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Field Applications of Non-Conventional Reinforcing and Strengthening Methods for Bridges and Structures

Editors:

Yail J. Kim, Steven Nolan, and Antonio Nanni

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Field Applications of Non-Conventional Reinforcing and Strengthening Methods for Bridges and Structures

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PREFACE

Field Applications of Non-Conventional Reinforcing and Strengthening Methods for Bridges and Structures

A Sustainable built-environment requires a comprehensive process from material selection through to reliable management. Although traditional materials and methods still dominate the design and construction of our civil infrastructure, non-conventional reinforcing and strengthening methods for concrete bridges and structures can address the functional and economic challenges facing modern society. The use of advanced materials, such as fiber reinforced polymer (FRP) and ultra-high performance concrete (UHPC), alleviates the unfavorable aspects of every-day practices, offers many new opportunities, and promotes strategies that will be cost-effective, durable, and readily maintainable. Field demonstration is imperative to validate the innovative concepts and findings of laboratory research. Furthermore, documented case studies add value to the evaluation of emerging and maturing technologies, identify successful applications or aspects needing refinement, and ultimately inspire future endeavors. This Special Publication (SP) contains nine papers selected from three technical sessions held during the ACI Virtual Concrete Convention in fall 2020. The first and second series of papers discuss retrofit and strengthening of super- and substructure members with a variety of techniques; and the remaining papers address new construction of bridges with internal FRP reinforcing and prestressing in beam, slabs, decks and retaining walls. All manuscripts were reviewed by at least two experts in accordance with the ACI publication policy. The Editors wish to thank all contributing authors and anonymous reviewers for their rigorous efforts. The Editors also gratefully acknowledge Ms. Barbara Coleman at ACI for her knowledgeable guidance.

Yail J. Kim, Steven Nolan, and Antonio Nanni

Editors

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Application of UHPC 2.0™ for a Non-Conventional Reinforcing and Strengthening of a Reinforced Concrete Beam for Bridges and Structures

Peter W. Weber and Su Wang

Synopsis: Conventional reinforcing and strengthening methods and material for bridges and structures has several limitations including include the increased weight of structure, the limited service life of the repair, short periods between repairs, uncertain strength of the reinforcement, extended time of repair and typically a heavy carbon footprint based on the materials used. Application of Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) solutions have shown the potential to replace traditional methods over the coming decade because the superior mechanical and durability properties reduce the required thickness of a repairing layer and extend the service life. Based on the overall cost of a given rehabilitation project, UHPFRC based solutions can already compete today but require certain specialized equipment and trained workforce creating real or perceived barriers. In this paper, a new type of nano-engineered UHPFRC based on carbon-nanofibers (CNFs) was introduced, named UHPC 2.0™. The test results show that UHPC 2.0™ possesses ultra-high mechanical properties, improved direct tension performance and durability. In addition, an analytical procedure is provided for case studies to show the performance and economic benefits of usage of UHPC 2.0™ compared to traditional UHPFRC.

Keywords: direct tension; durability; mechanical properties; nano-engineered binder; UHPFRC

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INTRODUCTION

The ultra-high performance fiber reinforced concrete (UHPFRC) was developed in the mid-1990s and characterized by a high amount of binder (cement, pozzolana and reactive powders), low water-to-binder ratio (normally 0.2), high packing density of fine ingredients and high volume fractions of steel fibers ($> 2\%$ by volume) [1, 2]. These characteristics create a concrete with superior compressive strength (17~29 ksi, 120~200 MPa), tensile strength (1.0~2.2 ksi, 7~15 MPa), deformation (0~5%) with unique strain-hardening behaviour, energy absorption capacity and durability [3]. The outstanding mechanical properties of UHPFRC resulted in a significant reduction in the volume and size of structures made from such type of concrete. Compared to conventional concrete, usage of UHPFRC not only reduces the overall construction costs but also provides a greener solution. Meanwhile, the high durability properties of UHPFRC address the problems of structures under severe exposure, resulting in less maintenance and long service life. Nowadays, there is a surge in applying UHPFRC to repairing of concrete elements (bridge decks, pavement, panel, etc.), element connection/joint grouting and manufacture of architectural facade panel, structural wind towers elements, tall buildings, and long-span bridges. Although the technology and development of UHPFRC have been well investigated and proved from micro- to macro-level, it is still difficult to promote UHPFRC widely due to the challenges of complicated production process and high cost. The complicated production process mainly results from too many categories of ingredients used (up to 15), which leads to difficult handling. Thus, the performance of produced UHPFRC is highly dependent on the quality of contractors and workers. Steel fibers content is another critical factor to control the cost of UHPFRC. Increasing the steel fiber content from 1.5 to 2.5% by volume could improve the tensile strength and strain capacity of UHPFRC from 1.2 to 2.0 ksi (8 to 14 MPa) and 1.7 to 2.4% (for straight steel fibers) [4]. Decreasing the volume of steel fibers without degrading the mechanical properties of UHPFRC is the key to reducing cost. Present UHPFRC needs high temperature and pressure curing treatment to increase performance. This process not only increases the cost of UHPFRC but also consumes high energy which is not environmentally friendly. In this study, a new type of nano-engineered UHPFRC based on carbon-nanofibers (CNFs) was introduced, named UHPC 2.0TM. The mechanical properties, especially the direct tension performance, and durability of UHPC 2.0TM were determined experimentally according to the standard methods. In addition, an analytical procedure is provided for case studies to investigate the bending performance of UHPC 2.0TM in repairing reinforced concrete beam compared to traditional UHPFRC.

RESEARCH SIGNIFICANCE

The present research introduces a new type of nano-engineered UHPFRC based on carbon-nanofibers (CNFs), named UHPC 2.0TM, for non-conventional reinforcing and strengthening of a reinforced concrete beam for bridges and structures. The modification and enhancement of microstructure by nanomaterials result in the outstanding mechanical properties, superior direct tensile performance and extraordinary durability properties of UHPC 2.0TM. By employing UHPC 2.0TM on bridge repairing and strengthening, the sizes and weight of elements could be reduced resulting in equal or lower construction costs compared to conventional methods due to the excellent strength and toughness. The lifetime under severe exposure of repaired structures would be also extended because of the high durability of UHPC 2.0TM. In general, UHPC 2.0TM provide a new solution with ideal engineering properties, easy handling, environment friendly and reduced cost for rehabilitation project of bridge structures.

MIX DESIGN

The mix design of UHPC 2.0TM was shown in Table 1. Graded quartz sands with a particle size of 100~600 μm (0.00039~0.024 in.) were used as fine aggregates. An optimal packing density of cement and quartz sands was achieved by considering the modified Andreasen and Andersen model [5]. A new type of CNF was produced by ceEntek based on catalytic vapor deposition method. The CNF has diameters of 10~100 nm (3.9×10^{-8} ~ 3.9×10^{-6} in.) and fiber lengths of 10~20 μm (0.00039~0.00079 in.). SEM images of the CNFs are shown in Fig. 1. The dispersion

process is through ultrasonication and the CNFs colloid still appears in stable homogenous form after 3 days due to the high surface zeta potential (+33mV). This makes it possible for CNFs to be dispersed in water through a physical process without the addition of chemical modification [6]. Standard steel fibers with a length of 0.5 in (13 mm) length, 0.008 in (0.2 mm) diameter and tensile strength of greater than 363 ksi (2500 MPa) have been used. All specimens were demoulded after 1 day of casting and water-cured at approximately 23°C (73°F).

EXPERIMENTAL PROGRAMME OF UHPC 2.0™

Mechanical properties

The compressive strength at 1, 7 and 28 days were tested according to BS EN 12390-3 [7] on 4 in (100 mm) standard cubes. Flexural tests at 7 and 28 days were measured on 1.6×1.6×6.3 in (40×40×160 mm) prisms based on ASTM C348 [8]. The 28-day elastic modulus tests complied with BS EN 12390-13 [9] and the test specimens were Φ4×8 in (Φ100×200 mm) cylinders. Direct tension tests were conducted by an MTS hydraulic testing machine with 56,200 lbs (250 kN) capacity in accordance with Swiss Standard SIA 2052 [10]. Two linear variable differential transducers (LVDTs) were attached to both sides of the dog-bone specimens to monitor the deformation of the sample with a gauge length of 4.7 in (120 mm).

Durability properties

The durability of UHPC 2.0™ was evaluated by chloride ion penetration and water absorption. Measurement of 28-day chloride ion penetration was conducted by rapid chloride penetration tests on Φ4×2 in (Φ100×50 mm) disks according to ASTM C1856-17 [11] and ASTM C1202-12 [12]. To avoid the interference of electrical conductivity of steel fibers to the results, the samples used for testing were without steel fibers. The 28-day water absorption was evaluated by ASTM C642-13 [13] and the samples are 100 mm cubes.

RESULTS AND DISCUSSION

Mechanical properties

The compressive strength development with the curing time of UHPC 2.0™ is presented in Fig. 3. The high packing density of the material facilitates a high compressive strength at different curing ages. Meanwhile, the filler function and more uniform hydration process of CNFs can further reduce the size and volume of capillary pores and micro voids to improve compressive strength [14], especially in the interfacial transition zone (ITZ) [15]. Moreover, a stable dispersion of CNFs is critical to the improvement of strength [6] due to reduced CNF-free volume of the base mix. The 7- and 28-day flexural strengths of UHPC 2.0™ are shown in Table 2. The ultra-high flexural strength is attributed to the high packing density of matrix which increases the shear capacity between sand particles and cement paste [16] and the improved bond of the paste to aggregates and fibers from the inclusion of CNF. Furthermore, the flexural strength is also increased by CNFs because the nanofibers work as bridges to delay the initiation and reduction of the propagation of nano- and micro-cracks [6]. The 28-day elastic modulus of UHPC 2.0™ is also listed in Table 2. Low water/cement ratio, higher packing density and increased density of the interstitial zone of UHPC 2.0™ significantly reduces the porous phase with low elastic modulus and increases the unhydrated phase with high elastic modulus. Moreover, CNFs will further reduce the amount of porous phase due to the filling effect and an increase in the stiffness of C-S-H [17].

The relationship between tensile stress and strain is shown in Fig. 3. As a type of UHPFRC, the direct tension performance of UHPC 2.0™ was evaluated according to SIA 2052 [10]. SIA 2052 [10] requires three critical parameters: elastic limit stress (f_{ute}), ratio between ultimate tensile stress and elastic limit stress (f_{utu}/f_{ute}), and strain at ultimate tensile stress (ϵ_{utu}), which are defined by three different categories. These three categories in ascending order are UO, UA, and UB. The comparison between testing results and requirements in SIA 2052 [10] is shown in Table 3. A comparison of the test data to the requirements shows that UHPC 2.0™ exceeds all of the requirements. Even for the highest category defined by UB, UHPC 2.0™ can be characterized by high tensile stress, high tensile strength, and high strain hardening. The highly dispersed CNFs could effectively improve the bond performance of steel fibers by densifying the microstructure and bridging nano-cracks in the ITZ between steel fibers and surrounding matrix. The increased interface frictional bond performance improved the fiber-bridging and energy absorption capacity resulting in high tensile strength and strain hardening. The outstanding direct tensile performance of UHPC 2.0™ can reduce the size of structural elements and amount of reinforcement to create equal or lower construction costs compared to conventional concrete material. Furthermore, the reduced self-weight of material eliminates or limits the strengthening of the structures and foundations, which results in significant economic effects.

Durability properties

The Rapid chloride penetration test results UHPC 2.0™ without steel fibers are shown in Table 4 and conducted in accordance with ASTM C1856/C1856M-17 and ASTM C1202-12 [12]. The results were evaluated based on the classification in ASTM C1202 (Table 5) [12] and UHPC 2.0™ was defined as very low Chloride ion permeability. The water absorption of UHPC 2.0™ is shown in Table 6. The measured values are very low and meet the requirements of Category DL200 in CSA A23.1-19 Annex U [18], which is the top category. According to Mindess *et al.* [19], water in capillary pore size from 10 nm to 10 µm is sensitive to permeability and the range of CNFs diameters used in this study falls into this range. CNFs enhanced and produced a denser matrix to eliminate capillary pores which contain evaporable bulk water [19].

