DEVELOPMENT OF PRELIMINARY DESIGN CHARTS FOR PRESTRESSED UHPC BRIDGE GIRDERS

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ABSTRACT

The original PCI Bridge Design Manual provides preliminary design charts that were developed based on the AASHTO Standard Specifications. These charts provide initial girder parameters including the girder size and number of prestressing strands required for a given span length and beam spacing for 28-day concrete compressive strengths of f'_c = 7,000 and 12,000 psi (48 and 83 MPa). Recently, a few states including Iowa and Virginia have built bridges using ultra-high performance concrete (UHPC) with an f'_c exceeding 15,000 psi (103 MPa) and other states such as New Mexico are also interested in this material. This paper presents a general procedure to develop preliminary design charts for prestressed concrete bulb-tee girders considering service load stress limits, flexural strength, and stress limits at transfer in accordance with UHPC tension and compression (release and 28-day) properties. The procedure is illustrated for a prestressed concrete BT-72 beam to determine the number of strands required versus span length and beam spacing. The results are first compared with the PCI design charts for purposes of verification. Using the verified procedure, preliminary design charts are then developed for UHPC girders to show the potential impact on prestressed bridge design.

Keywords: Preliminary design; Bulb-tee girder; Ultra-high performance concrete (UHPC); Flexure design.

INTRODUCTION

Ultra high performance concrete (UHPC), a new class of cementitious composite material, began with the development of reactive powder concrete (RPC) in the early 1980's. This development can be credited to researchers Bache[1] (1981) and Richard and Cheyrezy[2] (1995). Richard and Cheyrezy[2] (1995) implemented several new principles to produce UHPC. This type of concrete is different from normal and high performance concrete (HPC) in several ways. Mainly, coarse sand is replaced by fine sand and high strength steel fibers. Consequently, UHPC has been shown to develop a compressive strength greater than 21.7 ksi (150 MPa) and the steel fibers result in a sustained post-cracking tensile strength greater than 0.72 ksi (5 MPa)[3].

Since both the compressive and tensile strengths are greater for UHPC, the design of structural elements may be better optimized. Additionally, because its main aggregate is fine sand, the porosity of the concrete decreases and thus, penetration of liquids reduces significantly improving durability[4]. Hence, the mechanical and durability properties of UHPC make it a promising material for the construction of new prestressed concrete bridges as well as an option for repair and replacement of older bridges to address highway infrastructure deterioration[3]. Prestressed concrete bridges represent a significant amount (23%) of the structures in the National Bridge Inventory[5] and actually over 50% of the bridges built each year. In the United States, UHPC has been employed in three prestressed concrete girder bridges. The first two UHPC bridges were built in Iowa and Virginia using I-shaped girder members. The third UHPC bridge was constructed in Iowa with a prestressed deck-bulb-double-tee girder shape.

This paper presents a general procedure to develop preliminary design charts for prestressed concrete bulb-tee girders considering service load stresses, flexural strength, and stress limits at transfer based on UHPC tension and compression (release and 28-day) properties. A prestressed concrete girder model was developed using the MatLab software[6] for preliminary design purposes. A prestressed concrete BT-72 beam was considered to illustrate the procedure developed to compute the maximum span length for a given number of prestressing strands and girder spacing. Results were first compared with the PCI design charts for verification. Once the model was confirmed, preliminary design charts were developed for a UHPC BT-72 girder based on 28–day compressive strengths ranging from f'_c= 15,000 psi (103 MPa) to 20,000 psi (138 MPa). Both 0.6-in (15 mm) and 0.7-in (18 mm) prestressing strands were considered in this study. The potential impact on prestressed concrete bridges design (e.g., span lengths, number of prestressing strands, and girder spacings) resulting from the use of UHPC are examined.

PCI PRELIMINARY DESIGN CHARTS FOR NSC AND HPC

The design of an economical prestressed concrete girder bridge generally starts with a preliminary design. For a given span length and based on standard concrete strengths, the preliminary design includes selection of the girder size and shape; girder spacing; diameter and number of prestressing strands; and deck thickness. This section explains the preliminary design charts provided in the 2nd Edition of the PCI Bridge Design Manual[7] (2003) which were developed to satisfy the strength and serviceability limit states of the AASHTO Standard Specifications[8] (2002). These charts are hereafter referred to as the PCI-03 preliminary design charts and also provide a reasonable starting point for girders designed based on the AASHTO LRFD Bridge Design Specifications[9]. Recently, the 3rd Edition of the PCI Bridge Design Manual[10] (2011) was released but was not available at the time of this research. The new charts were developed based on LRFD design guidelines and are referred to as PCI-11. For reasons discussed later, it was decided to follow the AASHTO Standard Specifications[8].

CHART DESCRIPTION

The PCI-03 preliminary design charts were developed for different girder shapes including AASHTO box beams, AASHTO-PCI standard bulb-tees, AASHTO standard I-beams, to name a few. For each shape, different girder sizes were considered such as the PCI BT-54, BT-63, and BT-72 for bulb-tees. The first chart type for each shape provides the maximum attainable span length versus girder spacing for the different girder sizes. These charts are used in the early stage of design to select the girder size based on a span length and girder spacing. The second chart type for each shape provides the number of strands required for a specified span length and beam spacing; Figure 1 shows the PCI-03 chart for the BT-72 girder section.

