ANALYSIS OF UHPC BRIDGE GIRDERS

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ABSTRACT

Ultra-High Performance Concrete (UHPC) was developed in France approximately a decade ago. This material has been utilized in several bridges and other structures throughout the world and is beginning to gain more exposure in the U.S. UHPC has a compressive strength of 22 to 30 ksi and tensile strengths reported as high as 7 ksi with more typical values being in the range of 1-3 ksi. Typical analyses of concrete members takes the tensile strength of the concrete as negligible. This is common practice because the tensile strength is low in comparison to the compressive strength and the tensile strength is more difficult to quantify. However, the tensile strength of UHPC is significant and needs to be accounted for in analysis. This paper shows results from a strain compatibility procedure for analyzing girders that accounts for the tensile strength of UHPC. Results are compared with closed-form procedures in AASHTO and a strain compatibility approach that neglects the tensile strength.

Keywords: Ultra-High Performance Concrete, Bridge Girders, AASHTO LRFD, Flexural Capacity.

INTRODUCTION

Ultra-High Performance Concrete (UHPC) was developed in France approximately a decade ago. UHPC is effectively a new class of concrete with a compressive strength of 22 to 30 ksi. UHPC, also known as Ultra-high performance fiber reinforced concrete (UHPFRC), is defined by the Association Française de Génie Civil (AFGC) as a material with a cement matrix, a compressive strength that exceeds 22 ksi and containing steel fibers.[1] The steel fibers in UHPC create a ductile behavior under tension. The fibers in the mix give UHPC a direct tensile strength of 2 ksi.[2]

Different brands of UHPC are produced by several different concrete companies. These different brands vary slightly in mechanical properties and mix designs. Table 1 shows a representation of the mix design for typical UHPC. As shown in Table 1, UHPC is made mostly from Portland cement and sand. It should also be noted that no large aggregate exists.

Table 1: UHPC Mix

Component	Amount (lb/yd^3)	Amount (kg/m^3)
Portland Cement	1,188	710
Silica Fume	378	230
Quartz Powder	351	210
Fine Sand	1,701	1020
Steel Fibers	54-270	40-160
Superplasticizer	22	13
Water	243	140

UHPC in general is considered a self placing material, can be pumped from a truck, and does not require vibration. Vibration can be used, however, to ease the filling of formwork.[3] Heat treatment will give the UHPC some additional ductility, reduce future shrinkage and creep, and also increase the mechanical properties by approximately 15 percent.[3] Although a heat treatment is not required for all UHPC mix designs.

UHPC has been utilized in several bridges and other structures throughout the world and is beginning to gain more exposure in the U.S. The FHWA is currently investigating UHPC and is in the process of testing a test bridge at the Turner Fairbank Research facility utilizing a unique PI shaped section.[2] Virginia and Iowa[4] also have plans to utilize UHPC in bridges through the Innovative Bridge Research and Construction (IBRC) program.

Though UHPC is gaining exposure in the U.S., questions still remain on design utilizing UHPC. If bridge designers are to use current AASHTO LRFD philosophies to design UHPC bridges, the concepts and equations must be reviewed to determine their validity for UHPC. The first item is that the AASHTO LRFD equations do not account for the tensile strength of concrete. This is conservative and relatively accurate due to the low tensile strength of normal and even high strength concretes. However, UHPC has a significantly higher tensile strength and this should be accounted for in the overall flexural capacity of a girder. This paper examines the effect of the UHPC tensile strength on flexural capacity by investigating several standard sections in addition to two modified sections. Results from a strain compatibility approach that accounts for tensile strength as well as neglects tensile capacity are compared with closed-form AASHTO LRFD equations.

BACKGROUND

In order to evaluate UHPC members by strain compatibility procedures, stress-strain diagrams are needed. The compressive behavior, as specified in AFGC[1], can be taken as linear elastic, perfectly plastic. However, compressive tests on UHPC cylinders at Ohio University showed that the compressive behavior is generally linear elastic. To simplify the analyses and utilize test data, this study utilized a linear elastic compressive stress-strain curve.

The tensile stress-strain curve is much more difficult to determine and model. A multi linear tensile stress-strain softening diagram, depicted in Fig. 1, has been proposed in AFGC.[1] The strain softening diagram used in the analyses was developed from Fig 1 with minor modifications. The modifications were done to incorporate load deflection test results on small samples and to simplify analyses for this initial research effort.

