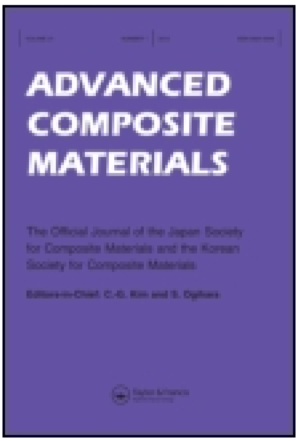

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The application of composites for the rehabilitation of concrete bridge infrastructure

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Abstract—This paper describes recent efforts at the Materials & Manufacturing Directorate of the US Air Force Research Laboratory (AFRL) to demonstrate the feasibility of rehabilitating concrete beams in bridge decks with fiber-reinforced composites. Early work in this program built on the success in Europe in repairing bridge beams by bonding thin composite plates to their lower surfaces. Materials and processes were appropriately selected, validated with flexural tests on scaled-down and full-sized concrete beams and implemented in a vehicular bridge in the field. The durability of this rehabilitation scheme was evaluated from actual exposure in service. From the results emerged a novel concept: the use of composite rods (instead of plates), embedded in longitudinal grooves in the lower face of the beam to improve flexural strength and stiffness. In addition to improved affordability, convenience and performance, this approach provides the unique ability to rehabilitate *deteriorated* concrete beams, in service, without the necessity of strengthening or otherwise preparing the concrete. The results demonstrate the viability of this repair scheme.

Keywords: Concrete beam; rehabilitation; composite plate and rod; stiffening; strengthening; bending moment; deflection.

1. INTRODUCTION

With the institution of the interstate highway system in the US in 1956, a large number of bridges were added to the national inventory. Many of these bridges are 40–50 years old (past their average life span) and in a state of disrepair. By one estimate [1], reconstruction of these bridges will average $860 per square meter of deck area for a total of about $200 billion in today's dollars. However, annual expenditures in this area are insufficient to overcome annual deterioration. This points to the need to repair or rehabilitate these aging bridges, thereby extending their useful lives and allowing staggered replacement in line with state or federal

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budgets. Rehabilitation itself is not inexpensive, and new and innovative materials, processes and designs are required to further lower costs.

Repair of concrete bridges has progressed from external post-tensioning and additional supports to flexural stiffening or strengthening of concrete beams with bonded steel plates. The advantages of the latter are lower cost, ease of application and maintenance, elimination of special anchorages, and the ability to strengthen the structure while it remains in use. The bonded plate serves as a second layer of reinforcement that can increase flexural strength by as much as 40% [2] and provide crack control for the concrete member. Corrosion at the steel/adhesive interface over time, however, results in deterioration in bond strength and reinforcing efficiency which led to a switch to carbon fiber-reinforced plastics (CFRP) as the reinforcing material of choice. CFRP offer distinct advantages over steel in this capacity with higher specific stiffness and strength, excellent durability in a saline environment, resistance to corrosion by acids and salts over a wide range of temperatures, and the ability to be tailored to provide the desired mechanical properties. Although much costlier than steel, composites have lighter weight and better corrosion resistance that can result in significant reductions in fabrication and long-term costs. Research in this area originated in the mid-1980s in Switzerland [3] and has continued to expand [4–6], with many commercial repair systems currently on the market.

Successful incorporation of nontraditional materials and processes (M&P) in the repair of concrete structures requires an interdisciplinary teaming effort between the M&P experts and the civil engineering industry. A second requirement is implementation of the technology in the field to evaluate its performance under actual service conditions. Accordingly, the AFRL Materials & Manufacturing Directorate established a partnership with an architectural firm, Lockwood, Jones & Beals, Inc., the Butler County Engineer's Office in Ohio and the University of Dayton Research Institute to develop repair methodologies using fiber-reinforced composites and implement them in the field. This paper describes the results of those studies, including laboratory evaluations, scale-up to tests on full-sized beams, and field implementation.