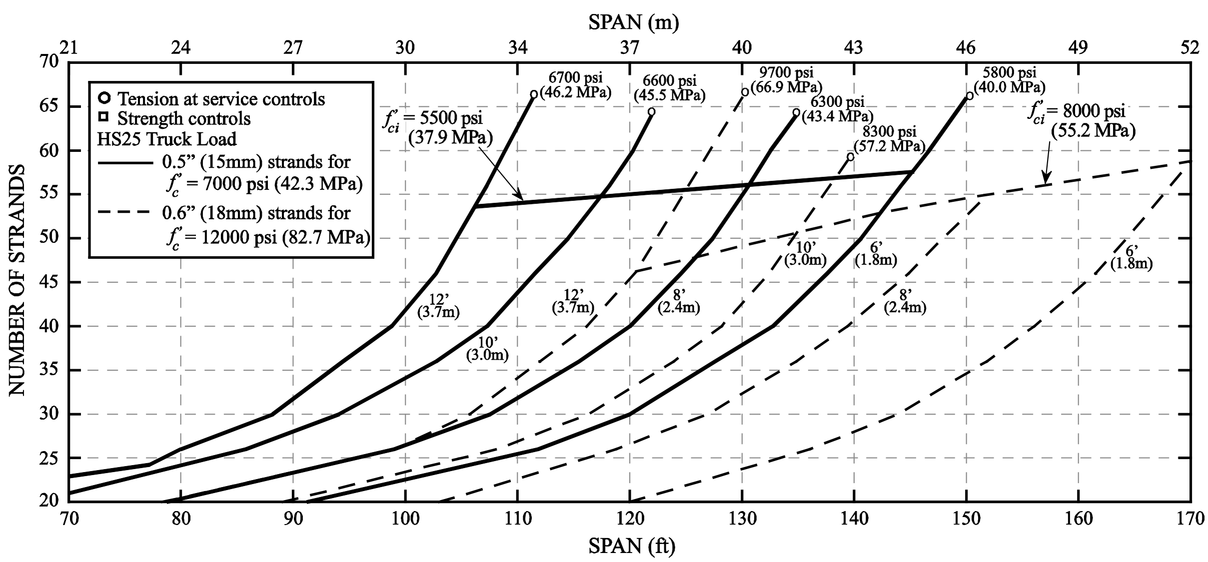

Fig. 1: Reproduction of PCI-03 chart for BT-72 girder section

This information can be used to obtain a preliminary cost estimate based on girder requirements and also to determine if local producers are able to fabricate the girder as designed (e.g considering hold-down force limits), for example. The girder size, spacing, and/or strand layout may have to be changed to satisfy the budget constraints and/or fabrication restrictions. This study focuses on AASHTO standard bulb-tee sections as shown in Figure 1 for a PCI BT-72. The maximum span length in the charts is controlled by the strength or serviceability (tension at service) limit states or allowable stresses at release. An upper bound limit labeled with the specified concrete stress at release, f'_{ci}, indicates cases where compressive stresses at release are the controlling criteria for the maximum span length. The curves are continued past this line and either tension at service or strength controls (usually tension at service), and the end point of each curve is labeled with the minimum required value of f'_{ci} not exceeding f'_{c}. In the next section, a review of the assumptions used in the development of the PCI-03 preliminary design charts is provided.

DESIGN CRITERIA

Dead and Live Loads

The PCI-03 preliminary design charts were developed based on the live-load effects for an AASHTO HS25 truck which is 25% heavier than the standard HS20 design truck. During the formulation of the PCI-03 charts, several states reported similar designs from the AASHTO LRFD Bridge Design Specifications[9] under HL-93 loading and the AASHTO Standard Specifications[8] under HS25 truck loading.

The live-load distribution factor for moment was taken as (S/5.5) under wheel loading, where S is the girder spacing in feet. The AASHTO Standard Specifications[8] employ this factor for I-beam systems under multiple lane loading which ignores the effects of span length, slab thickness, and composite girder stiffness in computing the distribution factor. However, these parameters are considered in the LRFD Bridge Design Specifications[9]. For example, in LRFD Article A4.6.2.2.2b the moment distribution factor (mg) under axle loading for an interior girder under HL-93 and multiple design lanes is computed as:

$$mg = 0.075 + \left[\frac{S}{9.5}\right]^{0.6} \left[\frac{S}{L}\right]^{0.2} \left[\frac{K_g}{12Lt_s^3}\right]^{0.1} \qquad (Eq. 1)$$

where L = span length (ft); S = girder spacing (ft); and t_s = slab thickness (in). The longitudinal stiffness, K_g, is computed as:

$$K_g = n\left[I + Ae_g{}^2\right] \qquad (Eq. 2)$$

where n = modular ratio between beam and deck material; I_g = moment of inertia of beam (in^4); A_g= cross-section area of beam (in^2); and e_g = distance between the centers of gravity of the beam and deck (in).

The live load impact factor, I, used in developing the PCI-03 charts was computed as :

$$I = \frac{50}{L + 125} \leq \ 30\% \qquad (Eq. 3)$$

The girder, slab, and haunch weight were considered as non-composite dead loads. For composite dead load, a value of 40 psf (1915 N/m^2) superimposed dead load was assumed which accounts for the barriers and railing weight as well as 2 in. (51 mm) of concrete overlay for a future wearing surface.

Deck Properties

An 8-in (203 mm) thick, concrete deck plus a 0.5-in (13 mm) haunch were assumed to develop the PCI-03 charts. It is important to note that using the same thickness for different girder spacings, the 8-in (203 mm) deck thickness is not the most feasible at larger spacings because it would require a significant amount of steel reinforcement and therefore, the cost of the reinforced concrete deck would be excessive. In actuality, the deck thickness should increase with the beam spacing to achieve a more cost-effective design. For instance, the New Mexico Department of Transportation (NMDOT) uses a standard slab thickness that increases from 7.5 to 11 in. (191 to 279 mm) for beam spacings from 6 to 11 ft (1.83 to 3.35 m) as specified in the NMDOT Bridge Procedures and Design Guide[11]. The cast-in-place deck was considered to have a 28–day compressive strength of f'_c = 4000 psi (28 MPa).

Girder Concrete and Allowable Stresses

For the precast girders, the concrete compressive strengths were taken as f'_{ci} = 5500 psi (38 MPa) at release and f'_c = 7000 psi (48 MPa) at service in the PCI-03 charts. The allowable concrete tensile stresses were taken as $7.5\sqrt{f'_{ci}}$ at release and $6\sqrt{f'_c}$ at service while the allowable concrete compression stresses were taken as $0.6f'_{ci}$ at release and $0.6f'_c$ at service, in accordance with the AASHTO Standard Specifications[8]. Presently, high strength concrete girders ranging from 10,000 to 15,000 psi (69 to 103 MPa) are being produced. Accordingly, the PCI-03 preliminary design charts for I-beams and bulb-tee girders also considered the case of concrete having f'_{ci} = 8000 psi (55 MPa) and f'_c = 12,000 psi (83 MPa). In the state of New Mexico, a 28-day compressive strength up to 9,500 psi (66 MPa) is currently the standard, but prestressing plants are allowed to provide up to 12,000 psi (83 MPa) using a 56-day curing period[11]. The NMDOT is currently sponsoring research towards the development of higher strength concretes up to 22,500 psi (155 MPa) using local materials. The effect of UHPC on prestressed concrete girder design will be addressed later in this paper. For high strength concrete (i.e., f'_c > 10,000 psi or 69 MPa), the allowable tensile stresses at release and service were considered 33 percent higher than those allowed for normal concrete. That is, the allowable tensile stresses were set at $10\sqrt{f'_{ci}}$ at release and $8\sqrt{f'_c}$ at service, while the allowable compressive stresses remained the same as normal strength concrete (i.e., $0.6f'_{ci}$ and $0.6f'_c$).

Prestressing Strands and Spacing

For normal strength concrete, 0.5-in (13 mm) diameter, seven-wire, 270 ksi (1.86 GPa) low relaxation strands were used in all cases. The center-to-center strand spacing was 2 in. (51 mm) and all strands were assumed to have an initial tension of 202.5 ksi (1.40 Gpa) before release. End stresses can be controlled either by strand debonding (shielding) and/or harping but no information was provided as to which method was used in the PCI-03 charts. Relative humidity was assumed as 75%. The PCI-03 charts for high strength concrete were developed using 0.6-in (15 mm) diameter strands at 2 in. (51 mm) spacing. Note that a 0.6-in (15 mm) diameter strand provides 40% more tensile capacity than a 0.5-in (13 mm) diameter strand.

