FRP Reinforcement for Prestressed Concrete Applications

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UNIVERSITY OF MIAMI

FRP REINFORCEMENT FOR PRESTRESSED CONCRETE APPLICATIONS

By

Marco Rossini

A DISSERTATION

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of the University of Miami
in partial fulfillment of the requirements for
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FRP REINFORCEMENT FOR PRESTRESSED CONCRETE APPLICATIONS

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Corrosion of steel reinforcement is the primary cause of durability problems in aged Prestressed and Reinforced Concrete (PC and RC) structures. Fiber-Reinforced Polymer (FRP) reinforcement is a reliable non-metallic solution, able to ensure the required mechanical performance and ensure long-term durability.

This dissertation includes three self-contained but closely related studies that tackle three fundamental components of applied research: field deployment and critical assessment of existing technologies; development and investigation of innovative solutions; and, generation of new knowledge.

The first study addresses the opportunities and challenges related to Carbon FRP (CFRP) prestressing while developing the design, construction, and load testing of a short-span bridge entirely reinforced and prestressed with FRPs. The lack of design guidance was identified as a limiting factor for wider applicability of FRP prestressing. To address this knowledge gap, a unified framework was developed for the design of FRP reinforced and prestressed structures that was later formalized in the second edition of the AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete and is consistent with the first edition of the AASHTO Guide Specifications for the Design of Concrete Bridge Beams Prestressed with CFRP Systems.
The experience gathered highlighted some limitations of CFRP prestressing including the inherent complexity of the tensioning operations, the brittleness at pull, the tendency to cause concrete splitting, and the relevant material cost. Therefore, in the second study, mild pre-tensioning using GFRP reinforcement was proposed as a novel approach to the design and construction of those elements that require a relatively low level of prestress and are most exposed to environmental weathering and chloride penetration in coastal areas. To limit the level of prestress greatly eases tensioning operations and allows to use traditional steel anchors available at any precast yard. It also prevents failures at pull and concrete splitting. The use of a cost-efficient material system that is also less prone to prestress losses offsets the need for a larger number of strands. Experimental evidences to support this innovative approach are gathered for the first time on a prototype GFRP strand specifically developed through a federally-funded partnership with industries.

To be effectively used in prestressing, a material system must maintain its initial pull without delayed failures. Historically, the main limitation to GFRP prestressing laid in the relatively low creep-rupture strength reported in codes and standards because of the lack of experimental evidence and reliable predictive models in archival literature. To address this gap, the third study collects and analyzes a large number of creep-rupture and tensile test results to develop a rational predictive model based on statistical considerations. This novel approach allows for a reliable assessment of the long term properties of GFRP reinforcement and shows how previous limitations may be overly conservative and GFRP can be effectively used in prestressing applications.
Quaecaret ora cruore nostro?
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List of Published Works

The author contributed to enlarging the body of knowledge through the publication of journal papers, book chapters, contribution to conference proceedings, and technical reports. A portion of these works became integral part of this dissertation. Others were fundamental to assemble the critical background that helped the author in developing this work. A list of published works and articles undergoing publication is reported below.

*Journal Papers and Special Publications*


Book Chapters


Conference Proceedings


Technical Reports and Other Publications


Chapter 1

Introduction

One of the primary causes of premature failure in reinforced concrete (RC) and prestressed concrete (PC) is corrosion of the steel reinforcement. When steel corrodes, its effective cross-section reduces, and so does its load-carrying capacity. Furthermore, corrosion products have an expansive action that causes the adjacent concrete to crack and spall, promoting the penetration of aggressive chemicals. In addition, a chemical electrolytic cell often develops within the structure, causing a battery effect which accelerates corrosion of the steel reinforcement. A chloride-rich environment accelerates the corrosion rate. Such is the case of a bridge deck that is subjected to de-icing agents, or a marine structure exposed to saltwater. Other sensitive applications include RC or PC elements subjected to low-pH solutions such as containment vessels for waste-water treatment or chemical storage.

One traditional solution to the corrosion problem consists in protecting the reinforcement by reducing the porosity of the concrete. This can be achieved increasing the amount of cement or using concrete admixtures. Another traditional approach consists in increasing the concrete clear cover to retard aggressive agents from reaching the
reinforcement layer. The use of ferrous corrosion-resistant solutions has also been tried with various degree of effectiveness. Epoxy-coated and galvanized steel bars and strands have proved to be ineffective in highly corrosive marine environments leading some governing agencies in the US, including the Florida Department of Transportation, to ban their use. Stainless steel bars are readily available and feature good corrosion resistance. However, their prohibitive cost prevents large scale deployment.

Being a non-ferrous material, FRP is non-corrosive and impervious to chloride attack. Therefore, the use of FRP reinforcement extends the service life of RC and PC structures. Typical applications for FRP include highly corrosive chloride-contaminated or low-pH environments, or simply concrete elements that are required to be thin with inadequate cover for traditional steel reinforcement. Other applications take advantage of the electromagnetic transparency of FRP, should concrete members be adjacent to high voltages, magnetic fields, stray currents, radio or high-frequency signals. Typical applications include the reinforcement of magnetic resonance imaging units in hospitals, high-voltage substations, transformer pads, cable ducting, inductance loops in highways and tollbooths. In addition to being non-corrosive and electrically transparent, FRP reinforcement is also thermally non-conductive, and some applications take advantage of this property including thermal shunts for cast balcony units and connections in sandwich walls.

Unlike steel, FRP reinforcement is highly anisotropic, meaning it features good mechanical properties in the direction of the fibers and low transverse strength and stiffness. Some temporary applications take full advantage of this anisotropic behavior that allows for easier cutting and demolition. Examples include soft-eye for tunnel-boring machines (TBM) to penetrate and launch excavation, and sequential excavation tunneling.
where FRP reinforcement is used for temporary soil stabilization then consumed by tunneling and mining equipment. The same concept facilitates the use of FRP soil nails in passive removable applications.

FRP reinforcement has been used in concrete structures for more than 40 years. To date, over 270 bridges have been built using FRP reinforcement in the US and Canada according to statistics collected by the American Composites Manufacturers Association (ACMA 2016). To these, add several applications in China, Russia, and the Middle East, plus some experiences in Japan, Europe, Oceania, and South America. The Halls River Bridge (Figure 1) represents a successful example of a structure entirely built using corrosion-resistant solutions and mostly FRP reinforcement.

**FRP Reinforcement for New Construction**

**Material properties**

Fiber-reinforced polymer (FRP) reinforcement is made from continuous fibers, typically glass, basalt, carbon, or aramid. Fibers are impregnated with a polymeric thermosetting resin, typically vinyl ester, epoxy, polyester, or acrylic. The fibers provide tensile strength, and the resin acts as a binder. The resin serves an important role in transferring stresses from fiber to fiber as well as protecting the fibers from environmental exposure. Figure 2 shows FRP bars of different sizes and surface preparations as produced by various manufacturers.

Unlike steel that features an isotropic ductile behavior, FRP reinforcement is linear elastic up to failure. Furthermore, it shows an anisotropic and asymmetric behavior with good tensile strength and stiffness, poor transverse properties, and a tendency to fiber-
buckling that limits compressive properties. Furthermore, FRP reinforcement exhibits a size effect known as shear-lag: as the diameter of an FRP bar increase, loads cannot be efficiently transferred from the external layers to the core of the bar; thus, the tensile strength in terms of stresses of an FRP bar reduces at increasing diameter. FRP reinforcement is typically stronger, but less stiff with respect to steel. These are appealing features for PC applications where losses are directly proportional to the elastic modulus of the reinforcement, and the initial pull is limited by the elastic capacity of the material. Prestressing of FRP reinforcement is possible if adequate anchoring is provided at pull.

The mechanical properties of FRP reinforcement are mostly dictated by the choice of fiber. Glass FRP (GFRP) is the most common and the least expensive alternative; it is typically deployed in RC applications in the shape of solid bars. The ultimate tensile strength of GFRP reinforcement ranges from 500 to 1,500 MPa depending on the size, shape, constituents, and manufacturing. The elastic modulus ranges from 45 to 50 GPa. Glass fibers are manufactured by large multinational companies such as Owens Corning, Pittsburgh Plate and Glass, Vetrotex CertainTeed, St Gobain and others. Basalt FRP (BFRP) has recently been developed as an alternative to GFRP; it is undergoing standardization to ensure adequate quality control on basalt fibers and their composites. Carbon FRP (CFRP) is the most expensive and the best performing among commercially available FRP reinforcement solutions. It is typically deployed in PC applications in the shape of multi-wire strands. The ultimate tensile strength of CFRP reinforcement ranges from 2,000 to 3,000 MPa depending on the size, shape, constituents, and manufacturing. The elastic modulus is approximately 155 GPa. Aramid fibers are organic fibers largely used in ballistic industry, and Aramid FRP (AFRP) has seen deployment in PC applications
in the past. However, its poor durability performance and relevant material cost have limited its development.

Even if FRP reinforcement does not corrode, it may experience degradation phenomena when exposed to certain environments. In general, the degradation rates are order of magnitudes lower than what experienced by steel and the phenomena do not entail loss of effective cross section that may result in the complete dissipation of steel reinforcement. The degradation of FRP is predictable and is accounted for at the design stage by simply introducing an environmental knock-down factor to the material strength. No further actions are required to ensure the performance is met through the service life of the structure. Conversely, the degradation of steel reinforcement is difficult to predict, and typically is not accounted for at the design stage. Therefore, constant monitoring and repairing is required to ensure that steel-reinforced structures meet their design performance through their service life. The durability performance of GFRP bars after up to 15 years of service exposure in bridge decks and superstructures have been validated through an extensive monitoring campaign in the US and Canada (Gooranorimi & Nanni 2017, Benmokrane et al. 2018).

The physical and durability properties of FRP reinforcement are dictated by both the choice of fiber and resin. Different types of glass fibers feature different resistance to alkaline and corrosive environments. Electrical grade glass (E-Glass) was used in the past but has been replaced by a corrosion resistant alternative (ECR-Glass) to meet durability requirements beyond 100 years of service life. Considering other types of fibers, carbon has a very good chemical resistance; the performance of basalt fibers is being validated; conversely, the issues related to the durability of aramid fibers are well known and related
to their organic nature. Concerning the choice of resin, unsaturated polyester was commonly applied in the past, but its poor durability in moist environments is limiting its use. In the US, only vinyl ester and epoxy resins are allowed, given their superior performance. These resins are produced by multinational companies such as Ashland Chemical, Dow Chemical, Reichhold Chemical, DSM and others.

The role of an FRP reinforcement manufacturer is to combine fibers and resin into a pultruded composite rod. During pultrusion, fibers are impregnated with resin and drawn through a heated die from which they emerge as a semi-final product. Then, various surface preparation techniques can be employed to enhance the bond of the FRP bar to the concrete. These include sand-coating, surface deformations similar to those of traditional steel bars, and carving. In addition, a number of small-diameter composite rods can be twisted into a single strand for PC applications. There are a number of variables affecting the quality of the final product. These include temperature, pultrusion speed, resin formulation, and fiber sizing. Nevertheless, once defined and controlled, pultrusion is a stable and reliable process.

The emphasis in commercial FRP manufacturing is currently in the area of GFRP bars for RC. A number of GFRP bars manufacturers exists worldwide, whereas few CFRP strands manufacturers do exist and PC commercial applications represent a small portion of the market. Recently, some manufacturers are opening to the production of BFRP bars alongside GFRP (Ruiz et al. 2018b). Conversely, AFRP commercial solutions that underwent wide development through the 80s and 90s currently experience limited commercialization.
Material specifications and testing

The standardization of FRP reinforcement began in the early 90s and is eventually reaching maturity. In the US, standard specifications for GFRP bars are covered by ASTM D7957 (ASTM 2017b). Specifications for other FRP products are variously covered by documents published by ACI, ICC, and AASHTO (ACI 2008a, ICC 2017, AASHTO 2018a). These documents are being withdrawn or harmonized once relevant ASTM standards are published. In Canada, standard specifications for FRP bars are covered by CSA S807 (CSA 2010). In Europe, while a consensus standard is under preparation, test methods are standardized by ISO 10406 (ISO 2015). In Russia, standard specifications for FRP bars are covered by GOST 31938 (EASC 2012). In China, standard specifications for FRP bars are covered by GB/T 26743 (SAC 2011).

To ensure that commercial FRP products meet standard requirements, testing is performed as detailed by relevant specifications. Historically, testing of FRP reinforcement has been performed based on the test methods outlined in ACI 440.3R (ACI 2012). The document was developed to facilitate the preparation of design guidelines and for transition to ASTM standards. In the US, most individual test methods have already been published as ASTM standards, such as ASTM D7205 (ASTM 2006) that provides a standard test method to measure the tensile properties of FRP bars. In Europe, ISO 10406 (ISO 2015) is typically preferred to ASTM counterparts. In Canada, Russia, and China, national standards (CSA 2010, EASC 2012, SAC 2011) provide specific test methods in addition to material specifications. Harmonization is being promoted among different national standards where possible; thus, test methods and minimum requirements are typically similar among different standards.
Specifications must standardize FRP reinforcement without limiting the potential for innovation nor penalizing virtuous manufacturers. To this purpose, taking GFRP bars standardized by ASTM D7957 (ASTM 2017b) as an example, manufacturers are free to refine constituents, optimize the pultrusion process, and select the surface preparation of their choice as long as the bar is round, and it meets the minimum requirements in terms of mechanical performance and durability. Some variations on the shape and size of bars produced by different manufacturers do exist. To address the issue, nominal bar sizes are defined to be equivalent to traditional steel bar sizes. A range is defined for the measured area of a GFRP bar to fall in-between to meet ASTM D7957 (ASTM 2017b) specification. Mechanical properties for certification are measured on the nominal area that is also the reference for design.

Standardized tests allow for consistent determination of relevant engineering properties and give the designer, contractor, and owner confidence that the FRP reinforcement deployed will meet the properties that were assumed for design calculations. Standardization enables independent quality control and allows the pre-qualification of certified manufacturers that are therefore allowed to bid on public projects.

