

An ACI Technical Publication

SYMPOSIUM VOLUME



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

SP-333

Editors:

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



American Concrete Institute
Always advancing

AVA Documents
Document Sharing Hub

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

Sponsored by
ACI Committees 342, 343, and 345

The Concrete Convention and Exposition
March 25-29, 2018
Salt Lake City, UT, USA

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



American Concrete Institute
Always advancing

SP-333

First printing, October 2019

Discussion is welcomed for all materials published in this issue and will appear ten months from this journal's date if the discussion is received within four months of the paper's print publication. Discussion of material received after specified dates will be considered individually for publication or private response. ACI Standards published in ACI Journals for public comment have discussion due dates printed with the Standard.

The Institute is not responsible for the statements or opinions expressed in its publications. Institute publications are not able to, nor intended to, supplant individual training, responsibility, or judgment of the user, or the supplier, of the information presented.

The papers in this volume have been reviewed under Institute publication procedures by individuals expert in the subject areas of the papers.

Copyright © 2019
AMERICAN CONCRETE INSTITUTE
38800 Country Club Dr.
Farmington Hills, Michigan 48331

All rights reserved, including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by any electronic or mechanical device, printed or written or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.

Printed in the United States of America

Editorial production: Susan K. Esper

ISBN-13: 978-1-64195-078-7

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.

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

University of Colorado Denver, USA
Missouri University of Science and Technology, USA
University of Miami, USA

TABLE OF CONTENTS

SP-333-1:

A Numerical Analysis Methodology for the Strengthening of Deep Cap Beams 1-18

Authors: Rafael A. Salgado and Serhan Guner

SP-333-2:

Surrogate Modeling for Self-Consolidating Concrete Characteristics Estimation
for Efficient Prestressed Bridge Construction 19-39

Authors: Junwon Seo and Jharna Pokhrel

SP-333-3:

On the Application of Basalt-Fiber Reinforced Polymer (BFRP) Bars to Prestressed
Slab Elements Typical of the Precast Concrete Industry 40-59

Authors: Bruno Dal Lago, Davide Bisi and Liberato Ferrara

SP-333-4:

Monitoring and Rehabilitation of Damaged Bridge Beam of Middle Ring Road
in Shanghai 60-79

Authors: Huayong Wu, Rongxin Zhao, and Yun Xing

SP-333-5:

Seismic Experiments and Analysis of Repaired Bridge Columns Using
CFRP Donut 80-95

Authors: Ruo-Yang Wu and Chris P. Pantelides

SP-333-6:

Pedestrian Bridge as Clarifying Example of FRP-RC/PC Design 96-118

Authors: Marco Rossini, Saverio Spadea, Antonio Nanni

SP-333-7:

Shear Tests on Prestressed Concrete Continuous Beams 119-135

Authors: Martin Herbrand, Viviane Adam, Josef Hegger

SP-333-8:

Concrete Beams with Fully Corroded Steel Repaired with CFRP Laminates 136-158

Authors: Needa Lingga, Yasir Saeed, Anas Yosefani, and Franz Rad

SP-333-9:

Rational Fuzzy Logic Condition Rating Model of Reinforced Concrete
Bridge Decks Using Nondestructive Testing and Visual Inspection 159-174

Authors: Tarek Omar and Moncef L. Nehdi

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

ACI member **Rafael A. Salgado** is a Ph.D. student at the University of Toledo. He received his BSCE degree in 2015 from the Universidade Federal do Espírito Santo (Brazil). His research interests include finite element analysis, including nonlinear and dynamic analyses of structures.

ACI member **Serhan Guner** is an Assistant Professor in the Department of Civil Engineering at the University of Toledo, Toledo, OH. He received his Ph.D. from the University of Toronto, Toronto, ON, Canada. He is a member of Joint ACI-ASCE Committee 447, Finite Element Analysis of Reinforced Concrete Structures. His research interests include finite element modeling of reinforced concrete structures, shear effects in concrete, development of analysis software, and structural response to impact, blast, and seismic loads.

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 VecTor2⁷. 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 Model⁸, which is a refined version of the Modified Compression Field Theory⁹ (MCFT), a theory that has been recognized and adopted by the AASHTO¹⁰ and CSA A23.3⁴ 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 model⁷. 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 elsewhere⁷. The concrete model includes nonlinear hysteresis rules for the unloading and reloading conditions⁷ (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 Seckin¹⁸.

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 fabrics²⁴. 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.

$$\tau_{bFy} = (54f'_c)^{0.19} \leq f_r = 0.6(f'_c)^{0.5} \quad (1)$$

$$G_f = (\tau_{bFy}/6.6)^2 \quad (2)$$

$$s_{Fy} = 0.057G_f^{0.5} \quad (3)$$

$$s_{Fu} = 2G_f/\tau_{bFy} \quad (4)$$

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.

$$f_{c3} = -f_{sz}\rho_z \quad (5)$$

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 AASHTO¹⁰ 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 64²⁸), which is not feasible for practical applications. In the proposed analysis methodology, the global capacity factor method proposed by Cervenka²⁸ 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 method²⁸. 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.

$$\gamma_G = \exp(\alpha_R \beta V_R) \quad (6)$$

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

$$R_d = \frac{R_m}{\gamma_G} \quad (8)$$

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 3.8²⁹, 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, AASHTO¹⁰ 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.

$$\gamma_G = \exp(3.04V_R) \quad (9)$$

The mean material properties of the reinforcing steel and concrete strengths can be calculated using Equations 10 and 11³⁰. 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).

$$f_{ym} = 1.1f_{yk} \quad (10)$$

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

$$f_{tm} = 1.18 \sim 1.20 f_{tk} \quad (12)$$

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 strengthening³¹ (see Figure 3a); and another with longitudinal CFRP fabrics for flexural strengthening anchored by U-wrapped fabrics³² (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 beam³¹: 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 beam³²: 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., $1-P_{cal}/P_{exp}$) 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 ratios 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-

bonding on the beams were based on the relative displacements between the CFRP fabrics and the concrete exceeding the slip at the maximum bond stress, after which bonding stresses decrease (i.e., as shown in Figure 2e). For the B70PW beam, the experimentally reported failure mode was a shear-tension failure with the initial flexural-shear cracks followed by the de-bonding of the flexural CFRP fabrics due to splitting cracks in the concrete³². The calculated failure mode of the beam captured the experimental response successfully as shown in Figure 4c, with splitting cracks at the bottom part of the beam that caused the de-bonding of the flexural CFRP reinforcement.

SYSTEM-LEVEL VERIFICATION OF THE NLFEA APPROACH

The proposed NLFEA approach was verified using a cap beam of an existing overpass structure (see Figure 6). Cross-sectional and a strut-and-tie model (STM) analyses calculated the cap beam to be overloaded. Thus, an NLFEA following the proposed modeling methodology was employed to calculate an accurate loading capacity of the cap beam.

The finite element model was developed in VecTor2, as shown in Figure 7, with the cross-sectional dimensions, reinforcement layout, beam configuration, and unfactored loading condition shown in Figure 6. The concrete compressive strength and steel reinforcement yield stress were reported on the original design drawings as 4 ksi (27.6 MPa) and 40 ksi (275 MPa), respectively. The geometric symmetry of the beam allowed for a half-model of the cap beam, which significantly reduced the numerical model size and lowered the modeling efforts. The support conditions applied included rollers on the axis of symmetry and pins at the lowermost ends of the pier columns (not shown in Figure 7).

Three of the considered second-order models (see Table 1) were found to be particularly important for the cap beam examined: the concrete compression softening (i.e., the reduction in the uniaxial compressive strength and stiffness due to transverse tensile cracking), the concrete tension stiffening (i.e., the ability of cracked reinforced concrete to transmit tensile stresses across cracks), and the dowel action (i.e., the additional shear strength provided by the main reinforcing bars). The very low amounts of stirrup reinforcement present in the cap beam make it prone to shear cracking, which reduces the effectiveness of the concrete struts and, thus, requires the consideration of the ‘concrete compression softening’. The cap beam is also prone to flexural cracking due to the lack of well-distributed layers of reinforcement, and thus its response is sensitive to the amount of tension transmitted across cracks, requiring the modeling of the ‘concrete tension stiffening’ effects. Finally, the low amount of stirrups reduce the shear capacity of the beam, such that the additional shear resistance due to the ‘dowel action’ becomes important.

The pushover loading procedure was performed for the assessment of existing cap beams (i.e., as the second proposed pushover loading method, see the “System-Level Capacity Determination” section). To obtain the design resistance of the cap beam, two analyses were performed: one with characteristic and one with mean properties of materials (as discussed in the “System-Level Capacity Determination” section). The mean values of the reinforcing steel and concrete strengths were calculated, using Equations 10 and 11, to be 44 ksi (275 MPa) and 3.32 ksi (22.9 MPa). For brevity, only the analysis results using the characteristic material properties are shown in this paper.

The characteristic pushover analysis calculated a maximum load capacity of 3447 kips (15333.2 kN), which represented approximately 90% of the ultimate load combination. At the failure condition (see Figure 8), an extensive shear and flexural cracking pattern were calculated at the cantilever span, which indicated the formation of the deep beam strut-action through shear cracks spanning from the point loads to the pier supports. The yielding of both top and bottom flexural reinforcement of the cap beam at the cantilever span caused the crushing of the concrete (i.e., high compressive strains) at the beam-column interface, contributing significantly to the propagation of the cracks (i.e., vertical cracks), resulting in a flexure-shear failure of the cap beam.

To obtain the system-level load capacity, the two performed pushover analyses were combined using Equations 7 and 9. The applied force versus displacement of the cantilever and inner span of the beams, for both analyses, are shown in Figure 9. The global capacity factor was calculated to be 1.10 and the design capacity of the cap beam was calculated, as per Equation 8, to be 3178 kips (14135 kN). Consequently, the un-retrofitted cap beam was found to be 17% overloaded. Thus, a retrofit study of the cap beam shall be performed to guarantee that the load resistance of the retrofitted cap beam surpasses its ultimate load demand.

Retrofit of the cap beam using externally bonded FRP

A suitable FRP retrofit layout needs to be developed based on the failure mechanisms developed on the un-retrofitted structure. When studying an effective FRP retrofit layout, it is important to ensure that the new cap beam is capable not only to resist its un-retrofitted failure mechanisms but also to be able to resist new failure modes that can occur due to the redistribution of stresses caused by the added FRP fabrics. The critical failure modes for externally bonded FRP retrofitted structures are: concrete shear, concrete flexure, concrete compression (i.e., crushing), fabric debonding, and fabric rupture. A general FRP retrofit layout that covers the critical failure modes is proposed. As shown in Figure 10, this layout includes fabrics to increase shear capacity that are completely wrapped (i.e., black fabrics on Figure 10) and U-wrapped (i.e., gray fabrics on Figure 10) around the concrete beam, and longitudinal fabrics that are bonded to the top and bottom of the beam to increase the flexural capacity (i.e., blue and green fabrics, respectively, on Figure 10). The completely wrapped FRP fabrics also provide effective anchorage to the longitudinal fabrics and confinement effects to the edges of the concrete beam. Consequently, besides its effective flexural and shear retrofit, this layout also improves the compressive capacity of the concrete (i.e., increasing its crushing resistance) and the bond-slip mechanism of the fabrics.

To determine the amount of FRP fabrics necessary to strengthen the beam (i.e., thickness and width of each fabric section of the FRP retrofit layout), different numerical models can be created (i.e., each one with a proposed amount of FRP fabrics) until the NLFEA calculates a safe structural condition under the ultimate loading condition. This process allows to effectively visualize the contribution and importance of each section of the FRP retrofit layout to the deficient structure. For conciseness, the fabric amounts for the final retrofit layout implemented on the studied cap beam is presented in Figure 10. The top longitudinal FRP fabrics were separated in two sections due to the locations of the bearing plates and some corners of the concrete beam were smoothed to help the fabrics application (see Figure 10). Similarly, the completely wrapped FRP fabrics were substituted by U-wrapped due to the presence of the bearing plates.

Once the externally bonded FRP layout was determined, it was implemented in the NLFEA model of the un-retrofitted structure (see Figure 11). The proposed FRP finite element material modeling approach was essential to ensure an accurate structural response and assessment of the additional capacity of the cap beam. In addition, the material modeling approach also modeled the critical failure modes (as discussed above): the concrete flexural and shear failure modes are covered by the employed concrete constitutive models; the compressive failure mode is covered by the concrete and FRP confinement effect models; the de-bonding is covered by the presented bond-slip relationship and the discussed perfect bonded regions; and the fabrics rupture is covered by the employed linear-elastic FRP material model. Thus, the determined effective FRP retrofit layout could be accurately incorporated in the NLFEA modeling approach, which makes it a suitable analysis procedure for accurate responses calculation.

The CFRP fabrics used had a modulus of elasticity, tensile strength, and thickness of 8200 ksi (56.5 GPa), 105 ksi (724 MPa), and 0.02 in. (0.51 mm), respectively. Using Equations 1-4, the calculated bond-slip properties of maximum bond stress, the slip at the maximum bond stress, and the slip at the ultimate bond stress were, 0.4 ksi (2.87 MPa), 9.83×10^{-4} in. (0.025 mm), and 5.20×10^{-3} in. (0.132 mm), respectively. Figure 11 shows the numerical model of the cap beam with the applied CFRP fabrics. Following the proposed FRP modeling approach, the CFRP fabrics were modeled with truss elements using the effective area of each truss and the bond-slip relationship. The wrapped fabrics (i.e., red truss elements in Figure 11) were perfectly bonded on the top and bottom edge of the concrete beam due to their complete wrapping (except over the bearing plate at the inner span). The bottom flexural fabrics were modeled as perfectly bonded due to the anchorage provided by the wrapped CFRP fabrics, and the top flexural fabrics were perfectly bonded at the anchorage regions. Out-of-plane confinement reinforcement was added to the concrete elements at the edges where the fabrics are wrapped.

For the system-level load capacity determination, the pushover loading procedure was performed following the method for the assessment of retrofitted structures (i.e., the third proposed pushover loading method, see the “System-Level Capacity Determination” section). The characteristic pushover analysis of the retrofitted structure showed an improvement in structural performance when compared to the same loading condition that caused the failure of the un-retrofitted cap beam (see Figure 12). Besides reducing the shear and flexural cracking condition at the cantilever and the inner span of the cap beam, the FRP retrofit layout also lowered the bottom reinforcement stress state and, in consequence, the concrete compressive strains at the cantilever-column interface.

The characteristic pushover analysis of the retrofitted cap beam calculated a maximum capacity of 4050 kips (18018 kN). At the structural failure condition (exaggerated in Figure 13), extensive shear and flexural cracking patterns were

observed. The high shear stresses on the cantilever span caused the FRP fabrics to de-bond following the main shear crack pattern (see Figure 13). The top and bottom reinforcing steel yielded and high compressive strains (i.e., concrete crushing) developed at the cantilever-column interface. The flexural capacity of the cantilever section relied mainly on the top longitudinal FRP fabrics, which, due to the high-stress demand, de-bonded through the split cracking of the adherent concrete, causing a flexure-shear failure of the cap beam.

Using Equations 7 and 9 with the results of the two analyses performed with the retrofitted finite element model (i.e., one with characteristic properties of material and one with mean properties of material), the calculated global capacity factor was 1.00 (both analyses calculated the same resistance) and the system-level load capacity of the retrofitted cap beam was determined to be 4050 kips (18018 kN). Figure 14 compares the characteristic pushover analysis responses of the un-retrofitted and retrofitted structure. In Figure 14, the last data point of each curve (i.e., the failure load and displacement) was selected for the loading condition at which the failure mechanisms that developed in each beam (and were described above) occurred. After this loading condition, the subsequent load stages of the numerical model were no longer representative of the real structural behavior (excessive displacements, zero-stresses, etc.) since failure mechanisms had already developed. The curves in Figure 14 do not present a strength peak (and subsequent loss of strength) due to the force-based nature of the numerical analysis performed. The calculated load capacity represented an increase of 27% in strength, when compared to the un-retrofitted cap beam (3178 kips or 14135 kN) and indicated an extra capacity of 6% over the ultimate loading condition. These results corroborate the benefits of the determined CFRP retrofit layout and the effectiveness of the modeling approach. As a result, the proposed methodology was successful in providing an effective FRP retrofit to the overloaded cap beam.

SUMMARY AND CONCLUSIONS

Deep bridge cap beams retrofitted with fiber reinforced polymers require special analysis methods in order to effectively account for behaviors that are characteristic of deep elements (such as the nonlinearity of the strain distribution and the effects of cracking on the stress distribution) and retrofit-related mechanisms (such as bond-slip relationships, confinement effects, and redistribution of stresses are not considered). Despite some equations given by available retrofit codes to calculate the extra capacity that FRP fabrics might produce on a general concrete element, no provisions account for the aforementioned behaviors that directly affect deep cap beams retrofitted with FRP fabrics. This study proposed an analysis methodology for deep cap beams retrofitted with externally bonded FRP fabrics. Details were given regarding the constitutive modeling of the critical behavior of materials, which were verified using experimental results available in the literature. Finally, to exemplify the application of the proposed methodology, a real/large structure was analyzed, and an effective retrofit solution was calculated based on the calculated beam response. The results of this study support the following conclusions:

1. The proposed analysis methodology presented a comprehensive set of material behaviors and numerical modeling formulations that are essential for the analysis of deep cap beam elements. Three different loading approaches were defined to obtain a more accurate response of cap beams depending on the state of the bridge, i.e., non-existing bridge, existing bridge, retrofitted bridge. Finally, the proposed methodology presented a global capacity factor procedure based on probabilistic fundamentals for the determination of the design resistance of a cap beam.
2. The proposed methodology and its constitutive modeling approaches were successfully verified using two different experimental studies from the literature. The overall load-displacement deviation between the experimental results and the created methodology was calculated to be within 6%. In addition, the verification studies successfully verified the proposed FRP-concrete composite structural behavior, including the bond-slip constitutive model, concrete confinement effects caused by the fabric wrap, and the perfectly bonded conditions of longitudinal fabrics anchored by U-wrapped fabrics. As a result, the numerical model created using the proposed methodology was able to capture the experimentally observed failure modes.
3. An existing deep cap beam structure was analyzed to illustrate the benefits and in-depth information provided by the proposed methodology. The analyzed cap beam was calculated to be 17% overloaded considering the design ultimate load condition. The calculated response of the cap beam confirmed the deep beam action occurring on the beam through the calculated crack pattern, which also helped identify the most critical spans of the beam, the type of failure (i.e., flexure, shear, or flexure-shear), and critical effects that should be considered in the retrofit design.
4. The detailed damage pattern and failure mechanism calculated by the numerical model of the un-retrofitted cap beam were essential in the determination of an efficient distribution of the FRP fabric throughout the analyzed cap beam to capture the critical failure mechanisms. Such a design approach differs significantly from the use of code provisions, in which simplified equations are given to obtain the added capacity of the member due to the usage of

FRP fabrics retrofit while neglecting important behaviors such as the deep beam effects, confinement effects, and bond-slip interaction.

5. The information obtained from the un-retrofitted numerical model was used to determine a CFRP retrofit layout on the overloaded cap beam. The proposed methodology was again used to create a retrofitted numerical model of the analyzed cap beam, which enabled an increase in the load carrying capacity of the cap beam by 27%, allowed it to be safe under its ultimate loading condition, and developed an additional 6% extra capacity.

