2c of AASHTO LRFD BDS

τ shear stress (ksi)

Ψ creep coefficient

$\Psi(t, t_i)$ creep coefficient at time t for loading applied at t_i

ψ curvature (rad/inch)

CHAPTER 1. INTRODUCTION

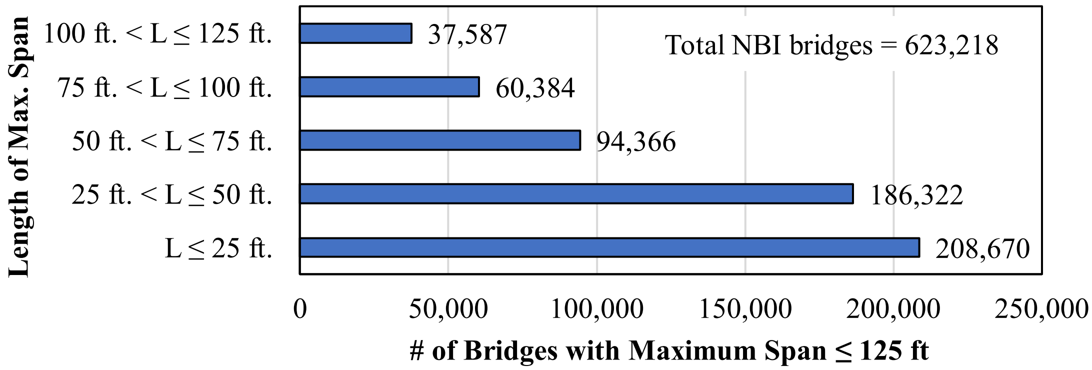
INTRODUCTION

The recent release of the AASHTO *Guide Specifications for Structural Design with Ultra-High Performance Concrete* (referred to hereafter as "UHPC Structural Design Guide") enables the optimization of ultra-high performance concrete (UHPC) structural components by leveraging the enhanced mechanical and durability properties of UHPC. Owners and designers are investigating different possible applications for UHPC structural components in the transportation industry, as these components offer many benefits over alternative options. Some of these benefits include the following.

- UHPC structural components can be optimized to be significantly lighter than conventional concrete section shapes, making them easier to transport and erect. Lighter sections may allow for more components to be loaded per truck and may enable the use of smaller cranes for placement. Wider components may also be possible, which would decrease the number of connections between elements and decrease the number of picks during erection.
- UHPC structural components can be designed to be stronger than conventional concrete components, allowing for shallower sections, longer spans, and wider girder spacings. Longer spans can facilitate the removal of intermediate piers on multi-span bridges. Wider girder spacings and shallower, optimized section shapes can create significantly lighter superstructures, which decreases the demand on the substructure.
- UHPC structural components can be designed with an integral deck, eliminating the need for a separate cast-in-place (CIP) concrete deck, which facilitates accelerated bridge construction.
- UHPC structural components are expected to be significantly more durable than conventional concrete or steel structural components, extending the service life of bridge superstructures by requiring less maintenance demands over the life of the structure.

Short-span bridges are likely an early entry point for UHPC superstructures for several reasons.

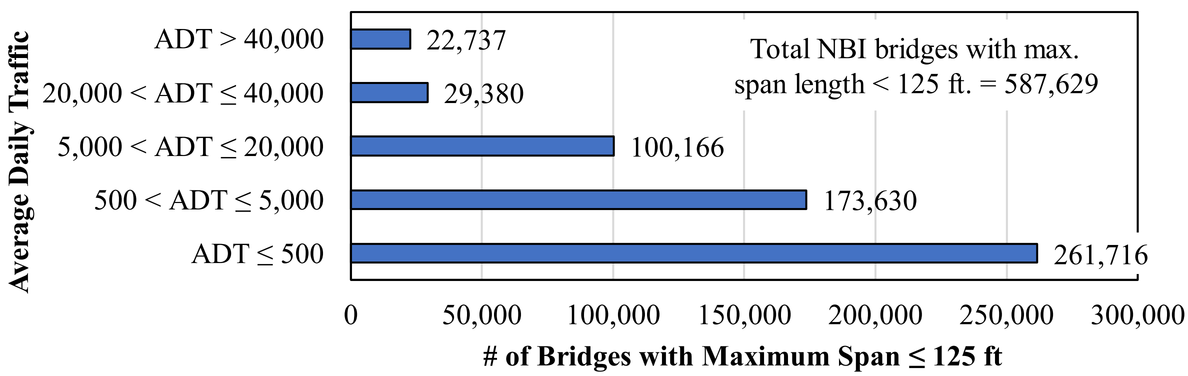
- Short-span bridges make up the largest portion of the national bridge inventory. Approximately 94 percent of the current inventory (587,629 of 623,218) has a maximum span length of less than 125 feet. A further break down of the maximum span lengths of the current national bridge inventory is shown in Figure 1.



Source: FHWA.

Figure 1. Graph. Number of NBI bridges with maximum span length less than 125 feet.

- A large portion of the short-span bridge inventory has a low average daily traffic (ADT), as shown in Figure 2. Approximately 44.5 percent of bridges with a maximum span length less than 125 feet have an ADT less than 500. These low ADT bridges are typically lower risk bridges for initial implementation of new technologies.



Source: FHWA.

Figure 2. Graph. Average daily traffic for bridges with maximum span length less than 125 feet.

- There are many short-span bridges that are past their original design life and in poor condition. Of the bridges in the national bridge inventory with a maximum span length less than 125 feet, more than 41 percent are more than 50 years old, approximately 10 percent have a rating factor less than 1.0, and approximately 4 percent have a superstructure condition rating of poor or worse.
- Shorter span bridges are typically less expensive and lower risk to construct than longer span bridges.
- Beams used for short-span bridges can possibly be cast using a single mix from the precasters batch plant or ready-mix truck. This would also reduce the risk for the precaster.

This report uses provisions in the UHPC Structural Design Guide and builds from the guidance presented in the *Structural Design with UHPC Workshop Manual* (FHWA-RC-24-0006). Several different UHPC solutions for short-span bridges are presented along with background information on their development.

OBJECTIVES OF REPORT

The overall objective of this report is to help facilitate the implementation of UHPC structural components for short-span bridges and provide background on how owners and designers can create optimized UHPC sections. Four different possible UHPC cross sections are suggested for short-span bridges with their advantages highlighted in several hypothetical case studies.

REPORT OVERVIEW

Chapter 2 of this report first provides a discussion on section development and design considerations for UHPC structural components. This discussion includes a background on a few of the UHPC structural components used in the U.S. to date, general limitations on UHPC section geometry, typical controlling aspects of UHPC structural component design, and the effect of UHPC material properties on design. Next in Chapter 3, four possible UHPC section shapes for short-span bridges are presented: a UHPC multi-stem T-beam, an optimized UHPC box beam, an optimized voided slab beam, and an optimized UHPC NEXT D beam. Design tables, suggested span lengths, and general design considerations for three of the sections are provided. In Chapter 4, several different hypothetical case studies are presented to demonstrate a number of the advantages of using UHPC structural components in several different circumstances.

CHAPTER 2. SECTION DEVELOPMENT AND DESIGN CONSIDERATIONS

INTRODUCTION

UHPC structural elements are expected to offer clear advantages when designed for use in short-span bridges. This chapter begins by providing an overview of four different UHPC bridges that were constructed in the U.S. It provides details on their section shapes, span configurations, and other construction aspects to showcase what has been successfully achieved with UHPC to date. Afterwards, a discussion on the general limitations for UHPC section shapes is provided to assist with decisions on UHPC section optimization and development. In addition, the general design considerations for UHPC structural elements are surveyed with specific details provided on controlling failure mechanisms and the influence of UHPC material properties on the design of UHPC structural elements. Finally, other considerations related to adjacent members used for short-span superstructures are discussed, including options for the riding surface and differential camber between adjacent members.

SAMPLE OF PREVIOUSLY USED UHPC SECTION SHAPES

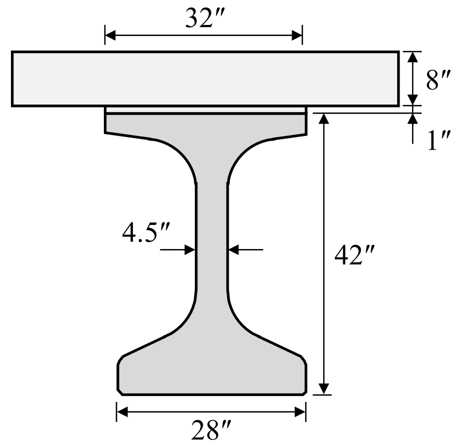
A sample of previously constructed UHPC superstructures is summarized in Table 1 and Figure 3 through Figure 6.

Table 1. Details for selection of previously constructed UHPC bridges in U.S.

Bridge	State	Year	Girder Type	Girder Depth	Deck	Span Length
Mars Hill Bridge, Wapello County	IA	2006	Iowa bulb-tee (pretensioned)	42 inches	CIP, CC, 8-inch	110 feet
Jakway Park Bridge, Buchanan County	IA	2008	Pi Girder (2^{nd} Generation, pretensioned)	33 inches	None	50 feet
Deacon Avenue Bridge, Buchanan County	IA	2015	Hawkeye Pi Girder (post-tensioned)	28 inches	None	52 feet
Bricker Road Bridge, St. Clair County	MI	2022	Triple-tee, ribbed deck (pretensioned)	13.5 inches	None	23.7 feet

The first UHPC structural elements used in a bridge in the U.S. were for the Mars Hill Bridge in Wapello County, Iowa, shown in Figure 3. This bridge consisted of three pretensioned UHPC bulb-tee girders with the web of the bulb-tee girder made thinner to take advantage of the enhanced tensile properties of UHPC, shown in Figure 3 (b). The 42-inch deep UHPC girder was topped with a 1-inch haunch and 8-inch cast-in-place (CIP) conventional concrete deck and spanned 110 feet. The bridge was opened to traffic in 2006. More details on this project can be found in Wipf et al. (2011).

Mars Hill Bridge, Wapello County, Iowa (2006)



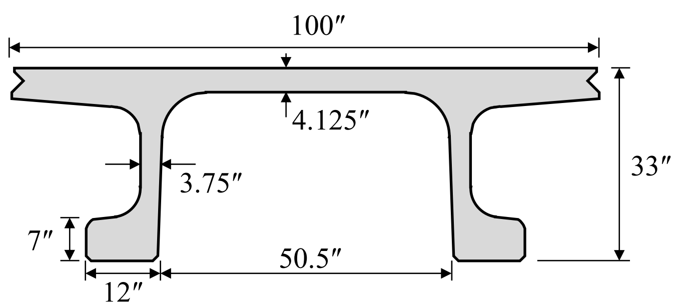
(a) (b)

Source: FHWA.

Figure 3. Photograph and Illustration. (a) Photograph and (b) section details for Mars Hill Bridge in Wapello County (IA) opened to traffic in 2006.

An optimized UHPC cross section was developed based on several research efforts, see Graybeal (2009). This pi-shaped section was used for the Jakway Park Bridge in Buchanan County, Iowa, shown in Figure 4. This bridge consisted of three pretensioned 33-inch-deep pi-girders spanning 50 feet. No CIP conventional concrete (CC) deck was used for this bridge, with the top flange of the UHPC girder serving as the riding surface. The top flange thickness (4.125 inches) and clear spacing between webs (50.5 inches) were determined based on the design for transverse flexural demands and stability of the section. The design required a 4-inch-thick top flange with the additional 0.125 inches included to accommodate potential air bubbles rising to the top and the potential need for surface grinding. Adjacent beams were connected using a grouted shear key between the top flanges and transverse HSS sections were installed to connect the bottom flanges at several sections along the length of the bridge. The bridge was opened to traffic in 2008. More details on the Jakway Park Bridge can be found in Wipf et al. (2011), Rouse et al. (2011), and Keierleber et al. (2010).

Jakway Park Bridge, Buchanan County, Iowa (2008)

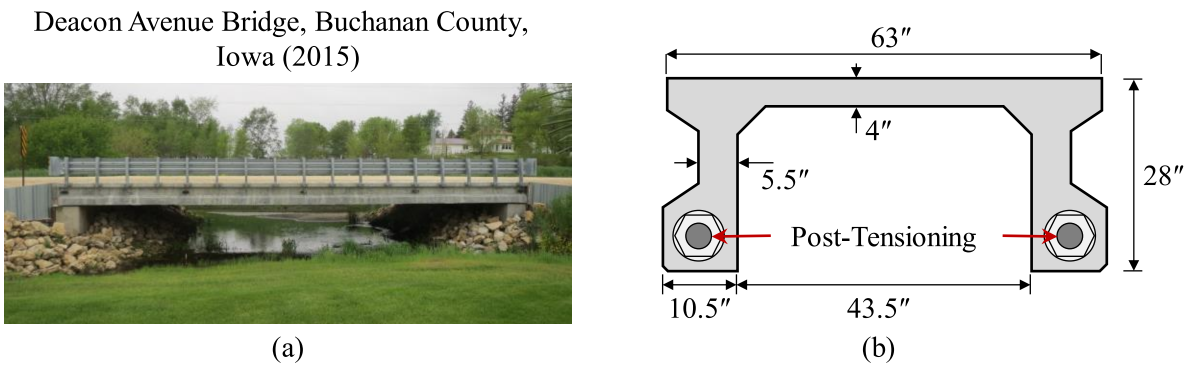


(a) (b)

Source: FHWA.

Figure 4. Photograph and Illustration. (a) Photograph and (b) section details for Jakway Park Bridge, Buchanan County (IA) opened to traffic in 2008.

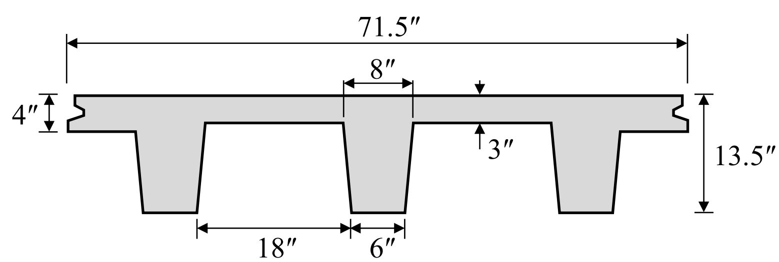
UHPC pi-girders were used again in Iowa for the Deacon Avenue Bridge in Buchanan County, shown in Figure 5. This bridge used a modified 28-inch-deep pi-girder shape with post-tensioned strands, shown in Figure 5 (b), to span 52 feet. The bridge opened to traffic in 2015. More information on this project can be found in Keierleber et al. (2015) and Shafei et al. (2019).



Source: FHWA.

Figure 5. Photograph and Illustration. (a) Photograph and (b) section details for Deacon Avenue Bridge, Buchanan County (IA) opened to traffic in 2015.

A UHPC triple-tee section shape (also called a ribbed deck or multi-stem T-beam) was used for the Bricker Road Bridge in St. Clair County, Michigan, shown in Figure 6. This bridge consisted of six 13.5-inch-deep conventionally reinforced triple-tee beams with the section shape shown in Figure 6 (b) to span 23.7 feet. The bridge was opened to traffic in 2022. More details on the project can be found in Hazelton et al. (2023) and El-Tawil and Hazelton (2024).



Source: FHWA.

Figure 6. Illustration. Section details for Bricker Road Bridge, St. Clair County (MI) opened to traffic in 2022.

The common themes from previous applications in the U.S. are thinner webs, generally shallower bridge profiles, and thin top flanges that can also be used as the riding surface for the bridge (for adjacent beam systems).

GENERAL SECTION LIMITATIONS

Cover Requirements (UHPC Structural Design Guide Article 10.1)

Some of the practical limitations on the section geometry are based on the cover requirements in UHPC Structural Design Guide Article 10.1.

> *The minimum cover shall not be less than the greater of 1.5 times the length of the longest type of fiber reinforcement included in the UHPC or 0.75 inches, unless adequate fiber distribution can be otherwise demonstrated for a specific application.*

The typical length of fiber reinforcement used in UHPC is 0.5 inches, but the UHPC Structural Design Guide Article C10.1 suggests that "… cover can be based on a conservative assumption of a 1-inch fiber length" if specific fiber length information is not available. This means that the cover should be a minimum of 0.75 inches for 0.5-inch fibers or 1.5 inches if the fiber length is not known but expected to not exceed 1 inch.

Design Considerations for Top Flange

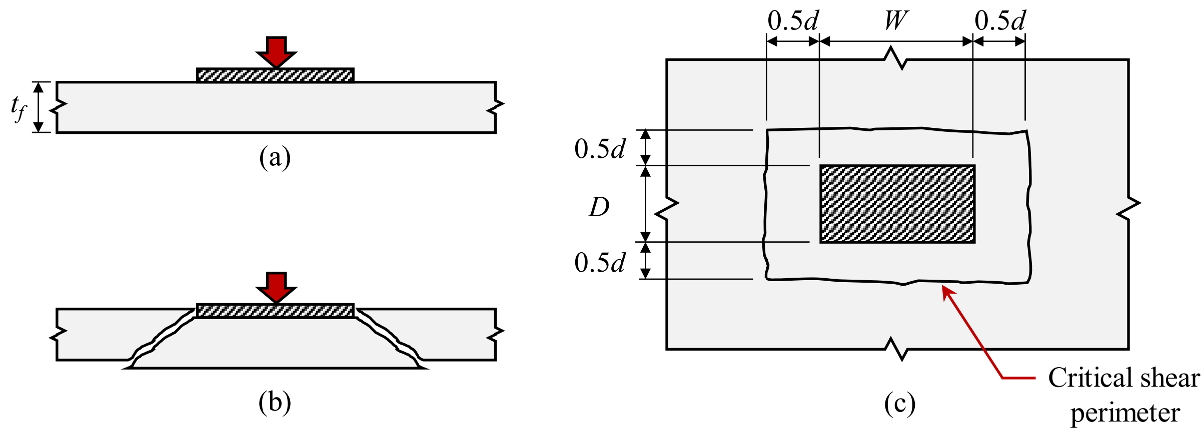
If the top flange of a beam is to be used for the riding surface, several additional design checks should be considered.

- **Transverse flexure**: The transverse flexural capacity of the top flange must be designed to ensure acceptable performance. Supporting elements (e.g., stems, webs) and intermittent breaks (e.g., field-cast connections between precast elements) should be considered, as these features affect the boundary conditions and thus the capacity of the plate. The design should align with the flexural resistance provisions in UHPC Structural Design Article 6. The use of the strip method, similar to that commonly used for the design of one-way bending of a conventional concrete plate supported by intermittent beams, may be acceptable.

- **Punching shear**: The punching shear capacity should be evaluated for the 10-inch by 20-inch tire footprint for thin top flanges that will be used as the riding surface. Punching shear is generally not evaluated for decks designed using the AASHTO *LRFD Bridge Design Specifications* (BDS) and is not covered by the UHPC Structural Design Guide. ACI 318-19 Article 22.6 on two-way shear strength could be used with a few modifications as a starting point for the evaluation of punching shear in the top flange. Some of the key parameters for punching shear are highlighted in Figure 7.

  Nominal shear strength for two-way members: $v_n = v_c$ ACI 318-19 Eqn. 22.6.1.2

  The nominal two-way shear strength is evaluated on a critical shear perimeter, highlighted in Figure 7 (c), which is based on the effective shear depth, *d*.



Source: FHWA.

Figure 7. Illustration. Punching shear failure mechanism with critical shear perimeter highlighted.

It is reasonable to assume the effective shear depth equal to 0.72h for UHPC sections, based on the minimum d_v definition in AASHTO LRFD BDS.

Assumed effective shear depth for two-way shear: $d = 0.72h$

The two-way shear strength is calculated using ACI 318-19 Table 22.6.5.2. For this design case, the strength can be assumed to be equal to the following.

Two-way shear resistance: v_c = minimum of ($v_{c(a)}$, $v_{c(b)}$, and $v_{c,(c)}$)

ACI 318-19 Table 22.6.5.2 (a)

where:

$$v_{c(a)} = 4\lambda_s\lambda\sqrt{f'_c}$$

$$v_{c(b)} = (2 + 4 / \beta)\lambda_s\lambda\sqrt{f'_c}$$

$$v_{c(c)} = (2 + \alpha_s\, d / b_0)\lambda_s\lambda\sqrt{f'_c}$$

In general, the direct tensile strength of concrete is assumed to be equal to $4\sqrt{f'_c}$, which is analogous to the design effective cracking strength for the UHPC, $f_{t,cr}$. Therefore, $f_{t,cr}$ can be reasonably used in place of $4\sqrt{f'_c}$ in ACI 318-19 Table 22.6.5.2 for the concrete direct tension strength. A reduction factor should be applied to account for the bi-directional tension in the UHPC; a reduction factor of 0.5 may be appropriate, but more research is needed. The size effect factor, λ_s, can be assumed equal to 1.0 for UHPC as the steel fibers are expected to mitigate size effect. Finally, the λ factor is the lightweight concrete factor and should be set equal to 1.0 for UHPC.

Previously constructed UHPC sections have used approximately 4-inch-thick top flanges with spacing between webs of between 43.5 inches and 50.5 inches.

Stability Considerations

UHPC structural elements will likely be more slender compared to similar depth conventional concrete elements, with narrower webs and possibly narrower top and bottom flanges, which may lead to stability concerns. There is currently no guidance related to specific stability considerations for UHPC members, such as lateral torsional buckling or web bend buckling. These behaviors may control some geometric aspects of short-span bridge members, such as the web width, top flange thickness, and spacing of webs. Until more experience is gained, and more guidance developed, a designer may need to consider a specialized stability analysis (e.g., three-dimensional finite element analysis) to ensure stability is not a concern.

Casting Volume

There may be limitations related to casting volume for a component. Precasters will initially perceive greater risk in association with large volume UHPC casts. It may be advantageous to work with local precasters to understand how much UHPC volume they can mix in their batch plants or ready-mix trucks.

As an example, if a precaster can comfortably mix and place 10 cubic yards of UHPC, it would be advantageous to try and design a UHPC component to require less than 10 cubic yards of material. One of the proposed box beam sections has a gross area of 554.1 inch^2. This box beam could be 65 feet long to have a volume less than 10 cubic yards, including 7 percent for quality control samples, waste, and loss.

Volume required for a 65-foot-long, 33-inch-deep box beam:

$$(554.1 \text{ inch}^2)(65 \text{ ft})(12 \text{ inch} / 1 \text{ ft}) = 432{,}198 \text{ inch}^3 = 9.26 \text{ cubic yards}$$

The area, depth, and length of a UHPC member could be optimized to keep under volume limits to allow a precaster to cast an entire component in a single cast.

DESIGN CONSIDERATIONS FOR UHPC STRUCTURAL ELEMENTS

The basic design steps for a UHPC structural elements are similar to conventional concrete structural elements as follows.

- **Step 1**: Determine initial bridge layout and girder sections.
- **Step 2**: Calculate moment and shear diagrams.
- **Step 3**: Estimate steel reinforcement (prestressed or non-prestressed)
- **Step 4**: Check service and fatigue limit states (stress and strain limits).
    - At transfer of prestressing for prestressed members.
    - At service due to sustained and total loads.
    - Check web shear stresses.
- **Step 5**: Check strength limit states.
    - Nominal flexural resistance and minimum flexural reinforcement.
    - Nominal shear resistance and minimum longitudinal reinforcement.
- **Step 6**: Check deflections (if required).

The general design procedure for UHPC structural elements is discussed in *Structural Design with UHPC Workshop Manual* (FHWA-RC-24-0006).

Typical Controlling Mechanisms for Beam Design

For typical prestressed beam elements, there are several different aspects of the design that may typically control. These aspects are generally similar to conventional concrete prestressed elements.

- **Release Stress at Top of Beam at Transfer Length**: Similar to conventional concrete pretensioned elements, the stresses in the top of the beam at the time of prestress transfer may exceed the allowable stresses at release at beam ends. The tensile stresses at release are limited to $\gamma_u f_{t,cri}$ per UHPC Structural Design Guide Article 5.2.1.3a, and Article 9.1.2 specifies that, unless determined by physical tests, $f_{t,cri}$ shall not be taken as a value to exceed $0.75 f_{t,cr}$ when the compressive strength at release is less than or equal to 90% of the specified compressive strength for design.

