

Rapid Prestressed Concrete Retrofit with Prestressed Mechanically-Fastened Fiber-Reinforced Polymer: Field Performance and Observation for a Deteriorated Prestressed Concrete Bridge

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Abstract

This paper presents repairs to rural bridges in North Carolina that deteriorated as a result variously of aging, overweight traffic, and exposure to salts and sulfates. The prestressed concrete C-channel superstructures exhibited prestressing strand loss and displayed significant concrete spalling, with one structure having to be closed to traffic after a routine inspection. Analysis conducted using the American Association of State Highway and Transportation Officials (AASHTO) bridge load rating criteria concluded that repair techniques which strengthen deteriorated flexural elements without also restoring lost prestressing forces are insufficient to maintain load ratings in C-channel structures with heavily damaged prestressing tendons. A prestressed mechanically-fastened fiber-reinforced polymer (MF-FRP) retrofit solution was developed and successfully installed on three structures by the authors and North Carolina Department of Transportation maintenance crews. The most extensive of these three repairs is presented here in detail. The field applications and associated analysis show the temporary MF-FRP repair system is capable of restoring lost prestressing forces, allowing original inventory and operating ratings to remain in place until a permanent superstructure replacement can be scheduled. The most heavily repaired bridge remains in service after 23 months, its performance demonstrated by long-term monitoring data. As currently implemented, the MF-FRP repair is a viable temporary solution for maintaining traffic on a degraded structure while a replacement structure is designed, programmed, and implemented. Efforts to expand the MF-FRP repair into a longer-term solution are underway.

Keywords

load rating, prestressed concrete, repair, fiber reinforced polymer, prestressed mechanically fastened FRP

Prestressed concrete beams of various forms are among the bridge superstructure systems currently in service in North Carolina and across the United States. Some of these bridges are in states of deterioration that will soon require a complete superstructure replacement. However, superstructure replacement commonly takes a year or more to design, schedule, and construct. Temporary repairs are thus desirable to maintain the useful service life of degraded structures until a permanent replacement can be implemented. Selected examples of deteriorated prestressed concrete beams presented in this paper include: (i) C-channel beams of Bridge No. 380093

(Figure 1*a*) on Service Route No. 1156 across Owen Creek in Granville County, NC, the subject of a 2014 study by the North Carolina Department of Transportation (NCDOT); and (ii) cored slabs of Bridge

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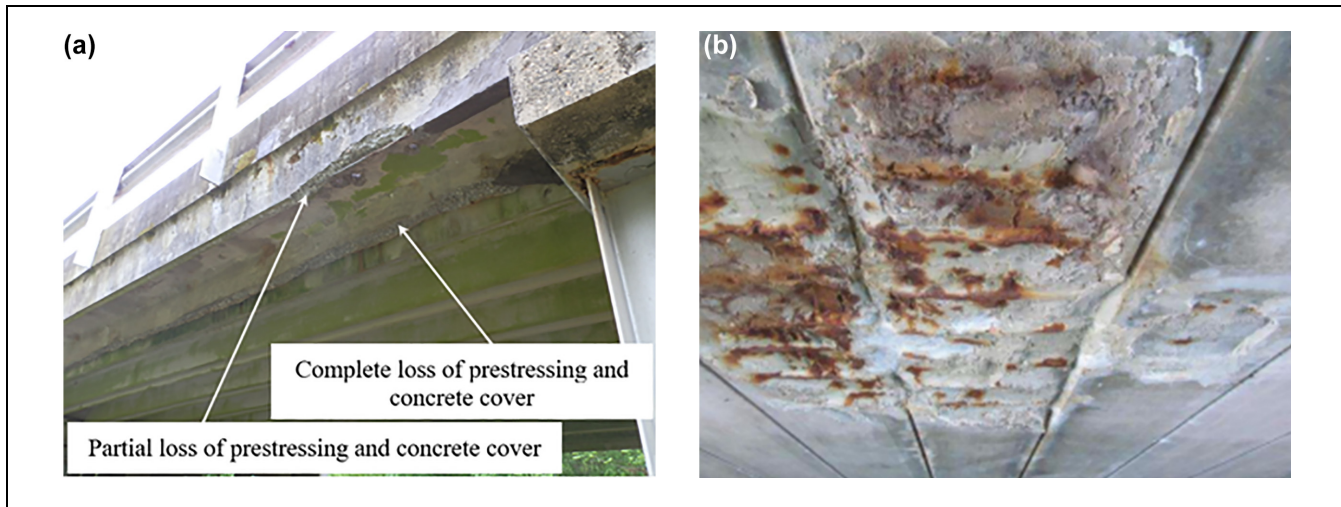


Figure 1. Examples of deterioration of prestressed concrete bridge beams that are repairable: (a) C-channel beam from Bridge No. 380093 and (b) cored slab from Bridge No. 150035 (1).

No. 150035 (Figure 1b) on US Highway 70 across Ward Creek in Carteret County, NC, the subject of a 2016 NCDOT study (1).

NCDOT currently maintains 269 bridges in inventory that have precast prestressed concrete C-channel beams tied together with grouted shear keys and transverse post-tensioning. The North Carolina Bridge Management System (2) shows that 226 of these 269 C-channel bridges are load-posted or closed because of their state of deterioration. The deterioration is often from corrosion of the prestressing strands as a result of natural aging or excessive concrete cracking from overloaded trucks, resulting in an average detour length of approximately 12 km. Bridge No. 380093 in Figure 1a is an example of a superstructure with deteriorated bottom faces of the edge beam. Even though this beam had a minor spall of concrete cover along the bottom edge, in extreme cases, strands can be exposed, corroded, or damaged, affecting the load capacity. Figure 2 shows an example of the bottom face of a beam with corroded and severed strands.

U.S. state Departments of Transportation (DOTs) face significant challenges in scheduling and budgeting bridge replacements or major repairs. Once a bridge is identified as deficient through routine inspection (3), a significant repair or replacement (that is not deemed an emergency) is commonly procured through the design-bid-build (DBB) project delivery method based on state and federal legislative contracting requirements. For the purposes of this discussion, a significant repair is defined as a repair that cannot be performed by DOT maintenance crews, and, therefore, must be performed through contracted resources. Contracted resources are procured through DBB project delivery, which typically awards

the project to the lowest qualified bid and, therefore, offers the lowest cost solution for the project. DBB is the most common project delivery method for government agencies because of the low up-front cost, as compared with other project delivery methods (4). While DBB project delivery provides a low-cost option, the awarding process within DBB is often time-intensive. It is not uncommon for the DBB process to last several years from the time of identifying a project need through project completion (4). In addition to DBB procurement time, capital improvement projects with budgetary requirements outside the annual maintenance budget often require several years for government appropriation of funds (5). Therefore, budgeting and project delivery requirements force state DOTs to impose load limits or even to close bridges while waiting for the approval and implementation of the corrective actions.

As with many of the deteriorating C-channel bridges in North Carolina, Bridge No. 810003 was already scheduled for replacement at the time when the bridge was closed because of deterioration in 2020. While the annual average daily traffic on this bridge is only 900 vehicles, closure of the bridge would have a significant impact on local traffic as the detour is over six miles long. A rapid, efficient, and easy-to-install temporary repair method, which can be budgeted within annual maintenance budget requirements, was needed to keep Bridge No. 810003, and similar bridges across the state, operational—without lowering the posted load limits or imposing closures—until replacement structures can be budgeted, scheduled, and implemented.

Current repair techniques using fiber-reinforced polymer (FRP) materials include externally-bonded (EB), mechanically-fastened (MF), and hybrid FRP systems.

