Structural Reliability of UHPC Bridge Girders in Flexure

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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. The FHWA is currently investigating UHPC and is in the process of constructing a small test bridge at the Turner Fairbank Research facility utilizing a unique pi shaped section. Two states also have plans to utilize UHPC in bridges through the Innovative Bridge Research (IBR) program. Though UHPC is gaining exposure in the U.S. many questions still remain on design utilizing UHPC. If bridge designers are to use current standards, such as AASHTO LRFD, to design UHPC bridges, the equations must be reviewed to determine their validity for UHPC. An example of this is how does the reliability of an UHPC girder design compare to current standard concrete designs when AASHTO LRFD equations are used along with accompanying load and resistance factors. This research begins to examine the reliability of the ultimate flexural strength of UHPC girders using current AASHTO LRFD procedures and investigates modifications to the resistance factor for this failure state. Monte Carlo simulations were performed to account for the variability in several parameters including the high tensile strength of UHPC, dimensions, other material properties, and load.

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 150 to 250 MPa (22 to 30 ksi). UHPC is also known as Ultra-high performance fiber reinforced concrete (UHPFRC) which is defined by the Association Française de Génie Civil (AFGC) as a material with a cement matrix, a compressive strength that exceeds 150 MPa (22 ksi), and contains steel fibers (AFGC/SETRA, 2002). The steel fibers in UHPC create a ductile behavior under tension. The fibers in the mix give UHPC a direct tensile strength of 14 MPa (2 ksi) (Graybeal and Hartmann, 2003). The relatively high tensile strength results in the stress-strain diagram depicted in Fig. 1 (AFGC/SETRA, 2002).

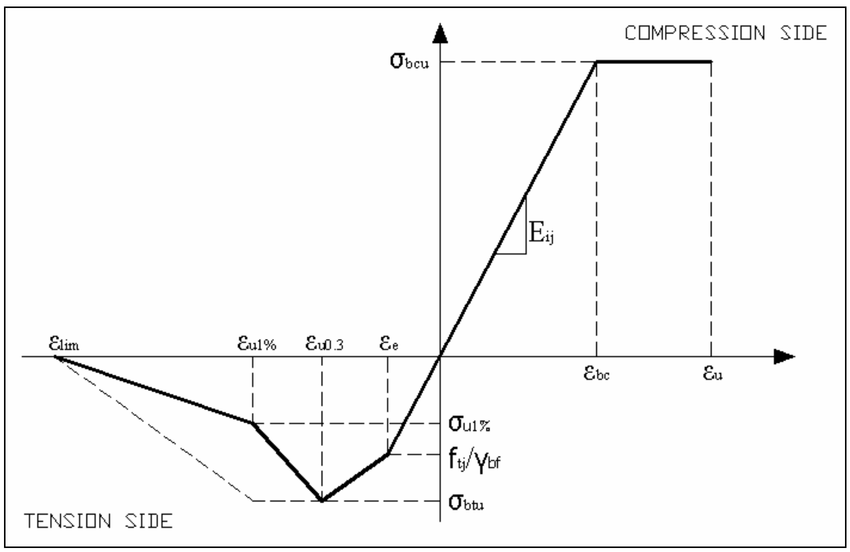

Figure1: Stress - Strain Diagram for UHPC

Different brands of UHPC are produced by several different concrete companies. These different brands very 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.

Table 1: UHPC Mix

Component	Amount (kg/m^3)	Amount (lb/ft^3)
Portland Cement	710	44
Silica fume	230	14
Quartz Powder	210	13
Fine Sand	1020	63
Steel Fibers	40-160	2-10
Superplasticizer	13	0.8
Water	140	9

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 (Ductal®, 2003). Heat treatment will give the UHPC some additional ductility, reduce future shrinkage and creep, and also increase the mechanical properties by approximately 15% (Ductal®, 2003). 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 (Graybeal and Hartmann, 2003) and is in the process of constructing a small test bridge at the Turner Fairbank Research facility utilizing a unique pi shaped section. Two states also have plans to utilize UHPC in bridges through the Innovative Bridge Research (IBR) program.

Though UHPC is gaining exposure in the U.S. many questions still remain on design utilizing UHPC. If bridge designers are to use current AASHTO LRFD standards to design UHPC bridges, the equations must be reviewed to determine their validity for UHPC. An example of this is how does the reliability of an UHPC girder design compare to current standard concrete designs when AASHTO LRFD equations are used along with accompanying load and resistance factors. This research begins to examine the reliability of the ultimate flexural strength of UHPC girders using current AASHTO LRFD procedures and investigates modifications to the resistance factor for this failure state.

Background

The parameters of structural design, no matter how much control is implemented, have some variability. Structural reliability analyses take into account some of the uncertainties and variability of structural design (Steinberg, 1997). The AASHTO LRFD Specifications are based on structural reliability and previous design specifications.

The reliability index, β, is typically used as a measurement of the level of reliability of a structural member or system for a specific failure mode. The relative reliability of a design based on probability distributions and statistics of resistance, load and load effects (Ellingwood, B. et al., 1980). β typically varies from 2 to 8 depending on the structural type, loading and resistance and loading factors (Ellingwood, B. et al., 1980).