Design guidelines

Based on the experience gained from past projects, and a voluminous amount of academic research, several design guidelines have been published internationally. In the US, guidelines for the design of buildings reinforced with FRP bars are provided by ACI 440.1R (ACI 2015), whereas guidelines for the design of buildings prestressed with FRP reinforcement are provided by ACI 440.4R (ACI 2004). A document written in mandatory language is under preparation within ACI committee 440 covering building code
specifications for building reinforced with GFRP bars. For what concerns infrastructural applications, the design of passive GFRP reinforcement is regulated by the second edition of *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete* (AASHTO 2018b), whereas the design of active CFRP reinforcement is regulated by the first edition of *AASHTO Guide Specifications for the Design of Concrete Bridge Beams Prestressed with CFRP Systems* (AASHTO 2018a). The two documents have been recently developed by two separate task-forces of researchers, practitioners, and government officials (Rossini et al. 2018c, Belarbi 2017). In Canada, two separate documents are published: CSA S806 *Design and Construction of Building Components with Fibre-Reinforced Polymers* (CSA 2012); and, CSA S6 *Canadian Highway Bridge Design Code* (CSA 2014), which includes provisions for the use of FRP bars and strands. In Europe, guidelines for the design of FRP reinforcement are included in *fib Model Code 2010* (fib 2013) and discussion is undergoing for the inclusion of FRP reinforcement in *Eurocode 2* (CEN 2004). In Russia, guidelines for the design of FRP reinforcement are provided by SP 295.1325800 *Concrete Structures Reinforced with Fibre-Reinforced Polymer Bars: Design Rules* (Minstroy 2018) Similarly, in China, design guidelines are provided by GB 50608 *Technical Code for Infrastructure Application of FRP Composites* (SAC 2010).

These documents provide designers with authoritative guidance using a traditional regulatory format.

Design with FRP reinforcement follows the same principles established for RC and PC members reinforced or prestressed with conventional steel bars and strands. Strain compatibility and stress equilibrium hold valid under traditional Bernoulli assumptions. The main difference is in the constitutive model used to describe the reinforcement
behavior and failure criterion. The brittle nature of FRP reinforcement implies the possibility to either have over-reinforced flexural members that may fail because of concrete collapse in the compression zone, or under-reinforced flexural members that may fail because of reinforcement rupture in the tension zone. Whereas under-reinforcement is typically preferred when designing for traditional steel bars and strands because it ensures a ductile failure, over-reinforcement is sometimes preferred when designing for FRP. Irrespectively of the failure mode, thanks to a relatively low elastic modulus, an FRP reinforced member will typically experience large-enough deformations to foresee upcoming collapse before it occurs. Namely, GFRP bars feature ultimate strains well beyond the ductile threshold traditionally set at 5000 microstrain for steel reinforcement (Rossini et al. 2018c) as shown in Figure 3. This pseudo-ductile behavior is adequate for static applications. However, the deployment of FRP reinforcement in earthquake-resisting members, where high ductility is required, is not allowed.

Taking the AASHTO approach to GFRP-RC design (AASHTO 2018b) as an example, resistance factors vary between the two failure modes. Resistance factors are applied a-posteriori to the flexural strength of a member. A very conservative safety factor of 0.55 is applied to the flexural strength of under-reinforced members whereas over-reinforced members have a safety factor of 0.75 in line with traditional steel reinforcement. Further conservativeness is added by defining the guaranteed strength of FRP reinforcement at the 99.9th strength percentile (average minus three standard deviations), in spite of a characteristic strength at the 95th percentile (average minus 1.64 standard deviations) as done for traditional construction materials under the assumption of ideal normal distribution. The design strength of FRP reinforcement is computed by multiplying the
guaranteed strength by an already mentioned environmental knock-down factor ranging from 0.7 to 0.8 for GFRP, depending on the exposure conditions.

The design of GFRP-RC is rarely governed by strength considerations. Given the relatively low elastic modulus of GFRP, serviceability considerations typically govern. These include limiting crack width and deflection. Furthermore, the stresses induced in the reinforcement by sustained and cyclic loads must be limited within allowable thresholds to avoid the occurrence of creep ruptures, cyclic fatigue failures, or a combination of the two.

For what concerns shear design, the second edition of *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete* (AASHTO 2018b) adopts the Modified Compression Field Theory (MCFT) that has become the standard approach in the eighth edition of *AASHTO LRFD Bridge Design Specifications* (AASHTO 2017). The sectional shear strength is computed as a sum of the contribution of the concrete to the contribution of the FRP shear reinforcement, if provided. The relatively low elastic modulus of GFRP longitudinal bars results in a reduced dowel action, affecting the concrete contribution. Furthermore, where GFRP stirrups are present, their exploitability is reduced to a portion of their strength to limit the opening of transverse cracks and avoid failures at the bent location that features reduced strength because of fibers buckling during bending operations.

Furthermore, the second edition of *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete* (AASHTO 2018b) is the first guideline covering the design of torqued and compressed GFRP-RC elements, allowing the design of any structural component of a bridge, from foundations, to traffic barriers. The approach to torque is consistent with the eighth edition of *AASHTO LRFD Bridge Design Specifications*.
Specifications (AASHTO 2017) and includes adjustments similar to the ones discussed for shear design. The design of compressed elements must account for the reduced strength and stiffness of GFRP bars under compression as opposed to their good tensile properties. To this purpose, the contribution of GFRP in compression is neglected.

These additional safety layers are being refined, and reduction factors are being progressively calibrated toward a more efficient design as experimental evidences become available and experience is gathered from real applications. With the publication of the second edition of AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete (AASHTO 2018b), the amount of FRP bars required to reinforce a standard bridge deck has been reduced from a factor of 2.0 to a factor of 1.5 with respect to a traditional steel reinforcement design. This corresponds to an increment of 10% in the initial investment for an estimated 25% reduction in total costs over the service life of the structure, and no need for extraordinary maintenance or repair operations (Rossini et al. 2019b).

Further optimization can be achieved by adopting an empirical design method, also included in the second edition of AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete (AASHTO 208b). The empirical method allows for the design of GFRP-RC bridge decks mostly relying on the arching effect that develops inside the concrete and using a limited amount of reinforcement, the approach is similar to the one prescribed by the Canadian Highway Bridge Design Code (CSA 2014).

**Construction specifications**

Generally, installation of FRP bars features little differences with respect to traditional steel bars. Conversely, prestressing with FRP strands entails some added complexity and
requires the use of specific anchors to avoid damaging the strand during pull. In the US, construction specifications for buildings reinforced with FRP bars are provided in mandatory language by ACI 440.5 (ACI 2008b). For what concerns infrastructural applications, construction specifications are provided alongside AASHTO design specifications for GFRP-RC and CFRP-PC (AASHTO 2018b, AASHTO 2018a). In Canada, construction specifications for building reinforced or prestressed with FRP are provided by CSA S806 (CSA 2012). Other national standards outside of the US include some handling and construction considerations alongside material or design specifications (CSA 2010, ISO 2015, CSA 2014, Minstroy 2018, SAC 2010).

For what concerns construction operations, FRP reinforcement has two fundamental differences with respect to steel bars and stands. Being FRP a brittle material, it cannot be bent on site, but must be shaped during the manufacturing process, before the resin sets, to form hooks and stirrups. Furthermore, being FRP flammable, it cannot be cut using a torch, but a grinder should be used instead. Recently, the use of thermoplastic resins has been proposed as an alternative to traditional thermosetting solutions. Theoretically, a thermoplastic bar could be heated and bent to shape at any time, easing the manufacturing of complex shapes, and potentially transforming the supply chain. However, the quality control required for bending would require the operation to be carried out in a controlled facility environment, and not at the construction site.

Other minor differences do exist including the relatively low modulus of elasticity of FRP that must be accounted for during construction. Typically, FRP bars need to be supported with chairs at a spacing of two-thirds with respect to steel bars. Furthermore, FRP is lighter than concrete, and concerns have been expressed that it may float during
concrete pouring. Therefore, it is recommended that at least 50% of bar intersections are tied using plastic-coated wire or zip ties. As an additional precaution, cages may be tied down in some locations. These provisions are conservative, and no issues have been recorded during field construction. The potential abrasion of FRP reinforcement during construction operations should be considered. Small scratches can be visually assessed and coated. Wider damage will require the bar to be rejected. Experience has proved FRP reinforcement to be able to withstand typical construction loads and handling without damaging, and without the need for particular precautions. Finally, while storing FRP reinforcement, it should be considered its sensitivity to prolonged UV exposure that may negatively affect the mechanical performance of the resin. Typically, FRP reinforcement should not be exposed to sunlight for more than four months during construction operations.

Recent experiences and rigorous investigations are proving construction with FRP reinforcement as efficient as the use of traditional steel bars and in some cases more convenient (Cadenazzi et al. 2019). Thanks to its light weight at ¼ of steel, eased transportation, and reduced need for machinery, construction with FRP reinforcement may be faster and less impacting from both an economic and energy perspective.

Successful projects

Over the last 40 years, a large number of successful projects involving the use of FRP reinforcement have been developed worldwide making it a challenge to provide a comprehensive summary. Metrics collected by ACMA (2016) count over 270 bridges built using FRP reinforcement in the US and Canada. The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS) published some key
metrics on the performance of bridge structures after 5 to 8 years of service (Mufti et al. 2005). More recently, ACI is conducting a similar investigation on structures that has been in service for more than 15 years. Analyses are showing no appreciable degradation of the GFRP bars, in line with experimental predictions. Some relevant projects are discussed hereinafter.

The Floodway Bridge over the Red River near Winnipeg features one of the largest deployments of FRP bars in a single project. It consists of two bridge structures of eight spans each for a total length of 348 meters. GFRP bars were used in the barrier walls and in the deck slab that was designed according to the empirical approach. The project required over 150 tonnes of GFRP bar, equivalent to approximately 500 tonnes of steel, accounting for the difference in weight. The choice to use FRP reinforcement was based on purely economic considerations after a full life cycle analysis was performed. The Floodway bridge was among the first GFRP-RC bridge decks to be constructed outside of a research project. This is important to note, as there were no external factors influencing the decision beyond a cost-versus-benefit analysis.

Recently built on the Coral Gables campus of the University of Miami, the Innovation Bridge is one of the first structures to be built using solely FRP reinforcement (Spada et al. 2018). Different material systems have been deployed to exploit the specific features of each type of fibre. CFRP strands have been used to pre-tension the two 20-meter double-tee girders constituting the main structure of the bridge. GFRP and BFRP bars have been used to reinforce the deck, abutments, and auger-cast piles. The structure marks one of the first deployment of BFRP in a bridge project, and one of the first deployment of FRP in a bridge substructure. The reinforcement is deployed in variety of peculiar shapes, including
closed-loop filament-winded stirrups intended to ensure ideal bond to the concrete, and pre-assembled pile cages and deck meshes that significantly sped-up the construction activities on site and allowed for better quality control.

The Florida Department of Transportation (FDOT) is one of the most proactive agencies in promoting the deployment of FRP reinforcement in bridges and other transportation infrastructures. Recently built in Homosassa FL, The Halls River Bridge (Figure 1) is a 5-span vehicular bridge entirely built using corrosion-resistant solutions and mostly FRP reinforcement. The bridge serves as a conceptual prototype for future FRP deployments in the state of Florida. The structure includes CFRP-PC bearing piles, CFRP-PC/GFRP-RC sheet piles, hybrid HSCS-PC/GFRP-RC sheet piles, GFRP-RC pile bent caps and bulkhead caps, a GFRP-RC bridge deck, GFRP-RC traffic railings, GFRP-RC approach slabs, and a GFRP-RC gravity wall. The unprecedented variety and completeness of the material and structural solutions deployed make the Halls River Bridge an invaluable source for data. To this end, monitoring protocols have been implemented at the design and construction stages and are implemented through the service life of the structure. A life cycle cost analysis has been conducted, alongside a complete life cycle assessment analysis, proving a complete FRP-RC/PC design to be the least impacting solution from both an economic and environmental perspective (Cadenazzi et al. 2019).

Research Significance

A voluminous amount of research has addressed the fundamental questions related to the deployment of FRP reinforcement in structural applications. These include mechanical characterization, design algorithms, and construction procedures. Nevertheless, research is undergoing to facilitate transfer to current practice and develop more efficient and rational
design and construction approaches. Furthermore, new research areas are opening to allow the exploitation of some untapped technology potential.

Prestressing of CFRP and AFRP reinforcement has historically been a technological challenge. Carbon and aramid fibers feature high tensile strength but come at a high material cost. To justify their deployment from an economic perspective, extreme pulling is required to exploit as much tensile strength as possible. This demand for complex anchor systems to account for the limited shear strength of FRP and come at the risk of failures during pulling operations. To limit the initial pull simplifies pre-tensioning operations and allows to achieve steel-like constructability with traditional anchors. However, this solution requires the choice of a material system that is mechanically more efficient and economical enough. Thanks to its relatively low elastic modulus resulting in reduced prestress losses, and a relatively low material cost, GFRP strands (Figure 4) are being developed for PC applications within the MILDGLASS project funded by NCHRP-IDEA (Rossini & Nanni 2019). The project focuses on those PC members for which mild levels of prestress are enough, whereas corrosion of steel reinforcement is a concern. These include bridge substructures – piles and sheet piles – exposed to seawater, but also precast deck panels in cold-weather regions.

Creative Contribution

This dissertation includes three self-contained but closely related studies that tackle three fundamental components of applied research: field deployment and critical assessment of existing technologies; development and investigation of innovative solutions; and, generation of new knowledge.
The first study addresses the opportunities and challenges related to Carbon FRP (CFRP) prestressing while developing the design, construction, and load testing of a short-span bridge entirely reinforced and prestressed with FRPs. The lack of design guidance was identified as a limiting factor for wider applicability of FRP prestressing. To address this knowledge gap, a unified framework was developed for the design of FRP reinforced and prestressed structures that was later formalized in the second edition of the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete* and is consistent with the first edition of the *AASHTO Guide Specifications for the Design of Concrete Bridge Beams Prestressed with CFRP Systems*.

The experience gathered highlighted some limitations of CFRP prestressing including the inherent complexity of the tensioning operations, the brittleness at pull, the tendency to cause concrete splitting, and the relevant material cost. Therefore, in the second study, mild pre-tensioning using GFRP reinforcement was proposed as a novel approach to the design and construction of those elements that require a relatively low level of prestress and are most exposed to environmental weathering and chloride penetration in coastal areas. To limit the level of prestress greatly eases tensioning operations and allows to use traditional steel anchors available at any precast yard. It also prevents failures at pull and concrete splitting. The use of a cost-efficient material system that is also less prone to prestress losses offsets the need for a larger number of strands. Experimental evidences to support this innovative approach are gathered for the first time on a prototype GFRP strand specifically developed through a federally-funded partnership with industries.