2. REHABILITATION WITH EXTERNALLY BONDED COMPOSITE PLATES

Unidirectional composite plates were fabricated from graphite/epoxy prepreg (AS4C/1919 from Hercules, Inc.) for this work, and the surfaces mechanically abraded to improve adhesion. For laboratory studies, the plates were bonded to 2.6-m $\times$ 46-cm $\times$ 15-cm deep construction-grade, steel-reinforced concrete beams with an ambient-cure, two-part epoxy adhesive (EA9460 from Dexter Hysol, Inc.). The relevant composite, concrete, adhesive and reinforcing steel properties are given in Table 1. Since joints are both practical and necessary in field rehabilitation with composite plates, various joint configurations were evaluated before settling on a doubler joint. Each doubler joint had a minimum plate overlap of 15.2 cm and was

Table 1.
Material properties for concrete test beam constituents

Property	Composite plate	Composite rod	Steel rebar	Concrete	Adhesive
Fiber volume (%)	59	56	—	—	—
Yield stress (MPa)	—	—	414	—	—
Tensile modulus (GPa)	138	122	200	—	2.8
Tensile strength (MPa)	1930	—	—	—	30
Ultimate strain (%)	1.4	—	—	—	3.5
Flex. strength (MPa)	—	1326	—	—	—
Compr. strength (MPa)	—	—	—	5.1	—

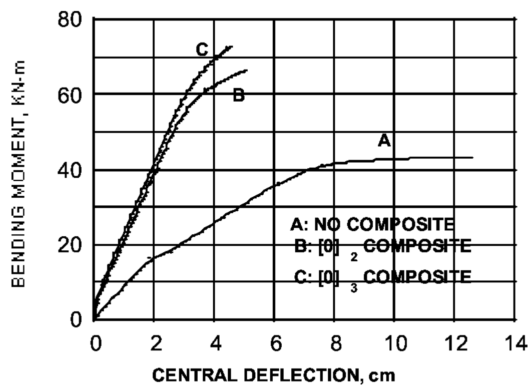

Figure 1. Bending moments *vs.* measured deflections of 2.6-m reinforced concrete beams with and without externally bonded composite plates.

constructed with the same adhesive used to bond the composite to concrete. The two-part adhesive was mixed in accordance with the manufacturer's instructions, applied to the beam's tensile surface, and overlaid with the composite plates. The entire plate area was then enclosed in a vacuum bag, sealed against the sides of the concrete beam and evacuated, to generate a pressure of approximately 82 kPa for a minimum of 24 hours. Bonding in this manner resulted in excellent adhesion, with the vacuum bag promoting intimate contact between adhesive and adherends during adhesive cure. Consequently, in all subsequent flexural testing, the composite plate did not peel away from the concrete beam up to final failure.

The composite-reinforced 2.6-m beams were tested in four-point flexure in accordance with ASTM C78-84, with leather pads between the loading pins and beam to evenly distribute the applied load. Typical results, shown in Fig. 1, demonstrate a substantial increase in beam stiffness and moment capacity with bonded composite plates. Failure in the control beam initiated via tensile cracks in the concrete and continued with yielding of the steel rebar. Failure in the plate-reinforced beam also initiated via tensile cracking of the concrete, albeit at higher loads. Subsequent failure depended on the cross-sectional area of the

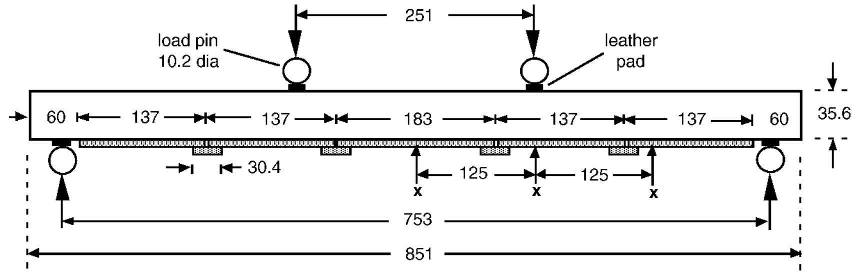

Figure 2. Experimental set-up for flexural testing of 8.5-m concrete beam with bonded composite plates (all dimensions in cm).

composite plate and the location of the composite joint. This failure ranged from longitudinal splitting and partial fracture of the outer plate in the doubler joint followed instantaneously by its debonding from the inner plate in the joint, to compressive failure of the concrete followed by tensile fracture of the composite plate in the central region of the beam. These results provided guidance for the design and application of composite plates (with joints) in full-sized concrete bridge beams.