There are several methods that exist to perform structural reliability analysis (Steinberg, 1997). Monte Carlo is one structural reliability technique that can easily be utilized when closed form solutions for the problem do not exist. In the Monte Carlo simulation technique, a random number between 0 and 1 is generated. This number is then used in the variable's cumulative distribution function (CDF), F(x), to produce a value for one of the variables. The type of cumulative distribution function is based on the statistical information of the variable. Figure 2 shows how the random number determines a value from the CDF of the variable. This process is repeated to generate values for all the variables that compose the strength of the member such as the concrete compressive strength, prestressing steel strength, and member dimensions. A similar process is used to generate values for the variables that compose the loading on the member. The values of the member are then utilized in an analysis to determine if the strength of the member exceeds the loading. The entire process is repeated numerous times with continually changing random values. The number of times the strength exceeds the loading determines the level of reliability for the member.

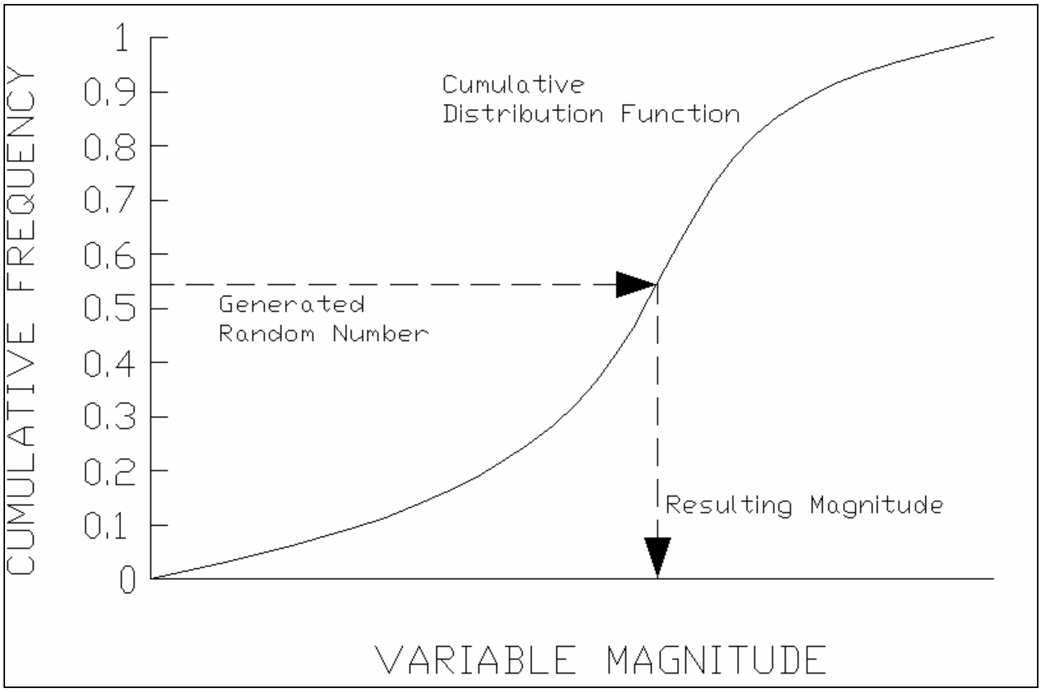

Figure 2: Cumulative Distribution Function

One of the basic equations for the ultimate limit state of flexural beam design in the AASHTO LRFD specification is:

$$\varphi\ M_n \geq \gamma_{DC}\ M_{DCn} + \gamma_{DW}\ M_{DWn} + \gamma_L\ M_{(Ln+I)n} \qquad (1)$$

where φM_n is the factored moment capacity, M_{DCn} is the nominal dead load moment caused by structural components and γ_{DC} is its dead load factor equal to 1.25. M_{DWn} is the nominal dead load moment caused by wearing surfaces and utilities, and γ_{DW} is its dead load factor equal to 1.5. $M_{(Ln + I)n}$ is the nominal live plus impact load, and γ_L is the live plus impact load factor equal to 1.75 (PCI, 2003). The moments are broken down into three types, each with its own factor, to account for the difference in variability of each type of load.

The nominal moment capacity for a prestressed concrete beam can be found by Eqn. 2.

$$M_n = A_{PS}\ f_{PS}\left(d_P - \frac{a}{2}\right) \qquad (2)$$

Eqn. 2 assumes no mild tension or compression reinforcement. A_{PS} in Eqn. 2 is the area of the prestressing reinforcement, d_P is the depth from the extreme compression fiber to the centroids of the prestressing steel, a is the depth of the compression block, and f_{PS} is the stress in the prestressing strands at ultimate. Eqn. 2 assumes the compression block to be rectangular. In addition, several procedures can be used to determine f_{PS}. Closed form equations exist and iterative procedures that utilize stress strain relationships can be performed.

Iterative procedures can also be used in replace of Eqn. 2 to determine the nominal moment capacity. One such procedure is often referred to as the Moment-Curvature (M-κ) approach. In this procedure, a linear strain distribution at ultimate is assumed. This determines the depth to the neutral axis. Stresses are determined along the depth of the member through stress-strain relationships of the concrete and prestressing steel. Numerical integration is performed to determine the compression force. This is compared to the tension force to see if equilibrium is met. If equilibrium is not achieved, a new strain distribution is assumed and the process is repeated. Once equilibrium is met, the moment arm between the forces is determined and the nominal capacity is calculated. This procedure can account for the nonlinear behavior of the concrete and steel. It can also account for the tensile capacity of the concrete, which can be significant in UHPC.