To be effectively used in prestressing, a material system must maintain its initial pull without delayed failures. Historically, the main limitation to GFRP prestressing laid in the
relatively low creep-rupture strength reported in codes and standards because of the lack of experimental evidence and reliable predictive models in archival literature. To address this gap, the third study collects and analyzes a large number of creep-rupture and tensile test results to develop a rational predictive model based on statistical considerations. This novel approach allows for a reliable assessment of the long term properties of GFRP reinforcement and shows how previous limitations may be overly conservative and GFRP can be effectively used in prestressing applications.
Figure 1 – Halls River Bridge under construction (a), and existing bridge before demolition (b). [Courtesy of Astaldi Construction Corporation] (Rossini et al. 2018b)
Figure 2 – GFRP bars from various manufacturers.
Figure 3 - Stress-strain diagrams for steel and GFRP bars. (Rossini et al. 2018c)

Figure 4 - GFRP strand prototype compared to a commercially-available CFRP strand. (Rossini & Nanni 2019)
Chapter 2

Prestressing with CFRP

This study deals with the philosophy behind the design of the CFRP-PC double-tee girders and the BFRP-RC/GFRP-RC auxiliary members constituting the structure of a recently built pedestrian bridge, named the “Innovation Bridge”. Located on the University of Miami (UM) campus in Coral Gables (FL), the bridge is a 20-meter-long single span PC bridge. Although the bridge serves as a pedestrian passageway, it comprises a number of peculiar features intended to ensure a minimum 75-year service life with minor maintenance cost. The structure is entirely steel-free. It includes a variety of non-corrosive reinforcement solutions and is designed for resilience in aggressive subtropical exposure. The structure contributes to a comprehensive effort to leverage non-corrosive technologies in infrastructural applications. It serves as demonstrator for the deployment of innovative reinforcement solutions and shapes similar to the ones discussed by Spadea et al. (2017a, 207b). Furthermore, it serves as field-proof for the development and validation of the next generation of design and construction guidelines for FRP-RC/PC. Details on the construction of the Innovation Bridge are discussed by Nanni et al. (2016), whereas the flexural testing of one of the double-tee girders is discussed by Spadea et al. (2018).
This study is an attempt to unify the design approaches to FRP-RC and FRP-PC members in a consistent fashion. A critical state-of-the-practice review of FRP-RC/PC design is provided, and guidance is given through the design phases. The experimental validation of the proposed design approach serves as a first assessment of the unified rationale adopted. The results support a slight relaxation of the design limits for CFRP-PC and GFRP-RC. Furthermore, they support the application to BFRP-RC design of the same parameters defined for GFRP-RC.

The Innovation Bridge

The Innovation Bridge (Figure 5a) serves as a pedestrian passageway and guarantees access to the UM intermural sport facilities. The bridge has a total length of 21 m, a free span of 20 m, and a width of 4.3 m. It is designed for a live load of 4.8 kN/m² in addition to its self-weight. The structure includes two CFRP-PC/BFRP-RC precast thin-walled double-tee girders (Figure 5b), a cast-in-place BFRP-RC deck, two cast-in-place BFRP-RC curbs, two cast-in-place BFRP/GFRP-RC bent caps, eight BFRP-RC auger cast piles, BFRP-RC abutments, and GFRP-RC construction joints. The bridge encloses innovative material solutions and peculiar reinforcement shapes. CFRP strands are used as prestressing reinforcement in each double tee. BFRP reinforcement is included as bidirectional preassembled meshes in the stem and deck of each double tee and in the finishing pour. BFRP closed stirrups serve as shear reinforcement and shear connectors in the curbs, bent caps, and abutments. BFRP preassembled cages are used as reinforcement for the auger cast piles. GFRP bent bars are used as reinforcement for the construction joints between deck and abutments. Reinforcement details for the double tee are shown in Figure 6.
Design Approach

Relevant codes and guidelines

GFRP-RC infrastructural applications are spreading. Conversely, CFRP-PC structures are still considered state of the art prototypes. Furthermore, BFRP-RC infrastructural applications are not regulated at the national level nor BFRP bars are typically covered by existing design guidelines. A number of documents including provisions for the design of FRP-RC and FRP-PC members exist internationally. These documents may hold binding status as building code requirement or exist as design and construction guidelines.

The reports published by ACI Committee 440 hold relevance in the United States and abroad, but internal discrepancies exist in the way FRP-RC and FRP-PC are treated. These can be tracked to the fact that ACI 440.4R, which covers FRP-PC design, was not updated since the first generation of design regulations (ACI 2004). Conversely, ACI 440.1R which covers FRP-RC design has been regularly updated through the last decade (ACI 2015). Furthermore, ACI 440.4R origins can be tracked back to the JSCE recommendation for the design of FRP-PC structures (JSCE 1997), whereas ACI 440.1R featured an independent development since the early 2000s (ACI 2001). Discrepancies include the definition of the mechanical properties of FRP reinforcement, the values assigned to design factors, and the definition of design factors themselves.

In this section, the general framework defined by ACI 440.1R (ACI 2015) is integrated with relevant guidelines to include prestress (ACI 2004), and relevant material specifications (ICC 2017, ASTM 2017b, FDOT 2019b). The approach aligns to the next generation of design guidelines recently published by AASHTO, including the second edition of the AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced
Concrete (AASHTO 2018b) and the first edition of the Guide Specifications for the AASHTO Guide Specifications for the Design of Concrete Bridge Beams Prestressed with CFRP Systems (AASHTO 2018a). The documents were under development at the time of design by two separate taskforces of researchers, practitioners, and government officials (Rossini et al. 2018c, Belarbi 2017).

**Design factors**

FRPs are brittle composite materials, elastic until failure, stronger, but less stiff with respect to MCS and HSCS. The guaranteed strength ($f_{gu}^*$) of an FRP bar is defined in ACI 440.1R as the experimental average value minus three times the value of standard deviation (ACI 2015), which corresponds to the 99.9th strength percentile under the assumption of normal distribution. The same approach can be applied to FRP strands. However, the concept of guaranteed strength and an explicit definition of the design strength of an FRP strand are not present in ACI 440.4R (ACI 2004). The approach is more conservative with respect to the calculation of characteristic strengths for steel reinforcement and for concrete that is defined as the average value minus 1.64 standard deviations – 95th strength percentile – under the assumption of ideal normal distribution (CEN 2004). The definition is reported in Eq. 1 for clarity.

$$f_{gu}^* = f_{jm} - 3\sigma_f$$  \hspace{1cm} (1)

The minimum values for the guaranteed tensile strength of GFRP bars are defined by ASTM D7957 (ASTM 2017b). CFRP strands are not regulated at the national level but are covered by the FDOT Standard Specifications (FDOT 2019b). The deployment of BFRP
bars in infrastructural applications is not regulated at the National nor State level. Following the principles of AC 434 (ICC 2017), the same minimum requirements for GFRP bars is also applied to BFRP bars. In line with the principles of AASHTO (AASHTO 2018b, 2009), the minimum guaranteed tensile strength per applicable standard specification is taken as the specified strength (\(f_{fu}^\prime\)) for design purposes, unless the manufacturer can provide evidence to support higher values. The specified strength must always be lower than or equal to the guaranteed experimental strength for the specific reinforcement batch deployed, as specified in Eq. 2.

\[
\bar{f}_{fu} \leq f_{fu}^* 
\]  

(2)

FRP composites are known to experience strength degradation following long-term exposure to moisture and environmental effects (ACI 2015). To account for the phenomenon, the design strength (\(f_{fd}\)) of the material is defined per Eq. 3 including an environmental reduction factor (\(C_E\)). The approach is in line with the principles of ACI 440.1R (ACI 2015). An explicit definition of the design strength is not present in ACI 440.4R (ACI 2004).

\[
f_{fd} = C_E \bar{f}_{fu} 
\]  

(3)

The design strength of the material is the reference value for calculations both at the strength and service limit state. Furthermore, the strength of FRP under sustained load is reduced to avoid creep rupture (ACI 2015). Resorting to the nomenclature suggested by Rossini et al. (2018a), a creep rupture reduction factor (\(C_c\)) is applied to the design strength in order to define the design strength against creep rupture under sustained load (\(f_{f,s}\)) as in
Eq. 4. Similarly, the initial level of tension in prestressed reinforcement is limited to avoid creep rupture over the long period (ACI 2004). A jacking reduction factor ($C_j$) can be defined in order to compute the jacking strength of the material ($f_{f,j}$) as in Eq. 5.

$$f_{f,s} = C_s f_{f,d} = C_s C_c f_{f,u}$$  \hspace{1cm} (4)

$$f_{f,j} = C_j f_{f,d} = C_j C_c f_{f,u}$$  \hspace{1cm} (5)

The application of creep and jacking reduction factor in addition to the environmental reduction factor is in line with ACI 440.1R (ACI 2015). It is the responsibility of the strand manufacturer to provide a jacking apparatus able to allow initial pulling of the material up to its jacking strength. Otherwise, the jacking strength must be reduced accordingly. Figure 7 shows the tensioning apparatus and anchoring devices used to apply the initial prestress to the CFRP strands in the Innovation Bridge. The proposed approach rationalizes the requirements of JSCE (1997) that limits the design strength of the material ($f_{f,d}$) to allowable jacking values ($f_{f,j}$). The approach is in line with ACI 440.4R (ACI 2004) that separately defines maximum values of tension in the strands before release ($f_{f,i}$), and after release ($f_{f,c}$).

The brittle nature of FRP reinforcement and the absence of a plastic plafond result in the possibility to either have over-reinforced flexural members that may fail because of concrete collapse in the compression zone, or under-reinforced flexural members that may fail because of reinforcement rupture in the tension zone (ACI 2004, 2015). The two failure modes are characterized by two different strength reduction factors – $\phi_c$ and $\phi_t$, respectively – defined to guarantee the same level of safety in the two cases. A flexural member can also undergo shear failure. In this case the strength reduction factor $\phi_s$ is aligned to values prescribed for MCS-RC and HSCS-PC in ACI 318 (ACI 2014).
The different bond characteristics of FRP bars and strands with respect to steel reinforcement is accounted for introducing a bond reduction factor \( (C_b) \). For consistency with the other reduction factors, the bond reduction factor increases at increasing performances. The parameter is defined in Eq. 6 as the inverse of the bond reduction coefficient \( (k_b) \) as defined in ACI 440.1R (ACI 2015).

\[
C_b = \frac{1}{k_b}
\]  

\[ (6) \]

Table 1 through Table 3 provide a summary of design factors as reported by various international guidelines, along with the values selected for this study. Design factors for CFRP strands are in line with ACI 440.4R (ACI 2004) with the exception that higher values were selected for both the jacking \( (C_j = 0.75) \) and creep rupture \( (C_c = 0.70) \) reduction factors. These are higher than the values recommended by AASHTO (2018a), close to the values recommended by JSCE (1997) and set the upper bound of the current practice (Table 4). However, an environmental factor \( (C_E) \) equal to 0.9 is selected, that is more conservative with respect to the value of 1.0 recommended by AASHTO (2018a). The selection of higher-than-standard jacking and creep rupture factors for CFRP is supported by experimental testing conducted by the manufacturer (Tokyo Rope 2014) and independently by the Structures and Material Laboratory (SML) at UM (SML 2017). Furthermore, the design factor prescribed by JSCE (1997) were first calibrated on the specific material system deployed in this study. Design factors for GFRP bars are in line with AASHTO (2018b). Design factor for BFRP bars have been taken equal to the ones for GFRP. The approach is in line with national (ICC 2017) and international (Minstroy 2018) guidelines that already cover BFRP. The choice to align design factors for BFRP to
design factors for GFRP, or even use higher reduction factors, is supported by experimental testing conducted at UM (SML 2016) and a more comprehensive research effort supported by FDOT (Kampmann et al. 2019).

**Material properties**

*Reinforcement properties*

Structural reinforcement for the Innovation Bridge includes 15.2 mm CFRP strands with an epoxy resin matrix. M10, M13, and M25 BFRP bars with an epoxy resin matrix, and M16 GFRP bars with ECR glass fibers and a vinyl ester resin matrix. Stainless steel is used in some details including the bearing plates, the lifting hooks for the double-tees, the anchor bolts for the lampposts, and the railings mounted on the side curbs. The mechanical behavior of 15.2 mm CFRP strands, M13 GFRP Bars, and M13 BFRP Bars is shown in Figure 8 and compared to 15.2 mm HSCS strands and M13 MCS bars. The mechanical properties of FRP reinforcement are reported in Table 5 through Table 8.

The mechanical properties of the BFRP bars are as reported by Spadea et al. (2018). The mechanical properties of the GFRP bars are as reported by Ruiz et al. (2018a). The specified properties for both GFRP and BFRP bars are set equal to the minimum requirements reported by ASTM D7957 (ASTM 2017b). The mechanical properties of the CFRP strands are measured by the manufacturer on a total of 5 samples from the production lot used for the Innovation Bridge. Specified strength, area, and diameter are as reported by FDOT Standard Specifications (FDOT 2019b). In absence of applicable minimum requirements, the specified modulus of the CFRP strands is set equal to the experimental value reported by the manufacturer for the specific production lot. The approach is in line with standard practice for HSCS-PC (ACI 2014).
Concrete properties

The experimental and design properties of the three mixes used for the girders, deck, curbs and abutments are reported in Table 9. Experimental properties of the concrete used in the girders are as reported by Spadea et al. (2018) with the addition of analytical extrapolation at 154 days, and 75 years. The compressive strengths of the concrete used for the deck, curbs and abutments were measured at 154 days and analytically extrapolated at 75 years. The analytical predictions are based on an extrapolating function fitted on the available measurements as suggested by Naaman (2012). Concrete tensile strength and elastic modulus are computed per ACI 318 (2014). The elastic modulus extrapolated from measured values of concrete strength includes the aggregate characterization factor recommended by the FDOT Design Manual (FDOT 2019a).

Prestress and prestress losses

Concrete creep and shrinkage along with reinforcement relaxation result in the progressive loss of prestress over time. The step-by-step procedure discussed by Naaman (2012) was applied to the calculation of prestress losses. The relaxation of CFRP strands was modelled as reported by Tokyo Rope (2014). The level of prestress in the strands was periodically measured using vibrating gages installed at midspan of one of the girders. The values reported in Table 10 and shown in Figure 9 are the mean of the measurements recorded on the top strand (T9) and bottom strands (T1).