6. The general FRP retrofit layout configuration used in the studied cap beam (i.e., position and distribution of fabrics, but not their quantities) could be applied for a general cap beam as long as its failure conditions follow the trend observed in the structure analyzed in this paper. The number of fabric's layers and material properties on each section of the FRP retrofit layout can be studied to result in the desired structural performance.

ACKNOWLEDGMENTS

The authors would like to thank the Ohio Department of Transportation for providing the structural design drawings and the contributions of Dr. Douglas K. Nims and Mr. John R. Morganstern for extracting the main structural details and participating in several meetings to discuss the various aspects of the bridge.

REFERENCES

- [1]. American Concrete Institute, "Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars (ACI 440.1R-15)," American Concrete Institute (ACI), Farmington Hills, MI, 2015, 88 pp.
- [2]. Canadian Standards Association / National Standard of Canada, "Canadian Highway Bridge Design Code (CAN/CSA S6-14)," 11th ed., CSA Group, Mississauga, Ontario, Canada, 2014, 894 pp.
- [3]. American Concrete Institute, "Building Code Requirements for Structural Concrete and Commentary (ACI 318-19)," American Concrete Institute, Farmington Hills, MI, 2019, 624 pp.
- [4]. Canadian Standards Association, "Design of Concrete Structures (CSA A23.3-14)," 6th ed., CSA Group, Mississauga, Ontario, Canada, 2014, 290 pp.
- [5]. Guner, S.; Vecchio, F. J., "Pushover Analysis of Shear-Critical Frames: Formulation," *ACI Structural Journal*, V. 107, No. 1, 2010, 63-71 pp., [http://www.utoledo.edu/engineering/faculty/serhan-guner/docs/JP1-Guner and Vecchio.pdf](http://www.utoledo.edu/engineering/faculty/serhan-guner/docs/JP1-Guner%20and%20Vecchio.pdf).
- [6]. Guner, S.; Vecchio, F. J., "Pushover Analysis of Shear-Critical Frames: Verification and Application," *ACI Structural Journal*, V. 107, No. 1, 2010, 72-81 pp., http://www.utoledo.edu/engineering/faculty/serhan-guner/docs/JP2_Guner_Vecchio_2010b.pdf.
- [7]. Wong, P. S.; Vecchio, F. J.; Trommels, H., "Vector2 & Formworks User's Manual," 2nd ed., 318 pp., http://www.vectoranalysisgroup.com/user_manuals/manual1.pdf.
- [8]. Vecchio, F. J., "Disturbed Stress Field Model for Reinforced Concrete: Formulation," *Journal of Structural Engineering*, V. September, 2000, 1070-1077 pp., http://www.vectoranalysisgroup.com/journal_publications/jp34.pdf.
- [9]. Vecchio, F. J.; Collins, M. P., "The Modified Compression-Field Theory for Reinforced Concrete Elements Subjected to Shear," *ACI Journal*, V. 83, No. 2, 1986, 219-231 pp., http://www.vectoranalysisgroup.com/journal_publications/jp2.pdf.
- [10]. American Association of State Highway and Transportation Officials. "AASHTO LRFD Bridge Design Specifications," 4th ed., American Association of State Highway and Transportation Officials, Washington, DC, 2017, 566 pp.
- [11]. Hognestad, E., "A Study of Combined Bending and Axial Load in Reinforced Concrete Members," *Bulletin Series* No 399, 1951, 128 pp., <https://www.ideals.illinois.edu/handle/2142/4360%0Ahttp://hdl.handle.net/2142/4360>.
- [12]. Coulomb, C. A., "Essai Sur Une Application des Regles de Maximis et Minimis a Quelques Problemes de Statique Relatifs a l'Architecture (Essay on Maximums and Minimums of Rules to Some Static Problems Relating to Architecture)," *Memoires De La Mathematique Et De Physique*, V. 7, 1973, 343-387 pp.
- [13]. Bunni, N. G.; Scott, B.; Park, R.; Priestley, M., "Stress-Strain Behavior of Concrete Confined by Overlapping Hoops at Low and High Strain Rates," *ACI Journal*, V. 79, No. 6, 1982, 496-498 pp.
- [14]. Vecchio, F. J., "Analysis of Shear-Critical Reinforced Concrete Beams," *ACI Structural Journal*, V. 97, No. 1, 2000, 102-110 pp.
- [15]. Vecchio, F. J., "Finite Element Modeling of Concrete Expansion and Confinement," *Journal of Structural Engineering*, V. 118, No. 9, 1992, 2390-2406 pp.

- [16]. Bentz, E. C., "Explaining the Riddle of Tension Stiffening Models for Shear Panel Experiments," *Journal of Structural Engineering*, V.131, No. 9, 2005, 1422-1425 pp., <http://ascelibrary.org/doi/10.1061/%28ASCE%290733-9445%282005%29131%3A9%281422%29>.
- [17]. Walraven, J. C.; Reinhardt, H. W., "Theory and Experiments on the Mechanical Behaviour of Cracks in Plain and Reinforced Concrete Subjected to Shear Loading," *Heron*, V. 26, No. 1A, 1981, <https://repository.tudelft.nl/islandora/object/uuid:3d68bd1a-465d-4590-b33c-7ede99bbc251/datastream/OBJ/download>.
- [18]. Seckin, M., "Hysteretic Behavior of Cast-In-Place Exterior Beam-Column-Slab Assemblies," *Ph.D. Thesis*, University of Toronto, Toronto, Canada, 1982.
- [19]. Kupfer, H.; Hilsdorf, H., "Behavior of Concrete Under Biaxial Stresses," *ACI Journal*, V. 66, No. 8, 1969, 656-666 pp., <http://www.academia.edu/download/33736623/kupfer.pdf>.
- [20]. Richart, F.; Brandtzaeg, A.; Brown, R. L., "A Study of the Failure of Concrete under Combined Compressive Stresses," *University of Illinois Bulletin*, V. 26, Bullet(12), 1928, 104 pp., <http://www.academia.edu/download/33736623/kupfer.pdf>.
- [21]. He, X. G.; Kwan, A. K. H., "Modeling Dowel Action of Reinforcement Bars for Finite Element Analysis of Concrete Structures," *Computers and Structures*, V. 79, No. 6, 2001, 595-604 pp.
- [22]. Dhakal, R. P; Maekawa, K., "Modeling for Postyield Buckling of Reinforcement," *Journal of Structural Engineering*, V. 128, No. 9, 2002, 1139-1147 pp., <http://ascelibrary.org/doi/10.1061/%28ASCE%290733-9445%282002%29128%3A9%281139%29>.
- [23]. Dhakal, R. P; Maekawa, K., "Reinforcement Stability and Fracture of Cover Concrete in Reinforced Concrete Members," *Journal of Structural Engineering*, V. 128, No. 10, 2002, 1253-1262 pp.
- [24]. Grelle, S. V; Sneed, L .H., "Review of Anchorage Systems for Externally Bonded FRP Laminates," *International Journal of Concrete Structures and Materials*, V. 7, No. 1, 2013, 17-33 pp.
- [25]. Sato, Y.; Vecchio, F. J., "Tension Stiffening and Crack Formation in Reinforced Concrete Members with Fiber-Reinforced Polymer Sheets," *Journal of Structural Engineering*, V. 129, No. 6, 2003, 717-724 pp.
- [26]. Wong, R. S.; Vecchio, F. J., "Towards Modeling of Reinforced Concrete Members with Externally Bonded Fiber-Reinforced Polymer Composites," *ACI Structural Journal*, V. 100, No. 6, 2003, 47-55 pp., http://www.vectoranalysisgroup.com/journal_publications/jp45.pdf.
- [27]. Ceroni, F.; Pecce, M.; Matthys, S.; Taerwe, L., "Debonding Strength and Anchorage Devices for Reinforced Concrete Elements Strengthened with FRP Sheets," *Composites Part B: Engineering*, V. 39, No. 3, 2008, 429-441 pp., doi:10.1016/j.compositesb.2007.05.002
- [28]. Cervenka, V., "Global Safety Format for Nonlinear Calculation of Reinforced Concrete," *Beton- und Stahlbetonbau*, V. 103, No. S1, 2008, 37-42 pp., <http://doi.wiley.com/10.1002/best.200810117>.
- [29]. Sangiorgio, F., "Safety Format for Non-linear Analysis of RC Structures Subjected to Multiple Failure Modes," *Ph.D. Thesis*, KTH Royal Institute of Technology, Stockholm, Sweden. 2015, 45pp.
- [30]. European Committee for Standardization. *Eurocode 2 - Design of Concrete Structures - Concrete Bridges - Design and Detailing Rules (EN 1992-2:2005)*. European Committee for Standardization (CEN), 2005, 95 pp.
- [31]. Khalifa, A.; Nanni, A., "Rehabilitation of Rectangular Supported RC Beams with Deficiencies Using CFRP Composites," *Construction and Building Materials*, V. 16, No. 3, 2002, 135-146 pp.
- [32]. Al-Tamimi, A. K.; Hawileh, R.; Abdalla, J.; Rasheed, H. A., "Effects of Ratio of CFRP Plate Length to Shear Span and End Anchorage on Flexural Behavior of SCC RC Beams," *Journal of Composites for Construction*, V. 15, No. 6, 2011, 908-919 pp.

Table 1 – Material models included in VecTor2

Material behavior	Default model	Material behavior	Default model
Compression base curve	Hognestad parabola ¹¹	Cracking criterion	Mohr-Coulomb (Stress) ¹²
Compression post-peak	Modified Park-Kent ¹³	Crack width check	Max crack width of $\text{Agg}/5$ ¹⁴
Compression softening	Vecchio 1992-A ¹⁵	Concrete hysteresis	Nonlinear w/plastic offsets ⁷
Tension stiffening	Modified Bentz 2003 ¹⁶	Slip distortion	Walraven ¹⁷
Tension softening	Linear ⁷	Rebar hysteresis	Seckin w/Bauschinger ¹⁸
Confinement strength	Kupfer/Richart ^{19,20}	Rebar dowel action	Tassios (Crack slip) ²¹
Concrete dilatation	Variable - Orthotropic ¹⁹	Rebar buckling	Refined Dhakal-Maekawa ^{22,23}

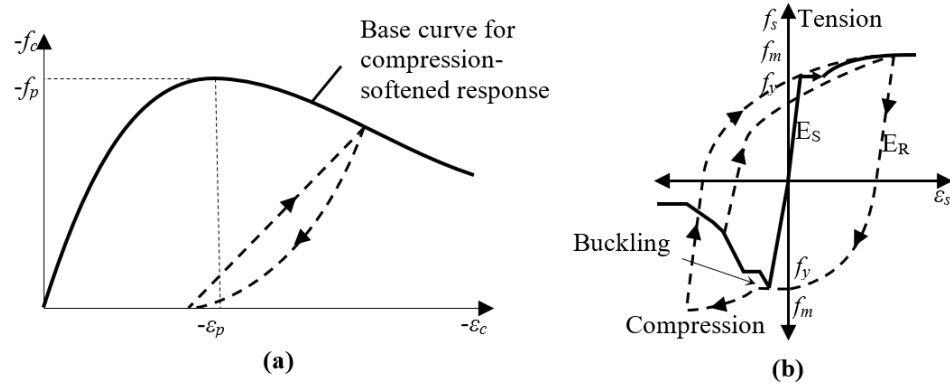


Figure 1 – (a) Concrete and (b) reinforcing steel material constitutive models.

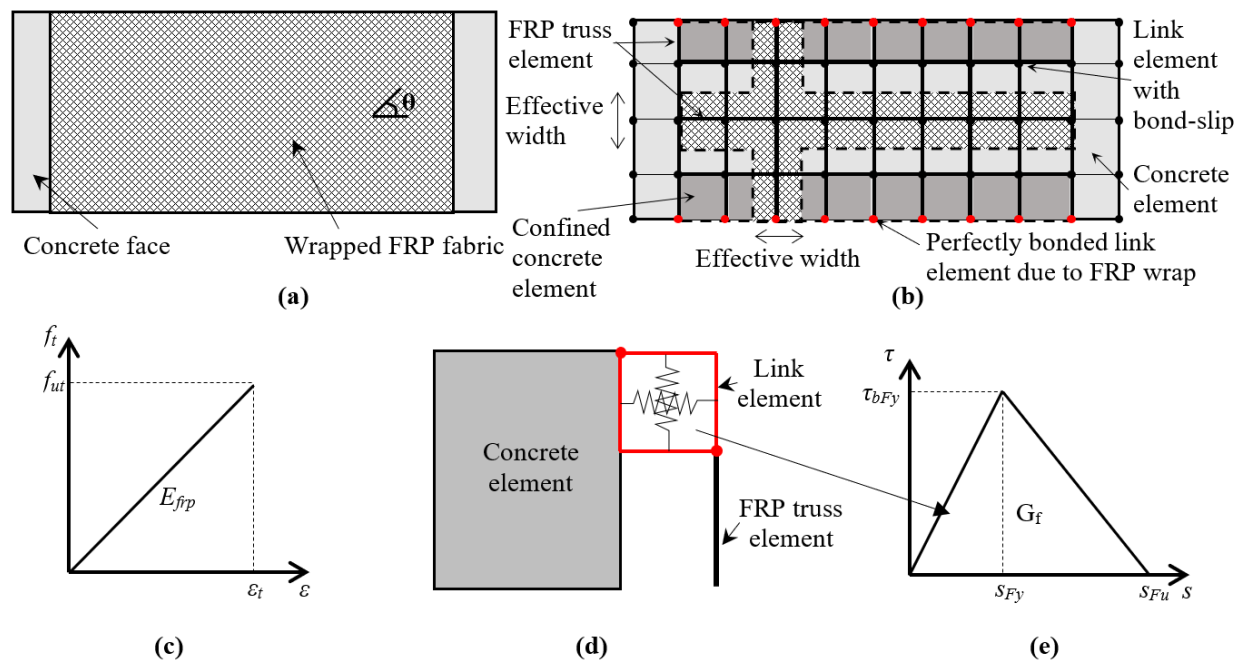
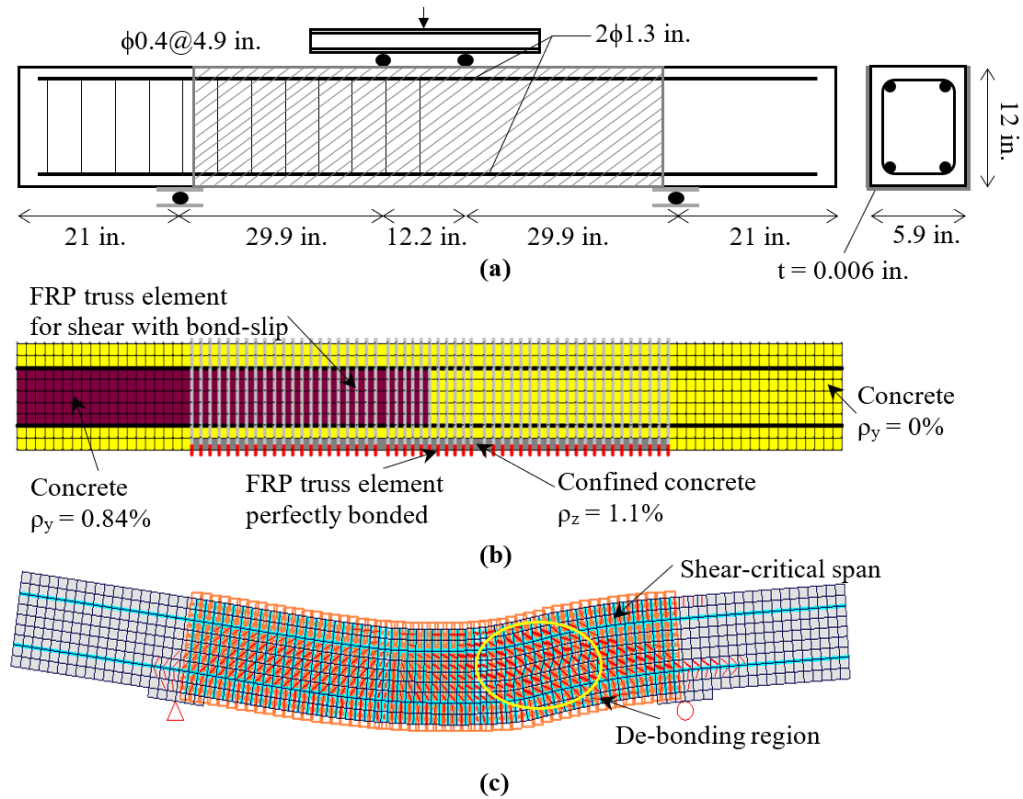
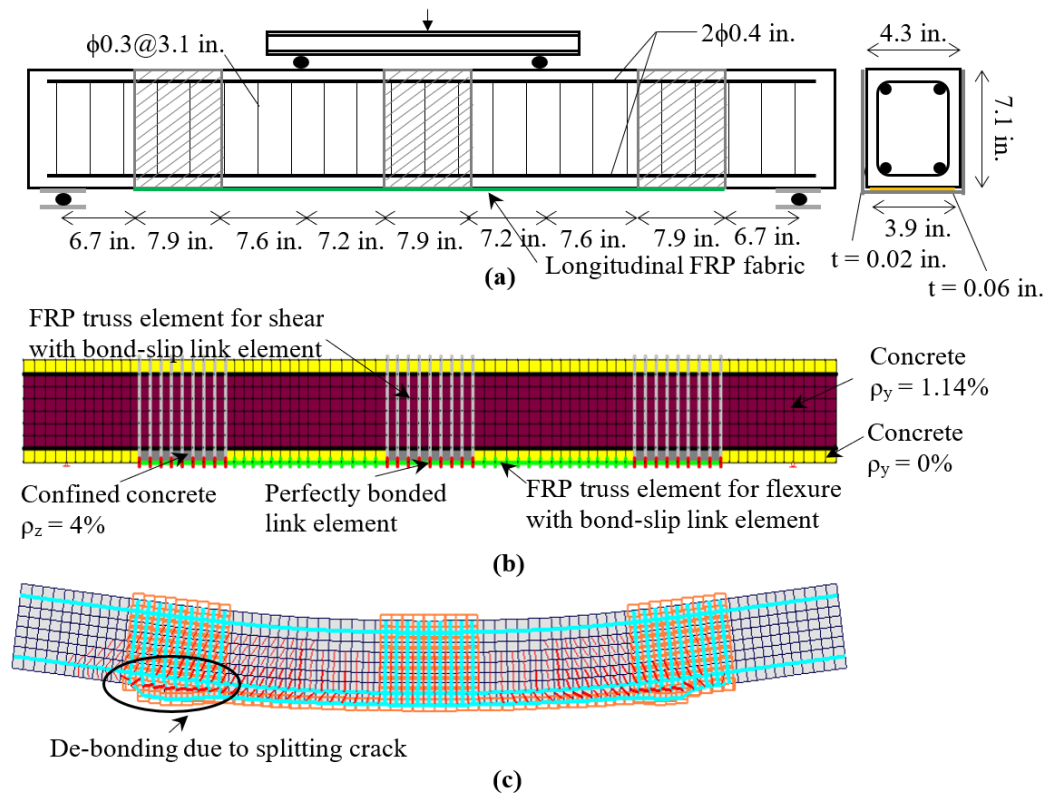


Figure 2 – (a) FRP fabric wrapped around the concrete element, (b) finite element modeling of FRP fabrics, (c) FRP constitutive model, (d) detail of link element between concrete and FRP fabric, and (e) bond-slip constitutive model.