Analyses

In order to account for the variability in the flexural capacity of a UHPC member and determine the validity of the current ASSHTO LRFD specifications for UHPC, reliability analyses were conducted on pre-stressed UHPC box beams. The reliability analysis consisted of a Monte Carlo simulation of several types of box beams and the use of the M-κ approach to calculate the flexural capacity. The first AASHTO standard box beam analyzed was a BI-36 bridge box beam girder. Figure 2 shows the AASHTO standard beam BI-36.

This box beam had a nominal width of 914 mm (36 in) and a nominal height of 685 mm (27 in). The box beam had 17 half inch 270 grade low-relaxation strands located in a single row at a nominal 635 mm (25 in) from the top of the beam. The second box beam was an AASHTO standard box beam BI-36 but with 34 strands in two rows. The center of the two rows of prestressing strands was located at 613 mm (24 in) from the top of the beam. The third and final beam analyzed was AASHTO standard BIII-36 bridge box beam girder. This box beam had a nominal width of 914 mm (36 in) and a nominal height of 990 mm (39 in). The box beam had 17 half inch 270 grade low-relax strands located in a single row at a nominal 939 mm (37 in) from the top of the beam. Table 2 is a list of the analyses that were completed. In current LRFD bridge design standards, the value for φ in Eqn.1 was set equal 1.0. This value was changed to evaluate the reliability of UHPC in bridge beam design using current AASHTO design criteria and to test the applicability of the value for φ.

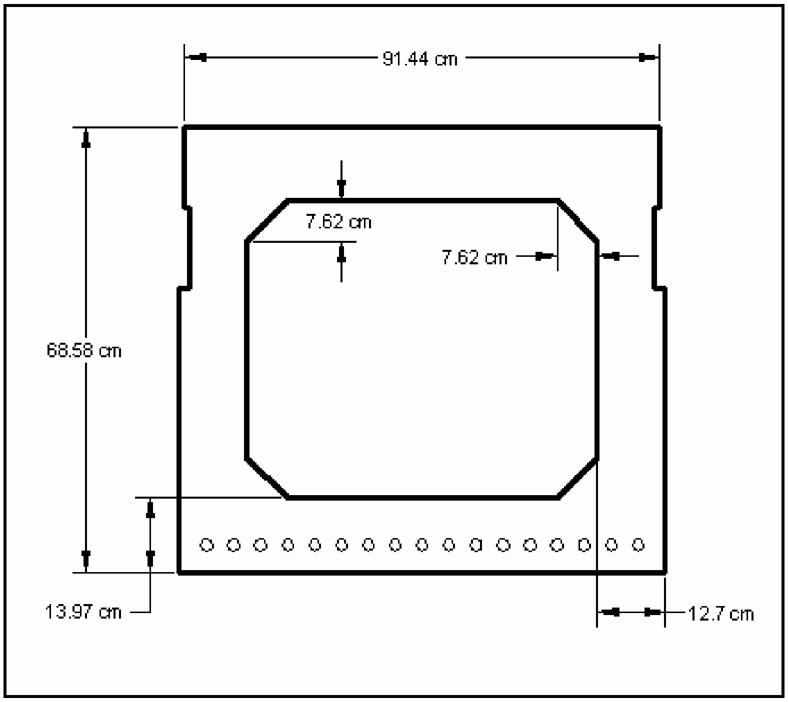

Figure 3: AASHTO Standard Beam BI-36

Table 2: Values used in each analysis

Analysis Number	Box Beam Type	Φ	# of prestressing strands	Nominal Moment Capacity kip*in	Nominal Moment Capacity kN*m	Nominal Calculation Method
1-5	BI-36	1.0-1.4	17	29,100	3,300	M & κ
6-11	BI-36	1.0-1.7	17	19,300	2,200	AASHTO closed form
12	BI-36	1.0	34	37,200	4,200	M & κ
13-14	BI-36	1.0-1.1	34	34,500	3,900	ACI closed form
15	BIII-36	1.0	17	46,800	5,300	M & κ
16-20	BIII-36	1.0 - 1.7	17	29,200	3,300	ACI closed form
21	BIII-36	1.0	34	61,500	7,000	M & κ
22-23	BIII-36	1.0 - 1.1	34	54,200	6,100	ACI closed form

The last column in Table 2 is the method utilized to determine the nominal flexural capacity. The AASHTO closed form approach was a conservative approach and was not as accurate as the M-κ approach. The nominal moment was calculated using the AASHTO closed form approach in order to test the feasibility of using this method for design of beams made with UHPC.

Monte Carlo simulation requires more then the nominal dimensions of the AASHTO standard box beams. The statistical distribution, mean, and standard deviation of each dimension are also required. Table 3 describes all the statistical information used for the box beam design and loading.