Analytical procedure and case study

The bending performance of a UHPC 2.0™ repaired Reinforced Concrete (RC) beam can be evaluated based on the direct tensile strength. At the negative moment point of a UHPFRC repaired RC beam, the UHPFRC layer is in tension while the bottom of the RC beam is in compression. The ultimate bending moment of the composite UHPFRC-RC cross section can be calculated according to the conventional sectional model with the extension to account for the UHPFRC layer as shown in Fig. 4 [20]. At the ultimate limit state, the bottom surface of the original concrete beam is crushed, in other words, the concrete strain at bottom surface has reached the ultimate value ϵ_c . At the same time, for a tension failure mode, the steel reinforcement in the UHPFRC layer is yielding. According to Fig. 4 and the force equilibrium, the depth of compressive zone x can be calculated according to following equations:

$$f_{utu}bh_u + f_{su}A_{su} + \sigma_{sc}A_{sc} = 0.85x \cdot f_c b \quad (1)$$

$$\sigma_{sc} = E_{sc}\epsilon_c \cdot \frac{d_{sc}-x}{x} \quad (2)$$

where b is the width of RC beam cross section; h_u is the thickness of UHPFRC layer; f_{utu} is the tensile strength of UHPFRC; f_{su} is the yield strength of steel reinforcement at UHPFRC layer; A_{su} is the steel reinforcement area at UHPFRC layer; σ_{sc} is the tension stress at the top steel reinforcement of the original concrete beam; A_{sc} is the steel bar area of the top steel reinforcement of the original concrete beam; E_{sc} is the elastic modulus of the top steel reinforcement of the original concrete beam; d_{sc} is the distance between the top steel reinforcement and bottom surface of the original concrete beam; and f_c is the compressive strength of original concrete. The obtained depth of compressive zone x was used to calculate σ_{sc} and if σ_{sc} is less than the yield strength of the top steel reinforcement f_{sc} , the ultimate bending moment of the composite UHPFRC-RC cross section M_u can be calculated as follows

$$M_u = f_{utu}bh_u(h + h_u - 0.425x - \frac{h_u}{2}) + f_{su}A_{su}(d_{su} - 0.425x) + \sigma_{sc}A_{sc}(d_{sc} - 0.425x) \quad (3)$$

where h is the height of the beam cross section; and d_{su} is the distance between the steel reinforcement at UHPFRC layer and bottom surface of the original concrete beam. If $\sigma_{sc} \geq f_{sc}$, then the depth of compressive zone x should be re-calculated by following Equation (4) and M_u can be obtained by inputting this x value into Equation (3).

$$x = (f_{utu}bh_u + f_{su}A_{su} + f_{sc}A_{sc})/(0.85f_{cd}b) \quad (4)$$

Malena and Eugen [21] conducted a series of beam tests to study the contribution of UHPFRC strengthening layers to RC beams. In this paper, the beam sizes and material properties from [21] were used as case studies to compare the performance between UHPC 2.0™ and traditional UHPFRC repairing RC beam based on the proposed analytical procedure. The characteristic values of tensile strength of UHPC 2.0™ and traditional UHPFRC used for analysis is 2.3 ksi (16 MPa) and 1.5 ksi (10 MPa) respectively. The targets of the case studies are to evaluate how much thickness of UHPFRC layer and steel reinforcement ratio at the UHPFRC layer can be reduced by using UHPC 2.0™ when the bending resistance is kept the same. Table 7 shows Case 1 that the thickness of UHPC 2.0™ h_u is 40% lower than that of traditional UHPFRC when $\rho_{su} = 0\%$ (no steel reinforcement at the UHPFRC layer). Table 8 shows Case 2 that h_u of UHPC 2.0™ is 36% lower than that of traditional UHPFRC when $\rho_{su} = 2.70\%$. Table 9 shows Case 3 that ρ_{su} of UHPC 2.0™ is 32% lower than that of traditional UHPFRC when $h_u = 50$ mm. It can be found that the ultra-high tensile strength of UHPC 2.0™ can significantly reduce the thickness of the UHPFRC layer and the steel reinforcement ratio at the UHPFRC layer.

CONCLUSION

As a type of UHPFRC, UHPC 2.0™ shows ultra-high mechanical properties including 25 ksi (172 MPa) compressive strength, 6.5 ksi (45 MPa) flexural strength, 2.6 ksi (18 MPa) tensile strength, and 7100 ksi (49 GPa) elastic modulus at 28 days. The improved direct tension performance satisfies the highest requirements for both strength and ductility as defined in SIA 2052 [10]. Low values of chloride ion penetration and water absorption reflect the excellent durability of UHPC 2.0™. The case study highlights up to 40% reduction in the amount of material used in the bridge RC beam repair work. Use of UHPC 2.0™ can lead to equal or lower construction costs compared to conventional methods due to less material resulting in a greener solution. The reduced dead load eliminates or limits the strengthening of the structures and foundations. The extended lifetime of UHPC 2.0™ addresses today's durability problems of structures under severe exposure resulting in less maintenance and long service life.

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Table 1—Mix design of UHPC 2.0™ (unit: a relative portion in mass)

Compositions	Binder	Graded Sands	Water	SP	CNFs	Steel fibers
UHPC 2.0™	1.00	0.79	0.19	0.0052	0.0006	0.13

Table 2—Flexural strength and elastic modulus of UHPC 2.0™

Curing ages	7 days	28 days
Flexural strength (ksi/MPa)	6.1 (0.2*)/42.0 (1.6*)	7.6 (0.4*)/52.2 (2.5*)
Elastic modulus (ksi/GPa)	5613 (58*)/38.7 (0.4*)	7107 (87*)/49.0 (0.6*)

* represents standard deviation.

Table 3—Comparison of the requirements of SIA 2052 for types U0, UA and UB with measured values in the direct tensile test

Parameter	Required minimum value			Measured value			Requirements for		
	U0	UA	UB	Sample 1	Sample 2	Sample 3	U0	UA	UB
f_{ute} , ksi (MPa)	≥ 1.0 (7.0)	≥ 1.0 (7.0)	≥ 1.5 (10.0)	2.1 (14.3)	2.1 (14.3)	2.1 (14.7)	Fulfilled	Fulfilled	Fulfilled
f_{utu} , ksi (MPa)	≥ 0.7 (4.9)	≥ 1.1 (7.7)	≥ 1.7 (12)	2.6 (18.1)	2.7 (18.3)	2.6 (17.8)	Fulfilled	Fulfilled	Fulfilled
f_{utu}/f_{ute}	> 0.7	> 1.1	> 1.2	1.27	1.28	1.21	Fulfilled	Fulfilled	Fulfilled
ϵ_{utu} (%)	-	> 0.15	> 0.2	0.23	0.21	0.51	Fulfilled	Fulfilled	Fulfilled

Table 4—Rapid chloride penetration test results of UHPC 2.0™ without steel fibers

	Sample 1	Sample 2	Sample 3	Average	Classification
Charge passed (Coulombs)	205	186	211	200	Very low

Table 5—ASTM C1202 classification

Charge passed (Coulombs)	> 4000	2000 - 4000	1000 - 2000	100 - 2000	< 100
Chloride ion permeability	High	Moderate	Low	Very low	Negligible

Table 6—Water absorption test results of UHPC 2.0™

	Sample 1	Sample 2	Sample 3	Sample 4	Average
Absorption after immersion %	0.7	0.65	0.55	0.60	0.625

Table 7—Case 1: h_U reduction by UHPC 2.0™ when $\rho_{sU} = 0\%$

	b, in (mm)	h, in (mm)	f_c , ksi (MPa)	f_{sc} , ksi (MPa)	ρ_{sc} (%)	f_{utu} , ksi (MPa)	h_U , in (mm)
Traditional UHPFRC	5.9	9.8	7.3	82	0.66	1.5 (10)	2 (50)
UHPC 2.0™	(150)	(250)	(50)	(566)		2.3 (16)	1.2 (30)

Table 8—Case 2: h_U reduction by UHPC 2.0™ when $\rho_{sU} = 2.70\%$

	b, in (mm)	h, in (mm)	f_c , ksi (MPa)	f_{sc} , ksi (MPa)	ρ_{sc} (%)	f_{sU} , ksi (MPa)	ρ_{sU} (%)	f_{utu} , ksi (MPa)	h_U , in (mm)
Traditional UHPFRC	5.9	9.8	7.3	82	0.66	103	2.70	1.5 (10)	2 (50)
UHPC 2.0™	(150)	(250)	(50)	(566)		(710)		2.3 (16)	1.3 (32)

Table 9—Case 3: ρ_{sU} reduction by UHPC 2.0™ when $h_U = 2$ in (50 mm)

	b, in (mm)	h, in (mm)	f_c , ksi (MPa)	f_{sc} , ksi (MPa)	ρ_{sc} (%)	f_{sU} , ksi (MPa)	h_U , in (mm)	f_{utu} , ksi (MPa)	ρ_{sU} (%)
Traditional UHPFRC	5.9	9.8	7.3	82	0.66	103	2 (50)	1.5 (10)	2.70
UHPC 2.0™	(150)	(250)	(50)	(566)		(710)		2.3 (16)	1.83

where b is the width of RC beam cross section; h is the height of RC beam cross section; f_c is the compressive strength of original concrete; f_{sc} is the yield strength of the top steel reinforcement in RC beam; ρ_{sc} is the top steel reinforcement ratio in RC beam; f_{sU} is the yield strength of steel reinforcement at UHPFRC layer; ρ_{sU} is the steel reinforcement ratio at UHPFRC layer; h_U is the thickness of UHPFRC layer

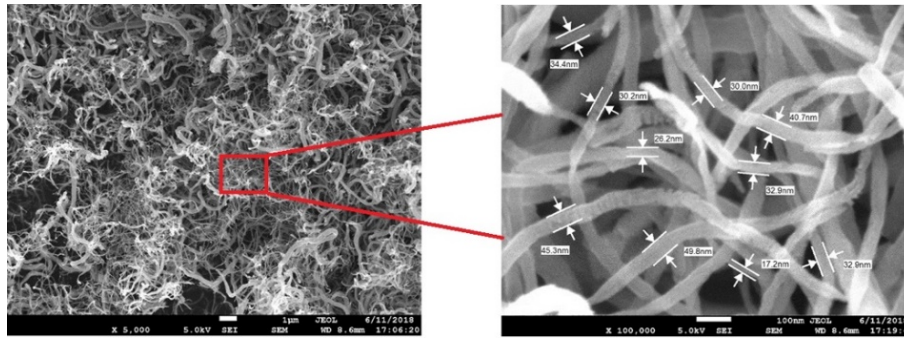


Fig. 1—SEM pictures of CNFs

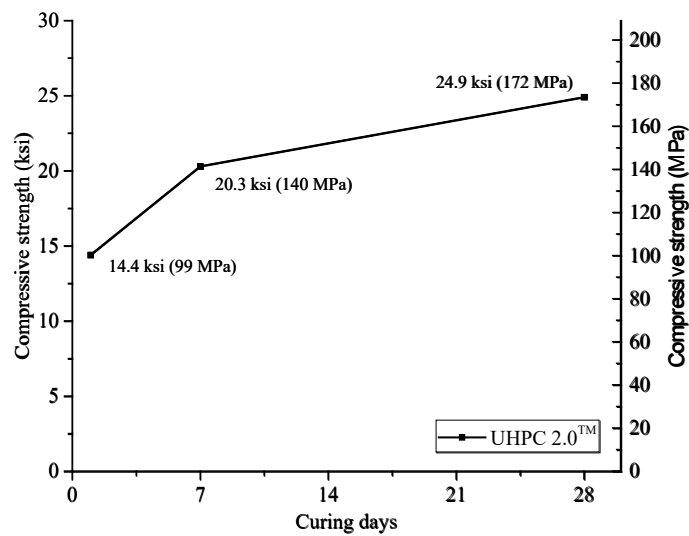


Fig. 2—Compressive strength development of UHPC 2.0TM

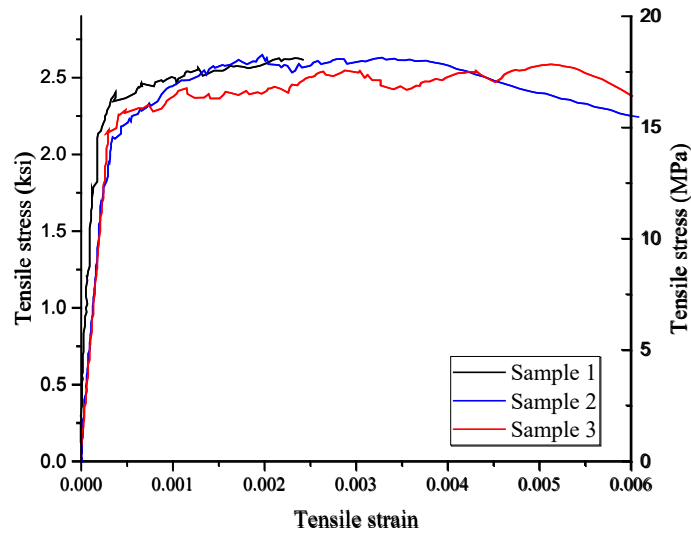


Fig. 3—Relationship between tensile stress and strain of UHPC 2.0TM

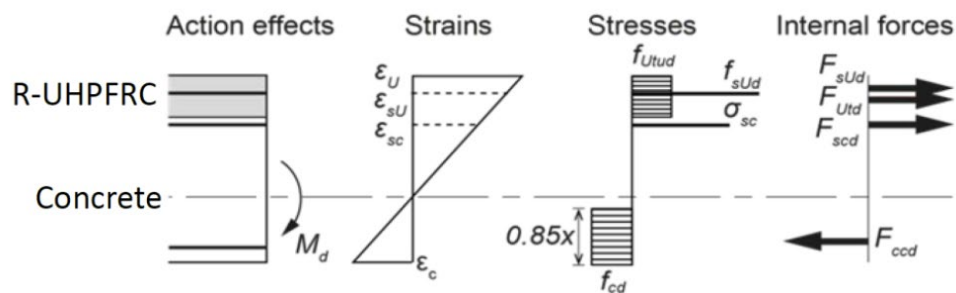


Fig. 4—Bending failure mechanisms of composite UHPFRC-RC beam [20]

FRP Retrofitting and Non-Destructive Evaluation for Corrosion-Deteriorated Bridges in West Virginia

Wael Zatar, Hai Nguyen, and Hien Nghiem

Synopsis: Many aging concrete bridges across the United States have exhibited severe deteriorations and in urgent need of rehabilitation, retrofitting or replacement. The deterioration is caused by a combination of factors including corrosion of reinforcing steel, freeze-thaw damage and chloride/water ingress. Fiber-Reinforced Polymer (FRP) composite fibers, laminates, reinforcing bars and prestressed tendons have been successfully employed in civil infrastructure applications in the past three decades. The State of West Virginia has one of the highest percentages of structurally deficient bridges in the United States and this study covers a few case studies of the use of FRP composites for rehabilitating the State's deficient bridges. Non-destructive ultrasonic pulse-echo testing is employed to map reinforcing rebars and detect delaminations of reinforced concrete slabs. A software, that employs the modified synthetic aperture focusing technique (SAFT) image reconstruction algorithm and signal processing, is developed to effectively visualize the reinforcing rebars and delaminations.

