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Advances in Concrete Bridges: Design, Construction, Evaluation, and Rehabilitation

Editors:
Yail J. Kim, John J. Myers, and Antonio Nanni

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American Concrete Institute
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PREFACE

Advances in Concrete Bridges: Design, Construction, Evaluation, and Rehabilitation

Concrete bridges play an important role in the efficiency and reliability of transportation civil infrastructure. Significant advancements have been made over the last decades to enhance the performance and durability of bridge elements at affordable costs. From an application perspective, novel analysis techniques and construction methods are particularly notable, which have led to the realization of more sustainable built-environments. As far as the evaluation and rehabilitation of constructed bridges are concerned, new nondestructive testing approaches provide accurate diagnosis and advanced composites, such as carbon fiber reinforced polymer (CFRP), have become an alternative to conventional materials. This Special Publication (SP) contains nine papers selected from two technical sessions held at The ACI Concrete Convention and Exposition – Spring 2018, in Salt Lake City, UT. The objective of the SP is to present technical contributions aimed to understand the state of the art of concrete bridges, identify and discuss challenges, and suggest effective solutions for both practitioners and government engineers. All manuscripts were reviewed in accordance with the ACI publication policy. The Editors wish to thank all contributing authors and reviewers for their rigorous efforts. The Editors also gratefully acknowledge Ms. Barbara Coleman at ACI for her knowledgeable guidance in the development of the SP.

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TABLE OF CONTENTS

SP-333-1

A Numerical Analysis Methodology for the Strengthening of Deep Cap Beams

Rafael A. Salgado, Serhan Guner

Synopsis: A significant number of in-service bridges have been subjected to loads above their original design capacities due to the increase in traffic and transported freight in the past decades. Externally bonded fiber reinforced polymers (FRP) is a non-destructive retrofit technique that has become common for the strengthening of overloaded cap beams of bridges. However, there is a lack of analysis methods for the retrofitted cap beams that can accurately predict the retrofitted structural response while accounting for the critical material behaviors such as bond-slip relationships, confinement effects, and redistribution of stresses. In this study, an analysis methodology using nonlinear finite element models is proposed for cap beams retrofitted with externally bonded FRP fabrics. A two-stage verification of the proposed methodology was employed: a constitutive modeling and critical behavior of materials verification using experimental results available in the literature; and a system-level load capacity determination using a large, in-situ structure. The proposed methodology was able to capture the FRP-concrete composite structural behavior and the experimentally observed failure modes. The FRP retrofit layout created using the results of this study increased the capacity of the initially overloaded cap beam in 27%, granting it a 6% extra capacity under its ultimate loading condition.

Keywords: deep beams; nonlinear analysis; cap beam; structural assessment; FRP; retrofit; analysis methodology

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INTRODUCTION

Externally bonded fiber reinforced polymer (FRP) is a non-destructive and efficient retrofit technique that has been increasingly common for the strengthening of overloaded bridge cap beams. Despite its large applicability, there is still a lack of analytical methods for the retrofitted cap beams that can accurately predict their structural response due to the added FRP fabrics. Despite some simple equations given by codes[1,2] to obtain an estimate of the added flexural and shear capacity due to the FRP fabrics, several material behaviors that are critical to obtain an accurate response of the retrofitted structure such as bond-slip relationships, confinement effects, and redistribution of stresses are not considered. On top of that, due to their small shear spans, cap beams are usually classified as deep elements that form a direct strut action (i.e., a diagonal compressive stress field between the load application point and the supports) and do not satisfy the Euler-Bernoulli theory (i.e., plane sections remain plane). By neglecting these important structural behaviors when performing retrofit studies using FRP fabrics, the calculated FRP retrofit layout is at risk of being ineffective or even detrimental to the original cap beam. Thus, the complexity and uniqueness of each cap beam require an effective analysis approach with an accurate FRP modeling methodology to substitute any ‘guess-work’ with a better understanding of the structural behavior.

This study proposes an analysis methodology for deep cap beams retrofitted with externally bonded FRP fabrics. The methodology is presented in two stages with respective verifications: constitutive modeling of the critical behavior of materials; and an overall methodology application using a large, in-situ structure. The material behavior models and the modeling procedure proposed are verified using experimental results available in the literature. The overall modeling process is presented to assist in accurately analyzing cap beams using the proposed methodology.

RESEARCH SIGNIFICANCE

FRP fabrics have been commonly used to retrofit deep cap beams of in-service bridges that have become structurally deficient due to the increase in loading condition over the decades. There is a lack of holistic analysis approaches to accurately calculate the load capacity of retrofitted cap beams while accounting for the concrete’s deep beam actions and the composite behavior introduced by the FRP fabrics. This study details a finite element approach that aims to provide a holistic understanding of the structural behavior and to accurately calculate the load capacity of FRP retrofitted deep cap beams.