Table 3: Statistical information used in Monte Carlo Simulation

Variable	Distribution	Nominal	Mean	Coefficient of Variation	Ref.
Beam Width	Normal	bn	bn + (5/32)	$(1/4)*(bn + (5/32))^{-1}$	7
Beam Height	Normal	hn	hn	1/(4*hn)	7
Dp	Normal	dpn	dpn + (1/8)	$(11/32)*(dpn + (1/8))^{-1}$	7
Aps, $cm^2(in^2)$	Normal	0.987 (0.153)	0.999 (0.1548)	0.0125	7
Eps, GPa(ksi)	Normal	200 (29,000)	202 (29,319)	0.01	8
fpu, MPa(ksi)	Normal	1,862 (270)	1,937 (281)	0.025	7
fpe, MPa(ksi)	Normal	1,049 (152)	1,121 (163)	0.04	7
Ec, GPa(ksi)	Normal	49.6 (7,200)	49.6 (7,200)	0.17	1,2,3,9
ε_e	Normal	315×10^{-6}	315×10^{-6}	0.14	2
σ_{btu} MPa(ksi)	Uniform	36.8 (5.34)	36.8 (5.34)	0.30	9
$\sigma_{u1\%}$ MPa(ksi)	Uniform	36.8 (5.34)	36.8 (5.34)	0.30	9
Dead Load	Normal	*	Nominal	0.1	7
Live Load	Type I	*	Ln * 0.894	0.25	7

* Value dependent on analysis.

As Table 3 shows, the majority of variables were assumed to have a normal distribution. The Live Load was considered to have a Type I distribution. The tensile stress variables, σ_{btu} and $\sigma_{u1\%}$, were assumed to have a uniform distribution. The uniform distribution was used to account for the wide range of values and to keep the values between the highest and lowest values presented. The nominal values changed depending on the beam being analyzed, however several nominal values like that of the area of a single steel strand, Aps, stay the same no mater what beam is being analyzed. The dead and live load nominal values were calculated using Eqn. 2. The values for the mean and coefficient of variation were found in the references sighted for each variable.

When conducting Monte Carlo simulation, it was necessary to produce hundreds of thousands of simulations in order to provide a reliable result. The need of so many simulations combined with the calculation intensive method of the M-κ approach required the aid of a computer program. MATLAB was chosen for programming this task for its ease of use and access.

The program boxbeam was written in MATLAB. The program was broken into three phases. The first phase includes the user input and the Monte Carlo Simulation. The second phase includes the calculation of the ultimate flexural strength of each UHPC box beam using the M-κ approach. An excel program was constructed before the second phase of the box beam program was written. The Excel program was used as a method for testing accuracy of the M-κ approach. Excel's visual interface helped to determine possible sources of error, verify results of the box beam program, and also produce the nominal moment capacity needed for the third phase of the boxbeam program. The third and final phase is the calculation of the probability of failure, Pf, and the reliability index, β.

Results

Figure 4 provides results of the analysis 1, which were for the AASHTO standard box beam BI-36 with a single row of 17 prestressing strands. The M-κ approach was used to determine the nominal capacity. The ratios of M_L/M_{DC} and M_{DW}/M_{DC} are the ratios of the nominal live moment to nominal dead moment and nominal dead moment for wearing surfaces to nominal dead moment, respectively.

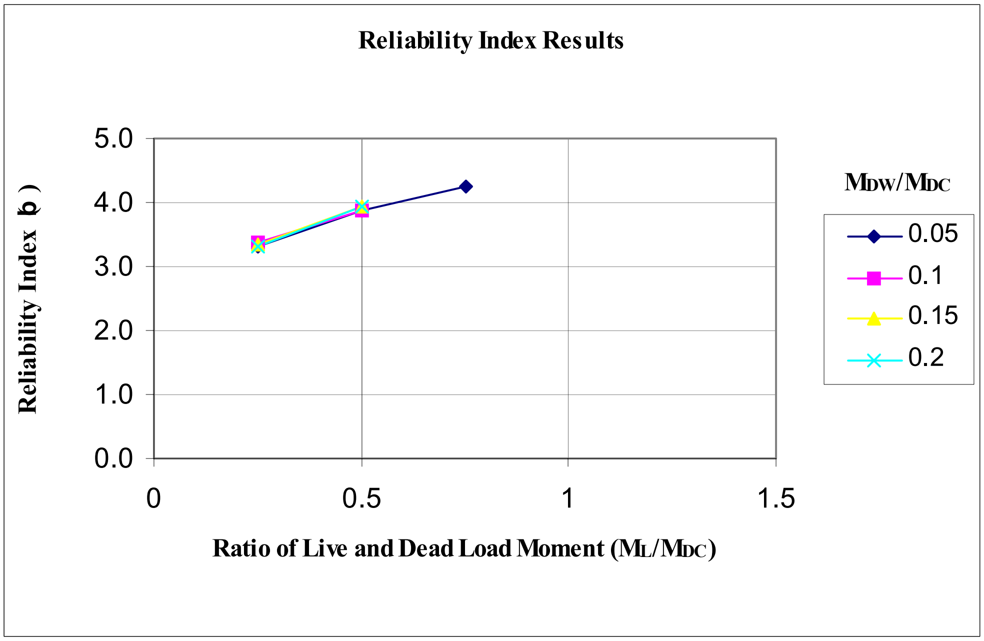

Figure 4: Reliability Results of Analysis 1

These results show that the reliability index stays with in the range of 3.3 to 4.3. Another important point illustrated by Fig. 4 is that β increases as M_L/M_{DC} increases, but remains relatively unchanged as M_{DW}/M_{DC} increases.