DESCRIPTION OF PRESTRESSED GIRDER MODEL

This section explains the general procedure followed to develop the preliminary design charts for bulb–tee girders considering service, strength, and release limit states. The procedure is illustrated for a prestressed concrete BT-72 girder to estimate the number of prestressing strands required versus span length and beam spacing. The prestressed girder model was developed using the MatLab software[6]. To determine the structural and economic impact of using UHPC in the superstructure of prestressed concrete bridges it was considered important to develop preliminary design charts not only for the BT-72 girder section but also for other girder sections, concrete strengths (for girders and deck), allowable stresses, and strand diameters.

DESIGN CRITERIA

The cross-section properties for a bulb-tee BT-72 shown in Figure 2 and Table 1 were used.

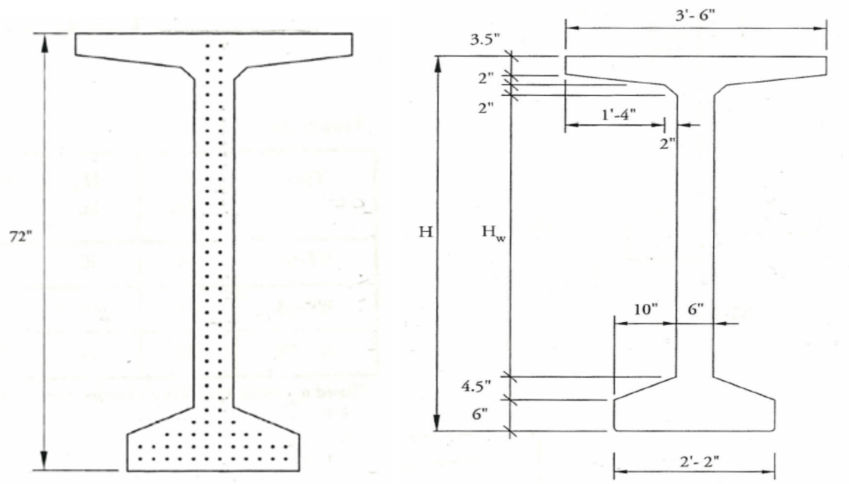

Fig. 2: Strand pattern and geometry of AASHTO-PCI Bulb-Tee BT-72

Table 1: Mechanical properties of AASHTO-PCI Bulb-Tee BT-72

Type	H in. (cm)	H_w in. (cm)	Area in^2. (cm^2)	Inertia in^4. (cm^4)	y_{bottom} in. (cm)	Weight kip/ft (N/cm)	Maximum Span,*ft (m)
BT-72	72 (2,195)	54 (1,646)	767 (712,566)	545,894 ($471.2x10^9$)	37 (1,128)	0.799 (117)	146 (45)

Note that the maximum span listed, 146 ft (45 m), corresponds to a 28-day compressive strength of 9500 psi (66 MPa). For the service limit state, the AASHTO Standard Specifications[8] were followed to compute the flexural stresses due to dead load and live load, and the axial/flexural stresses due to prestressing forces at midspan. A summary of the allowable stresses used in the prestressed girder model is shown in Table 2.

Table 2: Allowable stresses used in the prestressed girder model

Type of Concrete	At Release, psi		At Service, psi	
	Compression	Tension	Compression	Tension
Normal Strength Concrete (NSC)	$0.6\,f'_{ci}$	$7.5\sqrt{f'_{ci}}$	$0.6f'_c$	$6\sqrt{f'_c}$
High performance concrete (HPC)	$0.6\,f'_{ci}$	$10\sqrt{f'_{ci}}$	$0.6f'_c$	$8\sqrt{f'_c}$
Ultra high performance Concrete (UHPC)	$0.6\,f'_{ci}$	$10\sqrt{f'_{ci}}$	$0.6f'_c$	$8\sqrt{f'_c}$

A 28-day compressive strength f'_c= 7,000 psi (48 MPa) and f'_c = 12,000 psi (83 MPa) were used for NSC and HPC, respectively. Low relaxation strands at 2 in. (51 mm) spacing with 0.5-in (13 mm) diameter strands were used for NSC and 0.6-in (15 mm) diameter strands for HPC. An ultimate stress, f'_s = 270 ksi (1.86 GPa), and initial pretensioning, f_{si} = $0.75f'_s$, were considered. Setting the imposed stresses equal to the allowable stresses resulted in a polynomial equation as a function of L, the length of the girder. For a given number of strands and beam spacings, the approach taken was to solve for L from the polynomial equation. The use of the distribution and impact factor equations from the AASHTO Standard Specifications[8] for the service limit state simplified the function that needed to be solved to determine the maximum span length compared to the LRFD Specifications[9].

The AASHTO Standard Specifications[8] were also followed for the strength and release limit states to maintain consistency with service. For strength, the ultimate moment, M_u, was computed using the Group I load factor design combination. A nonlinear strain compatibility approach was considered to compute the flexural design strength, $\emptyset M_n$. This will be discussed later in the strength section related to the limit states for UHPC. Concrete strengths at release were taken as f'_{ci} = 5500 psi (38 MPa) and f'_{ci} = 8000 psi (55 MPa) for NSC and HPC, respectively. Similar to service, a polynomial equation as a function of L resulted from the strength and release limit states. For a given number of strands and girder spacings, solving for L provided the curves in the preliminary design charts. For purposes of verification, the results obtained from the prestressed girder model were first compared with the PCI-03 charts for NSC and HPC.

Figures 3 and 4, respectively, show the comparison between the prestressed girder model and PCI-03 charts for a BT-72 girder section with f'_c= 7,000 psi (48 MPa) and 0.5-in (13 mm) diameter strands and f'_c= 12,000 psi (83 MPa) and 0.6-in (15 mm) diameter strands.

