6). While no carbonation of concrete can be considered beneficial to steel rebars because the pH remains at high values, the opposite is true for GFRP reinforcement that is more sensitive to high alkalinity. Thus, the GFRP bars extracted from these cores were subject to an aggressive alkaline environment over the 17 and 11 years of service for Walker and Southview bridge respectively.

Chloride diffusion measurement

An adaptation of the rapid migration test (RMT) using silver nitrate solution was employed to determine the chloride diffusion in the concrete samples. Two concrete samples were cut in order to provide fresh split surfaces. A 0.1 mol/L silver nitrate solution was poured on the entire cut surface[9]. In the presence of chloride, a clearly visible white/silver precipitation takes place on the surface while in the absence of chlorides, the solution reacts with the hydroxides present in the concrete, changing the surface color to brown. No clear evidence of chloride diffusion was observed in all the tested specimens of both bridges using this method. It was noticed that the surface became darker, to a color similar to brown, while there was no visible gray area (Fig 7).

GFRP CHARACTERIZATION

SEM imaging

The GFRP microstructure was investigated since it is a critical parameter in performance and durability of GFRP bars[10]. The full cross-section of prepared GFRP coupons was scanned using SEM at different levels of magnification and images were taken at random locations. Attention was paid to the areas in the vicinity of the bar edges since possible degradation due to chemical attack starts at GFRP-concrete interface. Representative images are shown in Fig 8 and Fig 9. SEM analysis suggests that there was no apparent sign of deterioration in the GFRP coupons. No damage was observed in the matrix and at the matrix-fiber interface. Glass fibers appeared to be intact without no loss of cross-sectional area.

EDS analysis

EDS was performed at several locations of each GFRP slices with a focus on the edge of the bar to identify existing chemical elements. Results are shown in Fig 10 and Fig 11 where the vertical axis corresponds to the counts (number of X-rays received and processed by the detector) and the horizontal axis presents the energy level of those counts. Si, Al, Ca (from glass fibers) and C (from the matrix) were the predominant chemical elements in the extracted samples. No apparent sign of any chemical attack was observed in the bars.

Horizontal shear strength

The horizontal shear strength of the extracted GFRP coupons was determined following ASTM D4475[11] as a useful parameter for durability evaluation. The test was performed on three GFRP coupons extracted from Southview Bridge: i) one No. 4 GFRP bar with the total length of (58 mm) 2.3 in, and ii) two No. 6 GFRP bars with the total length of 76 mm (3 in.) and (74 mm) 2.9 in. No horizontal shear test was performed on samples extracted from Walker Bridge due to their small diameter. Since no historic data was available at the time of construction, the results were compared to the test performed on pristine bars produced by the same manufacturer in 2015 as a benchmark. Specimens were tested with the span-to-diameter ratio equal to three, according to standard and compared with pristine samples. The test was performed in displacement control with the rate of 1.27 mm/min (0.05 in/min) of the cross head (Fig 12).

All three specimens presented the horizontal shear mode of failure and the shear strengths were determined following ASTM-D4475 as:

$$S = 0.849\frac{P}{d^2} \tag{1}$$

where S is the horizontal shear strength, P refers to the breaking load and d corresponds to the nominal diameter of the specimen. A summary of the results is shown in Table 2 where S_c and S_s, refer to the shear strength of control samples tested in 2015 and extracted samples, respectively. The same notation is employed for the failure load. The extracted GFRP bars showed about 5% increase in horizontal shear strength compared to the samples produce in 2015.

Since the horizontal shear is affected by the resin properties, the increase may be a result of resin crosslinking over time especially if it was not cured 100% at time of construction. It is recognized that the number of tested samples was not sufficient for a proper statistical analysis. However, the result of horizontal shear strength provided an additional evidence of GFRP long-term durability.

Glass transition temperature (T_g)

The changes in T_g of the polymer matrix was determined by performing dynamic mechanical analysis (DMA) test on three specimens for each bridge. Rectangular specimens with dimensions of 1×5×50 mm (0.04×0.2×2.0 in.) were extracted from the bars according to ASTM E1640[12]. The DMA test was performed with a three-point-bending fixture for a temperature ranging from 30 to 130 °C (86 to 266 °F), and a heating rate of 1 °C/min (1.8 °F/min). Due to lack of T_g test data on GFRP bars at the time of construction, T_g tests were performed on samples from pristine bars produced in 2015 from the same manufacturer, to serve as a benchmark. Table 3 provides the result summary, where T_g^c and T_g^s respectively refer to glass transition temperature of the control and extracted GFRP samples.