This rehabilitation technology was scaled up to tests on three 8.5-m beams. The beam dimensions and composite joint locations are shown in Fig. 2. Beam 1 was the control beam. Beam 2 was flipped over for ease of bonding of the composite plates to its tensile surface, while composite plates were bonded overhead to the tensile surface of Beam 3 to simulate their application in the field. For convenience of bonding (in the field) the width of the plate was reduced for Beam 3; the plate cross-sectional area was maintained the same as in Beam 2, however, by adding another ply to the composite. Failure of Beam 1 under flexural loading occurred via yielding of the steel rebar at a bending moment of approximately 400 kN m. Beyond this point the beam continued to deflect with no significant increase in bending moment, with the concrete eventually failing in compression at a maximum bending moment of 435 kN m. Failure in Beam 2, reinforced with $[0]_4$ composite plates, initiated via longitudinal splitting of the plate in the region of maximum bending moment. Ultimate failure was precipitated by fracture of the concrete in compression. This was followed instantaneously by failure via composite/composite debonding in the joint accompanied by failure in the concrete adjacent to the composite. This failure scenario confirms the existence of a strong bond between the concrete and composite. On the other hand, interfacial failure at a lap instead of composite tensile fracture (following concrete compressive failure) suggests the need to improve the composite/composite bond. Ultimate failure occurred at a maximum bending moment of 565 kN m, representing a 30% increase in moment capacity over the control beam. The failure sequence in Beam 3 paralleled that observed in Beam 2 with two notable exceptions: ultimate failure was precipitated by debonding of the composite/composite lap (the reduced plate width results in a reduced lap area), and occurred at a lower maximum bending moment of 530 kN m.

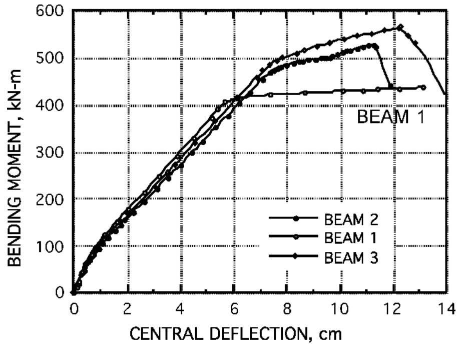

Figure 3. Bending moments of three 8.5-m reinforced concrete test beam *vs.* measured deflections. Beam 1: no composite reinforcement; Beam 2: $[0]_{4T}$ plate reinforcement; Beam 3: $[0]_{5T}$ plate reinforcement.

From a comparison of the flexural test data for these three beams (Fig. 3), it is apparent that the ratio of composite to reinforced concrete utilized in this study did not enhance beam stiffness. It may be concluded from the results that due to the difficulty in working overhead with Beam 3, the application of adhesive and the adhesive bond obtained were not as efficient as with Beam 2, resulting in the observed failure in the composite lap joint instead of concrete compressive failure. Up until ultimate failure, however, no plate peeling was observed either at the ends of the plate or in the composite joints. These results validate the improvements seen in tests on the scaled-down concrete beams and provide the materials, process and design for an approach to the rehabilitation of concrete beams.

3. FIELD IMPLEMENTATION OF PLATE BONDING

The technology developed in these laboratory studies was applied in the field to demonstrate the feasibility of strengthening concrete beams in a vehicular bridge deck. The bridge, on Fear Not Mills Road in Butler County, Ohio, was built in 1994. It has a single span of 8.05 m and a width of 9.14 m, with a conventionally reinforced precast concrete box-beam superstructure and an asphalt wearing surface. The 10 box beams that comprise the deck are identical to those used in the scaled-up lab studies. Composite plates, identical to those employed in Beam 3, were bonded to each of the two exterior box beams of the bridge deck using doubler joints. The only difference from Beam 3 was an increase in the composite/composite lap length in the joint from 15.2 cm to 22.9 cm. The exterior box beams were selected since they receive the most severe exposure to the natural environment as well as to gasoline, oil, deicing chemicals, etc. The lower faces of the beams were sandblasted to remove the outer weak cement layer, followed by a high-pressure water wash. All aggregate projecting below the plane of the lower face was mechanically ground down. Composite plates were bonded as described earlier, with pressure during adhesive cure once again applied with a vacuum bag.