Figure 5 contains the reliability results of analysis 2, which is the same as analysis 1 except φ was increased to 1.1. This is done to test the current AASHTO LRFD bridge design standards and the feasibility of increasing the value for φ. The results from analysis 2 show that the reliability index is in the range of 3.0 to 4.3, which are very acceptable values.

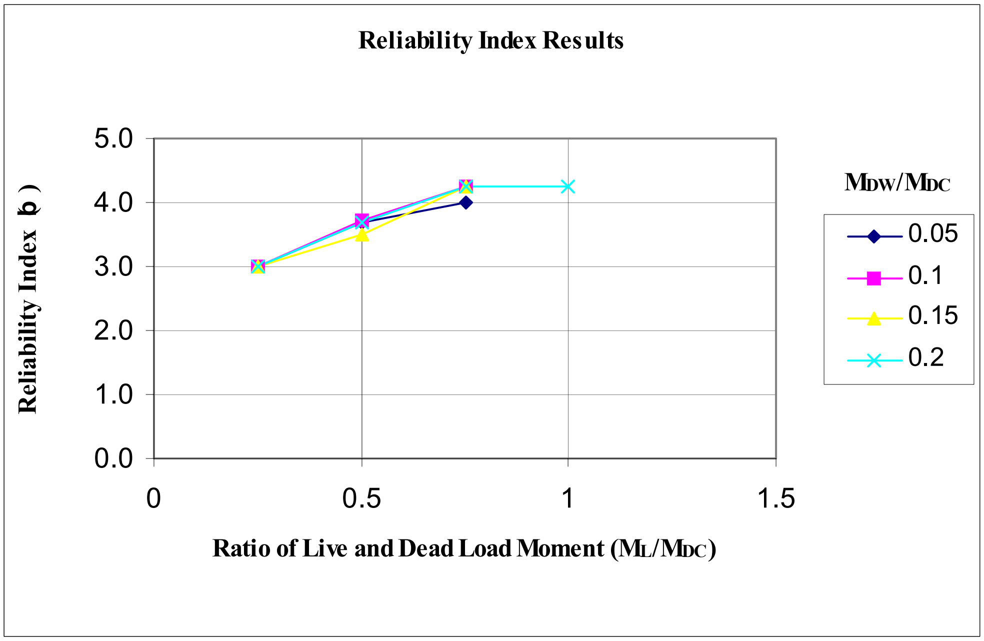

Figure 5: Reliability results of analysis 2

Figure 6 contains the results of analysis 5, which the value for φ is increased to 1.4. The reliability indices for M_L/M_{DC} ratio of 0.5 are now less than 3. In addition, the reliability indices for the lower M_L/M_{DC} ratio are significantly low. At this point it is easy to see that a φ of 1.4 is too high.

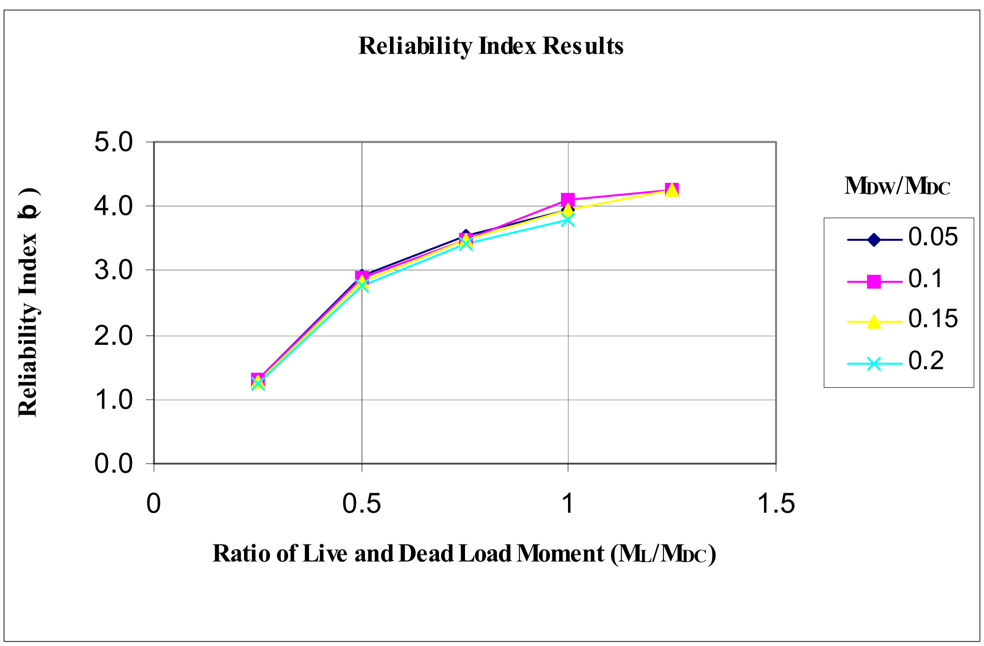

Figure 6: Reliability results of analysis 5

Analysis 6 was identical to the box beam in analysis 1 with the exception that the AASHTO closed form procedure (Eqn. 2) was used to calculate the nominal moment capacity of the UHPC box beam. The M-κ approached was still used in the Monte Carlo simulation. This procedures models the methodology when a beam is analyzed by an engineer with the AASHTO equations and the true behavior is more closely modeled by a mode detailed approach such as the M-κ analysis. This procedure accounts for some of the conservatism built into the AASHTO equations. The nominal moment capacity calculated using the M & κ approach was found to be 3,300 kN-m (29,100 kip-in). The nominal moment calculated using the more conservative AASHTO closed form approach was equal to 2,200 kN*m (19,300 kip*in). The results of analysis 6 did not produce many failures since the level of reliability was high and hence there was no need for a figure to show results. A β of 4.3 was found for low levels of M_L/M_{DC}. The high level of reliability was due to the low nominal moment capacity produced by the conservative AASHTO approach. With such high levels of reliability, it would seem logical to increase φ.