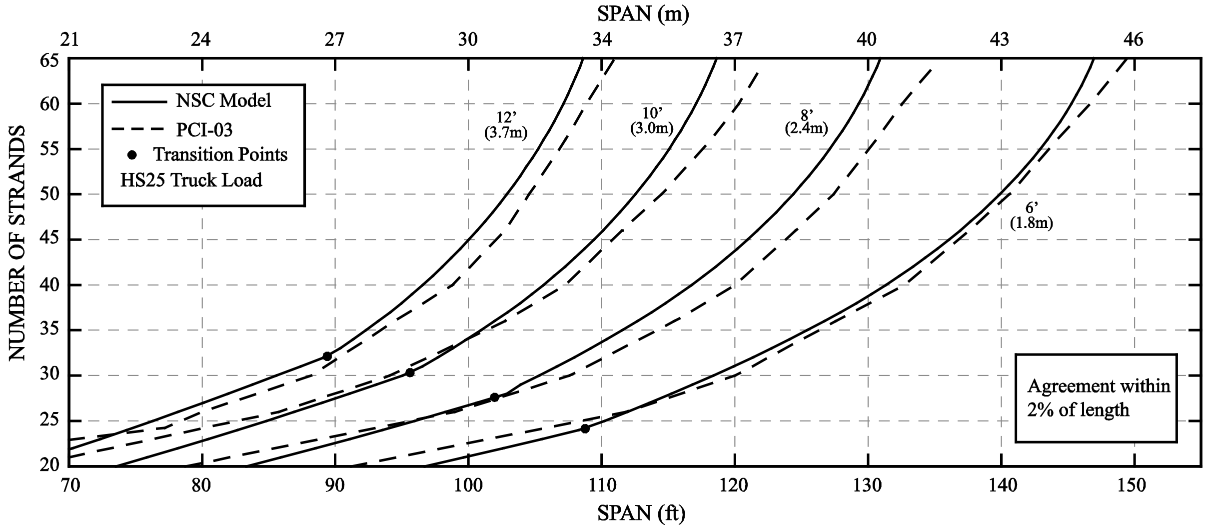

Fig. 3: Comparison between prestressed girder model and PCI-03 charts using f'_c= 7,000 psi (48 MPa).

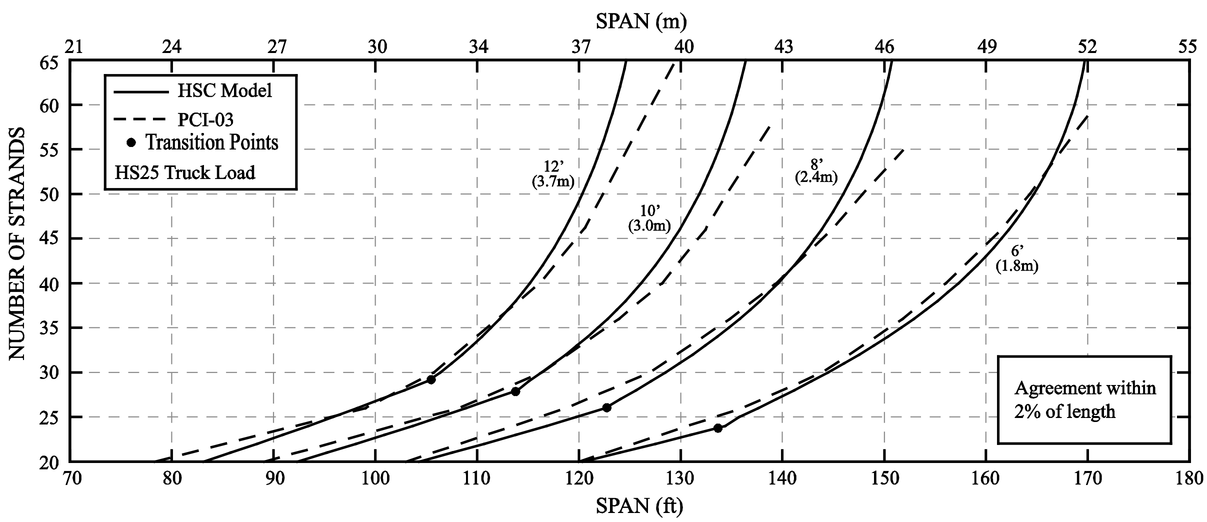

Fig. 4: Comparison between prestressed girder model and PCI-03 charts using f'_c= 12,000 psi (83 MPa).

These two figures show a good agreement between the NSC and HSC model results and the PCI-03 curves. For instance, Figure 3 for 40 strands shows that the required lengths from the NSC model were 97, 106, 117, and 131 ft (29.4, 32.2, 35.6 and 40 m), respectively, for girder spacings of 12, 10, 8, and 6 ft (3.7, 3.0, 2.4 and 1.8 m). In the PCI-03 charts for the same beam spacings, the required lengths were 99, 107, 120 and 133 ft (30, 32.7, 36.6 and 40.4 m).

The maximum percent difference in length between the NSC model and the PCI-03 charts for 40 strands was 2.8% at a girder spacing of 8 ft (2.4 m) while the minimum was 1.07% at a girder spacing of 6 ft (1.8 m). The average percent difference in length between the NSC model and the PCI-03 charts considering 40 strands and the four girder spacings was 1.95%. Based on Figure 4, the average percent difference in length between the NSC model and the PCI-03 charts was 0.86%. As the number of strands increases, the percent difference increases. The maximum percent difference in length between the HSC model and the PCI-03 charts was 3.5% corresponding to 65 strands and a girder spacing of 12 ft (3.7 m) while the minimum percent difference was 0.09% for 30 strands and a girder spacing of 10 ft (3.0 m). Overall, the average percent discrepancy between the NSC and HSC models and PCI-03 curves was less than 2%.

LIMIT STATES FOR UHPC

Using the verified procedure, for the prestressed girder model, preliminary design charts were obtained for UHPC BT-72 girders using f'_c ranging from 15,000 psi (103 MPa) to 20,000 psi (138 MPa) and both 0.6-in (15 mm) and 0.7-in (18 mm) diameter, strands were considered. For the strength limit state, the same procedure as described before was used to compute M_u and $\emptyset M_n$. The strength at release, f'_{ci}, was assumed 70% of the 28-day compressive strength for UHPC. Given a number of strands and beam spacings, the length of the girder L was determined by solving the polynomial equations for the different limit states as described before. Live load and dead load deflections were not considered. Further discussion of this process is provided in the following sections.

Service

To compute the maximum span length governed by the serviceability (concrete tension) limit state, the flow chart shown in Figure 5 was followed. In step 1, the girder spacing and BT-72 section properties (as given in Figure 2 and Table 1) were set and an 8-in (203 mm) thick concrete deck plus a 0.5-in (13 mm) haunch was specified. The 28-day concrete strengths, strand size, initial pretensioning, and tension stress limit at service loads, F_b, were taken as discussed earlier. The number of prestressing strands, N, was varied from 2 to 70 which is the maximum number of strands for a bulb-tee BT-72. The next three steps consisted of the following: composite section properties (step 2); bending moments due to dead and live loads (step 3); and flexural stresses at bottom fiber, f_{b1}, and required precompresssive stress, $f_{b1} - F_b$ (step 4). In step 5, the bottom fiber stresses due to prestress after all losses, f_{b2}, were calculated according to the procedures given in Chapter 9 of the 2nd Edition of the PCI Bridge Design Manual[7] .