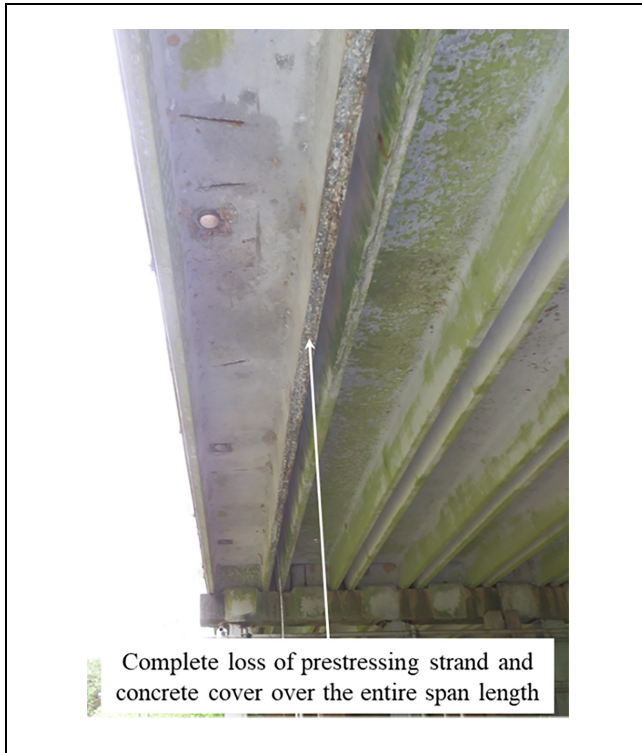


Figure 2. Example of deteriorated bottom face of a beam.

These available FRP systems increase the flexural capacity of deteriorated beams, but generally do not address the loss of prestressing forces caused by deterioration in prestressed concrete members. A comprehensive review of existing FRP repair systems used to strengthen or repair prestressed concrete beams and slabs can be found in McCoy (6). Increased flexural capacity of deteriorated beams is often necessary to maintain the bridge operating rating (7); however, inventory rating (7) requirements impose concrete stress limits that must be achieved through restoration of prestressing forces. Therefore, repairs to deteriorated prestressed concrete beams must restore flexural strength and prestressing lost to deterioration to maintain both operating and inventory rating levels. Other repair systems, such as unbonded external post-tensioned steel or carbon fiber prestressing strand systems, bonded concrete and reinforcement systems, and bonded or mechanically-fastened steel plates, are part of the current state of practice; however, they do not effectively and efficiently address all of the identified needs, including the need for rapid installation by DOT maintenance crews.

Research Significance

This paper presents an application and structural assessment of a deteriorated NCDOT bridge in which a

Table 1. Load Rating Comparison

Inventory rating	Operating rating
Routine traffic	Infrequent and heavier traffic loading than usual
No incremental damage	Small incremental damage
Assumed zero tensile stress limit in concrete	Maximum designed live load

retrofit concept has been developed for the rapid repair of prestressed C-channel beams. The concept presented uses mechanically-fastened FRP (MF-FRP) plates applied to the deteriorated beams using a post-tensioned prestressing method. The MF-FRP system can be designed to restore prestress losses on installation and maintain concrete tension within limits prescribed by AASHTO (7) to remove posted load limits. The following constraints were considered in the retrofit development:

1. The replacement of bridge superstructure through the DBB project delivery method and capital budgeting cycles may require a temporary repair to last between three and five years; long-term durability of the retrofitted structure is not likely a controlling consideration during this period,
2. Bridges shall be repaired with minimal disruption to traffic; therefore, solutions involving removal and replacement of deteriorated beams were not considered, and
3. A simple retrofit not requiring heavy or special equipment was desired.

AASHTO Rating Criteria

A retrofit solution that enables candidate bridges to remain in service without posted load limits or closures must address AASHTO (7) rating requirements. Load rating criteria are separated into two categories: inventory rating and operating rating (7). A brief comparison of the load rating categories is shown in Table 1. Load ratings based on inventory rating criteria address routine traffic, and specifically apply stress limits such that the bridge may be safely used at the inventory limit, across multiple lanes, for an indefinite period of time without the risk of incremental damage from traffic. The inventory rating loads typically correspond to serviceability stress limits, but reflect the existing bridge condition as determined through routine bridge inspections. Load limits imposed by operating rating criteria specifically address non-routine traffic using strength limit states to establish the maximum permissible live load.

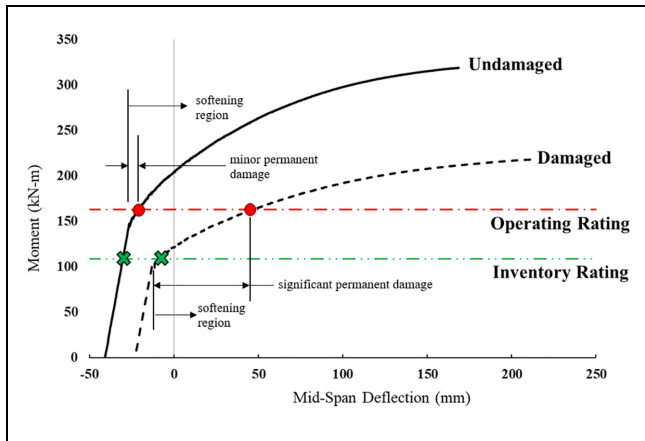


Figure 3. Moment–deflection response of undamaged and damaged prestressed concrete bridge beams with AASHTO limits.

The operating load limit typically allows a maximum live load that exceeds the inventory rating, and thus incremental damage may occur at the operating limit. This minor damage is acceptable, as the operating rating limit is for non-routine, infrequent traffic loading. Although the operating rating criteria are commonly used to post load limits on bridges, a retrofit solution which only increases the operating rating can create a situation in which the inventory load rating controls posted load limits, and therefore load restrictions, would remain in place. The opposite could be true for a retrofit solution that only addresses the inventory rating criteria. The limits can be better visualized through the typical flexural response of a prestressed concrete beam.

Figure 3 shows a typical moment–displacement response of an undamaged and a damaged prestressed concrete beam, with the AASHTO load limits defined above. The inventory limit intersects the response curve of the undamaged beam within the linear-elastic portion, meaning the beam does not suffer permanent damage at this load. At a higher moment, tension stresses will develop at the extreme tension face of the beam and cracking will occur. The operating limit of the undamaged beam intersects the curve at the softening stage, just past the elastic limit, such that the beam will accumulate minor permanent damage at this level of loading. On the other hand, when considering the effects of typical deterioration, the operating limit of the damaged beam occurs at a higher deflection level, leading to more permanent damage accumulation over time. In most cases, the operating rating controls bridge capacity and load restrictions; however, to adequately remove posted load restrictions from deteriorated prestressed concrete bridge superstructures, both operating and inventory rating criteria must be addressed.

Inventory and operating rating load limits for all bridges are calculated using the following general form of the AASHTO (7) rating equation:

$$RF = \frac{C - A_1 D}{A_2 L (1 + I)} \quad (1)$$

and

$$RT = (RF)W \quad (2)$$

where RF is the rating factor for the live load capacity of the bridge, C is the capacity of the member, D is the dead load effect, L is the live load effect, I is the impact factor, A_1 is the dead load factor (taken as 1.0 for allowable stress method analysis), A_2 is the live load factor (taken as 1.0 for allowable stress method analysis), RT is the bridge member rating tonnage, and W is the weight of the nominal vehicle used to determine L .

AASHTO (7) includes provisions for Allowable Stress Rating (ASR) and Load Factor Rating (LFR) methods to be used for bridge rating analysis. Currently, NCDOT requires all bridges to be designed in Load and Resistance Factor Design (LRFD) and rated for Load Resistance Factor Rating (LRFR). However, older bridges are rated based on the method used when they were designed (either ASR or LFR). For the purposes of this study and the development of the retrofit solution that addresses both inventory and rating criteria, ASR will be used to correspond with the ASD design methodology of the bridge. However, the use of LRFR or LFR would result in the same conclusions for the retrofit concept and the same impacts on inventory and operating rating analysis for bridges designed with LRFD or LFD design methodologies.