Each girder is prestressed with 18 CFRP strands of 15.2 mm diameter. Design considerations required each strand to be initially pulled at 183 kN for a total prestress of 3,305 kN on each girder. The value is in line with the initial measure of 3,207 kN recorded through strain monitoring at jacking. The design estimation of prestress losses at 75 years
(12%) is computed as a function of the design properties of reinforcement and concrete. The analytical prestress estimation at 75 years (13%) is a prediction based on a non-linear structural model calibrated on the experimental properties of the material. The prediction is in line with measured and design values. Losses are expressed as a percentage of the initial pull.

**Structural Members**

**CFRP-PC/BFRP-RC girders**

Each girder is shaped as a thin-walled double tee and is prestressed with 9 CFRP strands per stem of 15.2 mm diameter per stem. The end portions of each stem include two BFRP meshes acting as additional longitudinal and transverse reinforcement to strengthen the prestress transfer region (M10 at 150 mm in both directions). An M25 BFRP bar is located at the top and at the bottom of each stem to connect the meshes and ease constructability. The flange of each girder includes a BFRP mesh acting as longitudinal reinforcement in the longitudinal and transverse directions (M10 at 150 mm along the girder axis, M13 at 150 mm in the transverse direction). M13 BFRP closed stirrups were installed at 200 mm spacing to act as shear connector. Once installed, the girders were completed with a finishing pour to form the deck and with cast-in-place curbs. The finishing pour is reinforced with a BFRP mesh acting as shrinkage and temperature reinforcement (M10 at 20 mm). Furthermore, the BFRP mesh guarantees continuity through the wet joint that connects the two girders placed side-by-side. Four M16 BFRP bars were provided as longitudinal reinforcement in the curbs. Figure 6 shows the geometry and reinforcement layout of one girder.
Each girder has a free-span of 20 m and is designed for a live load of 4.8 kN/m² in addition to its self-weight. Assuming a simply supported configuration, this corresponds to a service moment demand of 1,284 kN-m, a service shear demand of 254 kN, a flexural strength demand of 1,716 kN-m, and a shear strength demand of 343 kN. Design resistances and demands are summarized in Table 11. A fully composite interaction with the cast-in-place deck and curbs is assumed.

Each girder is designed to remain uncracked under service load. The cracking moment is computed as the one required to relieve the extreme intrados fiber from its condition of initial prestress, and further apply a state of tension equal to the tensile strength of the concrete defined per ACI 318 (2014) and reported in Table 9. The approach is in line with ACI 440.4R (ACI 2004). The cracking moment is a function of the level of prestress. The higher the prestress, the higher the cracking moment. Therefore, as a safe side assumption, the level of prestress at 75 years – after long term losses – is considered for service calculation, as reported in Table 10. The service moment resistance is set equal to the cracking moment and corresponds to 1,424 kN-m. The service shear resistance is equal to 814 kN. It is computed considering the entire depth of the concrete section with an effective width of 305 mm and applying the ACI 440.1R formulation to compute the concrete contribution to shear (ACI 2015). This formulation is more refined with respect to the equation reported in ACI 440.4R (ACI 2004).

The flexural strength of each girder is computed as detailed in ACI 440.4R (ACI 2004). In spite of a closed-form solution, an iterative approach based on strain compatibility and equilibrium equations is implemented because of the complex shape of the section. The composite girder is under reinforced. Failure happens when the total strain in the bottom
strand T1 equals the ultimate design strain ($\varepsilon_{fd}$). The presence of a sustained state of tension in the strands ($\varepsilon_p$) and a preexisting state of tension in the concrete section ($\varepsilon_0$) that corresponds to the prestress losses shall be accounted for. The failure condition in the bottom strand is formalized in Eq. 7 and is shown in Figure 10.

$$\varepsilon_{fd} = \varepsilon_p + \Delta \varepsilon_f = \varepsilon_{p,i} - \varepsilon_0 + \Delta \varepsilon_f$$  \hspace{1cm} (7)

The nominal moment resistance is computed implementing flexural equilibrium over the failure configuration. It equals 3,127 kN-m. The failure configuration is tension-controlled, but close to a balanced condition. Thus, an intermediate value of 0.82 for the strength reduction factor shall be applied (ACI 2004) resulting in a factored moment resistance of 2,575 kN-m. Shear strength is computed with reference to ACI 440.4R (ACI 2004) but integrating the procedure with refined equations from ACI 440.1R (ACI 2015). The concrete contribution is computed assuming an effective width of 305 mm and considering the role of prestress in lowering the neutral axis. The contribution of the FRP stirrups is computed considering the M13 closed BFRP stirrups as the weak component in a shear transfer mechanism involving also the BFRP meshes in the stems of the girder. The contribution of prestress in the vertical direction ($V_{p,4R}$) can be computed according to ACI 440.4R (ACI 2004) and is equal to zero due to the straight path of the strands. The procedure entails simplifying assumptions but is deemed sufficient for design purposes, whereas safety is guaranteed by the inherent conservativeness of the ACI 440.1R and ACI 440.4R equations (ACI 2004) (ACI 2015). Further refinement can be achieved resorting to the Modified Compression Field Theory (MCFT) detailed as an alternative procedure in AASHTO (AASHTO 2018a, 2018b). However, the simplicity of the traditional approach
is preferred in this instance. The nominal shear strength \( (V_n) \) computed per Eq. 8 corresponds to 463 kN. The factored strength is computed applying a strength reduction factor of 0.75 and corresponds to 347 kN.

\[
V_n = V_{c,1R} + V_{f,jR} + V_{p,AR}
\]  

(8)

In addition to service and strength considerations, the member is designed to avoid creep rupture of the CFRP strands under sustained load. To limit the initial pull to values within the jacking strength \( (f_{j}) \) of the strand typically guarantees the creep load requirement to be fulfilled. However, the influence of external loads and prestress eccentricity shall be accounted for in calculations. The bottom strand is relieved of a portion of its initial prestress because of eccentric elastic losses. Conversely, the top strand retains a higher sustained pull and governs the design against sustained load. The issue can be visualized comparing the continuous lines representing the sustained strain in each strand in Figure 10. The stress in the top strand when the entire sustained load is applied (deck and curbs pouring) equals 1,468 MPa and corresponds to 99% of the factored material capacity. The demand will reduce over time as a consequence of long-term losses following concrete creep, shrinkage, and strand relaxation.

**BFRP-RC/GFRP-RC deck**

The deck of the Innovation Bridge is composed of a precast portion collaborating with a cast-in-place finishing pour. The precast portion – the flange of each girder – includes a BFRP mesh acting as longitudinal reinforcement against positive bending moment as shown in Figure 11a (M10 at 150 mm along the girder axis, M13 at 150 mm in the transverse direction). The finishing pour includes a BFRP mesh acting as shrinkage and
temperature reinforcement, and guaranteeing continuity through the wet joint connecting the two girders located side-by-side as shown in Figure 11b (M10 at 200 mm). The girders are designed as simply supported for positive bending moment. Nevertheless, they have fixed end connections to the abutments. These connections are reinforced with M13 GFRP bars at 200 mm running through the wet joints, as shown in Figure 11c.

This section discusses the flexural design for positive and negative bending moment of the BFRP-RC deck, the design for punching shear of the BFRP-RC deck, and the flexural design for negative bending moment of the GFRP-RC wet joints connecting the girders to the bridge abutments. The deck is designed considering a unitary width of 610 mm to resist the most demanding among a distributed live load of 4.8 kN/m² and a concentrated live load of 6.7 kN. The end joints are poured during deck and curbs pouring. Thus, they are only engaged by live loads, whereas the entire self-weight of the structure is resisted by the girders in a simply-supported configuration. The end joints are designed considering the entire width of 2.165 m of a single girder.

Flexural design and punching shear design follow the approach detailed in ACI 440.1R (ACI 2015) but adopting the design parameters specified in Table 2 and Table 3. Each section is designed for strength, service load, and sustained service load. The crack width requirement governs the design for service load in each section. The sustained load only includes a 20% portion of the live load. The approach is in line with ACI 440.1R (ACI 2015) and is adopted in AASHTO (2018b). The sectional behavior of a unitary strip of bridge deck under positive bending moment is visualized in Figure 12. The member is under reinforced and its failure is governed by FRP tensile failure (Tensile F.). Given the conservativeness of the strength reduction factor for FRP-controlled failures (ϕf), the
strains under Ultimate Load (Ultimate L.) lay below the flexural failure threshold defined as Nominal Resistance (Nominal R.) by a margin of more than 50%. Furthermore, the applied load is only a portion of the factored resistance. Design resistances and demands are summarized in Table 12.

**BFRP-RC substructure**

The girders rest on two BFRP-RC bent caps that are cast-in-place and monolithic with the other portions of the BFRP-RC abutments as shown in Figure 13a. The GFRP bars reinforcing the fixed end connections are spliced with an equal number of bent GFRP bars embedded in the pile cap as shown in Figure 13b. Thus, the abutments are made monolithic with the bridge superstructure. GFRP bars were selected for this application in reason of their sand-coated surface that ensures superior bond characteristics. Bond is critical to ensure stress transfer in structural joints. Each bent cap rests on four auger cast BFRP-RC piles reinforced with preassembled BFRP cages for eased installation as shown in Figure 14. Each bent cap is reinforced with 8 M25 longitudinal BFRP bars on top and bottom. Transverse reinforcement is provided by M13 4 legs BFRP stirrups. Auger cast piles are 12-meter long with a 401 mm diameter. They are reinforced with 6 M20 BFRP bars and a filament-winded M10 spiral with a pitch of 150 mm for the first 4.6 m and 300 mm for the remaining length. Pile cages are similar to the ones discussed by Spadea et al. (2018b). Both the bent caps and auger cast piles carry minimal loads.

The reinforcement in the bent caps is designed for constructability and crack control under thermal and shrinkage loading according to ACI 440.1R (ACI 2015). Its structural function is secondary. Longitudinal reinforcement in the auger cast piles is designed to have an area of reinforcement at least equal to 1% of the gross concrete cross sectional
area. Transverse reinforcement in the auger cast piles is designed to match ACI 440.1R minimum requirements for flexural members (ACI 2015). Shear reinforcement design of FRP-RC members is discussed by Razaqpur & Spadea (2015). In addition, a maximum pitch of 150 mm is imposed in the upper portion of the pile to ensure confinement. Below 4.6 m the lateral stability of the pile is guaranteed by soil-structure interaction and the requirement can be lifted. The design approach is in line with AASHTO (2018b) that represents the first design guideline to include provisions for the design of FRP-RC bridge substructures and abutments. Rossini et al. (2018a) discusses the design of a more representative GFRP-RC bent cap.

**Minimum reinforcement**

Minimum amounts of reinforcement are required in every RC and PC member for constructability, crack control under temperature and shrinkage loading, and to ensure a minimal level of flexural capacity (ACI 2014). In flexural RC and PC members, a minimum amount of reinforcement is required to ensure that, after cracking, the section is able to withstand a demand at least equal to its cracking moment \( M_{cr} \) (Gambles 2017). The minimum amount of reinforcement can be calibrated as a function of the mechanical properties of the reinforcing material. This ensures a minimum level of deflection at failure – pseudo-ductility – irrespectively of the brittleness of the reinforcing material (AASHTO 2017). The minimum reinforcement requirement can typically be waived whenever the factored flexural strength \( M_f \) of the element exceeds the flexural strength demand \( M_u \) by 33% (ACI 2014, ACI 2015, AASHTO 2017).

These concepts are formalized in different guidelines in a variety of implicit or explicit formulations. Implicit requirements are imposed on PC members in terms of a minimum
factored-flexural-strength-to-cracking-moment ratio (ACI 2004, 2014). Ratios range from
1.2 for HSCS-PC members (ACI 2014) to 1.5 for FRP-PC members (ACI 2004). Conversely, explicit requirements are imposed on RC members in terms of a minimum area of longitudinal reinforcement (ACI 2014, 2015). These correspond to a factored-flexural-strength-to-cracking-moment ratio of 1.6.

AASHTO (2017) recommends a unified implicit approach for MCS-RC and HSCS-PC. The approach is calibrated on the ductility of each material and can be adapted to include FRP-RC/PC members. FRP reinforcement is considered a purely brittle material. This results in an implicit factored-flexural-strength-to-cracking-moment ratio ranging from approximately 1.2 to 1.6 as a function of the level of prestress applied. The approach provides the advantage of formal consistency and general applicability. The approach is adopted in AASHTO (2018a, 2018b) and formalized in Eq. 9 in its most demanding formulation that is conservatively applied to both RC and PC elements in this study.

\[ M_r \geq \min(1.6M_u, 1.33M_u) \] (9)

The minimum reinforcement requirement can be verified for each flexural member of the bridge – either RC or PC – applying the proposed unified approach. Table 13 summarizes the results. The minimum reinforcement requirement at the deck joint is waived given a factored moment resistance \( M_r \) exceeding the factored moment demand \( M_u \) by more than 30%. Minimum reinforcement in the deck needs also to be provided for temperature and shrinkage requirement according to ACI 440.1R (ACI 2015). The requirement for minimum reinforcement in the bent cap is waived given its large section, short span and limited loading. The minimum reinforcement for temperature and shrinkage
in the bent cap is provided in line with ACI 440.1R (ACI 2015). Minimum reinforcement in the piles follows different requirements given their purely compressive nature and was discussed in the previous section.

**Load Tests**

**Instrumentation and test setup**

The Innovation Bridge is instrumented for continuous monitoring and has been load tested to validate its performances. Instruments include vibrating gages for strain monitoring, dial gages for deflection measurements during load test at the precast yard, surveying target and total stations for camber monitoring and deflection measurements during load test on site. Three vibrating gages are located at midspan and attached to the bottom CFRP strand (T1), the top CFRP strand (T9), and to one of the BFRP longitudinal bars in the flange. Dial gages and surveying targets were positioned over the span of one of the girders at one quarter, midspan, three quarters, and at the very ends.

The results of four load tests are reported in the following section. The first test was performed on one of the girders at the precast yard at 26 days from casting. The girder was loaded in a three-point-bending configuration with simply supported ends. A load of 120 kN was applied using a crane as shown in Figure 15a. The load applied in this configuration corresponds to 75% of the net cracking moment of the girder. Deflection measurements were taken using dial gages, while strain measurements were taken using vibrating gages.