Bending moment due to live load at midspan was computed as:

$$M_{LL} = \frac{9}{8}\left(L + \frac{21}{2}\right) - 14P \qquad (Eq. 5)$$

where P = 40 kips (due to HS25 truck loading).

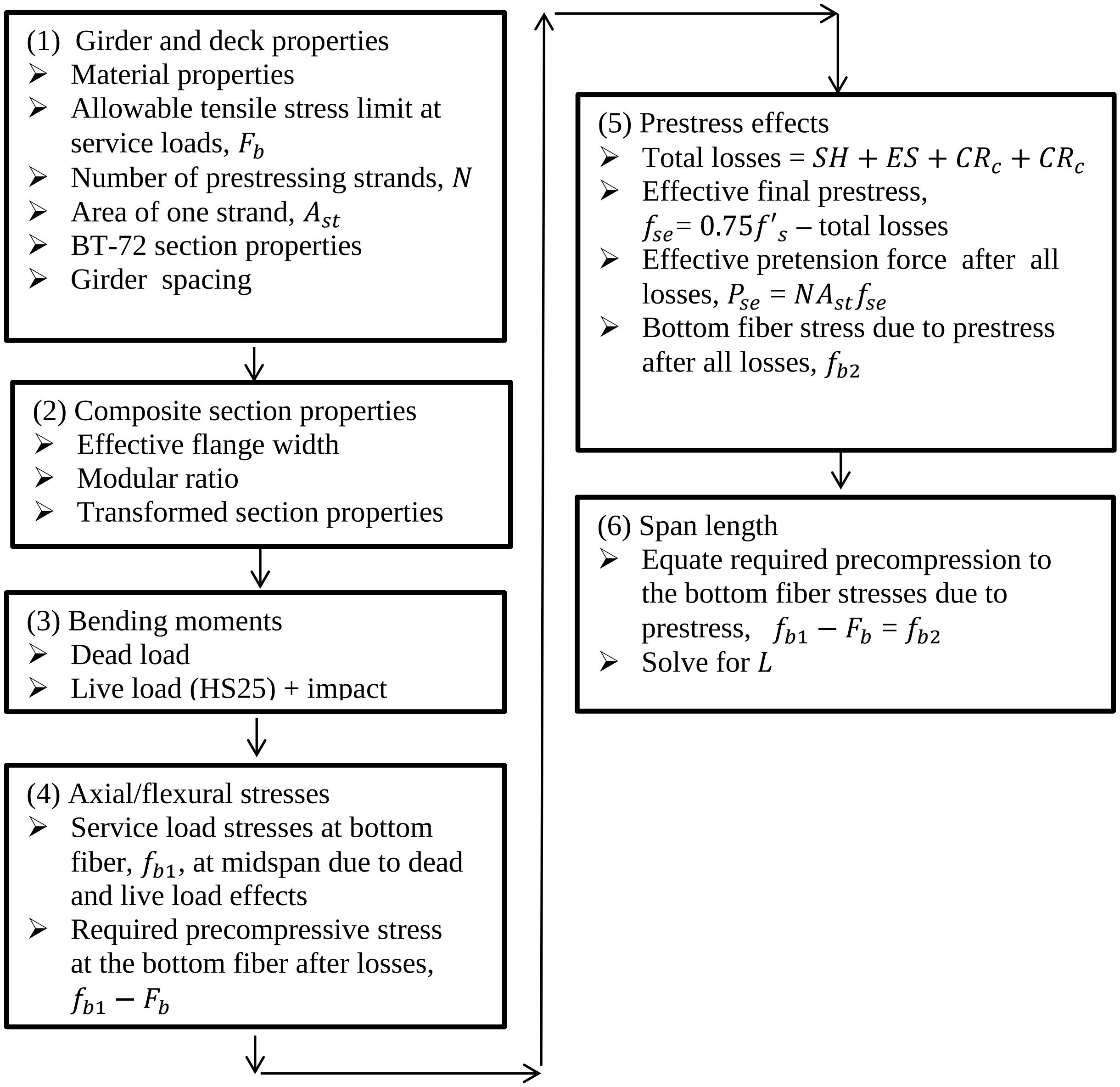

Fig. 5: Flow chart to compute maximum span length governed by service limit states

Considering the live load distribution factor, S/5.5, the bending moment due to live load plus impact at midspan was computed as:

$$M_{LL+I} = \left[\frac{9PL^2 + 1435PL - 24500P}{32L + 4000}\right]\left(\frac{S}{5.5}\right) \qquad (Eq.6)$$

The service load stresses under dead and live loading, f_{b1}, and the stresses due to the prestress force after all losses, f_{b2}, were computed as a function of L. Finally, in step 6 the required precompression was set equal to the bottom fiber stresses due to prestress, resulting in the following fourth degree polynomial which was then solved for the girder length L:

$$f(L) = aL^4 + bL^3 + cL^2 + dL + e \qquad (Eq. 7)$$

where $a, b, c, d, and\ e$ = constants. Equation 7 resulted in four roots, and the final girder length was equal to the maximum positive and non-imaginary root of the polynomial equation.

Strength

The flow chart shown in Figure 6 was used to compute the maximum span length governed by the strength limit state.

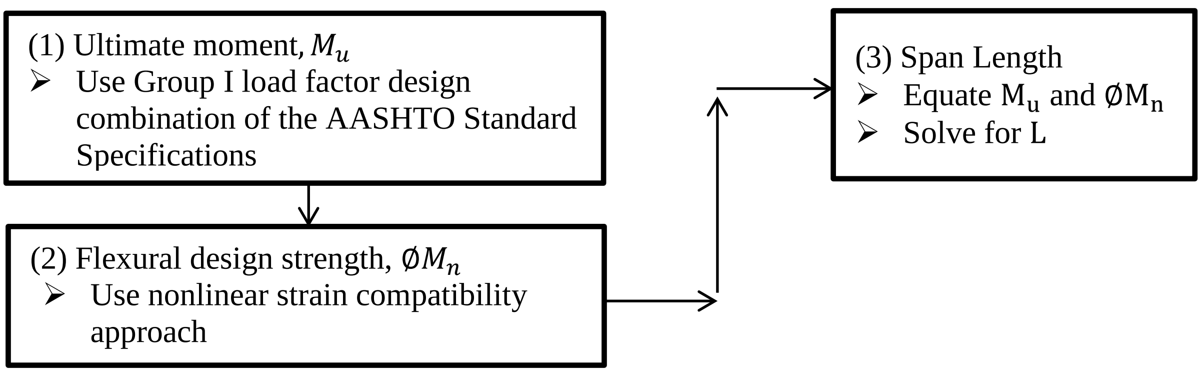

Fig. 6: Flow chart to compute maximum span length governed by the strength limit state

To compute the ultimate flexural moment, M_u, the Group I load factor design combination of the AASHTO Standard Specifications[8] was applied (step 1) using the dead and live load moments previously determined for service. The flexural design strength, $\emptyset M_n$, was determined using a non-linear strain compatibility approach[12]. With this approach, the material properties and geometry for any type of cross-section can be incorporated since the composite girder is divided into differential slices to compute strain and stresses over the height. Furthermore, design formulas related to flexural strength assume a parabolic stress-strain relationship for the concrete in compression and ignore the tensile strength of the concrete. For UHPC, the compressive behavior has been shown to be linear and the tensile strength to be significant[13]. Therefore, the capability to directly incorporate the concrete stress-strain relationship into the prestress girder model and recognizing that the tensile concrete strength may be substantial, a non-linear strain compatibility approach[12] was adopted to continue future work.