- **Service Stress in Extreme Tension Fiber**: Stress in the extreme tension fiber is limited to $\gamma_u f_{t,cr}$ per UHPC Structural Design Guide Article 5.2.1.3b. This stress limit is tightened to $0.95\gamma_u f_{t,cr}$ in UHPC Structural Design Guide Article 5.2.3 for components subjected to cyclic stresses, which will apply for bridge beam elements. The Service I load combination according to AASHTO LRFD BDS Article 3.4.1 should be used to check the stresses in components subjected to cyclic stresses.

  Service stress in extreme tension fiber (Service I): $f_{t,SeI} \leq 0.95\gamma_u f_{t,cr}$

  This design check may control the required total area of prestressing strands or the required section depth for longer spans depending on the properties of the UHPC.

- **Minimum Flexural Reinforcement**: The minimum amount of flexural reinforcement is specified in UHPC Structural Design Guide Article 6.3.3. This design check may control the number of strands required for shorter spans within a possible span range for a section.

  Minimum flexural reinforcement: $M_{r,m} = \phi M_{n,m} \geq$ minimum of ($1.33M_u$ and M_{cr})

  Non-prestressed reinforcement may be added instead of additional prestressing strands to satisfy this requirement.

- **Nominal Flexural Resistance**: The nominal flexural resistance is calculated using UHPC Structural Design Guide Article 6.3.2 and must be greater than the demand calculated using the Strength I load combination from AASHTO LRFD BDS Article 3.4.1.

  Flexural strength check: $M_r = \phi M_n \geq M_u$

  This design check may control the required total area of prestressing strands or the required section depth for most designs depending on the properties of the UHPC.

- **Optional Live Load Deflection Criteria**: AASHTO LRFD BDS Article 2.5.2.6.2 provides an optional live load deflection check, where the live load deflection is calculated using AASHTO LRFD BDS Article 3.6.1.3.2.

  Live load deflection: Δ_L = maximum of Δ_{LT} and $(0.25\Delta_{LT} + \Delta_{LL})$

  Live load deflection limit: $\Delta_L \leq L / 800$

  UHPC structural elements may be shallower than conventional concrete elements for similar span lengths, which means that this deflection criteria may control for longer spans within a possible span range for a section.

- **Nominal Shear Resistance**: The nominal shear resistance is calculated using UHPC Structural Design Guide Article 7.3 and must be greater than the demand calculated using the Strength I load combination from AASHTO LRFD BDS Article 3.4.1.

  Shear strength check: $V_r = \phi V_n \geq V_u$

  In particular, this design check may control the minimum web width for deeper sections with single webs (e.g., deep bulb tee sections) and the amount of transverse shear reinforcement, if needed.

The aspect of design that controls the section depth and required prestressing will be dependent on the UHPC material properties and section shape.

Material Properties and Design Implications

In conventional concrete design, the primary property to specify is the design compressive strength, f'_c. Designers must also specify a minimum compressive strength at release, f'_{ci}, which typically dictates the type of concrete used and concrete age at which the prestressing force can be transferred to the concrete. A higher release or design compressive strength may be specified to meet the stress limits and strength requirements. The contractor or producer must then provide a concrete mixture capable of achieving the design strength specified.

For UHPC components, additional material properties are required, and the process for determining their design values differs slightly from conventional methods. These additional properties include the effective cracking strength, $f_{t,cr}$, the crack localization strength, $f_{t,loc}$, and the crack localization strain, $\varepsilon_{t,loc}$. Designers must also specify the effective cracking strength at release, $f_{t,cri}$, if it is known from testing. However, this value need not be specified when the specified UHPC compressive strength at release is equal or less than 90 percent of the specified UHPC compressive strength, i.e., when $f'_{ci} \leq 0.9f'_c$, as designers can assume the value of $f_{t,cri}$ to be equal to $0.75f_{t,cr}$.

The process for determining the material property values for use in UHPC design involves qualification and acceptance procedures. A UHPC material supplier or precast plant should qualify their UHPC mixture to achieve specific design material properties values. The designer selects material properties based on the available results of the qualified mixtures. The maximum design values selected by the designers for a qualified mixture must be chosen based on statistical procedures that account for variability in the test results and the probability of failure. The contractor or producer would then perform acceptance testing to ensure that the UHPC used in the structural element meets the design material properties identified during qualification testing.

As with conventional concrete, specifying higher mechanical properties for a design can offer structural optimization advantages, potentially reducing material volumes. However, this may also decrease the number of available qualified UHPC mixtures meeting the criteria, which can increase the cost of material per unit volume. Conversely, specifying significantly lower property values than those of available qualified UHPC mixtures may lead to less economical designs that could be further optimized otherwise. Therefore, it is recommended that designers select design values that align with the qualifications of the available mixtures in their market.

There are other UHPC material properties that are allowed to be replaced by measured values as approved by an owner. These include the following.

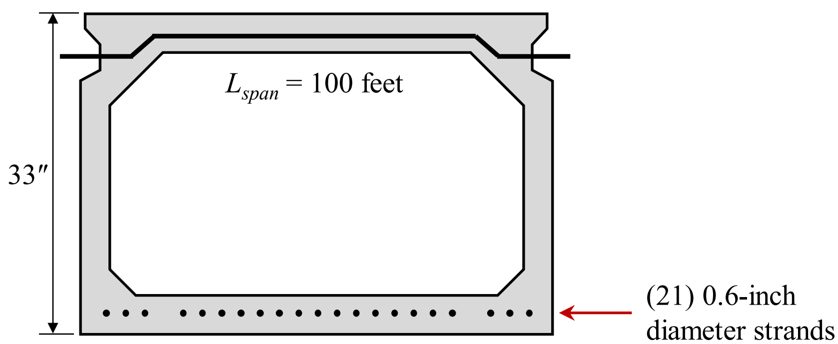
- **Modulus of elasticity (E_c modified by K_1)**: The modulus of elasticity for UHPC may be determined by physical tests in accordance with ASTM C1856 (UHPC Structural Design Guide Article 4.2.3). The K_1 correction factor facilitates the use of measured values for design. A higher modulus of elasticity will decrease the live load deflections, which may allow a design to meet the optional live load deflection criteria where it would not have otherwise.
- **Compression creep (Ψ modified by K_3)**: The creep coefficient may be measured from physical testing in accordance with ASTM C512 with modifications from ASTM C1856 and a sustained compressive stress of $0.65f'_c$ (UHPC Guide Article 4.2.8.2). These results can be used to calculate the K_3 correction factor. The K_3 correction factor can also be determined based on K_1: $K_3 = 1 / K_1$. A smaller creep coefficient will lead to smaller prestress losses and a higher effective stress to use in stress calculations. This can reduce the number of required strands for designs controlled by service stress in the extreme tension fiber.
- **Shrinkage strain (ε_{sh} modified by K_4)**: The shrinkage strain may be measured by physical testing according to ASTM C1856 (UHPC Guide Article 4.2.8.3). These results can be used to calculate the K_4 correction factor. Similar to the creep coefficient, a smaller shrinkage strain will lead to smaller prestress losses and a higher effective stress to use in stress calculations, which can reduce the number of required strands for designs controlled by service stress in the extreme tension fiber.
- **Effective cracking strength at release ($f_{t,cri}$)**: The effective cracking strength at release may be measured using the same method as the design effective cracking strength, according to AASHTO T 397-22. A higher effective cracking strength at release is beneficial for heavily prestressed sections where high tensile stresses are expected in the top fiber at release.
- **Ultimate compressive strain (ε_{cu})**: The ultimate compressive strain may be measured by physical testing in accordance with ASTM C1856 (UHPC Structural Design Guide Article 4.2.4.2). This would affect a design in which the nominal flexural resistance is controlled by concrete crushing and where the design is controlled by flexural strength. Most designs will be controlled by a crack localization flexural failure; changing the ultimate compressive strain will not affect these designs.

Measuring these material properties may or may not offer benefits to the design. However, if the designer has concern that a UHPC product's performance might differ from that predicted by the models (e.g., significantly more creep than predicted), alternative modeling or testing should be conducted. The use of more accurate values at the design stage will allow for more accurate designs and analyses.

There is an additional consideration for UHPC members with composite, cast-in-place decks. For the conventional concrete deck, the design compressive strength is specified, and all other properties assumed or calculated based on the compressive strength. Crushing of the concrete in the conventional concrete deck may control the flexural design capacity, thus, increasing the specified design compressive strength for the conventional concrete would increase the capacity of the member in these cases. Many UHPC-based short-span bridge solutions would not have a conventional concrete deck, so this would not be a concern.

Examples of UHPC Box Beam Designs with Different Material Properties

This section explores the impact of various UHPC material properties on the design considerations by analyzing a modified box beam section with a 33-inch depth and 100-foot span length, shown in Figure 8. Additional box beam section depths will be further explored in Chapter 3, which also provides details on the bridge cross-section used in the analyses presented herein.



Source: FHWA.

Figure 8. Illustration. Cross section geometry and strand layout of Base Design of 33-inch-deep modified box beam section and 100-foot span length for material properties analysis.

The Base Design for this analysis is considered Case #1 with the strand layout shown in Figure 8. The UHPC material properties for the Base Design are as follows.

- Design compressive strength: f'_c = 17.5 ksi
- Correction factor for modulus of elasticity: K_1 = 1.0
- Effective cracking strength: $f_{t,cr}$ = 1.0 ksi
- Crack localization strength: $f_{t,loc}$ = 1.0 ksi
- Crack localization strain: $\varepsilon_{t,loc}$ = 0.0035
- Creep correction factor: K_3 = 1.0
- Shrinkage correction factor: K_4 = 1.0

The Base Design meets all required service limit state, fatigue limit state, and strength limit state design checks, but does not meet the optional live load deflection criteria for the 100-foot span length.

Span length: L_{span} = 100 feet = 1,200 inches

Deflection from lane live load: Δ_{LL} = 0.71 inches

Deflection from design truck: Δ_{LT} = 1.61 inches

Live load deflection: Δ_L = maximum of Δ_{LT} and $(0.25\Delta_{LT} + \Delta_{LL})$
Δ_L = maximum of 1.61 inches and 1.11 inches = 1.61 inches

Live load deflection limit: Δ_L = 1.61 inches $\leq L$ / 800 = 1.50 inches → No Good

Thirteen analyses and designs were performed to highlight the impact of different UHPC material properties on design, summarized in Table 2. The comparisons provided are based on the Base Design for Case #1.

Table 2. Details on analyses for material property impact study.

Case #	Description	f'_c (ksi)	K_1	$f_{t,cr}$ (ksi)	$f_{t,loc}$ (ksi)	$\varepsilon_{t,loc}$	K_3	K_4
1	Base Design	17.5	1.0	1.00	1.00	0.0035	1.0	1.0
2	Increase K_1 to meet deflection limit	17.5	1.1	1.00	1.00	0.0035	1.0	1.0
3	Decrease $f_{t,cr}$ and $f_{t,loc}$ to the minimum threshold values (no modification to strands)	17.5	1.0	0.75	0.75	0.0035	1.0	1.0
4	Case 3 with increased strands to meet check	17.5	1.0	0.75	0.75	0.0035	1.0	1.0
5	Decrease $\varepsilon_{t,loc}$ to the minimum threshold value (no modification to strands)	17.5	1.0	1.00	1.00	0.0025	1.0	1.0
6	Case 5 with increased strands to meet check	17.5	1.0	1.00	1.00	0.0025	1.0	1.0
7	Decrease $f_{t,cr}$, $f_{t,loc}$, and $\varepsilon_{t,loc}$ to the minimum threshold values (no modification to strands)	17.5	1.0	0.75	0.75	0.0025	1.0	1.0
8	Case 7 with increased strands to meet check	17.5	1.0	0.75	0.75	0.0025	1.0	1.0
9	Increase $\varepsilon_{t,loc}$ to upper range (no modification to strands)	17.5	1.0	1.00	1.00	0.0060	1.0	1.0
10	Case 9 with modified strand pattern	17.5	1.0	1.00	1.00	0.0060	1.0	1.0
11	Decrease K_3 and K_4 (no modification to strands)	17.5	1.0	1.00	1.00	0.0035	0.6	0.6
12	Case 11 with modified strand pattern	17.5	1.0	1.00	1.00	0.0035	0.6	0.6
13	Increase f'_c to upper range (no modification to strands)	24.0	1.0	1.00	1.00	0.0035	1.0	1.0

A summary of the results from these designs is provided in Table 3. Some observations from these analyses are as follows.

- **K_1 can be increased to meet optional live load deflection limits**: K_1 was increased to 1.1 in Case #2, which directly increased E_c by 10 percent. Increasing E_c had a slight effect on the prestress losses (with the total losses decreasing from 42.8 ksi to 41.5 ksi), which led to a small decrease in the stresses (with f_b decreasing from -0.530 ksi to -0.517 ksi) and small increase in the nominal flexural resistance (with M_r increasing from 2,667 kip-ft to 2,697 kip-ft). The largest effect was on the live load deflection calculations, where a 10 percent increase in E_c led to a 10 percent decrease in the live load deflections (with live load deflections decreasing from 1.61 inches to 1.46 inches). This reduced live load deflection is less than the optional live load deflection limit of 1.5 inches for the 100-foot span, meeting the criteria.

- **Decreasing $f_{t,cr}$ and $f_{t,loc}$ affects stress limits and decreases M_r**: $f_{t,cr}$ and $f_{t,loc}$ were decreased to the minimum allowable values from UHPC Structural Design Guide Article 1.1 of $f_{t,cr} = f_{t,loc} = 0.75$ ksi in Case #3 and Case #4. The same strand configuration as the Base Design was used in Case #3. The strand configuration was modified in Case #4 to satisfy all design requirements. Reducing $f_{t,cr}$ and $f_{t,loc}$ caused M_r to decrease below M_u and $f_{t,cri}$ to drop below the tensile stress at release. More strands were required to increase M_r. Two top strands were also required to satisfy the end region stress checks at release.
- **Reducing $\varepsilon_{t,loc}$ decreases M_r and creates issues with minimum flexural reinforcement requirement**: $\varepsilon_{t,loc}$ was reduced to the minimum allowable value from UHPC Structural Design Guide Article 1.1 of $\varepsilon_{t,loc} = 0.0025$ in Case #5 and Case #6. The same strand configuration as the Base Design was used in Case #5. The strand configuration was modified in Case #6 to satisfy all design requirements. Reducing $\varepsilon_{t,loc}$ caused M_r to decrease below M_u and resulted in a failure to meet the minimum flexural reinforcement requirement. Larger diameter strands were required to increase M_r and two top strands were needed to satisfy the increased end region stresses caused by the additional flexural tension strands. Non-prestressed reinforcement needed to be added on the flexural tension side to further increase M_r without increasing M_{cr}.
- **Decreasing $f_{t,cr}$, $f_{t,loc}$, and $\varepsilon_{t,loc}$ reduces stress limits and decreases M_r**: $f_{t,cr}$, $f_{t,loc}$, and $\varepsilon_{t,loc}$ were reduced to the minimum allowable values from UHPC Structural Design Guide Article 1.1 summarized in the previous two bullet points in Case #7 and Case #8. The same strand configuration as the Base Design was used in Case #7. The strand configuration was modified in Case #8 to satisfy all design requirements. Decreasing all the tensile properties to the minimum values further reduced M_r and introduced the issues discussed in the previous two bullet points.
- **Increasing $\varepsilon_{t,loc}$ increases M_r and changes the controlling design check**: $\varepsilon_{t,loc}$ was increased to the upper range of current commercially available UHPC materials ($\varepsilon_{t,loc} = 0.006$) in Case #9 and Case #10. The same strand configuration as the Base Design was used in Case #9. The strand configuration was modified in Case #10 to satisfy all design requirements. Increasing $\varepsilon_{t,loc}$ from 0.0035 to 0.006 led to an increase in M_r from 2,669 kip-ft to 3,090 kip-ft (a 15.7% increase). This change allowed for three less 0.6-inch prestressing strands in the design and changed the controlling design check from Strength I limit state to the Service I cyclic tensile stress limit.
- **Smaller values of K_3 and K_4 decrease the tensile stresses and increase M_r**: K_3 and K_4 were reduced to lower ranges of current commercially available UHPC materials ($K_3 = K_4 = 0.6$) in Case #11 and Case #12. The same strand configuration as the Base Design was used in Case #11. The strand configuration was modified in Case #12 to satisfy all design requirements. Using a smaller value of K_3 and K_4 decreased the long-term prestress losses (from 32.9 ksi to 21.9 ksi). This reduction in prestress losses led to a decrease in the extreme tension fiber service stress (from -0.530 ksi to -0.312 ksi) and an increase in M_r (from 2,669 kip-ft to 2,846 ksi). These effects allowed for two less 0.6-inch diameter prestressing strands as compared to the Base Design while still satisfying the design requirements.

- **Increasing f'_c decreases deflections**: f'_c was increased to the upper range of current commercially available UHPC materials (f'_c = 24.0 ksi) in Case #13. This increase in the compressive strength had little effect on the stress and strength calculations. The increased compressive strength led to a higher calculated modulus of elasticity and decreased live load deflections. The estimated live load deflections (Δ_L = 1.45 inches) were less than the optional live load deflection limit of 1.5 inches for the 100-foot span.

The design tables and resources developed in this report are based on minimum values for compression properties and typical values for tensile properties, as summarized for the Base Design. These values could be modified by states and designers based on needs, locally available materials, and the observations summarized above.

Table 3. Summary of analysis results for material properties study on 33-inch-deep modified UHPC box beam section [a].

Case #	A_{ps} [b] (inch^2)	Δf_{pES} (ksi)	Δf_{pLT} (ksi)	$f_{t,i}$ (ksi)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [c] (inch)	Δ_L [c] (inch)	Controlling
1	4.557	10.2	32.6	-0.660	-0.530	3.282	2,669	2,503	-1.32	1.61 [d]	Strength I
2	4.557	9.4	32.2	-0.666	-0.517	3.275	2,697	2,503	-1.27	1.46	Strength I
3	4.557	10.2	32.6	-0.660	-0.530	3.282	2,544 [e]	2,308	-1.32	1.61 [d]	Strength I
4	4.991 [f]	11.1	34.1	-0.332	-0.321	3.533	2,708	2,430	-1.18	1.61 [d]	Strength I
5	4.557	10.2	32.6	-0.660	-0.530	3.282	2,403 [e,g]	2,503	-1.32	1.61 [d]	Strength I
6	5.586 [f,h]	12.9	35.9	-0.270	0.013	3.566	2,741 [g]	2,823	-1.83	1.61 [d]	Strength I
7	4.557	10.2	32.6	-0.660	-0.530	3.282	2,284 [e,g]	2,308	-1.32	1.61 [d]	Strength I
8	5.880 [f,h]	13.9	36.6	-0.312	0.183	3.530	2,719 [g]	2,726	-2.28	1.61 [d]	Strength I
9	4.557	10.2	32.6	-0.660	-0.530	3.282	3,090	2,503	-1.32	1.61 [d]	Service I (Cyclic)
10	3.906	7.9	30.4	-0.563	-0.942	3.363	2,737	2,271	-0.24	1.61 [d]	Service I (Cyclic)
11	4.557	10.2	21.9	-0.660	-0.312	3.245	2,846	2,612	-1.36	1.61 [d]	Strength I
12	4.123	8.7	20.9	-0.596	-0.615	3.304	2,696	2,443	-0.77	1.61 [d]	Strength I
13	4.557	10.2	32.9	-0.660	-0.551	3.281	2,674	2,495	-1.18	1.45	Strength I

[a] All analyses have L_{span} = 100 feet. The maximum moment demand at midspan from Strength I load combination is $M_{u,m}$ = 2,657 kip-ft for all designs.
[b] 0.6-inch diameter strands were used for all designs unless otherwise noted.
[c] Positive deflection is downward; negative deflection indicates upward deflection.
[d] Deflections do not meet the optional live load deflection checks for this analysis (L / 800 = 1.50 inches).
[e] design does not have sufficient nominal flexural resistance to resist the design moment ($M_{r,m} < M_{u,m}$).
[f] Two fully stressed top strands at 31 inches from bottom were required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.
[g] Design does not meet minimum flexural reinforcement requirement. This is a challenge for UHPC with lower crack localization strains. Non-prestressed reinforcement should be added to the flexural tension side of the member to meet this requirement.
[h] 0.7-inch diameter strands were needed for this design.

Additional analyses were performed to investigate how the same changes in material properties would influence the maximum possible span length, L_{max}, for this section shape, assuming the optional live load deflection check was relaxed. The maximum span lengths were checked in 5-foot increments; further refinement may be possible. Results are summarized in Table 4.

Table 4. Maximum span lengths for 33-inch-deep modified UHPC box beam section with material properties from different cases in Table 3.

Related Case #	f'_c (ksi)	K_1	$f_{t,cr}$ (ksi)	$f_{t,loc}$ (ksi)	$\varepsilon_{t,loc}$	K_3	K_4	L_{max} [a] (ft)	Controlling
1	17.5	1.0	1.00	1.00	0.0035	1.0	1.0	120	Strength I
3, 4	17.5	1.0	0.75	0.75	0.0035	1.0	1.0	115	Strength I
5, 6	17.5	1.0	1.00	1.00	0.0025	1.0	1.0	110	Strength I [b]
7, 8	17.5	1.0	0.75	0.75	0.0025	1.0	1.0	105	Strength I [b]
9, 10	17.5	1.0	1.00	1.00	0.0060	1.0	1.0	125	Service I (Cyclic)
11, 12	17.5	1.0	1.00	1.00	0.0035	0.6	0.6	125	Strength I

[a] Deflections do not meet the optional live load deflection checks for these analyses.
[b] Design requires non-prestressed reinforcement on the flexural tension side of the member to satisfy the minimum flexural reinforcement requirements.

In summary, decreasing $f_{t,cr}$, $f_{t,loc}$, and/or $\varepsilon_{t,loc}$ will reduce the maximum possible span length for a section. Conversely, increasing $\varepsilon_{t,loc}$ or decreasing K_3 and K_4 will increase the maximum possible span length for a section.

Having a lower crack localization strain may make it challenging to satisfy the minimum flexural reinforcement requirements with only prestressing strands. Non-prestressed reinforcement could be added to the flexural tension side to increase the nominal flexural resistance without also increasing the cracking moment. This design step was not completed for the designs in Table 4. A note was added to highlight where additional non-prestressed reinforcement would need to be added to satisfy the minimum flexural reinforcement requirement.

OTHER CONSIDERATIONS FOR ADJACENT MEMBERS

Riding Surface

Different states use different riding surfaces for adjacent member bridges. For adjacent UHPC member bridges, the top of the beam can be used as the riding surface, or an overlay can be installed on top of the UHPC beams.

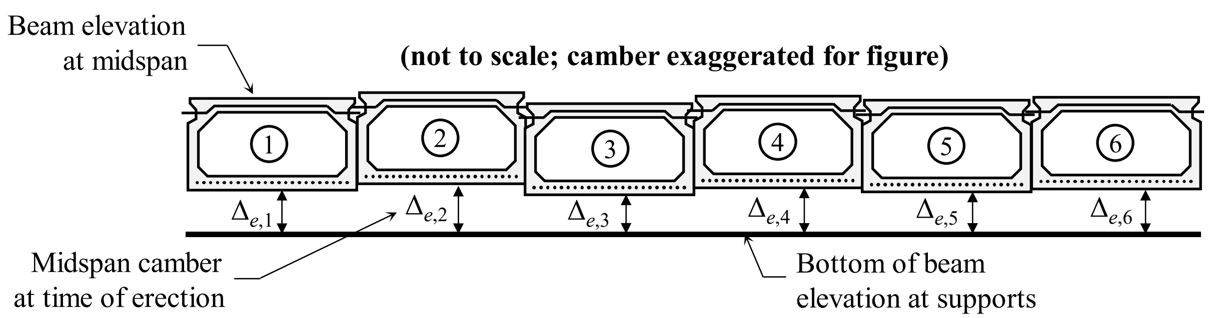
If the top of the beams and the field-cast connections are to be used as the riding surface, the beams and connections should be cast to include an additional sacrificial thickness on the top flange (e.g., $^1/_4$ to $^1/_2$ inch). This additional thickness will allow for grinding and grooving of the beams after installation. Note that the superstructure cross section should be designed and analyzed accounting for the dead weight of the sacrificial layer, while considering a structural depth that excludes its thickness.

