Reliability of CFRP-Prestressed Concrete Girders for Highway Bridges

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Abstract: This paper presents the reliability of concrete girders prestressed with carbon fiber reinforced polymer (CFRP) composite tendons for highway bridge application. After conducting a literature review, a calibration process is formulated to determine the strength reduction factors of the bridge girders subjected to flexural loading. The safety requirements of the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (BDS) are referenced and taken into consideration. The proposed design factors are recommended to replace those in published documents.

1. INTRODUCTION

The demand for sustainable materials is increasing these days to address socioeconomic challenges facing the infrastructure community. In lieu of conventional steel strands, carbon fiber reinforced polymer (CFRP) composite tendons have gained attention and are considered a promising alternative. For the implementation of such a non-conventional material, an adequate design approach is imperative so that practitioners can carry out necessary steps to complete an engineering project. Although the use of CFRP tendons is analogous to that of steel strands from an application point of view, their physical and mechanical properties differ and hence design factors that have been developed for the steel strands may not be adopted for CFRP tendons. Selected early efforts on CFRP-prestressed concrete are as follows. The static and fatigue behavior of prestressed girders with CFRP was studied in conjunction with bonded and unbonded tendons [1]. Notwithstanding noticeable deflection, the girders survived over seven million cycles of repeated loadings. An analytical study [2] clarified that the center of gravity of vertically distributed CFRP tendons played an important role in flexural design because the tendons may fail in a progressive manner without demonstrating a yield plateau. Since the constitutive relationship of CFRP is linear elastic, the definition of traditional ductility is invalid and another concept, called deformability, is often adopted to assess the flexural characteristics of structural members [3]. The most comprehensive document in the area of CFRP-prestressed concrete is ACI 440.4R-04 [4]; however, this guideline is outdated and contents need to be updated. Among many, the reliability of bridge girders is insufficiently stated; for instance, strength reduction factors were empirically developed without calibration. The present study aims to address such an identified concern and proposes new factors to promote CFRP-prestressed concrete technologies.

2. LITERATURE REVIEW

2.1. CFRP Prestressing Material

Two primary components are used to form CFRP tendons: unidirectional carbon fibers and a polymeric resin matrix. The volumetric ratio of the fibers dominates the properties of the composite tendons. Notable commercial products are Leadline consisting of woven high strength fibers with an epoxy and Carbon Fiber Composite Cables (CFCC) comprising straight fibers and an epoxy resin. A number of advantages were reported, namely, high strength and light density, high modulus, and durability; at the same time, several disadvantages were also known: cost, brittle failure, and insignificant dowel resistance.

2.2. Strength of CFRP

Currently, globally accepted strength requirements are unavailable and the capacity of CFRP is largely dependent upon manufacturer. On many occasions, the strength of CFRP is determined by either the mean value minus three standard deviations or the 95 percent inclusion strength. Considering that CFRP does not yield, practitioners should carefully examine the applicability of the reported strength and potential failure modes.

2.3. Pros of CFRP

The introduction of structural composites to the prestressed concrete community was a notable turning point for the construction of highway bridges. Unlike typical cases, members with CFRP are not corrosive and thus a significantly extended service life is anticipated. The unique nature of linearly responding CFRP enables predictable mechanical performance when subjected to gravity loadings. Furthermore, engineers can obtain products from simple calculations without conducting rigorous nonlinear analysis.

2.4. Cons of CFRP

The high strength and modulus of CFRP in the longitudinal direction are not preserved in the transverse direction. For this reason, CFRP tendons may experience premature failure due to dowel action where shear forces are applied. Such distress may occur when a member cracks and the split concrete parts are connected by CFRP. The transverse shear resistance of CFRP may be around 5% of the longitudinal tensile strength. The orthotropic CFRP material is susceptible to stress directions. To form a harped profile of prestressing elements, CFRP tendons may be bent at one or two locations along the span of a precast girder. The increased stress at harped points is proportional to a ratio between the tendon radius and the curvature radius of the saddle [4]. Some examples on harping stresses are provided in Table 1. These stresses should be taken into consideration when long-term performance is evaluated. Creep rupture resulting from sustained load is a critical factor for CFRP tendons. Upon reaching a creep limit, CFRP abruptly fractures without warning. To avoid this unfavorable failure mode, ACI 440.4R-04 limits a jacking stress level to 65% of the ultimate stress [4].