PROPOSED CAP BEAM NUMERICAL MODELING AND SYSTEM-LEVEL ANALYSIS METHODOLOGY

A numerical modeling and system-level analysis methodology for deep cap beams retrofitted with externally bonded FRP is proposed using nonlinear finite element analysis (NLFEA). NLFEA models are suitable for the assessment of deep cap beams due to its implementation of the nonlinear effects that are characteristic of deep elements, such as the nonlinearity of the strain distribution and the effects of cracking on the stress distribution[3,4]. Using NLFEA, the performance of the structure under both the serviceability and ultimate limit state conditions can be verified and it allows for the prediction of the progression of nonlinear events (i.e., concrete cracking, reinforcement yielding, concrete crushing, and the formation of the failure mechanism). Using the proposed methodology, if the NLFEA analysis of an un-retrofitted cap beam calculates an overloaded structural state, then a retrofit study using externally bonded FRP fabrics must be conducted to ensure the adequacy of the cap beam to its ultimate loading condition. In such cases, an NLFEA analysis is essential to get an accurate capacity of the deep beam and to determine an FRP retrofit layout that effectively captures the deficiencies of the beam.

Finite element material modeling approach

The proposed approach was developed using a two-dimensional continuum finite element model. When analyzing reinforced concrete structures, proper modeling of the constitutive response and important second-order material behaviors are crucial[5,6]. Thus, in this study, the model was developed using the computer program VecTor2[7]. Other specialized programs could also be used for this purpose; however, the selection of VecTor2 was made because it accounts for several second-order material behavior models that are particular to cracked reinforced concrete (see Table 1). VecTor2 uses a smeared rotating crack model based on the equilibrium, compatibility, and constitutive models of the Disturbed Stress Field Model[8], which is a refined version of the Modified Compression Field Theory[9] (MCFT), a theory that has been recognized and adopted by the AASHTO[10] and CSA A23.3[4] codes.

In the proposed methodology, the concrete is modeled using 8-degree-of-freedom quadrilateral elements (in geometrically uniform regions) or 6-degree-of-freedom triangular elements (in geometrically non-uniform regions such as inclined sections). The concrete material stress-strain response is accounted for using a plastic-offset-based nonlinear model[7]. Several pre- and post-peak models that vary in complexity and applicability are available in the literature; Table 1 summarizes the models used in this study with detailed formulation available elsewhere[7]. The concrete model includes nonlinear hysteresis rules for the unloading and reloading conditions[7] (see Figure 1a). Even though the proposed methodology includes a static pushover analysis, some parts of the cap beam will unload and some other parts will reload, as the concrete cracking and reinforcement yielding take place, thereby requiring the use of a hysteretic material behavior.

The shear reinforcement is accounted for through a smeared material model due to their even space across the element. On the other hand, the longitudinal reinforcement is modeled using discrete truss elements (1-degree-of-freedom per node) due to the large amount of steel in specific locations of the structure. The response of the reinforcing bars is modeled using a three-partite constitutive model (see Figure 1b), including a parabolic strain hardening region as per the model of Seckin[18].

The FRP fabrics are accounted for in the model through tension-only truss elements aligned vertically, horizontally, or in both directions depending on the fiber orientations of the fabrics. If the fabric has fibers oriented vertically, horizontally, or in both directions, the cross-sectional area of the truss elements is comprised of the effective width of each truss and the thickness of the combined FRP layers. On the other hand, if the fabric has fibers oriented in arbitrary directions, the vertical and horizontal truss-elements' sectional area are comprised of the equivalent horizontal, or vertical, fiber amount. Figures 2a and b show the case of FRP fabric with fibers oriented in an arbitrary direction, which is the most general case. The constitutive model of the fabrics is elastic up to their maximum tensile stress (see Figure 2c).