If an overlay is going to be installed on top of the UHPC beams, the weight of the overlay must be included in the design calculations.

The designs included in Chapter 3 show the structural depths, which exclude any extra material that would be removed through grinding or grooving and include the weight of a 2-inch-thick future asphalt overlay.

Differential Camber

One common concern for conventional concrete bridges constructed with adjacent members without cast-in-place, composite decks is the differential camber between adjacent members. Different beams may have different cambers at the time of erection, as shown in Figure 9, which can lead to difficulty with connection construction and an uneven riding surface if not planned for or corrected.



Source: FHWA.

Figure 9. Illustration. Possible differential camber between adjacent members at time of erection.

A complete description of the camber mechanism and the estimation of camber is outside the scope of this report. More details on camber and deflection calculations can be found in PCI (2024) for conventional concrete and FHWA-RC-24-0006 for UHPC. More details on controlling camber and sweep for conventional concrete in general are provided in the PCI MNL-137.

There are several possible solutions for planning for and correcting differential camber between adjacent members, some of which are as follows. Most of these are applicable to both conventional concrete and UHPC adjacent members, but some are specific to UHPC members.

- **Determine a confidence interval for camber**: UHPC material properties may be better defined than conventional concrete properties at the design stage due to the material qualification process. The designer may be able to determine a possible range and variability of compressive strengths and stiffnesses of the UHPC material at release to refine camber estimates and determine a confidence interval for camber. The combined design, tolerances, and construction process could then be based on the differential camber that may result from the confidence interval.
- **Review construction processes to achieve consistency between beam casts**: The design and construction team can review the construction process for the beams to ensure consistency between beam casts. Some of these processes include the process before transfer (e.g., curing process and time), time of release of pretensioned strands (i.e., material properties at time of transfer), and storage of beams (e.g., temperature, humidity, sun exposure, dunnage locations). Differences in these processes between beam casts may

lead to different cambers occurring in the beams. Camber can also be measured on each of the beams throughout the storage process to determine if any corrective actions need to be taken.

- **Plan for a supplemental riding surface**: A supplemental riding surface can be added to the bridge after casting of the joints to accommodate minor differential camber between members.
- **Include a sacrificial layer in the precast members**: A sacrificial layer can be included in the overall height of the member to possibly grind off after erection and connection casting for members that have a higher elevation than others. This can be done using traditional roadway grinding equipment. The weight of the sacrificial layer should be included in the beam design calculations (in case the entire layer is not ground off), but the thickness of the sacrificial layer should not be included in the resistance calculations (in case the entire layer is ground off).
- **Adjust support locations and add weight to members at precast plant**: It may be possible to affect the final camber by changing the moment in the beam during storage. This can be done by adjusting the support locations and/or adding weight on top of the beam. It may be difficult to estimate the effect this will have on the camber at the time of erection.
- **Adjust support height**: Different support heights can be used for different beams to correct differential camber at midspan. However, this correction may lead to differential elevations between adjacent members toward the beam ends.
- **Temporary hold down of beam during connection casting**: A temporary hold down (possibly a strongback or heavy weight placed on a beam) can be used to align a beam with larger camber with adjacent members during connection casting. This solution may be possible to accommodate minor differential camber, however, designers need to calculate stresses induced by the imposed deformation. More details on this type of correction can be found in Chitty and Garber (2021).

Additionally, the connection between adjacent members should be detailed to accommodate the allowable differential camber. The connection may need to be made taller so there is more vertical tolerance in the connection for the lap-spliced reinforcement.

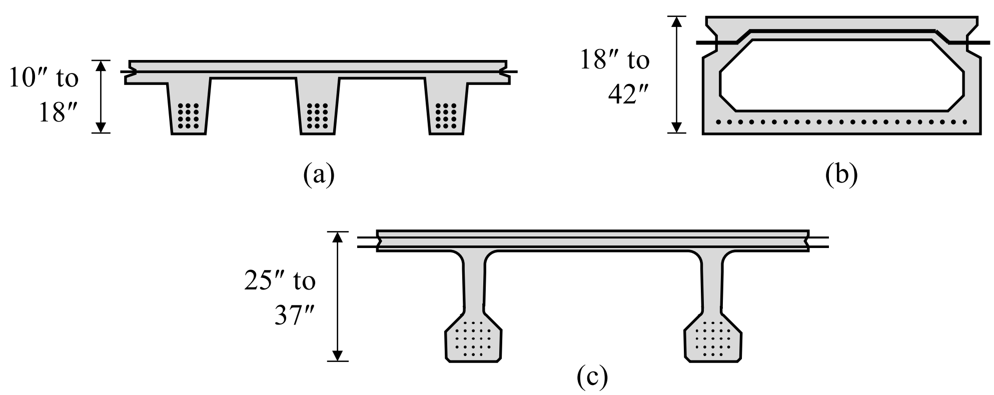
Different combinations of these possible solutions have been used successfully to control and accommodate differential camber in conventional concrete bridges constructed with adjacent members without cast-in-place, composite decks.

CHAPTER 3. SHORT TO MEDIUM SPANS: ADJACENT ELEMENTS

INTRODUCTION

Short- to medium-span bridges with span lengths between 20 and 125 feet are expected to be a common application for UHPC structural components. As described in Chapter 2, the four example UHPC bridge application in the U.S. had span lengths within this 20 to 125-foot range. UHPC superstructures in this span range can be constructed without a cast-in-place (CIP) conventional concrete deck simplifying and accelerating construction. This approach also allows for the entire superstructure to be constructed out of UHPC, further enhancing its durability.

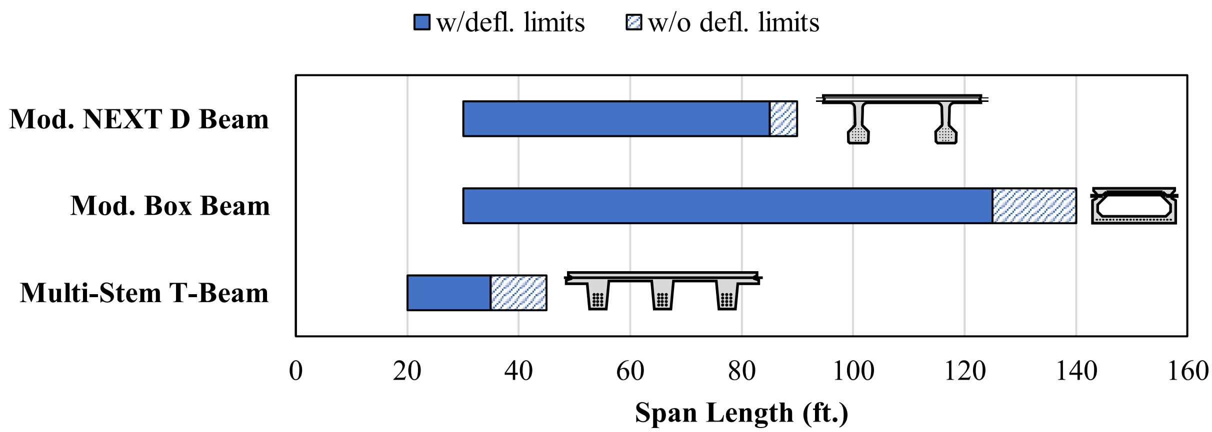
Three cross sections will be investigated in this chapter: (1) multi-stem T-beams, (2) box beams, and (3) modified NEXT D beams, as shown in Figure 10. A version of the multi-stem T-beam section was previously used for a UHPC superstructure in Michigan. Although multi-stem T-beams are not necessarily structurally efficient section shapes, they may prove to be an easily constructable option making them a suitable entry-level application for UHPC. On the other hand, box beam sections can be optimized to effectively leverage the enhanced mechanical properties of UHPC. They also feature a wide bottom flange that not only accommodates a large number of prestressing strands in the bottom layer at a large eccentricity but also increases the area of UHPC in tension, thereby increasing its contribution to the resisting tensile forces. NEXT D beams (PCINE, 2021) are a popular section because of their ease of fabrication making them a practical option for UHPC. NEXT D beam forms can accommodate geometrical modifications to create a more efficient section when utilized for UHPC. Design aids will be presented for different depths and variations of each section. Voided slab beams are also discussed as these types of members are popular in many parts of the U.S. Complete design aids are not provided for voided slab beams as they have many similarities to box beams.



Source: FHWA.

Figure 10. Illustration. Possible options for short- to medium-span UHPC superstructures.

Practical span lengths for using these possible sections for short-span bridges are shown in Figure 11. The multi-stem T-beam section is a reasonable solution for short spans with approximately 20- to 35-foot span lengths. The box beam section is reasonable for approximately 30- to 125-foot span lengths. The modified NEXT D beam section is reasonable for approximately 30- to 85-foot span and provides an alternate design option to the box beam for this span range. These sections can be used for longer span lengths if the optional live load deflection limit is relaxed or if a higher modulus of elasticity UHPC is used.



Source: FHWA.

Figure 11. Illustration. Practical span length ranges for multi-stem T-beam, modified box beam, and modified NEXT D beam sections.

The multi-stem T-beam section generally results in a lighter superstructure when suitable for a particular span length. Table 5 compares the total superstructure weight of a bridge constructed using 18-inch-deep multi-stem T-beams to one constructed using 18-inch-deep box beams for a 28-foot-wide bridge with a 35-foot span length. As shown in Table 5, at the same depth of 18 inches, the multi-stem T-beam with a 67-inch width is 6 percent lighter than the similar depth box beam with a 48-inch width, weighing 16.9 kips compared to 18.0 kips. Additionally, since the multi-stem T-beams are wider than the box beams, fewer multi-stem T-beams are required for the 28-foot bridge width resulting in a 28 percent lighter superstructure. Note that a 35-foot span length is the longest possible span length for the 18-inch-deep multi-stem T-beam while meeting the optional live load deflection limit. In contrast, the 18-inch-deep box beam can span up to 45 feet and meet all design requirements.

Table 5. Comparison of 18-inch-deep multi-stem T-beam and box beam sections for 35-foot span bridge with a 28-foot width.

Beam Type	Multi-Stem T-Beam	Box Beam
h_{beam} (inch)	18.0	18.0
w_{beam} (inch)	67.0	48.0
A_g (inch^2)	434.1	462.2
w_g (kip/ft)	0.482	0.514
W_{beam} (kips)	16.9	18.0
n_{beams}	5	7
$W_{superstructure}$ (kips) [a]	105.4	146.8

[a] Including 0.3 k/ft barriers on each side of the bridge.

The modified NEXT D beam section will generally result in a lighter and slightly deeper bridge profile than a bridge constructed with modified box beams. A comparison of the required section geometry and overall bridge weight of a 75-foot-long bridge with about a 28-foot width is shown in Table 6. A 33-inch-deep modified NEXT D beam is required for the 75-foot span, while a 27-inch-deep modified box beam can be used for this span length. The superstructure with the modified NEXT D beams only requires three beams (compared to seven for the modified box beam bridge) and weighs approximately 50 kips less than the modified box beam bridge.

Table 6. Comparison of 33-inch-deep modified NEXT D beam section and 27-inch-deep modified box beam section for 75-foot span bridge with a 28-foot width.

Beam Type	Mod. NEXT D beam	Mod. Box Beam
Section type	Mod. NEXT 33D-120	Mod. BI-48
h_{beam} (inch)	33.0	27.0
w_{beam} (inch)	120.0	48.0
A_g (inch^2)	1004.9	518.2
w_g (kip/ft)	1.117	0.576
W_{beam} (kips)	83.7	43.2
n_{beams}	3	7
$W_{superstructure}$ (kips) [a]	296.2	347.3

[a] Including 0.3 k/ft barriers on each side of the bridge.

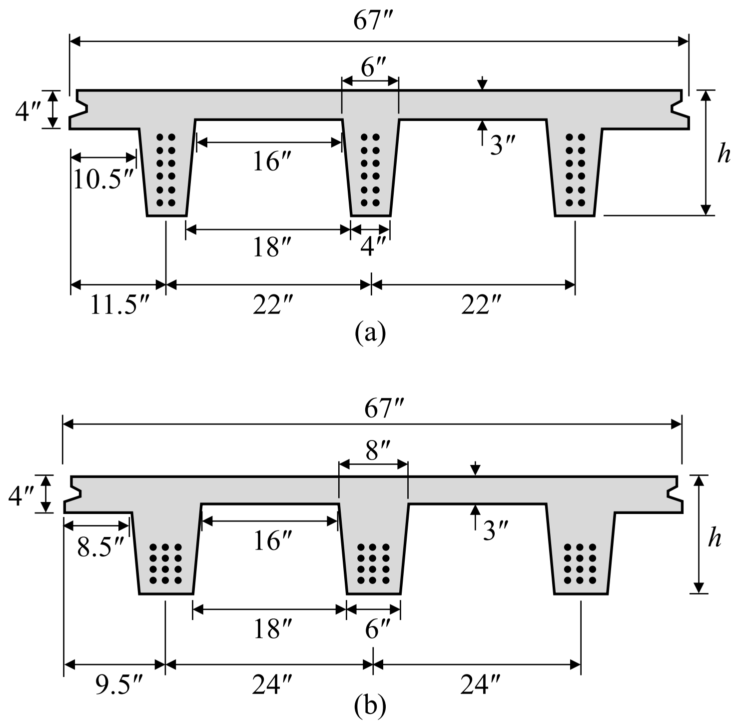
These cross-sectional shapes are not the only possible UHPC solutions for short- to medium-span bridges, but they can serve as a starting point for further refinements. The design aids and information provided in this chapter can be used to give a background on what section depths and quantity of prestressing is required for various span lengths.

MULTI-STEM T-BEAM

Introduction to Multi-Stem T-Beam Design Tables

As described above, a version of a UHPC multi-stem T-beam section was used for a short-span bridge in Michigan with the section shape shown in Figure 6. The multi-stem T-beam section may not be the most structurally optimized use of UHPC but may be easier to fabricate than voided sections with this shallow of a depth. A somewhat similar double tee conventional concrete section (i.e., NEXT beam) is receiving some interest due to its ease of fabrication, so it is reasonable to conceive that a similar UHPC section may be useful for short-span bridges.

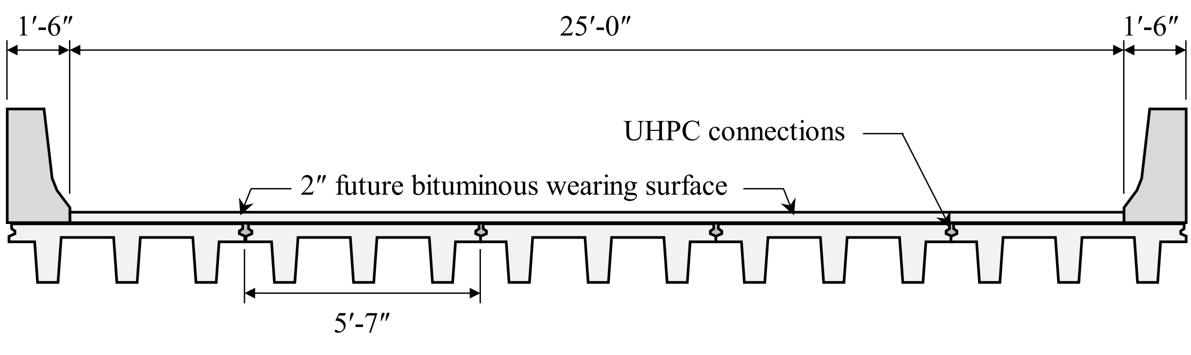
A multi-stem T-beam, similar to the one used in Michigan, was used as a starting point for two different pretensioned multi-stem T-beam UHPC sections, shown in Figure 12. The first option, Figure 12 (a), has narrower webs with only two strands per layer in each web. The second option, Figure 12 (b), has thicker webs that allow three strands per layer in each web.



Source: FHWA.

Figure 12. Illustration. Two options for multi-stem T-beam pretensioned UHPC section with (a) 4-inch tapered webs and (b) 6-inch tapered webs.

The design tables and graphs developed for this multi-stem T-beam section series were developed for the bridge cross section shown in Figure 13. The bridge consists of five 5-foot 7-inch-wide adjacent multi-stem T-beams with longitudinal field-cast UHPC connections between each precast member.



Source: FHWA.

Figure 13. Illustration. Bridge cross section for configuration used to develop design tables and design aids for the multi-stem T-beam section series.

Some of the details on the span configuration are as follows:

- Total beam length: $L_T = L + 1$ ft

- Support width: $b_s = 0.5$ ft

The UHPC material properties for the design guides are as follows:

- Compressive strength at transfer: $f'_{ci} = 14.0$ ksi
- Compressive strength for use in design and analyses: $f'_c = 17.5$ ksi
- Correction factor for modulus of elasticity: $K_1 = 1.0$
- UHPC unit weight (including reinforcement): $w_c = 0.160$ kcf
- Reduction factor for compression: $\alpha_u = 0.85$
- Ultimate compression strain: $\varepsilon_{cu} = 0.0035$
- Effective cracking strength: $f_{t,cr} = 1.0$ ksi
- Effective cracking strength at transfer: $f_{t,cri} = 0.75 f_{t,cr} = 0.75$ ksi
- Crack localization strength: $f_{t,loc} = 1.0$ ksi
- Crack localization strain: $\varepsilon_{t,loc} = 0.0035$
- Reduction factor for tension: $\gamma_u = 1.0$

The material properties for the conventional steel reinforcement (Grade 60) are as follows:

- Modulus of elasticity: $E_s = 29{,}000$ ksi
- Yield strength: $f_{sy} = 60$ ksi

The material properties for the prestressing strands are as follows:

- Low-relaxation
- Modulus of elasticity: $E_p = 28{,}500$ ksi
- Ultimate strength: $f_{pu} = 270$ ksi
- Yield strength: $f_{py} = 243$ ksi
- Specified rupture strain for strands: $\varepsilon_{pu} = 0.035$

The following stress-strain relationship was used for the prestressing strands:

$$f_{ps} = \varepsilon_{ps}\left[887 + \frac{27{,}613}{(1 + (112.4\varepsilon_{ps})^{7.36})^{(1/7.36)}}\right] \leq 270 \text{ ksi}$$

Two different strand diameters were used to develop the design guides. Strands with a 0.6-inch diameter were used when possible. 0.7-inch diameter strands were used for longer span lengths requiring a larger amount of prestressing.

- Area of one 0.6-inch diameter strand: $A_{p,0.6in} = 0.217$ inch^2
- Area of one 0.7-inch diameter strand: $A_{p,0.7in} = 0.294$ inch^2

The parameters related to creep coefficients and shrinkage strains used to develop the design tables are as follows.

- UHPC age at transfer: $t_i = 1$ day
- UHPC age at field placement: $t_d = 90$ days
- UHPC age at final time: $t_f = 36{,}500$ days

- Average ambient relative humidity: H = 70 percent
- Correction factor for creep: $K_3 = 1.0$
- Correction factor for shrinkage: $K_4 = 1.0$

Transformed section properties were used to calculate stresses. Gross section properties were used to calculate deflections.

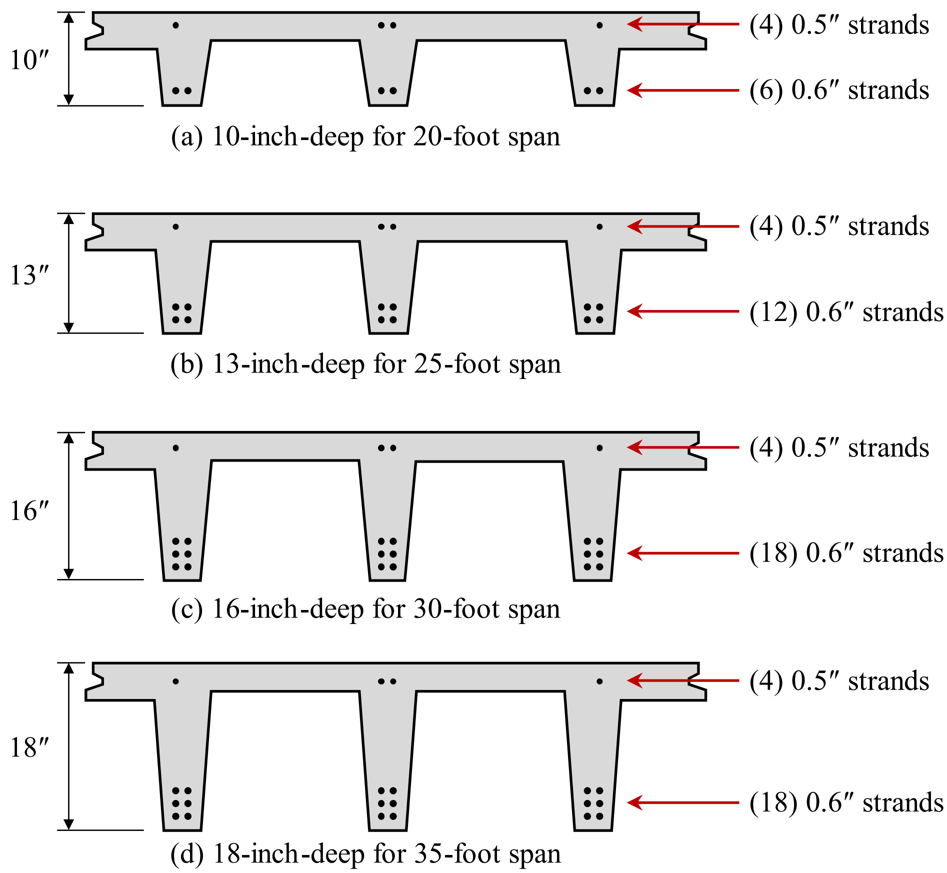
Possible Sections for 20- to 35-foot Spans

Several different depths of the UHPC multi-stem T-beam section with 4-inch-wide tapered webs were investigated for use on 20-, 25-, 30-, and 35-foot span lengths. The section properties for these sections and a few additional section depths are summarized in Table 7. The recommended sections for the four different span lengths are shown in the L_{span} column.

Table 7. Summary of section properties for UHPC multi-stem T-beam sections with 4-inch tapered webs with suggested span length.

Section Type	h (inch)	A_g (inch^2)	I_g (inch^4)	y_b (inch)	w_g (kip/ft)	L_{span} (ft)
TT4-10.0	10.0	314.9	2,120	6.80	0.350	20
TT4-11.0	11.0	329.8	2,809	7.49	0.366	-
TT4-12.0	12.0	344.7	3,631	8.17	0.383	-
TT4-13.0	13.0	359.6	4,598	8.84	0.400	25
TT4-14.0	14.0	374.5	5,716	9.49	0.416	-
TT4-15.0	15.0	389.4	7,005	10.14	0.433	-
TT4-16.0	16.0	404.3	8,447	10.77	0.449	30
TT4-17.0	17.0	419.2	10,080	11.40	0.466	-
TT4-18.0	18.0	434.1	11,899	12.03	0.482	35

The cross-section shapes and strand configurations for the multi-stem T-beam sections that can be used for 20-, 25-, 30-, and 35-foot span lengths are shown in Figure 14. These designs all meet the optional live load deflection criteria.



Source: FHWA.

Figure 14. Illustration. Possible multi-stem T-beam design for (a) 20-foot, (b) 25-foot, (c) 30-foot, and (d) 35-foot spans.

More details for these designs are provided in Table 9, Table 12, Table 15, and Table 17.

Additional Design Resources

Several additional design tables were created for different depth multi-stem T-beam sections with 4-inch-wide and 6-inch-wide tapered webs. The section depths analyzed for the 4-inch-wide tapered web are shown in Table 7; the 6-inch-wide tapered web section properties are summarized in Table 8.

Table 8. Summary of section properties for UHPC multi-stem T-beam sections with 6-inch tapered webs.