The T_g of the extracted samples were higher than the control samples pultruded in 2015. While due to the changes in glass fibers and resin formulation of the bars manufactured in 2015 compared to the ones produced in 1999 and 2004, a direct comparison is not possible. In general, T_g is expected to increase over time due to cross-linking of the resin if it is not 100% cured at the time of production[13].

Fiber content

The fiber content ratio of GFRP samples was determined following the ASTM D2584[14]. Three samples from each bridge were tested for change in mass. Samples were first placed inside the furnace for 40 minutes at 425 °C (797 °F) and then were left inside the furnace at 700 °C (1292 °F) for 30 minutes to burn off the resin completely. The weight of sand particles and wrapping strand at the GFRP surface was also eliminated to provide a precise estimation of fiber content. The result was compared with the same test performed on samples produced in 2015. Table 4 shows the summary of the result where α_c and α_s respectively correspond to fiber ratio of control and extracted samples. The measured fiber content after years of field exposure was consistent with the expected values and well above the minimum fiber content requirement of 70% by mass[15].

CONCLUSIONS

GFRP and concrete samples were extracted from two bridges more than a decade old. The concrete pH was in the range of 11-12 which was consistent with the concrete type and age. No indication of carbonation and chloride diffusion was observed in the concrete cores. Different tests were performed to investigate the condition of extracted GFRP bars. Microscopic examination did not show any GFRP degradation and no apparent sign of chemical attack was observed by performing EDS analysis. Fibers did not lose any cross- sectional area, the polymeric matrix was intact and no damage was observed at the fiber-matrix interface. T_g of the extracted GFRP bar was higher than that of the control samples produced in 2015 by the same manufacturer. T_g has probably increased over time due to cross-linking of the resin since the resin was not 100% cured at the time of production. The horizontal shear strength of the extracted GFRP samples from the Southview Bridge was about 5% higher compared to the average horizontal strength of the pristine bars manufactured in 2015. The increase may be a result of resin cross-linking over time. The result of fiber content measurement of extracted GFRP bars was consistent with that of the pristine bars manufactured in 2015 confirming that there was no apparent loss of fiber content in GFRP bars.

This study confirms that GFRP bars maintained their microstructural integrity after years of service in both bridges. In case of Walker Bridge, although the use of polyester resin GFRP bars is excluded presently, the extracted GFRP samples from Walker Bridge did not show any apparent sign of degradation after seventeen years of service which provide additional evidence that the accelerated laboratory conditioning tests could be overly conservative. This study suggests that GFRP bars can be a feasible solution for corrosion problem of the conventional steel-RC structures in order to increase the service life of the structures.

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Table 1—Guaranteed properties of the GFRP bars used in Walker & Southview Bridges provided by the manufacturer at the time of construction

Structure	Diameter (mm)	Tensile Strength f^* (MPa)	Elastic Modulus E_f (GPa)	Rapture Strain (%)
Walker	6	758	40.7	1.9
Southview	9	758	40.8	1.8
Southview	13	689	40.8	1.7
Southview	19	621	40.8	1.5

Note: 1 mm= 0.0394 in.; 1 MPa= 0.145 ksi.

Table 2—Results of the horizontal shear tests performed on extracted GFRP bars compared with the bars produced in 2015

Nominal Diameter (mm)	Span Length (mm)	P_c			P_s		S_c (MPa)	S_s (MPa)	Ratio (S_s/S_c)
		No. of Samples	Average (kN)	CoV (%)	No. of Samples	Average (kN)			
13	38	5	8.8	2.4	1	9.33	46.5	49.1	1.05
19	57	5	20.5	3.6	2	21.7	47.9	50.7	1.06

Note: 1 mm= 0.0394 in.; 1 kN= 0.2248 kips; MPa= 0.145 ksi.