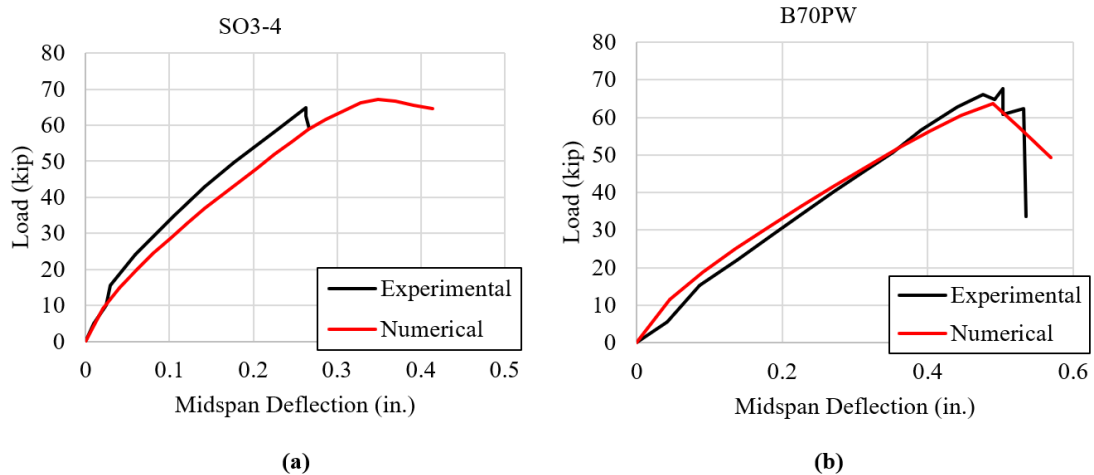
NOTE: 1 in. = 25.4 mm.

Figure 3 – (a) SO3-4 experimental setup, (b) finite element model, and (c) deflected shape at failure condition.



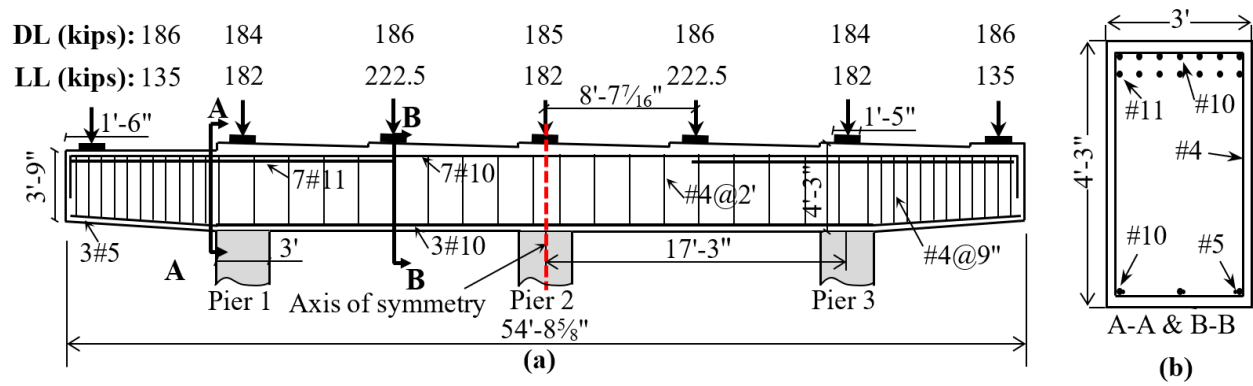
NOTE: 1 in. = 25.4 mm.

Figure 4 – (a) B70PW experimental setup, (b) finite element model, and (c) deflected shape at failure condition.



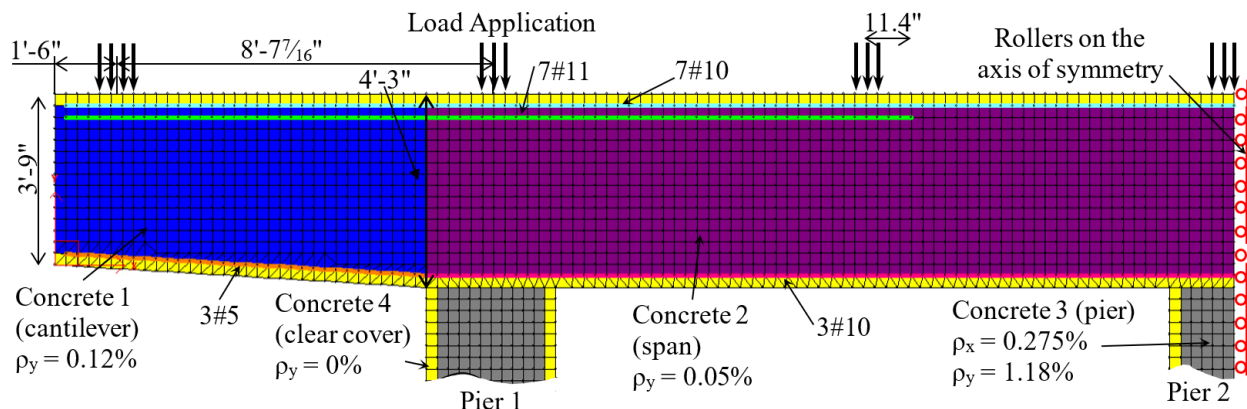
NOTE: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

Figure 5 – (a) Cantilever and (b) inner span total load versus displacement response.



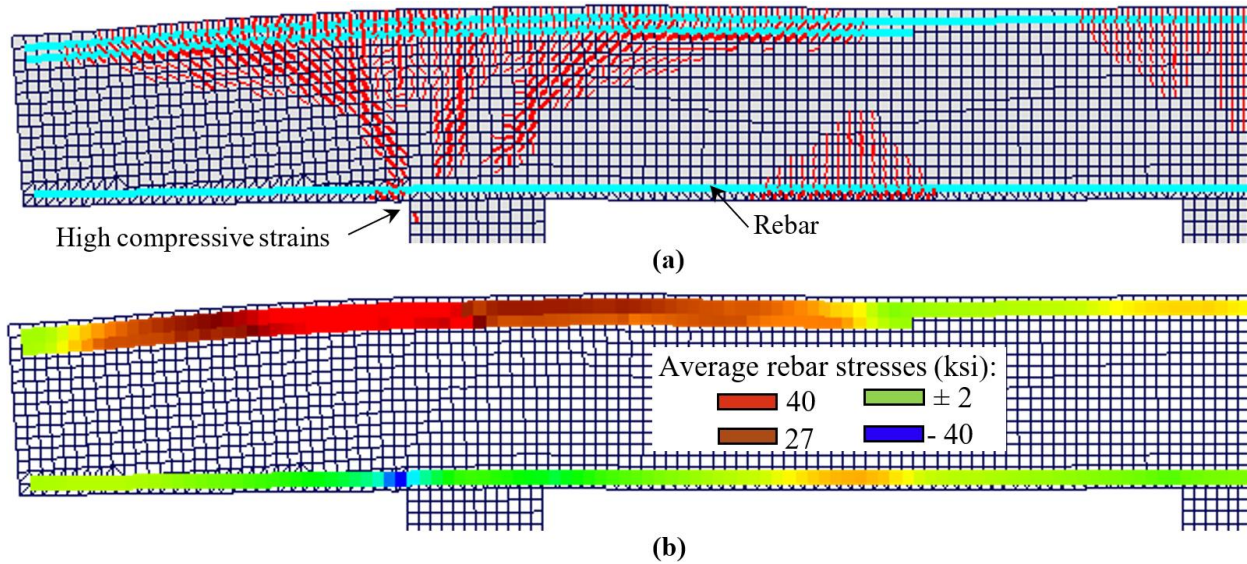
NOTE: 1 in. = 25.4 mm; 1 ft. = 0.305 m; 1 kip = 4.45 kN

Figure 6 – Cap beam examined: (a) elevation and (b) cross-section.



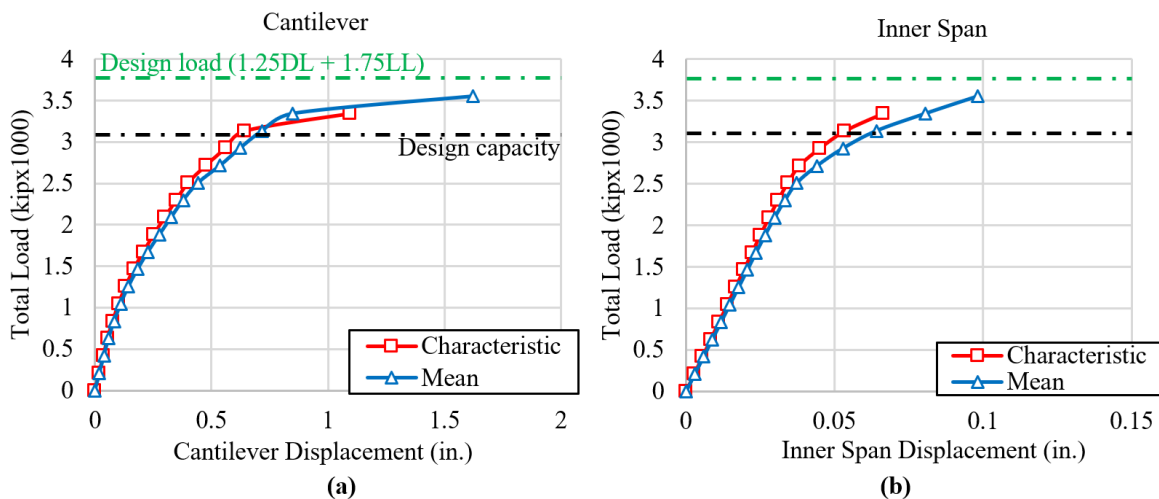
NOTE: 1 in. = 25.4 mm; 1 ft. = 0.305 m; 1 kip = 4.45 kN

Figure 7 – Finite element model developed for the NLFEA.



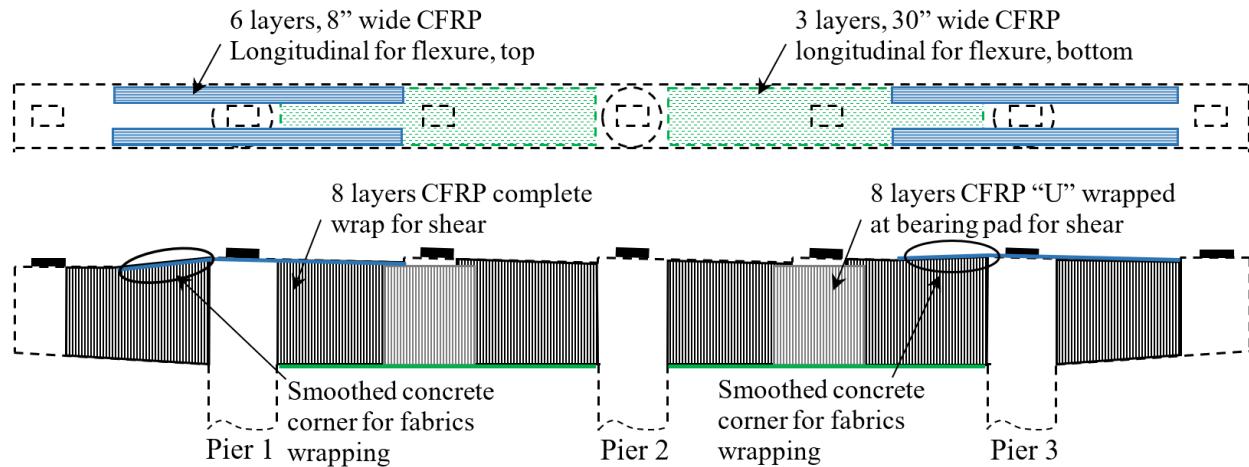
NOTE: 1 ksi = 6.9 MPa.

Figure 8 – Pushover analysis (a) crack pattern and (b) rebar stresses at failure loading condition (10 times actual deflection).



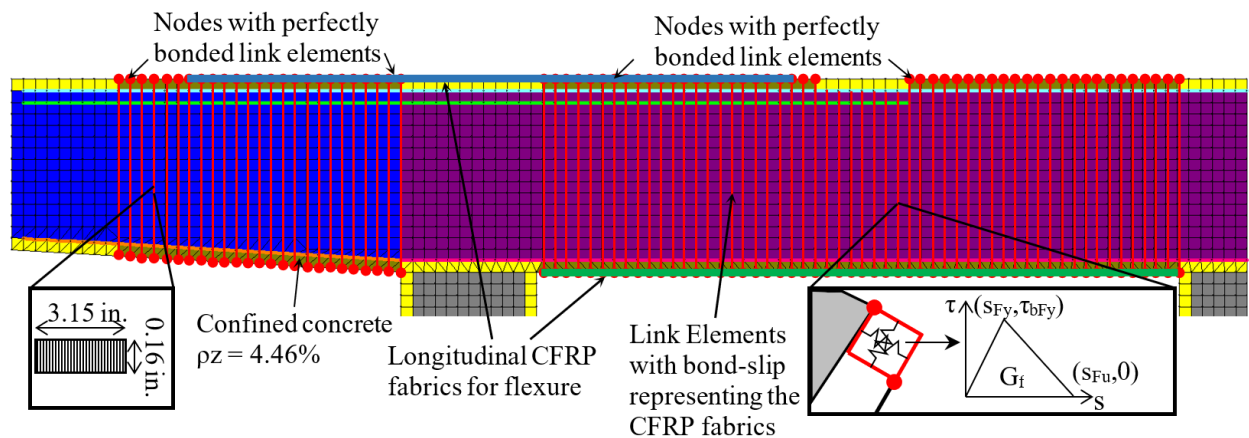
NOTE: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

Figure 9 – (a) Cantilever and (b) inner span total load versus displacement response.



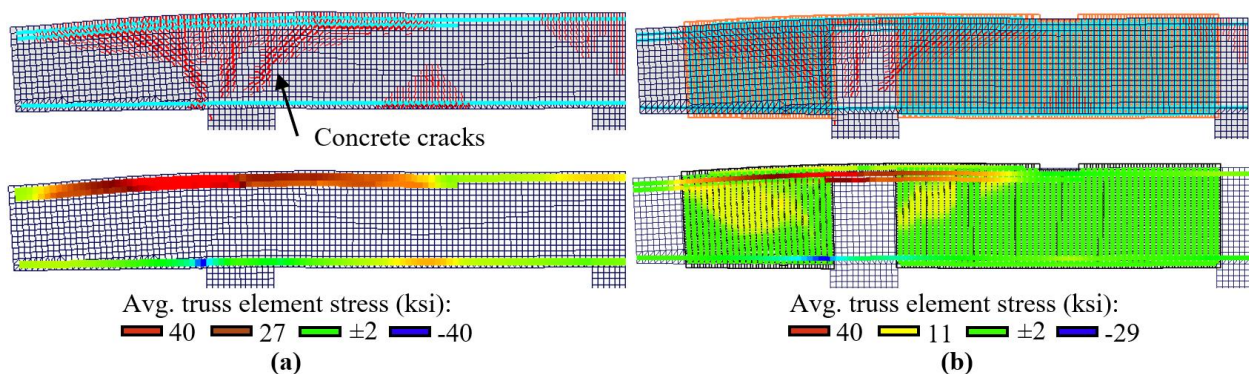
NOTE: 1 in. = 25.4 mm.

Figure 10 – FRP retrofit layout.



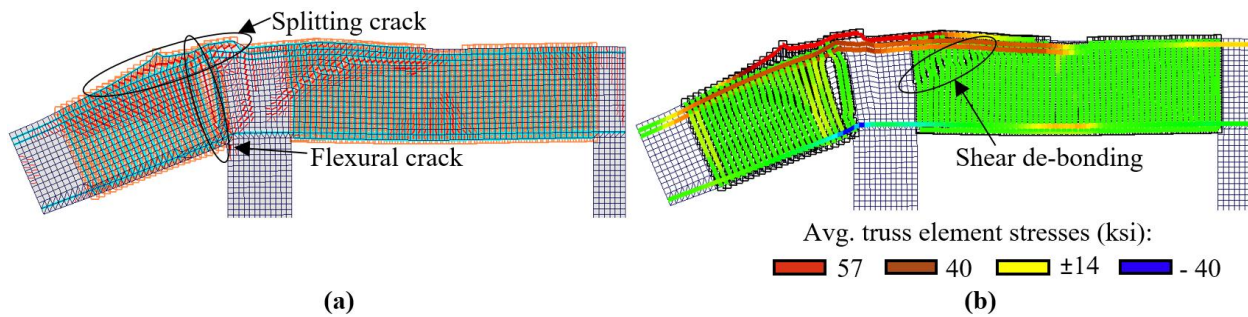
NOTE: 1 in. = 25.4 mm.

Figure 11 – Retrofitted finite element model.



NOTE: 1 ksi = 6.9 MPa.

Figure 12 – (a) Un-retrofitted model at failure condition and (b) retrofitted model at same loading condition.



NOTE: 1 ksi = 6.9 MPa.

Figure 13 – (a) Retrofitted cap beam response and (b) rebar stresses at failure loading condition (20 times actual deflection).

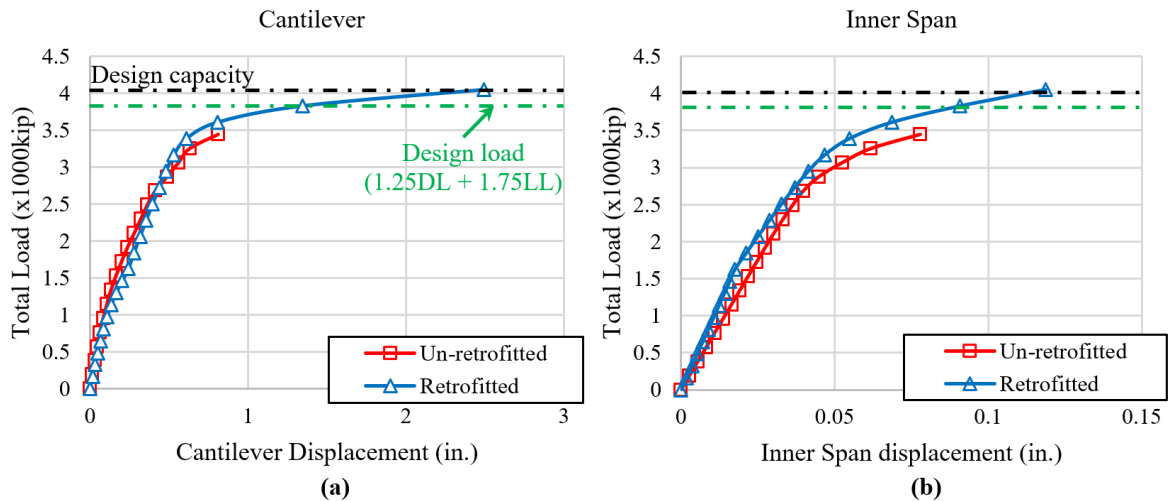


Figure 14 – (a) Cantilever and (b) inner span total load versus displacement response of the retrofitted model.