Section Type	h (inch)	A_g (inch^2)	I_g (inch^4)	y_b (inch)	w_g (kip/ft)	L_{span} (ft)
TT6-14.0	14.0	434.5	7,254	8.87	0.494	30
TT6-15.0	15.0	455.4	8,885	9.47	0.506	30
TT6-16.0	16.0	476.3	10,694	10.05	0.529	30
TT6-17.0	17.0	497.2	12,751	10.63	0.552	35
TT6-18.0	18.0	518.1	15,041	11.22	0.576	35

Details on the designs for different span length provided in Table 9 through Table 22 for different multi-stem T-beam sections. The details in these tables include the following.

The stresses due to the Service I load combination are provided.

- Stress in extreme tension fiber due to Service I load combination: $f_{t,SeI,m}$
- Stress in extreme compression fiber due to Service I load combination: $f_{c,SeI,m}$

These stresses are compared to the stress limits at service.

- Compressive stress limit due to Service I load combination: $f_{c,a} = 0.60\phi_w f'_c = 10.5$ ksi
- Tension stress limit due to Service III load combination: $f_{t,a} = \gamma_u f_{t,cr} = -1.0$ ksi
- Tension stress limit for members exposed to cyclic loads due to Service I load combination: $f_{t,a} = 0.95\gamma_u f_{t,cr} = -0.95$ ksi

The cracking moment (M_{cr}), demand due to the Strength I load combination at midspan ($M_{u,m}$), and factored resistance at midspan ($M_{r,m}$) are also provided in the table. The two associated design checks related to these are as follows.

- Flexural strength check: $M_{r,m} = \phi M_{n,m} \geq M_{u,m}$
- Minimum flexural reinforcement: $M_{r,m} = \phi M_{n,m} \geq$ minimum of (1.33M_u and M_{cr})

Two deflections are provided in the design tables. The total long-term deflection due to sustained loads ($\Delta_{sus,LT}$) includes the deflection due to the prestressing, self-weight, barrier weight, and weight of the future wearing surface. A positive deflection is downward, negative upward. The live load deflection is calculated based on AASHTO LRFD BDS Article 3.6.1.3.2.

Live load deflection: Δ_L = maximum of Δ_{LT} and $(0.25\Delta_{LT} + \Delta_{LL})$

The live load deflection can be compared to the optional live load deflection limit in AASHTO LRFD BDS Article 2.5.2.6.2.

Live load deflection limit: $\Delta_L \leq L / 800$

The live load deflections exceeding the optional limit are highlighted in the design tables.

Table 9. Design table for TT4-10.0 Multi-Stem T-Beam Section (10.0-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302 [c]	-0.504	1.671	183	193	156	-1.00	0.16	Strength I

[a] 0.6-inch diameter strands were used.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed 0.5-inch diameter top strands at 2 inches from top are required to meet stress checks at time of transfer. These are not included in A_{ps} shown.

Table 10. Design table for TT4-11.0 Multi-Stem T-Beam Section (11.0-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302 [c]	-0.104	1.414	188	221	180	-0.88	0.12	Strength I
25	3.906 [c]	0.321	2.786	341	347	308	-2.52	0.50 [d]	Strength I

[a] 0.6-inch diameter strands were used.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed 0.5-inch diameter top strands at 2 inches from top are required to meet stress checks at time of transfer. These are not included in A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 25 feet.

Table 11. Design table for TT4-12.0 Multi-Stem T-Beam Section (12.0-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302 [c]	0.183	1.221	193	250	205	-0.78	0.09	Min. Strands
25	3.906 [c]	1.083	2.274	350	405	363	-2.39	0.38 [d]	Strength I

[a] 0.6-inch diameter strands were used.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed 0.5-inch diameter top strands at 2 inches from top are required to meet stress checks at time of transfer. These are not included in A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 25 feet.

Table 12. Design table for TT4-13.0 Multi-Stem T-Beam Section (13.0-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302 [c]	0.388	1.073	197	280	231	-0.69	0.07	Min. Strands
25	2.604 [c]	0.370	1.812	358	396	346	-1.79	0.30	Strength I

[a] 0.6-inch diameter strands were used.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed 0.5-inch diameter top strands at 2 inches from top are required to meet stress checks at time of transfer. These are not included in A_{ps} shown.

Table 13. Design table for TT4-14.0 Multi-Stem T-Beam Section (14.0-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302 [c]	0.535	0.957	202	313	257	-0.62	0.06	Min. Strands
25	2.604 [c]	0.713	1.570	366	444	387	-1.63	0.24	Strength I
30	3.906 [c]	0.247	2.493	528	532	476	-2.73	0.57 [d]	Strength I

[a] 0.6-inch diameter strands were used.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed 0.5-inch diameter top strands at 2 inches from top are required to meet stress checks at time of transfer. These are not included in A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 30 feet.

Table 14. Design table for TT4-15.0 Multi-Stem T-Beam Section (15.0-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302	0.737	0.415	206	331	291	-0.65	0.05	Min. Strands
25	2.604 [c]	0.971	1.375	374	494	430	-1.49	0.20	Strength I
30	3.906 [c]	0.716	2.164	539	597	534	-2.55	0.46 [d]	Strength I

[a] 0.6-inch diameter strands were used.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed 0.5-inch diameter top strands at 2 inches from top are required to meet stress checks at time of transfer. These are not included in A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 30 feet.

Table 15. Design table for TT4-16.0 Multi-Stem T-Beam Section (16.0-inches deep).

Span (ft)	A_{ps} [a] ($inch^2$)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302	0.823	0.345	210	365	320	-0.59	0.04	Min. Strands
25	2.604 [c]	1.163	1.216	381	545	473	-1.36	0.16	Strength I
30	3.906 [c]	1.069	1.897	549	664	591	-2.37	0.38	Strength I

[a] 0.6-inch diameter strands were used.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed 0.5-inch diameter top strands at 2 inches from top are required to meet stress checks at time of transfer. These are not included in A_{ps} shown.

Table 16. Design table for TT4-17.0 Multi-Stem T-Beam Section (17.0-inches deep).

Span (ft)	A_{ps} [a] ($inch^2$)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302	0.885	0.286	214	399	350	-0.54	0.03	Min. Strands
25	1.302	-0.455	0.969	388	402	351	-0.75	0.14	Strength I
30	2.604 [c]	0.018	1.756	559	601	519	-1.64	0.32	Strength I
35	3.906 [c]	0.067	2.356	731	736	652	-2.77	0.60 [d]	Strength I

[a] 0.6-inch diameter strands were used.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed 0.5-inch diameter top strands at 2 inches from top are required to meet stress checks at time of transfer. These are not included in A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 35 feet.

Table 17. Design table for TT4-18.0 Multi-Stem T-Beam Section (18.0-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302	0.933	0.237	218	434	380	-0.49	0.03	Min. Strands
25	1.302	-0.289	0.865	395	438	382	-0.69	0.12	Strength I
30	2.604 [c]	0.245	1.588	569	655	564	-1.51	0.27	Strength I
35	3.906 [c]	0.399	2.112	744	807	711	-2.59	0.51	Strength I

[a] 0.6-inch diameter strands were used.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed 0.5-inch diameter top strands at 2 inches from top are required to meet stress checks at time of transfer. These are not included in A_{ps} shown.

Table 18. Design table for TT6-14.0 Multi-Stem T-Beam Section (14.0-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302	0.346	0.562	211	338	288	-0.53	0.05	Strength I
25	1.953 [c]	-0.211	1.643	382	443	363	-0.94	0.19	Strength I
30	3.255 [c]	-0.185	2.293	551	573	487	-1.95	0.45	Strength I
35	5.292 [c,e]	0.555	2.896	720	784	687	-3.86	0.83 [d]	Strength I

[a] 0.6-inch diameter strands were used up to 30-foot spans; 0.7-inch diameter strands were used for spans 35 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Three fully stressed top strands at 2 inches from the top are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 35 feet.
[e] Effective cracking strength at release of 0.86 ksi is required for this design.

Table 19. Design table for TT6-15.0 Multi-Stem T-Beam Section (15.0-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
25	1.953 [c]	-0.016	1.468	390	491	401	-0.85	0.16	Strength I
30	3.255 [c]	0.126	2.020	563	638	540	-1.79	0.36	Strength I
35	4.410 [c]	0.152	2.648	736	787	676	-3.02	0.68 [d]	Strength I
40	6.174 [c,e]	0.279	3.348	912	940	827	-4.65	1.12 [d]	Strength I

[a] 0.6-inch diameter strands were used up to 30-foot spans; 0.7-inch diameter strands were used for spans 35 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Three fully stressed top strands at 2 inches from the top are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 35 feet and longer.
[e] Effective cracking strength at release of 0.85 ksi is required for this design.

Table 20. Design table for TT6-16.0 Multi-Stem T-Beam Section (16.0-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
25	1.302	-0.659	1.112	399	419	354	-0.58	0.13	Strength I
30	2.604 [c]	-0.304	1.909	575	626	518	-1.31	0.30	Strength I
35	3.906 [c]	-0.126	2.399	752	786	669	-2.42	0.57 [d]	Strength I
40	5.292 [c,e]	0.125	2.948	932	972	840	-3.94	0.93 [d]	Strength I

[a] 0.6-inch diameter strands were used up to 35-foot spans; 0.7-inch diameter strands were used for spans 40 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Three fully stressed top strands at 2 inches from the top are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 35 feet and longer.
[e] Effective cracking strength at release of 0.89 ksi is required for this design.

Table 21. Design table for TT6-17.0 Multi-Stem T-Beam Section (17.0-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
25	1.302	-0.500	0.993	461	407	388	-0.53	0.11	Strength I
30	2.604 [c]	-0.100	1.725	586	686	566	-1.20	0.25	Strength I
35	3.255 [c]	-0.475	2.258	767	778	650	-1.85	0.47	Strength I
40	4.410 [c]	-0.315	2.771	950	958	811	-3.05	0.78 [d]	Strength I
45	6.174 [c,e]	-0.002	3.304	1,138	1,154	1,001	-4.71	1.19 [d]	Strength I

[a] 0.6-inch diameter strands were used up to 35-foot spans; 0.7-inch diameter strands were used for spans 40 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Three fully stressed top strands at 2 inches from the top are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 40 feet and longer.
[e] Effective cracking strength at release of 0.91 ksi is required for this design.

Table 22. Design table for TT6-18.0 Multi-Stem T-Beam Section (18.0-inches deep).

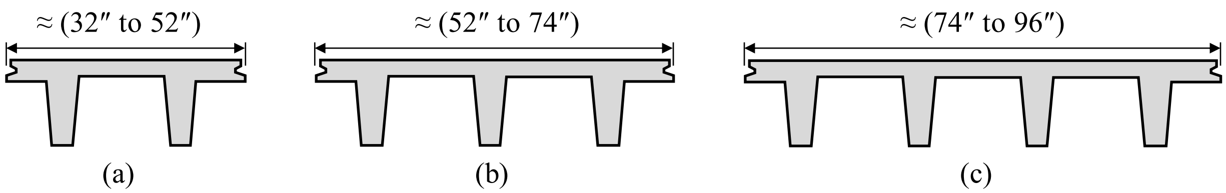
Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
25	1.302	-0.369	0.891	414	503	423	-0.49	0.09	Strength I
30	1.953 [c]	-0.608	1.693	597	649	521	-0.79	0.22	Strength I
35	3.255 [c]	-0.239	2.046	781	848	706	-1.72	0.40	Strength I
40	4.410 [c]	-0.035	2.509	968	1,043	879	-2.84	0.66 [d]	Strength I
45	5.292 [c,e]	-0.227	2.994	1,159	1,170	998	-3.94	1.01 [d]	Strength I
50	7.056 [c,f]	-0.058	3.591	1,355	1,359	1,181	-5.54	1.45 [d]	Strength I

[a] 0.6-inch diameter strands were used up to 35-foot spans; 0.7-inch diameter strands were used for spans 40 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Three fully stressed top strands at 2 inches from the top are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 40 feet and longer.
[e] Effective cracking strength at release of 0.89 ksi is required for this design.
[f] Effective cracking strength at release of 0.97 ksi is required for this design.

Further Discussion on Multi-Stem T-Beam Section Geometry

The section shapes presented herein represent just two possible general configurations of the multi-stem T-beam section and are not an exhaustive list of all possible variations. The multi-stem T-beam section could be reasonably modified to better suite local needs and facilitate fabrication at local precasters.

The web spacing (18-inch clear distance between the bottom of the webs) and top flange thickness (3-inch-thick interior and 4-inch-thick exterior) were based on the section constructed in Michigan. The webs could possibly be spaced out further, but additional analysis of section should be performed to confirm that the transverse flexural capacity and overall stability of the section were satisfactory. Additionally, the section could be detailed with more webs (e.g., 4 webs) or less webs (e.g., 2 webs) based on bridge widths, transportation width restrictions, available construction equipment, or other limitations, as shown in Figure 15. At least two webs should be used to help with the stability of the section during storage and transportation of the beams and erection of the bridge.



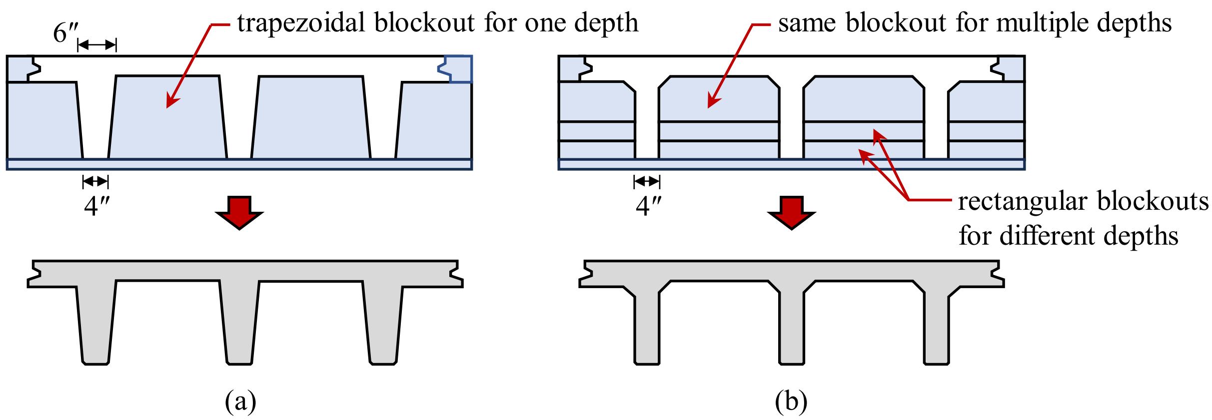
Source: FHWA.

Figure 15. Illustration. Possible web configurations using similar web spacing and overhang lengths.

Two different tapered web widths were investigated as part of the development of this report: 4-inch-wide and 6-inch-wide tapered. Compared to the section with a 4-inch-wide tapered web, the section with 6-inch-wide tapered web had a higher moment of inertia (average increase of 21.2 percent), higher weight (average increase of 14.5 percent), and allowed for one additional strand per web per layer (three strands per layer instead of 2 strands). Although the section with the 6-inch-wide tapered web is heavier than its 4-inch-wide web counterpart, its higher moment of inertia allowed the use of slightly shallower section depths in some cases while still meeting the optional deflection limits. For example, a TT6-15.0 (6-inch-wide tapered web and 15 inches deep) can be used for 30-foot spans, whereas a TT4-16.0 (4-inch-wide tapered web and 16 inches deep) is needed for the same span length. Other configurations of the web widths could be explored, but they are not likely to have a significant impact on the design based on the analyses provided in this report.

The multiple-stem tee sections analyzed in this report were designed with tapered webs to facilitate the release of the members from the forms, increase transverse flexure capacity, and improve the stability of the section. To simplify the calculation of the section geometry in the analysis, the top and bottom widths of the tapered web were maintained across all section depths. For example, for the case of the 4-inch-wide tapered web, the bottom of the web was kept at 4 inches and top at 6 inches regardless of the section depth, as shown in Figure 16 (a). However, the taper of the web could reasonably maintain the same slope for different section depths to facilitate fabrication.

Alternatively, straight webs can be constructed using a formwork blockout configuration that allows for multiple depths to be cast with the same set of formwork. However, this configuration may require smaller spacing between the stems to ensure the stability of the section. A possible cross-sectional shape option with straight webs is shown in Figure 16 (b).



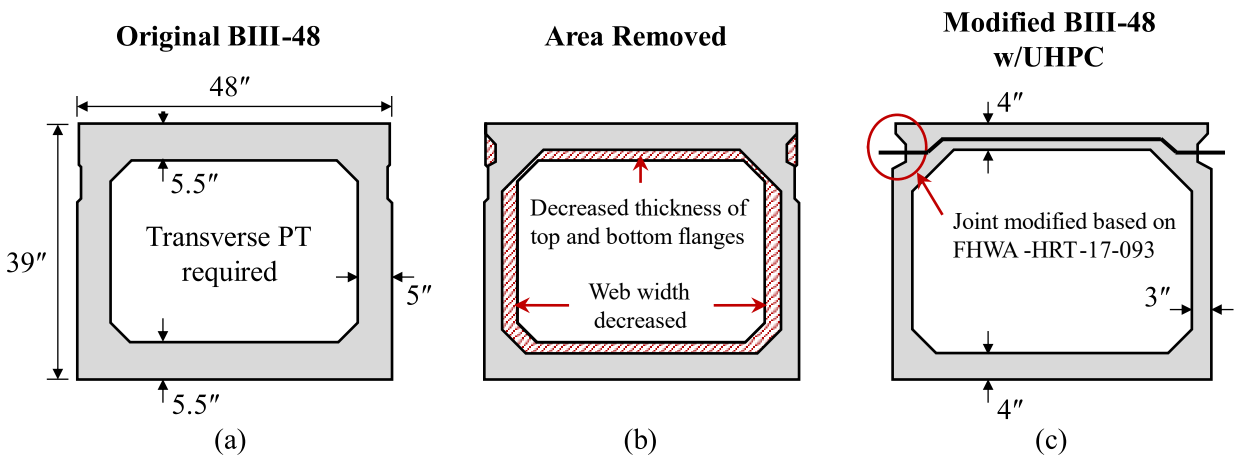
Source: FHWA.

Figure 16. Illustration. Possible options for formwork for multi-stem T-beam section.

MODIFIED BOX BEAMS

Introduction to Box Beam Design Tables

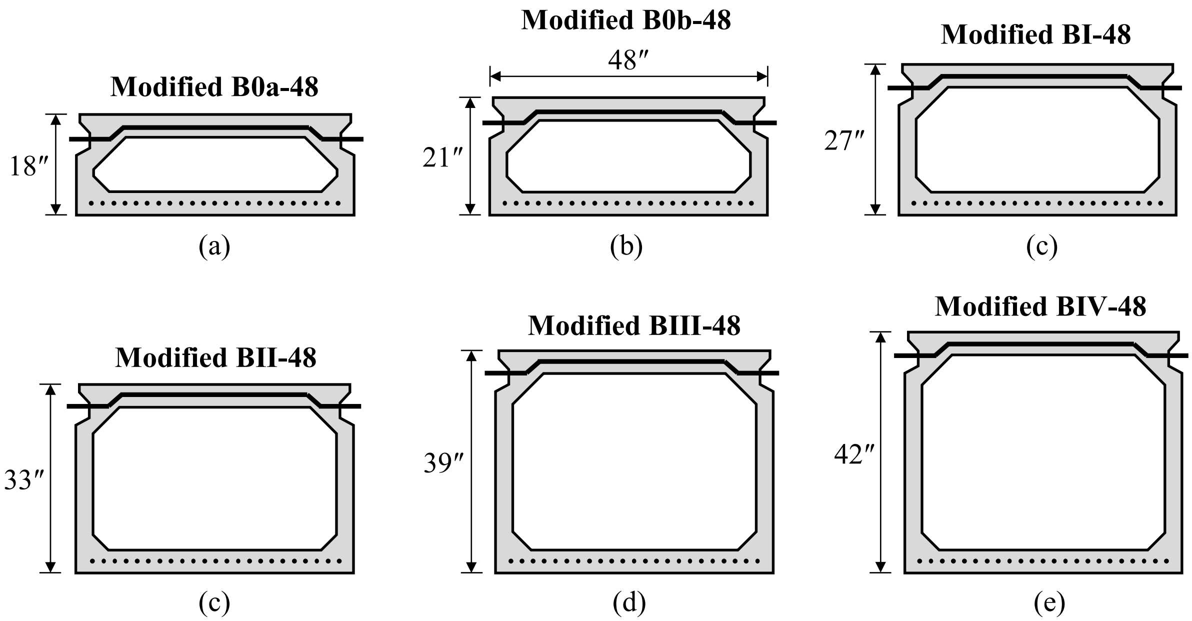
A modified UHPC box beam section was developed as a possible short-span solution without a cast-in-place composite deck. The modified box beam section shape is based on the AASHTO box beam series, as shown in Figure 17. The web widths and the top and bottom flange thicknesses were decreased to take advantage of the enhanced properties of UHPC and reduce the weight of the section. The shear key and transverse post-tensioning used for the conventional concrete adjacent box beams was replaced with a UHPC joint detail described in FHWA-HRT-17-093.



Source: FHWA.

Figure 17. Illustration. Example of optimization of previously used section for UHPC based on BIII-48 box beam section.

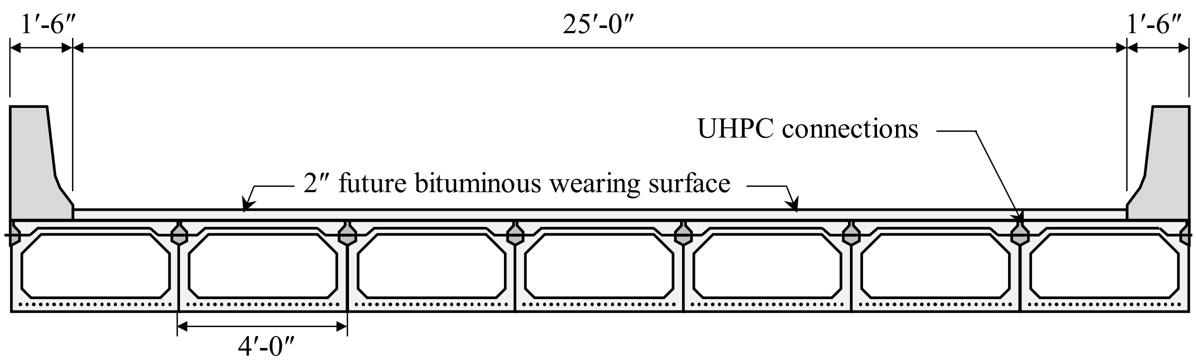
Section details for six different depths, ranging from 18 inches to 42 inches, were developed for the modified box beam shape, as shown in Figure 18. The modified box beam series was designed to achieve span lengths up to 125 feet considering the optional deflection limits in AASHTO LRFD BDS and up to 140 feet if the deflection limits are relaxed for typical UHPC material properties or if stiffer UHPC is used. All section shapes can accommodate up to 23 prestressing strands in the bottom layer, which is located 2 inches from the bottom face, and 2 top prestressing strands, located at 2 inches from the top face. A minimum of four strands in the bottom layer were used in the designs.



Source: FHWA.

Figure 18. Illustration. Possible modified box beam section series based on AASHTO Box Beam shapes for conventional concrete.

The design tables and graphs developed for this modified box beam series were designed using the bridge cross section shown in Figure 19. The bridge consists of seven 48-inch-wide adjacent box beams connected with UHPC connections and is designed to carry two lanes of traffic. This bridge configuration is similar to Example 9.4 from the PCI Bridge Design Manual.



Source: FHWA.

Figure 19. Illustration. Bridge cross section for configuration used to develop design tables and design aids for the modified box beam section series.