Table 3—Results of T_g performed on extracted GFRP bars compared with the bars produced in 2015

Structure	T_g^c			T_g^s		
	No. of Samples	Average(˚C)	CoV (%)	No. of Samples	Average (˚C)	CoV (%)
Walker	3	81.0	16.9	3	111.9	2.5
Southview	3	81.0	16.9	3	100.6	2

Note: ˚F=1.8˚C+32

Table 4—Results of fiber content measurement performed on extracted GFRP bars compared with the bars produced in 2015

Bridge	α_c			α_s		
	No. of Samples	Average (%)	CoV (%)	No. of Samples	Average (%)	CoV (%)
Walker	4	75.7	1.2	4	82.38	4.0
Southview	4	75.7	1.2	4	73.4	2.0

Fig. 1—Old (left) and new (right) Walker Bridge

Fig. 2—A complete GFRP cage before concrete casting

Fig. 3—A view of the former Southview Bridge

Fig. 4—Southview Bridge deck top-GFRP layer and CFRP tendons

Fig. 5—GFRP coupons extracted from the concrete cores of Walker (left) and Southview (left) Bridge

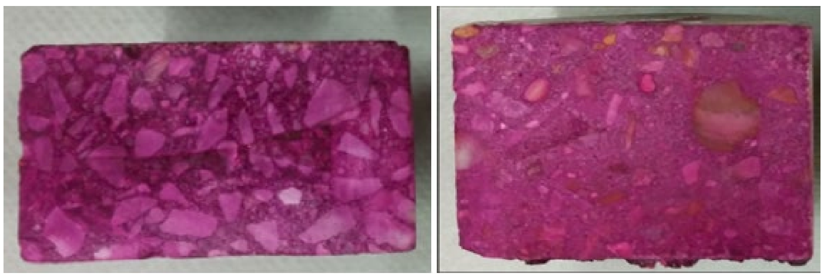

Fig. 6—Carbonation depth measurement of concrete samples extracted from Walker (left) and Southview (right) Bridge

Fig. 7—Chloride diffusion measurement of concrete samples extracted from Walker (left) and Southview (right) Bridge

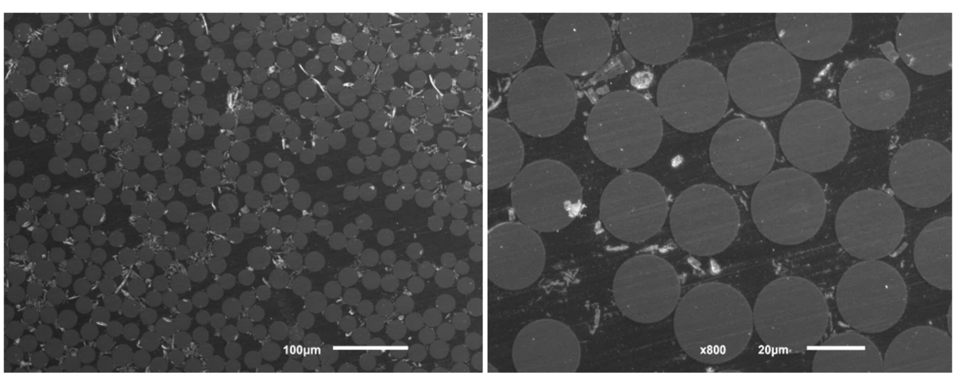

Fig. 8—SEM images of GFRP bar after 17 years of service in Walker Bridge in magnification levels of 200x (left) and 800x (right)

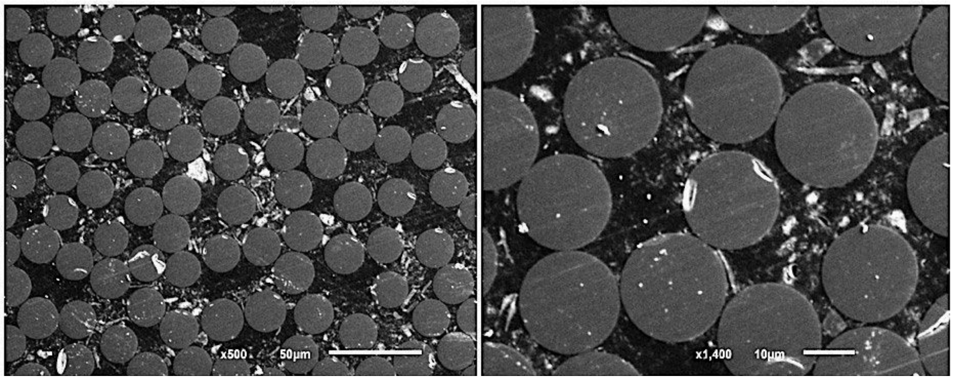

Fig. 9—SEM images of GFRP bar after 11 years of service in Southview Bridge in magnification levels of 500x (left) and 1400x (right)