SP-333-2

Surrogate Modeling for Self-Consolidating Concrete Characteristics Estimation for Efficient Prestressed Bridge Construction

Junwon Seo and Jharna Pokhrel

Synopsis: This paper investigates the effects of material constituents on fresh and hardened properties of Self-Consolidating Concrete (SCC) mixture necessary for efficient prestressed bridge girder fabrication using a surrogate modeling technique. Response surface methodology (RSM)-based surrogate models consisting of input parameters such as density of coarse and fine aggregate were created based upon the past laboratory testing results for different SCC mixture trials. These models were used to estimate various SCC material characteristics, including slump flow, J-ring flow, passing ability, filling capacity, Visual Stability Index (VSI), T_{50} (concrete spread time to reach the 50.8 cm [20 in] mark), column segregation, 16-hour compressive strength, and 28-days compressive strength, while examining the correlation between the input parameters on each material characteristic. To observe the effect of core input parameters in an efficient manner, 2D contour plot and 3D surface plot for material characteristics were also created. Then, statistical analyses with the testing results were performed to determine the accuracy of the surrogate models in terms of coefficient of regression (R^2). Most of the R^2 values are higher than 90%, indicating a higher degree of correlation among the testing and surrogate data. Average predicted-to-measure ratios of the surrogate models were almost equal to or slightly greater than 1.00, showing good agreement with the testing results, and specifically the surrogate and testing values for J-ring flow and 28-days compressive strength were nearly identical. Key findings indicate that the coarse aggregate content significantly affected the characteristics of the SCC mixtures.

Keywords: bridge; characteristics; girder; material; prestressed; SCC; surrogate modeling

Surrogate Modeling for Self-Consolidating Concrete Characteristics Estimation for Efficient Prestressed Bridge Construction

ACI member **Junwon Seo** is an Assistant Professor in the Department of Civil and Environmental Engineering at South Dakota State University, Brookings, SD. He is an associate member of ACI Committees 341 (Earthquake-Resistant Concrete Bridges), 343 (Concrete Bridge Design: Joint ACI-ASCE), and 345 (Concrete Bridge Construction, Maintenance, and Repair). His research interests encompass the structural behavior of irregular bridges and other structures, multi-hazard vulnerability, and sustainability assessment, repair, retrofit and rehabilitation, structural reliability and risk analysis, and structural health monitoring. He is a licensed Professional Engineer in Iowa.

ACI student member **Jharna Pokhrel** holds a MS degree in Civil Engineering from South Dakota State University, Brookings, SD. Her research interests comprise multi-hazard structural vulnerability assessment, finite element analysis of structural components, and structural reliability analysis.

INTRODUCTION

Prestressed bridge girders made with Self-Consolidating Concrete (SCC) mixture require a number of experimental trial batches to achieve the desired fresh and hardened material characteristics [1-5]. The experimental setup and testing typically require significant amounts of time and costs to determine the optimal SCC mixtures meeting the requirements for prestressed bridge girder design and construction. Therefore, a more efficient technique to determine the optimal SCC material constituent proportions at low cost and time for better understanding of its fresh and hardened properties affecting the structural performance of SCC girders is needed.

Response Surface Methodology (RSM) has been regarded as a type of surrogate modeling techniques to predict different responses [6-12]. It is well known that RSM in conjunction with Design of Experiments (DOE) has been widely used because DOE helps construct RSM with input and output parameters within an effective number of matrix (e.g., experimental batches) [6, 13]. In spite of the integrated RSM and DOE applications in many fields, including structural engineering [14-18] and material sciences [12,19-25], there have been limited applications of the RSM to the concrete research, especially for SCC characteristics prediction aimed at constructing prestressed girder bridges efficiently.

This paper aimed to develop RSM-based surrogate models to predict different SCC material characteristics. Each model was developed using the laboratory testing database which includes input (e.g., density of coarse aggregate) and output parameters (e.g., J-ring flow). Further, each of the models was plotted in the form of 2D contour and 3D surface graphs per output. Finally, each model was validated by comparing the surrogate model responses to the laboratory testing data, so as to evaluate its accuracy.

RESEARCH SIGNIFICANCE

A typical SCC mixture comprises cement, water, coarse aggregates, fine aggregates, and admixtures. Because each of the SCC mixture constituents has an influence on its material properties, numerous batches need to be prepared and used for several material tests to achieve its desired material characteristics. To efficiently identify the optimal SCC mixture constituents that meet their desired material characteristics, resulting in improvement of their performance and workability, a generation of surrogate models would be beneficial. Therefore, this paper developed the RSM-based surrogate model utilizing experimental data to predict fresh and hardened properties of the SCC mixtures. This demonstrated a significant effect of coarse and fine aggregates on the fresh and hardened properties of the SCC mixtures.

BACKGROUND

RSM was initially developed by Box and Wilson [26] for determining an optimum solution in chemical engineering. Many RSM applications over the past couple of decades have been done in different fields including physics [27-29], structural engineering [11, 30-32], material sciences [12, 19-25], and medical sciences [33-35]. Some studies have been done in the concrete research, and it was concluded that the RSM exhibited excellent correlation between the experimental and estimated responses. For example, Khayat et al. [1] applied RSM to assess the effect of SCC mixture parameters, such as cementitious material content, water-cement ratio, and volume of coarse aggregate on the material properties, including slump flow, filling capacity, relative torque viscosity, flow resistance, compressive strength, surface settlement and V-funnel flow time. Ghezal et al. [23] also used the statistical factorial design method in assessing the effect of SCC mixture parameters, such as cement content, filler content and high range water reducing admixtures (HRWR). Lofty et al. [11] employed RSM to develop a statistical model to explore the

relationship between the input parameters, such as water to binder ratio, affecting the material characteristics of SCC mixture and Ghafari et al. [19] developed a regression model to predict the performance of the ultra-high performance SCC reinforced with steel micro-fibers. However, these studies did not study fully the effects of various aggregates with other parameters on the characteristics of SCC mixture. RSM has been also applied to normal concrete other than SCC, in an attempt to achieve more optimized material properties. Simon et al. [31] used RSM to optimize the material properties in terms of certain outputs (e.g., slump flow) due to variance in different factors such as water-cement ratio. Baryamov et al. [24] also developed an analytical model utilizing RSM to enhance the ductile behavior of steel fiber reinforced concrete by optimizing key parameters such as aspect ratio of steel fiber in concrete. It was concluded from the literature review that RSM was able to efficiently predict the material properties of normal concrete mixtures, and the experimental and surrogated response exhibited good correlation.

FRESH AND HARDENED PROPERTY DATABASE

A set of fresh and hardened property database consisting of 28 tested SCC mixtures was utilized to create RSM-based surrogate models (see Table 1). The input parameters are 9.5 mm [0.35 in] coarse aggregate, 19 mm [0.75 in] coarse aggregate, fine aggregate, High Range Water Reduces (HRWR), and Viscosity Modifying Admixtures (VMA), while output parameters for the evaluation of fresh and hardened properties of SCC are slump flow, J-ring flow, passing ability, filling capacity, Visual Stability Index (VSI), T_{50} (time required for the concrete mixture to spread to a diameter of 50.8 cm [20 in]), column segregation, 16-hour compressive strength (initial compressive strength measured after 16 hours curing), and 28-days compressive strength (final compressive strength measured after 28 days curing). A constant value for the density of cement and water were maintained at 474 kg/m³ [29.59 lb/ft³] and 166 kg/m³ [10.36 lb/ft³], respectively. Note, three different plants of coarse aggregate (including Plant A – 1A through 11A mixtures, Plant B – 12B through 21B, and Plant C – 22C through 28C mixtures in Table 1) were chosen from various regions in Wisconsin [32]. The specific gravities of the coarse aggregate to Plant A, Plant B, and Plant C were 2.66, 2.59, and 2.77, respectively. The input parameters including coarse aggregate and fine aggregate have remarkable impact on the strength and flowability of the mixture. Inclusion of chemical admixtures such as HRWR and VMA aids in improving the workability of the mixture. Further detailed information for the fresh and hardened SCC material testing setup and procedure can be found elsewhere [32].

For the determination of the fresh and hardened properties, the American Society for Testing and Material (ASTM) testing standards were utilized. Specifically, the slump flow test was specified by ASTM C1611, which recommended the desired diameter for the mixture ranging from 533 mm [21 in] to 737 mm [29.02 in]. The J-ring flow, column segregation, and compressive strength tests followed ASTM C1621, ASTM C1610, and ASTM C39, respectively. The detailed information on the ASTM standards used for this study can be found elsewhere [32].

SURROGATE MODELING

RSM Overview

RSM-based surrogate models that can be created by a least-square fitting method [36] was used in this study, and these models were based on Central Composite Design (CCD), which is one of the commonly preferred DOEs [7]. It can be noted that each model determines a functional relationship between input parameters of interest and output response from experimental databases. The CCD-RSM consists of a second order polynomial surrogate model to predict the desired output of the mixture as shown in Eq. 1:

$$\hat{y} = \beta_0 + \sum_{i=1}^k \beta_i x_i + \sum_{i=1}^k \beta_{ii} x_i^2 + \sum_{i=1}^{k-1} \sum_{j>1}^k \beta_{ij} x_i x_j + \epsilon \quad (1)$$

where y is the predicted response of the SCC mixture (e.g., J-ring flow), x_i and x_j are input parameters (e.g., density of coarse aggregate), β_0 , β_i , β_{ii} , β_{ij} are regression coefficients determined from surrogate modeling, i and j are linear and quadratic coefficients, k is the number of input parameters and ϵ is the random error.

Surrogate Modeling for Self-Consolidating Concrete Characteristics Estimation for Efficient Prestressed Bridge Construction

RSM Development

To create RSM-based surrogate models, the 28 different SCC mixture combinations (see Table 1) were used via MATLAB (Math Laboratory Inc.) [23]. As stated previously, each mixture consisted of the combination of aggregates and admixture contents. The input parameters used in the creation of each surrogate model were X_1 (density of 9.5mm coarse aggregates), X_2 (density of 19mm coarse aggregates), X_3 (density of fine aggregates), X_4 (density of HRWR content), and X_5 (density of VMA content), while the output parameters were slump flow, J-ring flow, passing ability, filling capacity, VSI, T_{50} , column segregation, 16-hour compressive strength, and 28 days compressive strength. It is noteworthy that the range of each input parameter is 0-808 kg/m³ [50.44 lb/ft³] for X_1 , 0-763 kg/m³ [47.63 lb/ft³] for X_2 , 621-719 kg/m³ [38.77- 44.89 lb/ft³] for X_3 , 0.95-2.13 L/m³ [0.0071-0.016 gal/ft³] for X_4 , and 0-0.75L/m³ [0.0056 gal/L³] for X_5 .

The experimental results for J-ring flow along with input parameters were used to develop the surrogate model for J-ring flow. The surrogate model for the prediction of fresh properties of the SCC mixture in terms of J-ring flow here referred to as $\hat{y}_{JR}$ is expressed by the second order polynomial equation as shown in Eq. 2:

$$\begin{aligned}\hat{y}_{JR} = & 6424.187 - 8.17 * X_1 - 3.95 * X_2 - 10.47 * X_3 - 1081.94 * X_4 + 2277.408 * \\ & X_5 + 0.038 * X_1 * X_2 - 0.0276 * X_1 * X_3 - 0.55 * X_1 * X_4 - 0.16 * X_1 * X_5 - 0.035 * X_2 * \\ & X_3 - 0.68 * X_2 * X_4 + 0.21 * X_2 * X_5 + 2.29 * X_3 * X_4 - 2.23 * X_3 * X_5 - 574.22 * X_4 * \\ & X_5 + 0.018 * X_1 * X_1 + 0.019 * X_2 * X_2 + 0.024 * X_3 * X_3 + 69.27 * X_4 * X_4 - 197.87 * \\ & X_5 * X_5\end{aligned}\quad (2)$$

where X_1 = 9.5mm coarse aggregate content (kg/m³); X_2 = 19mm coarse aggregate content (kg/m³); X_3 = fine aggregate content (kg/m³); X_4 =HRWR content (L/m³); X_5 =VMA content (L/m³).

Similarly, the surrogate model for the prediction of hardened properties of SCC mixture in terms of 16-hour compressive strength herein denoted as $\hat{y}_{16hstrength}$ can be expressed as:

$$\begin{aligned}\hat{y}_{16hstrength} = & 678.74 - 1.922 * X_1 - 0.98 * X_2 - 0.511 * X_3 - 54.126 * X_4 + \\ & 278.99 * X_5 + 0.0036 * X_1 * X_2 - 0.00075 * X_1 * X_3 - 0.14 * X_2 * X_4 + 0.122 * \\ & X_1 * X_5 - 0.0022 * X_2 * X_3 - 0.1006 * X_2 * X_4 + 0.0627 * X_2 * X_5 + 0.23 * X_3 * \\ & X_4 - 0.415 * X_3 * X_5 - 50.69 * X_4 * X_5 + 0.00178 * X_1 * X_1 + 0.00175 * X_2 * X_2 + \\ & 0.0012 * X_3 * X_3 + 1.38 * X_4 * X_4 - 9.55 * X_5 * X_5\end{aligned}\quad (3)$$

The remaining equations for both fresh and hardened properties such as slump flow, passing ability, filling capacity, VSI, T_{50} , column segregation, and 28-days compressive strength are included in Appendix A section of the paper.

SCC CHARACTERISTICS PREDICTION

A surrogating modeling analysis of the RSM response was carried out to determine the influence of input parameters in characterizing the fresh and hardened properties of the SCC mixtures. To determine the interaction between the parameters in a surrogate response, 2D contour and 3D response surface plots were generated and used. Further details for the surrogate fresh and hardened characteristics are presented in the subsequent sections.

Fresh Properties

For the fresh property characterization, the surrogate models were established in terms of the following output parameters: 1) Slump flow; 2) J-ring flow; 3) Passing ability; 4) Filing capacity; 5) VSI; 6) T_{50} ; 7) Column segregation. The surrogate model for J-ring flow as a representative parameter among the outputs was first used. This is because the model has a high correlation coefficient with the testing results as seen in Table 2. The 2D contour and 3D response surface plots as shown in Figure 1 were utilized to examine the effects of influential input parameters on the surrogate J-ring flow. It appears that the 2D contour plot for J-ring flow experiences higher variability for the pairs of the following input parameters: X_1 and X_2 , X_2 and X_3 , and X_2 and X_4 . The effect of variation in X_1 and X_2 on the J-ring flow value can be observed in Figure 1a, resulting from a constant value of X_3 of 647.88 kg/m³ [40.44 lb/ft³], X_4 of 1.36 L/m³ [0.01 gal/ft³], and X_5 of 0.3 L/m³ [0.0022 gal/ft³]. This figure shows that the J-ring flow value decreases when increasing X_1 and maintaining a low value of X_2 , whereas the opposite

phenomenon is observed when a high value of X_2 is maintained. These phenomena can be attributed to the increase in X_1 and X_2 , leading to the increase in slump flow value which is directly related to J-ring flow and the improvement in the blending configuration which help increase the J-ring flow. Similarly, the effects of X_3 and X_2 on variation in the J-ring flow is shown in Figure 1b for a constant value of X_1 of 240.08 kg/m³ [14.99 lb/ft³], X_4 of 1.36 L/m³ [0.01 gal/ft³] and X_5 of 0.29 L/m³ [0.0022 gal/ft³]. It is observed that J-ring flow expands with increase in X_3 . Such an increase in the fine aggregate content amplifies the fluidity of the mixture, resulting in increase in the higher J-ring flow value, while the lower X_2 results in a relatively higher J-ring flow value due to the increased density of 19 mm coarse aggregate (X_2), causing lesser particle movement and larger particle size, which assist in reducing the J-ring flow. Such similar behaviors are observed, including variability in a J-ring flow value with change in X_4 and X_2 as seen in Figure 1c for a constant value of X_1 of 240.08 kg/m³ [14.99 lb/ft³], X_3 of 647.88 kg/m³ [40.44 lb/ft³], and X_5 of 0.29 L/m³ [0.0022 gal/ft³]. Increase in X_4 showed the improvement in the J-ring flow value as a result of the increase in the fluidity of the mixture.

Further investigation was done for estimating the effects of influential input parameters on reminders of the fresh properties of the SCC mixtures. 2D contour and 3D response surface plots corresponding to slump flow; passing ability; filling capacity; VSI; T_{50} ; and column segregation are shown in Figure B1 and B2 in Appendix B. Such visualization can be beneficial in efficient identification of influencing parameters for the observed response. As shown in Figure B1a and B2a, high fluctuations in the slump flow are observed for the pairs of the following input parameters: X_1 and X_2 , X_1 and X_3 , and X_1 and X_4 among others. This contributes to the fact that 9.5 mm coarse aggregate (X_1) plays a significant role in determining slump flow value of the mixture. Both figures show that the slump flow increases when both X_1 and X_2 are maintained at a high value. These graphs are created at a constant value of X_3 of 645.96 kg/m³ [40.32 lb/ft³], X_4 of 1.35 L/m³ [0.01 gal/ft³] and X_5 of 0.3 L/m³ [0.0022 gal/ft³]. Analogous to what was observed in the J-ring flow investigation, this phenomenon of increase in the slump flow is caused by increase in X_1 and X_2 . Similarly, to observe the slump flow variation with respect to change in X_3 and X_1 , representative plots are shown in Figure B1b and B2b for a constant value of X_2 of 430.86 kg/m³ [26.9 lb/ft³], X_4 of 1.35 L/m³ [0.01 gal/ft³], and X_5 of 0.3 L/m³ [0.0022 gal/ft³]. It is observed that reduction in slump flow occurs when X_1 increases within the considered range of X_3 . Decrease in the slump flow is observed when X_1 is fixed at 400 kg/m³ [24.97 lb/ft³] with an increase in X_1 . This phenomenon of decrease in the slump flow at a high value of coarse aggregate can be explained mainly because of less passing ability of the mixture. Similar behaviors are observed, while including the variation of slump flow with change in X_4 and X_1 content as shown in Figure B1c and B2c for a constant value of X_2 of 430.86 kg/m³ [26.9 lb/ft³], X_3 of 645.96 kg/m³ [40.32 lb/ft³], and X_5 of 0.3 L/m³ [0.0022 gal/ft³]. It is apparent that increment in X_4 improved the workability of the mixture, which means that this increase in the content of HRWR (X_4) improved the viscosity of the mixture, resulting in the improvement of the slump flow.

The passing ability undergoes high alteration for the pairs of input parameters: X_1 and X_2 , X_2 and X_4 , and X_2 and X_5 . To observe the variation of X_1 and X_2 in the passing ability, respective 2D and 3D plots are shown in Figure B1d and B2d. It can be observed that increment in passing ability occurs when both X_1 and X_2 are maintained at a high value. These plots are developed for X_3 of 647.88 kg/m³ [40.44 lb/ft³], X_4 of 1.36 L/m³ [0.01 gal/ft³], and X_5 of 0.29 L/m³ [0.0022 gal/ft³]. When a high value of X_2 is maintained, an increase in X_1 causes passing ability to increase. This behavior is well explained by a rise in viscosity and blending configuration observed which aid in improving the passing ability of the SCC mixtures. To look into the effect of X_4 and X_2 on the passing ability, representative plots are seen in Figure B1e and B2e for X_1 of 240.08 kg/m³ [14.99 lb/ft³], X_3 of 647.88 kg/m³ [40.44 lb/ft³], and X_5 of 0.29 L/m³ [0.0022 gal/ft³]. It can be noted that the increase in X_2 lowers the passing ability of the mixtures, while an increase in X_4 improves it. It should be noted that an addition of HRWR (X_4) in the mixtures improved the fluidity, which corresponds to increase in passing ability. Similarly, the behavior of passing ability with a change in X_5 and X_2 are illustrated in Figure B1f and B2f regarding X_1 of 240.08 kg/m³ [14.99 lb/ft³], X_3 of 647.88 kg/m³ [40.44 lb/ft³], and X_4 of 1.36 L/m³ [0.01 gal/ft³]. The increase in coarse aggregate caused reduction on the passing ability and on the J-ring flow. This resembles to the fact that the increase in an aggregate size of the mixture causes higher blockage lowering the passing ability of the mixture.