The ASR method defines C for the analyzed member as the allowable stress capacity prescribed by AASHTO (7) for the rating level desired. Inventory rating criteria determine C based on concrete or prestressing steel serviceability stress limits, or reduced strength for flexural and shear capacities; whereas operating rating criteria allow for higher capacities of nominal strength and prestressing steel stress limits. Specific forms of Equation 1 are used to determine the inventory rating factor for five limiting conditions: concrete tension, concrete compression, prestressing steel tension, and flexural and shear strength (7). Operating rating criteria use specific forms of Equation 1 to evaluate the rating factor for flexural and shear strength, and prestressing steel tension (7). Equation 1 provides an evaluation of the residual capacity of bridge elements after dead loads are applied for each of the evaluated limits. A rating factor ≥ 1.0 is necessary to avoid load posting for a given bridge since the

overall load rating of the bridge is controlled by the member with the lowest rating.

The allowable stress capacity, C , must remain constant regardless of current bridge condition, while the dead load effects, D , of the element increase as prestressing is lost. Thus, the rating factor for the bridge element is reduced, and the bridge load rating, RT , is reduced. Therefore, for the rating factor of a deteriorated bridge element to be equal to the original rating factor, either the dead load effect for the evaluated condition must be reduced, or the live load effect must be reduced. While the live load effect for deteriorated bridge elements with reduced capacity may be reduced to keep RF unchanged, a reduced live load effect also requires the nominal vehicle weight, W , to be reduced, and the overall member rating, RT , is reduced. Reducing live load effects, and thus imposing load limits which correspond to the appropriate nominal vehicle weight, is a common practice for DOTs to leave deteriorated bridges in service with restrictions.

State DOTs also have the option to repair deteriorated bridge elements to reduce dead load effects or increase capacity, and restore the RF to the pre-deteriorated value, keeping the bridge open without restrictions. In prestressed concrete bridge elements, the specific form of Equation 1 for the inventory rating criterion that places stress limits on concrete tension is of particular interest when considering candidate bridges that are experiencing loss of prestress effects because of corrosion. Under normal conditions, AASHTO (7) specifies the allowable concrete tensile limit as:

$$f_{t, \text{ allow}} = 0.5\sqrt{f'_c} \quad (3)$$

where f'_c is the concrete design compressive strength (MPa). However, AASHTO (7) provides a clause that allows bridge owners to limit $f_{t, \text{ allow}}$ to $0.25\sqrt{f'_c}$ (MPa) or $0\sqrt{f'_c}$ depending on varying conditions. NCDOT (8), in accordance with AASHTO (7), prescribes $f_{t, \text{ allow}}$ as 0 MPa for prestressed concrete elements in areas that exhibit increased corrosion, such as coastal regions where a large majority of the deteriorated candidate bridges are located. Therefore, a retrofit solution, which restores lost prestressing such that f_t is 0 MPa at service load levels considered in the inventory load rating, is necessary for a deteriorated prestressed concrete bridge to remain in service without restrictions.

Once the retrofit has been designed for the deteriorated prestressed concrete beams in a bridge, the increased capacity, C , of the retrofitted beams may be calculated for both the inventory and operating rating criteria. Equations 1 and 2 are then used to assess the new load rating of the retrofitted bridge. With other conditions and parameters remaining the same, increasing C of the deteriorated prestressed concrete beams after the

installation of the retrofit increases RF and RT . Consequently, a closed bridge may be brought back into service, or an existing load posting may be increased, until such time as the superstructure replacement is scheduled.

Background

The challenge to develop a retrofit technique that is easy to install, inspect, and maintain is complicated by the need for the technique to be applied to a range of prestressed concrete beam types with internal DOT resources and without capital budget appropriation and DBB project delivery time constraints. Critically, state DOTs must also be able to rate a bridge with the proposed retrofit installed using established AASHTO (7) inventory and operating rating methods before such a technique may be applied in the field.

FRP composites are non-metallic materials and therefore provide a retrofit system that is durable in highly corrosive environments. American Concrete Institute Subcommittee 440 (9) provides a design guide for EB FRP systems, making these systems more prevalent in practice than mechanically-fastened alternatives, for which there are currently no national design guides available. However, EB systems are critically dependent on the adhesive bond between the FRP and concrete substrate (9), which requires more concrete surface preparation during installation and makes routine inspection of bond quality difficult in service. Prestressed EB FRP systems are also available but in addition to concrete surface preparation, they require the use of hydraulic jacks and proprietary anchorage systems, and must be installed by certified personnel.

Therefore, the retrofit technique presented in this paper focuses on the use of mechanical fastening only (no adhesive), to provide a solution that is quick to install and traffic ready on installation. Additionally, the quality of the anchorage (in the form of mechanical fasteners) can be easily inspected at the time of installation and throughout the life of the system.

Non-prestressed MF-FRP systems are presented as an effective flexural strengthening technique to increase the AASHTO operational load rating for reinforced concrete elements throughout the literature. McCoy et al. (10) also developed a prestressed MF-FRP repair system (MF-FRP retrofit) to address the AASHTO inventory load rating. Through a series of material-level tension tests the bolted connection was optimized to maximize the efficiency of the MF-FRP plate. The effectiveness of the retrofit system was demonstrated in a series of full-scale laboratory tests on previously in-service C-channel beams. Bourara (11) studied the long-term behavior of the MF-FRP system under sustained load and fatigue.

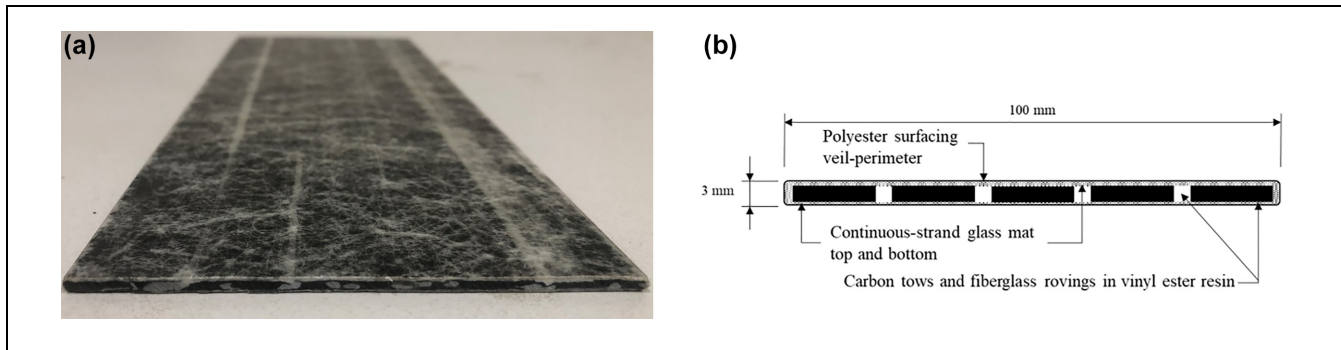


Figure 4. Fiber-reinforced polymer (FRP) plate used in the mechanically-fastened (MF)-FRP retrofit: (a) FRP plate sample and (b) FRP cross-section.

Source: Adapted from Strongwell (14).