Analysis 11 was identical to analysis 6 except for the value of φ was increased to 1.7. Figure 7 illustrates the results of analysis 11. β drops below 3.0 at the low M_L/M_{DC} ratio and again the M_{DW}/M_{DC} ratio has little effect.

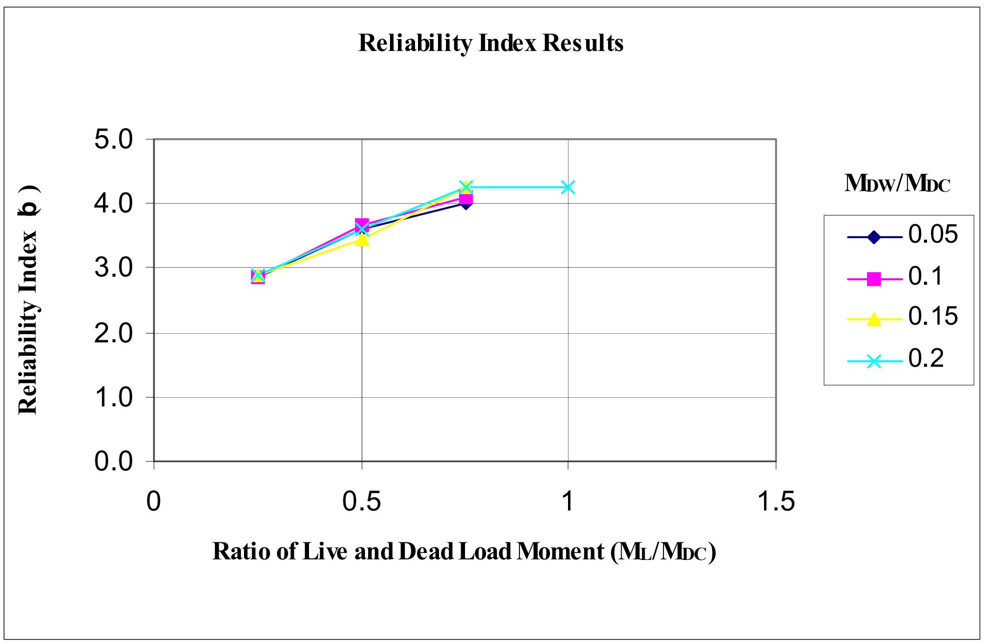

Figure 7: Reliability results of analysis 11

Analysis 12 was performed on a BI-36 box beam with the same nominal values as in analysis 1 except that the numbers of prestressing strands were increased to two rows of 17 making a total of 34 strands. Like analysis 1, the value for φ in analysis 12 was equal to 1.0. Also the nominal moment capacity in analysis 12 was calculated using the M - κ approach. Figure 8 contains the results of analysis 12. Analysis 12 was different than other analyses because the high number of prestressing strands produced a large tensile force in the UHPC box beam. As seen in Figure 8, the values for β range from 2.9 to 3.8.

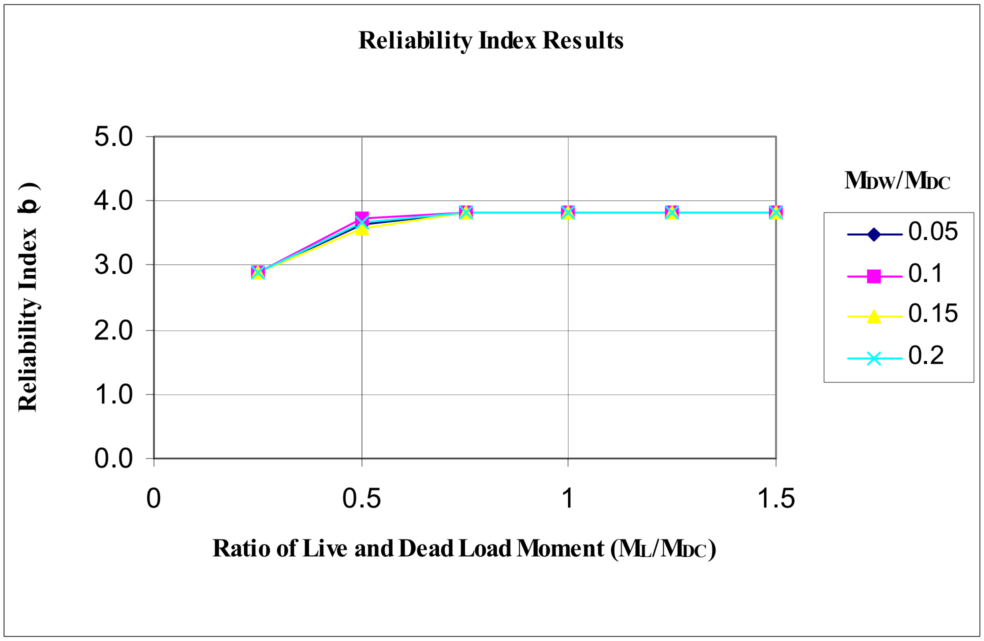

Figure 8: Reliability results of analysis 12

Analysis 13 was similar to analysis 12 expect the nominal moment capacity was calculated using the conservative AASHTO closed form approach for analysis 13. This resulted in a nominal moment capacity 3,900 kN*m (34,500 kip*in) compared to 4,200 kN*m (37,200 kip*in) calculated in analysis 12.