Table 1: CFRP stress increase at variable saddle radii

Radius (mm)	Tendon radius = 10 mm Elastic modulus = 142 GPa	Allowable (f_{fu} = 2,250 MPa) ($0.65f_{fu}$ = 1,463 MPa)
25	4,017	1,896%
100	1,004	474%
150	670	316%
300	335	158%
600	167	79%
1,500	67	32%
3,000	33	16%

2.5. Field Demonstration

It is acknowledged that the Beddington Trail Bridge in Alberta, Canada, was the first CFRP-prestressed concrete application in North America. The two-span bridge had a length of 43 m and a width of 23 m, and it was open to the public in 1993. The dimension of each girder was 1,500 mm wide and 1,100 mm deep alongside 26 pretensioned tendons (diameter = 15 mm). Another notable example is the Bridge Street Bridge in Michigan, USA, which was constructed in 2001. The bridge had two spans and its total length was 64 m, supported by 4 double-tee girders (width = 2,120 mm and depth = 1,220 mm). A combination of internally bonded and externally unbonded CFRP tendons was utilized to satisfy strength and deformability requirements. To monitor the in-situ performance of the bridge structure, sensors were installed and data were logged for a period of 5 years. The measured behavior of the bridge was close to the predicted values in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (BDS) [5].

2.6. Strength Reduction Factor

The strength reduction factors of CFRP-prestressed concrete members are suggested to be ϕ = 0.85 and 0.65 for tension-controlled and compression-controlled cases, respectively in ACI 440.4R-04 [4]. These factors were empirically developed using 30 beams, 22 of which were CFRP-prestressed concrete members [6]. Concerns include that there was no reliability calibration and the applicability of the ϕ factor was not fully appraised.

3. PROTOTYPE BRIDGE GIRDERS

3.1. Trial Design

Previously designed prestressed concrete bridge samples were reviewed to best select candidate superstructure types. The number of target bridges was 20 to 30, which can represent constructed bridges in the United States. Parameters considered were the types of girders, span lengths, and deck widths. In this study, only precast prestressed bonded tendons were of interest and other kinds (e.g., unbonded tendons) were not included.

3.2. Selected Bridge Configurations

Upon completion of the review process, a total of 25 cases were selected. All bridges were slab-on-girder types and their configurations involved box, tub, bulb-tee, and I-shape girders. The range of spans was from 17 m to 50 m with girder spacings of 1.2 m to 5.6 m. The depth of the individual girders varied between 640 mm and 2,300 mm.

Table 2: Bridge layout

Identification	Girder type	Span length (m)	Bridge width (m)	Structural depth (mm)	Girder spacing (mm)
1	Box	7.6	25.0	635	1,250
2	Box	7.6	25.0	635	1,250
3	Tub	24.4	16.6	2,032	4,176
4	Box	13.4	27.4	1,118	1,402
5	Box	13.4	27.4	1,118	1,402
6	Tub	21.0	21.6	1,753	5,486
7	Tub	21.0	21.6	1,753	5,486
8	Bulb-tee	15.2	13.1	1,270	1,829
9	Bulb-tee	18.9	15.7	1,575	1,981
10	Tub	24.4	16.6	2,032	4,176
11	Tub	19.2	8.8	1,600	2,743
12	I	21.9	47.9	1,829	2,987
13	I	21.9	47.9	1,829	2,987
14	Box	16.2	20.4	1,346	1,646
15	Tub	24.4	38.6	2,032	5,639
16	Tub	24.4	36.6	2,032	4,877
17	Box	17.1	20.4	1,422	1,646
18	Tub	24.4	38.6	2,032	5,639
19	Tub	22.6	36.6	1,880	4,877
20	Box	16.5	13.1	1,372	1,250
21	Box	16.2	46.3	1,346	1,890
22	Box	16.2	46.3	1,346	1,890
23	Tub	24.4	16.6	2,032	4,176
24	Tub	24.4	36.6	2,032	4,877
25	Bulb-tee	28.0	8.5	2,337	2,042

3.3. Implementation of Design

A commercial computer program was utilized to conduct preliminary designs in accordance with AASHTO LRFD BDS. The default geometric properties of all candidate bridges were adopted from as-built drawings (Table 2) and it was assumed that the prestressed concrete girders were precast members and assembled on site to form a continuous system. The properties of Leadline CFRP tendons were employed (Table 3) and minor adjustments were made to facilitate design processes. The jacking details of the tendons were based on the provisions of ACI 440.4R-04 [4]. Local stress concentrations were ignored; that is, an increased stress at a harping point was not taken into account. As far as prestress losses are concerned, simplifications were considered; for example, the CFRP tendons were regarded as a low relaxation material.