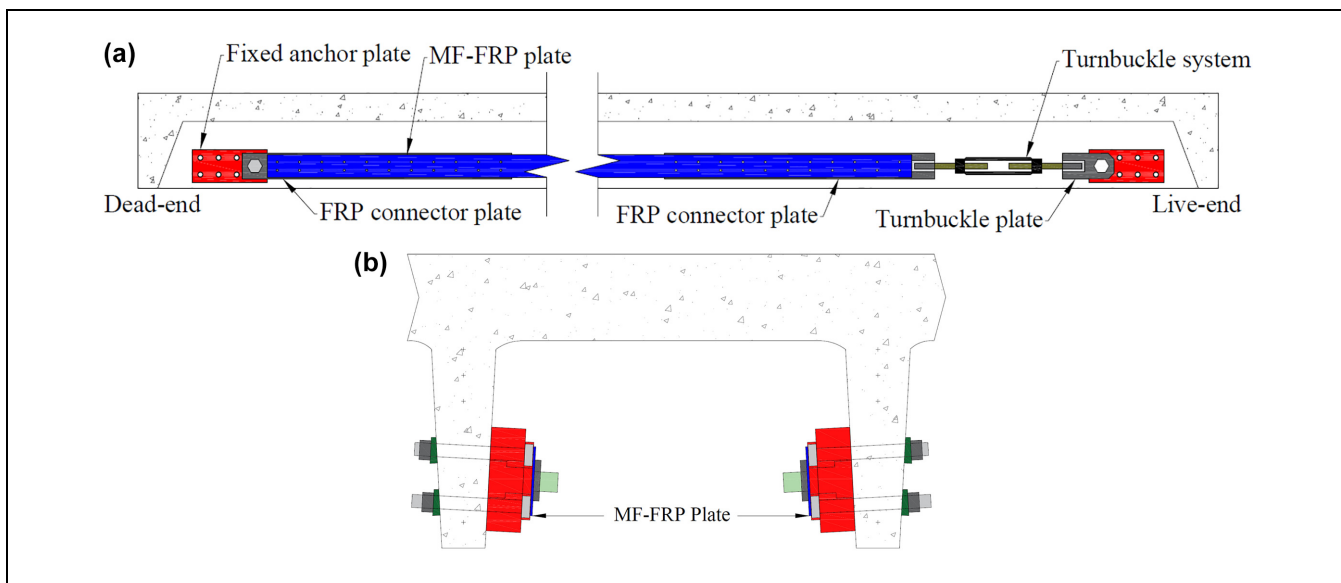


Figure 5. Schematic of mechanically-fastened fiber-reinforced polymer (MF-FRP) retrofit installed on a C-channel beam: (a) elevation view and (b) cross-section view (at the fixed anchor end).

The critical bolted connection was subject to a sustained load equal to the design prestress level for up to 30 months, and the maximum expected stress range for up to 1.25 million cycles. Bourara (11) and Lin (12) developed numerical analysis procedures for MF-FRP retrofitted cored slabs and C-channel beams, respectively, to predict the moment–deflection behavior to failure. Comprehensive information on the system throughout this research project can be found in the NCDOT report (13). The commercially available FRP plate used in the retrofit system is shown in Figure 4.

Figure 5 presents the MF-FRP retrofit developed by McCoy et al. (10) that was used to repair the deteriorated prestressed concrete C-channel beams presented in this paper. The FRP plate is bolted to a steel connector plate and anchored to the concrete section through a steel fixed anchor plate. A turnbuckle system on the live-end is used

to prestress the MF-FRP plate in the field. The repair system can be rapidly installed in the field with simple tools and common skills once the design plan is developed and the required materials are procured and made ready for installation.

Retrofit of In-Service C-Channel Bridges

During the NCDOT funded research program, the MF-FRP retrofit was installed on in-service deteriorated prestressed concrete C-channel bridges in North Carolina. The successful experiments presented in McCoy et al. (10) showed that the MF-FRP retrofit could restore the prestress losses and improve inventory and operating load limits for deteriorated prestressed concrete C-channel beams. To date, three in-service bridges have been retrofitted with the prestressed MF-FRP system. The

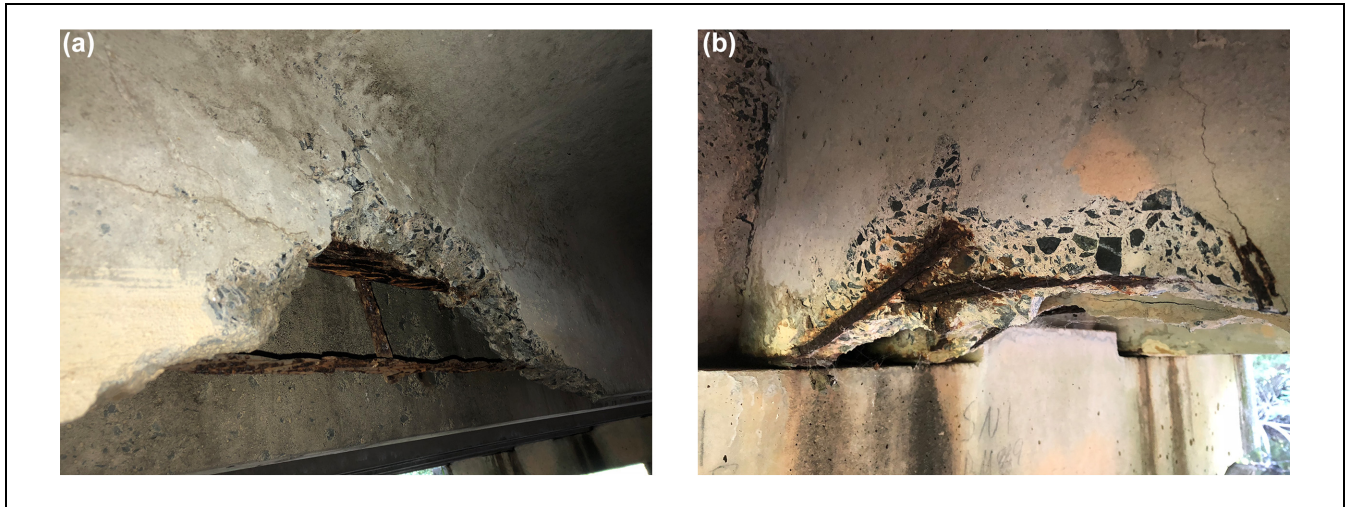


Figure 6. Observed deterioration of Bridge No. 810003: (a) mid-span deterioration and (b) end-region deterioration.

first repair served as a field demonstration on a deteriorated C-channel beam on Bridge No. 340080 in Franklin County, NC, in April 2019, with only one stem on one beam repaired. The retrofit was visually inspected in October 2020 after the system had been in service for more than 18 months, and no obvious deterioration related to the repair system was observed. This deteriorated C-channel bridge was replaced with a cored slab bridge in early 2022.

A modified MF-FRP retrofit was used to repair Bridge No. 810003 in Sampson County, NC, in November 2020, returning this bridge to service after closure following an inspection. The Sampson County bridge was the first known full bridge installation of prestressed MF-FRP, so a long-term monitoring plan was developed to document the in-service performance. It is planned to test the retrofitted beams once the superstructure is replaced in early 2023. Details of this retrofit are described in detail later in this paper. Most recently, the retrofit system was installed on Bridge No. 910180 in Wake County, NC, in July 2021 to increase the existing load posting from 5 tons for all vehicles to 19 and 25 tons for single vehicles and truck-tractor semitrailers, respectively, allowing school buses to cross the bridge.

Condition Assessment of Bridge No. 810003

Bridge No. 810003 is a three-span C-channel bridge on Service Route No. 1933 across Branch Six Run Creek in Sampson County, NC. The bridge was built in 1966, and was recently found to have severe deterioration from aging and environmental exposure, and likely from overweight vehicles. Severe corrosion of several prestressing strands was visible along with significant concrete spalling. The deteriorated prestressed concrete bridge was

closed by NCDOT in May 2020, following the biennial inspection when excessive deflection under traffic load was observed. The closure of the bridge resulted in a 9.5 km detour that affected local businesses and farming. The impact of the closure and the replacement schedule (early 2023) meant that immediate action to extend its service life was required to reopen the bridge.