The modeling of the bond-slip response of the fabrics is crucial for an accurate model because it is a dominant failure mode for structures retrofitted with FRP fabrics[24]. Thus, to account for the bond-slip behavior, link elements (i.e., bi-directional springs) are used to connect the FRP truss elements to the existing concrete elements (see Figure 2d). A bi-linear constitutive model based on the fracture energy of concrete (G_f) created for the tangential bond-slip relationship between Carbon FRP (i.e., CFRP) and concrete is attributed to the link elements (see Figure 2e), with characteristic points calculated as per Equations 1-4[25,26]. For the FRP fabrics that are completely wrapped around the concrete element, perfect bonding of the fabrics nodes at the edges of the concrete element is considered (see Figure 2b). Similarly, wrapped fabrics also confine the longitudinal fabrics and provide an effective anchorage to help avoid de-bonding of the longitudinal fabrics[24,27]. Thus, the nodes of the fabrics at the anchorage regions are also perfectly bonded to the concrete. Perfect bond is modeled by specifying a high maximum bond stress for the link elements.

$$\tau_{bFy} = (54f_c')^{0.19} \leq f_r = 0.6(f_c')^{0.5} \tag{1}$$

$$G_f = \left(\tau_{bFy}/6.6\right)^2 \tag{2}$$

$$s_{Fy} = 0.057G_f^{0.5} \tag{3}$$

$$s_{Fu} = 2G_f/\tau_{bFy} \tag{4}$$

where τ_{bFy} is the maximum bond stress in MPa, f'_c is the concrete compressive strength in MPa, f_r is the modulus of rupture of the concrete in MPa, G_f is the fracture energy in N/mm, s_{Fy} is the slip at the maximum bond stress in mm, and s_{Fu} is the slip at the ultimate bond stress (i.e., zero stress) in mm.
When the FRP fabrics are wrapped around the concrete element, they provide confinement to the concrete beam. The confinement is accounted for using a smeared FRP reinforcement component in the out-of-plane direction (referred as z-direction) of the concrete elements at the edges of the beam that are wrapped by the FRP fabrics (see Figure 2b), as per Equation 5.

$$f_{c3} = -f_{sz}\rho_z \tag{5}$$

where f_{c3} is the resulting confining pressure, f_{sz} is the stress in the out-of-plane reinforcement, and ρ_z is the out-of-plane reinforcement ratio.

System-Level Capacity Determination
To determine the structural capacity of the cap beam, a pushover analysis, where the finite element model is subjected to a monotonically increasing load up to the structural failure, is performed. Three loading procedures can be used, depending on the objective of the analysis:

The *first procedure* is used to assess the structural capacity of a non-existing cap beam, the pushover analysis is conducted from no load up to the maximum capacity of the structure, following the Strength I ultimate load combination as per the AASHTO[10] specifications of 1.25 x (Dead Load) + 1.75 x (Live Load). The *second procedure* is used when assessing the capacity of an existing cap beam, the pushover analysis is first conducted up to the Strength I ultimate load combination. Then, only the factored live load (LL) is continued to increase up to the structural failure. This loading procedure results in a more realistic assessment since the dead load (DL) that acts on the cap beam (i.e., the cap beam's own weight and bridge superstructure) is not expected to increase. The *third procedure* is used when analyzing the retrofitted structure, the FRP fabrics do not contribute to the original dead load that acts on the beam. Thus, a more realistic procedure is employed: the model is first loaded up to 100% factored dead load and no live load (i.e., 1.25DL + 0LL) with the retrofit elements turned off. From this point on, the retrofit elements are activated, and the dead load is kept constant while the factored live load (i.e., 1.75LL) is progressively increased up to the structural failure.

A global capacity factor method is preferred when calculating the design resistance of a member using NLFEA because nonlinear finite element constitutive models are highly sensitive to the material properties input values, particularly to the concrete strength (f'_c) and the reinforcement yield stress (f_y). Thus, the use of material resistance factors can artificially influence the response of the beam and may even change the failure mode. A full probabilistic analysis that considers the random distribution of the input parameters (i.e., material strengths) is considered the 'ultimate tool' for numerical performance assessments. However, such an approach would require several analyses (between 32 and 64[28]), which is not feasible for practical applications. In the proposed analysis methodology, the global capacity factor method proposed by Cervenka[28] is used. Cervenka studied different methods to calculate the design resistance of nonlinear analysis models and concluded that the estimate of the coefficient of variation method (ECOV), using only two analyses, yields results that are consistent with the full probabilistic method[28]. In the ECOV method, a global capacity factor (γ_G) is probabilistically obtained based on the coefficient of variation of the resistance (V_R) (see Equation 6), which is estimated based on the resistance of the structure using its characteristic (R_k) and mean (R_m) properties of materials, as defined by Equation 7. The design resistance is obtained from the mean resistance (R_m) and the calculated global capacity factor, as shown in Equation 8.