The second test of the performance of the girders corresponded to the pouring of the concrete to form deck and curbs. Before hardening, concrete acts as a dead weight of 0.6 kN/m on each of the two double tees corresponding to 40% of the net cracking moment.
Strains were monitored using the vibrating gages installed on the strands and in the reinforcement located in the compressed zone. Deflections were measured using a total station and surveying equipment.

The first test on site was performed on the finished structure on site at 154 days from casting. The bridge was loaded in a four-point-bending configuration with fixed ends. The load was applied using a truck weighting 53 kN in total, corresponding to 27 kN per girder as shown in Figure 15b. The load applied in this configuration corresponds to 8% of the net cracking moment of the entire structure. Deflection measurements were taken using surveying equipment, while strain measurements were taken using vibrating gages.

The second load test on site was performed one year into the service life of the structure. The bridge was loaded using three different trucks weighting 106 kN in total, corresponding to 53 kN per girder over six points of application. The load applied in this configuration corresponds to 11% of the net cracking moment of the entire structure. Deflection measurements were taken using surveying equipment, while strain measurements were taken using vibrating gages.

**Analytical model**

During each of the four load tests the cracking moment of the girder or the structure was deliberately never exceeded. This allows to model the behavior of the structure using a traditional linear-elastic approach. The flexural stiffness of the member is defined considering an uncracked homogenized section. When the entire bridge is considered, the deck and curbs are assumed to be perfectly collaborating with the underlying double-tee girders as a fully composite structure. The double-tees are assumed to be simply supported
during load test at the precast yard and during concrete pouring. Conversely, after completion, perfectly fixed connections are assumed at the end joints.

The material properties of both concrete and reinforcement are taken as their measured mean values. The model is refined to account for concrete hardening and stiffening overtime, and progressive loss of prestress as discussed in the section of this chapter dealing with material properties. The stiffness of FRP reinforcement do not significantly vary over time (ACI 2015) so further refinement is not required. The structure is not loaded up to rupture, therefore the strengths of its constituents do not affect this predictive model. Similarly, reduction factors defined in Table 1, Table 2, and Table 3 do not apply since the aim of this model is to provide the closest possible estimate of deflection and strains rather than investigate the strength of the section as discussed in the design section of this chapter.

**Results and discussion**

Deflection measurements along the span of the girder are shown in Figure 16a for the load test performed at the precast yard, and in Figure 17a for concrete pouring. Deflection measurements along the span of the bridge are shown in Figure 18a for the first load test performed on site, and in Figure 19a for the second load test performed on site. Strain measurements along the section of the girder are shown in Figure 16b for the load test performed at the precast yard, in Figure 17b for concrete pouring, in Figure 18b for the first load test performed on site, and in Figure 19a for the second load test performed on site. Deflection and strain measurements are two orders of magnitude less in the load tests performed on site with respect to the load test performed at the precast yard and concrete pouring. The difference in order of magnitudes is expected given the reduced entity of the load applied in the second case, the different load configuration adopted, the different
boundary conditions, and the increased stiffness of the structure following deck and curbs pouring with respect to the girder alone.

The expected and measured values of deflection at midspan and strain at the bottom CFRP strand (T1) are reported in Table 14. The error is measured as the difference of the analytical value minus the experimental value over the experimental value. Measurements taken during load tests are close to expected values. In the case of load tests performed on site the error is higher than in the load test performed at the precast yard. However, this is expected given the fact that the measurements taken on site lay closer to the noise range of the instruments. The results validate the proposed rationale and the performance of the structure.

**Conclusions**

This study is an attempt to develop a unified approach to FRP-RC/PC design that features consistency with procedures traditionally applied to MCS-RC and HSCS-PC. The Innovation Bridge is presented as an example of application of the proposed approach. Specific outcomes of this study are summarized as follows:

1. A unified approach to the design of FRP-RC/PC infrastructural elements is discussed.
   
   This includes: (a) the formalization of a consistent approach to the definition of the design properties of FRP reinforcement, (b) the identification of updated design parameters, and (c) the selection of efficient design equations.

2. The deployment of novel material systems (BFRP) and shapes (preassembled meshes and cages, closed stirrups) in infrastructural works is investigated. Similarities with GFRP bars were noted, and design parameters in line with the ones used for GFRP-RC
are proposed. The approach adopted is on the safe side and acceptable until the properties of BFRP reinforcement are assessed through a comprehensive study.

3. The design of CFRP-PC/BFRP-RC girders, BFRP-RC/GFRP-RC deck and joints, BFRP-RC bent caps and abutments, and BFRP-RC auger-cast piles is discussed. The design of bridge substructures reinforced with BFRP is discussed for the first time. The design approach is in line with the second edition of the AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete (AASHTO 2018b).

4. Design parameters for FRP-RC have been relaxed with respect to the state-of-the-practice where applicable. This includes the adoption of a flexural strength reduction factor for compression-controlled members of 0.75, instead of 0.65; the adoption of a creep rupture reduction factor of 0.30, instead of 0.20; and, the adoption of a bond reduction factor of 0.83, instead of 0.71. The values are in line with the second edition of the AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete (AASHTO 2018b).

5. The reduction factor that limits the jacking strength of CFRP strands is raised from 0.65 to 0.75. This results in performance comparable to HSCS strands. The performances over time of the CFRP strands are assessed through continuous strain monitoring. Prestress losses were estimated in 12% at 75 years at the design stage. The value is in line with the 13% prediction extrapolated from strain monitoring.

6. The performance of the structure is validated performing four load tests on the Innovation Bridge. The first load test was performed on a single girder at the precast yard and showed differences of 1% and 8% when comparing the expected deflection and strain values to measured ones. The second load test was performed while pouring
concrete for the deck and curbs. The other two load tests were performed on the entire bridge in service conditions. They featured good alignment in deflection measurements (±18%). The higher error on strain measurements is a consequence of measuring quantities that are close to the noise range of the instruments.

7. To foster the didactic component of this study, a complete set of design information is provided: (a) the mechanical properties of CFRP strands, GFRP bars, BFRP bars are provided, and design properties are computed and compared to traditional reinforcement materials; (b) the mechanical properties of concrete mixes used in the Innovation Bridge are provided, and their evolution over time is discussed; (c) load demands and structural resistances at the various limit states for the various members of the Innovation Bridge are computed and reported.
Table 1 – Summary of CFRP design guidelines and design factors.

<table>
<thead>
<tr>
<th>CFRP</th>
<th>JCSE CES23</th>
<th>CNR DT203</th>
<th>fib</th>
<th>CSA S806</th>
<th>CSA S6</th>
<th>ACI 440 4R</th>
<th>ACI 440 1R</th>
<th>Minstroy SP295</th>
<th>ICC AC454</th>
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<th>AASHTO CFRP</th>
<th>Design values</th>
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</table>

^ a refined approach to durability is discussed in fib bulletin 40 (fib 2007).
* ACI 440.4R (ACI 2004) lack an explicit definition of the design strength.
(1) from ACI 440.4R (ACI 2004).
(2) from ACI 440.1R (ACI 2015).
(3) Applied without environmental reduction factor.
(4) from fib bulletin 40 (fib 2007).
(5) from AASHTO LRFD Bridge Design Specifications (AASHTO 2017).
Table 2 – Summary of GFRP design guidelines and design factors.

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\(^{\wedge}\) a refined approach to durability is discussed in fib bulletin 40 (fib 2007).

\(^*\) CSA S806 (CSA 2012) includes a limit of 0.002 on the sustained strain.

(1) from ACI 440.1R (ACI 2015).

(2) from ACI 318 (ACI 2014).

(3) Applied without environmental reduction factor.

(4) from fib bulletin 40 (fib 2007).
Table 3 – Summary of BFRP design guidelines and design factors.

<table>
<thead>
<tr>
<th>BFRP</th>
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(1) from ACI 318 (ACI 2014).
(2) Applied without environmental reduction factor.
Table 4 – Jacking reduction factor in current practice.

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<td>Lapeer Road Bridges</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Ushijima et al. (2016)</td>
</tr>
<tr>
<td>0.72</td>
<td>Double-tee</td>
<td>Kittery Overpass</td>
<td>MA 2014</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Ushijima et al. (2016)</td>
</tr>
<tr>
<td>0.61</td>
<td>I girder</td>
<td>KY 70 Bridge</td>
<td>KY 2014</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>Ushijima et al. (2016)</td>
</tr>
<tr>
<td>0.60</td>
<td>Bulb-tee</td>
<td>Route 49 Bridge</td>
<td>VA 2015</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Ushijima et al. (2016)</td>
</tr>
<tr>
<td>0.62</td>
<td>Piles</td>
<td>Halls River Bridge</td>
<td>FL 2018</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Rossini et al. (2018b)</td>
</tr>
<tr>
<td>0.70</td>
<td>Sheet piles</td>
<td>Halls River Bridge</td>
<td>FL 2018</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td>Rossini et al. (2018b)</td>
</tr>
<tr>
<td>0.75</td>
<td>Double-tee</td>
<td>Innovation Bridge</td>
<td>FL 2016</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Spadea et al. (2018)</td>
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</table>

Table 5 – Experimental and design properties for No. 3 (M10) BFRP bars.

<table>
<thead>
<tr>
<th>BFRP M10</th>
<th>Stress / MPa</th>
<th>Strain %</th>
<th>Force kN</th>
<th>Modulus GPa</th>
<th>Area mm$^2$</th>
<th>Diameter mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>$f_{tn}$</td>
<td>1,227</td>
<td>2.71</td>
<td>88</td>
<td>45.5</td>
<td>97</td>
</tr>
<tr>
<td>Guaranteed</td>
<td>$f_{tu}^*$</td>
<td>1,089</td>
<td>2.39</td>
<td>77</td>
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<td></td>
</tr>
<tr>
<td>Specified</td>
<td>$f_{tu}'$</td>
<td>827</td>
<td>1.84</td>
<td>59</td>
<td>44.8</td>
<td>71</td>
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<tr>
<td>Design</td>
<td>$f_{td}$</td>
<td>579</td>
<td>1.29</td>
<td>41</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jacking</td>
<td>$f_{fj}$</td>
<td>n.a.</td>
<td>n.a.</td>
<td>n.a.</td>
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<td></td>
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<tr>
<td>Sustained</td>
<td>$f_{fs}$</td>
<td>172</td>
<td>0.39</td>
<td>12</td>
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Table 6 – Experimental and design properties for No. 4 (M13) BFRP bars.

<table>
<thead>
<tr>
<th>BFRP M13</th>
<th>Stress / MPa</th>
<th>Strain %</th>
<th>Force kN</th>
<th>Modulus GPa</th>
<th>Area mm$^2$</th>
<th>Diameter mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>$f_{tn}$</td>
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<td>2.11</td>
<td>130</td>
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<td>Guaranteed</td>
<td>$f_{tu}^*$</td>
<td>903</td>
<td>1.86</td>
<td>115</td>
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<td></td>
</tr>
<tr>
<td>Specified</td>
<td>$f_{tu}'$</td>
<td>758</td>
<td>1.69</td>
<td>96</td>
<td>44.8</td>
<td>129</td>
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<tr>
<td>Design</td>
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<td>1.18</td>
<td>67</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jacking</td>
<td>$f_{fj}$</td>
<td>n.a.</td>
<td>n.a.</td>
<td>n.a.</td>
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<td></td>
</tr>
<tr>
<td>Sustained</td>
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<td>159</td>
<td>0.3</td>
<td>20</td>
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Table 7 – Experimental and design properties for No. 5 (M16) GFRP bars.

<table>
<thead>
<tr>
<th>GFRP M16</th>
<th>Stress / MPa</th>
<th>Strain / %</th>
<th>Force / kN</th>
<th>Modulus / GPa</th>
<th>Area / mm$^2$</th>
<th>Diameter / mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>$f_{m}$</td>
<td>896</td>
<td>1.81</td>
<td>177</td>
<td>49.6</td>
<td>213</td>
</tr>
<tr>
<td></td>
<td>$f_{u}^*$</td>
<td>779</td>
<td>1.57</td>
<td>154</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Guaranteed</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Specified</td>
<td>$f_{u}'$</td>
<td>655</td>
<td>1.46</td>
<td>129</td>
<td>44.8</td>
<td>199</td>
</tr>
<tr>
<td>Design</td>
<td>$f_{u}$</td>
<td>455</td>
<td>1.02</td>
<td>91</td>
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<td></td>
</tr>
<tr>
<td>Jacking</td>
<td>$f_{j,j}$</td>
<td>n.a.</td>
<td>n.a.</td>
<td>n.a.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sustained</td>
<td>$f_{j,s}$</td>
<td>138</td>
<td>0.31</td>
<td>27</td>
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Table 8 – Experimental and specified properties of 0.6 in. (15.2 mm) CFRP strands.

<table>
<thead>
<tr>
<th>CFRP 15.2</th>
<th>Stress / MPa</th>
<th>Strain / %</th>
<th>Force / kN</th>
<th>Modulus / GPa</th>
<th>Area / mm$^2$</th>
<th>Diameter / mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>$f_{m}$</td>
<td>3,199</td>
<td>2.15</td>
<td>370</td>
<td>149</td>
<td>116</td>
</tr>
<tr>
<td>Guaranteed</td>
<td>$f_{u}^*$</td>
<td>2,889</td>
<td>1.94</td>
<td>334</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Specified</td>
<td>$f_{u}'$</td>
<td>2,337</td>
<td>1.57</td>
<td>270</td>
<td>149</td>
<td>116</td>
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<tr>
<td>Design</td>
<td>$f_{d}$</td>
<td>2,103</td>
<td>1.41</td>
<td>243</td>
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<tr>
<td>Jacking</td>
<td>$f_{j,j}$</td>
<td>1,579</td>
<td>1.06</td>
<td>182</td>
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<td></td>
</tr>
<tr>
<td>Sustained</td>
<td>$f_{j,s}$</td>
<td>1,475</td>
<td>0.99</td>
<td>170</td>
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</table>
Table 9 – Design, experimental, and analytically predicted concrete properties.