A detailed field assessment was performed by NCDOT personnel and the research team to evaluate the current condition of the bridge so that an appropriate retrofit may be designed. Figure 6a shows deterioration observed on the bridge, including concrete spalling, corrosion, and exposure or loss of prestressing strands, which reduced flexural capacity and violated AASHTO load rating limits. In addition, the loss of concrete sections and exposed strands at the end regions, as shown in Figure 6b, raised concerns about shear capacity. Sound concrete is required to prevent concrete splitting at the prestressed MF-FRP anchorage. A Schmidt hammer was used to estimate the in-situ concrete strength as 48.3 and 58.6 MPa for stem and flange concrete, respectively. Note that the measured values obtained from the Schmidt hammer are limited to 58.6 MPa by the device itself. Nonetheless, the in-situ concrete strength is estimated to be sufficiently above the design strength of 34.5 MPa. Although most of the target deteriorated C-channel beams had only one stem exhibiting severe deterioration, the research team decided to retrofit both stems of all deteriorated C-channel beams. It was determined that six beams (12 stems) required flexural strengthening, and seven damaged end regions needed additional treatment to address concerns about shear capacity. The end-region deterioration observed on this bridge was not considered previously when the original prestressed MF-FRP system was developed. A version of

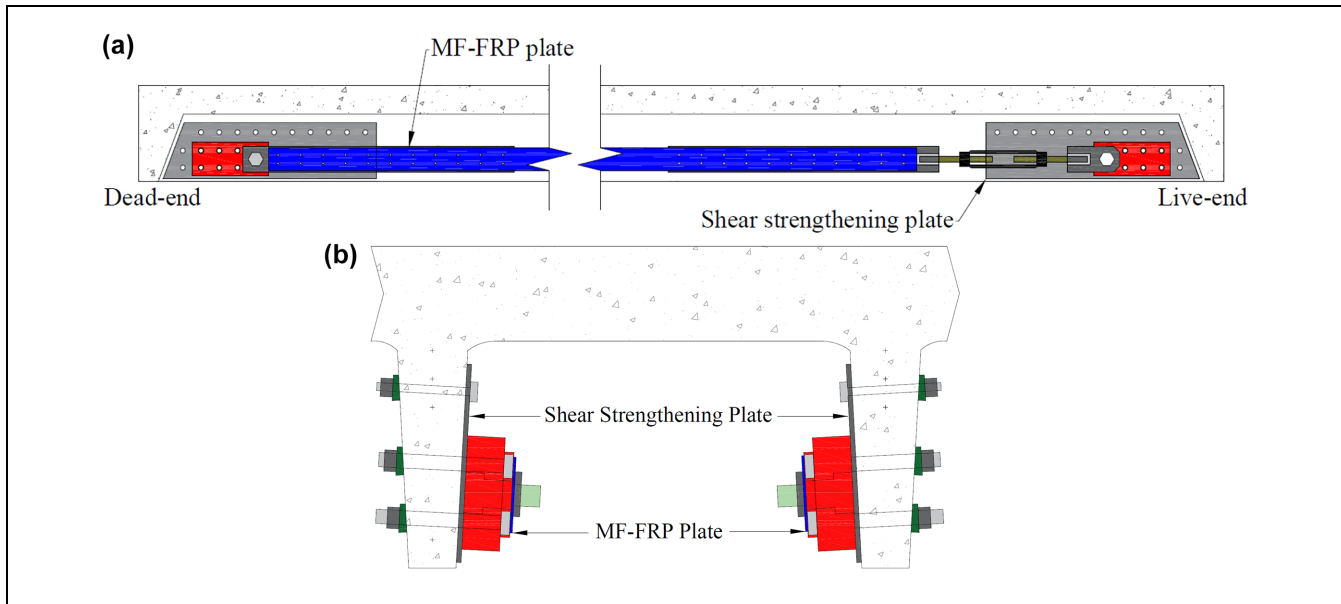


Figure 7. Schematic of the flexural and shear strengthening of mechanically-fastened fiber-reinforced polymer (MF-FRP) retrofit: (a) elevation view and (b) cross-section view.

the prestressed MF-FRP system developed by Lin (12) was used to address the flexural and shear retrofit needs of this bridge.

Retrofit Design and Installation Procedure

The purpose of the prestressed MF-FRP retrofit was to reopen the bridge until replacement of the bridge could be scheduled, likely in early 2023. In addition to flexural strengthening, deterioration observed at the end regions was addressed by an additional shear strengthening plate designed to integrate with the MF-FRP system. Figure 7 shows the retrofit system as used in this application. The shear strengthening plate was designed to be located between the fixed anchor plate and the concrete stem, aiming to distribute the applied shear force and to increase member shear capacity. The bottom two rows of the bolt hole grid on the shear strengthening plate follow the same pattern as the holes in the fixed anchor plate to provide flexibility in locating the anchor plate while installing the flexural repair system. The shear strengthening plate was anchored using the top and bottom two rows of bolt holes to engage the full depth of the plate.

After determining the extent of damage and completing the retrofit design, the installation procedure started with preparation of the FRP plate. Measurements were taken of each target beam to identify the best location for the fixed anchor plates. For the beam ends with only the flexural prestressed MF-FRP system, the fixed anchor plate was located as close to the end of the beam as possible so that the repair system would cover as much of the beam as possible. For the beams with shear deterioration,

the fixed anchor plate was located at least 300 mm away from the deteriorated end regions to avoid the potential for premature anchorage failure during prestressing. Damaged concrete sections at the end regions were patched using standard NCDOT repair methods one week before installation to provide sound concrete and a flat surface for anchoring the shear strengthening plate, and to minimize concrete splitting while prestressing the MF-FRP system. Although not strictly necessary, damaged concrete near the mid-span region could also be patched in an effort to reduce the rate of corrosion of the prestressing strands.

Internal steel prestressing strands and stirrups were located using a pachometer to avoid conflicts while drilling anchor holes. Strand patterns and stirrup details in NCDOT (15) standard drawings for these C-channel beams did not match the actual locations measured in-situ. The FRP plates were cut off-site to the necessary lengths using a wet saw. The 22-bolt pattern at both ends of each FRP plate was predrilled off-site to maintain the quality of workmanship and tolerances and installation time. Prefabricated wooden templates were used on site to align drill holes through the concrete stems for the fixed anchor plate and shear strengthening plate.

Installation activities on site started with clamping the wooden templates at previously determined locations. Holes for the dead-end and live-end anchor plates and the shear strengthening plate were drilled with a rotary hammer drill using a 22 mm diameter bit. All plates were anchored with 19 mm A325 steel bolts tightened to a maximum torque of 81 N-m to avoid failure of the wedge washers. The FRP was mechanically fastened to the FRP

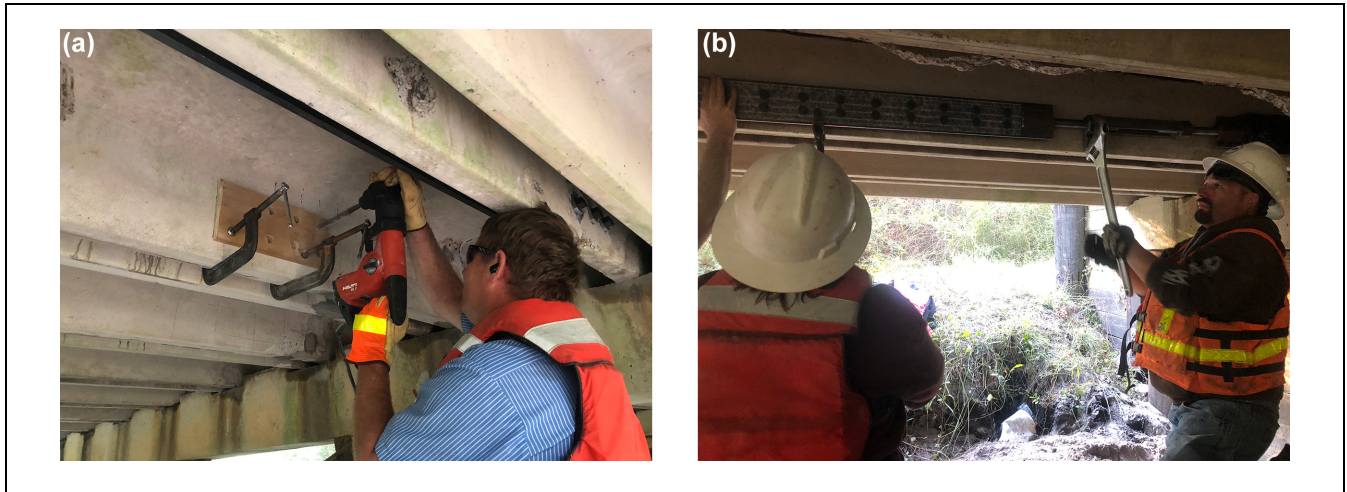


Figure 8. Key steps in retrofit installation conducted by NCDOT personnel: (a) drilling fixed anchor plate holes and (b) prestressing the mechanically-fastened fiber-reinforced polymer (MF-FRP) system.