Table 3: Engineering Properties of CFRP

Property	Value
Diameter	16 mm
Cross-sectional area	126 mm^2
Ultimate strength	2,250 MPa
Modulus of elasticity	142 GPa
Stress limit for jacking	212 MPa
Force for jacking	185 kN
Fiber volume ratio	0.65

4. MODEL DEVELOPMENT

4.1. Capacity Assessment

Two types of failure modes were considered when calculating the flexural capacity of the CFRP-prestressed concrete girders: compression-controlled and tension-controlled sections for concrete crushing at a strain of 0.003 and CFRP rupture, respectively. Pursuant to the provisions of ACI 440.4R-04 [4] in conjunction with force equilibrium and displacement compatibility, nominal moments (M_n) were determined.

4.2. Random Variation

The variables of the analytical model formulated in a previous section were stochastically simulated with the properties listed in Table 4. To appropriately handle the adequacy of CFRP from a statistics standpoint, ASTM E112 [7] was referenced.

4.3. Implementation of Model

The aforementioned statistical properties for the bridge girders and CFRP were incorporated into the modeling framework and the Monte-Carlo method was carried out. For realistic simulations, the resistance of the bridge girders (M_R) was obtained by multiplying some factors [8]

$$M_R = \psi_a \psi_F \psi_M M_n \tag{1}$$

where ψ_a, ψ_F, and ψ_M are the analysis, fabrication, and material factors, respectively. These factors account for potential differences between the theoretically assumed properties and actual ones in terms of strength, material, and geometry. Additionally, bias factors and the coefficient of variation were included in the model calculation.

Table 4: Statistical Properties Used for Stochastic Simulations

Variable	Distribution	Bias factor	Coefficient of variation	Reference
Geometry	Normal	1	0.03	[9]
Area of CFRP tendon	Normal	1	0.05	[10]
Compressive strength of concrete	Normal	1.14 to 1.40	0.1	[11]
Elastic modulus of concrete	Normal	1	0.1	[16]
Tensile strength of CFRP	Weibull	1.05	0.04	Authors
Elastic modulus of CFRP	Lognormal	1.04	0.04	Authors
Relaxation of CFRP	Normal	1	0.30	[12]
Relative humidity	Normal	1	0.75	[13]
Dead load	Normal	1	0.10	[8]
Wearing surface	Normal	1	0.25	[8]
Live load	Normal	1.28	0.18	[8]

5. MODEL CALIBRATION

5.1. Parameters

5.1.1. Load

In line with AASHTO LRFD BDS [5], Strength Limit State I was taken and the dead and live load factors were specified: 1.25 for gravity loads, 1.50 for a wearing surface, and 1.75 for live loads. The standard live load of HL-93 and associated distribution factors were employed to generate flexural distress.

5.1.2. Resistance

Through Monte-Carlo simulations with an assumed strength reduction factor of ϕ =1.0, the aforementioned design factors were estimated.

5.2. Calculation of Reliability Index

The reliability index of the bridge girders (β) was attained by [8]

$$\beta = \frac{\left[M_R(A)(1-\ln(A)) - M_E\right]}{\sqrt{(M_R V_R A)^2 - (M_E V_E)^2}} \tag{2}$$

where M_R is the section resistance; A is the characteristic number (A = 1 - kV_R, in which k is a shifting factor from the mean in standard deviation units and V_R is the coefficient of variation for the resistance); M_E is the mean load effects; and V_E is the coefficient of variation for the load effects. The target reliability index was set to β = 3.5.

5.3. Procedure for Calibration

Below is a summary of the procedure for calibrating the resistance factors of CFRP-prestressed concrete bridge girders:

1) Distribution of statistical parameters
2) Assumption of a trial reduction factor
3) Property simulations
4) Calculation of resistance
5) Computation of a reliability index
6) Implementation of Monte-Carlo simulations
7) Calibration of reduction factors
8) Determination of reduction factors

6. OUTCOMES OF SIMULATIONS

6.1. Resistance Parameters

A comparison is made between the test and nominal capacities of beam specimens in Fig. 1(a). The experimental data were collected from literature [14]. A bias factor of 1.097 was observed with a coefficient of variation of 0.18, which are higher than those of conventional steel-prestressed concrete members: 1.0 and 0.89, respectively [8,15].