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Jacking Measure</th>
<th>Transfer Measure</th>
<th>28 days Measure</th>
<th>154 days Mixed</th>
<th>75 years Analytic</th>
<th>Design ACI 318</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>$f'_c$ MPA</td>
<td>36.6</td>
<td>3.76</td>
<td>24.8</td>
<td>55.6</td>
<td>58.3</td>
<td>59.8</td>
</tr>
<tr>
<td></td>
<td>$f_r$ MPA</td>
<td></td>
<td></td>
<td>n.a.</td>
<td>55.6</td>
<td>58.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$E_c$ GPa</td>
<td></td>
<td></td>
<td></td>
<td>4.65</td>
<td>4.76</td>
<td>4.81</td>
</tr>
<tr>
<td></td>
<td>$w_c$ kg/m³</td>
<td></td>
<td></td>
<td></td>
<td>30.6</td>
<td>31.3</td>
<td>31.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>n.a.</td>
<td>35.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>n.a.</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2,249</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>n.a.</td>
<td>2,323</td>
</tr>
<tr>
<td>Deck</td>
<td>$f'_c$ MPA</td>
<td></td>
<td></td>
<td></td>
<td>50.6</td>
<td>55.1</td>
<td>27.6</td>
</tr>
<tr>
<td></td>
<td>$f_r$ MPA</td>
<td></td>
<td></td>
<td></td>
<td>4.43</td>
<td>4.62</td>
<td>3.27</td>
</tr>
<tr>
<td></td>
<td>$E_c$ GPa</td>
<td></td>
<td></td>
<td></td>
<td>29.0</td>
<td>30.2</td>
<td>25.1</td>
</tr>
<tr>
<td></td>
<td>$w_c$ kg/m³</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2,238</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>n.a.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2,323</td>
<td></td>
</tr>
<tr>
<td>Curbs</td>
<td>$f'_c$ MPA</td>
<td></td>
<td></td>
<td></td>
<td>53.1</td>
<td>59.7</td>
<td>27.6</td>
</tr>
<tr>
<td></td>
<td>$f_r$ MPA</td>
<td></td>
<td></td>
<td></td>
<td>4.54</td>
<td>4.81</td>
<td>3.27</td>
</tr>
<tr>
<td></td>
<td>$E_c$ GPa</td>
<td></td>
<td></td>
<td></td>
<td>29.7</td>
<td>31.5</td>
<td>25.1</td>
</tr>
<tr>
<td></td>
<td>$w_c$ kg/m³</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2,238</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>n.a.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2,323</td>
<td></td>
</tr>
</tbody>
</table>

Table 10 – Design, experimental, and analytically predicted prestress losses.

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Jacking Measure</th>
<th>Transfer Measure</th>
<th>28 days Measure</th>
<th>154 days Mixed</th>
<th>75 years Analytic</th>
<th>Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>$\varepsilon_{pm}$ %</td>
<td>1.04</td>
<td>0.97</td>
<td>0.92</td>
<td>0.91</td>
<td>0.90</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>$P_i$ kip</td>
<td>178</td>
<td>166</td>
<td>159</td>
<td>156</td>
<td>155</td>
<td>158</td>
</tr>
<tr>
<td></td>
<td>$P$ kip</td>
<td>3,207</td>
<td>2,994</td>
<td>2,856</td>
<td>2,807</td>
<td>2,785</td>
<td>2,851</td>
</tr>
<tr>
<td></td>
<td>$\Delta P$</td>
<td>n.a.</td>
<td>-7%</td>
<td>-11%</td>
<td>-12%</td>
<td>-13%</td>
<td>-12%</td>
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</table>
Table 11 – Design resistances and demands for the CFRP-PC/BFRP-RC girder.

<table>
<thead>
<tr>
<th></th>
<th>Strength</th>
<th></th>
<th>Service</th>
<th></th>
<th>Sustained stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flexure</td>
<td>Shear</td>
<td>Flexure</td>
<td>Shear</td>
<td></td>
</tr>
<tr>
<td></td>
<td>kN-m</td>
<td>kN</td>
<td>kN-m</td>
<td>kN</td>
<td>MPa</td>
</tr>
<tr>
<td>Nominal resistance</td>
<td>3,127</td>
<td>463</td>
<td>1,424</td>
<td>814</td>
<td>1,475</td>
</tr>
<tr>
<td>Factored resistance</td>
<td>2,575</td>
<td>347</td>
<td>1,424</td>
<td>814</td>
<td>1,475</td>
</tr>
<tr>
<td>Demand</td>
<td>1,716</td>
<td>343</td>
<td>1,284</td>
<td>254</td>
<td>1,469</td>
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<tr>
<td>Exploitation Ratio</td>
<td>0.67</td>
<td>0.98</td>
<td>0.90</td>
<td>0.31</td>
<td>0.99</td>
</tr>
</tbody>
</table>
Table 12 – Design resistances and demands for the BFRP-RC deck and GFRP-RC end joints.

<table>
<thead>
<tr>
<th></th>
<th>Deck</th>
<th>Deck joint</th>
<th>End joint</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Flexural</td>
<td>Flexural</td>
<td>Flexural</td>
</tr>
<tr>
<td></td>
<td>strength</td>
<td>strength</td>
<td>strength</td>
</tr>
<tr>
<td></td>
<td>kN-m</td>
<td>kN-m</td>
<td>kN-m</td>
</tr>
<tr>
<td>Nominal</td>
<td>31.3</td>
<td>19.7</td>
<td>815</td>
</tr>
<tr>
<td>Factored</td>
<td>17.2</td>
<td>10.8</td>
<td>449</td>
</tr>
<tr>
<td>Demand</td>
<td>4.9</td>
<td>2.4</td>
<td>420</td>
</tr>
<tr>
<td>Exploitation Ratio</td>
<td>0.28</td>
<td>0.23</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>Punching</td>
<td>Crack</td>
<td>Crack</td>
</tr>
<tr>
<td></td>
<td>shear</td>
<td>width</td>
<td>width</td>
</tr>
<tr>
<td></td>
<td>kN</td>
<td>kN-m</td>
<td>kN-m</td>
</tr>
<tr>
<td>Nominal</td>
<td>33.4</td>
<td>5.6</td>
<td>267</td>
</tr>
<tr>
<td>Factored</td>
<td>24.9</td>
<td>5.6</td>
<td>267</td>
</tr>
<tr>
<td>Demand</td>
<td>10.7</td>
<td>1.6</td>
<td>263</td>
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<td></td>
<td>Crack width</td>
<td>Sustained load</td>
<td>Sustained load</td>
</tr>
<tr>
<td></td>
<td>kN-m</td>
<td>kN-m</td>
<td>kN-m</td>
</tr>
<tr>
<td>Nominal</td>
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<td>5.0</td>
<td>233</td>
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<tr>
<td>Factored</td>
<td>13.6</td>
<td>5.0</td>
<td>233</td>
</tr>
<tr>
<td>Demand</td>
<td>3.1</td>
<td>0.5</td>
<td>53</td>
</tr>
<tr>
<td>Exploitation Ratio</td>
<td>0.23</td>
<td>0.10</td>
<td>0.23</td>
</tr>
</tbody>
</table>
Table 13 – Minimum reinforcement requirements for each flexural member of the bridge.

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Girder</th>
<th>Deck</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Midspan</td>
<td>End joint</td>
</tr>
<tr>
<td>$M_n$ kN-m</td>
<td>3,127</td>
<td>815</td>
</tr>
<tr>
<td>$M_r$ kN-m</td>
<td>2,344</td>
<td>449</td>
</tr>
<tr>
<td>$M_{cr}$ kN-m</td>
<td>1,423</td>
<td>251</td>
</tr>
<tr>
<td>1.33 $M_u$ kN-m</td>
<td>2,282</td>
<td>560</td>
</tr>
<tr>
<td>$M_r / M_{cr}$</td>
<td>1.65</td>
<td>1.79</td>
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</tbody>
</table>

Table 14 – Expected and measured values of deflection at midspan, and strain at the bottom CFRP strand.

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Load test at precast yard</th>
<th>Concrete pouring</th>
<th>First load test on site</th>
<th>Second load test on site</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>δ</td>
<td>ε_TI</td>
<td>δ</td>
<td>ε_TI</td>
</tr>
<tr>
<td>Analytical</td>
<td>1.09</td>
<td>0.038</td>
<td>0.46</td>
<td>0.017</td>
</tr>
<tr>
<td>Experimental</td>
<td>1.10</td>
<td>0.036</td>
<td>0.36</td>
<td>0.022</td>
</tr>
<tr>
<td>Error</td>
<td>-1%</td>
<td>+8%</td>
<td>+27%</td>
<td>-24%</td>
</tr>
</tbody>
</table>
Figure 5 – Innovation Bridge after completion (a), and one double-tee after demolding (b). (Rossini et al. 2019d)
Figure 6 – Geometry of the CFRP-PC/BFRP-RC double-tee girder (1 in. = 25.4 mm). (Rossini et al. 2019d)

Figure 7 – Jacking apparatus for CFRP strands (Spadea et al. 2018).
Figure 8 – Mechanical properties of various reinforcing materials. (1 ksi = 6.89 MPa).
(Rossini et al. 2019d)

Figure 9 – Measured level of prestress compared to design value before and after losses.
(1 kip = 4.45 kN). (Rossini et al. 2019d)
Figure 10 – Sectional analysis of the CFRP-PC/BFRP-RC. (1 in. = 25.4 mm; 1 ksi = 6.89 MPa). (Rossini et al. 2019d)

Figure 11 – Reinforcement for the flange of the girders (a), the deck pour (b), and the end joints (c). (Rossini et al. 2019d)
Figure 12 – Sectional analysis of a strip of BFRP-RC deck under positive moment. (1 in. = 25.4 mm). (Rossini et al. 2019d)

Figure 13 – BFRP reinforcement cage for the bent cap and abutments (a), and detail of the end joint (b). (Rossini et al. 2019d)
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Figure 15 – Load test setup at the precast yard (a), and on site (b). (Rossini et al. 2019d)
Figure 16 – Deflection measurements (a), and strain measurements (b) during the load test at the precast yard. (1 in. = 25.4 mm; 1 ft = 0.30 m) (Rossini et al. 2019d)
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Figure 18 – Deflection measurements (a), and strain measurements (b) during the first load test on site (Two orders of magnitude less than Figure 16 and Figure 17). (1 in. = 25.4 mm; 1 ft = 0.30 m) (Rossini et al. 2019d)
Figure 19 - Deflection measurements (a), and strain measurements (b) during the second load test on site. (Two orders of magnitude less than Figure 16 and Figure 17). (1 in. = 25.4 mm; 1 ft = 0.30 m)
Chapter 3

Prestressing with GFRP

The experimental portion of this project explores the potential of GFRP strands coupled with traditional steel anchors as an efficient, easy-to-deploy, and non-corrosive pre-tensioning solution in applications that require limited levels of prestress. An overview of available solutions and historic experiences in the state-of-the-practice of prestressing technologies with non-conventional tendons is also presented to determine whether mild prestressing with Glass Fiber Reinforced Polymer (GFRP) strands may represent a solution for some of the existing challenges. The strands are composed of seven twisted wires and have a nominal diameter of 15.2 mm. This study on GFRP strand prototypes covers: a) mechanical and geometrical characterization; b) jacking strength of the coupled strand-anchor under instantaneous and sustained load; c) load loss during anchor setting using various pulling techniques; and, d) some flexural properties and coilability. Results should only be considered indicative of the potential of the technology given also the limited material availability. Experimental outcomes are evaluated with reference to information available in archival literature with particular reference to GFRP reinforcing bars.
State-of-the-practice on FRP Prestressing

The state-of-the-practice of non-corrosive prestressing reinforcement is discussed with reference to mechanical properties of various material systems are summarized in Table 15 in terms of: nominal diameter ($d_n$) and effective area ($a_f$); guaranteed tensile strength ($f_{t,u}^*$) defined as the average minus three standard deviations (ACI 2015); guaranteed ultimate strain ($\varepsilon_{f,u}^*$); average modulus of elasticity ($E_f$); and, guaranteed jacking strength as a fraction of the guaranteed tensile strength.

High-strength carbon steel and epoxy-coated carbon steel

Low-relaxation High-strength Carbon Steel (HSCS) strands have been traditionally used in pre-tensioning and post-tensioning application. Material specifications for HSCS strands are covered by ASTM A416 (ASTM 2017a) while design provisions are in ACI 318 (ACI 2014) and AASHTO LRFD Bridge Design Specifications (AASHTO 2017). HSCS strands feature good mechanical performances with an ultimate tensile strength ranging from 1725 to 1860 MPa depending on grade, a modulus of elasticity of approximately 196 GPa, and a minimum ultimate strain equal to 3.5%. Strands that are composed of seven wires and have a diameter of 15.2 mm with an effective area of 140 mm$^2$ are commonly applied in pre-tensioning applications. Strands are delivered coiled at the precast yard where they are cut to length, positioned, and pre-tensioned. Typical construction time for a PC element is one day; thus, pre-tension is kept in place for approximately 12 hours via anchors with wedges made of hardened steel (Figure 20). The jacking stress of HSCS strands is limited to 75% of ultimate and relaxation loss is in the
order of 1.0% of ultimate. Concerns pertaining the durability of traditional HSCS are
detailed by various authors (Bertolini et al. 2013, Rostam 2008).

Epoxy-Coated Steel (ECS) bars are a corrosion-resistant metallic solution for concrete
reinforcement. High-strength Epoxy-Coated Steel (HECS) strands have been developed
for prestressing applications (ASTM 2004). However, the use of ECS has proven
ineffective in highly corrosive environments (Rostam 2008) and the deployment of ECS
and HECS is not allowed in the state of Florida where aggressive coastal exposure is
commonplace (FDOT 2019b).