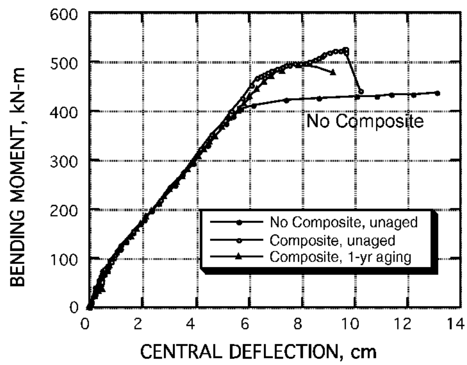

Figure 4. Comparison of maximum bending moments of 8.5-m concrete beams without external reinforcement, and with external composite plate reinforcement (unaged and aged for one year), *vs.* measured central deflections.

These rehabilitated beams were retained in service (with corresponding exposure to the elements, temperature fluctuations, deicing chemicals and static and dynamic loads) for one year. Over this period of time bond integrity was monitored using an acoustic tap-hammer technique to detect regions where the composite plate debonded from the adhesive layer. Measurements were made one, three and 10 months after the rehabilitation. The percentage of debonded area increased slightly with time from one to 10 months. After a year in service, one of these exterior beams was removed and tested in flexure under conditions identical to those used for Beam 3. A comparison of the test results is shown in Fig. 4. As is evident from the plot, a year's exposure to the outdoor application environment (and the resulting increase in composite/adhesive debonded area) resulted in a minimal reduction in the maximum bending moment of the beam. The sequence of failure events was also similar to that of Beam 3. Post-failure examination of the composite/composite lap region revealed areas between composite and adhesive that were initially unbonded, in addition to those that debonded later, in service. This points to a need to optimize the plate bonding conditions to minimize such unbonded areas on adhesive cure. In five to six years from the rehabilitation date, the other plate-reinforced exterior beam will be similarly removed and tested.

4. REHABILITATION WITH EMBEDDED COMPOSITE RODS

Although composite reinforcement of aging infrastructure has demonstrated benefits, various materials, material forms and processes may be employed to this end. Initial attempts focused on bonding precured composite plates to the concrete, followed by the lay-up and *in-situ* cure of composite prepreg on the concrete member. There are a number of disadvantages to rehabilitating concrete bridge beams with composite *plate* reinforcement:

- Precured plates have to be fabricated in sizes that are practical and convenient to transport to the field, necessitating the development of strong, efficient plate joining techniques for beams with long spans.
- Plate reinforcement typically covers the entire lower surface of the rehabilitated concrete beam, restricting the drainage of absorbed water that may accumulate at the bondline, accelerating its degradation.
- Optimum bonding requires proper surface preparation and application of pressure during adhesive cure, to promote intimate contact between adhesive and adherends and minimize voids.
- Free-edge, thermal and residual stresses, in conjunction with the service environment, can promote plate debonding and consequent loss of reinforcing capability over time.

Some of these disadvantages can be overcome by using composite rods (of the same cross-sectional area as the plate) embedded in parallel, longitudinal grooves cut into the tensile face of the beam. Continuous lengths of pultruded rods can be transported to the field in rolls and cut to the necessary length, thereby avoiding composite joints. Since the rods cover a small fraction of the beam's lower surface, drainage of absorbed water is not hindered. The surface-to-volume ratio of a rod is substantially lower than for a plate; this, coupled with the fact that the rods are completely embedded in the adhesive makes them less susceptible to interfacial degradation in service. Pultruded rods are also less expensive to fabricate than precured composite plates and easier to employ in the field, requiring no pressure application during adhesive cure. This approach to use pultruded CFRP rods to strengthen structural members is the subject of a patent application by Lockwood, Jones and Beals, Inc. One disadvantage of this rehabilitation technique is the difficulty in cutting grooves on the lower surface of a concrete beam in the field.