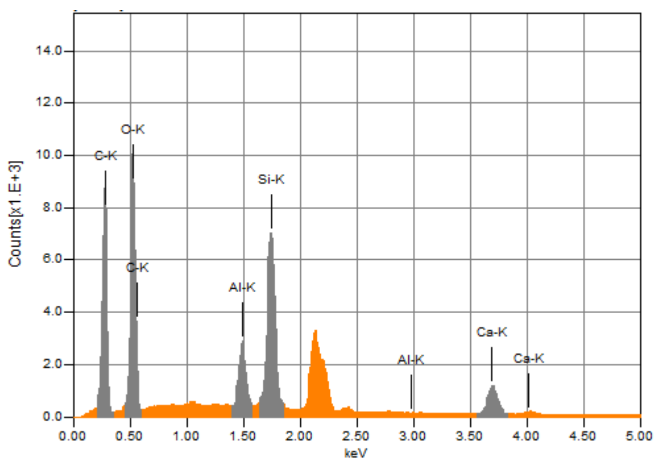

Fig. 10—Result of the EDS analysis performed on GFRP samples extracted from Walker Bridge

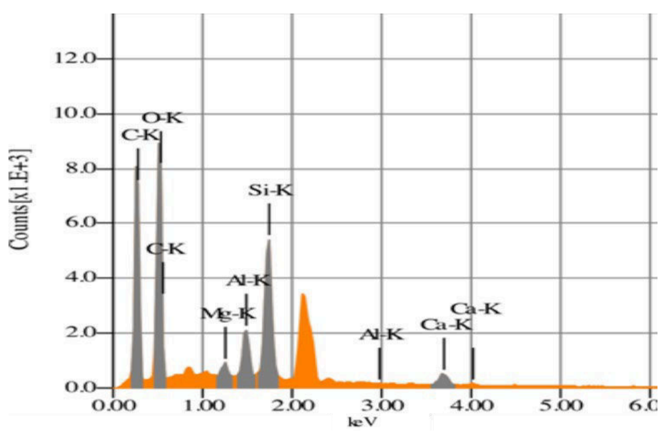

Fig. 11—Result of the EDS analysis performed on GFRP samples extracted from Southview Bridge

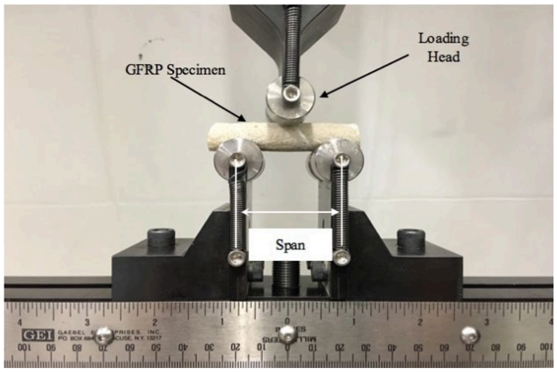

Fig. 12—Horizontal shear test performed on GFRP bars extracted from Southview Bridge

SP-331-6

Improving the Durability of Impact Damaged PC Bridge Girders Using CFRP Rod Panel Retrofit

Abheetha Peiris and Issam Harik

Synopsis: An exterior girder of a prestressed concrete bridge over Interstate 65 in Kentucky was damaged due to an over-height truck impact. The damaged section spanned two of the three northbound lanes of the highway. Two prestressing strands were severed and two additional strands were damaged by the impact. In addition, shear reinforcing bars in the vicinity of the impact were cut-off. CFRP Rod Panels (CRPs) were deployed to restore some of the load carrying capacity lost due to the severed prestressing tendons. CRP 195, with CFRP rods of 3.96 mm (0.156 in) diameter, having a capacity of 867 kN (195,000 lbs.) per 305 mm (1 ft.) width of panel, was selected for the flexural strengthening. A triaxial braided quasi-isotropic CFRP fabric was selected for shear strengthening and served as containment of crushed concrete in the event of future over-height impacts. Since the ACI and AASHTO Codes or Guides do not directly address the design with CRPs, strain limits based on debonding of the rods similar to externally bonded CFRP (EB-CFRP) are imposed when determining the retrofitted beam capacity. The load rating evaluation of the impacted beam, the retrofit analysis and design, and the field repair stages are presented and discussed.