For a given number of strands $\emptyset M_n$ was computed at midspan. First, the neutral axis depth c was assumed. Second, the composite girder section was divided into slices, and the strains and the corresponding stresses were calculated at the center of each slice based on the distance from the neutral axis. A maximum concrete compressive strain of 0.003 was assumed and the non-linear stress-strain relationships for the deck concrete, girder concrete, and prestressing steel were used[14,15]. Third, the average stress within each slice was then multiplied by the area of the slice to determine the associated compressive force. Next, the tension forces in the steel were computed based on the steel strain and the effective stress in prestressing steel after losses, f_{se} (ksi), which was estimated using the following equation[12]:

$$f_{se} = 158 - 0.2[N - 20] \qquad (Eq. 8)$$

rather than $f_{se} = 0.75f'_s - total\ losses$. The use of $(Eq. 8)$ simplified the function that needed to be solved to determine the girder length L.

Based on f_{se}, the stresses in the steel at ultimate were then computed using the power formula[15]. Final forces in the steel were obtained by multiplying the ultimate stresses times the area. Equilibrium was then checked and if the forces were not in equilibrium, another value for the neutral axis depth c was chosen and the process repeated. Finally, the flexural capacity of the girder was computed by summing moments due to the concrete forces with respect to the centroid of the prestresssing steel. The strength reduction factor, $\emptyset,$ was computing using the following equation[12]:

$$\emptyset = 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) \qquad 0.75 \leq \emptyset \leq 1.0 \qquad Eq. 9)$$

where d_t = distance from extreme compression fiber to extreme tension steel (in). and c = distance from extreme compression fiber to neutral axis (in). This equation represents the strength reduction factor considering the transition zone between tension-controlled and compression-controlled members.

Setting M_u = $\emptyset M_n$, resulted in a third-degree polynomial as shown below

$$f(L) = fL^3 + gL^2 + hL + i \qquad (Eq. 10)$$

where $f, g, h, and\ i$ = constants. Three roots were computed from this equation, and the final girder length for the strength limit state was taken as the maximum, positive and non-maginary root of the polynomial equation. The governing girder lengths resulting from Equations 7 and 10 were plotted versus number of strands for a given girder spacing.

Release

The flow chart shown in Figure 7 was used to compute the maximum span length governed by stresses at release. Three different longitudinal locations were considered to compute the maximum span length based on stresses at release as the controlling criterion. The first section was located at a distance equal to the transfer length (50 times the strand diameter) from the end of the beam. The second section considered was at the harp point location which is 40% of the beam length from the end of the beam. The third section was located at midspan.

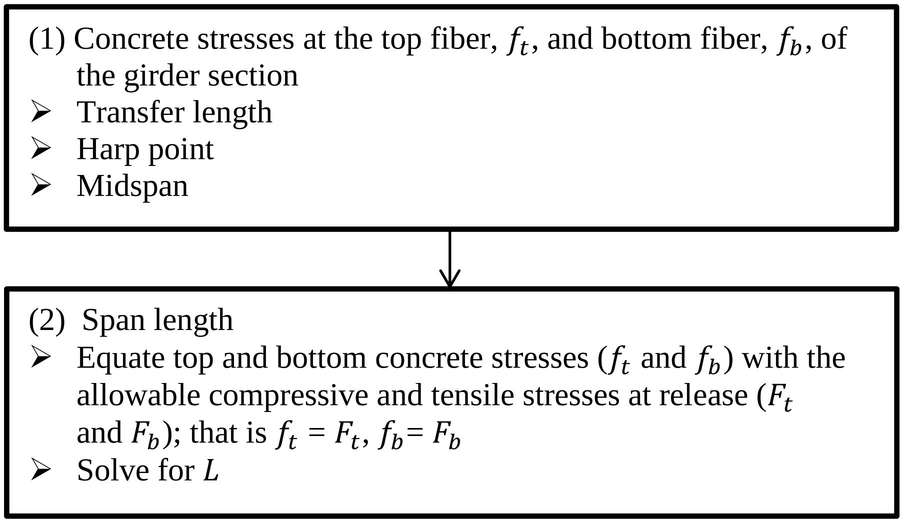

Fig. 7: Flow chart to compute maximum span length governed by stresses at release.

For a given number of strands, the concrete stresses at the top and bottom fibers fiber (f_t and f_b, respectively) of the girder section were computed at the three different locations described above (step 1). Equating the concrete stresses to the allowable stresses at release (step 2) resulted in two second-degree polynomials at each location (i.e., a total of six equations). Solving the polynomial equations, the required girder lengths for each location were obtained. At the transfer length location, different girder lengths for the same number of strands but with different number of harped strands were obtained. From this analysis, the optimum number of harped strands for a given girder length and total number of strands was determined. For instance, it was found that for a girder length equal to 48 ft (14.6 m) with 38 0.5-in (13 mm) strands and a compressive strength f'_c= 7,000 psi (48 MPa), the optimum number of harped strands was four. The midspan location at release was more critical than the harp point location. Therefore, the midspan stresses were used to compute the required length for release.

From the prestressed girder model, it was observed that concrete stresses at release did not affect the results given in the preliminary design charts which were governed by the service and strength limit states; hence, the release limit state was considered mainly to determine the number of harped strands. Consequently, only the results for the service and strength limit states for UHPC are presented in the next section.

UHPC RESULTS AND DISCUSSION

Figures 8, 9 and 10 show the preliminary design charts for UHPC BT-72 girders based on f'_c equal to 12, 15, 17.5, and 20 ksi (83, 103, 121, and 134 MPa) and strand diameters of 0.6 and 0.7 in. (15 and 18 mm). These charts were developed to satisfy the service and strength limit states from the AASHTO Standard Specifications[8].