Some of the details on the span configuration are as follows:

- Total beam length: $L_T = L + 1$ ft
- Support width: $b_s = 0.5$ ft

The UHPC material properties for the design guides are as follows:

- Compressive strength at transfer: f'_{ci} = 14.0 ksi
- Compressive strength for use in design and analyses: f'_c = 17.5 ksi
- Correction factor for modulus of elasticity: K_1 = 1.0
- UHPC unit weight (including reinforcement): w_c = 0.160 kcf
- Reduction factor for compression: α_u = 0.85
- Ultimate compression strain: ε_{cu} = 0.0035
- Effective cracking strength: $f_{t,cr}$ = 1.0 ksi
- Effective cracking strength at transfer: $f_{t,cri} = 0.75 f_{t,cr}$ = 0.75 ksi
- Crack localization strength: $f_{t,loc}$ = 1.0 ksi
- Crack localization strain: $\varepsilon_{t,loc}$ = 0.0035
- Reduction factor for tension: γ_u = 1.0

The material properties for the conventional steel reinforcement (Grade 60) are as follows:

- Modulus of elasticity: E_s = 29,000 ksi
- Yield strength: f_{sy} = 60 ksi

The material properties for the prestressing strands are as follows:

- Low-relaxation
- Modulus of elasticity: E_p = 28,500 ksi
- Ultimate strength: f_{pu} = 270 ksi
- Yield strength: f_{py} = 243 ksi
- Specified rupture strain for strands: ε_{pu} = 0.035

The following stress-strain relationship was used for the prestressing strands:

$$f_{ps} = \varepsilon_{ps}\left[887 + \frac{27{,}613}{(1 + (112.4\varepsilon_{ps})^{7.36})^{(1/7.36)}}\right] \leq 270 \text{ ksi}$$

Two different strand diameters were used to develop the design guides. Strands with a 0.6-inch diameter were used when possible. 0.7-inch diameter strands were used for longer span lengths requiring a larger amount of prestressing.

- Area of one 0.6-inch diameter strand: $A_{p,0.6in}$ = 0.217 inch^2
- Area of one 0.7-inch diameter strand: $A_{p,0.7in}$ = 0.294 inch^2

The parameters related to creep coefficients and shrinkage strains used to develop the design tables are as follows.

- UHPC age at transfer: t_i = 1 day
- UHPC age at field placement: t_d = 90 days
- UHPC age at final time: t_f = 36,500 days
- Average ambient relative humidity: H = 70 percent
- Correction factor for creep: K_3 = 1.0

- Correction factor for shrinkage: $K_4 = 1.0$

Transformed section properties were used to calculate stresses. Gross section properties were used to calculate deflections.

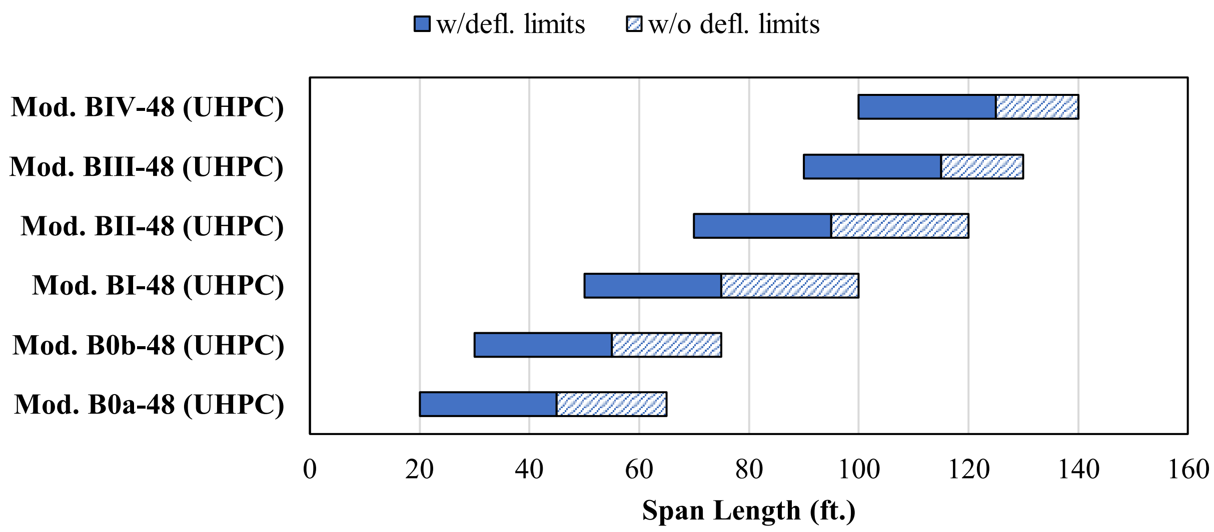
Possible Sections for 45- to 125-foot Spans

The section properties for the box beam sections and the maximum possible span lengths are summarized in Table 23. The maximum span lengths are provided with and without considering the optional live load deflection limits in AASHTO LRFD BDS.

Table 23. Summary of section properties and maximum span lengths for UHPC box beam sections for provided design tables.

Section Type	h (inch)	A_g (inch^2)	I_g (inch^4)	y_b (inch)	w_g (kip/ft)	L_{max} (w/defl. limit)	L_{max} (w/o defl. limit)
Mod. B0a-48 (UHPC)	18	462.2	19,737	8.91	0.516	45 ft.	65 ft.
Mod. B0b-48 (UHPC)	21	482.0	29,416	10.40	0.536	55 ft.	75 ft.
Mod. BI-48 (UHPC)	27	518.2	55,301	13.45	0.576	75 ft.	100 ft.
Mod. BII-48 (UHPC)	33	554.1	90,567	16.45	0.616	95 ft.	120 ft.
Mod. BIII-48 (UHPC)	39	590.2	135,832	19.47	0.656	115 ft.	130 ft.
Mod. BIV-48 (UHPC)	42	608.5	162,455	20.97	0.676	125 ft.	140 ft.

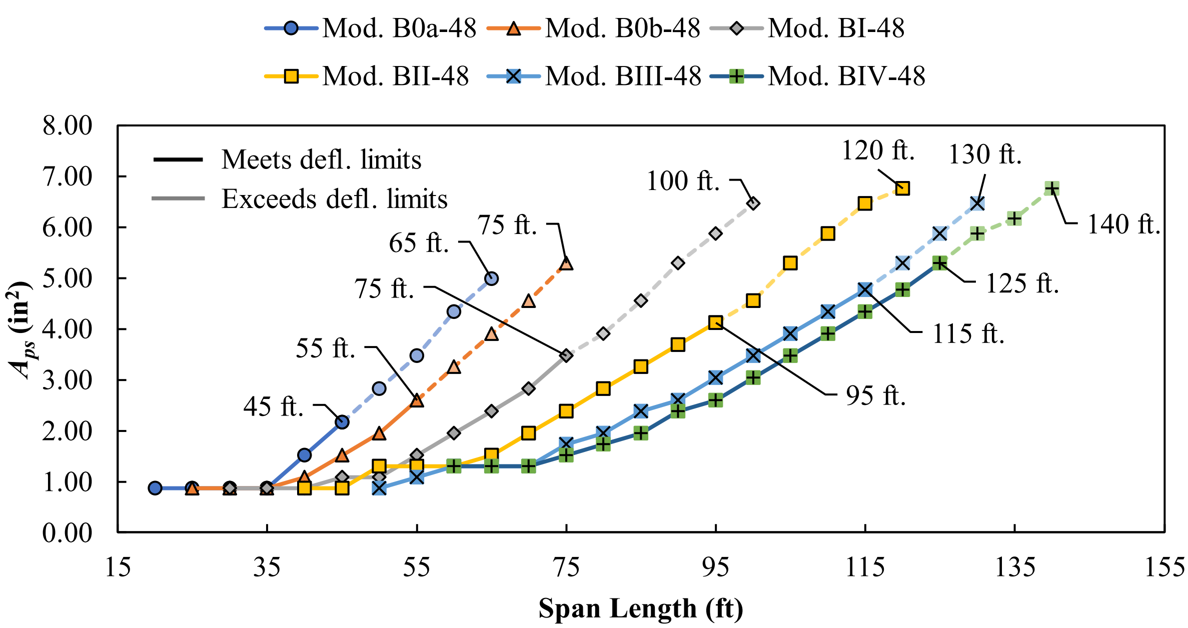
The span length ranges for each beam in the modified box beam series are shown in Figure 20. A 25-foot range was considered for each section depth, considering optional live load deflection criteria. Deeper sections could be used as needed for shorter span lengths than those shown herein.



Source: FHWA.

Figure 20. Graph. Practical span length ranges for modified UHPC box beam series.

The total area of prestressing steel on the flexural tension side of the member (A_{ps}) required for different span lengths in the modified box beam section series is shown in Figure 21. The span lengths at which the design no longer satisfies the optional live load deflection criteria specified in AASHTO LRFD BDS Article 2.5.2.6.2 are plotted as dashed lines in Figure 21.



Source: FHWA.

Figure 21. Graph. Required prestressing area versus span length for modified UHPC box beam series.

More details on each of the analysis points shown in Figure 21 are provided in the following section.

Additional Design Resources

Details on the designs for each span length are provided in Table 24 through Table 29 for the modified box beam section series.

The stresses due to the Service I load combination are provided.

- Stress in extreme tension fiber due to Service I load combination: $f_{t,SeI,m}$
- Stress in extreme compression fiber due to Service I load combination: $f_{c,SeI,m}$

These stresses are compared to the stress limits at service.

- Compressive stress limit due to Service I load combination: $f_{c,a} = 0.60\phi_w f'_c = 10.5$ ksi
- Tension stress limit due to Service III load combination: $f_{t,a} = \gamma_u f_{t,cr} = -1.0$ ksi
- Tension stress limit for members exposed to cyclic loads due to Service I load combination: $f_{t,a} = 0.95\gamma_u f_{t,cr} = -0.95$ ksi

The cracking moment (M_{cr}), demand due to the Strength I load combination at midspan ($M_{u,m}$), and factored resistance at midspan ($M_{r,m}$) are also provided in the table. The two associated design checks related to these are as follows.

- Flexural strength check: $M_{r,m} = \phi M_{n,m} \geq M_{u,m}$
- Minimum flexural reinforcement: $M_{r,m} = \phi M_{n,m} \geq$ minimum of ($1.33M_u$ and M_{cr})

Two deflections are provided in the design tables. The total long-term deflection due to sustained loads ($\Delta_{sus,LT}$) includes the deflection due to the prestressing, self-weight, barrier weight, and weight of the future wearing surface. A positive deflection is downward, negative upward. The live load deflection is calculated based on AASHTO LRFD BDS Article 3.6.1.3.2.

Live load deflection: Δ_L = maximum of Δ_{LT} and ($0.25\Delta_{LT} + \Delta_{LL}$)

The live load deflection can be compared to the optional live load deflection limit in AASHTO LRFD BDS Article 2.5.2.6.2.

Live load deflection limit: $\Delta_L \leq L / 800$

The live load deflections exceeding the optional limit are highlighted in the design tables. Designs are shown up to an upper live load deflection limit of L / 400. The upper range of the 18-inch-deep and 21-inch-deep sections are controlled by this L / 400 limit.

The details in the design tables are provided assuming only one layer of 0.6-inch or 0.7-inch diameter prestressing strands on the flexural tension side of the box beam.

Table 24. Design table for Modified B0a-48 Box Beam Section (18-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	0.868	0.300	0.337	137	471	460	-0.13	0.01	Min. Strands
25	0.868	-0.062	0.712	246	474	461	-0.15	0.05	Min. Strands
30	0.868	-0.437	1.101	357	478	461	-0.14	0.12	Min. Strands
35	0.868	-0.829	1.508	473	482	462	-0.06	0.22	Strength I
40	1.519	-0.671	1.820	592	614	583	-0.37	0.36	Strength I
45	2.170	-0.553	2.157	717	741	700	-0.77	0.55	Strength I
50	2.821	-0.474	2.521	847	867	816	-1.25	0.79 [c]	Strength I
55	3.472	-0.430	2.911	983	990	930	-1.79	1.09 [c]	Strength I
60	4.340 [d]	-0.328	3.659	1,124	1,170	1,065	-2.00	1.45 [c]	Strength I
65	4.991 [d]	-0.351	4.098	1,271	1,292	1,176	-2.54	1.89 [c]	Strength I

[a] 0.6-inch diameter strands were used up to 65-foot spans; 0.7-inch diameter strands were used for spans 70 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Deflections do not meet the optional live load deflection checks for span lengths of 50 feet and longer.
[d] Two fully stressed top strands at 16 inches from bottom are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.

Table 25. Design table for Modified B0b-48 Box Beam Section (21-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
25	0.868	0.078	0.543	248	580	579	-0.14	0.03	Min. Strands
30	0.868	-0.219	0.852	361	585	580	-0.14	0.08	Min. Strands
35	0.868	-0.531	1.175	478	592	581	-0.11	0.15	Min. Flex. Reinf.
40	1.085	-0.673	1.479	599	650	631	-0.14	0.24	Strength I
45	1.519	-0.654	1.765	725	760	729	-0.32	0.37	Strength I
50	1.953	-0.658	2.070	857	868	827	-0.53	0.53	Strength I
55	2.604	-0.520	2.363	995	1,023	970	-0.97	0.73	Strength I
60	3.255	-0.414	2.678	1,138	1,179	1,110	-1.48	0.98 [c]	Strength I
65	3.906	-0.338	3.015	1,288	1,330	1,249	-2.03	1.27 [c]	Strength I
70	4.557 [d]	-0.291	3.375	1,443	1,479	1,387	-2.61	1.61 [c]	Strength I
75	5.292 [d]	-0.296	4.208	1,605	1,654	1,518	-2.23	2.01 [c]	Strength I
80	5.880 [d]	-0.340	4.613	1,773	1,788	1,641	-2.58	2.46 [c]	Strength I

[a] 0.6-inch diameter strands were used up to 70-foot spans; 0.7-inch diameter strands were used for spans 75 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Deflections do not meet the optional live load deflection checks for span lengths of 60 feet and longer.
[d] Two fully stressed top strands at 19 inches from bottom are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.

Table 26. Design table for Modified BI-48 Box Beam Section (27-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
30	0.868	0.016	0.562	369	807	824	-0.13	0.04	Min. Strands
35	0.868	-0.204	0.788	488	814	825	-0.13	0.08	Min. Strands
40	0.868	-0.435	1.026	613	818	826	-0.09	0.13	Min. Flex. Reinf.
45	1.085	-0.506	1.243	742	902	894	-0.14	0.20	Min. Flex. Reinf.
50	1.085	-0.761	1.507	878	906	895	-0.03	0.28	Strength I
55	1.519	-0.689	1.718	1,019	1,062	1,030	-0.19	0.39	Strength I
60	1.953	-0.636	1.944	1,167	1,208	1,164	-0.39	0.52	Strength I
65	2.387	-0.603	2.186	1,321	1,353	1,296	-0.60	0.67	Strength I
70	2.821	-0.587	2.443	1,481	1,502	1,427	-0.83	0.86	Strength I
75	3.472	-0.440	2.687	1,648	1,708	1,619	-1.32	1.07	Strength I
80	3.906	-0.461	2.975	1,822	1,850	1,748	-1.56	1.31 [c]	Strength I
85	4.557	-0.357	3.252	2,002	2,053	1,937	-2.09	1.58 [c]	Strength I
90	5.292 [d]	-0.300	3.965	2,189	2,289	2,117	-1.61	1.89 [c]	Strength I
95	5.880 [d]	-0.277	4.277	2,383	2,471	2,286	-1.94	2.24 [c]	Strength I
100	6.468 [d]	-0.272	4.605	2,583	2,654	2,456	-2.23	2.63 [c]	Strength I

[a] 0.6-inch diameter strands were used up to 85-foot spans; 0.7-inch diameter strands were used for spans 90 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Deflections do not meet the optional live load deflection checks for span lengths of 80 feet and longer.
[d] Two fully stressed top strands at 25 inches from bottom are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.

Table 27. Design table for Modified BII-48 Box Beam Section (33-inches deep)

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
40	0.868	-0.212	0.758	626	1,059	1,082	-0.11	0.08	Min. Strands
45	0.868	-0.399	0.951	759	1,059	1,083	-0.07	0.12	Min. Flex. Reinf.
50	1.302	-0.275	1.090	898	1,261	1,254	-0.23	0.17	Min. Flex. Reinf.
55	1.302	-0.481	1.303	1,043	1,274	1,256	-0.15	0.24	Min. Flex. Reinf.
60	1.302	-0.697	1.527	1,195	1,278	1,258	-0.02	0.32	Min. Flex. Reinf.
65	1.519	-0.763	1.731	1,354	1,387	1,344	-0.01	0.41	Strength I
70	1.953	-0.685	1.916	1,519	1,577	1,513	-0.19	0.52	Strength I
75	2.387	-0.623	2.112	1,691	1,760	1,680	-0.38	0.65	Strength I
80	2.821	-0.576	2.322	1,870	1,947	1,846	-0.58	0.80	Strength I
85	3.255	-0.544	2.543	2,056	2,128	2,011	-0.79	0.97	Strength I
90	3.689	-0.526	2.777	2,249	2,309	2,176	-0.99	1.16	Strength I
95	4.123	-0.522	3.023	2,449	2,489	2,340	-1.17	1.37	Strength I
100	4.557	-0.530	3.282	2,657	2,669	2,503	-1.32	1.61 [c]	Strength I
105	5.292 [d]	-0.447	3.924	2,871	2,965	2,729	-0.69	1.87 [c]	Strength I
110	5.880 [d]	-0.391	4.185	3,092	3,198	2,945	-0.97	2.16 [c]	Strength I
115	6.468 [d]	-0.351	4.459	3,321	3,430	3,161	-1.21	2.47 [c]	Strength I
120	6.762 [d]	-0.490	4.780	3,557	3,560	3,275	-0.78	2.82 [c]	Strength I

[a] 0.6-inch diameter strands were used up to 100-foot spans; 0.7-inch diameter strands were used for spans 105 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Deflections do not meet the optional live load deflection checks for span lengths of 100 feet and longer.
[d] Two fully stressed top strands at 31 inches from bottom are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.

Table 28. Design table for Modified BIII-48 Box Beam Section (39-inches deep).

Span (ft)	A_{ps} [a] ($inch^2$)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
50	0.868	-0.392	0.910	918	1,316	1,351	-0.05	0.12	Min. Flex. Reinf.
55	1.085	-0.407	1.052	1,067	1,448	1,455	-0.08	0.16	Min. Flex. Reinf.
60	1.302	-0.431	1.203	1,223	1,579	1,558	-0.12	0.21	Min. Flex. Reinf.
65	1.302	-0.616	1.394	1,386	1,580	1,561	0.00	0.27	Min. Flex. Reinf.
70	1.302	-0.810	1.594	1,556	1,581	1,563	0.18	0.35	Strength I
75	1.736	-0.709	1.743	1,733	1,843	1,768	0.02	0.43	Strength I
80	1.953	-0.769	1.931	1,918	1,974	1,872	0.07	0.53	Strength I
85	2.387	-0.692	2.100	2,110	2,196	2,075	-0.11	0.64	Strength I
90	2.604	-0.771	2.308	2,309	2,321	2,178	-0.02	0.77	Strength I
95	3.038	-0.718	2.497	2,515	2,549	2,379	-0.18	0.91	Strength I
100	3.472	-0.677	2.696	2,729	2,769	2,579	-0.34	1.07	Strength I
105	3.906	-0.648	2.906	2,951	2,988	2,778	-0.48	1.25	Strength I
110	4.340	-0.631	3.126	3,180	3,207	2,976	-0.60	1.44	Strength I
115	4.774	-0.624	3.356	3,416	3,426	3,174	-0.69	1.65	Strength I
120	5.292	-0.579	3.586	3,660	3,680	3,407	-0.89	1.88 [c]	Strength I
125	5.880 [d]	-0.587	4.204	3,911	3,970	3,615	0.27	2.13 [c]	Strength I
130	6.468 [d]	-0.526	4.444	4,170	4,252	3,876	0.08	2.40 [c]	Strength I

[a] 0.6-inch diameter strands were used up to 115-foot spans; 0.7-inch diameter strands were used for spans 120 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Deflections do not meet the optional live load deflection checks for span lengths of 120 feet and longer.
[d] Two fully stressed top strands at 37 inches from bottom are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.

Table 29. Design table for Modified BIV-48 Box Beam Section (42-inches deep)

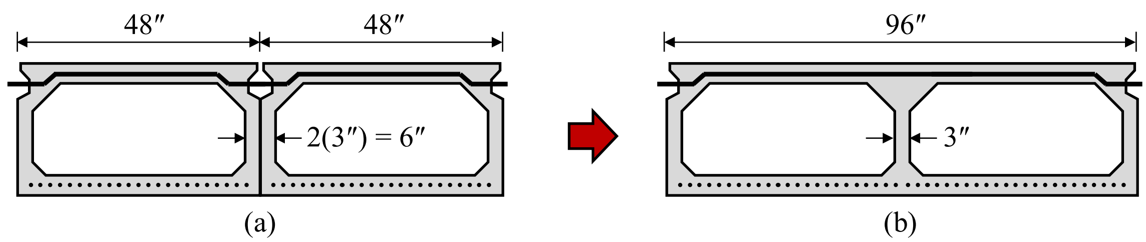
Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
60	1.302	-0.336	1.082	1,237	1,736	1,712	-0.14	0.18	Min. Flex. Reinf.
65	1.302	-0.506	1.257	1,402	1,737	1,715	-0.04	0.23	Min. Flex. Reinf.
70	1.302	-0.683	1.440	1,575	1,737	1,717	0.10	0.29	Min. Flex. Reinf.
75	1.519	-0.720	1.602	1,755	1,880	1,830	0.12	0.36	Strength I
80	1.736	-0.765	1.772	1,942	2,023	1,943	0.15	0.45	Strength I
85	1.953	-0.818	1.951	2,137	2,165	2,055	0.21	0.54	Strength I
90	2.387	-0.738	2.109	2,339	2,428	2,275	0.05	0.64	Strength I
95	2.604	-0.809	2.306	2,549	2,565	2,388	0.16	0.76	Strength I
100	3.038	-0.750	2.483	2,766	2,812	2,606	0.01	0.90	Strength I
105	3.472	-0.703	2.670	2,991	3,052	2,823	-0.13	1.04	Strength I
110	3.906	-0.668	2.866	3,223	3,290	3,039	-0.27	1.20	Strength I
115	4.340	-0.642	3.072	3,464	3,528	3,255	-0.38	1.38	Strength I
120	4.774	-0.628	3.287	3,712	3,766	3,469	-0.46	1.57	Strength I
125	5.292	-0.575	3.502	3,967	4,041	3,723	-0.66	1.78	Strength I
130	5.880 [d]	-0.575	4.095	4,231	4,356	3,946	0.48	2.01 [c]	Strength I
135	6.174 [d]	-0.668	4.353	4,502	4,530	4,096	0.91	2.25 [c]	Strength I
140	6.762 [d]	-0.607	4.585	4,780	4,837	4,380	0.78	2.52 [c]	Strength I

[a] 0.6-inch diameter strands were used up to 120-foot spans; 0.7-inch diameter strands were used for spans 125 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Deflections do not meet the optional live load deflection checks for span lengths of 130 feet and longer.
[d] Two fully stressed top strands at 40 inches from bottom are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.

Further Discussion on Box Beam Section Geometry

The design resources and tables presented in this report for the proposed modified box beam series generally represent other voided shapes with similar section depths.

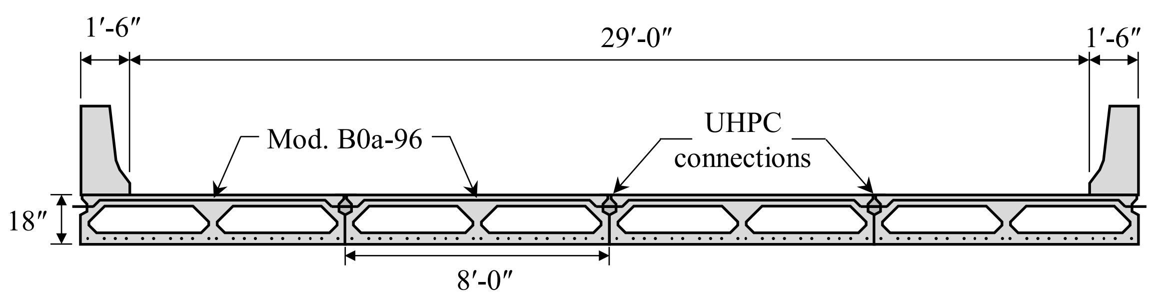
Additionally, adjacent members could be reasonably combined into a single member, as shown in Figure 22. This approach can offer several benefits, including fewer picks during construction and reduced number of connections, which would facilitate and accelerate construction. It would also further reduce the weight of the section and superstructure, as the 3-inch webs for adjacent members could be combined into one 3-inch web.