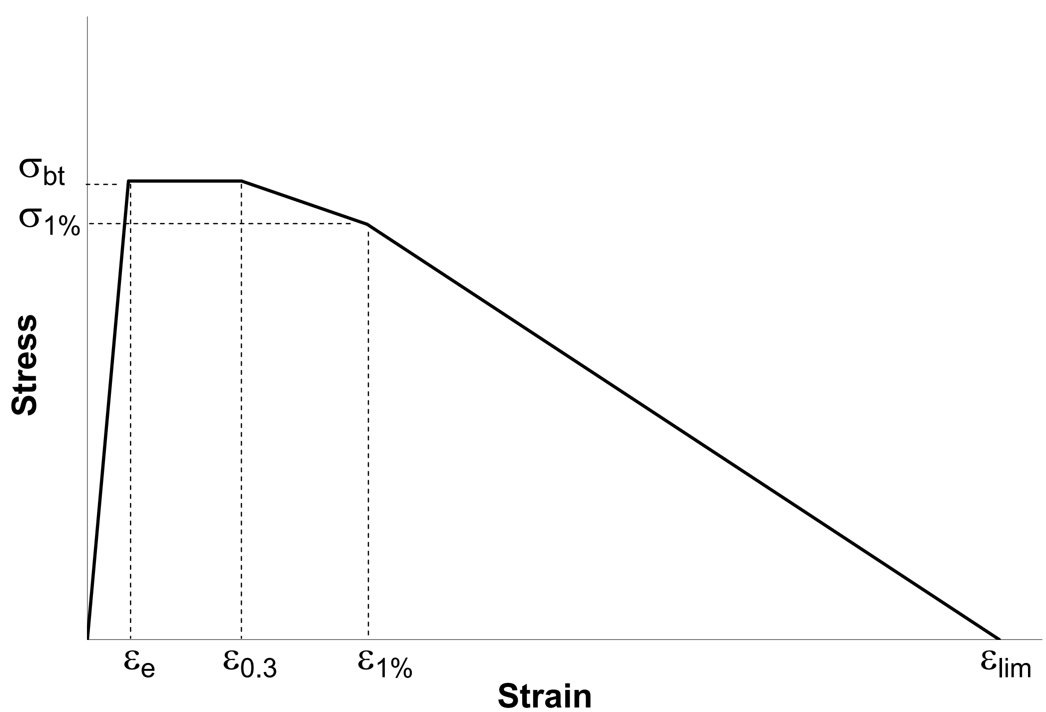

Figure 1: Tensile Stress - Strain Diagram for UHPC

where

ε_e = elastic tensile strain limit
$\varepsilon_{0.3}$ = strain when the crack width, $w_{0.3}$, is 0.3 mm.
$\varepsilon_{1\%}$ = strain when the crack width, $w_{1\%}$, is 1% of the specimen height, H
ε_{lim} = tensile strain limit
$\sigma_{1\%}$ = stress corresponding to $\varepsilon_{1\%}$
σ_{bt} = maximum stress in elastic range

The strain when the crack width is 0.3 mm, $\varepsilon_{u0.3}$, can be estimated by

$$\varepsilon_{u0.3} = \frac{w_{0.3}}{l_c} + \frac{f_{tj}}{\gamma_{bf}\, E} \tag{1}$$

where

$w_{0.3}$ = the crack width (0.3 mm)
l_c = the characteristic length (2/3 h)
f_{tj} = tensile strength at end of initial linear behavior
γ_{bf} = partial safety factor to account for manufacturing defects
E = slope of initial linear portion of stress/strain diagram.

The strain when the crack width is 1% of the specimen height, $\varepsilon_{u1\%}$, can be estimated by

$$\varepsilon_{u1\%} = \frac{w_{1\%}}{l_c} + \frac{f_{tj}}{E} \tag{2}$$

The tensile strain limit, ε_{lim}, can be estimated by

$$\varepsilon_{lim} = \frac{l_f}{4\, l_c} \tag{3}$$

where

l_f = the length of the fibers.

ANALYSES

To evaluate the effect of the tensile strength on the flexural capacity of bridge girders, analyses were performed on box beams, bulb tees, and two other cross sections specially developed for UHPC usage. Table 1 provides the details of the sections analyzed. Figures 2 and 3 provide the cross-section for the Modified Iowa 45 and the FHWA PI Test section, respectively. All sections were analyzed for flexural capacity using closed-form procedures, a strain compatibility approach that neglected the tensile strength of UHPC and another strain compatibility approach that considered the tensile strength of UHPC.