Keywords: CFRP, Impact Damage, Load Rating, Prestressed Concrete, Residual Capacity, Retrofit, Rod Panel

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INTRODUCTION

Damage to bridge beams 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. 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 on traffic. Fiber Reinforced Polymer (FRP) materials have emergence over the last few decades as a reliable and effective method of externally strengthening bridge beams, especially concrete bridges. They can be highly durable and cost effective due to their non-corrosive nature, high strength to weight ratio and rapid application capability when compared with traditional methods. The use of bonded FRP laminates and fabrics to repair and strengthen concrete structures is well established, with design guidelines by the American Concrete Institute (ACI) [1], American Association of State Highway and Transportation Officials (AASHTO) [2], Japan Society of Civil Engineers (JSCE) [3], International Federation for Structural Concrete (fib) [4] and several others. Among FRP material, Carbon FRP (CFRP) has been the primary material utilized for the repair and retrofit of over-height impacted bridge girders. Considerable research has been done on strengthening Prestressed Concrete (PC) bridges using CFRP, but compared to RC bridges; only a few field applications have been documented. CFRP in the form of externally bonded CFRP (EB-CFRP) pultruded laminates and wet-layup sheets/fabric has been successfully used to repair and strengthen bridges with impact damage [5-7]. Prestressed CFRP sheets have also been utilized in the retrofit of PC bridges [8]. A recent National Cooperative Highway Research Program report [9] highlighted these methods utilizing CFRP for PC bridge retrofit. The report categorized the use of different methods based on the degree of impact damage to the PC girder. Experimental research on prestressed Near Surface Mounted (NSM) CFRP bars for strengthening impacted PC beams have also been carried out [10]. But as Kasan et al [11] points out, when compared to an EB-CFRP application, NSM-CFRP might not be efficient for use in positive bending regions due to higher cost in resources and installation.

Impact damage to PC girders typically occurs at or near the maximum positive moment region of the girder. While large impacts causing collapse or replacement of PC girders have occurred [9], due to vertical clearance limits with the roadway grade, most impacts tend to be on one of the exterior girders. Depending on the location and size of impact, the damage can span multiple lanes of traffic, causing traffic related delays during the retrofit construction period. While the concrete repair work can be carried out segmentally, the application of CFRP laminates typically requires the closure of all affected traffic lanes to maintain continuity over the entire retrofit area. This would always be the case for most prestressed CFRP applications. While utilizing a splice plate to maintain continuity is viable for non-prestressed EB-CFRP laminates, studies have shown that splice plate debonding is the primary failure mode, especially when the splice is near the maximum moment region [12]. Carbon fabric/sheets can be overlapped to maintain continuity, but typically require multiple layers of application to achieve the capacity limits required for bridge strengthening. A more durable and modular strengthening system using CFRP Rod Panels (CRPs), originally developed by the authors using ultra high modulus CFRP strips for steel beam strengthening, has been successfully adopted for strengthening reinforced concrete beams [13-14]. The CRPs are produced using small-diameter CFRP rods, which are mounted on a fiberglass backing to maintain spacing greater than the rod diameter between individual rods as seen in Figure 1. While the length of the panels can be varied, the currently developed panels are 1220 mm (48 in.) in length. The selected length allows individual workers to handle and mount the panels on the soffit of a girder. As seen in Fig. 1, the modular construction of the system is possible through the use of a finger joint between adjacent panels. The CRPs are bonded to the concrete surface using the same two part structural epoxies utilized for typical EB-CFRP laminate applications. Each alternate rod panel, identified as the '+' panel in Figure 1, is produced with an extra rod to establish symmetry at the finger joint. The 152 mm (6 in.) overlap for the finger joint was a conservative selection based on results of double lap shear tests [13-15] and concrete beam tests [13,16]. The present design was developed following both concrete bond tests and concrete beam tests with CFRP rods of diameters varying from 1.98 mm (0.078 in.) to 3.96 mm (0.156 in.), and utilize CFRP with manufacturer reported tensile modulus of 134 GPa (19,500 ksi) and an ultimate tensile strength of 2,200 MPa (320 ksi).