The filling capacity with influential input parameters shows comparatively high change on the 2D and 3D plots for X_1 and X_2 , X_3 and X_2 , and X_4 and X_2 , as illustrated in Figure B1g and B2g for X_3 of 647.88 kg/m³ [40.44 lb/ft³], X_4 of 1.36 L/m³ [0.01 gal/ft³]. It is found that the filling capacity increases when X_2 increases with a high range of X_1 , while the behavior is opposite to that for a low range of X_1 . This can be attributed to the fact that increase in 9.5 mm coarse aggregate (X_1) results in relatively higher viscosity [37] and improvement in particle packing ability observed, whereas increase in 19 mm coarse aggregate (X_2) lowers the passing ability of the mixture because of the

Surrogate Modeling for Self-Consolidating Concrete Characteristics Estimation for Efficient Prestressed Bridge Construction

particle sizes. The variation in X_3 and X_2 on filling capacity are also shown in Figure B1h and B2h with X_1 of 240.08 kg/m³ [14.99 lb/ft³], X_4 of 1.36 L/m³ [0.01 gal/ft³], and X_5 of 0.29 L/m³ [0.0022 gal/ft³]. With increase in X_2 , the filling capacity of the mixtures is reduced for all values of X_3 , while an increase in X_3 improves the filling capacity. Increase in fine aggregate (X_3) correlated to the rise in viscosity of the mixtures. As shown in Figure B1i and B2i for X_1 of 240.08 kg/m³ [14.99 lb/ft³], X_3 of 647.88 kg/m³ [40.44 lb/ft³], and X_5 of 0.29 L/m³ [0.0022 gal/ft³], it is evident that increasing X_2 in the mixtures lowers the filling capacity, whereas increment in the filling capacity is observed due to the increase in X_4 . This tendency is similar to that observed from the J-ring flow and slump flow-based results.

The 2D and 3D plots for the change in VSI value shows that high variation is observed on the plots for X_1 and X_2 , X_3 and X_2 , and X_4 and X_2 as shown in Figure B1j and B2j for X_3 of 645.96 kg/m³ [40.33 lb/ft³], X_4 of 1.35 L/m³ [0.01 gal/ft³], and X_5 of 0.3 L/m³ [0.0022 gal/ft³]. A high value for X_2 and X_1 produced more favorable VSI because of less bleeding or segregation observed in the mixtures. Both the plots for change in VSI for X_2 and X_3 is displayed in Figure B1k and B2k for X_1 of 266.86 kg/m³ [16.66 lb/ft³], X_4 of 1.35 L/m³ [0.01 gal/ft³], and X_5 of 0.3 L/m³ [0.0022 gal/ft³]. It is indicated that the VSI is reduced when X_2 increases for all values of X_3 . The increase in X_3 with high values of X_2 results in improving VSI. Increase in the fine aggregate (X_3) help enhance viscosity of the mixtures, which is helpful in maintaining the stability of the concrete. Similar behaviors are observed for the variation in the X_2 and X_4 as shown in Figure B1l and B2l for X_1 of 266.86 kg/m³ [16.66 lb/ft³], X_3 of 645.96 kg/m³ [40.33 lb/ft³], and X_5 of 0.3 L/m³ [0.0022 gal/ft³].

The 2D and 3D plots for T_{50} can be found in Figure B1m and B2m for X_1 of 645.96 kg/m³ [40.33 lb/ft³], X_4 of 1.35 L/m³ [0.01 gal/ft³], and X_5 of 0.3 L/m³ [0.0022 gal/ft³]. These plots indicate that the response time for the concrete mixture to reach 50.8 cm diameter is highly dependent on the variation for X_1 and X_2 , X_3 and X_2 , X_4 and X_2 . When X_2 is increased for the mixtures maintained at high X_1 values, the T_{50} accelerates with increase in X_2 value. This is caused by the fact that high content of X_1 and X_2 offers improvement in viscosity which increased T_{50} values, appropriate for slow curing process. Similarly, lowering the content of coarse aggregate improves the flowability of the mixture which in turn improves the T_{50} response as seen the Figure B1m and B2m, suitable in congested concrete placement area. The variation in X_3 and X_2 on T_{50} value are shown in Figure B1n and B2n for X_1 of 266.86 kg/m³ [16.66 lb/ft³], X_4 of 1.35 L/m³ [0.01 gal/ft³], and X_5 of 0.3 L/m³ [0.0022 gal/ft³]. It can be observed that increase in X_3 improves the T_{50} value as higher binding ability achieved in conjunction to X_2 . The variation in X_2 and X_4 in estimating T_{50} values is shown in Figure B1o and B2o for X_1 of 266.86 kg/m³ [16.66 lb/ft³], X_3 of 645.96 kg/m³ [40.33 lb/ft³], and X_5 of 0.3 L/m³ [0.0022 gal/ft³]. Increase in X_2 caused reduction in the T_{50} values of the mixtures until X_2 of around 400 kg/m³ is reached. This phenomenon is explained as improvement in viscosity is observed leading to reduction in passing ability of the mixture.

Figure B1p and B2p can show 2D and 3D plots for high variability in column segregation due to the effects of X_1 and X_2 , X_3 and X_2 , and X_5 and X_2 . These figures were created with a constant value of X_3 of 648.96 kg/m³ [40.51 lb/ft³], X_4 of 1.36 L/m³ [0.01 gal/ft³], and X_5 of 0.29 L/m³ [0.0022 gal/ft³]. It can be seen that increase in X_1 causes the decrease in column segregation. This behavior is detected due to increase in weight of aggregates causing higher settlement rate. The change in column segregation with variation in X_3 and X_2 are shown in Figure B1q and B2q for X_1 of 244.4 kg/m³ [15.25 lb/ft³], X_4 of 1.36 L/m³ [0.01 gal/ft³] and X_5 of 0.29 L/m³ [0.0022 gal/ft³]. Increase in X_2 lowers the column segregation of the SCC mixtures until X_2 is equal to 400 kg/m³ [24.97 lb/ft³]. Further increment causes the response to improve slowly. It is also found that increase in X_3 slightly increases the column segregation. To observe the variation in column segregation for the change in X_5 and X_2 , Figure B1r and B2r for X_1 of 244.4 kg/m³ [15.25 lb/ft³], X_3 of 648.96 kg/m³ [40.51 lb/ft³] and X_4 of 1.36 L/m³ [0.01 gal/ft³] are illustrated. Increase in X_5 slightly gets down the column segregation value, while the change in X_2 showed similar results. This behavior is comparable to that observed for the passing ability.

Hardened properties

The surrogate models were generated for estimate of the following hardened property outputs, including 16-hour compressive strength and 28-days compressive strength. Specifically, the generated model for 16-days compressive strength as the representative output was considered to discuss in more detail. This model was chosen due to a high correlation coefficient with the experimental results as listed in Table 2. As shown in Figure 2, 2D contour and 3D response surface plots were utilized to observe the effects of influential input parameters on the surrogate 16-hour compressive strength of the SCC mixtures. It is observed that the response is highly dependent on the plots for X_1 and X_2 , X_2 and X_4 , and X_2 and X_5 . The effect of change in X_1 and X_2 on 16-hour strength is shown in Figure 2a. The

plot is generated for X_3 of 645.96 kg/m^3 [40.32 lb/ft^3], X_4 of 1.35 L/m^3 [0.01 gal/ft^3], and X_5 of 0.3 L/m^3 [0.0022 gal/ft^3]. This figure shows that increment in X_1 improves the 16-hour strength of the mixtures at high values of X_2 , whereas there is gradual decrement in the strength with low X_2 values. It can be inferred that this increment in the strength at X_1 results in increase in viscosity and workability, whereas increase in X_2 lowers the strength due to the presence of weak zone at the aggregate-cement interface where cracking is induced. The effects of X_2 and X_4 on the strength are shown in Figure 2b for X_1 of 266.86 kg/m^3 [16.66 lb/ft^3], X_3 of 645.96 kg/m^3 [40.32 lb/ft^3], and X_5 of 0.3 L/m^3 [0.0022 gal/ft^3]. It can be observed that increase in X_2 lowers the strength at initial points until X_2 reaches 400 kg/m^3 , and X_2 on further increment improves the strength. This behavior for change in X_2 on the strength is analogous to that of J-ring flow value, as increase in X_2 has a high particle size which blocks particle movements within each mixture. The increase in HRWR (X_4) in the mixtures is observed to improve the strength value slightly, because of improvement in workability. Figure 2c with X_1 of 266.86 kg/m^3 [16.66 lb/ft^3], X_3 of 645.96 kg/m^3 [40.32 lb/ft^3], and X_4 of 1.35 L/m^3 [0.01 gal/ft^3] can be seen for the visualization of the effects of X_5 and X_2 . Increase in X_2 initially lowers the strength until X_2 reaches 400 kg/m^3 [24.97 lb/ft^3] and on further increment improved the strength. The increase in X_5 slightly improved the strength. Similar results for 28 days compressive strength can be observed in Figure B3 and B4 in Appendix B.

COMPARISON TO EXPERIMENTAL DATA

The proposed surrogate models are presented to predict the response of each of the fresh and hardened SCC mixture properties. These models were tested to evaluate the adequacy of model fit. The measure for verifying the model accuracy in such computational simulation is estimated by determining the coefficient of regression (R^2). R^2 statistics gives a correlation between the experimental data and the predicted response. It should be higher for a particular model to be significant. However, it might be misleading in some cases having less number of input parameters. In such cases, adjusted R^2 is introduced for evaluating the goodness-of-fit. Finally, to verify the overall accuracy of the surrogate models, statistical tests at additional random data points was performed. Those tests included Mean Absolute Error (%), Maximum Absolute Error (%), and Root Mean Square Error (%). The Mean Absolute Error (MAE) is used to measure the spread of data while including the effect of total data set. The Maximum Absolute Error (MAX) is the sum of maximum relative error and maximum absolute error. The Root Mean Square Error (RSME) is the expected value of the square of the error and useful in the measurement of the average magnitude of the error.

The analysis results for the full regression analysis with the surrogate and testing data is shown in Table 2. The R^2 value for the J-ring flow, passing ability, filling capacity, column segregation, and 28-days compressive strength is higher than 90%, showing high correlation between the testing and surrogate response. Similarly, the adjusted R^2 value has a high value when the input parameters added to the mixtures improved the accuracy of the surrogate models. This table indicates that J-ring flow, column segregation, and 28-days compressive strength have relatively higher adjusted R^2 values when compared to the other outputs. However, the adjusted R^2 value for VSI and T_{50} are observed to be negative because some test data has less variability among the surrogate responses for given mixtures. The MAE for all the surrogate outputs is within a relatively low range of 0.00 to 5.51%, while VSI value (MAX = 177.82%) and for T_{50} value (MAX = 77.28%) has much higher MAX than that for J-ring flow value (MAX = 2.47%). It should be worthwhile to note that the other outputs have a MAX range of 2.47 to 56.57%. The RSMEs for VSI and T_{50} are 57.94% and 25.41%, which are higher than that for J-ring flow (RSME = 0.74%). Note that the range for other outputs lie between 0.74 to 18.35%. Lower RSME indicates a strong association between the testing and surrogate responses, while higher RSME results in presence of large number of outliers in the model prediction. The overall comparison of the surrogate response was done with testing data for the fresh and hardened properties. The accuracy of the surrogate models was also determined based on the ratio of surrogate-to-testing response. The summary of the testing and surrogate responses are listed in Table 3. The ratio of surrogate-to-testing values for slump flow, J-ring flow, filling capacity, passing ability, VSI, T_{50} , column segregation and compressive strength at 16-hour and 28-days on average were 1.0, 1.0, 1.0, 1.02, 0.88, 1.06, 1.02, 1.02, and 1.0 respectively. The ratio of surrogate-to-testing values only differs from 1.0 to 1.06, indicating extremely high accuracy for predicting all response except for VSI ratio (surrogate-to-testing ratio = 0.88). The reason why VSI has the relatively lower ratio is that the testing results only had three levels of VSI, which are 0, 0.5, or 1 and those from the surrogate model ranged from 0 to 1.05. Based on the analysis, it can be suggested that additional testing data covering a wide range of input parameters be used to more accurately determine the actual VSI values.

Surrogate Modeling for Self-Consolidating Concrete Characteristics Estimation for Efficient Prestressed Bridge Construction

The graphical representation for the testing and surrogate responses can be seen in Figure 3. The graph shows the ratio of surrogate-to-testing response of the RSM model. In this figure, the dotted line represents the upper and lower estimated errors at 95% confidence limit, while the diagonal line determines the inclination of the model. Note that points above the diagonal line indicate that the surrogate model has overestimated the testing response. For the derived model to show perfect alignment, the testing data line up along the 1:1 dashed line, indicating that the majority of data for each response lie within one standard region, which is sufficiently accurate to replicate the testing data. The majority of data for the testing response lie close to 1:1 diagonal line, resulting in a mean value of the surrogate-to-testing responses ratio within 1.00 ± 0.2 , indicating the high accuracy of the RSM model [38]. Thus, it can be deduced that the surrogate response of the mixture for the considered fresh and hardened properties demonstrated good agreement with the testing results, excluding VSI. As pointed out above, the graph in Figure 3e for VSI values provided less agreement to the target ratio of 1.0 than the others, due to less variability of VSI experimental results for the given SCC mixtures. Further details on physical significance of VSI can be found elsewhere [39].

CONCLUSIONS

The developed surrogate model could be applied extensively in the SCC concrete industry to predict the required fresh and hardened properties of the SCC mixtures. These models were valid for SCC mixtures made with a range of 0.00 to 763 kg/m³ [47.63 lb/ft³] of 9.5 mm coarse aggregates (X_1), 0.00 to 808 kg/m³ [50.44 lb/ft³] of 19 mm coarse aggregates (X_2), 621 to 719 kg/m³ [38.77- 44.89 lb/ft³] of fine aggregates (X_3), 0.95 to 2.13 kg/m³ [0.0071-0.016 gal/ft³] for HRWR (X_4), and 0.00 to 0.71 L/m³ [0.0056 gal/L³] of VMA (X_5). These models were then utilized in creating the 2D contour plots to examine the effects of the SCC mixture input parameters, which affect each of the desired output parameters, including slump flow, J-ring flow, passing ability, filling capacity, Visual Stability Index (VSI), T_{50} , column segregation, 16-hour and 28-days compressive strength of the tested SCC mixtures. It is noteworthy that the alteration in X_1 (9.5 mm coarse aggregate) and X_3 (fine aggregate) exhibited significant variation in the surrogate response in terms of the fresh and hardened properties of the SCC mixtures. The 3D response surface plot using the surrogate models were also generated to confirm the effect of input parameters on each of the outputs. It was found that most of the surface plots were an elliptical shape with the highest or lowest values at the mid-point, demonstrating the close interaction between coarse aggregate parameters and the others (e.g., fine aggregate). Finally, the accuracy of each model was evaluated with the testing fresh and hardened properties statistically and graphically, demonstrating a high level of accuracy of the models for all the outputs, excluding VSI. Note, the limited VSI database ranging from 0 to 1 used for this study resulted in the relatively low accurate surrogate model. To improve the accuracy of the surrogate VSI model in practice, a further statistical study with sufficient VSI datasets is needed.

REFERENCES

1. Khayat, K., Ghezal, A., and Hadriche, M., 2000, "Utility of statistical models in proportioning self-consolidating concrete", *Materials and structures*, 33(5), 338-344.
2. Berry, M., Kappes, B., and Kappes, L., 2015, "Optimization of Concrete Mixtures Containing Reclaimed Asphalt Pavement", *ACI Materials Journal*, 112(6), 723-733.
3. Akalin, O., Akay, K. U., and Sennaroglu, B., 2010, "Self-Consolidating High-Strength Concrete Optimization by Mixture Design Method", *ACI Materials journal*, 107(4), 43-53.
4. Shilstone, J. S., 1990, "Concrete mixture optimization", *Concrete International*, 12(6), 33-39.
5. Khayat, K., 2000, "Optimization and performance of air-entrained, self-consolidating concrete", *Materials Journal*, 97(5), 526-35.
6. Myers, R. H., Montgomery, D. C., and Anderson-Cook, C. M., 2016, "Response surface methodology: process and product optimization using designed experiments", John Wiley & Sons.
7. Montgomery, D. C., 2017, "Design and analysis of experiments", John Wiley & Sons.
8. Berry, M., Kappes, B., and Schroeder, D., 2017, "Reclaimed Asphalt Pavement as Aggregate in Portland Cement Concrete", *Special Publication*, 314, 1-14.
9. Nambiar, E. K., and Ramamurthy, K., 2006, "Models relating mixture composition to the density and strength of foam concrete using response surface methodology", *Cement and Concrete Composites*, 28(9), 752-760.
10. Mohammed, B. S., Fang, O. C., Hossain, K. M. A., and Lachemi, M., 2012, "Mix proportioning of concrete containing paper mill residuals using response surface methodology", *Construction and Building Materials*, 35, 63-68.