Keywords: concrete bridges; concrete imaging; deterioration; FRP composites; non-destructive testing and evaluation; structural rehabilitation; ultrasonic pulse-echo

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INTRODUCTION

According to the ASCE infrastructure report card [1], many constructed facilities in the United States such as bridges, roads, dams, levees, and other infrastructures (e.g. aviation, rail, ports, transit, etc.) have been deteriorated. For example, 9.1% of the nation's bridges were structurally deficient in 2016 and millions of trips are taken every day across these structurally deficient bridges. Major contributors to such deterioration include, but not limited to, water, ultraviolet radiations, temperature, freeze-thaw cycles, traffic and impact loading, and chemicals [2]. Proper inspection, maintenance, and rehabilitation, therefore, play critical roles in the sustainability of America's infrastructure and in the nation's economy and public safety. FRP composite materials have been increasingly used worldwide in structural strengthening and repair applications due to their high strength-to-weight ratios, high stiffness-to-weight ratios, superior environmental durability, anti-corrosion, favorable maintenance/labor costs, tailored properties, ease of handling and installation, and rapid construction [3-6]. Major FRP bridge applications include FRP deck/panel (e.g. sandwich construction and adhesively bonded pultrusions), non-prestressed and prestressed internal FRP reinforcement (e.g. FRP rebar/grid, FRP cable/tendon), externally bonded reinforcements (e.g. precured strips or uncured sheets for flexural/shear strengthening of structural elements such as beams, slabs, columns, shear walls, joints, trusses, etc.). FRP materials have also been used to enhance confinements in structurally deficient reinforced-concrete columns and masonry/timber/metal structures. West Virginia has been known as one of the pioneer states to deploy FRP composites in construction. Out of 220 bridges nationwide with the FRP composites, 35 bridges are in the mountaineer state of West Virginia [7]. West Virginia Department of Transportation (WVDOT) has employed FRP composites for bridge construction and rehabilitation since 1996. Many FRP-bridge projects (FRP decks or FRP rebar/grid-reinforced-concrete decks) were constructed during the 1996-2004 period (Table 1). The FRP composites have been applied to strengthen or retrofit beams/girders, slabs, bents, columns/piles/pier caps, and abutments/footings. A list of major FRP-retrofitted bridge projects in West Virginia can be found elsewhere [8]. The first part of this paper provides an overview of bridge types and conditions in the state of West Virginia and gives two examples of FRP-strengthened bridges in West Virginia. The second part of the paper will address the non-destructive testing technique in detecting subsurface defects and mapping reinforcing rebars in concrete structures.

RESEARCH SIGNIFICANCE

The use of externally-bonded fiber-reinforced polymers has recently been widely accepted to retrofit reinforced concrete (RC) bridge members. Complete sets of drawings for some of these older aging RC bridges may not always be available, thus hindering accurate condition assessment and production of FRP-retrofit designs. Advanced non-destructive testing and evaluation methods could potentially assist addressing data unavailability for aging structural members. The breakthrough ultrasonic pitch and catch technology is used in this study to locate rebars and subsurface defects. By employing a progressive signal processing technique, the authors produced a software that accurately generates high-resolution images. The research deliverables assist capturing data necessary to reproduce structural drawings and generate FRP-retrofit designs for aging RC bridge structures in West Virginia.

BRIDGE CONDITIONS AND FRP-REHABILITATED BRIDGE PROJECTS IN WEST VIRGINIA

Overview of Bridge Types and Conditions in West Virginia

According to the 2017 National Bridge Inventory (NBI) database, West Virginia has 7,228 highway bridges and 19% of these bridges (1,372 bridges) were rated as structurally deficient (SD). In addition, 1,394 bridges (19.3%) were rated as functional obsolete (FO). As shown in Table 2, the bridge type (categorized based on bridges' main structure type) with a large population of SD bridges includes slab type (32.5% deficient out of 517 bridges), girder and floorbeam system type (46.7% out of 229 bridges), tee beam type (47.1% out of 104 bridges), truss – thru type (43.3% out of 180 bridges), arch – deck type (39.8% out of 399 bridges), channel beam type (54.8% out of 115 bridges).

Examples of FRP-Rehabilitated Bridges in West Virginia

This section briefly introduces two aging bridges in West Virginia retrofitted by the FRP composites. The FRP wraps externally bonded to the concrete surface to compensate for strength loss due to corrosion, deterioration, or fire/impact damage. The use of FRP wraps allows the rehabilitation of the existing concrete, resulting in an economic repair as substructure replacement generally requires replacing the entire bridge. These repairs have saved the WVDOT thousands of dollars compared to conventional repairs.

East Street Viaduct Bridge (carries CSX railroad track)—East Street Viaduct bridge is located 0.14 mi (0.23 km) south of WV 618 in Parkersburg, West Virginia (Wood County, District 3). This reinforced concrete (RC) bridge was built in 1907 with two traffic lanes under and 6,748 average daily traffic (as of 2015). It has a single span with a span length of 23.3 ft (7.1 m) and a total length of 64.7 ft (19.72 m). According to WVDOT 2017 bridge inspection report, the bridge substructure is composed of unreinforced concrete full-height abutments, RC wing walls, and bents (which are constructed of ten vertical riveted steel columns with concrete footings and a riveted steel box beam cap). The bridge superstructure consists of 3 ft (0.91 m) RC top slab, which carries 2.5 ft (0.76 m) of railroad slag fill and ten sets of railroad tracks. In 2001, the bridge was rehabilitated with GFRP wrapping of the abutments, wing walls, concrete bases of the bents, and top slab. In July 2012, WV State Forces, touched up the FRP on the headwall above the southbound lane with fiberglass repair kit, applied paint to GFRP areas showing wear, and repaired the weep drains.

Flag Run Bridge—Flag Run bridge is located 0.03 mi (0.05 km) north of county route 72/6 in Preston county (District 4), West Virginia. This reinforced concrete (RC) bridge was built in 1940 with two lanes of traffic and 650 average daily traffic (as of 2014). It has a single span with a total length of 43.2 ft (13.2 m) and a span length of 40 ft (12.2 m). According to WVDOT 2016 bridge inspection report, the bridge superstructure consists of four RC T-beams (33 in. [838.2 mm] high and 16.5 in. [419.1 mm] wide) topped with cast-in-place RC slab and supported by two full-height concrete abutments. The entire bottom face and side faces at both ends of T-beams were wrapped with CFRP composites in 2002 to achieve an HS-25 design loading. The HS-25 design loading is 20% higher than the HS-20 loading, which is defined as the HS20 truck consisting of two 32-kip axles, varied between 14 ft and 30 ft apart, to produce extreme force effects [9]. Abutments were also wrapped with CFRP and the backwalls were patched.

NON-DESTRUCTIVE TESTING OF REINFORCED-CONCRETE SLABS

Of the 614,387 bridges in the National Bridge Inventory, approximately 239,610 bridges (39%) are over 50 years or older [1]. Design drawings of these aging bridges may not be available which makes prediction of the remaining structural capacities of the bridge components more challenging, or even impossible. Fortunately, advanced non-destructive testing (NDT) techniques can provide great solutions to address these deteriorated bridges. Existing structures can be reliably assessed by the NDT methods as they can facilitate fast, cost-effective, efficient, rapid data collection. These techniques are also becoming increasingly important given the facts that they can be carried out without significant detriment to bridge structures and potentially without the need for traffic interruptions [10-12]. In the second part of this paper, the ultrasonic pitch & catch (UPC) technique for non-destructive testing of a reinforced-concrete slab is discussed. The developed software provides accurate predictions of cover thickness and diameters of steel rebars, which can help engineers to produce design drawings for aging in-service bridges and assess their current conditions. The software also precisely predicts the depth, thickness, and delaminations of the RC slab.

Ultrasonic Pitch and Catch Technique

Ultrasonic pitch and catch (UPC) technique is based on the ultrasonic pulse-echo method using transmitting and receiving transducers in a “pitch-catch” configuration (Fig. 3a). In the pitch-catch method, one channel (consists of 4 transducers) acting as a transmitter emits shear-wave pulse into the test object and the other channels sequentially act as a receiver picking up direct and reflected pulse (a.k.a. echo) [13]. Figure 3b shows the first channel acting as the

transmitter and other channels acting as receivers while Fig. 3c exhibits the second channel as the transmitter and other channels (except the first and second channels) as receivers. Based on this principle, there are a total of 66 transmitting-receiving pairs or 66 data arrays for each B-scan. The depth of the reflecting interface (i.e. defects, anomalies, discontinuities, irregularities, etc.), Z , can be calculated by Equation 1.

$$Z = \sqrt{\left(C_s \frac{\Delta t}{2}\right)^2 - X^2} \quad (1)$$

where C_s = shear-wave speed; Δt = measured travel time; X = horizontal spacing between the first channel and the test location (Fig. 3a). A signal processing technique called synthetic aperture focusing (SAFT) is used to reconstruct 2-D images of the interior of the test object (i.e. the RC slab in this study) at the test location. SAFT was originally developed in the early 1970s based on synthetic aperture airborne radar systems [14]. Its algorithm numerically superimposes pulse-echo signals measured at several positions, thus structural noise is suppressed by spatial superposition and resulting in a high-resolution image [15]. This study aims at developing inhouse software with an advanced signal processing technique called modified synthetic aperture focusing (SAFT) and proposes novel signal filtering method for accurate predictions of internal defects of reinforced concrete slabs (e.g., locations and dimensions of the defects) with high-resolution imaging. The modified SAFT is known to be more effective than the original SAFT and the aforementioned SAFT-C algorithms.

Developed Software

Computer code (written in Borland Delphi with a fully graphical interface) for ultrasonic imaging of concrete structures was developed to overcome the shortcoming of the proprietary software, which comes with the commercially-available UPC device used in this study. The most significant advantages of the in-house developed software include 1) developed a new procedure to calculate the approximate shear-wave velocity in a media using direct arrival time from each measurement pairs; 2) produced a single panoramic reconstruction from multiple overlapping scans by using superposition algorithm; 3) proposed novel signal filtering and processing (i.e. sharp and high-resolution reconstructed images of the embedded objects are obtained with the images synthesized from the ultrasonic linear array); 4) determined the embedded object in the RC structure from signal amplitudes obtained on vertical and horizontal sections of the 2D reconstructed image that reveals information on dimensions and exact locations of the embedded objects.

Modified SAFT

In the ultrasonic pitch and catch (a.k.a. pulse-echo) method, travel time from transmitter to receiver and amplitude of pulses reflected at an object embedded in the concrete structures are measured. Synthetic apertures focus the signal amplitude to any point in the sampling area of a large transducers by coherent superposition. Initially, synthetic aperture focusing technique (SAFT) is developed to interpret signals with transmitting and receiving at the same location. This technique also can be applied for an array of transmitters and receivers located at different locations. Two-dimensional (2D) SAFT is obtained from the linear aperture and represented by a B-Scan cross-section. A three-dimensional (3D) SAFT is synthesized from the planar synthetic apertures of two perpendicular directions, such as B-Scan and A-Scan. In this study, the original SAFT is modified and improved to interpret signals from transmitting and receiving transducers located at different locations on one side of the concrete component. In an ultrasonic linear array of transmitting and receiving pair, the impulse respect to a travel time represents response characteristics of positions in the media that have the same travel-path length. Synthesizing the overall signal produces a reconstructed image referred to as a SAFT B-Scan. The relationships between the synthesized signal and the ultrasonic linear array are given in the following equations:

$$g_{ij} = \sum_{t=1}^{N-1} \sum_{r=t+1}^N A(x_t, x_r, x_i, z_k) \phi_{tr}(x_i, z_j) \quad (2)$$

$$\phi_{tr}(x_i, z_j) = s \left(x_t, x_r, \frac{1}{c} \left(\sqrt{(x_i - x_t)^2 + z_j^2} + \sqrt{(x_i - x_r)^2 + z_j^2} \right) \right) \quad (3)$$

where g_{ij} is synthesized impulse at each position within the region of interest (ROI); i and j are horizontal and vertical indexes of a position in the ROI; N is number of transducer channels; t and r are indexes of the transmitting and receiving transducers; x_t and x_r are horizontal coordinates of the transmitting and receiving transducers; x_i and z_j are

horizontal and vertical coordinates of a position in the ROI with the i^{th} and j^{th} indexes; ϕ_{tr} and s is a function of the magnitude of the impulse at an evaluated time; and A is the apodization factor expressed as follows:

$$A(x_t, x_e, x_i, z_k) = \alpha_t(x_t, x_i, z_k) \alpha_r(x_r, x_i, z_k) \quad (4)$$

$$\alpha_t(x_t, x_i, z_k) = \frac{z_j}{\sqrt{(x_i - x_t)^2 + z_j^2}} \quad (5)$$

$$\alpha_r(x_r, x_i, z_k) = \frac{z_j}{\sqrt{(x_i - x_r)^2 + z_j^2}} \quad (6)$$

where: α_t and α_r are cosines of the angles between travel paths and vertical lines at the locations of transmitting and receiving transducers, respectively. The above equations allow for eliminating the direct arrival impulse (i.e. reduction of the impulse near the surface, where z_j is negligible). To cover for a large ROI, multiple overlapping scans are combined into a single panoramic reconstructed image. The scans are conducted at equal spacings in the direction along the aperture of the linear array. In this case, all impulses from the individual scan are accumulated according to their physical locations. The amplitude of the impulse, G_{ij} , is determined from the individual impulse as the following equation:

$$G_{ij} = \sum_{m=1}^M g_{ij}^m \quad (7)$$

where M is the number of overlapping scans; m is the index of a scan. The Hilbert transform focuses the instantaneous amplitude within the ROI to change in acoustic impedance that can obtain high reflectivity in reconstruction for the location of the embedded objects. However, it cannot distinguish rebars and voids when they appear in the same form in the reconstructed image. Therefore, in this study, the Hilbert transform is dismissed for detecting not only the location but also the characteristics of embedded objects in the RC structure. Figure 4 demonstrates the SAFT panoramic algorithms, which three overlapping B-scans of two rebars (embedded in a 178 mm thick concrete slab and with 125 mm spacing) are combined. The scans were conducted from left to right with a spacing of 50 mm, as shown in Fig. 4b-4d. The overlapping scans were then combined using Equation 7 and the resulting reconstructed image is presented in Fig. 4e. All scans show high-intensity reflections (dark-red color) at the rebar locations and slab bottom (i.e. backwall reflection). This example proved that the proposed algorithm effectively reconstructs the visualization image from multiple overlapping scans.