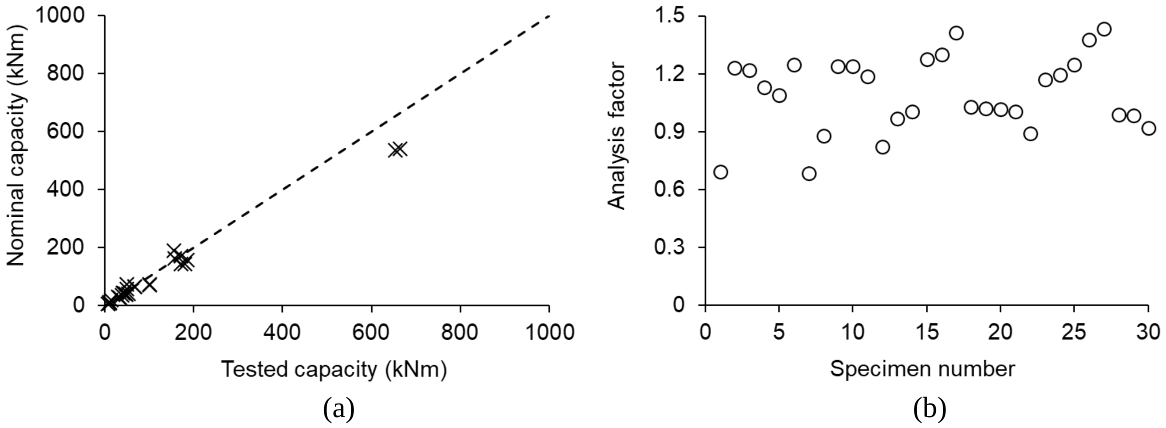

Fig. 1. Calibration of the ψ_a factor: (a) flexural capacity; (b) variation

6.2. Determination of reduction factor

Figure 2 displays the variation of reduction factors (ϕ), depending upon the level of safety (β = 2.5, 3.0, and 3.5). With the decreased safety index, the minimized $(\beta - \beta_T)^2$ term increased (β_T = target safety). The difference between the tension- and compression-controlled sections (Figs. 2(a) and (b), respectively) is attributed to their distinct failure modes: CFRP rupture vs. concrete crushing. Overall, the compression-controlled sections provided higher reduction factors in comparison with the tension-controlled sections.

7. DESIGN RECOMMENDATIONS

To implement the calibrated reduction factors in practice, the ϕ factors for the tension- and compression-controlled sections were proposed to be 0.75 and 0.80, respectively. These newly proposed values differ from those of ACI 440.4R-04 (0.85 and 0.65), which were not calibrated using reliability theory.

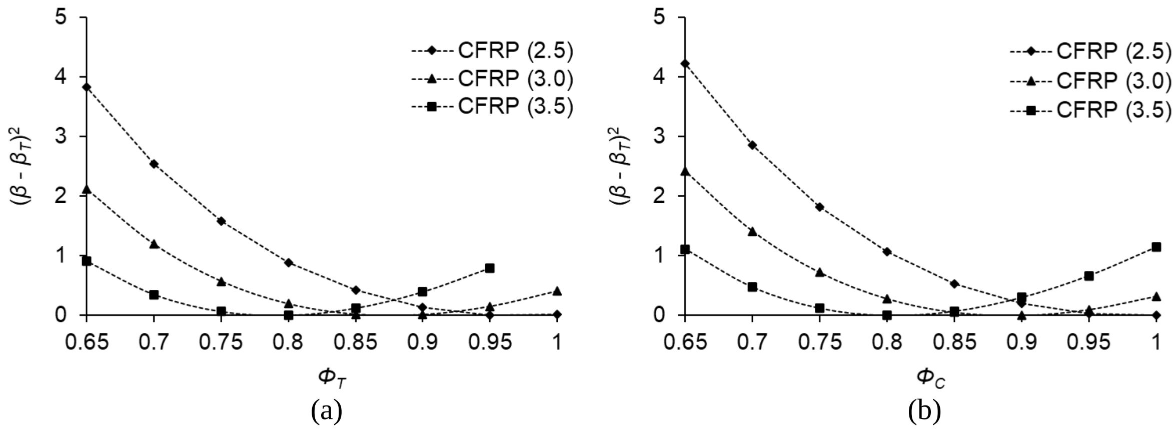

Fig. 2. Variation of reduction factor for CFRP-prestressed concrete girders: (a) tension-controlled section; (b) compression-controlled section

8. CONCLUSIONS

This paper has elaborated on the calibration of reduction factors (ϕ) for CFRP-prestressed concrete bridge girders, based on reliability theory. Benchmark bridges were designed on the basis of bridges constructed in the United States. The level of safety varying from β = 2.5 to 3.5 was considered and design recommendations were proposed: ϕ = 0.75 for tension-controlled sections and 0.80 for compression-controlled sections.

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