High-strength stainless steel

High-strength Stainless Steel (HSSS) strands have recently been proposed as a
corrosion-resistant alternative. HSSS strands are not covered by national standards in the
US, but material specifications and design guidelines are available from some state
Departments of Transportation (DOTs) (FDOT 2019b). HSSS adoption may be eased by
its similarity with HSCS; however, differences do exist in the material properties as
reported by Paul et al. (2017) and Rossini et al. (2018b). Commercially available HSSS
strands feature an ultimate tensile strength ranging from 1665 to 1724 MPa depending on
manufacturing and constituents. Furthermore, they feature an elastic modulus of
approximately 162 GPa, and an ultimate strain ranging from 1.1% to 1.6%. The higher the
strength, the lower the ultimate strain. The 7-wire strand with a diameter of 12.7 mm has
an effective area ranging from 93 mm² to 99 mm² and is the only commercial product at
the moment because of manufacturing challenges not fully addressed. HSSS strands are
coilable and can be stressed similarly to HSCS strands at a jacking stress of 75% of
ultimate. Relaxation loss in air is reported by Schuetz (2013) to be similar to HSCS strands.
Cold-forming of HSSS wires and twisting into a strand shape is currently a technological challenge (Schuetz 2013). In fact, a ductile behavior cannot be always achieved, and ultimate strain values can be as low as 1.0% corresponding to the conventional yield point for HSCS according to ASTM A416 (ASTM 2017a). The brittle behavior may represent a safety concern during both pulling operations and service. The cost of HSSS is significantly higher than the cost of traditional HSCS (Paul et al. 2017).

**Carbon FRP and aramid FRP**

Coilable Carbon FRP (CFRP) strands made of epoxy resin and Poly-AcryloNitrile (PAN) carbon fibers were developed into a commercial product in the 80s in Japan (Tsuji et al. 1993). The technology has undergone minimal changes during the last 40 years (Spadea et al. 2018). CFRP material specifications and design guides are issued by state DOTs and ACI (FDOT 2019a; FDOT 2019b; ACI 440.4R 2004) while an AASHTO guide was published recently (AASHTO 2018a). CFRP is elastic up to failure, stronger, but less stiff compared to HSCS. Commercially available CFRP strands feature a nominal tensile strength of approximately 2336 MPa corresponding to a nominal strain of 1.5% as declared by manufacturer. However, testing reveals values of guaranteed tensile strength as high 2990 MPa corresponding to a guaranteed strain of 1.9% as reported by Rossini et al. (2018c). The guaranteed strength is defined as the average value of the ultimate strength minus three standard deviations (ACI 2015). When experimental data are not available, the guaranteed strength may conservatively be taken equal to the nominal strength as declared by the manufacturer. CFRP strands feature an elastic modulus of approximately 155 GPa.

The 7-wire strand with a diameter of 15.2 mm has an effective area of 116 mm². As per any FRP composite, the design strength results from multiplying the guaranteed strength
by an environmental knock-down factor to account for exposure conditions. For CFRP, the value for the environmental knock-down factor varies between 0.9 and 1.0 depending on the guideline. Recently published guidelines suggest an environmental knock-down factor of 1.0 for internal prestressing and a jacking stress equal to 70% of the guaranteed tensile strength.

The level of prestress in CFRP strands must be restrained to limit the incidence of viscous phenomena on the mechanical performance of the reinforcement. When FRP is engaged by sustained load, creep deformation develops; if the sustained load exceeds a fixed threshold, creep rupture caused by the excessive accumulation of creep deformation may occur within the service life of the structure. Furthermore, relaxation is a viscous phenomenon specular to creep; when FRP reinforcement is forced to maintain a certain elongation, the stress it retains reduces over time. The creep rupture strength of CFRP strands is set equal to 65% of their guaranteed tensile strength in AASHTO (2018a), and relaxation losses over one-million-hour are reported to be in the order of 2% of the guaranteed tensile strength by Enomoto et al. (2009).

A CFRP-PC member can be either designed for concrete crushing or CFRP rupture. The two failure modes are characterized by different strength reduction factors calibrated to guarantee the same reliability in the two cases. The weakest mechanism among the two governs the behavior of the system.

CFRP strands are orthotropic and feature limited stiffness and strength in the transverse direction. They require a specific anchor system to be pulled at the desired level of prestress without damaging the strand. The wedges must be longer than for traditional anchors to reduce stress concentration. For the same reason, CFRP strands are sleeved using a steel
mesh at the wedge location. Furthermore, splicing with traditional HSCS strand is necessary to connect the anchor to the jacking system. Figure 21 shows CFRP and HSCS strands coupled and ready to be inserted in the prestressing bed. The added complexity slows precasting operations, requires workers to undergo specific training, and demands the constant presence of a supervisor on site to ensure a correct implementation.

The high material cost of CFRP strands (Cadenazzi et al. 2019) demands for extreme pulling to maximize efficiency. Considering the brittle nature of the material, particular care is required during pulling operations. Additionally, the high pre-tension may also cause transverse cracking in the concrete once released (Figure 22) as reported by Roddenberry et al. (2014).

Historic FRP prestressing solutions include Aramid FRP (AFRP) strands. Aramid fibers are organic fibers common in ballistic applications (ACI 2007). From the structural perspective, AFRP shares many of the properties of CFRP. However, it features lower elastic modulus and strength at comparable costs. Furthermore, the durability of AFRP in alkaline and moist environment remains an unaddressed concern (Derombise et al. 2010). Design guidelines are provided by ACI 440.4R (ACI 2004) that also report typical mechanical properties. Even if not used in the US and not regulated by national standards, commercial AFRP prestressing solutions are available. Prestressing with AFRP is permitted by the Canadian Highway Bridge Design Code (CSA 2014) that limits its deployment to dry conditions and restrains jacking stresses to 40% of the guaranteed tensile strength. The creep-rupture strength is limited to 35% of the guaranteed tensile strength. Relaxation losses are reported to be approximately 6 times those of CFRP strands over 100 hours of testing (ACI 2004).
Glass FRP and basalt FRP

Currently, there are no commercially available Glass FRP (GFRP) prestressing strands. Conversely, the use and number of GFRP bar manufacturers are steadily growing (Ruiz et al. 2017). GFRP is characterized by a relatively low modulus of elasticity required to be at least 45 GPa by ASTM D7957 (ASTM 2017b) and a strength that depends on the fiber-resin system. The material cost per volume is estimated at approximately 1.5 times the cost of steel bars on the US market by Cadenazzi et al. (2019) with the expectation that it will significantly decrease as a result of an economy of scale.

The development of GFRP bars and tendons dates back to the 50s and 60s (Rubinsky & Rubinsky 1951; Wines & Hoff 1966). The fiber used was Electrical-grade Glass (E-Glass) whereas the type of resin was not reported. These early efforts were abandoned because of concerns for the durability of E-Glass exposed to the alkaline pore solution (EES, 1952). The first commercial GFRP tendon was developed in the 80s in Germany (Erki & Rizkalla 1993b) and installed in a number of field applications including several bridges prone to corrosion (Wolff & Miesseler 1993). E-Glass and unsaturated polyester were the constituents of a tendon made of 19 rods of 7.5 mm diameter each. The applied prestress was as high as 774 MPa corresponding to 51% of the nominal tensile strength. The target applications were post-tensioning and a complex anchoring system was developed to keep the load constantly applied (Erki & Rizkalla 1993a). The tendons were shipped on specific metallic coils larger than traditional wooden coils used for HSCS strands.

The application of GFRP to post-tensioning was also investigated in the US. The material system was a 7-wire strand made with E-Glass. The applied prestress was as high
as 848 MPa corresponding to approximately 63% of the ultimate tensile strength. The strands were sleeved and protected at the ends to avoid ruptures at the location of the anchors. This technology was applied in a limited number of exploratory projects including the first FRP-PC vehicular bridge built in the US in Rapid City, SD, and the pile cap of a dock pier (Iyer & Lampo 1996).

More recently, the alkali resistance of GFRP bars made with E-Glass and polyester resin was thoroughly investigated by Benmokrane et al. (2002) and, as a result, the use of polyester is no longer considered acceptable (ASTM 2017b). Furthermore, GFRP post-tensioning requires anchors able to continuously maintain the load without damaging the tendons (Erki & Rizkalla, 1993a). This problem is avoided in pre-tensioning applications because the anchor is kept in place only for a limited time.

In the 90s, an attempt was made to develop a GFRP strand for pre-tensioning applications in coastal environments (Sen et al. 1992) using Strength-grade Glass (S-Glass) and epoxy. Each strand was made of seven 2.9 mm wires and given 3.3 twists per meter for an effective area of 45 mm². A proprietary anchor system was developed to apply a pre-tension as high as 1,080 MPa, corresponding to approximately 55% of the guaranteed tensile strength. Demonstrative beams, columns and piles were cast and driven for test purposes. However, the effort was discontinued because of the poor environmental resistance of S-Glass/epoxy in alkali exposure (Sen et al. 1992).

Recently, a limited number of experimental studies have been conducted on the application of pre-tension to GFRP solid bars. PC members considered were either slabs (Fornusek et al. 2009, Singh 2014) or beams (Atutis et al. 2015, Zawam et al. 2017) with prestress applied as high as 500 MPa corresponding to approximately 42% of the nominal
capacity of a round GFRP M16 bar (Zawam et al. 2017). The GFRP bars were made with vinyl ester resin and the fiber grade was not reported, even if likely to be Electrical-grade Corrosion-Resistant Glass (ECR-Glass). ECR-Glass features mechanical properties slightly superior to E-Glass and good resistance to alkali without a significant increment in cost (OC, 2011). The alkali resistance of GFRP bars that are representative of the current manufacturing practice has been investigated by Benmokrane et al. (2017) and Ali et al. (2018). Their findings suggest that the environmental knock-down factor currently set at 0.7 for GFRP bars (ACI 2015) could be raised as high as 0.9.

The use of GFRP bars in prestressing is covered by the Canadian Highway Bridge Design Code (CSA 2014) which specifies a jacking stress at 30% of the guaranteed tensile strength. However, the use of solid bars in commercial pre-tensioning applications is limited because precast yards require prestressing reinforcement to be coilable and cuttable to length. Solid bars are not practically coilable and can be shipped at maximum lengths of around 12 meters.

A standard test method to validate the creep rupture performance of FRP bars has been established by ASTM D7337 (ASTM 2012) and the characterization of commercially available GFRP products is currently undergoing. The experimental results reported by Benmokrane et al. (2019) at approximately 50% of the ultimate tensile strength lay significantly above the limitation imposed in the past at 14% to 25% of the guaranteed tensile strength (ACI 2015, CSA 2014). Relaxation losses of a GFRP tendon for prestressing applications made with E-Glass and polyester resin were investigated by Wolff & Miesseler (1993) that reported a value of approximately 2% of the nominal strength at one-million-hour. Creep performance of GFRP bars made with E-Glass and vinylster
resin are reported by Youssef & Benmokrane (2011) that recorded creep strains ranging from 1.04 to 1.12 of the initial value after 10,000 hours of loading at stress levels ranging from 25% to 30% of the ultimate tensile strength.

Basalt FRP (BFRP) has recently been proposed as an alternative to GFRP. Basalt fibers feature mechanical properties comparable to ECR-Glass (Fiore et al. 2015) but no material standard is yet available in ASTM. BFRP bars are covered by AC454 (ICC 2016) for use as passive reinforcement in buildings with the same requirements of GFRP bars. Limited investigations in the performance of BFRP in prestressing applications have been conducted by Shi et al. (2015) who suggest a level of prestress as high as 46% of the guaranteed tensile strength. However, the properties of basalt fibers are susceptible to fluctuation as a function of the composition of the raw material (basalt rock) used and the quality control implemented during the filament production process (Fiore et al. 2015).

**Mild Prestressed Concrete with GFRP Strands**

The state-of-the-practice with prestressing reinforcement identifies durability as the limiting factor for traditional HSCS strands. Conversely, corrosion-resistant reinforcement solutions present constructability challenges and high material costs. Mild Prestressed Concrete (MPC) with GFRP strands (GFRP-MPC) is proposed as an alternative approach that may allow overcoming some of these challenges. GFRP-MPC is at a prototypical stage and the validation of its attributes and viability goes beyond the scope of this study. Also, it is meant for: a) pre-tensioning applications so that the anchor engages the strand only for a limited time and on a portion of the strand that will not contribute to the strength of the PC element once completed; and, b) PC elements that require a limited level of prestress, are exposed to aggressive environments, and are not subjected to severe cyclic fatigue
loading. These include piles, sheet piles, pile caps, and other elements of the bridge substructure. Precast and prestressed deck panels may also be an application in regions where deicing salts are used.

The fundamental feature of GFRP-MPC consists in limiting the level of prestress at about 40% of the guaranteed tensile strength of the strand. The value is set as a target required to achieve effective mild prestressing of substructure elements while guaranteeing adequate safety against failures due to pulling and creep rupture. This choice results in a number of advantages when applied to a mechanically- and cost-efficient material system:

1. Constructability.
   a. It allows to couple the strand to simpler and shorted anchors than the ones historically used for CFRP, GFRP, and AFRP (Erki & Rizkalla 1993a). Higher jacking stresses would cause failures during pulling if ad-hoc anchors are not deployed. The ability to couple FRP strands with traditional steel anchors that are available at any precast yard would play a critical role in easing constructability.
   b. It prevents the splitting of concrete at strand release as a consequence of pseudo-Poisson effect (Roddenberry et al. 2014). The splitting of concrete is a localized effect and can be restrained by adding transverse reinforcement. However, the reinforcement layout of typical PC members tends to be congested and additional reinforcement is not always an option. Furthermore: a) the confining capacity of FRP transverse reinforcement is limited by its relatively low modulus of elasticity; and, b) manufacturing of bent shapes is still a technological challenge.
   c. The relatively high guaranteed strain and the low modulus of elasticity of GFRP strands represent appealing features during pre-tensioning operations. They allow
for elongations during pulling similar to those of steel strands even under lower levels of force.

d. To be efficiently deployed in prestressing applications at the industrial scale, a tendon requires to be coilable, shippable in long lengths, and cuttable to length as needed at the precast yard. The reduced elastic modulus and the larger guaranteed strain of GFRP are expected to ease coilability.