Figure 9. Overview of Bridge No. 810003 retrofit: (a) view of live-end and (b) overall view.

connector plates at both ends with 12.7 mm A325 steel bolts with 81 N-m torque to be consistent with the previous full-scale laboratory testing. This level of torque was determined experimentally to prevent premature fatigue failure of the MF-FRP plate (11). The turnbuckle system (see Figure 5a) was attached to the FRP connector plate and to the turnbuckle plate. The FRP connector plate and turnbuckle plate were then attached to the dead-end and live-end anchor plates, respectively. Slack in the FRP was removed by slightly rotating the turnbuckle body until the system remained straight under its own weight. Before prestressing, witness marks were aligned on the FRP plate and C-channel stem to indirectly monitor the applied prestress force through elongation. As this was the first full retrofit installed on an in-service bridge, strain gauges (FLA-5-11-3LJCT) were also attached to

the FRP plate before prestressing to further verify the prestressing operation. For research purposes, the strain gauges also provide the opportunity for long-term monitoring of FRP strain. In this retrofit the MF-FRP was prestressed to the design force of 82.3 kN, equivalent to the current predicted effective prestress force in the steel prestressing strands, by rotating the turnbuckle body by hand with a wrench. A second wrench was used to prevent the FRP plate from rotating with the turnbuckle. The retrofit is complete once the FRP plate is prestressed to the design force level. Figure 8 shows the key steps of the installation procedure on site. An overview of the installed retrofit can be seen in Figure 9. The entire process required five days of fieldwork by a six-person NCDOT maintenance crew under the supervision of the research team. No specialized equipment was required.

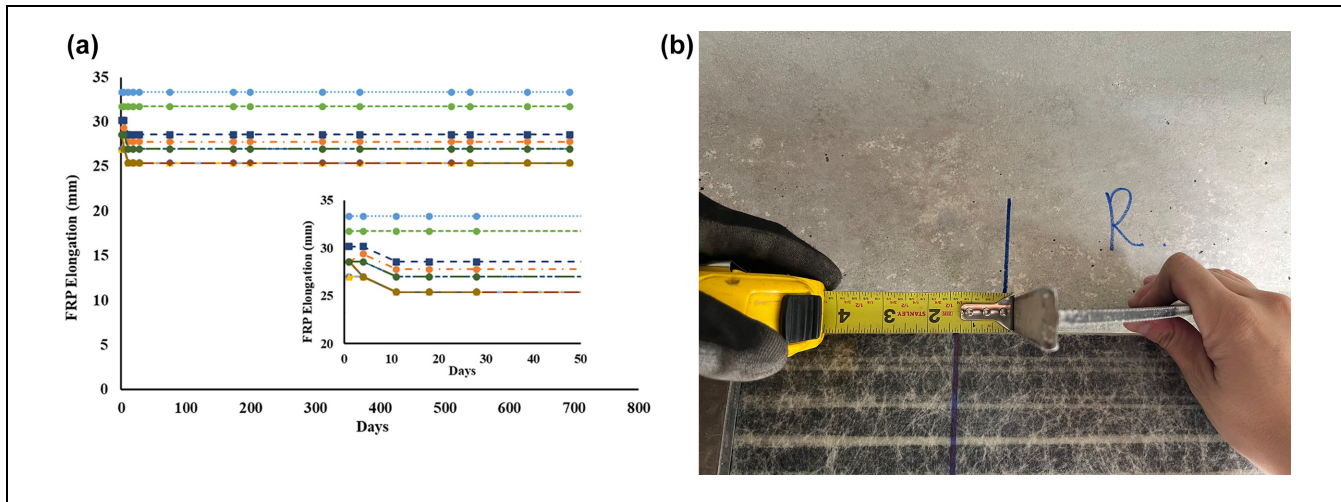


Figure 10. Long-term monitoring of fiber-reinforced polymer (FRP) elongation using witness marks: (a) field measurement and (b) field measurement of FRP elongation.

The installation was completed on November 4, 2020, and the bridge reopened to traffic the next day with the original load posting.

Long-Term Monitoring

Instrumentation was installed on the retrofit system to conduct short-term and long-term monitoring, including beam camber, FRP elongation, and FRP strain. Beam camber was measured before and after prestressing the MF-FRP plate using a stretched-wire system and a tape measure. The stretched-wire method was used to measure camber according to the outcomes from O'Neill and French (16). A 356 N fishing wire was selected for the method. It was attached to each end to screws anchored in the concrete near each end of the beam. Three C-channel beams adjacent to each other were selected, two repaired and one unrepaired, to allow for comparisons of camber. Any difference in restored camber will also provide information on the condition of the transverse prestressing.

Measured FRP elongation provides an easy and economical way to obtain and monitor the applied prestressing force using witness marks on the FRP plate and concrete stem. The required FRP elongation can be determined using the procedure described in McCoy et al. (10), considering the elastic elongation of the FRP plate, and the slip of the bolts in their holes. The initial measured FRP elongation in this retrofit application varied between 28 to 33 mm depending on the FRP gauge length.

Electrical resistance strain gauges were installed on selected FRP plates to measure FRP strain directly and to compare against the calculated FRP elongation

required to achieve the design prestress force. The required FRP strain is determined using Hooke's Law. With 82.3 kN of applied prestress force on the FRP plate, the target FRP strain was $4100 \mu\epsilon$. It showed the difference between the measured FRP elongation from the witness mark and the calculated FRP elongation based on the applied prestress force is within 5%, which is sufficiently accurate for the retrofit application to restore the prestress losses. Long-term FRP strain readings were always taken between 9:30 and 10:00 a.m. to minimize any daily temperature effects. FRP temperature measurements were taken using an infrared gun when FRP strain readings were being recorded.

Results and Discussion

The retrofit of deteriorated prestressed C-channel beams was successful, and Bridge No. 810003 was reopened to traffic on November 5, 2020, one day after the repair was completed. The short- and long-term measurements taken during and after installation are presented as follows.

Limited access to the mid-span of the instrumented beam over Branch Six Run Creek meant that the beam camber was only measured for six months after installation. The restored beam camber was observed to be 4.8 mm because of prestressing the MF-FRP system. However, there was no measurable camber restored for the adjacent unrepaired beam, which is an indication of lost transverse prestressing. This restored beam camber remained constant during the first six months of service.

Figure 10 presents the measured FRP elongation using the witness marks after installation of the retrofit system. All 12 MF-FRP plates installed were

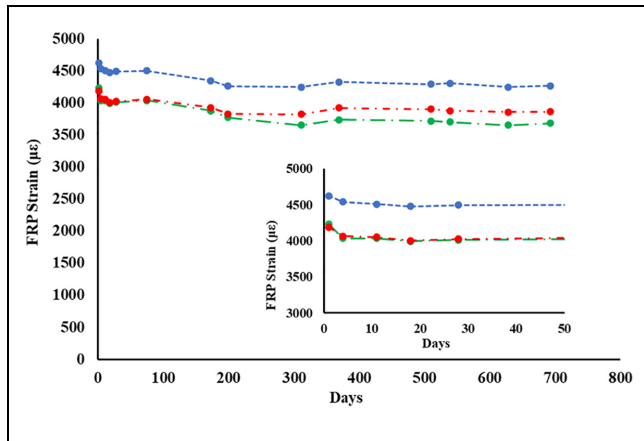


Figure 11. Long-term monitoring of fiber-reinforced polymer (FRP) strain using strain gauges.

instrumented and monitored. The results show that after an initial reduction in elongation during the first 10 days of service, no additional losses are measurable. The initial reduction in elongation is a result of the combined actions of the bolt bearing on the FRP plate under sustained loading, creep of the beam, and relaxation of the repair system. The insert included in Figure 10 more clearly illustrates the measured reduction in elongation during the first 50 days of service. The measured FRP elongation has remained stable after being in service for 693 days (approximately 23 months) at the time of collecting the last measurement.