It is interesting to note the difference in the values of nominal moment capacities by the two different analysis approaches was only 300 kN*m (2,700 kip*in) for two rows of strands and 1,100 kN*m (9,800 kip*in) for a single row of strands. This is due to the high level of tensile stress created by a large number of prestressing strands. The conservative AASHTO closed form approach does not account for the high tensile strength of UHPC. This is why when there is only one row of strands the difference in the methods of calculating the nominal moment capacity is so large. When there are two rows of prestressing strands the tensile strength of UHPC is small when compared to the tensile strength in the prestressing strands and therefore, including the tensile strength of UHPC is not as vital. Figure 9 contains the results of analysis 13. β values ranged from 3.2 to 3.8.

Analysis 14 was the same as analysis 13 except φ was increased to 1.1. This resulted in β becoming less than 3 for low M_L/M_{DC} ratios.

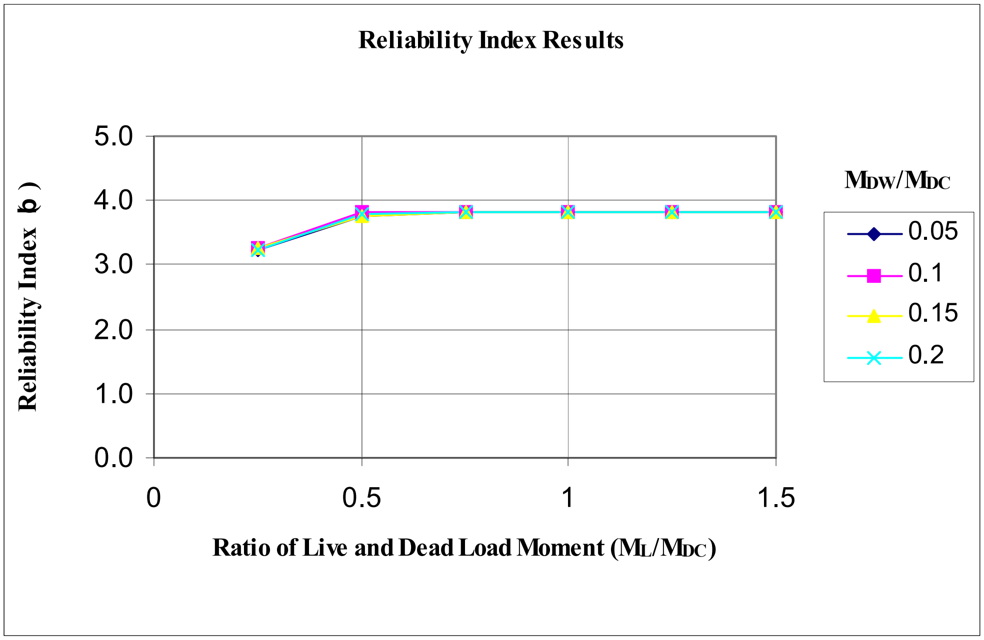

Figure 9: Reliability results of analysis 13

Analysis 15 investigated the AASHTO BIII-36 standard box beam with a single row of 17 prestressing strands. The value for φ in analysis 15 was set equal to 1.0. Figure 10 contains the results of analysis 15. The box beam in analysis 15 was almost identical to the box beam in analysis 1 with the exception that the nominal height of the box beam in analysis 1 was 685.8 mm (27 in) and the nominal height of the box beam in analysis 15 was 990.6 mm (39 in). As can be seen from the results, the reliability indices where relatively high and similar to previous results.

Figure 11: Reliability results of analysis 15

Analysis 16 was similar to analysis 15 with the only difference being that the AASHTO closed form approach was used to determine the nominal moment. In analysis 15, the nominal moment capacity was calculated using the M - κ approach and found to be equal to 5,300 kN*m (46,800 kip*in). For analysis 16 the nominal moment was calculated using the more conservative AASHTO closed form approach and was 3,300 kN*m (29,200 kip*in). The simulations did not produce any failures for analysis 16 in which φ was equal to 1.0. Therefore, φ was increased to reduce the reliability index.

The reliability indices did not become reasonable until analysis 20 where φ was 1.7. Even at a φ = 1.7, the reliability indices for M_L/M_{DC} ratios at or above 0.5 were approximately 4. Figure 12 contains the results of analysis 20.

Figure 12: Reliability results of analysis 20

Analysis 21 investigated the AASHTO BIII-36 standard box beam with two rows of 17 strands making a total of 34 strands. Like analysis 15, the value for φ in analysis 21 was equal to 1.0. Also the nominal moment capacity in analysis 21 was calculated using the M - κ approach. Figure 13 contains the results of analysis 21. β ranges from 3.1 to 4.1.