Experiments were conducted with unidirectional, pultruded composite rods from DFI Pultruded Composites, Inc. The composite consists of a matrix of seven parts vinyl ester (Hetron 922HV) and one part polyester (Aropol 703) from Ashland Chemical Co., reinforced with T-300 PAN-based carbon fiber. Longitudinal rectangular grooves were machined on the lower faces of beams using a portable masonry cutter and lined with beads of the same epoxy adhesive used for plate bonding. Composite rods, the length of the beam, were sanded, wiped clean with acetone, and embedded in the epoxy within the grooves. The adhesive was then allowed to cure overnight under ambient conditions. In initial studies, four composite rods (with a total cross-sectional area of 1.97 cm^2) were used in 2.6-m beams to replace the plate (1.45 cm^2 cross-sectional area) employed in earlier studies. The larger cross-sectional area of the rods was compensated by their closer proximity (in the grooves) to the beam's neutral axis. The test data for the two reinforcement types were similar. The failure modes, however, were different. Ultimate failure in both beams initiated with compression fracture of the concrete. In the plate-reinforced beam, secondary, catastrophic failure occurred almost

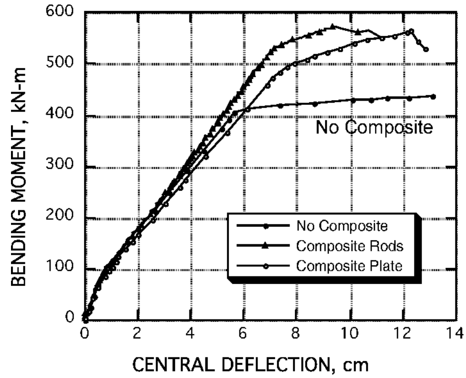

Figure 5. Comparison of maximum bending moments of 8.5-m concrete beams without external reinforcement, reinforced with composite plates, and reinforced with composite rods, *vs.* measured central deflections.

instantaneously through composite fracture or composite/composite lap failure. In the rod-reinforced beam, however, secondary failure was noncatastrophic, with limited debonding and partial fracture of the embedded rods, allowing the beam to sustain the applied load with gradual deformation up to the limits of the test fixture. Total extension of the rods during loading did not exceed 1%, and post-failure examination revealed no slippage of the rods at the ends of the beam. In this method of rehabilitation, machining the longitudinal grooves for the composite rods entails cutting through the steel stirrups that support the steel rebars during beam fabrication. However, this does not appear to adversely affect the mechanical performance of the beam in flexure.

The success with rod reinforcement of 2.6-m beams led to a scale-up of this reinforcing scheme to as-fabricated 8.5-m beams. Eleven composite rods (with a total cross-sectional area of 5.43 cm^2) were embedded in parallel grooves equally spaced on the lower surface of the beam. This beam was tested in flexure, and the data are compared in Fig. 5 with those from flexural tests on similar beams with plate reinforcement and without any composite reinforcement. The beam with rod reinforcement compares well against the beam with plate reinforcement, even though the comparison favors the latter because of its higher composite cross-sectional area (6.13 cm^2). For an equivalent or slightly larger rod area (to compensate for the closer proximity of the rods to the beam's neutral axis), the beam would have a significantly higher maximum bending moment (at ultimate failure) compared to a beam with plate reinforcement. This difference in maximum bending moments may be explained by the differences in failure modes. In the beam with plate reinforcement, premature composite/composite lap failure triggers ultimate failure of the beam; in the beam with rod reinforcement, the concrete fails first in compression, at a higher applied bending moment, followed by noncatastrophic secondary failure events (partial rod debonding and fracture).