Source: FHWA.

Figure 22. Illustration. Option to combine adjacent modified box beam sections.

An example of where this modification may be beneficial is for a 45-foot span length with the bridge cross section shown in Figure 23. In this case, the bridge can be constructed using four adjacent Mod. B0a-96 UHPC twin box beams.



Source: FHWA.

Figure 23. Illustration. Example of a bridge cross section for 45-foot span with Mod. B0a-96 UHPC box beams.

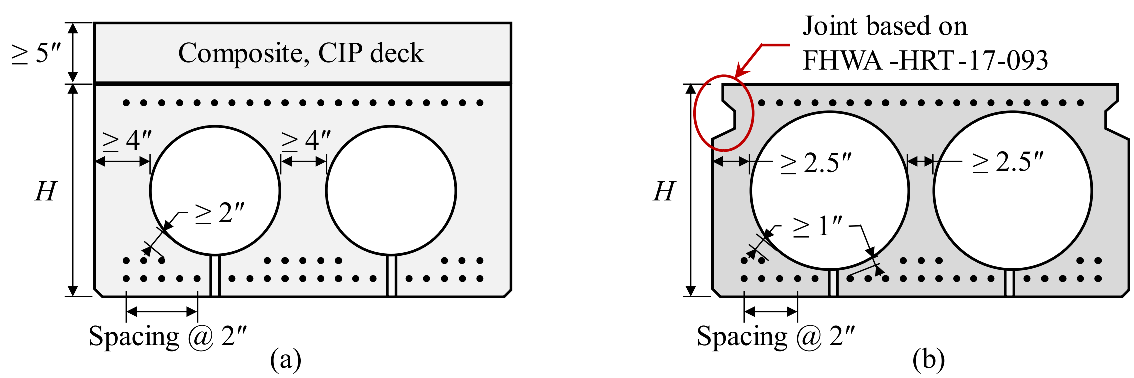
The area, weight per beam, and overall bridge weight for a 45-foot span bridge constructed using Mod. B0a-48 and Mod. B0a-96 box beams are summarized in Table 30. The wider Mod. B0a-96 box beam section offer several advantages compared to the two Mod. B0a-48 beams. These advantages include a lighter superstructure, with each Mod. B0a-96 approximately 2.76 kips lighter than two Mod. B0a-48, and a reduced number of crane picks and cast-in-place construction joints during erection of the bridge. Additionally, the 46-foot-long Mod. B0a-96 could be shipped on a single truck without requiring a permit.

Table 30. Beam and bridge weights for 45-foot span with B0a-48 and B0a-96 UHPC box beams.

Section	w_{beam} (inch)	A_g (inch^2)	w_g (kip/ft)	W_{beam} (kips)	n_{beams}	W_{bridge} (kips)
Mod. B0a-48	48	462.2	0.514	23.6	8	216.0
Mod. B0a-96	96	870.4	0.967	44.5	4	204.9

MODIFIED VOIDED SLAB BEAMS

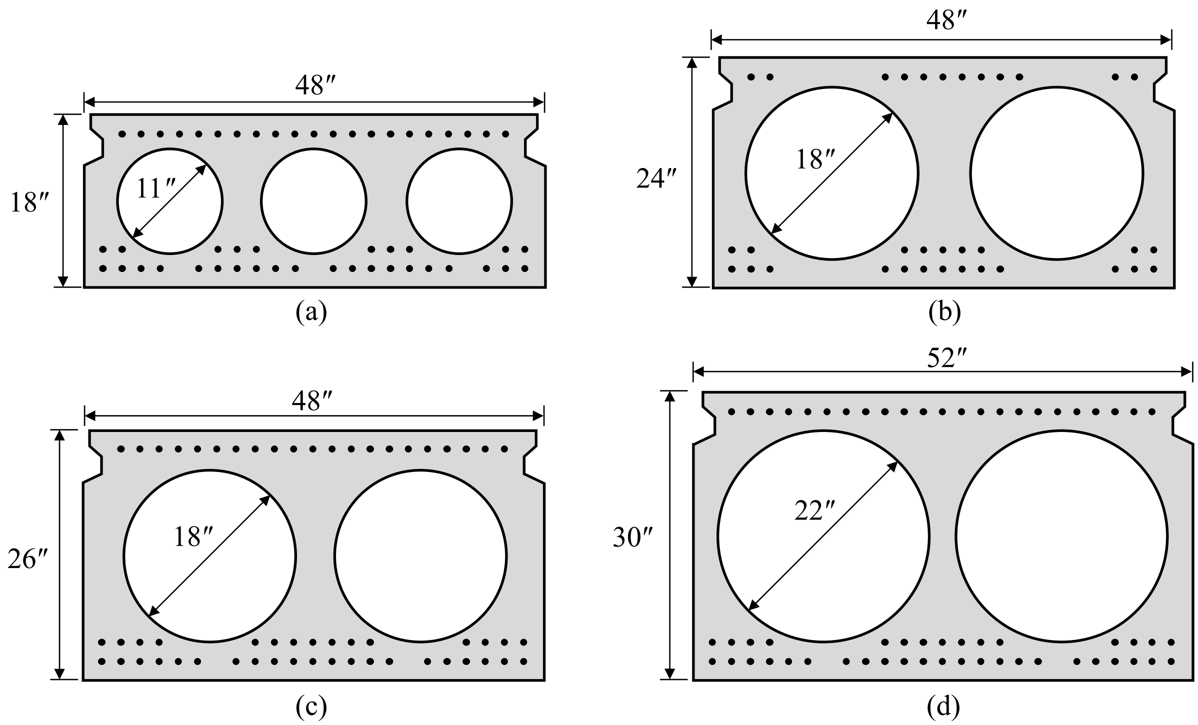
Many states have a voided slab beam series for short span bridges (e.g., Idaho, Washington, Oregon, Montana). Voided slab beams are similar to box beams except with different shaped voids. One example is the Washington DOT voided slab section, shown in Figure 24 (a), which can span up to 75 feet with a 30-inch-deep section and 5-inch composite, cast-in-place conventional concrete deck. This voided slab section has a different number of circular voids of different diameters based on the section depth. There are minimum cover distances specified between the void and sides (4 inches), between voids (4 inches), and between the void and closest prestressing strands (2 inches).



Source: FHWA.

Figure 24. Illustration. Example of (a) conventional concrete voided slab section from Washington DOT design standards (2021) and (b) possible modified UHPC voided slab section.

Similar to the UHPC box beam section described earlier in this chapter, the conventional concrete voided slab section could be optimized for UHPC by increasing the void sizes, decreasing the cover requirements, removing the cast-in-place composite deck, and adding a joint block out for connecting adjacent members, as shown in Figure 24 (b). The minimum cover between voids, void and side, and void and strands for the UHPC section would depend on the allowable tolerances for the location of the void. Some strand lines may need to be removed as the void size increases for some section depths, but this may be offset by utilizing 0.7-inch diameter strands for longer span lengths. Possible void diameters and section shapes for a UHPC voided slab section are shown in Figure 25. These void diameters and strand patterns are based on the current voided slab beam heights and widths for the Washington DOT voided slab beam series.



Source: FHWA.

Figure 25. Illustration. Possible modified UHPC voided slab section based on Washington DOT design standards (2021).

The UHPC modified voided slab section does not result in as large of a weight per unit length savings when compared against the results from the UHPC modified box beams . A comparison between the gross area and unit weight of the Washington DOT conventional concrete voided slab section compared to the modified UHPC voided slab section is provided in Table 31. The UHPC modified voided slab sections are on average 11.4 percent lighter than the similar depth conventional concrete voided slab sections. However, there would be a much larger weight savings for the UHPC sections when factoring in that a 5-inch cast-in-place, composite deck would no longer be required for the UHPC section; the UHPC section without a deck is on average 32.3 percent lighter than the conventional concrete section with 5-inch deck.

Table 31. Summary of area and unit weight for conventional concrete and UHPC voided slab sections.

Section Type	h (inch)	$A_{g,CC}$ (inch^2)	$w_{g,CC}$ (kip/ft)	$A_{g,UHPC}$ (inch^2)	$w_{g,UHPC}$ (kip/ft)	% Lighter
Mod. 18-inch Voided Slab	18	655.0	0.682	559.6	0.622	8.9%
Mod. 24-inch Voided Slab	24	741.0	0.772	624.0	0.693	10.2%
Mod. 26-inch Voided Slab	26	835.0	0.870	720.0	0.800	8.0%
Mod. 30-inch Voided Slab	30	1021.0	1.064	780.7	0.867	18.4%

The section properties for the UHPC modified voided slab sections and the maximum possible span lengths are summarized in Table 32. The maximum span lengths are provided with and without considering the optional live load deflection limits in AASHTO LRFD BDS. The section and maximum spans were determined using the same bridge configuration, shown in Figure 19, and the same material properties used for the box beam analyses. The maximum span lengths possible without considering the optional live load deflection limits depend on the possible strand configuration shown in Figure 25.

Table 32. Summary of section properties and maximum span lengths for UHPC voided slab sections.

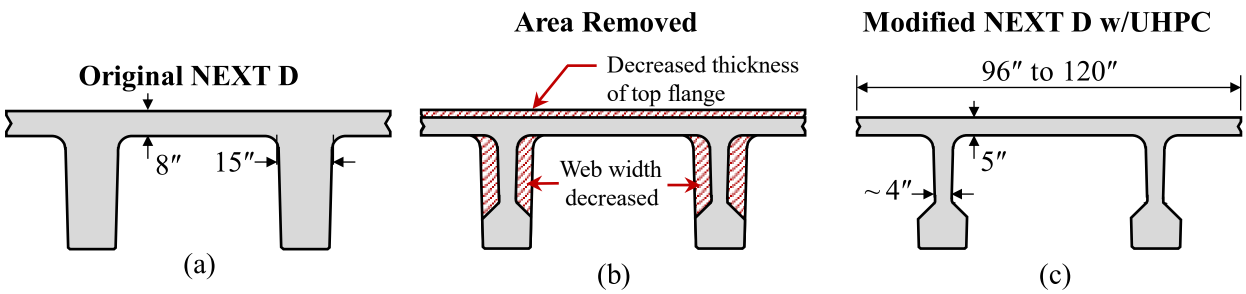
Section Type	h (inch)	A_g (inch^2)	I_g (inch^4)	y_b (inch)	w_g (kip/ft)	L_{max} (w/defl. limit)	L_{max} (w/o defl. limit)
Mod. 18-inch Voided Slab	18	559.6	20,562	8.82	0.622	45 ft.	80 ft.
Mod. 24-inch Voided Slab	24	624.0	43,591	11.75	0.693	65 ft.	85 ft.
Mod. 26-inch Voided Slab	26	720.0	58,256	12.76	0.800	75 ft.	115 ft.
Mod. 30-inch Voided Slab	30	780.7	91,458	14.72	0.867	90 ft.	125 ft.

In general, the maximum span lengths possible considering the deflection limits for the UHPC modified voided slab section are similar to the maximum span lengths for the UHPC modified box beam, comparing Table 23 and Table 32, while the modified voided slab sections will be heavier than the modified box beam sections. The maximum span lengths without deflection limits are longer for the modified voided slab section than the modified box beam section because there are more available strand locations on the tension side of the member for the voided slab section.

MODIFIED NEXT D BEAMS

Introduction to NEXT D Beam Design Tables

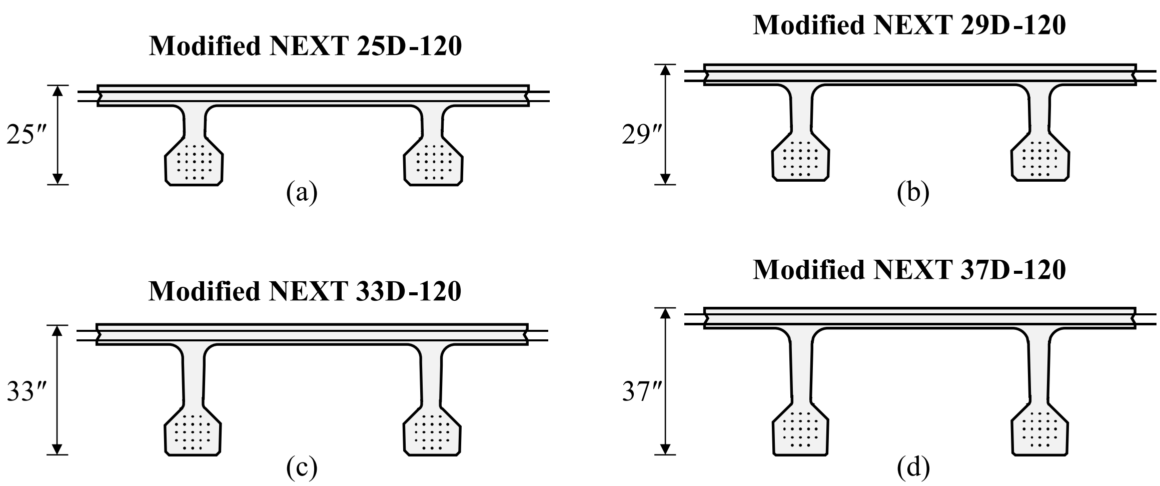
A modified UHPC NEXT D beam section was developed as another possible short span solution without a cast-in-place composite deck. The modified NEXT D beam section is based on the NEXT D beam series, as shown in Figure 26. Many precasters have NEXT D beam forms and many states are regularly using this section type, which may make a UHPC section shape utilizing these forms an attractive solution for early implementation. The web width can be reduced by using blockouts in the forms, as shown in Figure 26 (b) and (c). To create an optimized UHPC NEXT D beam section, the top flange thickness can be decreased from 8 inches thick to 5 inches thick to further reduce the weight of the section. Internal voids or eccentric blockouts may also be used to decrease the web width; alternate section configurations are discussed in a later section.



Source: FHWA.

Figure 26. Illustration. Example of optimization of previously used section for UHPC based on NEXT D beam.

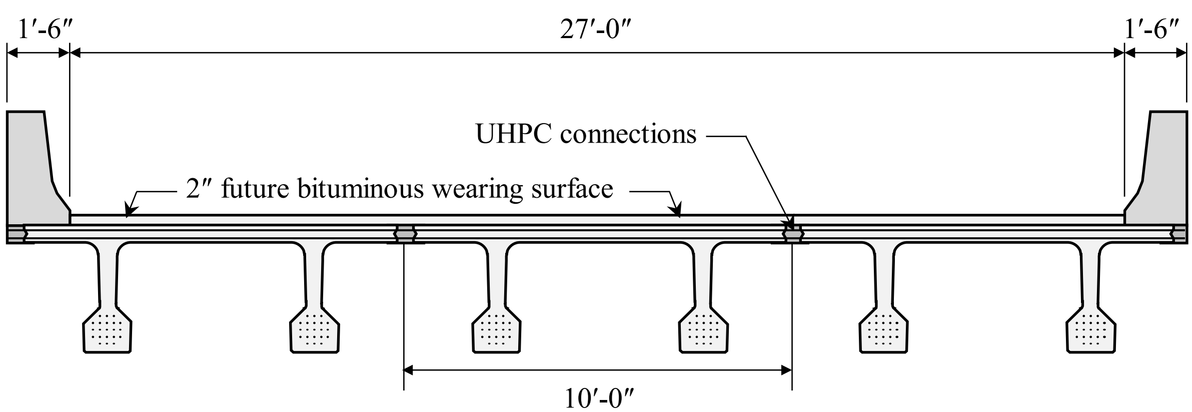
The existing NEXT D beam series has section depths of 28, 32, 36, and 40 inches with the 8-inch-deep top flange. The proposed UHPC geometry has a reduced top flange thickness of 5 inches, which reduces the overall height of each section by 3 inches, as shown in Figure 27. The sections can be fabricated using existing NEXT D beam forms with blockouts and with the same existing strand pattern. A minimum of six strands (three in the bottom of each stem) are used in the designs.



Source: FHWA.

Figure 27. Illustration. Possible modified NEXT D beam section series.

The design tables and graphs developed for this modified NEXT D beam series were designed using the bridge cross section shown in Figure 28. The bridge consists of three 120-inch-wide adjacent NEXT D beams connected with UHPC connections and is designed to carry two lanes of traffic.



Source: FHWA.

Figure 28. Illustration. Bridge cross section for configuration used to develop design tables and design aids for the modified NEXT D beam section series.

Some of the details on the span configuration are as follows:

- Total beam length: $L_T = L + 1$ ft
- Support width: $b_s = 0.5$ ft

The UHPC material properties for the design guides are as follows:

- Compressive strength at transfer: $f'_{ci} = 14.0$ ksi
- Compressive strength for use in design and analyses: $f'_c = 17.5$ ksi

- Correction factor for modulus of elasticity: $K_1 = 1.0$
- UHPC unit weight (including reinforcement): $w_c = 0.160$ kcf
- Reduction factor for compression: $\alpha_u = 0.85$
- Ultimate compression strain: $\varepsilon_{cu} = 0.0035$
- Effective cracking strength: $f_{t,cr} = 1.0$ ksi
- Effective cracking strength at transfer: $f_{t,cri} = 0.75 f_{t,cr} = 0.75$ ksi
- Crack localization strength: $f_{t,loc} = 1.0$ ksi
- Crack localization strain: $\varepsilon_{t,loc} = 0.0035$
- Reduction factor for tension: $\gamma_u = 1.0$

The material properties for the conventional steel reinforcement (Grade 60) are as follows:

- Modulus of elasticity: $E_s = 29{,}000$ ksi
- Yield strength: $f_{sy} = 60$ ksi

The material properties for the prestressing strands are as follows:

- Low-relaxation
- Modulus of elasticity: $E_p = 28{,}500$ ksi
- Ultimate strength: $f_{pu} = 270$ ksi
- Yield strength: $f_{py} = 243$ ksi
- Specified rupture strain for strands: $\varepsilon_{pu} = 0.035$

The following stress-strain relationship was used for the prestressing strands:

$$f_{ps} = \varepsilon_{ps}\left[887 + \frac{27{,}613}{(1 + (112.4\varepsilon_{ps})^{7.36})^{(1/7.36)}}\right] \leq 270 \text{ ksi}$$

Two different strand diameters were used to develop the design guides. Strands with a 0.6-inch diameter were used when possible. 0.7-inch diameter strands were used for longer span lengths requiring a larger amount of prestressing.

- Area of one 0.6-inch diameter strand: $A_{p,0.6in} = 0.217$ inch^2
- Area of one 0.7-inch diameter strand: $A_{p,0.7in} = 0.294$ inch^2

The parameters related to creep coefficients and shrinkage strains used to develop the design tables are as follows.

- UHPC age at transfer: $t_i = 1$ day
- UHPC age at field placement: $t_d = 90$ days
- UHPC age at final time: $t_f = 36{,}500$ days
- Average ambient relative humidity: $H = 70$ percent
- Correction factor for creep: $K_3 = 1.0$
- Correction factor for shrinkage: $K_4 = 1.0$

Transformed section properties were used to calculate stresses. Gross section properties were used to calculate deflections.

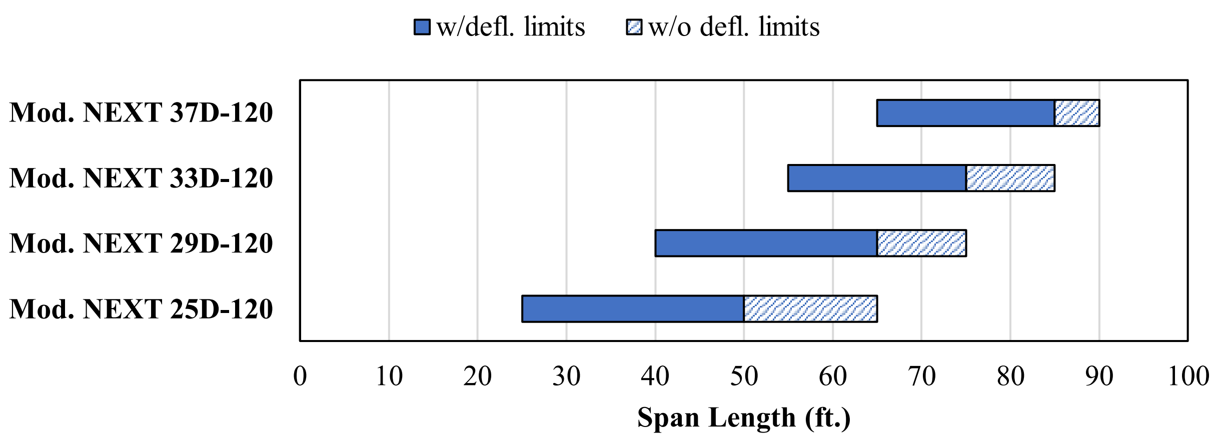
Possible Sections for 40- to 85-foot Spans

The section properties for the NEXT D beam sections and maximum possible span lengths are summarized in Table 33. The maximum span lengths are provided with and without considering the optional live load deflection limits in AASHTO LRFD BDS.

Table 33. Summary of section properties and maximum span lengths for UHPC NEXT D beam sections for provided design tables.

Section Type	h (inch)	A_g (inch^2)	I_g (inch^4)	y_b (inch)	w_g (kip/ft)	L_{max} (w/defl. limit)	L_{max} (w/o defl. limit)
Mod. NEXT 25D-120	25.0	940.9	59,492	17.03	1.045	50 ft	65 ft
Mod. NEXT 29D-120	29.0	973.4	89,459	19.66	1.082	65 ft	75 ft
Mod. NEXT 33D-120	33.0	1004.9	126,471	22.24	1.117	75 ft	85 ft
Mod. NEXT 37D-120	37.0	1036.9	171,041	24.78	1.152	85 ft	90 ft

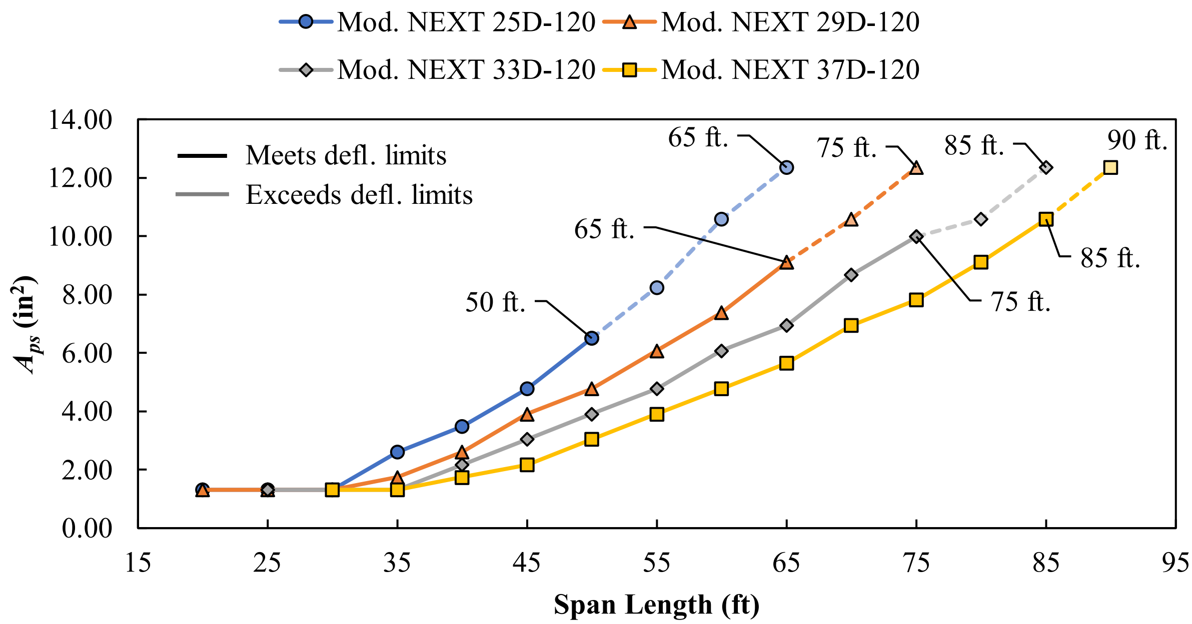
The span length ranges for each beam in the modified NEXT D beam series are shown in Figure 29. A 20- to 25-foot range was considered for each section depth, considering optional live load deflection criteria. Deeper sections could be used for shorter span lengths than those shown herein.