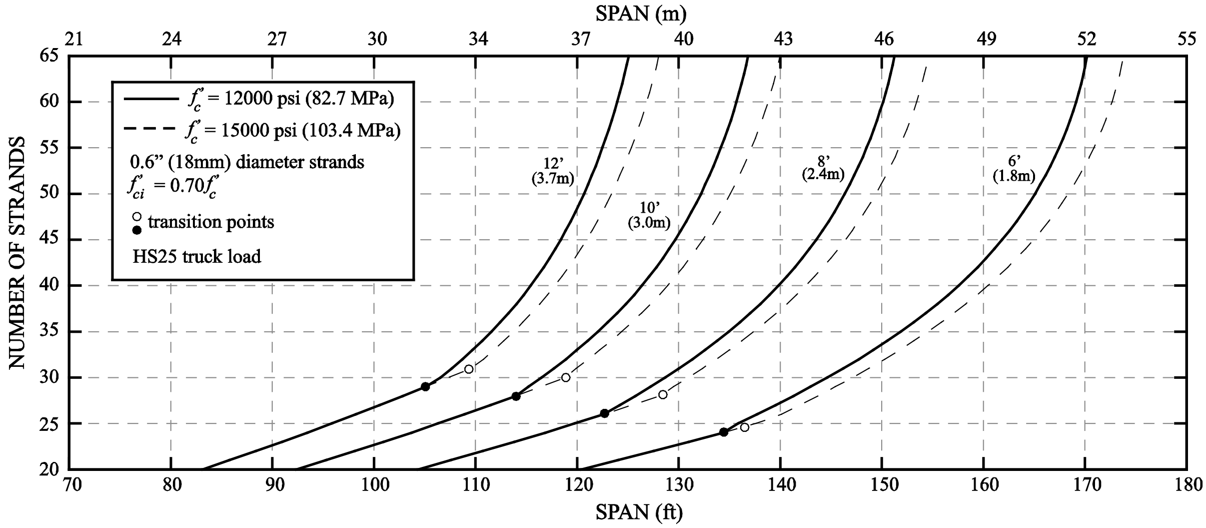

Fig. 8: Preliminary design chart for UHPC BT-72 girders using f'_c= 12,000 psi (82.7 MPa) and f'_c= 15,000 psi (103.4 MPa) with 0.6-in (15 mm) diameter strands.

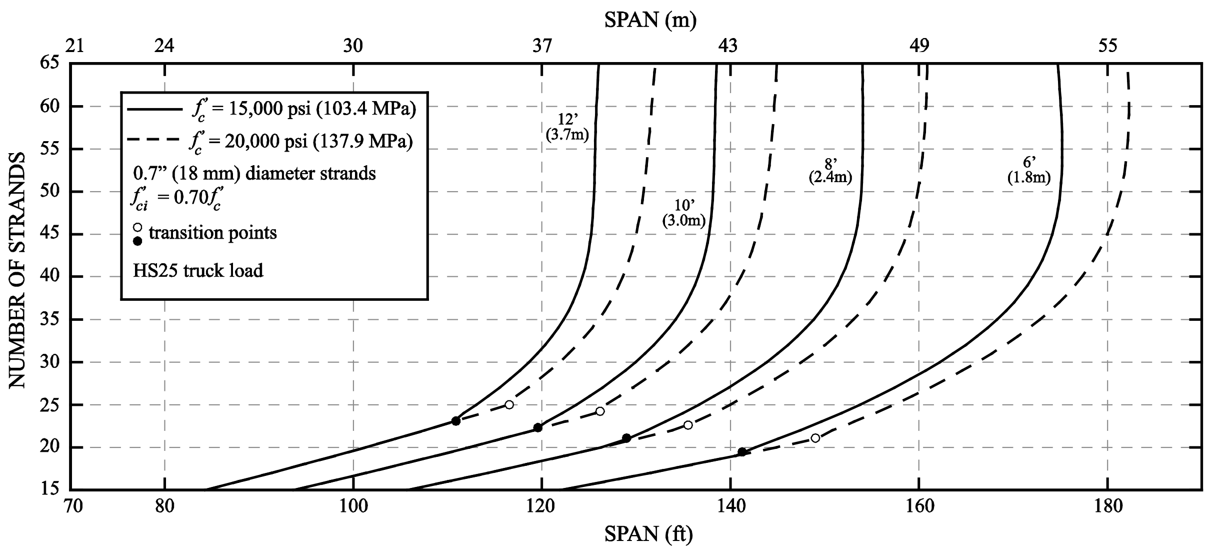

Fig. 9: Preliminary design chart for UHPC BT-72 girders using f'_c= 15,000 psi (103.4 MPa) and f'_c= 20,000 psi (137.9 MPa) with 0.7-in (18 mm) diameter strands.

The transition point where the span length changes from being controlled by the strength to service limit state is evident in the figures by the change in shape from a “smooth” straight line to a “pronounced” curve line for the four girder spacings. These points are marked by a dark and white circle for 12,000 and 15,000 psi (83 and 103 MPa), respectively, in Figure 8. Similarly, the points are marked for 15,000 and 20,000 psi (103 and 138 Mpa) in Figure 9.

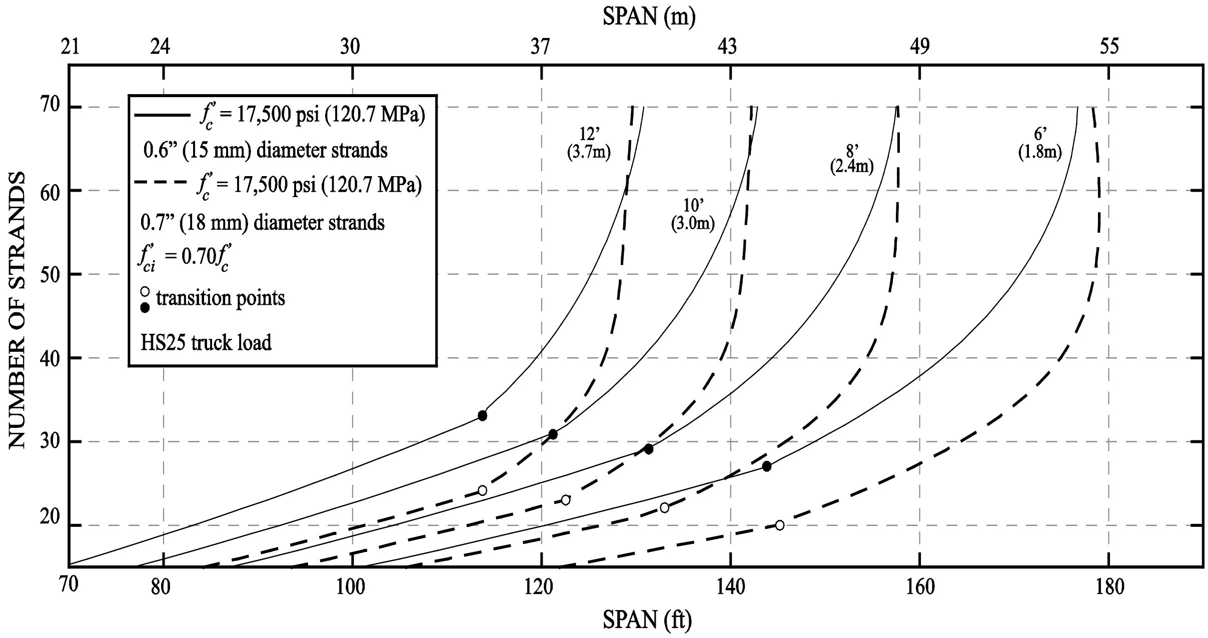

Fig. 10: Preliminary design chart for UHPC BT-72 girders using f'_c= 17,500 psi (121 MPa) with 0.6-in (15 mm) and 0.7-in (18 mm) diameter strands.