$$\gamma_G = \exp(\alpha_R \beta V_R) \tag{6}$$

$$V_R = \frac{1}{1.65}\ln\left(\frac{R_m}{R_k}\right) \tag{7}$$

$$R_d = \frac{R_m}{\gamma_G} \tag{8}$$

where α_R is the sensitivity factor for the resistance reliability, β is the reliability index, and R_d is the design resistance of the model. For a structural service life of 50 years, the recommended values of α_R and β are 0.8 and 3.8[29], respectively, for the ultimate limit state condition. For a service life of 75 years, α_R and β are 0.8 and 3.2, respectively. Similarly, AASHTO[10] recommends a reliability index of 3.5 for bridges. In this study, the reduction factor is calculated considering the service life of 50 years. As such, the global factor can be calculated using Equation 9.

$$\gamma_G = \exp(3.04V_R) \tag{9}$$

The mean material properties of the reinforcing steel and concrete strengths can be calculated using Equations 10 and 11[30]. Since there is a lack of studies that indicate the mean tensile strength of FRP fabrics, this study used 25 technical sheets of different FRP fabrics manufacturer (15 of CFRP and 10 of GFRP) to obtain this factor for FRP fabrics. The factor for CFRP fabrics was calculated to be 1.18, which was slightly lower than the 1.20 factor for GFRP fabrics (see Equation 12). The mean bonding properties are inherently accounted for by the consideration of the mean concrete properties (see Equations 1-4).

$$f_{ym} = 1.1f_{yk} \tag{10}$$

$$f_{cm} = 1.1\left(\frac{\gamma_s}{\gamma_c}\right)f_{ck} \tag{11}$$

$$f_{tm} = 1.18\sim1.20f_{tk} \tag{12}$$

where f_{yk} and f_{ck} are the characteristic material properties for the reinforcing steel and concrete, respectively; γ_s and γ_c are the partial factors for materials for the ultimate limit states; and f_{tk} is the characteristic tensile strength of the FRP.

VERIFICATION OF THE PROPOSED MODELING APPROACH

The accuracy of the proposed material modeling approach was verified using two simply-supported beams experimentally retrofitted with CFRP fabrics: one with continuum CFRP U-wrap fabrics for shear strengthening[31] (see Figure 3a); and another with longitudinal CFRP fabrics for flexural strengthening anchored by U-wrapped fabrics[32] (see Figure 4a). The first specimen (originally referred to as SO3-4) was used to verify the bond-slip constitutive models (i.e., Equations 1-4) and the confinement effect of the fabrics (i.e., Equation 5). The second specimen (originally referenced as B70PW) was used to verify the bonding of the flexural FRP fabrics due to the provided anchorage fabrics.

The details of the experimental setup of each reinforced concrete beam are discussed elsewhere[31,32]. In short, the material properties experimentally reported and used in the NLFEA discussed herein were, for the SO3-4 beam[31]: concrete strength of 4 ksi (27.5 MPa), reinforcing steel modulus of elasticity, yield stress, and ultimate stress of 29000 ksi (200 GPa), 67 ksi (460 MPa), and 106 ksi (730 MPa), respectively, and CFRP modulus of elasticity and tensile strength of 33000 ksi (228 GPa) and 550 ksi (3790 MPa), respectively; and for the B70PW beam[32]: average concrete strength of 8 ksi (54 MPa), steel reinforcement modulus of elasticity and yielding strength of 29300 ksi (202 GPa) and 89 ksi (611 MPa), respectively, and CFRP modulus of elasticity and tensile strength of 31200 ksi (215 GPa) and 363 ksi (2500 MPa). Figures 3 and 4 presents the experimental setup, the created finite element model, and the beam deformations at failure for each specimen. Because U-wrap CFRP fabrics were used, only the nodes at the bottom edge of the beams were modeled as perfectly bonded. Similarly, the out-of-plane confinement reinforcement was modeled only for the concrete elements wrapped in the fabrics at the bottom edge of the beams (see Figures 3 and 4).

Figure 5 shows the load-deflection response experimentally obtained and numerically calculated by the created finite element model. The peak load, peak displacement and overall stiffness response of both beams were well captured by the finite element model. The calculated-to-experimental ratios (i.e., $1\text{-}P_{cal}/P_{exp}$) of the peak load capacity were -2.5% and 5.9% for the SO3-4 and the B70PW specimens, respectively. For the peak displacement, the calculated-to-experimental rations were 32.9% and 2.9% for the SO3-4 and the B70PW specimens, respectively. It is believed that the difference in peak displacement in the SO3-4 beam, despite its good overall response, was due to differences in the experimentally reported and actual material properties, which resulted in a slight stiffness deviation. The failure mode of the SO3-4 beam was experimentally reported to be the de-bonding of the CFRP U-wrap fabrics at a load of 65 kips (289 kN)[31]. The finite element model successfully calculated the failure mode as de-bonding of the CFRP fabrics starting at a load of 64 kips (285 kN) at the shear-critical span (see Figure 3c). The criteria used to identify de-