11. Lotfy, A., Hossain, K. M., and Lachemi, M., 2014, "Application of statistical models in proportioning lightweight self-consolidating concrete with expanded clay aggregates", *Construction and Building Materials*, 65, 450-469.
12. Gao, Y., Xu, J., Luo, X., Zhu, J., and Nie, L., 2016, "Experiment research on mix design and early mechanical performance of alkali-activated slag using response surface methodology (RSM)", *Ceramics international*, 42(10), 11666-11673.
13. Upasani, R. S., and Banga, A. K., 2004, "Response surface methodology to investigate the iontophoretic delivery of tacrine hydrochloride", *Pharmaceutical research*, 21(12), 2293-2299.
14. Seo, J., Dueñas-Osorio, L., Craig, J. I., and Goodno, B. J., 2012, "Metamodel-based regional vulnerability estimate of irregular steel moment-frame structures subjected to earthquake events", *Engineering Structures*, 45, 585-597.
15. Seo, J., and Linzell, D. G., 2013, "Use of response surface metamodels to generate system level fragilities for existing curved steel bridges", *Engineering Structures*, 52, 642-653.
16. Seo, J., and Linzell, D. G., 2012, "Horizontally curved steel bridge seismic vulnerability assessment", *Engineering Structures*, 34, 21-32.
17. Cheng, J., Jiang, J. J., and Xiao, R. C., 2003, "Aerostatic stability analysis of suspension bridges under parametric uncertainty", *Engineering Structures*, 25(13), 1675-1684.
18. Kim, D. H., and Lee, S. G., 2015, "Reliability analysis of offshore wind turbine support structures under extreme ocean environmental loads", *Renewable Energy*, 79, 161-166.
19. Ghafari, E., Costa, H., and Júlio, E., 2014, "RSM-based model to predict the performance of self-compacting UHPC reinforced with hybrid steel micro-fibers", *Construction and Building Materials*, 66, 375-383.
20. Kockal, N. U., and Ozturan, T., 2011, "Optimization of properties of fly ash aggregates for high-strength lightweight concrete production" *Materials & Design*, 32(6), 3586-3593.
21. Aldahdooh, M., Bunnori, N. M., and Johari, M. M., 2013, "Evaluation of ultra-high-performance-fiber reinforced concrete binder content using the response surface method", *Materials & Design*, 52, 957-965.
22. Muthukumar, M., Mohan, D., and Rajendran, M., 2003, "Optimization of mix proportions of mineral aggregates using Box Behnken design of experiments", *Cement and Concrete Composites*, 25(7), 751-758.
23. Ghezal, A., and Khayat, K. H., 2002, "Optimizing self-consolidating concrete with limestone filler by using statistical factorial design methods", *Materials Journal*, 99(3), 264-272.
24. Bayramov, F., Taşdemir, C., and Taşdemir, M., 2004, "Optimisation of steel fibre reinforced concretes by means of statistical response surface method", *Cement and Concrete Composites*, 26(6), 665-675.
25. Sonebi, M., Grunewald, S., and Walraven, J., 2007, "Filling ability and passing ability of self-consolidating concrete", *ACI Materials Journal*, 104(2), 162-170.
26. Box, J., and Wilson, W., 1951, "Central composites design", *JR Stat Soc*, 1, 1-35.
27. Lee, J. H., Park, S. J., and Jeon, S. J., 2006, "Optimum design criteria for a synchronous reluctance motor with concentrated winding using response surface methodology", *Journal of applied physics*, 99(8), 08R3251-08R3253.
28. Chen, Y., Zhao, L., Liu, B., and Zuo, S., 2012, "Application of response surface methodology to optimize microwave-assisted extraction of polysaccharide from Tremella", *Physics Procedia*, 24, 429-433.
29. Gao, N., Zhu, S. A., and He, B., 2006, "A new magnetic resonance electrical impedance tomography (MREIT) algorithm: the RSM-MREIT algorithm with applications to estimation of human head conductivity", *Physics in medicine and biology*, 51(12), 3067-3083.
30. Seo, J., Kim, Y. J., and Zandyavari, S., 2015, "Response surface Metamodel-based performance reliability for reinforced concrete beams Strengthened with FRP Sheets", *Special Publication*, 304, 1-20.
31. Simon, M. J., Lagergren, E. R., and Synder, K. A., 1997, "Concrete Mixture Optimization Statistical Mixture Design Methods," *International Symposium on High Performance Concrete*, New Orleans, LA, 230-244.
32. Torres, E., Seo, J., and Lederle, R. E., 2017, "Experimental and Statistical Investigation of Self-Consolidating Concrete Mixture Constituents for Prestressed Bridge Girder Fabrication", *Journal of Materials in Civil Engineering*, 29(9), 040171411-040171411.
33. Li, C., Bai, J., Cai, Z., and Ouyang, F., 2002, "Optimization of a cultural medium for bacteriocin production by Lactococcus lactis using response surface methodology", *Journal of Biotechnology*, 93(1), 27-34.
34. Mojarrad, J. S., Nemati, M., Valizadeh, H., Ansarin, M., and Bourbour, S., 2007, "Preparation of glucosamine from exoskeleton of shrimp and predicting production yield by response surface methodology", *Journal of agricultural and food chemistry*, 55(6), 2246-2250.

Surrogate Modeling for Self-Consolidating Concrete Characteristics Estimation for Efficient Prestressed Bridge Construction

35. Wang, X., Wu, Y., Chen, G., Yue, W., Liang, Q., and Wu, Q., 2013, "Optimisation of ultrasound assisted extraction of phenolic compounds from Sparganii rhizoma with response surface methodology", *Ultrasonics sonochemistry*, 20(3), 846-854.
36. Öktem, H., Erzurumlu, T., and Kurtaran, H., 2005, "Application of response surface methodology in the optimization of cutting conditions for surface roughness," *Journal of materials processing technology*, 170(1), 11-16.
37. Hu, J., and Wang, K., 2011, "Effect of coarse aggregate characteristics on concrete rheology", *Construction and Building Materials*, 25(3), 1196-1204.
38. Alshihri, M. M., Azmy, A. M., and El-Bisy, M. S., 2009, "Neural networks for predicting compressive strength of structural light weight concrete", *Construction and Building Materials*, 23(6), 2214-2219.
39. Seo, J., Torres, E., and Schaffer, W., 2017, "Self-consolidating concrete for prestressed bridge girders", *Wisconsin Department of Transportation (WisDOT), WisDOT 0092-15-03*, 181.

Table 1—Selected SCC concrete mixture proportions reproduced based on [23]

Combination	9.5mm course aggregate (kg/m ³)	19mm coarse aggregate (kg/m ³)	fine aggregate (kg/m ³)	HRWR (L/m ³)	VMA (L/m ³)	slump flow (mm)	J- ring flow (mm)	Passing ability (mm)	Filling capacity (%)	VSI	T50	Column segregation (%)	Compressive strength (MPa)	
													16- hour	28- days
1A	645	0	640	1.18	0.24	610	622	12	84.8	0	9.4	2.7	44.42	82.73
2A	763	0	621	1.18	0.47	610	-	-	-	0	7.4	-	41	-
3A	516	129	640	1.42	0.47	603	610	0	83.1	0	5.3	6.3	48.45	81.87
4A	387	258	640	1.18	0	622	610	12	83.7	0.5	12	6.4	46.58	76.23
5A	467	305	621	1.18	0.24	622	-	-	-	0.5	8.5	-	44.71	-
6A	258	387	640	1.3	0.71	629	622	7	85.7	1	3.9	2.8	47.98	70.08
7A	305	471	635	1.42	0.54	629	610	19	84	1	5.2	4.7	49.2	68.1
8A	129	516	640	1.42	0.24	578	635	57	85	0	10.6	9.1	58.14	
9A	152	610	621	1.66	0.35	641	622	19	86.3	1	4.8	4.2	48.6	65
10A	0	645	640	1.42	0.24	635	610	25	84.3	1	4.6	6.3	48.22	63.72
11A	0	763	621	1.18	0	584	622	0	85.4	0	7.3	10.1	49.94	68.32
12B	662	0	640	1.18	0	622	571	51	78.2	0	6.3	1.6	36	55.59
13B	530	132	640	1.42	0	660	622	38	87.3	1	7.1	3.3	38.06	60.13
14B	397	264	640	1.18	0.24	622	597	25	81.9	0	9.3	5.1	42.66	62.39
15B	264	397	640	1.18	0.24	643	622	21	86.4	1	13.6	8	48.25	-
16B	264	397	640	1.42	0.35	629	597	31	82.2	0.5	3.4	2	49.05	60.33
17B	264	397	621	1.18	0.35	625	603	22	82.9	0	3.6	10.1	47.98	71.66
18B	132	530	640	1.18	0.24	622	597	25	81.9	0	8.2	9.5	40.81	-
19B	132	530	621	1.42	0.47	635	597	38	82.5	0.5	5.9	-	46.11	-
20B	132	530	640	1.18	0.35	622	610	12	83.7	0	4.8	9.9	49.29	68.91
21B	0	662	640	1.18	0.47	660	616	44	86.4	1	5.7	11.8	40.47	67.08
22C	288	432	719	1.42	0.47	667	635	19	83.5	1	6.1	5.1	46.45	57.28
23C	317	402	647	0.95	0.47	641	603	38	83.7	1	5.8	3.4	47.73	61.58
24C	324	409	662	1.42	0	622	565	57	77.4	0	5.3	2.2	46.26	58.72
25C	144	576	719	1.42	0	667	622	45	87.6	1	3.1	9.9	47.31	62.58
26C	0	808	662	1.66	0.47	610	571	39	77.6	0	2.5	4.7	34.77	46.38
27C	0	794	647	1.66	0.47	610	597	13	82.1	0	3.4	3.1	37.59	46.98
28C	0	720	710	2.13	0.24	610	603	7	84.8	0	4	12.2	41.22	49.17

[1 kg/m³ = 0.0624 lb/ft³; 1 L/m³ = 0.0075 gal/ft³; 1mm = 0.0394 in; 1 MPa = 145.04 psi]

Table 2—Statistical Performance Measures for the derived RSM model

Statistics	multiple R-squared	adjusted R-squared	MAE (%)	MAX (%)	RMSE (%)
Slump flow	0.78	0.15	0.00	5.86	1.59
J-ring flow	0.95	0.73	0.31	2.47	0.74
Passing ability	0.92	0.57	1.27	56.57	18.35
Filling capacity	0.91	0.55	0.12	4.02	1.25
VSI	0.71	-0.12	5.51	177.82	57.94
T50	0.65	-0.36	0.16	77.28	25.41
Column segregation	0.96	0.79	0.00	29.32	9.87
16-hour compressive strength	0.82	0.32	1.84	12.62	5.02
28-days compressive strength	0.99	0.75	0.00	4.95	1.63

Table 3—Predicted-to-measured ratios of the proposed models

Results	Slump flow	J-ring flow	Passing ability	Filling capacity	VSI	T ₅₀	Column segregation	Compressive strength	
								16-hour	28-days
Ratio	1.00	1.00	1.02	1.00	0.88	1.06	1.02	1.02	1.00

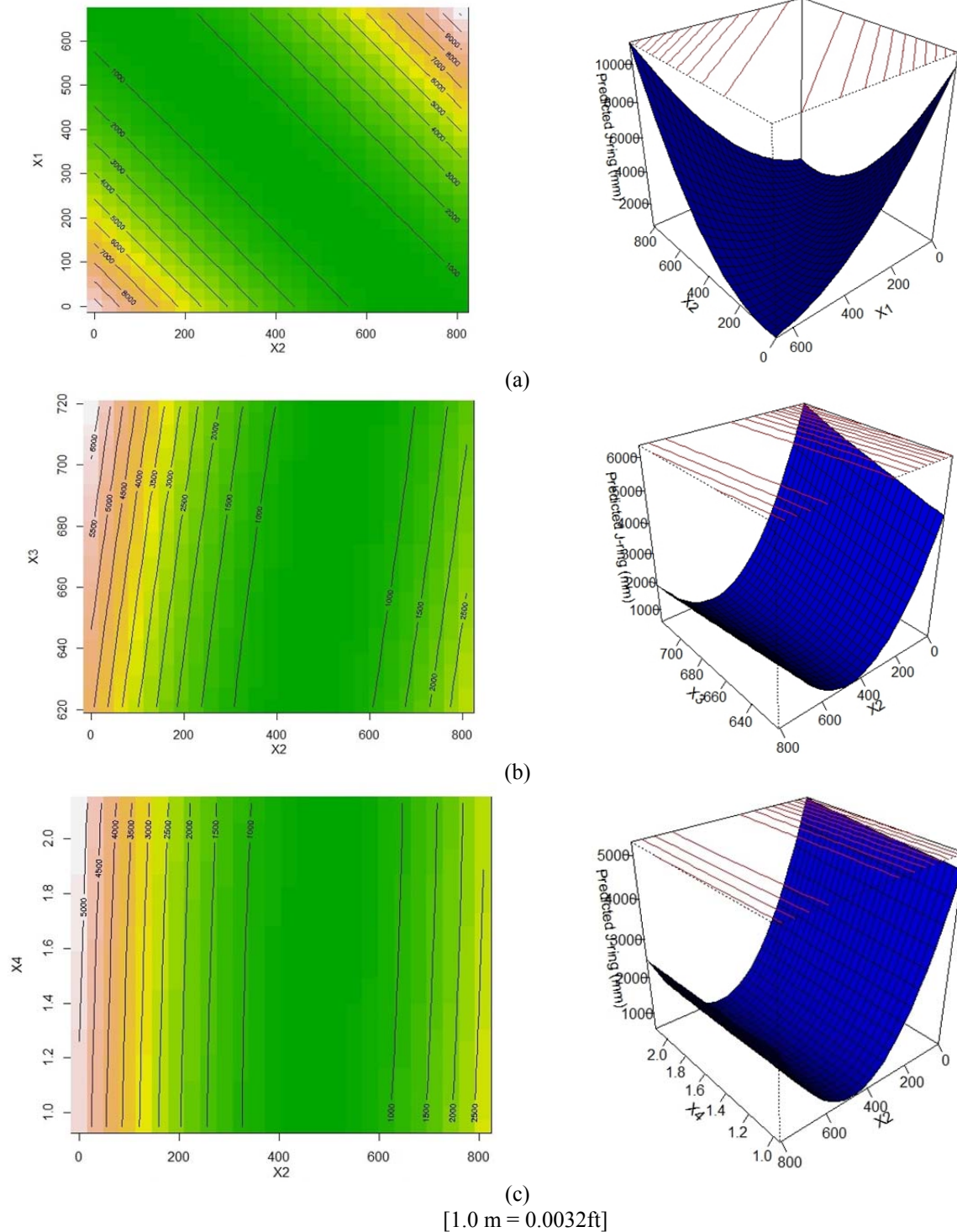
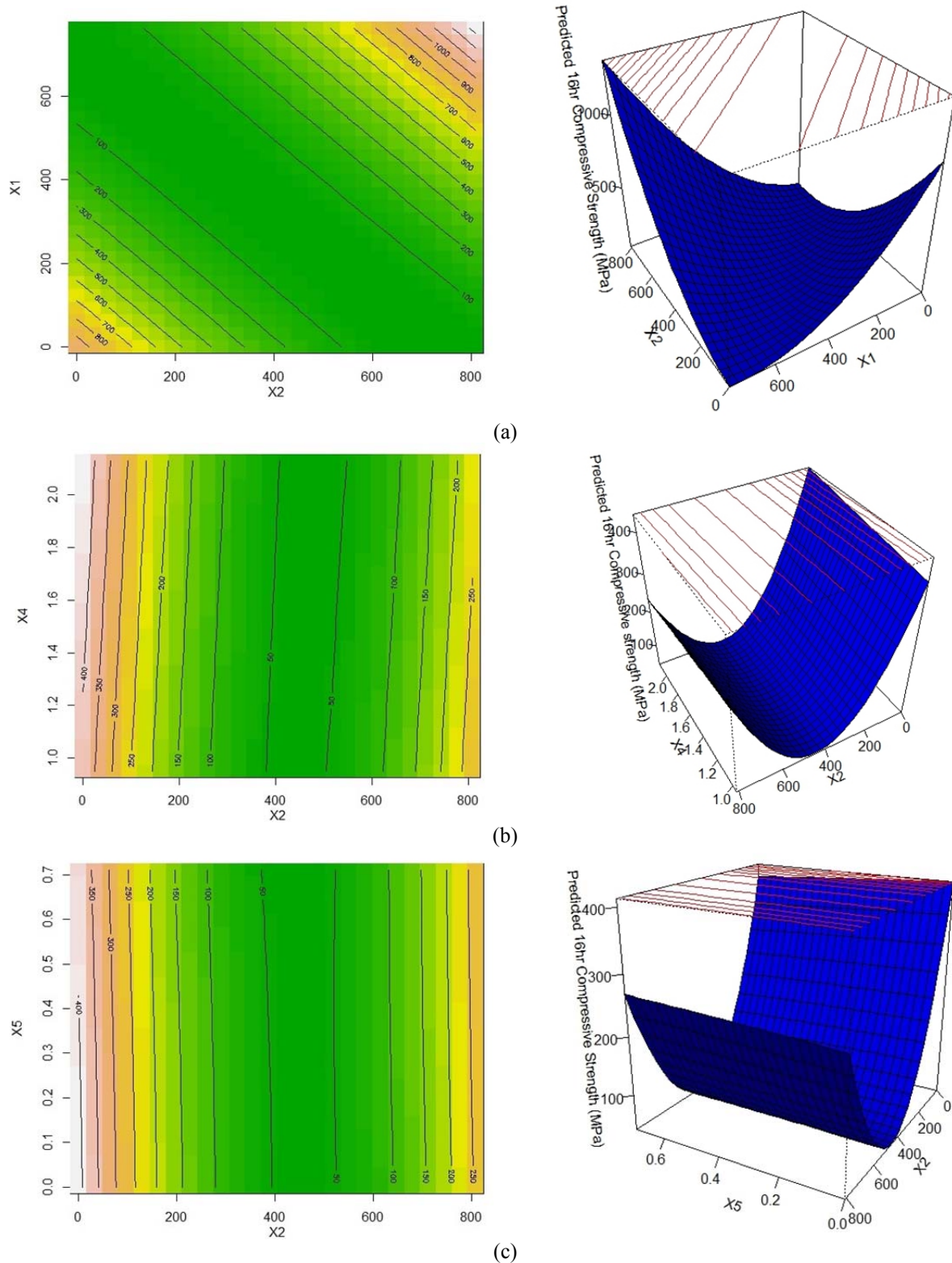


Figure 1—2D contour plot and 3D surface plot for the J-ring flow values on other parameters: (a) X_1 and X_2 ; (b) X_3 and X_2 ; (c) X_4 and X_2 .

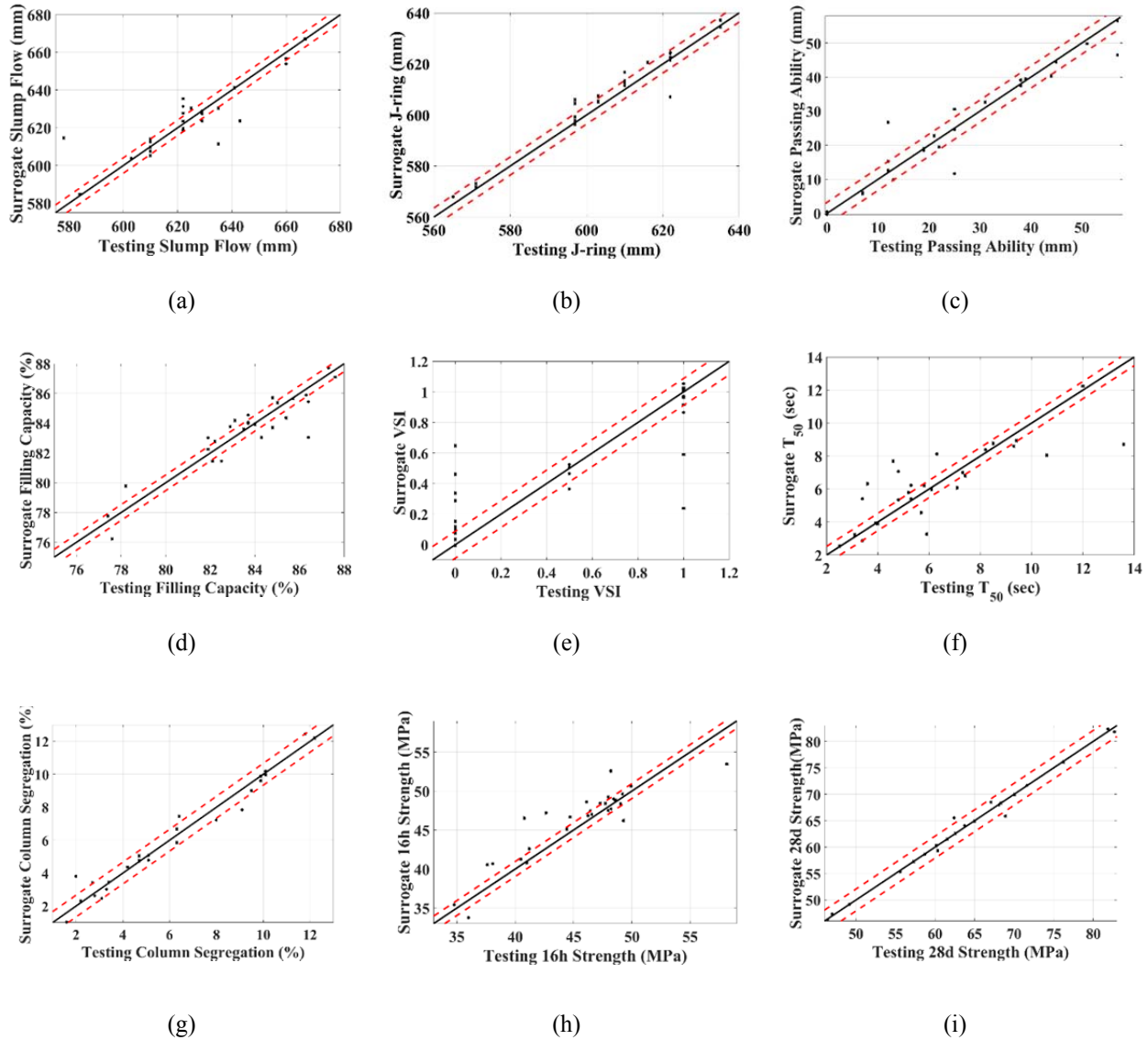
Surrogate Modeling for Self-Consolidating Concrete Characteristics Estimation for Efficient Prestressed Bridge Construction



[1 MPa = 145.04 psi]

Figure 2—2D contour plot and 3D surface plot for the 16-hour compressive strength values on other parameters:

(a) X_1 and X_2 ; (b) X_2 and X_4 ; (c) X_2 and X_5 .