EXPERIMENTAL PROGRAM

Specimen Preparation and Details

To validate the effectiveness of the inhouse developed software, a reinforced concrete (RC) slab was designed with two layers of #5 steel rebars and artificial delaminations (Fig. 5). The overall dimensions (width $\times$ length $\times$ thickness) of the RC slab are 45 $\times$ 48 $\times$ 7 in. (1,143 $\times$ 1,219 $\times$ 178 mm). The artificial delaminations, made from off-the-shelf hard-plastic sheets, were embedded in the RC slab. Two sizes of delaminations (width $\times$ length $\times$ thickness) were considered including 4 $\times$ 8 $\times$ 0.4 in. (102 $\times$ 203 $\times$ 10 mm) and 8 $\times$ 12 $\times$ 0.4 in. (203 $\times$ 305 $\times$ 10 mm). These delaminations were located at different depths as shown in Fig. 5. Ready-mix concrete from in-transit mixers was used for the casting of the RC slab. The slab specimen was then cured and tested in laboratory conditions.

Data Collection and Acquisition

The data collection was performed in both horizontal and vertical directions with equal spacing of 2 in. (50.8 mm) (Fig. 6). A total of 420 data points (corresponding to the grid point locations in Fig. 6) were collected on the RC slab. The operating frequency was set to a default value of 50 kHz while the shear-wave velocity was set to be determined by actual measurements. It should be noted that the shear-wave pulse velocity of 2,450 μ s was assumed for concrete and this value was used in SAFT reconstruction by the UPC device.

RESULTS AND DISCUSSIONS

The signal amplitudes for reconstructed image of the concrete slab obtained from SAFT without applying the Hilbert transform can be positive and negative values and mapped in a color range from black (minimum negative) to dark red (maximum positive). The color spectrum is defined in Fig. 7 with “dark” designates black to blue colors and “bright” indicates light-blue to dark-red colors. Figure 8 exhibits the 2D B-scan reconstructed images using the in-

house developed software for the detection of rebars and artificial delaminations embedded in the RC slab. The bottom of the concrete slab (a.k.a. backwall reflection) is usually observed as a long bright-color strip near the bottom section (Fig. 8). The backwall reflection can also be determined from the positive-negative waveform pattern (Fig. 9a). The thickness of the concrete slab can be approximated as the depth of the positive peak of the signal amplitude. The calculated thickness of the concrete slab (based on the reconstructed images) ranges from 7.16 in. [182 mm] to 7.52 in. [191 mm], which is approximately 2.2%-7.3% higher than the design thickness of the slab. These discrepancies may be caused by the uneven surface of the concrete slab.

In Fig. 8a, the first layer of the rebars are highly visible in the reconstructed image (i.e. circular areas with bright color and dark shadows on top). Locations of rebars can be determined not only from the reconstructed images but also from the signal amplitudes either in vertical or horizontal directions. In the vertical direction (Fig. 9b), the rebar is represented by a waveform with a minimum peak followed by a maximum peak, and the location of the rebar is approximately equal to the mean value of those two-peak coordinates. The rebar locations in the horizontal direction are accurately identified based on peak amplitudes of the waveform (refer to circular marks on the horizontal section of 2D reconstructed image in Fig. 9c). It is noted that the color spectrum of the rebars in the second layer is not extremely bright as the first-layer rebars in the reconstructed image (Fig. 8a) because of the signal attenuation. However, these rebars have the similar waveform pattern as the rebars in the first layer. Locations of the second-layer rebars can be determined based on the waveform pattern although the peaks of the signal amplitudes of the second-layer rebars are lower than those of the first-layer rebars (Fig. 9c). The rebars near the edges of the concrete slab appear in different sizes and shapes in comparison with the interior rebars. It can be observed that those rebars located in the areas with fewer scans causing limited aperture effects [16]. Finer scan spacings near the edges of the RC structure is thus recommended to overcome the reconstructed 2D images.

As earlier mentioned in the experimental program, the artificial delaminations (D1 to D5 in Fig. 5a) were placed at various depths and locations in the RC slab. In contrast with the rebars, the artificial delaminations appear in the form of a bright-color strip (Fig. 8b through Fig. 8d). This is attributed to the fact that the ultrasonic shear wave reflected from an object embedded in concrete slab with a lower density than concrete material. In the vertical sections of the 2D reconstructed image, the waveform has a sequential form of positive-negative peak amplitude (Fig. 9d through Fig. 9i) as a recognizable sign of the delaminations. The center of the bright strips (Figs. 8b, 8d, 8c) corresponding to the positive peak amplitudes in the vertical sections (Figs. 9d, 9e, 9h) are assumed to be the predicted depths of the delaminations. The predicted depths from the reconstructed images (Table 3) correlates relatively well with the measured depths. The breadths of the delaminations can be predicted based on the bright-color strips where they are highly visible in the reconstructed images (Fig. 8b through Fig. 8d). It can be observed from Figs. 9f, 9g, 9i that the delaminations are represented by the high-amplitude regions. Table 3 shows the comparisons between the predicted and measured breadths with a good agreement obtained for the delaminations D1, D3, and D5. As observed in Fig. 5, delaminations D2 and D4 were located just under the top-layer rebars. The rebars perpendicular to the scan direction blocked the ultrasonic wave from reaching the delaminations D2 and D4. As a result, the predicted breadths of the delaminations D2 and D4 are slightly smaller than their measured values (Table 3).

Figures 10 shows 2D images obtained from the UPC device's proprietary software for the rebars and artificial delaminations. In general, the software provides fairly good visualization of the embedded objects in terms of their locations and horizontal dimensions. However, the proprietary software used the Hilbert transform, which made the embedded objects much bigger than their actual measured dimensions in the vertical direction. For example, the delaminations in Fig. 10b through Fig. 10d are exhibited in very thick areas indicated by red color. In addition, the proprietary software failed to display the exact locations of the two-layer rebars as they are close to each other (3 in. [76 mm]). As a result, the proprietary software mixed up the rebars from two layers as the color spectrum of the first-layer rebars are overlapped with the second-layer rebars (Fig. 10).

CONCLUDING REMARKS AND FUTURE WORK

The first part of this paper gives an overview of FRP wraps for the rehabilitation of deteriorated bridges in the state of West Virginia. Overall conditions of all highway bridges in the state of West Virginia (based on the latest National Bridge Inventory by U.S. Department of Transportation, Federal Highway Administration) are presented. Examples of two FRP-retrofitted bridge projects by the West Virginia Department of Transportation are provided. The second part addresses the non-destructive testing technique in detecting subsurface defects and mapping reinforcing rebars in concrete structures. A methodology based on a modified synthetic aperture focusing technique (SAFT) to reconstruct images of reinforced-concrete slabs is proposed. A computer software for ultrasonic imaging of concrete structures

was developed to overcome the shortcoming of the proprietary software, which comes with the commercially-available UPC device used in this study. The in-house developed software was effectively visualized the embedded objects in the concrete slab including rebars and artificial delaminations. The results reveal that 2D reconstructed images of the rebars and delaminations can be interpreted based on specific patterns of color spectrum. The UPC is proved to be an excellent non-destructive testing (NDT) technique for an accurate evaluation of concrete structures. In our future study, we will extend the application of the NDT techniques such as UPC, ground-penetrating radar, and infrared thermography for examining debondings/delaminations/defects of FRP-wrapped concrete bridge elements. Experimental works are underway and results will be published in the near future.

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Table 1—List of bridges in West Virginia using FRP composites

No.	Bridge Name	Year Built/Reconstructed or Rehabilitated	Total Length, ft [m]	Deck Width, ft [m]	Bridge Type (Main)	FRP System
1	Goat Farm	2004	42.3 [12.9]	15.0 [4.6]	SSWB	FRP deck
2	Kite Creek	2004	34.7 [10.6]	24.2 [7.4]	SSWB	FRP deck
3	Howell's Mill (a.k.a. "Rimmer- White")	2003	237.5 [72.4]	32.5 [9.9]	CSBM	FRP deck
4	Robert C. Beach (a.k.a. West Buckeye)	2003	148.7 [45.3]	36.0 [11.0]	Timber Arch – Through Type (STTA)	FRP deck/panel
5	La Chein	2003	42.8 [13.0]	24.0 [7.3]	SSWB	FRP deck
6	Market Street	2001	180.5 [55.0]	56.0 [17.1]	SSPG	FRP deck
7	Boy Scout Camp	2001	33.1 [10.1]	25.2 [7.7]	SSWB	FRP deck
8	Wickwire Run	1997	34.5 [10.5]	21.8 [6.6]	SSWB	FRP deck
9	Hanover	1976/2010	118.4 [36.1]	28.2 [8.6]	SSWB	FRP deck/panel
10	Katy Truss	1912/2001	90.1 [27.5]	13.9 [4.2]	SSPT	FRP deck
11	Martha Queen's	2001	49.5 [15.1]	30.1 [9.2]	SCBB	Deck with GFRP C-bar and abutment with GFRP rebar
12	Montrose	2001	40.7 [12.4]	27.5 [8.4]	SSWB	Deck with FRP rebar/grid
13	Dans Run Slab	2000	25.3 [7.7]	24.3 [7.4]	SCSL	Deck with FRP rebar/grid
14	Buffalo Creek (a.k.a. McKinleyville, 1st vehicular bridge with FRP rebar in the US)	1996	180.0 [54.9]	29.5 [9.0]	CSWB	Deck with FRP rebar/grid
15	North Kayford	1940/2000	43.0 [13.1]	24.2 [7.4]	SCBB	Deck with FRP rebar/grid
16	North Acme	1940/2001	35.3 [10.8]	24.0 [7.3]	SCBB	Deck with FRP rebar/grid
17	South Acme (FRP Rebars)	1940/2001	34.4 [10.5]	24.1 [7.3]	SCBB	Deck with FRP rebar/grid
18	Barrackville Covered	1853/1999	150.0 [45.7]	15.3 [4.7]	STCO	FRP tendon/cable
19	Laurel Lick	1997	20.0 [6.1]	16.0 [4.9]	All- composite	FRP deck, beam, and substructure

Note: SSWB = Simple Steel Wide-Flange Beam; SSPT = Simple Riveted Pony Truss Spans; CSBM = Continuous Steel Stringer/Multi-Beam or Girder; STCO = Single-Span Timber Covered Bridge; SCTB = Simple-Span Concrete T-Beams; SSPG = Structural Steel Plate Girder; CSWB = Continuous Steel Wide-Flange Beam; SCBB = Simple Prestressed Concrete Box Beam; SCSL = Single Reinforced Cast-In-Place Concrete Slab Spans.

Table 2—Bridge conditions in West Virginia

Main Structure Type	Code	SD (a)	FO (b)	Bridge Total (c)	a/c (%)	b/c (%)
<i>Slab</i>	01	168	144	517	32.5	27.9
Stringer/Multi-beam or Girder	02	592	516	3085	19.2	16.7
<i>Girder and Floorbeam System</i>	03	107	41	229	46.7	17.9
<i>Tee Beam</i>	04	49	25	104	47.1	24.0
Box Beam or Girders - Multiple	05	92	401	1905	4.8	21.0
Box Beam or Girders - Single or Spread	06	2	8	55	3.6	14.5
Frame (except frame culverts)	07	5	14	52	9.6	26.9
Orthotropic	08	0	2	2	0	100
Truss – Deck	09	0	4	11	0	36.4
<i>Truss – Thru</i>	10	78	35	180	43.3	19.4
<i>Arch – Deck</i>	11	159	134	399	39.8	33.6
Arch – Thru	12	1	2	8	12.5	25.0
Suspension	13	2	1	3	66.7	33.3
Stayed Girder	14	0	0	3	0	0
Movable – Lift	15	NA	NA	NA	NA	NA
Movable – Bascule	16	NA	NA	NA	NA	NA
Movable – Swing	17	NA	NA	NA	NA	NA
Tunnel	18	NA	NA	NA	NA	NA
Culvert (includes frame culverts)	19	46	36	539	8.5	6.7
Mixed types	20	1	0	1	100	0
Segmental Box Girder	21	1	1	3	33.3	33.3
<i>Channel Beam</i>	22	63	25	115	54.8	21.7
Other	00	6	5	17	35.3	29.4
Total		1,372	1,394	7,228	19.0	19.3

Note: SD = Structurally Deficient; FO = Functionally Obsolete

Table 3—Predicted and measured breadths and depths of the delaminations

Delamination	Predicted breadth in. [mm]	Measured breadth in. [mm]	Predicted depth in. [mm]	Measured depth in. [mm]
D1	7.71 [196]	8.00 [203]	2.44 [62]	2.32 [59]
D2	7.05 [179]	8.00 [203]	5.08 [129]	5.00 [127]
D3	7.87 [200]	8.00 [203]	2.36 [60]	2.32 [59]
D4	7.32 [186]	8.00 [203]	5.23 [133]	5.00 [127]
D5	7.48 [190]	8.00 [203]	3.31 [84]	3.62 [92]



(a) Elevation view



(b) GFRP wrapping on a wing wall

Fig. 1—Details of East Street Viaduct bridge (WVDOT bridge inspection reports 2012 and 2017)



(a) Elevation view



(b) CFRP wraps in underside of a T-beam and
abutment #1

Fig. 2—Details of Flag Run bridge (WVDOT bridge inspection reports 2016)