The variation of FRP strain is shown in Figure 11, representing the average tensile strain readings on the installed MF-FRP plates. The average initial measured FRP strain was $4,280 \mu\epsilon$, close to the target strain of $4,100 \mu\epsilon$ needed to achieve the design prestress force. Strain measurements indicate similar behavior as the FRP elongation (refer to Figure 10), where the greatest variation of strains occurred during the first 10 days after installation. The insert included in Figure 11 more clearly illustrates the measured strain variation during the first 50 days of service. However, the more accurate strain gauge readings are able to capture seasonal temperature effects. To date, the measured FRP strains have varied, on average, between $85 \mu\epsilon$ and $280 \mu\epsilon$. This change in FRP strain results in less than 1 mm change of FRP elongation, which is less than the accuracy of the tape used to measure the long-term FRP elongation. The average initial short-term losses of FRP strain were approximately 3.9%, and long-term losses were 6.0% after 693 days of service, including the effects of concrete creep, bolt bearing on the FRP plates, relaxation of the strengthening system, and fatigue because of traffic loading.

The MF-FRP retrofit successfully repaired the deteriorated prestressed concrete C-channel bridge and enabled the bridge to be put back in service under the

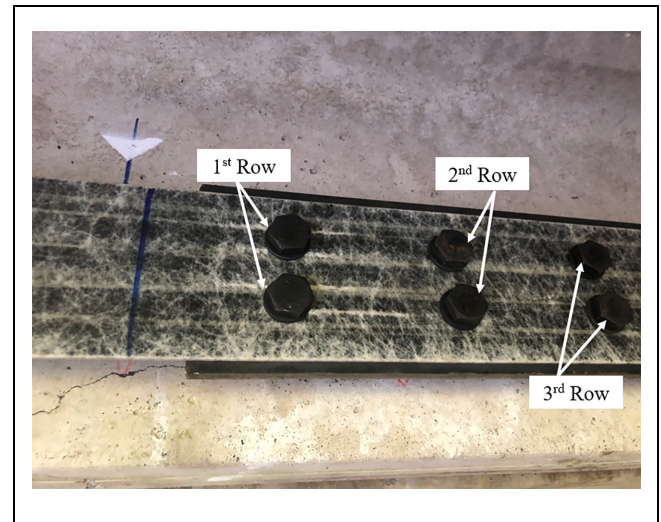


Figure 12. Minor longitudinal cracking of the mechanically-fastened fiber-reinforced polymer (MF-FRP) plate at the bolted connection.

current load posting condition. It remains in good condition after being in service for more than 23 months. Only minor longitudinal cracks were observed on some of the FRP plates, located at the first one or two rows of FRP bolts, as shown in Figure 12. The extent of these cracks is easily visible and consistent with experimental observations in the laboratory (11). These longitudinal cracks develop early in the service life of the system and result from slippage of the bolted connection engaging the MF-FRP plate as design trucks pass over the bridge. Longitudinal cracking in the FRP plate was observed after the bridge reopened but remained unchanged after the initial two weeks of service, not raising any concern about the condition of the system. Besides longitudinal cracking of the FRP plates, no signs of concrete splitting or deterioration of the repair system have been observed except for surface rusting of the steel elements. It is recommended that future repairs coat all steel elements per DOT standard specifications. Figure 13 shows the condition of the repair system at the time of installation and as of late September, 2022.

The MF-FRP retrofit is expected to have a service life in the order of three to five years, depending on the typical bridge replacement schedule set by DOTs. The condition of the MF-FRP retrofit should be assessed as part of the required biennial bridge inspections. The prestress force in the MF-FRP plate can be confirmed by measurement of the relative movement of the witness marks. Short-term limits on relative movement of the witness marks within two weeks of the installation are prescribed, which is an indication of excessive bolt bearing stresses on the FRP plate. Similarly, long-term limits on relative movement of the witness marks are prescribed, indicating

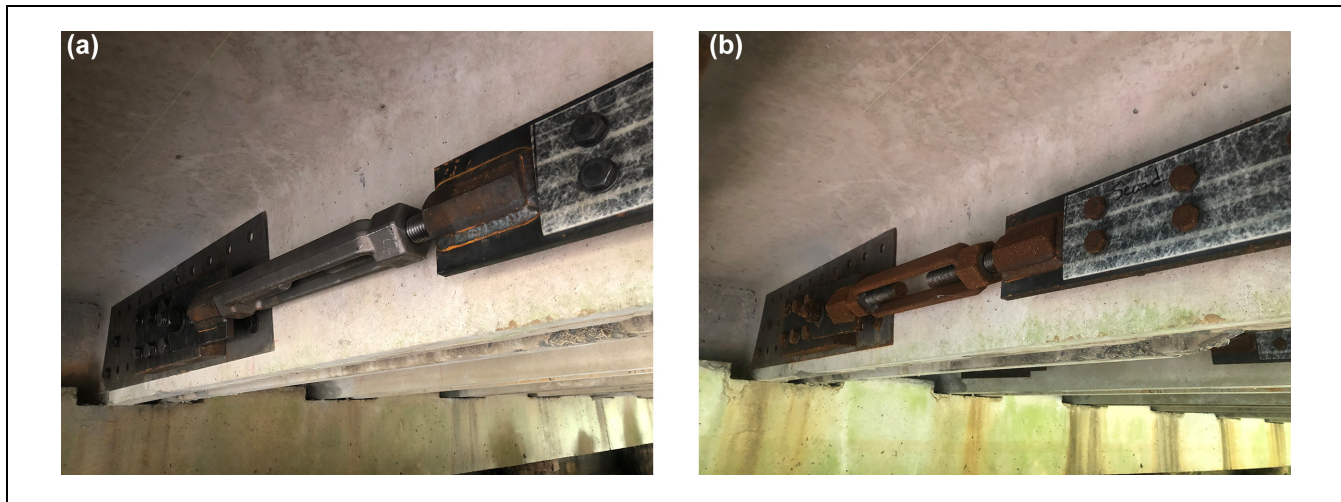


Figure 13. Condition of the retrofit system: (a) at time of installation and (b) current condition.

that the FRP plate is approaching the end of its fatigue life. If these limits are exceeded for an FRP plate, the plate should be replaced. The FRP plate and the concrete should be inspected for longitudinal cracking near the bolt hole locations. Some longitudinal cracking at the time of installation is expected, but these cracks should not widen or propagate after tension is applied to the FRP plate. All bolts must be inspected to confirm the 81 N-m torque remains. It should be noted that the system can be re-tensioned if prestress force in the MF-FRP plates has been lost, assuming all other components of the retrofit (and the beam) remain in good condition.

Cost Benefit

As mentioned previously, one alternative available to DOTs is to replace a deteriorated C-channel with another C-channel in good condition. However, DOTs are no longer using C-channel beams in new construction. Therefore, a limited stockpile of good quality C-channel beams is available to conduct such repairs, making traditional replacement less feasible. In addition, C-channel beam replacement requires a mobile crane to handle the beam on site. The cost of renting a mobile crane and hiring an operator for three or four days is roughly estimated as \$50,000, according to NCDOT personnel. The use of a crane adds significant cost to the repair budget, and requires road closure during the replacement process, resulting in a temporary inconvenience for the local community.

McCoy et al. (10) showed that the optimized field-level installation time was 4.1 labor-hours for the flexural prestressed MF-FRP retrofit system on a deteriorated C-channel beam (two stems). The MF-FRP retrofit can provide a rapid and economical solution with little labor,

simple tools, and common skills. All components of the MF-FRP retrofit and the required installation tools are readily available commercially. At the time of retrofitting Bridge No. 810003, the total material cost (steel elements, hardware, and FRP plate) for a single beam (two stems) was approximately \$2,000. The total material cost of retrofitting the six C-channel beams (approximately \$12,000) is thus significantly lower than the cost of traditional beam replacement. Further, the witness marks used during installation allow for long-term monitoring of the prestress force without the need for additional instrumentation. And, if needed, the turnbuckle can be used to adjust the prestress level in the future. Most elements of the repair, except for the FRP plate, can be recovered and reused on future repairs after permanent bridge replacement, which is valuable from a budget perspective.