[1 mm = 0.0394 in; 1 MPa = 145.04 psi]

Figure 3—Comparison between testing and surrogate values: (a) Slump flow; (b) J-ring flow; (c) Passing ability; (d) Filling capacity; (e) VSI; (f) T_{50} ; (g) Column segregation; (h) 16-hour compressive strength; and (i) 28-days compressive strength

APPENDIX A—RSM FUNCTION FOR SCC MIXTURE

$$\hat{y}_{SF} = 831.20 + 8.31 * X_1 + 6.70 * X_2 - 9.66 * X_3 + 671.38 * X_4 - 301.19 * X_5 - 0.0083 * X_1 * X_2 - 0.0050 * X_1 * X_3 + 0.58 * X_1 * X_4 + 0.11 * X_1 * X_5 - 0.002 * X_2 * X_3 + 0.37 * X_2 * X_4 + 0.38 * X_2 * X_5 - 1.47 * X_3 * X_4 + 0.57 * X_3 * X_5 - 200.68 * X_4 * X_5 - 0.0042 * X_1 * X_1 - 0.0042 * X_2 * X_2 + 0.01 * X_3 * X_3 + 8.22 * X_4 * X_4 + 46.19 * X_5 * X_5 \quad (A1)$$

$$\hat{y}_{JR} = 6424.187 - 8.17 * X_1 - 3.95 * X_2 - 10.47 * X_3 - 1081.94 * X_4 + 2277.408 * X_5 + 0.038 * X_1 * X_2 - 0.0276 * X_1 * X_3 - 0.55 * X_1 * X_4 - 0.16 * X_1 * X_5 - 0.035 * X_2 * X_3 - 0.68 * X_2 * X_4 + 0.21 * X_2 * X_5 + 2.29 * X_3 * X_4 - 2.23 * X_3 * X_5 - 574.22 * X_4 * X_5 + 0.018 * X_1 * X_1 + 0.019 * X_2 * X_2 + 0.024 * X_3 * X_3 + 69.27 * X_4 * X_4 - 197.87 * X_5 * X_5 \quad (A2)$$

$$\hat{y}_{PA} = 8001.97 - 21.34 * X_1 - 21.05 * X_2 - 6.99 * X_3 + 1734.66 * X_4 + 2704.47 * X_5 + 0.039 * X_1 * X_2 - 0.0030 * X_1 * X_3 - 2.18 * X_1 * X_4 - 3.35 * X_1 * X_5 - 0.0021 * X_2 * X_3 - 2.24 * X_2 * X_4 - 2.44 * X_2 * X_5 - 1.22 * X_3 * X_4 - 1.05 * X_3 * X_5 + 81.85 * X_4 * X_5 + 0.019 * X_1 * X_1 + 0.0184 * X_2 * X_2 + 0.0085 * X_3 * X_3 + 198.17 * X_4 * X_4 - 435.73 * X_5 * X_5 \quad (A3)$$

$$\hat{y}_{FC} = 135.43 + 0.65 * X_1 + 0.95 * X_2 - 0.81 * X_3 - 134.11 * X_4 + 228.86 * X_5 + 0.0026 * X_1 * X_2 - 0.0039 * X_1 * X_3 - 0.008 * X_1 * X_4 + 0.11 * X_1 * X_5 - 0.0046 * X_2 * X_3 - 0.027 * X_2 * X_4 + 0.14 * X_2 * X_5 + 0.27 * X_3 * X_4 - 0.29 * X_3 * X_5 - 93.46 * X_4 * X_5 + 0.0012 * X_1 * X_1 + 0.0014 * X_2 * X_2 + 0.0027 * X_3 * X_3 - 0.52 * X_4 * X_4 - 4.52 * X_5 * X_5 \quad (A4)$$

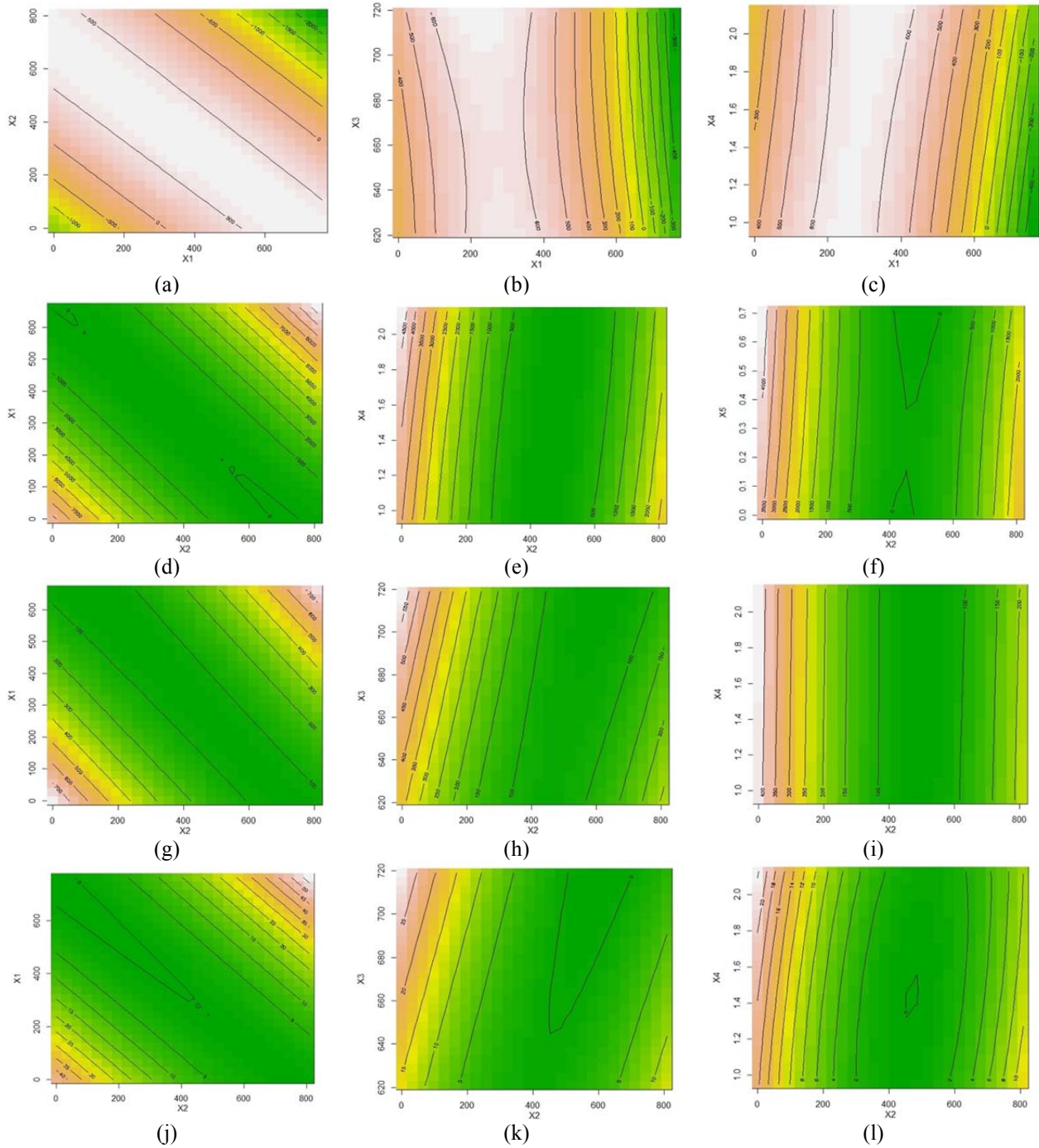
$$\hat{y}_{VSI} = -17.69 + 0.087 * X_1 + 0.0923 * X_2 - 0.089 * X_3 + 24.15 * X_4 - 1.7 * X_5 + 0.0002 * X_1 * X_2 - 0.0003 * X_1 * X_3 - 0.0081 * X_1 * X_4 + 0.0139 * X_1 * X_5 - 0.0003 * X_2 * X_3 - 0.0099 * X_2 * X_4 + 0.018 * X_2 * X_5 - 0.032 * X_3 * X_4 - 0.0024 * X_3 * X_5 - 7.26 * X_4 * X_5 + 0.0001 * X_1 * X_1 + 0.0001 * X_2 * X_2 + 0.0003 * X_3 * X_3 + 1.92 * X_4 * X_4 + 3.40 * X_5 * X_5 \quad (A5)$$

$$\hat{y}_{T_{50}} = 363.51 - 1.88 * X_1 - 0.86 * X_2 + 0.23 * X_3 - 116.27 * X_4 - 35.96 * X_5 + 0.002 * X_1 * X_2 - 0.0006 * X_1 * X_3 + 0.06 * X_1 * X_4 + 0.036 * X_1 * X_5 - 0.00097 * X_2 * X_3 + 0.058 * X_2 * X_4 + 0.024 * X_2 * X_5 + 0.023 * X_3 * X_4 - 0.011 * X_3 * X_5 + 15.04 * X_4 * X_5 + 0.0011 * X_1 * X_1 + 0.00095 * X_2 * X_2 + 0.00028 * X_3 * X_3 + 18.3 * X_4 * X_4 - 6.27 * X_5 * X_5 \quad (A6)$$

$$\hat{y}_{CS} = 2955.01 - 4.64 * X_1 - 3.83 * X_2 - 4.04 * X_3 - 359.89 * X_4 + 451.61 * X_5 + 0.0062 * X_1 * X_2 + 0.0003 * X_1 * X_3 + 0.038 * X_1 * X_4 - 0.197 * X_1 * X_5 - 0.00055 * X_2 * X_3 - 0.065 * X_2 * X_4 - 0.134 * X_2 * X_5 + 0.3 * X_3 * X_4 - 0.55 * X_3 * X_5 + 30.65 * X_4 * X_5 + 0.0032 * X_1 * X_1 + 0.0030 * X_2 * X_2 + 0.00305 * X_3 * X_3 + 61.61 * X_4 * X_4 - 51.69 * X_5 * X_5 \quad (A7)$$

$$\hat{y}_{28 \text{ days strength}} = 13865.99 - 28.85 * X_1 - 22.575 * X_2 - 14.805 * X_3 - 1160.41 * X_4 + 3381.129 * X_5 + 0.0438 * X_1 * X_2 - 0.00141 * X_1 * X_3 - 0.726 * X_1 * X_4 - 1.372 * X_1 * X_5 - 0.0102 * X_2 * X_3 - 0.966 * X_2 * X_4 - 0.968 * X_2 * X_5 + 1.459 * X_3 * X_4 - 3.782 * X_3 * X_5 - 13.448 * X_4 * X_5 + 0.0221 * X_1 * X_1 + 0.0215 * X_2 * X_2 + 0.0148 * X_3 * X_3 + 295.48 * X_4 * X_4 - 277.056 * X_5 * X_5 \quad (A8)$$

APPENDIX B—CONTOUR PLOT FOR PREDICTED RESPONSES OF SCC MIXTURE



Surrogate Modeling for Self-Consolidating Concrete Characteristics Estimation for Efficient Prestressed Bridge Construction

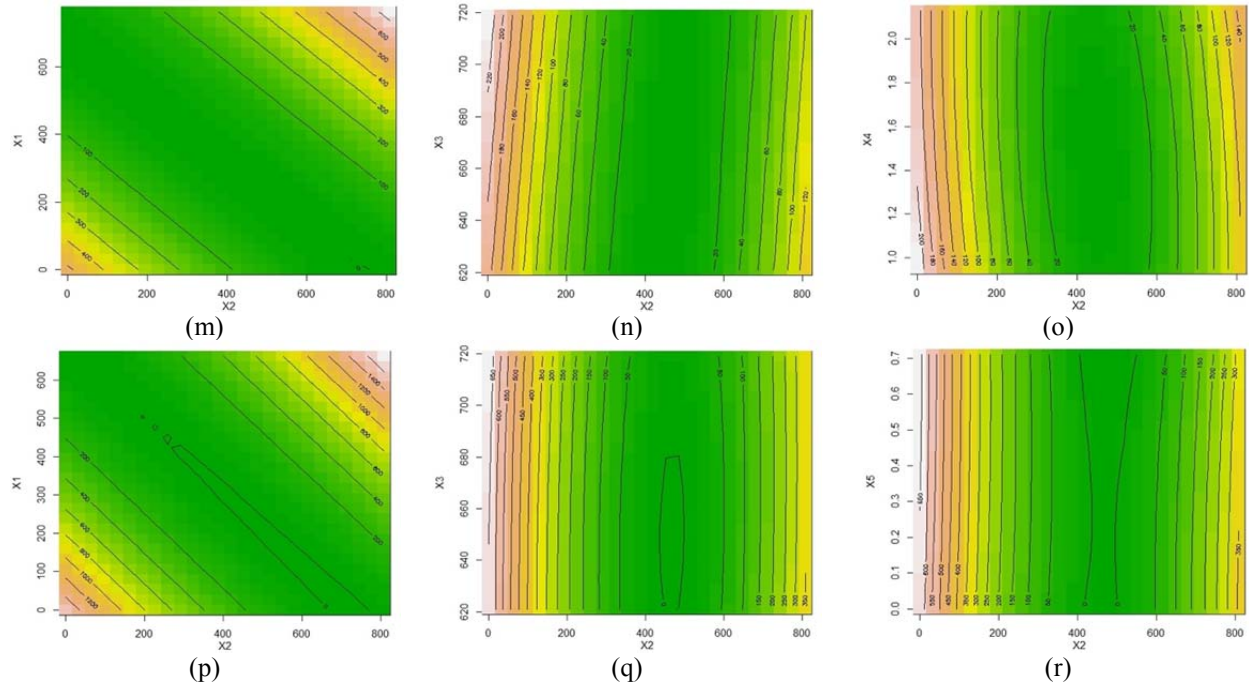
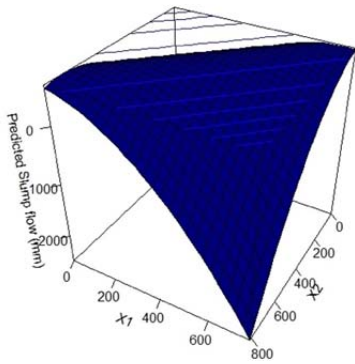
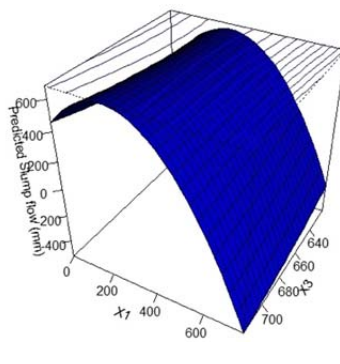


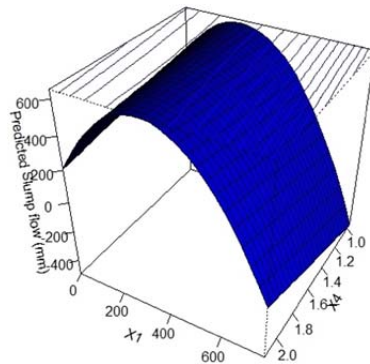
Figure B1—2D contour plot for the effect of significant input parameter on the fresh properties of SCC mixture: (a) X_1 and X_2 on slump flow; (b) X_3 and X_1 on slump flow; (c) X_4 and X_1 on slump flow; (d) X_1 and X_2 on passing ability; (e) X_2 and X_4 on passing ability; (f) X_2 and X_5 on passing ability; (g) X_1 and X_2 on filling capacity; (h) X_2 and X_3 on filling capacity; (i) X_2 and X_4 on filling capacity; (j) X_1 and X_2 on VSI; (k) X_2 and X_3 on VSI; (l) X_2 and X_4 on VSI; (m) X_1 and X_2 on T_{50} ; (n) X_2 and X_3 on T_{50} ; (o) X_2 and X_4 on T_{50} ; (p) X_1 and X_2 on column segregation; (q) X_2 and X_3 on column segregation; (r) X_2 and X_5 on column segregation.