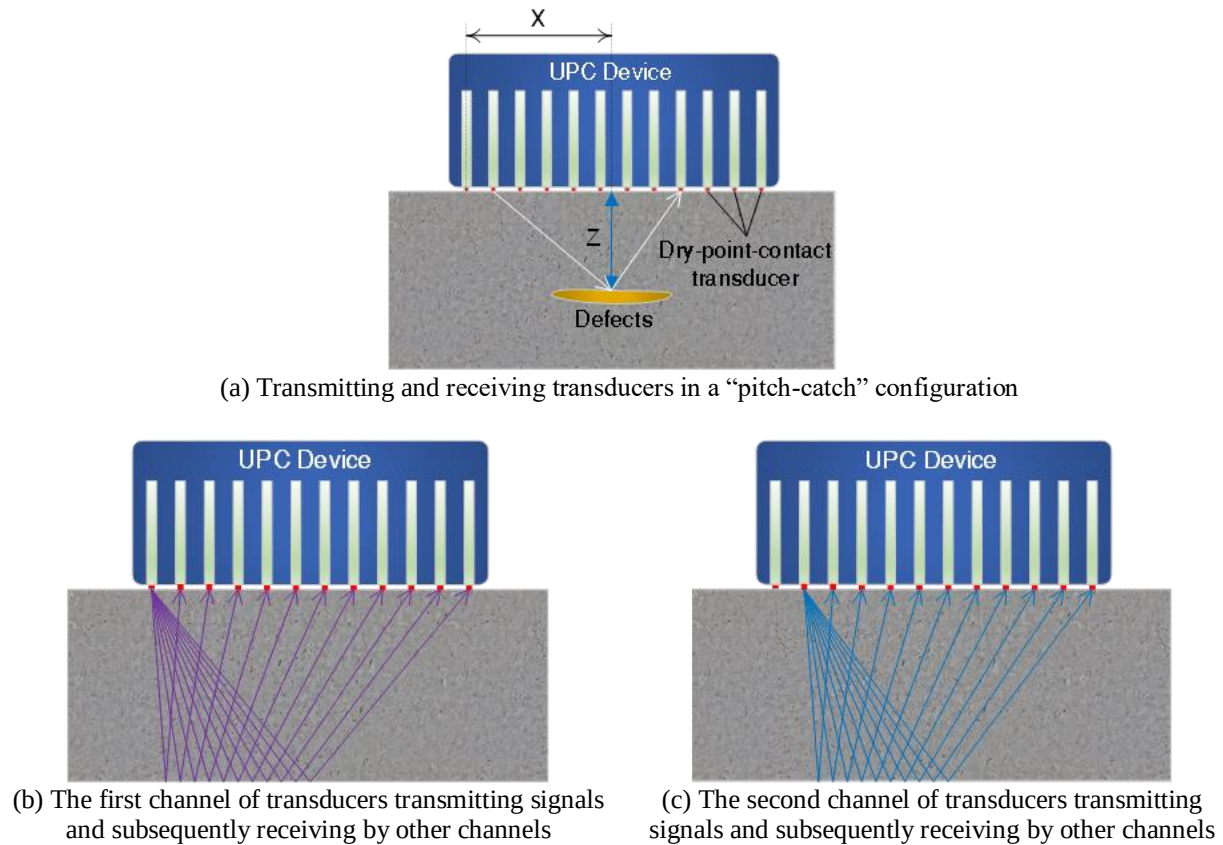


Fig. 3—Basic principles of the UPC technique

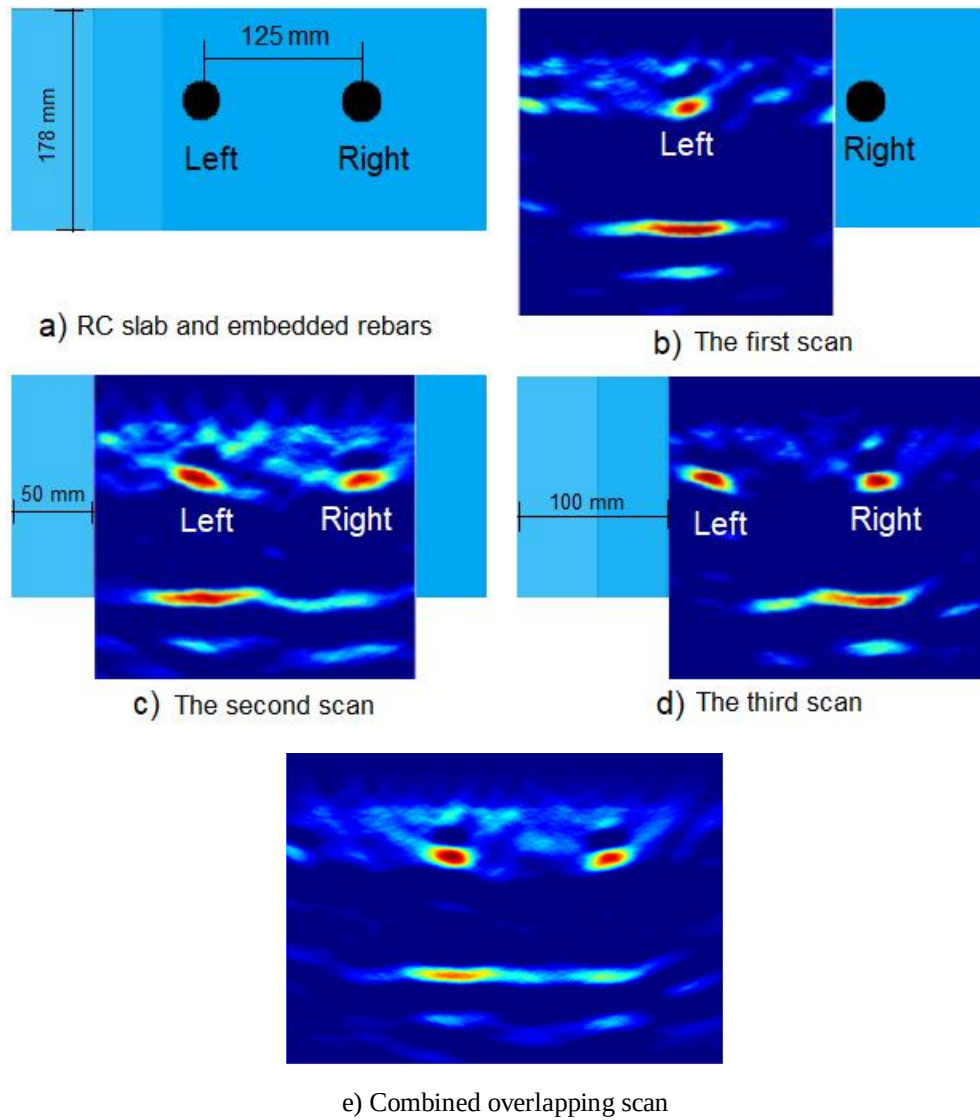
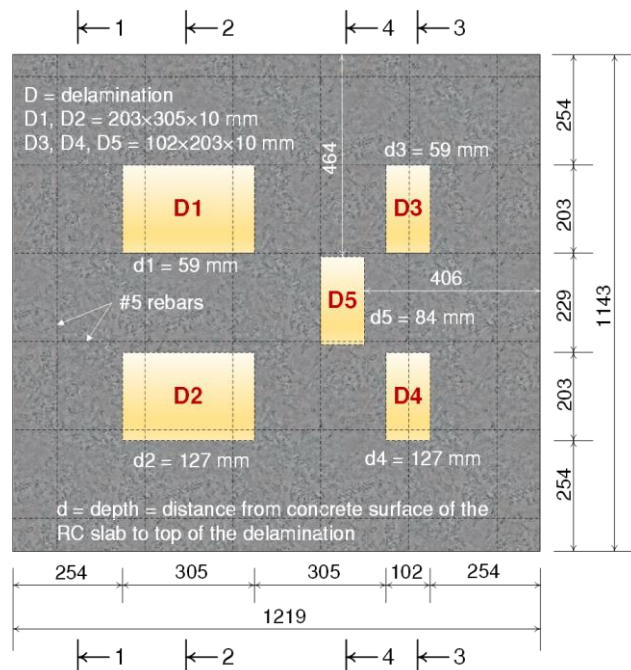
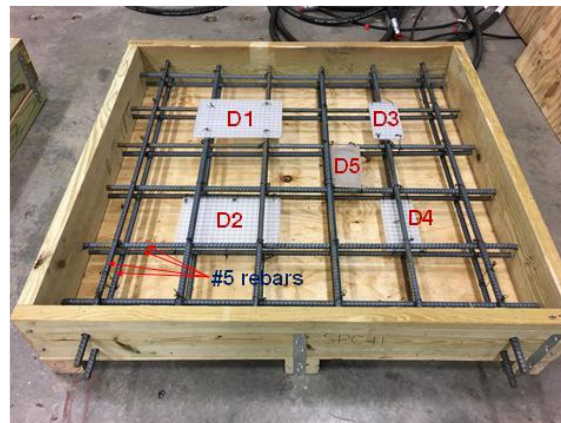


Fig. 4—Three overlapping B-scans and their combination



(a) Locations of artificial delaminations (units in millimeters; 1 mm = 0.039 in.)



(b) Schematic view

Fig. 5—Details of the RC slab specimen

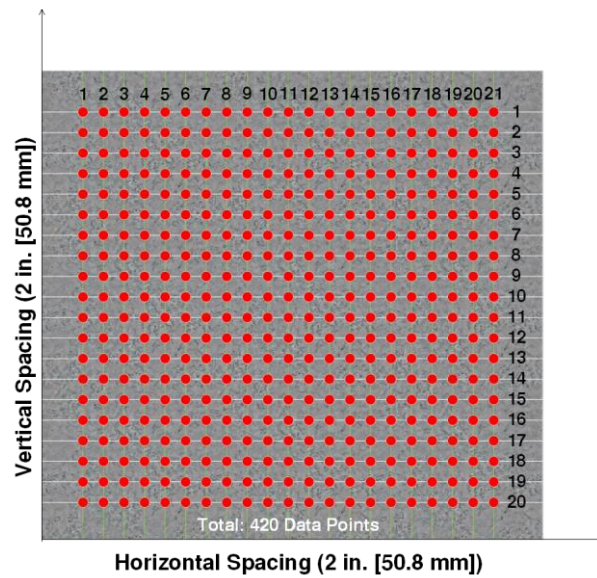


Fig. 6—Data collection on the RC slab using the UPC technique

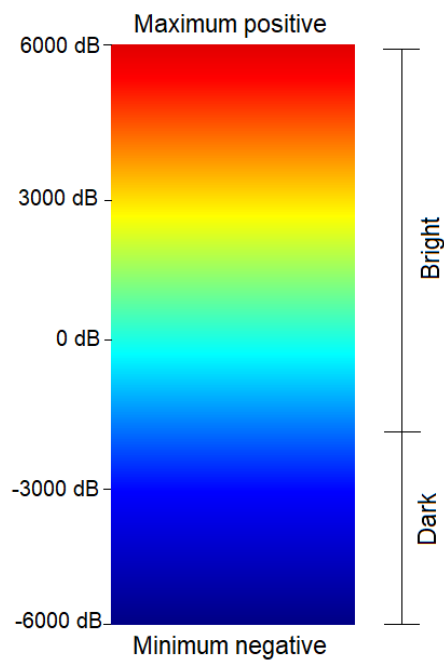
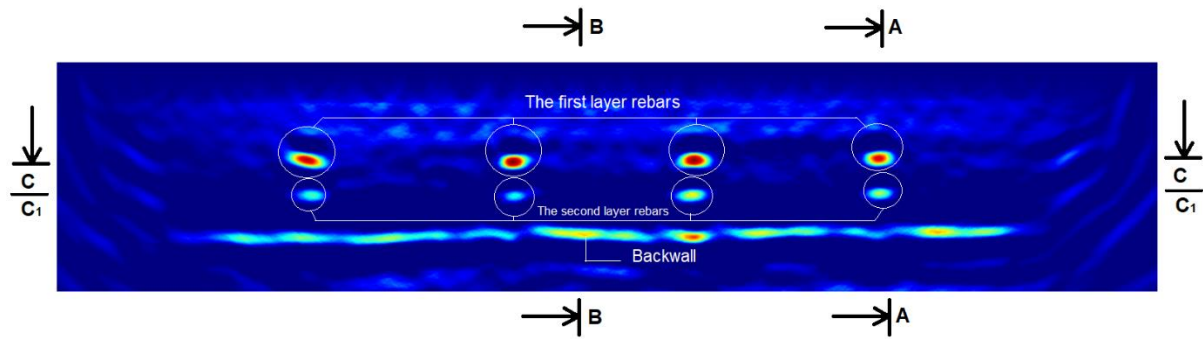
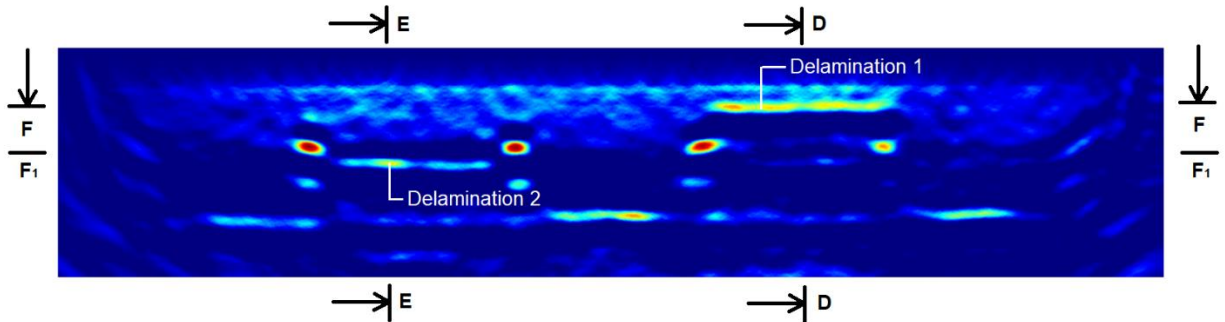