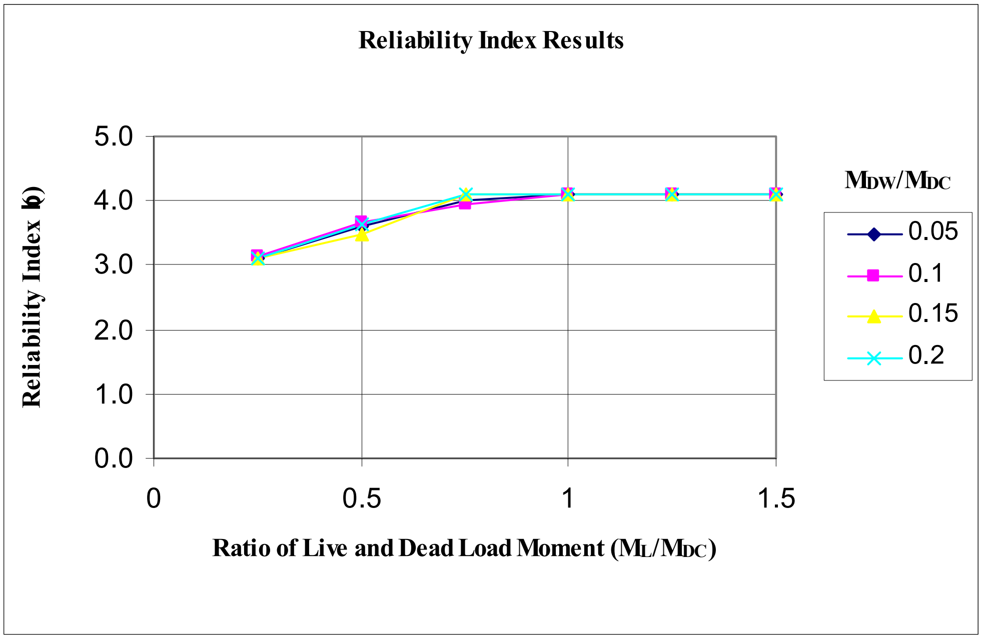

Figure 13: Reliability results of analysis 21

Analysis 22 was similar to analysis 21 in all respects expect one. For analysis 22, the nominal moment capacity 6,100 kN*m (54,200 kip*in) was calculated using the AASHTO closed form approach. Figure 14 contains the results of analysis 22. The values for β are very similar to the results of analysis 21.

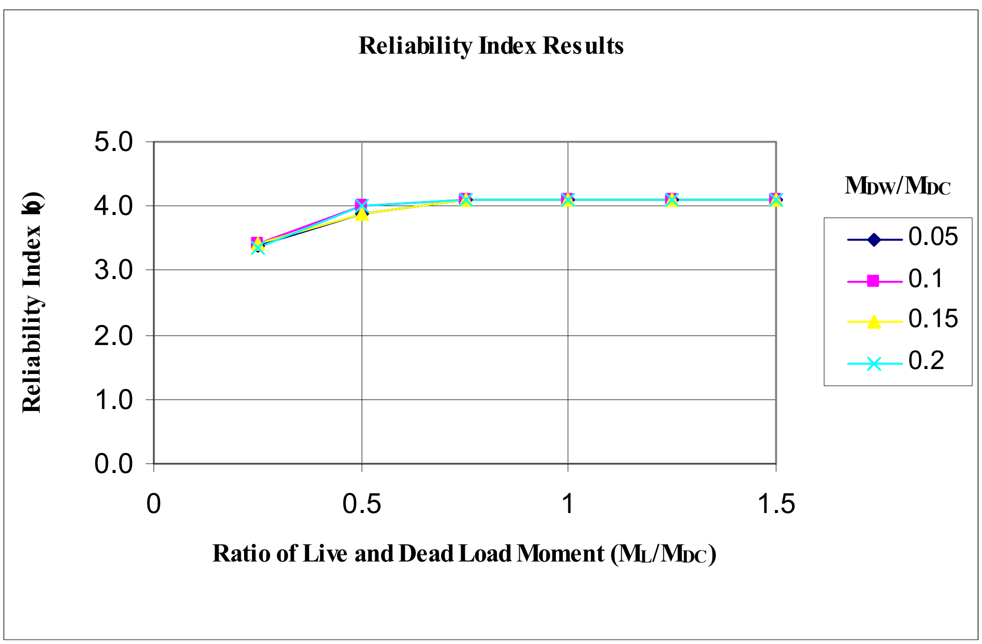

Figure 14: Reliability results of analysis 22

Due to the results of analysis 22 being slightly high, φ was increased to 1.1 for analysis 23. β decreased slightly at low M_L/M_{DC} ratios but still remained greater than 3.0.

Conclusions

The initial reliability analyses study presented has shown that UHPC box beam flexural design, in accordance to current AASHTO LRFD bridge design standards, produces a conservative reliability index. The design of lightly reinforced UHPC box beams, in accordance to current AASHTO LRFD bridge design standards, may be over conservative. To rectify this, either a more advanced analysis procedure, such as the M - κ approach, should be used to evaluate the UHPC Box beam's flexural capacity or the φ factor should be increased. The advanced analysis procedure should account for the significant contribution the UHPC tensile capacity makes to the flexural strength.

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