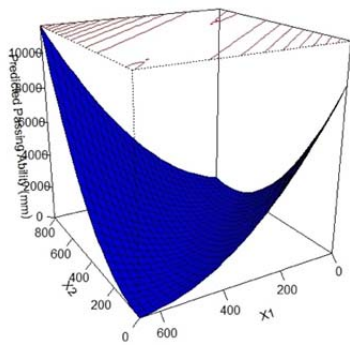
(a)



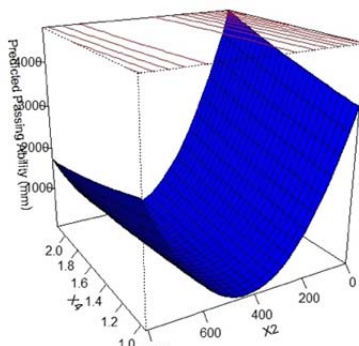
(b)



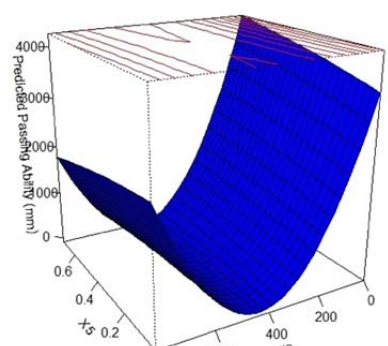
(c)



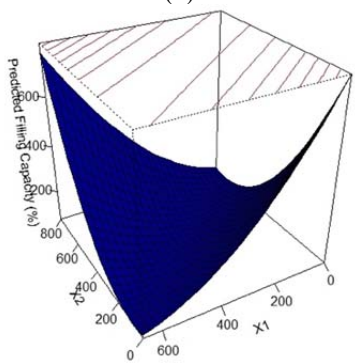
(d)



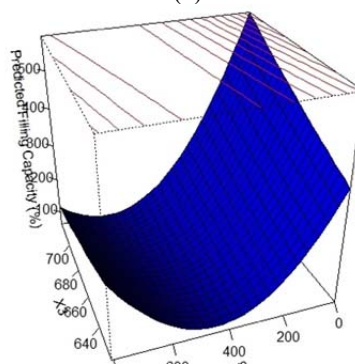
(e)



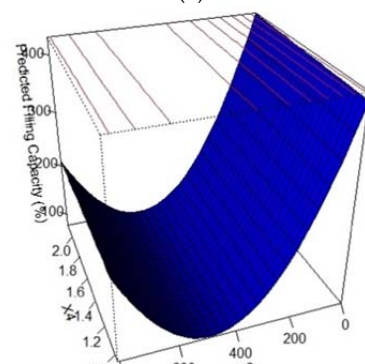
(f)



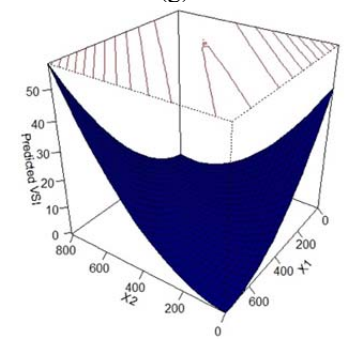
(g)



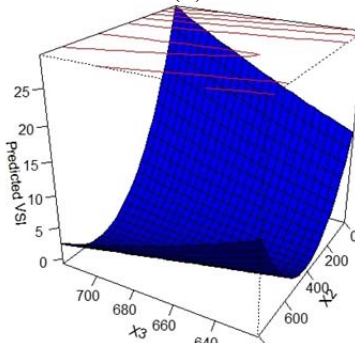
(h)



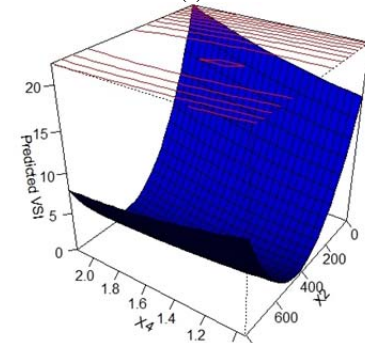
(i)



(j)



(k)



(l)

Surrogate Modeling for Self-Consolidating Concrete Characteristics Estimation for Efficient Prestressed Bridge Construction

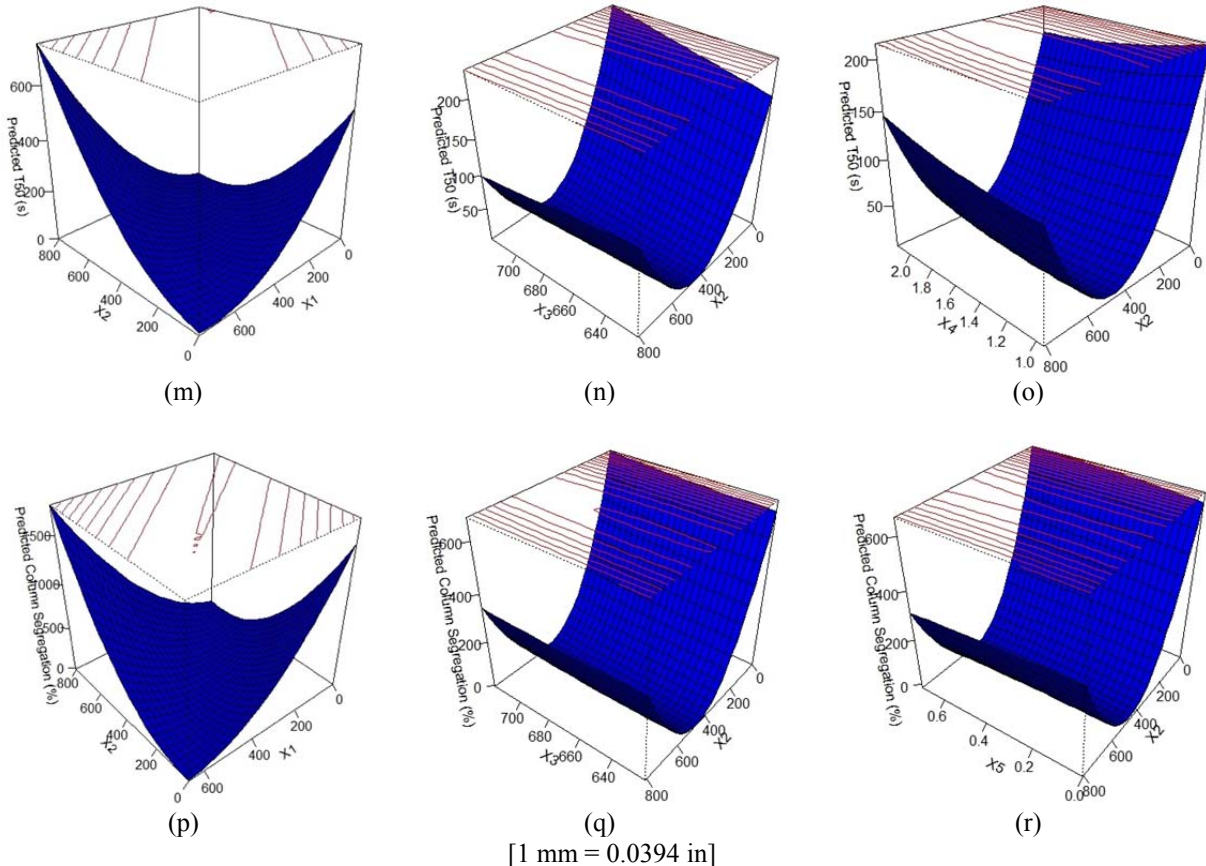


Figure B2—3D surface plot for the effect of significant input parameter on the fresh properties of SCC mixture: (a) X_1 and X_2 on slump flow; (b) X_3 and X_1 on slump flow; (c) X_4 and X_1 on slump flow; (d) X_1 and X_2 on passing ability; (e) X_2 and X_4 on passing ability; (f) X_2 and X_5 on passing ability; (g) X_1 and X_2 on filling capacity; (h) X_2 and X_3 on filling capacity; (i) X_2 and X_4 on filling capacity; (j) X_1 and X_2 on VSI; (k) X_2 and X_3 on VSI; (l) X_2 and X_4 on VSI; (m) X_1 and X_2 on T_{50} ; (n) X_2 and X_3 on T_{50} ; (o) X_2 and X_4 on T_{50} ; (p) X_1 and X_2 on column segregation; (q) X_2 and X_3 on column segregation; (r) X_2 and X_5 on column segregation.

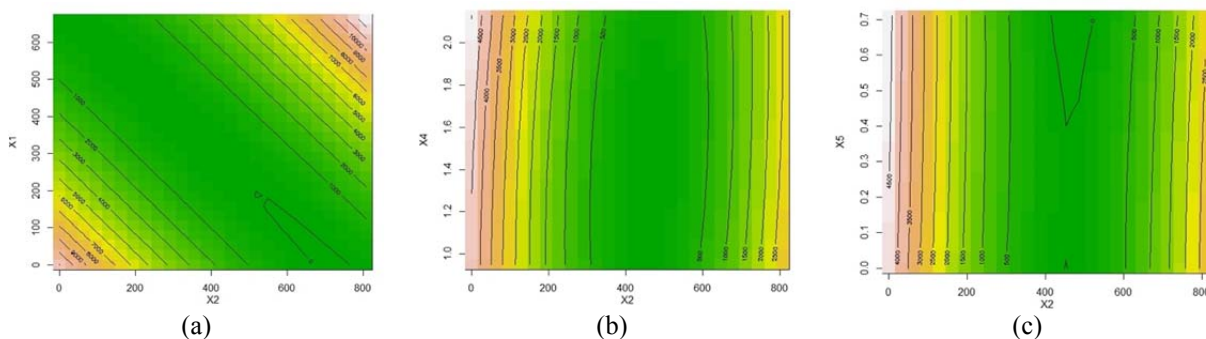
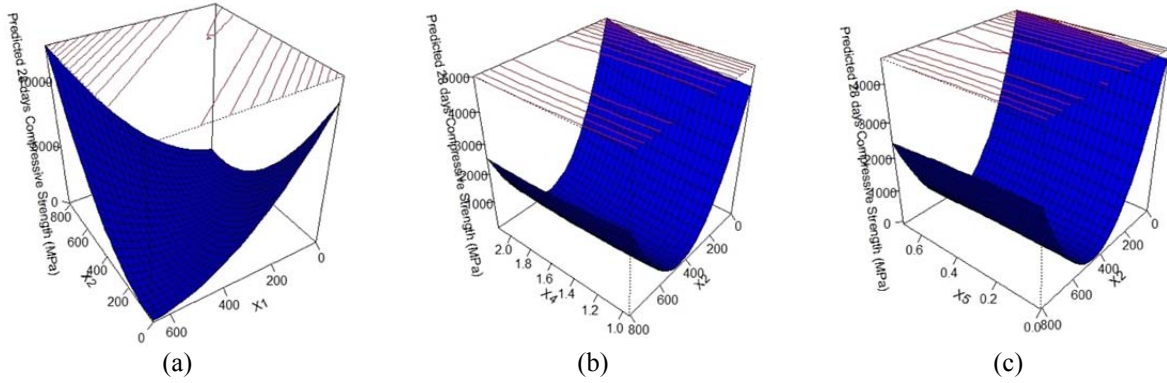


Figure B3—2D contour plot for the 28-days compressive strength values on other parameters: (a) X_1 and X_2 ; (b) X_2 and X_4 ; (c) X_2 and X_5 .



[1 MPa = 145.04 psi]

Figure B4—3D surface plot for the 28-days compressive strength values on other parameters: (a) X_1 and X_2 ; (b) X_2 and X_4 ; (c) X_2 and X_5 .

SP-333-3

On the Application of Basalt-Fiber Reinforced Polymer (BFRP) Bars to Prestressed Slab Elements Typical of the Precast Concrete Industry

Bruno Dal Lago, Davide Bisi and Liberato Ferrara

Synopsis: Basalt-Fiber Reinforced Polymer (BFRP) bars have been recently proposed to be used to prestress precast concrete elements. Mechanical properties, potential low production cost, low carbon footprint, and enhanced durability make the application of BFRP to prestressed concrete promising. Nevertheless, some issues related to anchorage and sustained stress still need to be fully addressed. Applications are so far limited to few laboratory tests. This paper discusses how the Serviceability Limit State (SLS) and Ultimate Limit State (ULS) checks of pre-stressed elements employing this technology vary with respect to elements pre-stressed with steel tendons. Furthermore, an attempt is made to investigate the potential application into the precast concrete industry, by analyzing several typical roof and floor slab elements with different cross-section. This investigation highlights which type of element could be more advantageously switched to the use of pre-stressed BFRP bars, and at which cost in terms of structural performance.

Keywords: basalt; creep; losses; nonlinear analysis; prestressing; serviceability; slab elements; structural design

Bruno Dal Lago, Ph.D., P.E., is a Research Associate and Adjunct Professor of Structural Analysis and Design at the Department of Civil and Environmental Engineering, Politecnico di Milano, Italy. He received his BS, MS, Specialization, and Ph.D., from Politecnico di Milano. His research interests include seismic behavior of precast concrete structures, experimental testing and numerical modeling of structures and mechanical connections, seismic effects and influence of non-structural components, innovation in dry-assembled precast structures and prestressing techniques in reinforced concrete and timber structures.

Davide Bisi is a Master student of Civil Engineering at Politecnico di Milano, where he received his BS. His research activity is orienting towards design of pre-stressed concrete elements and composite reinforcement.

ACI member **Liberato Ferrara**, Ph.D., P.E., is Associate Professor of Structural Analysis and Design at the Department of Civil and Environmental Engineering, Politecnico di Milano, Italy and was Fulbright visiting scholar at Northwestern University (USA). His research interests include material concept and structural applications of advanced cementitious composites, self-healing concrete, 3D printing with cementitious materials and sustainability of cement-based materials and structures. He is chair of ACI 544-FRC and coordinates a Horizon 2020 project on the application of nano-engineered HPFRCCs in extremely aggressive environments.

INTRODUCTION

The precast concrete industry is nowadays deeply involved into the worldwide challenges of sustainability and durability. A response to these issues must be provided, among the others, also at material level, through an effective reduction of the emissions of equivalent CO₂ involved in the production of both concrete and reinforcement. Some proposed solutions, including the use of stainless-steel tendons, only address the durability issue, whilst being hardly viable from an economic point of view. An effective solution alternative to the use of classic high-strength-steel tendons is provided by composite bars. In the '90s, composite reinforcement as a pre-stressing material was experimentally investigated with the use of the composite aramid [1,2], carbon [3,4] and glass bars [5] originally developed for non-prestressed reinforced concrete. More recently, a novel composite material was proposed in [6-8] to be used in different engineering fields, including civil engineering and reinforced concrete structures: basalt. Basalt, which is a natural fiber with diameter larger than the breathability critical level (which made the use of asbestos a crucial public health issue, being it below this level), is nowadays regarded as a promising material which could meet the challenges described above at a potentially affordable cost. The application of Basalt Fiber Reinforced Polymers (BFRP) bars into non-prestressed concrete was investigated experimentally and numerically, mainly focusing on the bending [9-12] or compression-bending [13] behavior of beam elements.

BFRP bars are characterized by a tensile strength ranging from 920 to 1650 MPa (133 to 239 ksi) and a relatively low Young modulus ranging from 45 to 59 GPa (6.5 to 8.6 msi). The high strength suggests that they could be used to prestress concrete elements. Furthermore, their low elastic modulus may imply that elastic and creep prestressing losses will be reduced by about four times with respect to steel. Because of this, few authors investigated the possible application of BFRP bars to prestressed concrete [14-18] through numerical and experimental works. In particular, the authors of [14,15] manufactured and tested up to failure a full-scale steel-free prestressed slab element. Following recent trends in structural engineering [19], the application of BFRP bars to prestress timber elements was also investigated [20].

Design codes for concrete elements prestressed by BFRP are not currently available, even though generic information related to FRP reinforcement in concrete is available in ACI 440.1R-15 [21] as well as in other guidelines [22,23]. In addition to the above cited papers focusing on the use of BFRP [15-18], the scientific literature provides information about the design of beams prestressed with other kinds of composite material reinforcement concerning flexure [24-26] and long-term deformation [27-33]. BFRP bars, due to their orthotropic behavior, are vulnerable in the area of the anchorage, where classic wedges apply high lateral pressures [15]. Special solutions were proposed in [34-37] for different composite bars, even though none of them has been yet proved to be applicable to a fully industrialized process. Furthermore, no information has been obtained and provided so far on the resistance of basalt fibers to sustained high stress. The similarity of its molecular structure with that of glass, which is susceptible to strength decay under sustained high stress [38-41], may suggest that further studies over this subject are needed before a safe implementation in the precast concrete industry can be fully exploited.

On the Application of Basalt-Fiber Reinforced Polymer (BFRP) Bars to Prestressed Slab Elements Typical of the Precast Concrete Industry

RESEARCH SIGNIFICANCE

This paper focuses on the applicability of BFRP bars to typical precast prestressed roof and floor slab elements. A full design process has been conducted on four slab elements prestressed with either steel tendons or with BFRP bars: two typically employed for roof decks (TT and wing-shaped) and two commonly used for floor slabs (hollow core and box cross section). A semi-analytical procedure based on a sectional approach for the evaluation of Serviceability Limit State (SLS) and a mechanically non-linear procedure for the evaluation of Ultimate Limit State (ULS) flexural behavior of fiber-reinforced concrete elements pre-stressed with BFRP bars have been used, whose predictions were previously validated through the comparison with experimental results [15,42]. The results provide a quantitative comparison of the structural performances associated to the replacement of traditional steel tendons with BFRP bars for each considered element.

SCOPE OF STUDY

The aims of this study are the following:

- to investigate and compare the non-linear cross-sectional behavior of several slab elements typical of the precast concrete industry production;
- to investigate and compare the structural performance at sectional level of these elements prestressed with steel tendons or BFRP bars;
- to find the maximum span of each element with ordinary or composite prestressing reinforcement through a full design flow accounting for bending, stress and deflection limit states;
- to compute and compare the elastic and long-term prestressing losses of each element with ordinary or composite prestressing reinforcement;
- to identify which element is the most suitable for the application of basalt-based composite reinforcement.

SLAB ELEMENTS ANALYZED

Four pretensioned slab elements are considered, two typical for roofing (TT and wing-shaped) and two typical for flooring (hollow core and box). TT and hollow core sections are typical worldwide, while wing-shaped and box sections are typical of southern European countries [42,43]. A box section with misaligned thick outer flanges is typical of the prefabrication market of the British Isles [15]. Both roof elements are about 1 m (3.28 ft) deep, and both floor elements are about 0.4 m (1.3 ft) deep, in order to perform a fair comparison. The geometry and the position of the prestressing reinforcement of each cross-section are described in Figure 1. The roof elements are supposed to be installed spaced at 5 m (16.4 ft) (center to center), with the clear spacing covered by thin barrel vault shells, whilst floor elements are supposed to be installed in adjacency (Figure 2). The prestressing reinforcement layout employed is that allowing for the longer span considering the application of steel tendons, designed according to the procedure described in the paper. The elements with BFRP have been designed with the same geometrical reinforcement ratio considering typical commercial sizes. Thus, steel tendons of diameters 15.24 mm (0.6 in.) and 12.70 mm (0.5 in.) were respectively substituted by BFRP bars of diameter 12 mm (0.47 in.) and 16 mm (0.63 in.). The reinforcement layout respects the EC2 prescriptions on the minimum concrete cover and the minimum bar spacing, assumed equal for both steel and BFRP.

MATERIAL PROPERTIES

The design of the elements analyzed in this paper is based upon the definition of their material properties over time. A class C70/85 concrete has been assumed. It is characterized by a cylindric compressive strength at 28 days equal to 70 MPa (10.15 ksi) and a cubic one of 85 MPa (12.33 ksi). Whilst the typical declared class of precast concrete is C45/55, this higher class has been chosen as representative of the real concrete obtained from the traditional mixes in Europe, mainly due to a careful production of concrete in high quality control batching plants and by the need of attaining a high strength after few hours from casting to carry on the daily shifts typical of an industrialized production process of prestressed elements. The considered mix is the one described in [44], characterized by self-compacting performance in its fresh state. Furthermore, the calculations have been carried out on the plain mix and on the same mix with polypropylene fibers with a dosage of 4 kg/m³ (0.25 lb/ft³), which were used together with the basalt prestressing reinforcement in [15] to provide toughness to the structural element after the potential failure of the basalt bars. In the latter mix, a residual tensile strength of 0.51 MPa (0.074 ksi) has been considered following the rules indicated in [45] after the peak of mean tensile strength at 4.60 MPa (0.667 ksi), evaluated according to fib-Model Code 2010 [45-47]. The average compressive strength of 76 MPa (11.02 ksi) [46] has been used in the definition of the non-linear stress-strain relationship of concrete shown in Figure 3a, based on the modified Sargin curve [47]. The material is considered failed under compression at a strain equal to 0.35%. The evolution of tensile