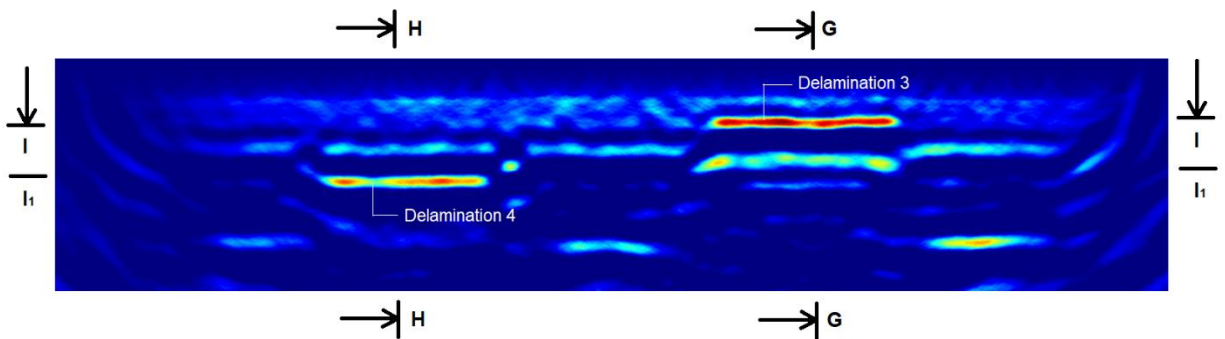
Fig. 7—Color spectrum for signal amplitudes



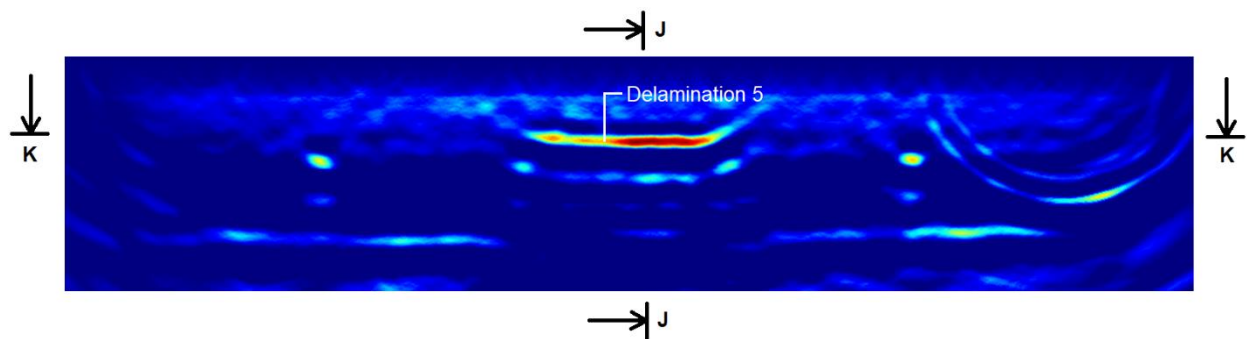
a) 2D reconstructed image at the location of the fully bonded rebar (refer to section 1-1 in Fig. 5a)



b) 2D reconstructed image at the location of the artificial delaminations 1 and 2 (refer to section 2-2 in Fig. 5a)

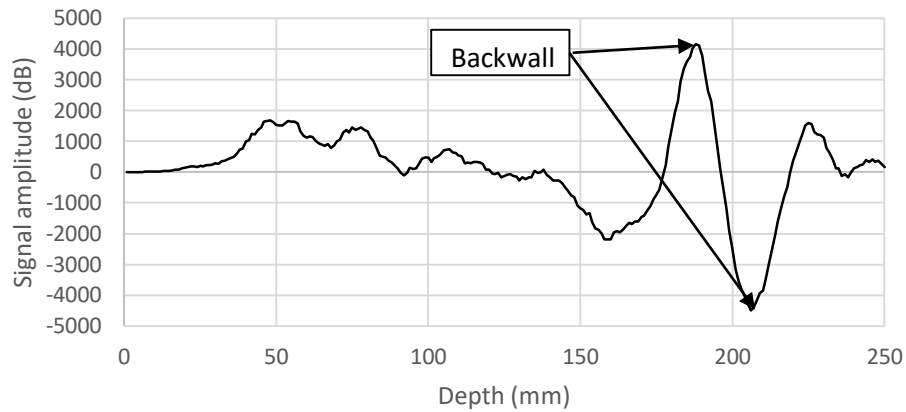


c) 2D reconstructed image at the location of the artificial delaminations 3 and 4 (refer to section 3-3 in Fig. 5a)

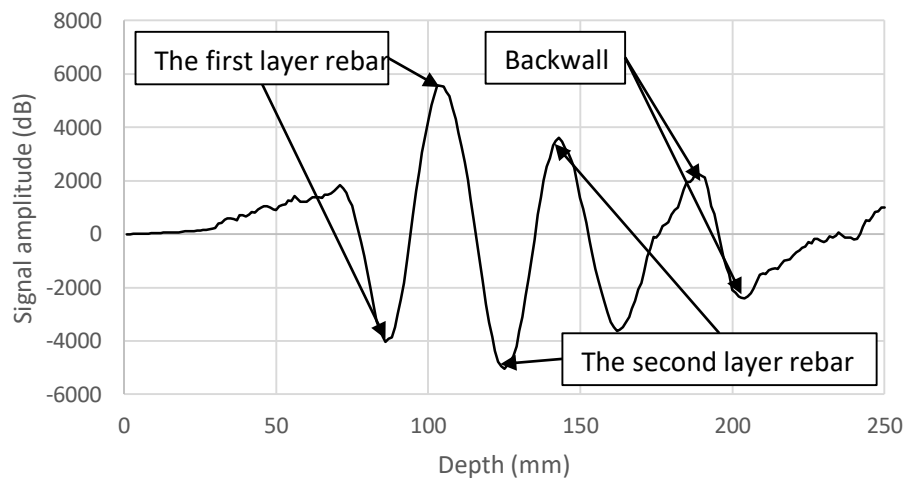


d) 2D reconstructed image at the location of the artificial delamination 5 (refer to section 4-4 in Fig. 5a)

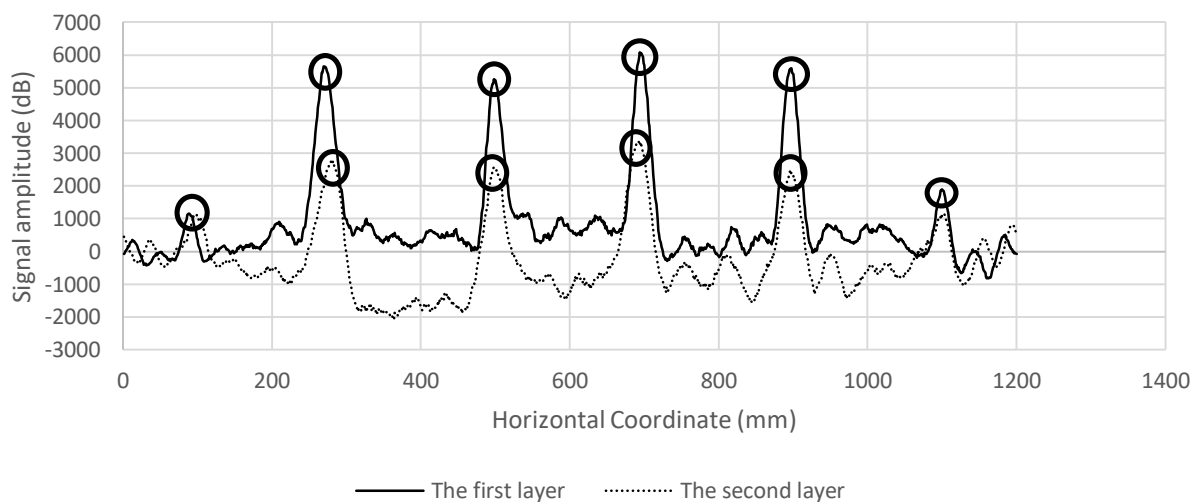
Fig. 8—2D reconstructed images



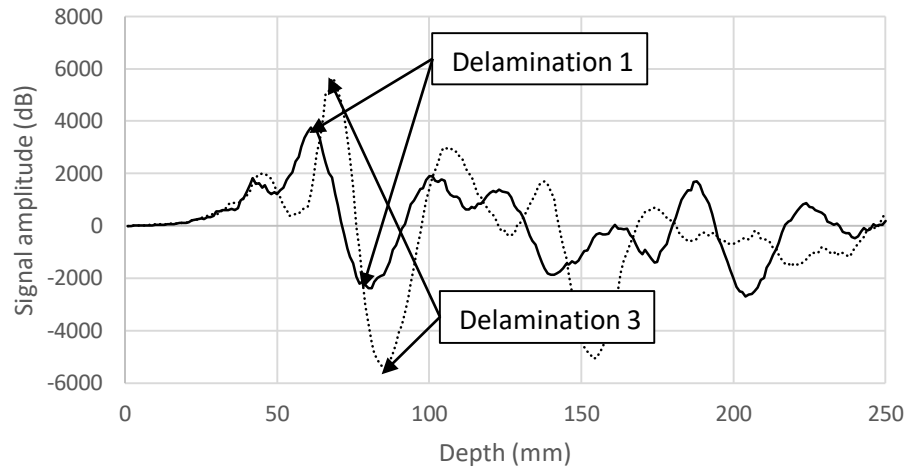
a) Backwall reflection at section B-B (refer to Fig. 8a)



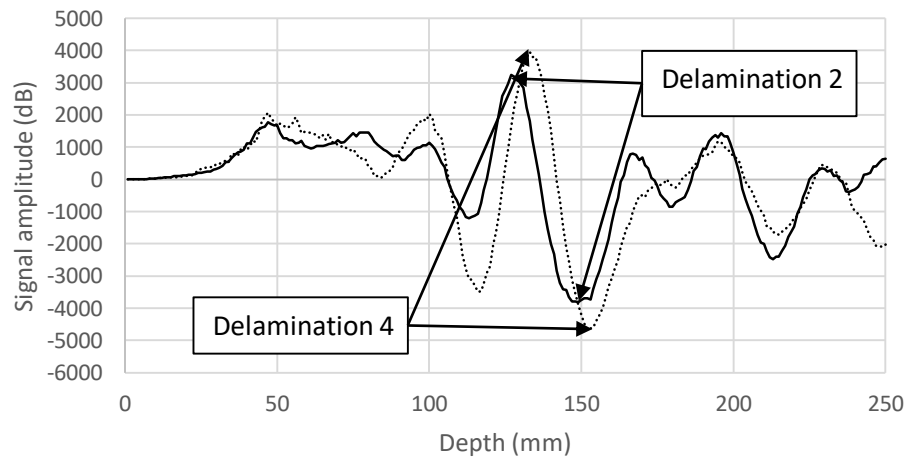
b) Signal amplitude versus depth relationship of a fully-bonded rebar (section A-A in Fig. 8a)



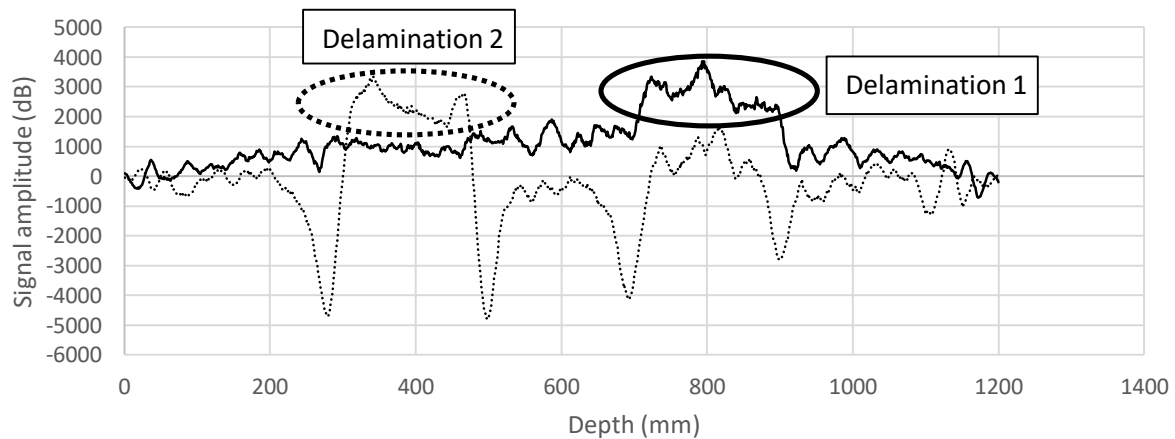
c) Signal amplitude versus horizontal coordinate relationship of rebars (sections C-C and C₁-C₁ in Fig. 8a)



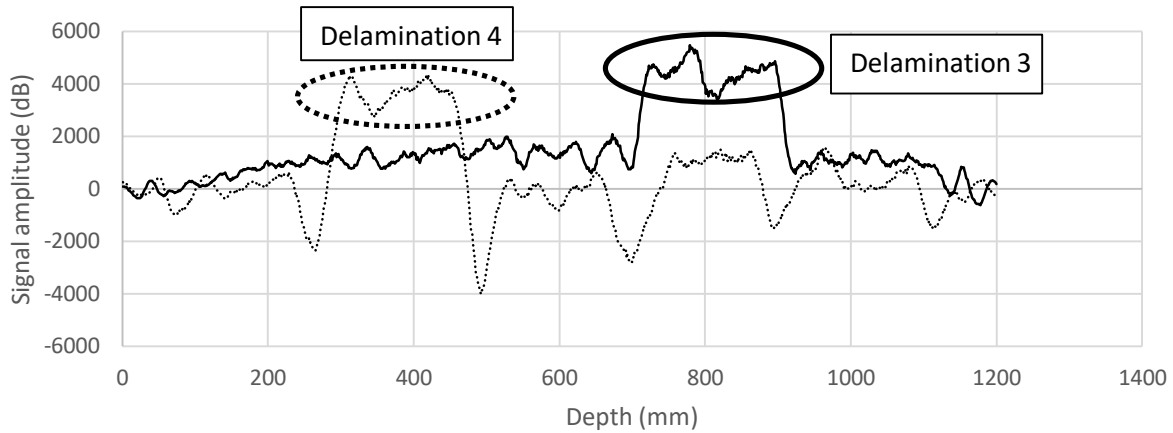
d) Signal amplitude versus depth relationship of the delaminations 1 and 3 (sections D-D and G-G in Fig. 8b-8c)