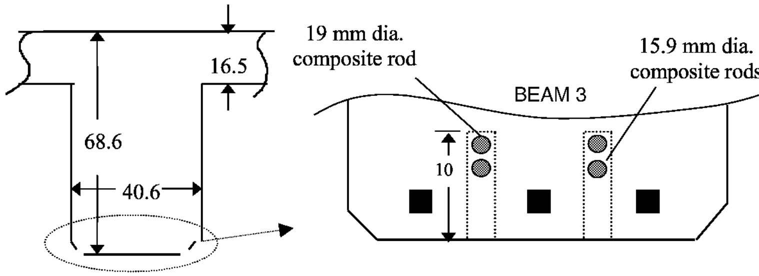

Figure 6. Schematic of Beam 3 from deteriorated bridge deck, tested in flexure (dimensions in cm).

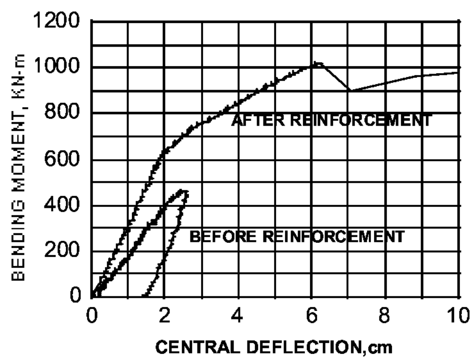

Figure 7. Comparison of maximum bending moments of deteriorated 10.4-m concrete beams as functions of central deflections, shown before and after reinforcement with composite rods.

In laboratory tests, rehabilitation with composites is generally evaluated on as-fabricated concrete beams or those subjected to controlled damage. Little work has been reported on the rehabilitation with composites of beams *aged in service* in which, in addition to mechanical damage, the constituent properties have been severely degraded. Any reinforcing scheme is only as good as its weakest link. The drawback with using external plate bonding to rehabilitate concrete beams deteriorated in service is that the plate is bonded to weakened concrete which becomes the locus of failure under flexural loading, at significantly lower bending moments than in virgin beams. Therefore, while laboratory tests demonstrate the efficacy of this rehabilitation scheme, the question remains as to whether weakened concrete severely undermines the reinforcing capacity of the bonded composite plate. In rehabilitation with embedded composite rods, on the other hand, this problem does not arise; grooves may be machined down to a depth at which the concrete has its original strength, and the composite rods embedded with adhesive in this sound concrete. This was the approach taken to evaluate this reinforcing concept with three 10.4-m beams recovered from a bridge deck, replaced after 80+ years in service.

All three beams cut from the concrete bridge deck were of slightly different dimensions, in different states of disrepair, and had different amounts of steel reinforcement. Consequently, it was not possible to use the results from the flexural testing of any one beam as a baseline for comparison. Grooves were therefore first machined in two of the beams. They were then preloaded in flexure to obtain a baseline, reinforced with composite rods, and then tested again in flexure to failure. Figure 6 shows a schematic of the cross section of one of the beams (Beam 3). Composite reinforcement for this beam consisted of two rods 15.9 mm in diameter, and two rods 19 mm in diameter, located as shown in the figure. The flexural test data for this beam (the most deteriorated of the three beams) are displayed in Fig. 7. This beam was loaded up to the first sign of steel yielding, as evidenced from the start of a knee in the corresponding load trace, then unloaded, and reinforced with the composite rods. As evidenced from the trace for the second loading, the rod-reinforcement results in a significant increase in flexural stiffness and an approximately 40% increase in maximum bending moment at which the steel yields. Ultimate failure is precipitated by compressive fracture of the concrete. This event is not catastrophic, however, and the beam continues to deflect while sustaining significant load. Post-failure examination revealed no slippage of the composite rods within the epoxy adhesive; secondary failure occurred in the concrete below the machined grooves and through partial fracture of the composite rods before the test was terminated. An interesting feature in the secondary failure process was the tensile fracture of an exposed steel rebar.

5. SUMMARY

Concrete bridge beams can be significantly strengthened by external reinforcement with fiber-reinforced composites. Composite rods, embedded in grooves in the beams' tensile faces offer several advantages over externally bonded composite plates, chief among which is the ability to use the former to rehabilitate deteriorated concrete.

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