Other Applications

A 2016 study conducted by the NCDOT examined Bridge No. 150035 (Figure 1b), which consists of nine 12.2-m spans, with 16 cored slabs per span. The cored slab cross-section of Bridge No. 150035 is shown in Figure 14. Inspections during site visits revealed that the greatest deterioration was at the end spans of the bridge. This deterioration caused the partial loss of the bottom layer of prestressing strands in multiple cored slabs within each end span. Figure 1b is a representative photograph of the patchwork attempting to slow the rate of deterioration; however, the patchwork cannot restore lost prestressing.

At the time of the site visits, posted load limits were active for Bridge No. 150035 because of reduced flexural capacity and excessive concrete tensile stress. The

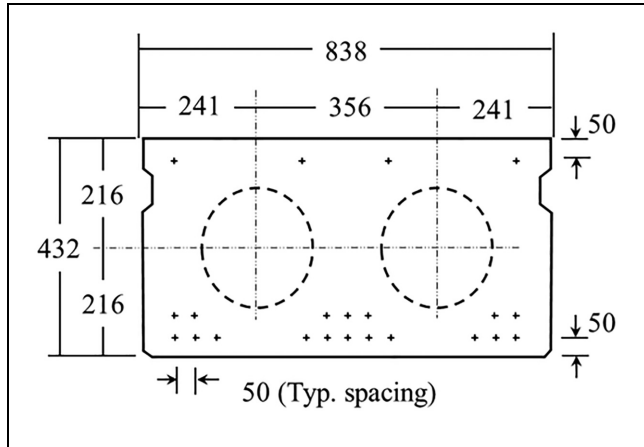


Figure 14. Bridge No. 150035 cored slab cross-section.
Source: Adapted from Van Brunt et al. (1).

prestressed MF-FRP retrofit presented for C-channel beams may be easily adapted to cored slabs experiencing prestress loss caused by deterioration in aggressive environments. The retrofit solution for cored slabs can be applied directly to the tension face of the section, as shown in Figure 15.

Bourara (11) conducted an experimental and analytical research program on deteriorated cored slabs repaired with the MF-FRP retrofit, showing the system capable of restoring both lost prestressing and the ultimate moment capacity of damaged cored slabs. As shown in Figure 15b, the typical strand layout in cored slab provides sufficient space to accommodate the bolts

required to attach the anchor plates at each end. Further, the 0.9m long end regions of typical cored slabs are zones of solid concrete, such that the anchor bolts do not interact with the hollow cores. As with C-channel beams, the extent of deterioration must be considered when determining the number of prestressed MF-FRP plates needed to retrofit the cored slabs such that the bridge may remain open without posted limits or closure.

Conclusions and Recommendations

Both inventory and operating rating criteria (7) must be addressed to remove posted load limits and closures on deteriorated prestressed concrete bridges. While operating rating criteria typically control the overall member rating, a retrofit solution that provides strengthening only, without restoring lost prestressing, would likely create a situation in which inventory rating criteria control would still apply. Therefore, a complete retrofit solution must also address the concrete stress limits within the inventory rating criteria to effectively remove posted load limits or closure, and allow the bridge to remain in service without restrictions.

A deteriorated prestressed concrete C-channel bridge located in Sampson County, NC, was repaired successfully using the flexural/shear prestressed MF-FRP retrofit system by a six-person NCDOT maintenance crew with no prior experience with the system and the North Carolina State University research team in five days of fieldwork. The retrofit was installed on six deteriorated C-channel beams (12 stems) and the bridge was reopened

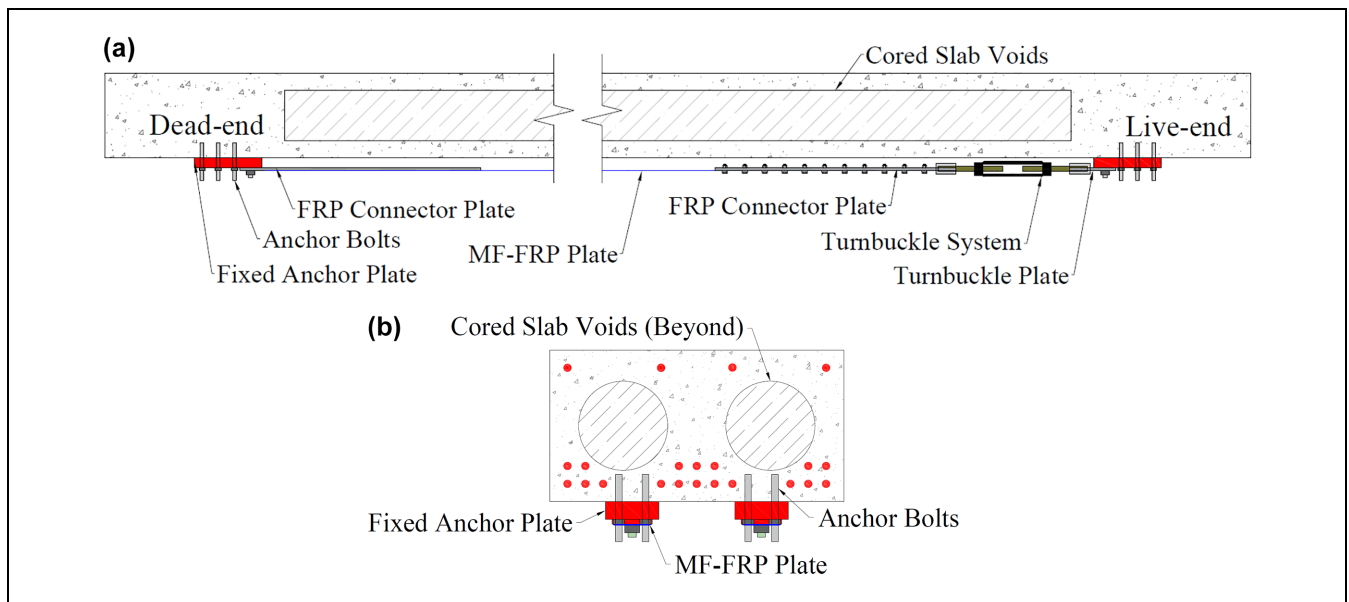


Figure 15. Schematic of the mechanically-fastened fiber-reinforced polymer (MF-FRP) retrofit applied to a cored slab: (a) elevation view and (b) dead-end cross-section.

to traffic the day after the repair was completed. Monitoring was conducted during and after the repair. The retrofit remained in good condition after being in service for more than 23 months. No damage was observed on the retrofit system except for minor longitudinal cracking of the MF-FRP plate at the first and second rows of bolts, and surface rust on the steel elements. Measured strains in the FRP plate became stable within the first two weeks of service, and measured FRP elongation and beam camber also stabilized within this time. Consistent with these field observations of this case study, it is concluded that the prestressed MF-FRP retrofit can serve as an ideal repair method to extend the service life of C-channel or cored slab prestressed concrete bridges by more than 23 months with significant economic benefits.

In addition, the following recommendations are made based on the case study:

- The prestressed MF-FRP retrofit is recommended for prestressed concrete C-channel beams (and prestressed concrete cored slabs) with the full loss of up to two prestressing strands (typically the bottom-most strand of each stem).
- The prestressed MF-FRP retrofit system is widely applicable on prestressed concrete C-channel beam and cored slab bridges, including cross-section geometry, material properties, span length, and steel prestressing strand layout.
- All steel elements of the retrofit should be coated to delay corrosion. Alternatively, stainless steel equivalent components may be used. All steel elements in good condition at the time of bridge replacement can be recovered and stored for future reuse. The development of more durable long-term prestressed MF-FRP solutions with composite anchorages is underway.

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Author Contributions

The authors confirm contribution to the paper as follows: study conception and design: S-H. Lin, B. McCoy, G. Lucier, N. Pierce, R. Seracino; data collection: S-H. Lin; analysis and

interpretation of results: S-H. Lin, B. McCoy; draft manuscript preparation: S-H. Lin, B. McCoy, G. Lucier, N. Pierce, R. Seracino. All authors reviewed the results and approved the final version of the manuscript.

Declaration of Conflicting Interests

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