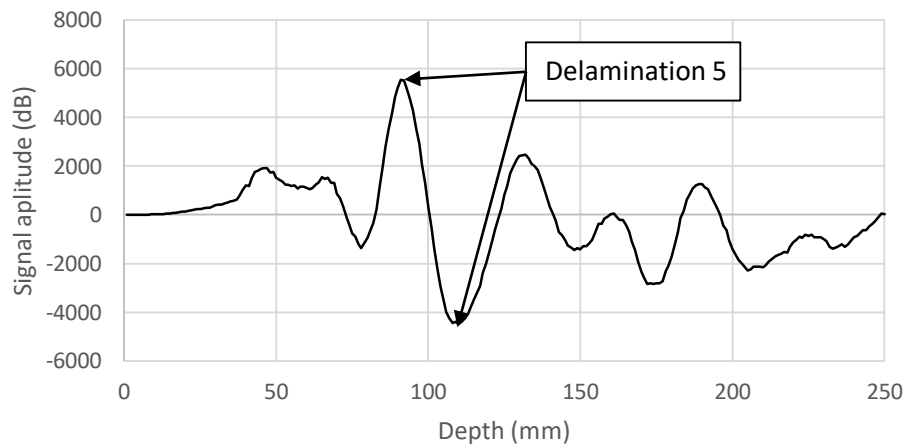
e) Signal amplitude versus depth relationship of the delaminations 2 and 4 (sections E-E and H-H in Fig. 8b-8c)



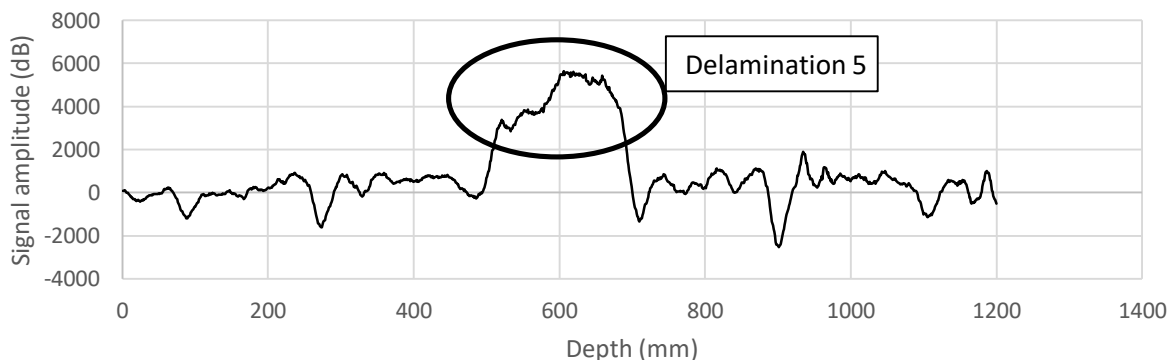
f) Signal amplitude versus horizontal coordinate relationship of delaminations 1 and 2 (sections F-F and F₁-F₁ in Fig. 8b)



g) Signal amplitude versus horizontal coordinate relationship of delaminations 3 and 4 (sections I-I and I₁-I₁ in Fig. 8c)

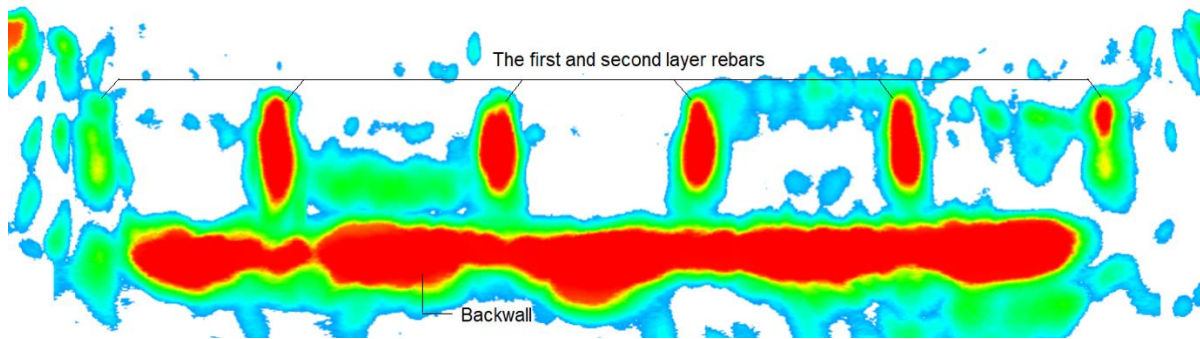


h) Signal amplitude versus depth relationship of the delamination 5 (section J-J in Fig. 8d)

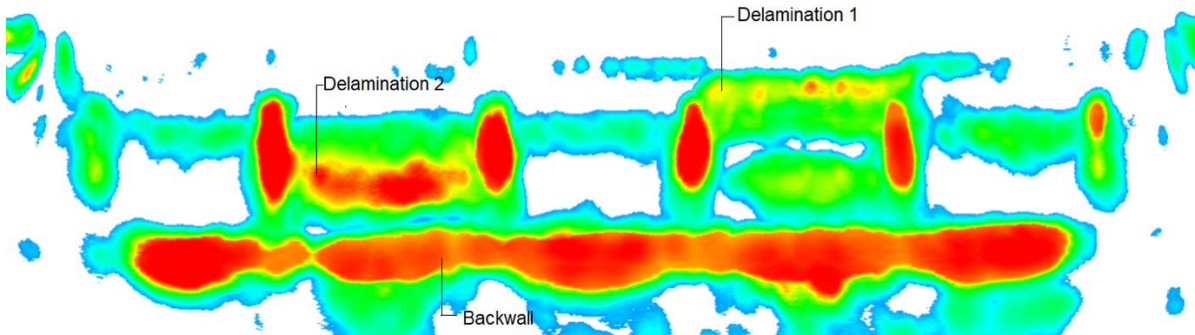


i) Signal amplitude versus horizontal coordinate relationship of delamination 5 (section K-K in Fig. 8d)

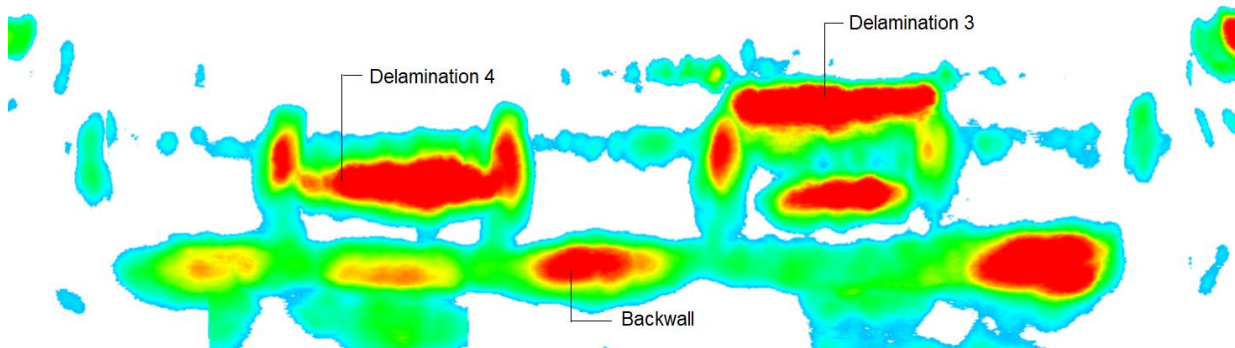
Fig. 9—Signal amplitude at different sections [1 mm = 0.0394 in.]



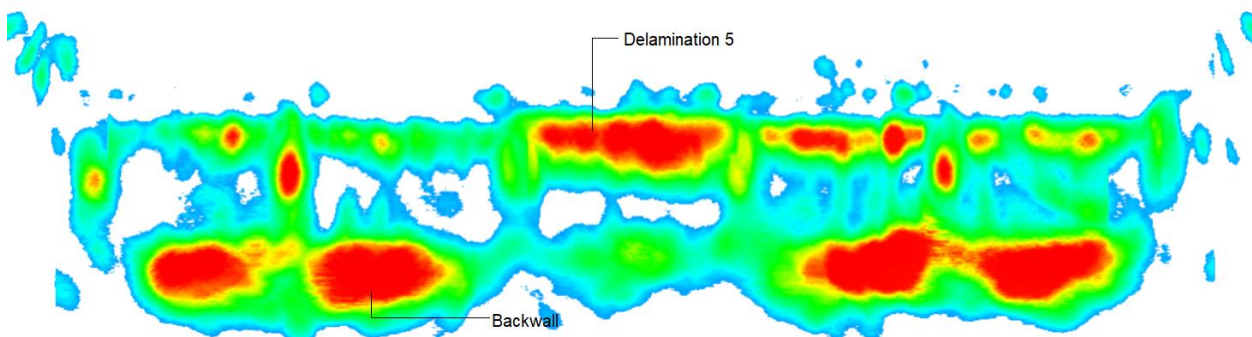
a) 2D reconstructed image at the location of the fully bonded rebars (refer to section 1-1 in Fig. 5a)



b) 2D reconstructed image at the location of the artificial delaminations 1 and 2 (refer to section 2-2 in Fig. 5a)



c) 2D reconstructed image at the location of the artificial delaminations 3 and 4 (refer to section 3-3 in Fig. 5a)



d) 2D reconstructed image at the location of the artificial delamination 5 (refer to section 4-4 in Fig. 5a)

Fig. 10—2D images obtained from the UPC device's proprietary software

Impact Damage Retrofit of RC Bridge Girder Previously Retrofitted with CFRP Fabric

Abheetha Peiris and Issam Harik

Synopsis: Following an over-height truck impact, Carbon Fiber Reinforced Polymer (CFRP) fabric was used to retrofit the exterior girder in a four-span Reinforced Concrete Deck on Girder (RCDG) Bridge on route KY 562 that passes over Interstate 71 in Gallatin County, Kentucky. The impacted span (Span 3) traverses the two northbound lanes of Interstate 71. While the initial retrofit was completed in May 2015, a second impact in September 2018 damaged all four girders in Span 3. The previously retrofitted exterior girder (Girder 4) suffered the brunt of the impact, with all steel rebars in the bottom layer being severed. Damage to Girders 1, 2, and 3 was minor and none of the bars were damaged. A two-stage approach for the containment and repair of the damaged girders following an over-height truck impact was implemented when retrofitting the bridge. The repair and strengthening of all the girders using CFRP fabric was the economical option compared to the alternative option of replacing the RCDG bridge. The initial CFRP retrofit was found to have failed in local debonding around the impact location. The CFRP retrofit material that was not immediately near the impact location was found to be well bonded to the concrete. The removal of this material and subsequent surface preparation for the new retrofit was time consuming and challenging due to traffic constraints. In Girder 4 all but one of the main rebars were replaced by removing the damaged sections and installing straight rebars connected to the existing rebars with couplers. One of the rebars could not be replaced. A heavy CFRP unidirectional fabric, having a capacity of 534 kN (120,000 lbs.) per 305 mm (1 ft.) width of fabric, was selected for the flexural strengthening and deployed to replace the loss in load carrying capacity. A lighter unidirectional CFRP fabric was selected for anchoring and shear strengthening of all the girders, and to serve as containment of crushed concrete in the event of future over-height impacts. The retrofit with spliced steel rebars and CFRP fabric proved to be an economical alternative to bridge replacement.

Keywords: bridge; CFRP; heavy fabric; impact damage; reinforced concrete; retrofit

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INTRODUCTION

Bridges with girders impacted by over-height trucks may require a load posting if the impact causes moderate to severe damage. While height restrictions exist and are enforced, these impacts occur frequently, and in certain instances when the damage is excessive, require bridge closure. Due to vertical clearance limits with the roadway grade, most impacts tend to be on one of the exterior girders. Damage due to over-height truck impacts affects the safety of traffic on both the roadway on the bridge, as well as the one below it. Damage caused by impacts includes cracking of beams, damage to reinforcement and/or prestressing strands, yielding of steel, spalling of concrete, and failure of joints and connections. Due to cracking and spalling of concrete, damage to both Reinforced Concrete (RC) and Prestressed Concrete (PC) girders sometimes require immediate attention even when the impact may not be severe due to the possibility of loose concrete blocks falling on vehicles travelling under the impacted beams. Many traditional methods of repair and retrofit of such damages can be costly and time-consuming due to the location of the damage and impact to traffic. Fiber Reinforced Polymer (FRP) materials have emerged over the last few decades as a reliable and effective method of externally strengthening concrete bridge beams. They can be highly cost-effective due to their non-corrosive nature, high strength-to-weight ratio, and rapid application capability when compared with traditional methods. Among FRP material, Carbon FRP (CFRP) has been the primary material utilized for the repair and retrofit of over-height impacted bridge girders. In the United States, the field application of externally bonded (EB) CFRP-pultruded laminates and wet-layup fabrics for strengthening Reinforced Concrete (RC) beams is well established, with design guidelines by the American Concrete Institute (ACI)¹ and American Association of State Highway and Transportation Officials (AASHTO)². Numerous field applications of over-height damage retrofit using EB-CFRP have been reported in literature³⁻⁷ along with the recent deployment of CFRP Rod Panels for the retrofit of impact damaged RC and PC bridge girders in Kentucky⁸⁻¹⁰. The 2012 National Cooperative Highway Research Program (NCHRP) report¹¹, dealing with collision damage and repair of PC girders, categorized the use of different FRP retrofit methods based on the degree of impact damage to the PC girders. Many findings related to PC bridge girders also extend to RC bridge girders. According to the latest Scan by the NCHRP¹², the use of FRP for the retrofit of over-height truck impact damage is considered as ‘practice ready’, with thorough research and sufficient design guidance. The majority of impacted locations on bridge girders have low clearances and are often impacted more than once. This paper presents the steps that were followed to retrofit the route KY 562 RC bridge girder with FRP following the first and second impact. The analysis and design of the second retrofit depend on the steps followed during the first retrofit and the degree of damage during the second impact. The second impact does provide insight into the type of failure that can be expected in the FRP material.