Source: FHWA.

Figure 29. Graph. Practical span length ranges for modified NEXT D beam series.

The total area of prestressing steel on the flexural tension side of the member (A_{ps}) required for different span lengths in the modified NEXT D beam section series is shown in Figure 30. The span lengths at which the design no longer satisfies the optional live load deflection criteria specified in AASHTO LRFD BDS Article 2.5.2.6.2 are plotted as dashed lines in Figure 30.



Source: FHWA.

Figure 30. Graph. Required prestressing area versus span length for modified NEXT D beam series.

More details on each of the analysis points shown in Figure 30 are provided in the following section.

Additional Design Resources

Details on the designs for each span length are provided in Table 35 through Table 38 for the modified NEXT D beam section series.

The stresses due to the Service I load combination are provided.

- Stress in extreme tension fiber due to Service I load combination: $f_{t,SeI,m}$
- Stress in extreme compression fiber due to Service I load combination: $f_{c,SeI,m}$

These stresses are compared to the stress limits at service.

- Compressive stress limit due to Service I load combination: $f_{c,a} = 0.60\phi_w f'_c = 10.5$ ksi
- Tension stress limit due to Service III load combination: $f_{t,a} = \gamma_u f_{t,cr} = -1.0$ ksi
- Tension stress limit for members exposed to cyclic loads due to Service I load combination: $f_{t,a} = 0.95\gamma_u f_{t,cr} = -0.95$ ksi

The cracking moment (M_{cr}), demand due to the Strength I load combination at midspan ($M_{u,m}$), and factored resistance at midspan ($M_{r,m}$) are also provided in the table. The two associated design checks related to these are as follows.

- Flexural strength check: $M_{r,m} = \phi M_{n,m} \geq M_{u,m}$

- Minimum flexural reinforcement: $M_{r,m} = \phi M_{n,m} \geq$ minimum of ($1.33M_u$ and M_{cr})

Two deflections are provided in the design tables. The total long-term deflection due to sustained loads ($\Delta_{sus,LT}$) includes the deflection due to the prestressing, self-weight, barrier weight, and weight of the future wearing surface. A positive deflection is downward; negative is upward. The live load deflection is calculated based on AASHTO LRFD BDS Article 3.6.1.3.2.

Live load deflection: Δ_L = maximum of Δ_{LT} and $(0.25\Delta_{LT} + \Delta_{LL})$

The live load deflection can be compared to the optional live load deflection limit in AASHTO LRFD BDS Article 2.5.2.6.2.

Live load deflection limit: $\Delta_L \leq L / 800$

The live load deflections exceeding the optional limit are highlighted in the design tables.

The details in the design tables are provided assuming the possible strand configuration summarized in Table 34.

Table 34. Possible strand location in modified NEXT D beam series.

Layer #	y_p (in)	$n_{strands}$
1	2	6
2	4	10
3	6	10
4	8	10
5	10	6
Top	$h - 2$	4

Strands are added filling the layers from bottom to top. 0.6-inch diameter strands are used until all strand locations are filled; 0.7-inch diameter strands are then used for longer spans. Four top strands were added when release stresses exceeded tensile stress limits.

Table 35. Design table for Modified NEXT 25D-120 Beam (25-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302	0.403	0.149	354	918	859	-0.15	0.01	Min. Strands
25	1.302	-0.171	0.423	634	927	861	-0.20	0.04	Min. Strands
30	1.302	-0.748	0.699	913	939	863	-0.24	0.09	Min. Strands
35	2.604	-0.358	0.839	1,194	1,276	1,198	-0.66	0.17	Strength I
40	3.472	-0.340	1.039	1,482	1,496	1,417	-1.08	0.28	Strength I
45	4.774	-0.175	1.265	1,777	1,777	1,697	-1.70	0.43	Strength I
50	6.510	0.110	1.517	2,082	2,116	2,037	-2.56	0.61	Strength I
55	8.246	0.243	1.837	2,397	2,408	2,334	-3.46	0.84 [c]	Strength I
60	10.584 [d]	0.785	2.317	2,723	2,952	2,825	-4.85	1.12 [c]	Strength I
65	12.348 [d]	0.728	2.711	3,060	3,195	3,081	-5.85	1.46 [c]	Strength I

[a] 0.6-inch diameter strands were used up to 55-foot spans; 0.7-inch diameter strands were used for spans 60 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Deflections do not meet the optional live load deflection checks for span lengths of 55 feet and longer.
[d] Four fully stressed top strands at 23 inches from bottom are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.

Table 36. Design table for Modified NEXT 29D-120 Beam (29-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
20	1.302	0.462	0.108	367	1,113	1,076	-0.12	0.01	Min. Strands
25	1.302	0.004	0.330	658	1,122	1,078	-0.17	0.03	Min. Strands
30	1.302	-0.456	0.554	946	1,134	1,080	-0.20	0.06	Min. Strands
35	1.736	-0.612	0.738	1,238	1,281	1,220	-0.34	0.11	Strength I
40	2.604	-0.486	0.888	1,536	1,559	1,492	-0.64	0.19	Strength I
45	3.906	-0.147	1.026	1,842	1,942	1,874	-1.18	0.28	Strength I
50	4.774	-0.155	1.229	2,157	2,186	2,110	-1.64	0.41	Strength I
55	6.076	0.014	1.434	2,482	2,519	2,442	-2.32	0.56	Strength I
60	7.378	0.095	1.679	2,819	2,822	2,745	-3.05	0.75	Strength I
65	9.114	0.260	1.968	3,167	3,179	3,104	-3.96	0.97	Strength I
70	10.584 [c]	0.512	2.400	3,527	3,692	3,545	-5.01	1.23 [d]	Strength I
75	12.348 [c]	0.574	2.724	3,900	4,023	3,889	-6.05	1.54 [d]	Strength I

[a] 0.6-inch diameter strands were used up to 65-foot spans; 0.7-inch diameter strands were used for spans 70 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed top strands at 27 inches from bottom are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 70 feet and longer.

Table 37. Design table for Modified NEXT 33D-120 Beam (33-inches deep).

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
25	1.302	0.114	0.268	679	1,328	1,305	-0.14	0.02	Min. Strands
30	1.302	-0.266	0.456	977	1,340	1,308	-0.17	0.04	Min. Strands
35	1.302	-0.654	0.649	1,278	1,351	1,310	-0.19	0.08	Min. Strands
40	2.170	-0.467	0.766	1,585	1,687	1,635	-0.44	0.13	Strength I
45	3.038	-0.315	0.894	1,900	2,010	1,952	-0.77	0.20	Strength I
50	3.906	-0.219	1.044	2,224	2,320	2,251	-1.15	0.29	Strength I
55	4.774	-0.172	1.216	2,559	2,609	2,533	-1.58	0.40	Strength I
60	6.076	0.049	1.387	2,906	3,013	2,934	-2.25	0.53	Strength I
65	6.944	0.001	1.602	3,264	3,271	3,186	-2.78	0.69	Strength I
70	8.680	0.267	1.829	3,634	3,733	3,646	-3.70	0.87	Strength I
75	9.996 [c]	0.473	2.251	4,018	4,286	4,107	-4.54	1.09	Strength I
80	10.584 [c]	0.237	2.501	4,414	4,462	4,280	-5.05	1.34 [d]	Strength I
85	12.348 [c]	0.372	2.778	4,823	4,883	4,712	-6.11	1.62 [d]	Strength I

[a] 0.6-inch diameter strands were used up to 70-foot spans; 0.7-inch diameter strands were used for spans 75 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed top strands at 31 inches from bottom are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 80 feet and longer.

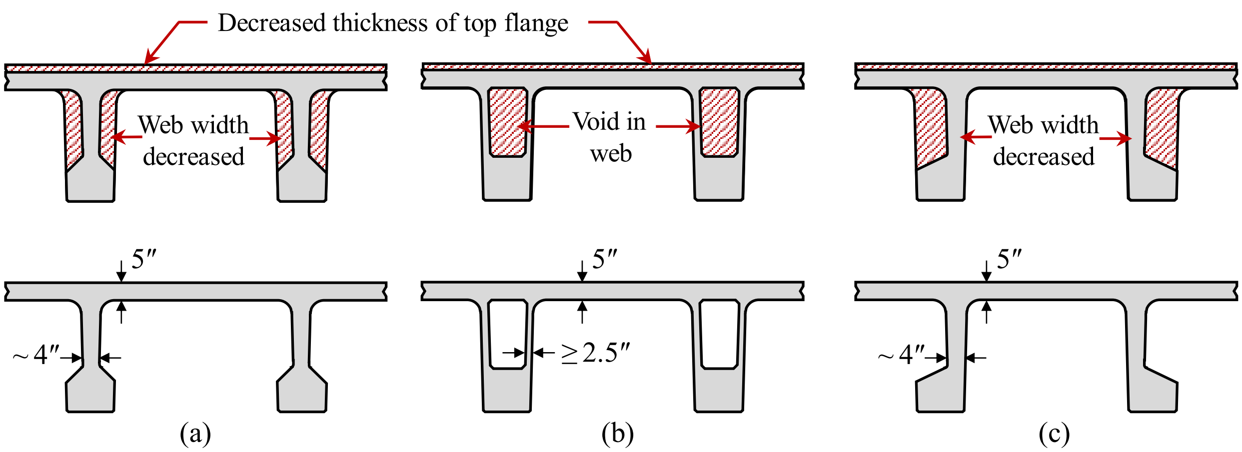
Table 38. Design table for Modified NEXT 37D-120 Beam (37-inches deep)

Span (ft)	A_{ps} [a] (inch^2)	$f_{t,SeI,m}$ (ksi)	$f_{c,SeI,m}$ (ksi)	$M_{u,m}$ (k-ft)	$M_{r,m}$ (k-ft)	M_{cr} (k-ft)	$\Delta_{sus,LT}$ [b] (inch)	Δ_L [b] (inch)	Controlling
30	1.302	-0.136	0.386	1,005	1,558	1,545	-0.15	0.03	Min. Strands
35	1.302	-0.465	0.552	1,314	1,569	1,547	-0.17	0.06	Min. Strands
40	1.736	-0.522	0.684	1,629	1,777	1,737	-0.29	0.10	Strength I
45	2.170	-0.594	0.824	1,953	1,977	1,925	-0.43	0.15	Strength I
50	3.038	-0.417	0.935	2,286	2,351	2,291	-0.74	0.21	Strength I
55	3.906	-0.291	1.065	2,630	2,711	2,638	-1.12	0.29	Strength I
60	4.774	-0.209	1.215	2,986	3,049	2,968	-1.53	0.39	Strength I
65	5.642	-0.152	1.374	3,354	3,384	3,295	-2.01	0.51	Strength I
70	6.944	0.041	1.549	3,734	3,835	3,735	-2.69	0.65	Strength I
75	7.812	0.018	1.745	4,127	4,138	4,032	-3.23	0.80	Strength I
80	9.114	0.110	1.966	4,533	4,537	4,428	-3.96	0.99	Strength I
85	10.584 [c]	0.392	2.362	4,953	5,226	5,011	-4.86	1.19	Strength I
90	12.348 [c]	0.582	2.597	5,386	5,741	5,530	-5.92	1.43 [d]	Strength I

[a] 0.6-inch diameter strands were used up to 80-foot spans; 0.7-inch diameter strands were used for spans 85 feet and longer.
[b] Positive deflection is downward; negative deflection indicates upward deflection.
[c] Four fully stressed top strands at 35 inches from bottom are required to meet stress checks at time of transfer. These are not included in the A_{ps} shown.
[d] Deflections do not meet the optional live load deflection checks for span lengths of 90 feet.

Further Discussion on Modified NEXT D Beam Section Geometry

NEXT beams are a popular section because of their ease of fabrication. The web width needs to be reduced to make them a more efficient section and a practical solution when constructing them with UHPC. The reduction of the web width will increase the complexity of fabrication. There are several different options that may be considered to modify the NEXT beam geometry for use with UHPC while still utilizing existing NEXT beam forms and maintaining the strand configuration in the bottom of each stem, shown in Figure 31.



Source: FHWA.

Figure 31. Illustration. Possible options for reducing web width while maintaining available strand locations in bottom of stems.

A blockout could be installed on each side of the stem, as shown in Figure 31 (a). This would keep the symmetry of each stem without requiring a void. Having one web in each stem would also allow for the minimum overall web width, as a 3-to-4-inch web can be used in each stem. A challenge related to fabrication of this section is that the blockouts would need to be created such that they could be lifted out of the forms with the beam. There may also be challenges with this section associated with release of the prestressing strands as the blockouts may restrain camber depending on the blockout material.

Voids could also be used to reduce the width of the webs, as shown in Figure 31 (b). The use of a void would allow for the reduction in web width while still allowing the beam to be removed from forms like conventional concrete NEXT beams. The two disadvantages to this are: (1) web widths for UHPC members cannot practically be less than about 2.5 or 3 inches. This would create thicker overall webs than the option shown in Figure 31 (a). Additionally, producers have had trouble with holding the foam blockouts used to create the voids in place within specified tolerances. These blockouts are often very buoyant, creating large uplift forces during casting.

The third option would be to install a blockout only on the outside of each of the stems, as shown in Figure 31 (c), which would create a section like the pi girder. This may allow for outside forms to be created that could be removed to allow for easier removal of the modified NEXT D beam. The eccentricity between the center of the strands and center of the web is a potential challenge which would need to be investigated further for the more heavily prestressed modified NEXT D beam section.

The restrained shrinkage in the top flange of the modified NEXT D beam shape should also be considered. Depending on the UHPC formulation used and its rate of hydration, shrinkage characteristics, and creep characteristics, as well as the stiffness of the formwork, tensile stresses can be developed in the deck. Special formwork was used for pi girders to minimize the tensile stresses generated across the top flange of the girder during fabrication.

CHAPTER 4. CASE STUDIES

INTRODUCTION

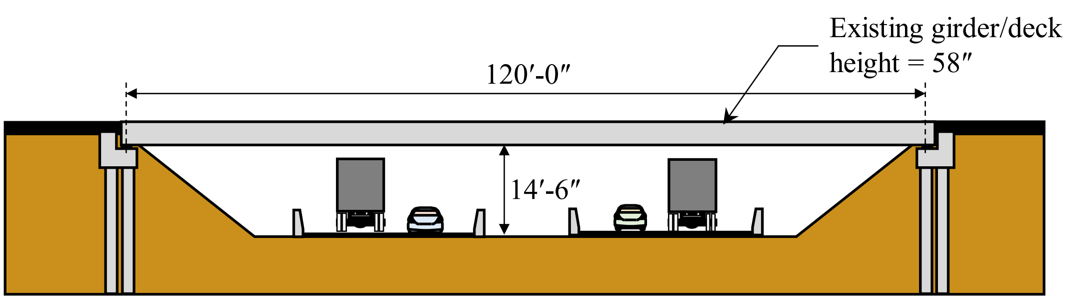
UHPC short-span solutions offer advantages over conventional concrete solutions. The following case studies are based on hypothetical bridges that are representative of many bridges found in the national bridge inventory. The case studies were selected to highlight some of the benefits gained by using several of the short-span UHPC solutions discussed in Chapter 3. These cases studies include the following.

- Case Study #1: Reducing superstructure weight and increasing overhead clearance.
- Case Study #2: Increasing span length to eliminate pier in water.
- Case Study #3: Eliminating shoulder piers and adding lanes underneath.
- Case Study #4: Short-span slab beam alternative.
- Case Study #5: Increased hydraulic clearance.

Some of the case studies discuss conventional concrete design alternatives. These conventional concrete designs were based on available design aids for conventional concrete sections in the PCI *Bridge Design Manual* or state DOT resources.

CASE STUDY #1: REDUCING SUPERSTRUCTURE WEIGHT AND INCREASING OVERHEAD CLEARANCE

An existing bridge with a 120-foot span is being considered for replacement, as shown in Figure 32. The existing bridge carries a high average daily traffic (ADT) route and spans a high ADT highway; therefore, accelerated bridge construction (ABC) techniques are being considered. The current bridge has a low overhead clearance (14 feet 6 inches) with the depth of the girder and deck totaling 58 inches.



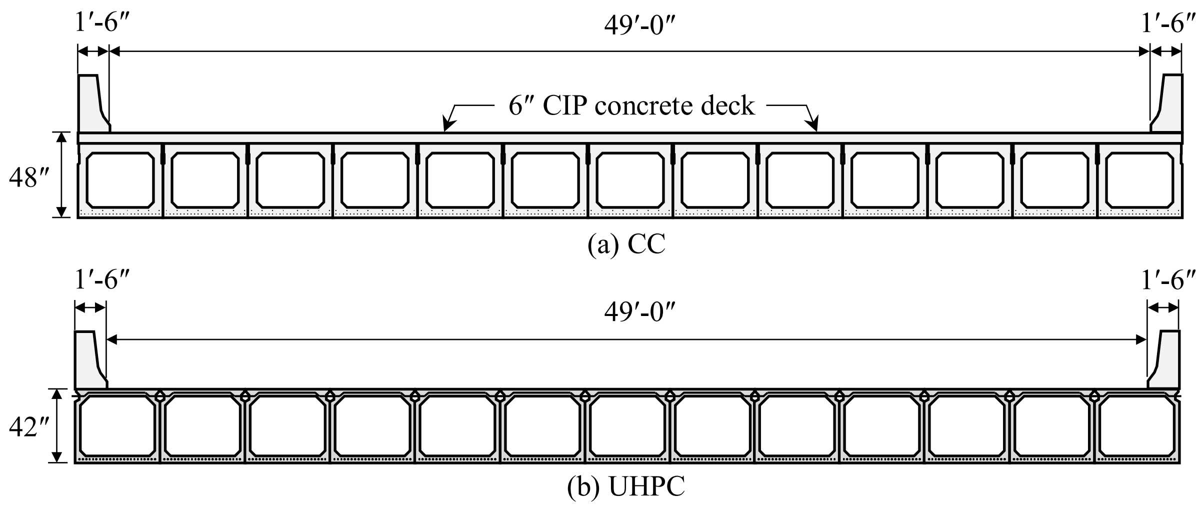
Source: FHWA.

Figure 32. Illustration. Elevation of existing bridge being considered for replacement in Case Study #1.

While the current superstructure was in poor condition, the substructure was generally in good condition. However, the substructure was designed using older design standards and would not meet current LRFD design criteria for new construction.

Adjacent box beams are being considered for the new bridge because they will allow for an accelerated construction schedule and a decreased superstructure depth, resulting in an increase in the overhead clearance for the highway under the bridge.

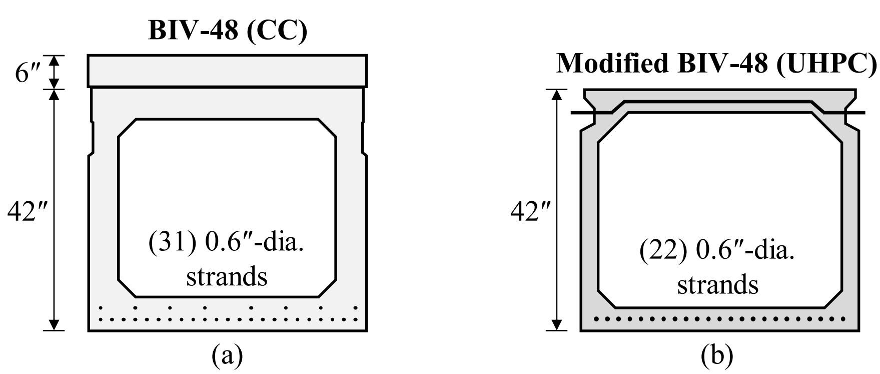
The two designs that are being considered are shown in Figure 33. The conventional concrete box beam design is based on the PCI *Bridge Design Manual* Preliminary Design Example No. 2. The UHPC box beam design is based on the design tables presented in Chapter 3.



Source: FHWA.

Figure 33. Illustration. Bridge cross section for 52-foot-wide bridge with (a) conventional concrete and (b) UHPC adjacent box beams.

The cross-section designs for the conventional concrete and UHPC box beam sections are shown in Figure 34. The conventional concrete design is 6 inches deeper than the UHPC section because it requires a 6-inch-thick cast-in-place composite concrete deck.



Source: FHWA.

Figure 34. Illustration. Cross section of design for 120-foot span for (a) conventional concrete and (b) UHPC adjacent box beam bridge.

The overall weight of the superstructure was calculated for the two design options using the following assumed parameters.

- Density of conventional concrete (including reinforcement): w_c = 0.150 kcf
- Density of UHPC (including reinforcement): w_c = 0.160 kcf
- Span length: L_{span} = 120 ft
- Beam length: L_{beam} = 121 ft
- Weight of barrier: 0.3 kips/ft/side

The total weight of the superstructure is shown Table 39.

Table 39. Superstructure weight for 120-foot span with conventional concrete and UHPC adjacent box beams.

Material	h_c (inch)	w_g [a] (k/ft)	W_{beam} [b] (kips)	W_{bridge} [c] (kips)
CC	48	1.178	142.5	1,925
UHPC	42	0.676	81.8	1,136

[a] Weight includes the cast-in-place deck for the conventional concrete design.
[b] Weight of a single beam including the cast-in-place deck for the conventional concrete design.
[c] Total weight of superstructure including the barrier weight.

Some discussion points related to the decision between the conventional concrete and UHPC box beam alternatives are as follows.

- **Construction Time**: The conventional concrete box beam design requires casting of a 6-inch cast-in-place concrete deck, which adds additional time to the construction schedule and may necessitate staged construction. The 42-inch-deep conventional concrete box beam cannot be used without a composite deck for a 120-foot span and no deeper box sections are available. The UHPC box beam does not require any concrete deck or overlay, and its top surface can be used as the riding surface. The UHPC joints connecting the adjacent beams can be opened to traffic when they reach 14 ksi, which can be achieved with some UHPC mixtures within one day. It may be possible to replace this superstructure in a weekend closure using the adjacent UHPC box beam configuration.
- **Possible Reuse of Substructure**: The current substructure is in good condition but is under-designed based on the current AASHTO LRFD BDS design criteria. The use of the UHPC superstructure option would lead to a superstructure that is 789 kips lighter than the conventional concrete option. This may allow for the substructure to be reused where it would not have been able to otherwise. Reusing the substructure would decrease the construction time, project cost, and carbon factor.
- **Overhead Clearance**: The current overhead clearance (14 foot 6 inches) is below the typically desired overhead clearance of 16 feet to lessen the risk of accidents involving vehicles hitting the low bridge. Both the conventional concrete and UHPC design options would improve the overhead clearance compared with the current superstructure. Using the UHPC superstructure would allow for the overhead clearance to increase to 16 feet without needing to raise the grade of the existing bridge.

- **Long-Term Durability**: The UHPC superstructure is expected to have a longer service life and necessitate less maintenance compared to a conventional concrete superstructure. This would be an important consideration for these high ADT routes.