In Figure 8, compressive strengths of 12,000 psi (83 MPa) and 15,000 psi (103 MPa) with a 0.6-in (15 mm) diameter strand were considered. Note that the change in concrete strength had no observable effect on the required span length for the strength limit state. This is attributed to the fact that the tensile strength of the concrete for UHPC was ignored. As shown in Figure 8, as the girder spacing increases, the number of strands at the transition points increases signifying the strength limit state becomes more the controlling criterion. The transition point ranges between 24 and 30 strands, meaning that span lengths are governed by the service limit state above this range of strands. Also note that the number of strands at the transition points increases as the concrete compressive strength increases. For instance, at 8 ft (2.4 m) girder spacing, the number of strands at the transition point changes from 26 with f'_c= 12,000 psi (83 MPa) to 28 strands with f'_c= 15,000 psi (103 MPa). Transition points provide important information since fully prestressed and partially prestressed members are directly related to the coordinates of the preliminary design charts.

Fully prestressed girders are those that do not experience stresses above the modulus of rupture and are governed by Figures 8-10. Partially prestressed girders are those that are allowed to experience stresses larger than the modulus of rupture. In other words these girders have less prestressing strands than required to remain below the allowable tension stresses under service loads given in Table 2 and therefore may experience some cracking. Partially prestressed members will require a fewer number of strands above the transition point shown in Figures 8-10 and may be considered as an economical option for the final design of the girder.

Keeping the number of prestressing strands constant, higher span lengths can be achieved when the concrete compressive strength is increased. For example, considering 40 strands and increasing the compressive strength from 12,000 (83 MPa) to 15,000 (103 MPa) the average percentage increment in length was 1.9% from Figure 8. Furthermore, for a given span length, the number of prestressing strands decreases as the compressive strength increases. For instance, considering a span length of 120 ft (37 m) and increasing the compressive strength from 12,000 (83 MPa) to 15,000 (103 MPa) the percentage decrease in number of strands was 12%. Note that small changes occurred since the strand diameter was not increased.

In Figure 9, compressive strengths of 15,000 psi (103 MPa) and 20,000 psi (138 MPa) with a 0.7-in (18 mm) diameter strand were investigated. For a compressive strength of 15,000 psi (103 MPa), the transition point ranges from 18 to 24 prestressing strands while for 20,000 psi (138 Mpa) it was 20 to 26 strands for the four girder spacings. Based on this data, the strength limit state governs only for a low number of strands. As discussed earlier the use of UHPC results in an increase in the span lengths for a given number of prestressing strands; alternatively, the number of prestressing strands decreases for a given span length. This is important for new construction and bridge replacement, respectively. For instance, at 40 strands, the girder length increased by an average of 4.5 ft (1.4 m) due to the 5000 psi (34 MPa) increase in f'_c. At a span length equal to 137.5 ft (42 m) with a 10 ft (3.0 m) girder spacing, the required number of strands were 46 and 34 for compressive strengths of 15,000 psi (103 Mpa) and 20,000 psi (138 MPa), respectively. It can also be observed that as the number of strands increases, the different curves corresponding to the four girder spacings reach a maximum span length in the vertical axis and become more pronounced as the girder spacings decrease. It is not certain what is causing this behavior and further investigations will address this issue.

Figure 10 shows the impact of using different strand diameters with no change in compressive strength. A compressive strength at 28-days of f'_c= 17,500 psi (121 MPa) with 0.6-in (15 mm) and 0.7-in (18 mm) strands were selected to develop this graph. As shown in the figure, the effect of using a larger strand diameter was more significant than increasing the concrete compressive strength since the change in span length was larger for a given number of strands and girder spacing. For instance at 40 strands, the span length increased by approximately 10 ft (3 m) by increasing the strand diameter whereas increasing the compressive strength from 15,000 to 20,000 psi (103 to 138 MPa) resulted in a 4.5 ft (1.4 m) longer span length as illustrated in Figure 9. For a given girder spacing and span length, the number of strands reduces more significantly as the strand diameter increases. For example, for a 10 ft (3.0 m) girder spacing and a span length of 120 ft (37 m), the number of prestressing strands reduced from 40 to 30 when the strand diameter was changed from 0.6-in (15 mm) to 0.7-in (18 mm). Finally, it can also be noticed that the curves corresponding to the four girder spacings become more pronounced than in Figures 8 and 9. This is still being investigated.

CONCLUSIONS

Development of the preliminary design charts for prestressed concrete BT-72 girder sections, has led to the following conclusions:

- Based on the service and strength limit states of the AASHTO Standard Specifications, the prestressed girder model for NSC and HPC agreed well with the PCI-03 charts.
- The results of the preliminary design charts were not affected by concrete stresses at release. The release limit state was used mainly to determine the number of harped strands. Therefore, the preliminary design charts for UHPC were dependent only on the service and strength limit states.
- The transition points between the strength and the service limit states for the prestressed girder model, showed that the span lengths were governed mostly by the service limit state for NSC, HPC and UHPC.
- Fully prestressed and partially prestressed members can be related to the coordinates of the UHPC prestressed girder model for the transition points. Partially prestressed members (members that may experience cracking and stresses above the modulus of rupture) will require a fewer number of strands above the transition point and may be considered as a reasonable option for the final design of the girder.
- The preliminary design charts demonstrate the impact of using UHPC. Increasing the concrete compressive strength for a given number of strands and girder spacing resulted in an increase of the span length. However, the effect of using the combination of UHPC and a larger strand diameter was found to be more significant than solely increasing the concrete compressive strength since the resulting span length increment was larger.
- As the number of prestressing strands increases, the different curves corresponding to the four girder spacings reached a maximum span length and become more pronounced curves as the girder spacing decreases. This issue is still under investigation.
- Preliminary design charts for UHPC provided the framework by which to investigate the structural and economic impact on the superstructure of prestressed concrete bridges. These graphs were developed only for the BT-72 girder sections. In future work, the method described in this paper will be extended to other girder sections, concrete strengths (for girders and deck), allowable stresses, and strand diameters to further evaluate the efficiency of using UHPC in prestressed concrete bridges.

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