The limiting criteria for the analyses was taken as the typical compressive strain of 0.003. This compressive strain value agrees fairly well with compressive test results on cylinders. Analyses also checked that the strands did not exceed ultimate capacity.

Table 1: Sections Analyzed

Type	Section	Height	Width	Number of Strands
Box	BI-36	27	36	17 and 34
	BI-48	27	48	23 and 46
	BIV-36	42	36	17 and 34
	BIV-48	42	48	23 and 46
Bulb-Tee	BT-54	54	42	36 and 38
	BT-63	63	42	36 and 38
	BT-72	72	42	36 and 38
	Modified Iowa 45	42	32	47
Deck Bulb-Tee	DBT-35	35	72	30
	DBT-53	53	72	30
	DBT-65	65	72	30
PI	FHWA Test Bridge	33	96	22

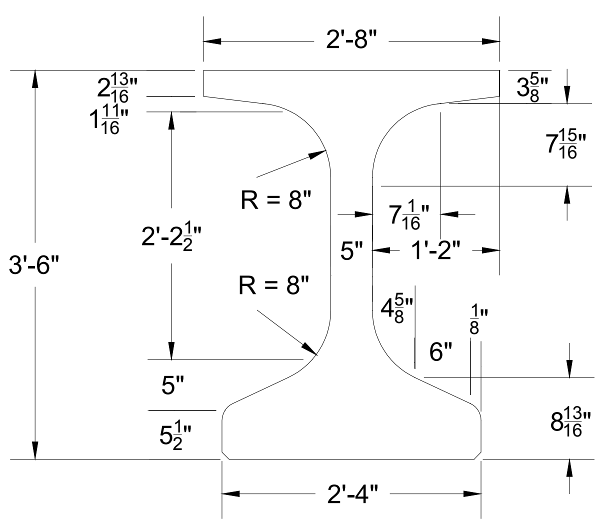

Figure 2: Modified Iowa 45

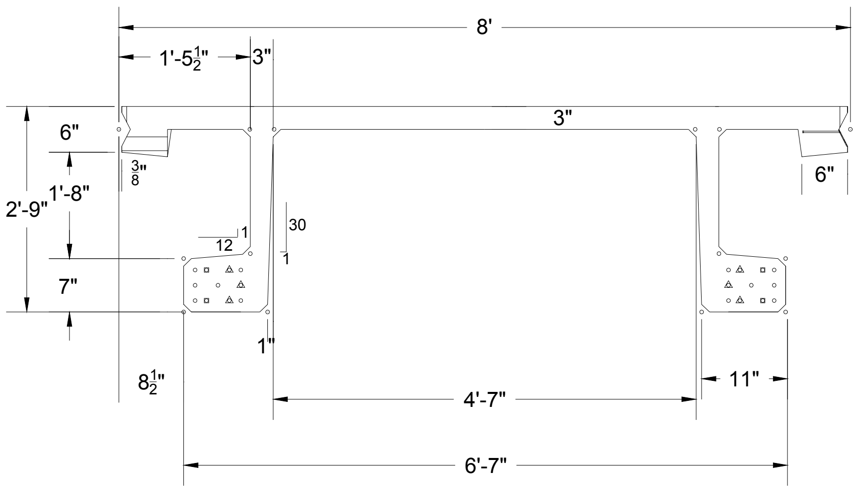

Figure 3: FHWA PI Test section

The data for the stress strain curves is shown in Table 2. The values for the strains were developed using Eqns 1-3. In addition, data from flexural tests from previous work performed at Michigan Technological University[5] and Ohio University[6] were utilized to verify strains and develop stress values. The stress values for σ_{bt} and $\sigma_{1\%}$ were taken as the same magnitudes. This was done because test results were in the form of load and deflection, and stress strain curves had to be developed and then used to predict these quantities in the nonlinear portion of the data. In addition, it was difficult to distinguish differences in the averages of these stress values.

Table 2: UHPC Stress-Strain Properties

Stress/Strain	Magnitude
ε_e	315 $\mu\varepsilon$
$\varepsilon_{0.3}$	4,074 $\mu\varepsilon$
$\varepsilon_{1\%}$	14,650 $\mu\varepsilon$
ε_{lim}	47,753 $\mu\varepsilon$
σ_{bt}	1,766 psi
$\sigma_{1\%}$	1,766 psi

RESULTS

Moment capacity results from the analyses are shown in Fig. 4 and Table 3. Fig. 4 provides the results of select sections that were chosen based on their representation of the type of section. Table 3 provides all the analyses results. As expected the moment capacity shows an increase from the closed-form approach to the strain compatibility approach when not considering the tensile strength of the UHPC, but this increase is minimal, testifying to the applicability of the closed-form approach. However, the increase in moment capacity is significant when the strain compatibility approach is used and the tensile strength of the UHPC is taken into account.