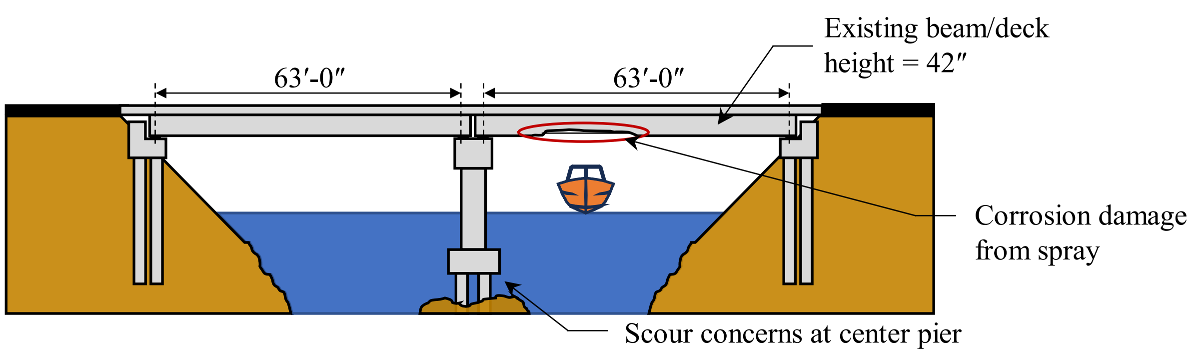
This hypothetical case study demonstrates several of the advantages that a UHPC superstructure may have over conventional concrete alternatives.

CASE STUDY #2: INCREASING SPAN LENGTH TO ELIMINATE PIER IN WATER

Vessel impacts of piers in or near navigable waterways is a long-standing challenge for bridge owners. Piers also affect and restrict the flow of water under the bridge, potentially leading to scour around the pier, challenging currents, restricted passageways near the bridge, and undesirable water flow actions on land nearby. Bridge owners and the broader public can benefit from the elimination of piers within waterways.

An existing two-span bridge over a tidal, navigable river near its mouth leading to the ocean needs to be replaced, as shown in Figure 35. The existing bridge has a two-span configuration with 63-foot-long spans and an existing beam and deck height of 42 inches. The bottoms of the existing beams are in the splash zone of water vessels passing at high speeds under the bridge. Additionally, there are concerns with scour undercutting the center pier.

The owner requires that new designs meet the optional live load deflection criteria in AASHTO LRFD BDS Article 2.5.2.6.2.



Source: FHWA.

Figure 35. Illustration. Elevation of existing bridge being considered for replacement in Case Study #2.

Adjacent box beams would be appropriate for this design due to the short span and shallow depth of the existing cross section. UHPC box beams are being considered for this design to extend the service life of the beams as they are exposed to the salt spray from the boats under the structure. Modified NEXT D beams could also be used for spans less than 85 feet. The adjacent box beams and modified NEXT D beams will also offer an accelerated construction schedule as discussed in Case Study #1.

Three of the possible configurations for this bridge are:

- Two spans with a shallower box beam section,
- Two spans with a shallower modified NEXT D beam, or
- One span with a deeper box beam section.

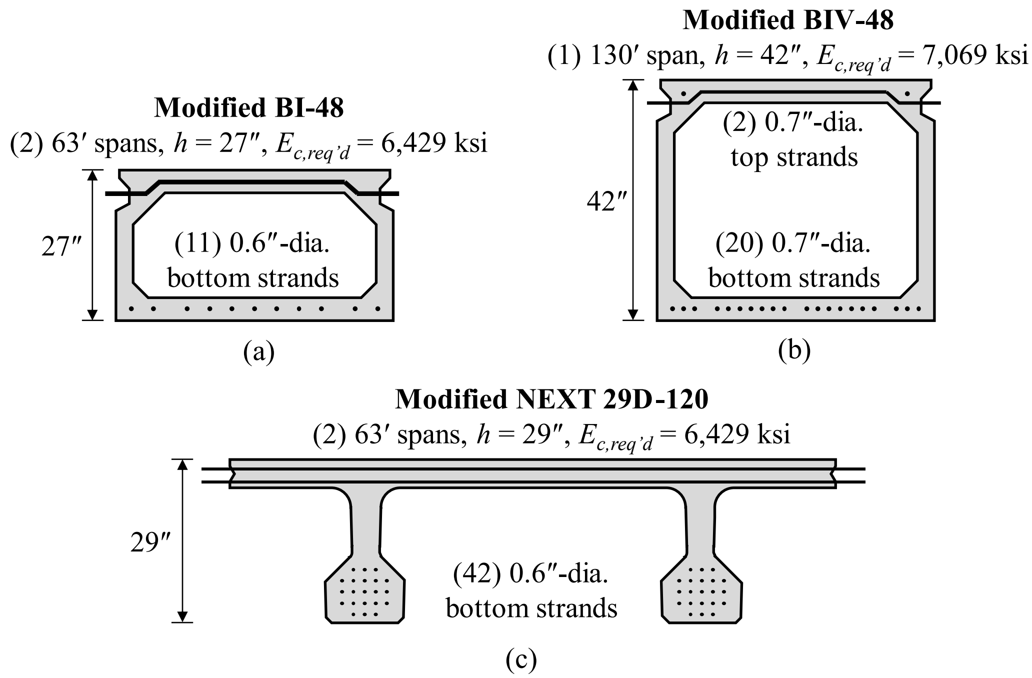
A two-span bridge configuration could be used with a shallower box beam or modified NEXT D beam section to gain some clearance in the navigable river. A one-span bridge configuration with a deeper box beam section could be used to eliminate the pier in the water. A summary of some of the different possible design configurations is shown in Table 40. The designs for the 63-foot span lengths are based on 65-foot span lengths from the design tables in Chapter 3.

A higher modulus of elasticity is required for some of the design to meet the optional live load deflection criteria. The higher required modulus can either be achieved by using a UHPC with a higher K_1 or higher compressive strength. The modulus of elasticity required for the design to meet the optional deflection criteria, $E_{c,req'd}$, is provided in Table 40. The modulus of elasticity required for Option A, $E_{c,req'd}$ = 6,429 ksi, is based on a K_1 = 1.0 and f'_c = 17.5 ksi.

Table 40. Summary of design options for Case Study #2.

Option	n_{spans}	L_{span} (feet)	Section	h (inch)	$E_{c,req'd}$ (ksi)	Δ_L (inch)	Δ_{limit} (inch)
A	2	63	Mod. BI-48	27	6,429	0.67	0.98
B	2	63	Mod. B0b-48	21	8,358	0.98	0.98
C	2	63	Mod. NEXT 29D-120	29	6,429	0.97	0.98
D	1	130	Mod. BIV-48	42	7,069	1.83	1.95

Four different combinations of bridge configuration and UHPC section types and depths are shown depending on the preference of the owner and the locally available UHPC material. Option A, with two 63-foot spans and a 27-inch-deep box beam section, and Option C, with two 63-foot spans and 29-inch-deep modified NEXT D-beam section, would be the only possibilities if a UHPC with higher modulus of elasticity was not available. If a higher modulus of elasticity UHPC material was available, then Options B or D may be possible. There are currently commercially available UHPC materials with a modulus of elasticity up to 9,400 ksi, but it is more common for UHPC to have modulus of elasticity less than 7,500 ksi. For this reason, the designs that will be considered for this case study are Options A, C, and D, as shown in Figure 36.



Source: FHWA.

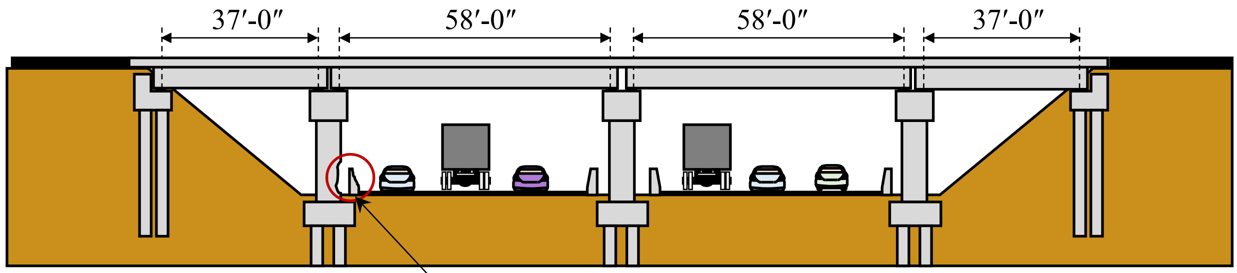
Figure 36. Illustration. Three different design possibilities for Case Study #2 with (a) Option A with two 63-foot spans with a Modified BI-48 section, (b) Option D with one 130-foot span with a Modified BIV-48 section and (c) Option C with two 63-foot spans with a NEXT 29D-120 section.

The two-span Option A would allow for the beams to be raised 15 inches while keeping the same riding surface elevation. Option C would be similar to Option A except using modified NEXT D beams; this option would allow for the beams to be raised 13 inches. However, Options A and C would require reconstructing the center pier in the water and adding some type of scour mitigation strategy.

The one-span Option D would allow for the center pier to be removed, so there would no longer be any pier in the water. This would eliminate concerns with scour in the center of the channel, vessel impact on the pier, disturbing aquatic organisms during construction and operation, and deterioration and maintenance of the pier in the water. Option D would maintain the same girder and deck thickness and the same riding surface, but no additional clearance would be added to the channel. Option D would likely be the preferred option for this case study but would require the use of a UHPC with a higher modulus of elasticity.

CASE STUDY #3: ELIMINATING SHOULDER PIERS AND ADDING LANES

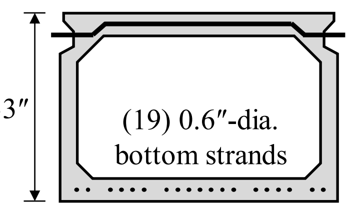
An existing four-span bridge with span lengths of 37-feet, 58-feet, 58-feet, and 37-feet is in need of replacement, as shown in Figure 37. The existing adjacent box beam structure, with a 33-inch total structural depth, has deterioration due to failure of the grouted shear keys between the beams. There is corrosion damage to one of the shoulder piers due to splashing from adjacent traffic under the bridge. There is also a desire to add an additional lane in one direction, to have four travel lanes in each direction.



Source: FHWA.

Figure 37. Illustration. Elevation of existing bridge being considered for replacement in Case Study #3.

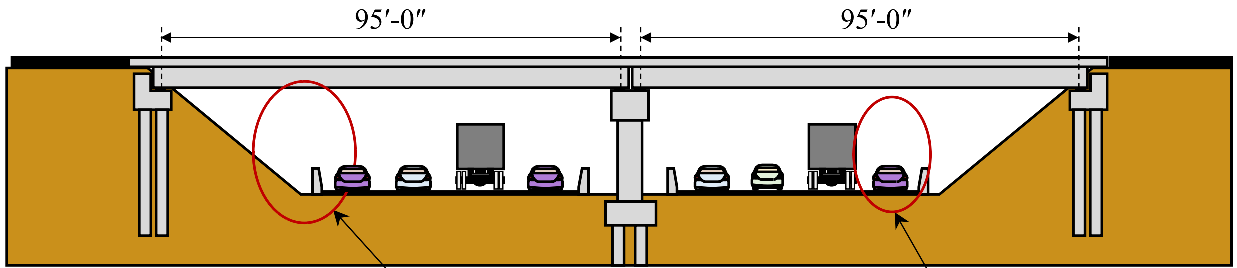
The shoulder piers can be eliminated if Modified BII-48 UHPC box beams are used with (19) 0.6-inch diameter prestressing strands on the flexural tension side of the beam, as shown in Figure 38.



Source: FHWA.

Figure 38. Illustration. Cross-section design option for Case Study #3 using Modified BII-48 UHPC box beam for 95-foot span.

This would change the configuration from a four-span bridge to a two-span bridge, as shown in Figure 39. The elimination of the shoulder piers also allows for an additional traffic throughput under the bridge.



Source: FHWA.

Figure 39. Illustration. Elevation of bridge configuration with UHPC box beams for Case Study #3.

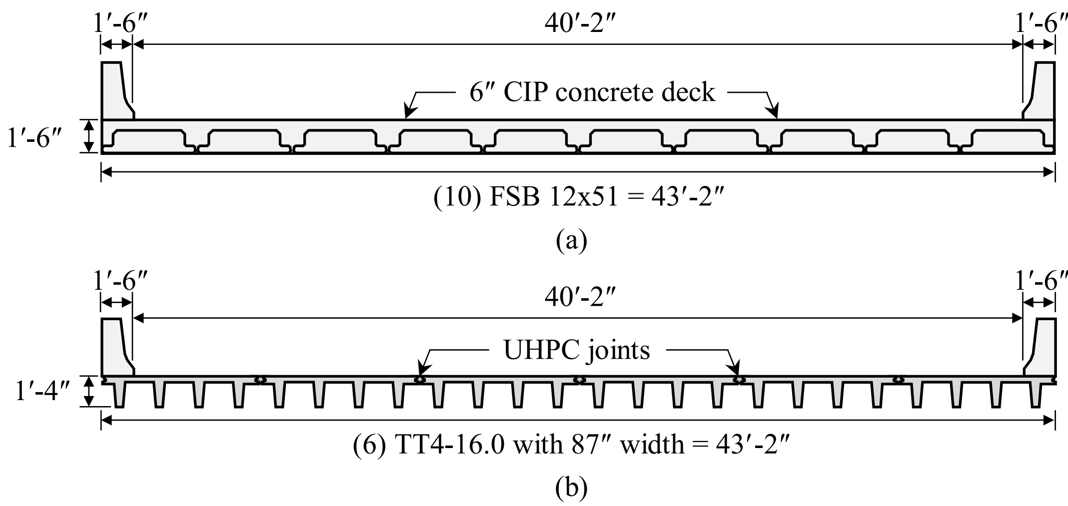
This design would also have many of the same benefits discussed in Case Study #1 and #2.

CASE STUDY #4: SHORT-SPAN SLAB BEAM ALTERNATIVE

Different slab beam systems are frequently used for short-span bridges; slab beams can be used for spans up to approximately 60 feet. One example of a slab beam system being used in the U.S. is the Florida Slab Beam (FSB). An FSB is a self-forming pretensioned slab beam where the joint and cast-in-place (CIP) composite deck are placed during the same cast. Standard FSB sections are available in 12-inch, 15-inch, and 18-inch depths with 6 or 6.5-inch composite decks for maximum span ranges up to approximately 63 feet.

An existing single span bridge with a span length of 30 feet is in need of replacement. The bridge sits low over brackish water in a relatively remote location with some issues with access.

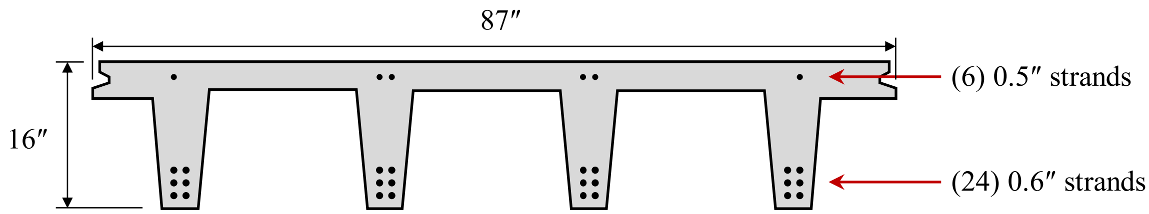
Two different designs are being considered, shown in Figure 40. The conventional concrete design is based on a 12-inch-deep FSB section with a 6-inch CIP composite deck, shown in Figure 40 (a). The UHPC design is based on a 16-inch-deep multi-stem T-beam section with four webs, as shown in Figure 40 (b).



Source: FHWA.

Figure 40. Illustration. Bridge cross section for 30-foot-span with (a) Florida Slab Beam (FSB) section and (b) multi-stem T-beams with four webs, 87-inch width, and 16-inch depth.

The UHPC multi-stem T-beam section and design for a 30-foot span length is shown in Figure 41. A multi-stem T-beam section with a 16-inch depth is sufficient for the 30-foot span length.



Source: FHWA.

Figure 41. Illustration. Cross section design for 30-foot span with multi-stem T-beams.

The overall weight of the superstructure was calculated for each design option using the following assumed parameters.

- Density of conventional concrete (including reinforcement): w_c = 0.150 kcf
- Density of UHPC (including reinforcement): w_c = 0.160 kcf
- Span length: L_{span} = 30 ft
- Beam length: L_{beam} = 31 ft
- Weight of barrier: 0.3 kips/ft/side

The total weight of the superstructure is shown Table 41.

Table 41. Superstructure weight for 30-foot span with conventional concrete FSBs and UHPC multi-stem T-beams.

Material	h_c (inch)	w_g (k/ft)	w_s (k/ft)	W_{beam} (kips)	W_{slab} [a] (kips)	n_{beams}	W_{bridge} [b] (kips)
CC	18	0.531	0.425	16.5	13.2	10	315
UHPC	16	0.449	0.000	13.9	0.0	6	102

[a] Slab weight is per beam.
[b] Total weight of superstructure including the barrier weight.

Some discussion points related to the decision between the conventional concrete slab beams and UHPC multi-stem T-beams alternatives are as follows.

- **Construction Considerations**: There are several construction-related advantages for the UHPC multi-stem T-beams over the CC slab beams. The UHPC beams are about 16 percent lighter than the precast slab beams, allowing for more beams to be transported per truck and allowing for lighter lifting equipment to be used for installation. Additionally, no CIP composite deck is required for the UHPC multi-stem T-beam system as they are connected by five narrow UHPC joints. This configuration can decrease the construction duration and allow for on-site missing of the field-placed material. Additionally, a shorter construction schedule can be beneficial for remote bridges where long detours are common. On-site mixing of the joint material can be a major advantage for remote locations where delivery of ready-mix concrete, needed for a CIP CC deck, can be challenging.
- **Superstructure Weight**: The overall weight of the UHPC superstructure is one third the weight of the CC superstructure (315 kips compared to 102 kips). This reduces the

demand on the substructure, potentially allowing for its reuse or unlocking substructure design options that would not have been possible with the heavier CC superstructure.

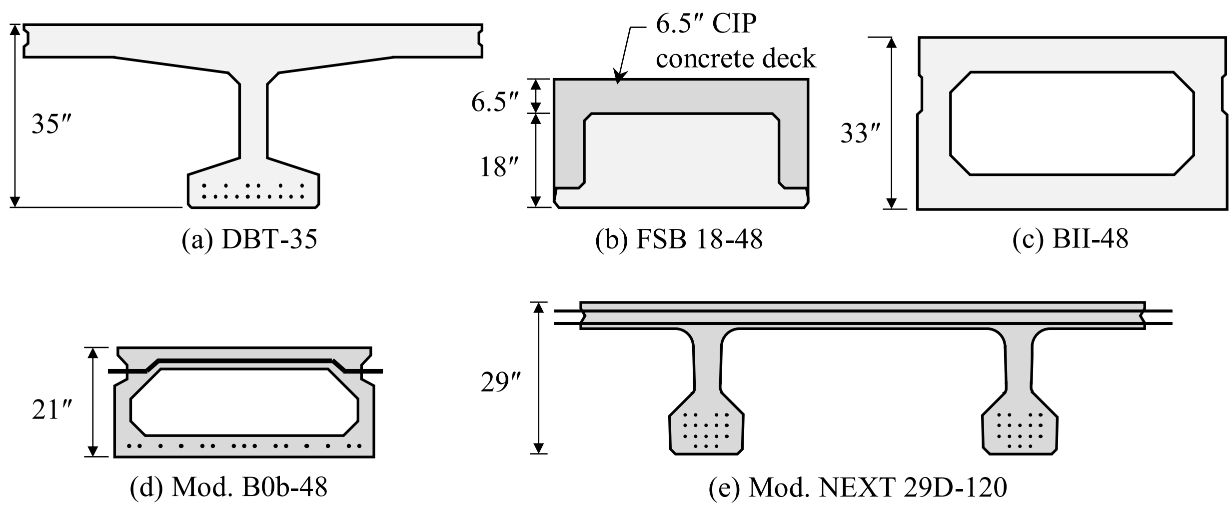
- **Long-Term Durability**: The UHPC superstructure would be expected to have a longer service life that requires less maintenance than the CC alternative. This is an important consideration for a remote bridge that may be difficult to access and where its loss could lead to a long detour.

This hypothetical case study demonstrates several of the advantages that a UHPC superstructure may have over conventional concrete alternatives for short-spans.

CASE STUDY #5: INCREASED HYDRAULIC CLEARANCE

A 60-foot-span water crossing in a rural location needs to be replaced. The bridge has hydraulic flow concerns. There are advantages to the bridge replacement and substructure construction for this case study if the overall weight of the bridge can be kept under 350 kips. The bridge has an overall width of 28 feet with a clear roadway width of 25 feet. The existing bridge has a 36-inch-deep superstructure.

There are five different options that are being considered for this bridge, as shown in Figure 42. The three conventional concrete options can all be used with accelerated construction techniques: 35-inch-deep decked bulb-tee (DBT) section, 18-inch-deep Florida Slab Beam (FSB) with a 6.5-inch CIP composite deck, and 33-inch-deep BII-48 adjacent box beam with transverse post-tensioning and no CIP deck. The 21-inch-deep Modified B0b-48 UHPC box beam can be used for this span length as well if the UHPC has a $E_c \geq 7{,}069$ ksi or if the deflection criteria are relaxed. Otherwise, the 27-inch-deep Modified BI-48 UHPC box beam would be required. The 29-inch-deep Modified NEXT 29D-120 could also be used for this span length.



Source: FHWA.

Figure 42. Illustration. Options for 60-foot span length: conventional concrete (a) decked bulb-tee (DBT), (b) Florida Slab Beam (FSB), and (c) BII-48 box beam and (d) UHPC Mod. B0b-48 box beam.

The overall weight of the superstructure was calculated for each design option using the following assumed parameters.

- Density of conventional concrete (including reinforcement): w_c = 0.150 kcf
- Density of UHPC (including reinforcement): w_c = 0.160 kcf
- Span length: L_{span} = 60 ft
- Beam length: L_{beam} = 61 ft
- Weight of barrier: 0.3 kips/ft/side

Some of the characteristics of the designs and weights are summarized in Table 42.

Table 42. Superstructure weight for 60-foot span with conventional concrete sections and UHPC box beams.

Option	Material	Cross Section	h_c (inch)	w_g (k/ft)	w_s (k/ft)	W_{beam} (kips)	W_{slab} [a] (kips)	n_{beams}	W_{bridge} [b] (kips)
A	CC	DBT-35	35.0	0.857	0.000	52.3	0.0	5	298
B	CC	FSB 18-48	24.5	0.719	0.507	43.8	30.9	7	560
C	CC	BII-48	33.0	0.721	0.000	44.0	0.0	7	345
D	UHPC	Mod. B0b-48	21.0	0.449	0.000	27.4	0.0	7	228
E	UHPC	Mod. NEXT 29D-120	29.0	1.082	0.000	66.0	0.0	3	235

[a] Slab weight is per beam.
[b] Total weight of superstructure including the barrier weight.

Some discussion points related to the decision between the conventional concrete and UHPC design alternatives are as follows.

- **Improved Hydraulic Clearance**: The UHPC section (Option D) is the shallowest of the four options. It is 3.5-inches shallower than the slab beam option (Option B) and is 8 to 14 inches shallower than the other sections. Overall, the UHPC box beam option would allow for an increase in the hydraulic clearance for the bridge without affecting the approach or current superstructure elevation.
- **Accelerated Construction**: Options A, C, D, and E do not require a CIP deck. These options would allow for a more accelerated construction window compared to Option B, which requires a 6.5-inch CIP concrete deck. The UHPC options would not require an overlay, as the top of the superstructure can be used as the riding surface.
- **Beam Weight**: The UHPC box beams are the lightest of the four options, between 37 and 48 percent lighter than the CC options. The lighter weight UHPC box beams would allow for easier transportation and erection of the beams compared to the CC beams. The UHPC box beams would also allow for a lighter substructure.

Overall, the UHPC box beam option would allow for up to a 14-inch gain in hydraulic clearance compared with the CC options, would facilitate accelerated construction, and would decrease the demand on the substructure.

CHAPTER 5. SUMMARY AND CONCLUSIONS

The release of the AASHTO *Guide Specifications for Structural Design with UHPC* allows for the optimization of UHPC structural components by leveraging its enhanced mechanical properties. One of the early entry points for owners and engineers to begin designing and implementing UHPC structural components is in short-span bridges where numerous advantages exist for this application. The potential solutions discussed in this report can be used to help owners and engineers with the implementation of UHPC structural components for short-span bridges with span ranges between 20 and 125 feet.

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