RESEARCH SIGNIFICANCE

The presented case study details the failure observed in a CFRP fabric retrofitted bridge girder following an over-height truck impact, and the steps taken to remove the existing CFRP material and reapply an externally bonded CFRP fabric system to strengthen the damaged girder. The study highlights the complications of removing existing CFRP material, the nature of the concrete surface following the removal, as well as the additional surface preparation needed prior to application of the new CFRP fabric system. The two-stage approach implemented for the retrofit of the bridge girder is outlined along with detailed information related to the new CFRP retrofit design and construction.

IMPACT DAMAGE RETROFIT

Following an over-height truck impact, the damage is evaluated and, if required, the bridge is load posted or closed. Depending on the location and size of impact, the damage can span multiple lanes of traffic, leading to traffic related delays during the retrofit construction period. In some cases, and unless the girder is severely damaged, it may take several months following the impact to carry out the repairs. The prolonging of the repair may result in subsequent impacts being undetected, with the damage not being evaluated, leading to possible debris falling on the traffic below.

Furthermore, corrosion of exposed rebars or prestressing strands may lead to section loss and reduction in member capacity. Following several over-height truck impact damaged bridge repair projects in the State of Kentucky, a two-stage approach was developed for the containment and repair of the damaged area⁶. The first stage outlines the immediate action following an impact to prevent loose concrete blocks from falling on the traveling public. The second stage outlines the methodology for repairing the impacted area and also providing containment in the event future impacts were to occur. This two-stage approach was applied on the girder in the bridge on route KY 562.

Stage 1: Containment of Damage Area

The first stage focuses on the steps to follow immediately after an over-height truck impact, where there is visible damage to the concrete with the possibility of concrete spalling. The steel or the prestressing strands may or may not be exposed at this time. The primary focus of Stage 1 is to contain any loose debris in the damage area until the retrofit can be carried out. Stage 1 is not necessary if the retrofit construction is carried out immediately following the impact and the road underneath is closed. For the KY 562 Bridge, retrofit construction was delayed for 7-months. Since the steel will be exposed during this period, it was cleaned and primed to prevent rust accumulation. The loose concrete can be removed by hand and any pieces trapped within steel rebars or prestressing strands can be hammered lightly or broken down to smaller pieces using a chisel and hammer. Containing the damage area is the next step to prevent any concrete from coming loose as well as to provide some containment in case of future impacts prior to the retrofit. Commercially available fiberglass mesh can be used for containment. The mesh used on this bridge has approximately 5 mm × 5 mm (0.2 in × 0.2 in) openings and weighs less than 160 g/m² (4.72 oz/yd²). Laboratory tests were conducted to evaluate the capacity of the fiberglass mesh when bonded to concrete in order to ensure that it can hold concrete blocks that may come loose. The mesh and the epoxy, used to apply the mesh, are available at many home improvement stores. The mesh is bonded on to the undamaged vertical face of the beam using the epoxy and wrapped in a 'U' shape around the impact area. Self-adhesive meshes are also available that can be wrapped easily on to the undamaged concrete, and then coated with a strong adhesive using a roller or brush.

Stage 2: Repair and Containment of Future Damage

The second stage outlines the methodology for repairing the impacted area and providing containment in the event future impacts were to occur. For concrete repair, detailed information is available in guides such as ICRI 310.1R-2008¹³, ACI 564R-04¹⁴ and NCHRP Report 609¹⁵. A summary of the repair steps for impacted beams is presented herein:

- (1)- Removal of any loose concrete debris as described in Stage 1 and, in addition, any concrete that is deemed unsound, cracked, or unsuitable for application of FRP must be removed. This is typically accomplished by pneumatic or electric chipping hammers.
- (2)- Simple geometric shapes should be cut around the damaged area and the concrete should be chipped off up to the edges of the geometric shapes.
- (3)- Concrete around the ends of any exposed steel should be removed until a minimum of 25 mm (1 inch) of non-corroded steel is exposed. Any splicing of the rebars as well as replacement of any damaged shear stirrups should be carried out at this point.
- (4)- The steel should be cleaned by sand blasting or mechanical means to have a bright metal finish. This is followed by cleaning the steel surface with a solvent and the application of a primer either by brush or spraying equipment to cover all exposed steel. Typically, a minimum of twelve hours is required for the primer to dry.
- (5)- Following the application and curing of the steel primer, the girder should be brought back to its original shape using a rapid-set repair mortar. Depending on the severity of the damage and location, formwork and aggregates may be necessary. A bonding agent is recommended when applying repair mortar on to overhead or vertical surfaces. Numerous types of repair mortar are commercially available, and one can be selected depending on the strength gain required, work time desired, environmental conditions, etc.
- (6)- When cracks are present, and prior to any strengthening work, they should be evaluated and, if required, repaired in accordance with the guidelines. If crack injection is required, the concrete around the cracks should be cleaned mechanically or sand blasted to improve bonding of epoxy around the cracks. Cracks should be cleaned by blowing compressed air to remove dust and any loose particles. Crack injection ports should be mounted, and the cracks completely covered by a trowel-able epoxy. Following the cure time for the epoxy, a low viscosity crack filling epoxy should be injected through the injection ports. Following the cure time for the injected epoxy, the injection ports and excess epoxy used to cover the cracks can be grinded off.

- (7)- In order to carry out any externally bonded FRP strengthening, the concrete surface should be blast cleaned, or roughened using other mechanical means. It should be noted that this step is not required if utilizing Near Surface Mounted (NSM) FRP but would involve creating grooves on the girder soffit.

Several methods are available for strengthening RC and PC girders to compensate for damaged reinforcing bars and prestressing strands. These include splicing back the rebar/tendons, externally post-tensioned rods/cables, externally bonded FRP, prestressed/post-tensioned FRP, and a combination of these methods. It should be noted that unless the damage is minor, the exclusive use of FRP material for the retrofit without splicing rebar/tendons may not be possible based on the current design guidelines. Both ACI¹ and AASHTO² guides require that the damaged (or in the case of only strengthening, the existing) structure be capable to carry all dead loads and the majority of the live loads. ACI¹ requires that the damaged structure be capable of resisting 75% of the service live loads (equation 1). Due to the nature of vehicular live loads, AASHTO² requires the damaged structure to resist all service live loads including any impact loading (equation 2). As highlighted in the impact damage case study later in the manuscript, coupling of damaged rebar can be carried out until the capacity of the damaged girder satisfies the ACI (or AASHTO) requirements of the guide specifications for externally bonded repair in equation 1 (or equation 2).

Existing strength of the structure based on ACI¹:

$$(\phi R_n)_{existing} \geq (1.1S_{DL} + 0.75S_{LL})_{new} \quad (1)$$

Where;

R_n = nominal strength of a member
 S_{DL} = dead load effects
 S_{LL} = live load effects
 ϕ = strength reduction factor

Existing strength of the structure based on AASHTO²:

$$R_r \geq \eta_i[(DC + DW) + (LL + IM)] \quad (2)$$

Where;

R_r = factored resistance computed in accordance with AASHTO LRFD;
 η_i = load modifier taken as 1.0;
 DC = force effects due to component and attachments;
 DW = force effects due to wearing surfaces and utilities;
 LL = force effects due to live loads; and
 IM = force effects due to dynamic load allowance.

The NCHRP report by Harries and Miller¹¹ provides good insight on the use of different FRP retrofit methods based on the degree of impact damage to PC girders. The choice of the retrofit material and method would be dictated by the degree of required strengthening, the access to the girder for retrofit construction, as well as minimizing the amount of lane closures during construction. Provided that the damaged girder has the minimum capacity dictated by the relevant design guide, externally bonded FRP is a very economical retrofit method. Precured CFRP plates/laminates and wet-layup of CFRP sheet/fabric are the two most common types of retrofit, with their design governed by the debonding strain dictated by the respective design guide. The AASHTO Guide Specifications² limit the strain at the FRP/concrete interface to 0.005 mm/mm (in./in.). In the ACI guide¹, the strain at which debonding occurs (ϵ_{fd}) for externally bonded FRP is given by Eq. 3.

$$\epsilon_{fd} = 0.041 \sqrt{\frac{f'_c}{nE_ft_f}} \leq 0.9\epsilon_{fu} \quad \text{in SI units} \quad (3)$$

$$\epsilon_{fd} = 0.083 \sqrt{\frac{f'_c}{nE_ft_f}} \leq 0.9\epsilon_{fu} \quad \text{in in.-lb. units}$$

where;

f'_c = Compressive strength of concrete in MPa (psi)
 n = Number of plies of FRP
 E_f = Tensile modulus of elasticity of FRP in MPa (psi)
 t_f = Nominal thickness of one ply of FRP reinforcement in mm (in.)

When working over multilane roadways with limited access, CFRP Rod Panels (CRPs) that can be applied to the girder soffit one panel at a time in a modular fashion, could be more efficient. The CRPs can be conservatively designed as externally bonded FRP^{8,9}. Near Surface Mounted (NSM) FRP rods and strips, that are inserted into grooves cut into the soffit of the girder, provide more efficient use of the FRP material, but may require specialized equipment and personnel for the installation. When strengthening beam soffits, whose width is usually less than 610 mm (24 in.), the contribution of NSM FRP is limited due to groove depth and spacing requirements.

The containment of the damage, from possible future impacts, is achieved using FRP fabric U-wraps, covering the repaired area and the FRP strengthening. In addition to providing containment, the U-wraps anchor the FRP material applied for flexural strengthening, which permits the longitudinal FRP to develop a larger portion of its tensile strain capacity^{16,17}. The U-wraps also confine the concrete around the damaged area and provide additional shear strength to the section. While uniaxial fabric can be used for the U-wrap, a biaxial or triaxial fabric can also be used for the combined shear and flexural strength. The U-wrap application area should be blast or mechanically cleaned, and this can be carried out concurrently with the cleaning of the surface for the FRP used for the flexural strengthening. Following the completion of FRP application, it is recommended that a UV protective coating be applied to protect the epoxy from deterioration over time.

BRIDGE DESCRIPTION AND 2014 IMPACT DAMAGE

The KY 562 Bridge over Interstate 71 (I-71) in Gallatin County, Kentucky, is a four-span Reinforced Concrete Deck on Girder (RCDG) bridge built in 1967. The bridge's two center spans traverse two lanes of northbound and two lanes of southbound I-71 traffic. Each span has four reinforced concrete girders, with a 190 mm (7.5 inch) reinforced concrete deck. The general layout of the bridge and cross section details are presented in Figure 1. The cast-in-place RC girders have a varying depth across the center piers, and constant depth at the middle of the two center spans and near the abutments (Figure 1). Based on the design plans, the reinforcing steel had a tensile yield strength of 276 MPa (40 ksi). The beam concrete had a design compressive strength of 20.7 MPa (3000 psi). Span 3 of the bridge has a posted height clearance of 4.57 m (15 ft) over the roadway. In April of 2014, an over-height truck struck and damaged the exterior girder (Girder 4 of Span 3) over the right lane of northbound I-71. As shown in Figure 2, the over-height truck impact produced significant concrete spalling, especially on the backside of the girder exposing all the rebar. Out of the five #11 rebars (diameter of 35.81 mm) that made the bottom rebar layer, significant out-of-plane bending was seen in one of the #11 rebars, and slight bending in two other rebars. Damage was observed in all the exposed shear stirrups, with one severed and all the other stirrups being bent. While the damaged section was over 3.66 m (12 ft) along the length of the girder, no additional visible cracking was observed in the girder. The damaged section (Figure 2) spanned almost the entire right lane of the two northbound lanes of the interstate highway. While the impact occurred on the right lane, the concrete spalling at the back of the girder spanned approximately 0.3 m (1ft.) across the left lane of the freeway. Although the road on the bridge is a rural road with an Annual Average Daily Traffic (AADT) of less than 400, the northbound two lanes I-71 freeway below carry an AADT of 32,700.

2014 IMPACT DAMAGE RETROFIT

The impacted girder was inspected, and the extent of the damage documented. As detailed in the first stage of the two-stage approach, since the retrofit construction was expected to start several months following the impact, all loose concrete was removed and a glass fiber mesh was applied soon after the inspection to prevent any concrete from falling on to the roadway below. The application of the mesh is shown in Figure 3(a). Due to the low volume of traffic on the bridge, traffic control was setup on top of the bridge to prevent vehicles from traversing on the lane over the damaged girder. As seen in Figure 3(b), a site visit just prior to retrofit construction revealed that the glass fiber mesh may have been damaged by another over-height impact. This prompted another inspection to check if any additional damage had occurred. As no new damage was detected, retrofit construction began in November 2014, more than six months after the impact. Analysis had revealed that the impact was near an inflection point in the multi-span continuous girder bridge. Consequently, the expected flexural demands were low. Based on the observed damage and rebar deformation, the excessively bent #11 rebar was removed and a new straight bar was attached using couplers. The two bars that were slightly bent were mechanically straightened. Although all the rebar was now available to carry the flexural load, the loss of one #11 rebar was conservatively assumed and the FRP strengthening was designed accordingly. A 305 mm (12 in.) wide unidirectional CFRP fabric, able to carry 803 kN/m (55 kips/ft) was applied longitudinally to the bottom soffit as well as the two vertical faces to strengthen the girder at the location damaged by the impact. The shear capacity at the impact section was restored by removing the damaged portions of shear stirrups and splicing new sections back to the existing undamaged portions. Additional U-shaped partial stirrups were placed between two of the repaired original stirrups to increase confinement. These were attached to anchor bolts that were drilled vertically