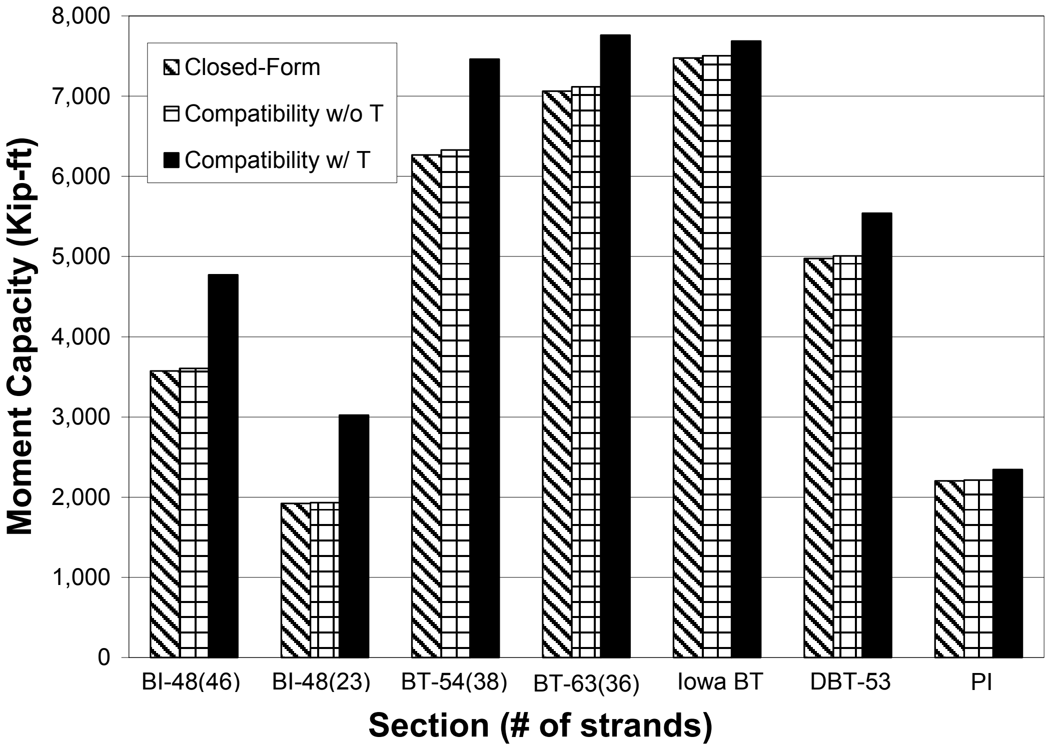

Figure 4: Moment Capacity of Select Sections

Table 3: Moment Capacity Results

Type	Section	Number of Strands	Moment Capacity (Kip-ft)		
			Closed-Form	Strain Compatibility w/o Tensile	Strain Compatibility w/ Tensile
Box	BI-36	34	2,644	2,667	3,604
		17	1,421	1,428	2,344
	BI-48	46	3,574	3,607	4,771
		23	1,921	1,931	3,021
	BIV-36	34	4,397	4,424	6,168
		17	2,298	2,306	3,613
	BIV-48	46	5,945	5,984	7,957
		23	3,109	3,119	4,348
Bulb-Tee	BT-54	36	5,967	6,001	7,031
		38	6,268	6,327	7,460
	BT-63	36	7,063	7,117	7,761

		38	7,425	7,504	8,255
	BT-72	36	8,159	8,232	8,877
		38	8,582	8,682	9,388
	Modified Iowa 45	47	7,477	7,503	7,689
Deck Bulb-Tee	DBT-35	30	3,135	3,149	3,968
	DBT-53	30	4,975	5,008	5,539
	DBT-65	30	6,202	6,247	6,756
PI	FHWA Test Bridge	22	2,203	2,213	2,344

Fig. 5 and Table 4 provide the results of the percentage increase compared to the closed-form approach. Fig. 5 is for select representative sections and Table 4 provides complete results. As can be seen, the percentage increase for the strain compatibility approach without accounting for the UHPC tensile strength is less than 1.5 percent for all sections. However, when the tensile strength is considered in the analyses, the percentage increase varies from approximately 30 to 60 percent for the boxes and from approximately 10 to 20 percent for the standard bulb-tees. The Modified Iowa Bulb-Tee and PI shaped section showed only a 3 percent and 6 percent increase, respectively. This is likely due to the sections being specifically optimized for UHPC.

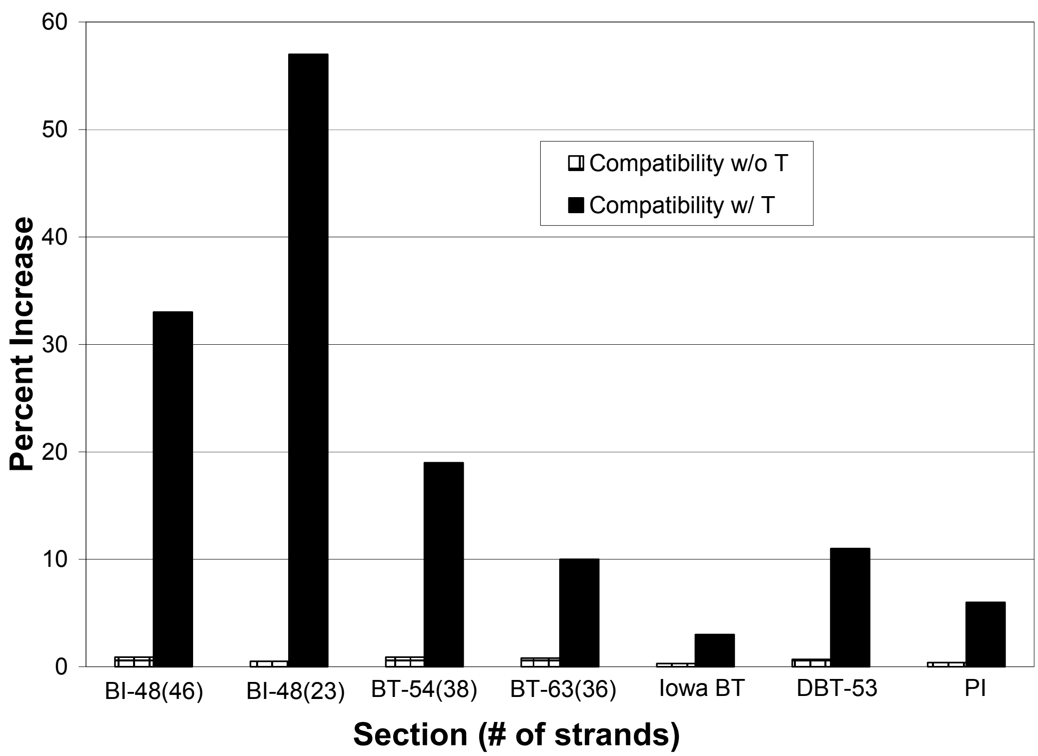

Figure 5: Percent Increase Compared to Closed-Form Approach

Table 4: Increase in Moment Capacity

Type	Section	Number of Strands	Increase Compared to Closed-Form Approach (%)	
			Strain Compatibility w/o Tensile	Strain Compatibility w/ Tensile
Box	BI-36	34	0.9	36
		17	0.5	65
	BI-48	46	0.9	33
		23	0.5	57
	BIV-36	34	0.6	40
		17	0.3	57
	BIV-48	46	0.6	34
		23	0.3	40
Bulb-Tee	BT-54	36	0.6	18
		38	0.9	19
	BT-63	36	0.8	10
		38	1.1	11
	BT-72	36	0.9	9
		38	1.2	9
	Modified Iowa 45	47	0.3	3
Deck Bulb-Tee	DBT-35	30	0.4	27
	DBT-53	30	0.7	11
	DBT-65	30	0.7	9
PI	FHWA Test Bridge	22	0.4	6

CONCLUSIONS

The results of this initial study show that the tensile strength of UHPC needs to be incorporated in the flexural analysis of standard sections to accurately predict the capacity. However, this may not be the case for sections optimized for UHPPC. In addition, more research is necessary to develop stress-strain curves for UHPC, to account for differences in utilizing small specimens for the curve and then applying the curves to large samples, and to account for variability that is inherent in all materials. AASHTO concepts and philosophies must also be evaluated for UHPC in order for designers to have procedures to design with this unique material.

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