2. Strands and PC elements performance.

a. It is envisioned that a safe value of approximately 40% of the guaranteed tensile strength will be possible in GFRP-MPC based on recent experimental results on the creep rupture performance of GFRP bars (Benmokrane et al. 2019) and experiences with similar material systems (Wolff & Miesseler 1993, Zawam et al 2017).

b. The durability of GFRP has been addressed through the adoption of ECR glass fibers and vinyl-ester resin that meet the durability requirements of ASTM D7957M (ASTM 2017b). Experimental evidence was provided by Benmokrane et al. (2017) and Ali et al. (2018) for GFRP reinforcing bars. Stress corrosion can be a concern for strands subject to sustained loads in aggressive environments. To account for the influence of environmental degradation on the creep rupture performance of FRP reinforcement, an environmental knock down factor is applied to the creep rupture strength of the material, as it is applied to its guaranteed strength for flexural strength calculations. The approach is traditional (ACI 2015) and is being confirmed in recent design guidelines for FRP material systems in prestressing application (AASHTO 2018).
c. Prestress losses are the product of concrete deformations – that are the sum of an elastic, a viscous, and a shrinkage component – multiplied by the stiffness of the tendon. With its relatively low modulus of elasticity, GFRP allows to retain a higher portion of the prestress initially applied with respect to stiffer materials like HSCS and CFRP. Similar considerations date back to the first investigations into GFRP-RC/PC (Wines & Hoff 1966) and have been confirmed ever since (Zawam et al. 2017). Relaxation – that is solely a function of the viscous properties of the reinforcement – also contributes to prestress losses but is typically the least relevant component in terms of magnitudes. Relaxation in GFRP material systems is reported to be only slightly higher with respect to CFRP and HSCS strands (Wolff & Miesseler 1993).

d. Ductile behavior is one appealing feature of traditional HSCS tendons. GFRP strands feature linear elastic behavior up to failure that occurs at guaranteed strains of approximately 2.0%. Accounting for an initial pre-tensioning at approximately 40% of the guaranteed tensile strength, the remaining strain reserve exceeds the threshold of 0.5% set by current standards for conventional PC (ACI 2014; AASTHO 2017). Thus, the deformation experienced by a GFRP-MPC element may provide enough warning before collapse occurs.

e. The twisted geometry of traditional HSCS strands ensure ideal bond to the concrete in addition to coilability. Similar geometries have been successfully used in CFRP commercial applications. Iyer & Lampo (1998) report development lengths for twisted GFRP strands to be in line with the ones of traditional HSCS strands.
Experimental Investigation

A limited amount of GFRP strands was produced to verify manufacturing capabilities and conduct an exploratory investigation. The reported material properties and mechanical performances are indicative of the potential of the technology but should not be considered representative of the quality of an industrial production. In fact, the full characterization of the material system is of secondary importance with respect to investigating the technological challenges related to pulling.

The mechanical properties of the GFRP strand prototype are compared to the ones of other material systems in Table 15. The complete list of specimens tested is reported in Table 16. A limited number of tensile tests (T) was conducted to determine the tensile strength \( f_{tu} \) of the GFRP strand. Pull tests (P) were conducted to measure the jacking strength of the GFRP strand coupled with traditional steel anchors under instantaneous \( f_{j,i} \). Pseudo-creep-rupture (CR) and pseudo-creep (C) tests were conducted on the GFRP strand coupled with traditional steel anchors under sustained stress \( f_{j,s} \) for time durations comparable to pre-tensioning operation \( t_{failure} \). Runout times are indicated with a star (*). Pseudo-relaxation (R) tests were conducted to measure the load loss following anchor setting \( f_{12h/i} \) together with the influence of various pre-tensioning procedures including the application of preload (RP) for a certain amount of time \( t_{preload} \) and the application of re-pulling (RR). A limited number of transverse shear tests (S) was conducted to measure the transverse shear strength \( \tau_{f,u} \). A flexural test (F) was conducted to measure the limit deflection at which first damaging occurs \( \delta_F \) to investigate the potential for coilability. A total of four different geometrical configurations were investigated by varying the number
of wire twists per meter from 1.25 to 4.50 that proved to have minimal influence on mechanical properties.

**Material characterization**

The GFRP strand considered in this study is made with ECR glass fibers and vinyl ester resin and is shown in Figure 4 with a commercially-available CFRP strand. The strand is composed by seven wires with a nominal diameter of 5.1 mm each. The nominal diameter of the strand is 15.2 mm while the average measured diameter is 14.5 mm with a coefficient of variation equal to 4.1% measured over 13 specimens. The average effective area of the strand is 128 mm² with a coefficient of variation equal to 3.7% measured according to ASTM D7205 subsection 11.2.5.1 (ASTM 2006) over 23 specimens. This corresponds to an average effective diameter equal to 12.8 mm. The average measured diameter of the strand is 4.6% smaller than the nominal diameter. Stresses reported in the tables are computed over the average effective area while diagrams are plotted in terms of forces and force ratios.

Tensile tests (T) were conducted according to ASTM D7205 (ASTM 2006) but over a reduced free length of 450 mm (30 diameters). The average ultimate tensile strength measured is equal to 859 MPa with a coefficient of variation equal to 1.0%. The guaranteed tensile strength, defined as the average minus three standard deviations (ACI 2015), is equal to 834 MPa. The value is slightly lower than the minimum set by ASTM D7957 (ASTM 2017b) at 844 MPa for straight M6 bars but is significantly higher than the minimum set at 653 for M16 bars. M6 bars are the closest to the diameter of the single wires, M16 bars are the closest to the diameter of the strand. The average modulus of elasticity is equal to 44.5 GPa with a coefficient of variation equal to 2.4%. The value is
slightly lower than the minimum set by ASTM D7957 (ASTM 2017b) at 44.8 GPa for straight bars of any diameter. The guaranteed strain is equal to 1.9%. Transverse shear tests (S) were conducted according to ASTM D7617 (ASTM, 2011). The average transverse shear strength measured is equal to 163 MPa with a coefficient of variation equal to 4.2%. The guaranteed transverse shear strength is equal to 142 MPa. The value is higher than the minimum set by ASTM D7957 (ASTM 2017b) at 131 MPa for straight bars of any diameter.

Shear lag is known to reduce the guaranteed tensile strength at increasing bar diameter. Lumping together smaller-size wires to create a twisted strand allows achieving M6-like performance with a 15.2 mm diameter strand. However, twisting reduces the contribution of the fibers in the longitudinal direction determining lower values of elastic modulus and tensile strength without apparently affecting transversal properties.

**Jacking strength under instantaneous load**

The test setup is consistent with ACI 440.3R subsection B.10 (ACI 2012) but with conventional steel anchors applied to both ends of the specimen (Figure 23). The free length is 900 mm (60 diameters). The goal of this experiment was to determine whether the failure of the GFRP-anchor system happens at a load level that allows sufficient stressing of the GFRP strand.

Pull tests (P) were conducted up to failure. Results are reported in Table 16. An average jacking strength under instantaneous load equal to 524 MPa was measured with a coefficient of variation equal to 3.8%. The guaranteed jacking strength under instantaneous load, computed as the average minus three standard deviations, is equal to 464 MPa corresponding to 56% of the guaranteed tensile strength of the GFRP strand. The value
meets the selected target and should be considered indicative of the potential of the technology.

Figure 24 shows the load-displacement diagrams for four strands representative of each of the geometrical configuration considered. The displacement was measured at the cross-heads of the testing frame and is inclusive of the slipping between the strand and the anchors at the two ends. A strain hardening behavior can be observed as the wedges dig into the GFRP surface and gripping improves. Figure 25 shows the statistical distribution of the results of the pull tests. Even if the data set is limited, the results align along a gaussian distribution (represented by the black line) confirming that the quantity measured is a stable property of the strand-anchor system and can be analyzed using the usual statistical assumption.

**Jacking strength under sustained load**

The strand-anchor system is required to maintain a certain level of sustained jacking load for an estimated 12 hours. The system will undergo a creep-like phenomenon with the wedges slowly digging into the GFRP strand and slowly slipping under sustained load. The target in terms of guaranteed jacking strength under a 12-hour sustained load is set at approximately 40% of the guaranteed tensile strength, a value deemed sufficient to achieve an effective prestressing design.

Using the same set-up of the instantaneous test, a total of six tests were conducted allowing to record two failure points (labeled as pseudo-creep-rupture) and four runout points (labeled as pseudo-creep) interrupted at different times. Once the pre-set load level was reached, the load was maintained constant on the strand-anchor system until failure or runout time. In field pre-casting, the strand-anchor system is subject to a load that decreases
over time as sitting losses occur; thus, the experimental setup is more demanding. Results of the pseudo-creep (C) and pseudo-creep-rupture (CR) tests are reported in Table 16. Failure points and runout points are reported in Figure 26. Times to failure and times to runout are read on the x axis. Sustained jacking forces are read on the y axis.

It is known that GFRP undergoes creep rupture when subject to sustained load for a certain amount of time (ACI 2015). The higher the load applied, the shorter the time to failure. Various formulations have been proposed for the experimental creep rupture curve that links the sustained load to the respective time to failure with logarithmic functions representing the current standard (ASTM 2012). In this research, the same approach is applied to the modelling of the pseudo-creep-rupture behavior of the strand-anchor system. An average pseudo-creep-rupture function can be defined as in Eq. 10 in terms of stresses.

\[
\frac{f_{f,js}(t)}{f_{f,ji}} = 1 - B \log_{10} \left( \frac{t}{t_0} \right)
\]

(10)

Where: \(f_{f,js}(t)\) is the average jacking strength under sustained load at the time \(t\); \(f_{f,ji}\) is the average jacking strength defined by pull testing under instantaneous load; \(t_0\) is the failure time associated to instantaneous load conditions set equal to 0.0001 hours (0.36 seconds); and, B is a non-dimensional mechanical parameter representative of damage propagation in the specific system tested (Budelmann & Rostasy 1993). The higher the value of B, the faster damages propagate, and the sooner failure occurs. Fitting the failure points reported in Table 16 gives a B equal to 0.045 with a \(R^2\) equal to 0.93.

Eq. 10 is shown as a red dotted line in Figure 26 in terms of forces. To account for the variability in jacking strength one can translate the curve down by a quantity equal to three standard deviations. The approach is in line with the work of Budelmann & Rostasy (1993).
and implies that the variability in jacking strength remains constant over time. This allows to define a guaranteed pseudo-creep-rupture function simply by writing Eq. 10 in term of guaranteed jacking strengths.

This approach allows to estimate the 12-hour guaranteed jacking strength at 345 MPa corresponding to 41% of the guaranteed tensile strength of the GFRP strand. The 12-hour guaranteed jacking strength threshold is shown as a black continuous horizontal line in Figure 26. The limit is confirmed by four runout points reported as round dots with a horizontal arrow in Figure 26. The 12-hour guaranteed jacking strength meet the target set at approximately 40%.

**Pseudo-creep behavior during jacking**

The pseudo-creep behavior of the strand-anchor system was monitored and is shown in Figure 27 for 4 specimens representatives of each of the configurations tested. The displacement is plotted as a ratio with respect to the initial value recorded at the end of the loading ramp. The behavior is consistent with an average 12-hour displacement ratio equal to 1.11 with a coefficient of variation equal to 1.2%.

The displacement shown in Figure 27 was measured at the frame cross-heads and includes the creep deformation occurring within the GFRP strand, plus the slipping occurring between the anchor and the strand. To appreciate the entity of each contribution, one of the specimens (STR #1.50-CR) was instrumented with a 100 mm extensometer located at midlength for the first half hour of testing. The results are shown in Figure 28. The extensometer is representative of the creep deformation occurring within the GFRP strand. The ratio of the loss related to GFRP creep with respect to the total displacement measured at the frame cross-heads at half hour is equal to 0.19.
The measure confirms how the creep behavior of the GFRP strand only accounts for a relatively small portion of the damage propagation that results in the pseudo-creep-rupture of the anchor-strand system. The main contribution is provided by the wedges of the anchor digging into the GFRP material and slowly slipping. Furthermore, shear lag at the anchors’ location promotes stress concentrations on the external wires that are in direct contact with the wedges and prevents the load from being uniformly distributed over the effective area of the strand. This exacerbates damage propagation on the external wires that are the ones that eventually fail. The issue is acknowledged from historical references (Erki & Rizkalla, 1993a). However, it is a necessary trade-off that allows to deploy an anchoring system that is optimal from the constructability standpoint.

Traditional steel anchors coupled with GFRP strands are not able to provide the same level of performances achieved with HSCS strands (i.e. jacking strength of approximately 70% of the ultimate tensile strength). However, they can guarantee performance that are deemed sufficient for an effective deployment in MPC applications.

**Load loss during anchor setting**

As the wedges of the anchor engage the GFRP strand setting occurs, and load loss does follow. Furthermore, some relaxation occurs within the GFRP strand. Such loss needs to be limited so that the load retained at the time of releasing (12 hours) is sufficient to meet design requirements. The phenomenon is labelled as pseudo-relaxation.

A total of 10 pseudo-relaxation tests have been conducted using different pulling procedures. The test setup is consistent with ACI 440.3R subsection B.10 (ACI 2012) but with anchors applied to both ends of the specimen. After the initial pulling, the total displacement is maintained constant on the strand-anchor system until a test duration of 12
hours is reached. The setup is representative of the actual conditions during pre-casting
when the anchor lays against the jacking frame and is prevented from moving. The imposed
initial pull is equal to 348 MPa corresponding to 42% of the guaranteed tensile strength of
the GFRP strand. The free length is equal to 900 mm.

_Pseudo-relaxation tests_

A total of four pseudo-relaxation tests (R) have been conducted. Results are reported
in Table 16. In Figure 29 the ratio of the retained pull at time $t$ over the initial value is
plotted for the four specimens tested. The behavior is consistent with an average 12-hour
loss equal to 0.20 of the initial pull with a coefficient of variation equal to 11.7%. The
coefficient of variation recorded for load loss is higher than the value recorded for pseudo-
creep displacements because the quantity compared are one order of magnitude smaller,
and the relative difference raises therefore. However, the dispersions in term of absolute
values (i.e. standard deviations) are comparable being 0.01 and 0.02 respectively.

The load losses reported in Figure 29 include a component related to the slipping
between strand and anchors that results in the elastic shortening of the strand, plus a
relaxation component that occurs within the GFRP strand. To appreciate the entity of each
contribution, one of the specimens (STR #1.50-R) was instrumented with a 100 mm
extensometer located at midlength for the first half hour of testing. The results are shown
in Figure 30. The extensometer is representative of the elastic shortening of the strand that
is directly proportional to the load loss caused by slipping. The ratio of the extensometer
measure to the total loss is equal to 0.49; thus, slipping accounts for about half of the total
loss.
The contribution of slipping is significant but not predominant as it was in the case of pseudo-creep. Furthermore, the shortening of the strand following slipping at the anchors’ location relieves the strand of some of the imposed displacement, thus reducing the severity of relaxation occurring within the strand as compared to a perfectly fixed case. The entity of the slip is constant at varying length of the strand. However, the reliving strain associated to the slip reduces at increasing length. Therefore, on longer strands, the influence of internal relaxation is expected to raise in relative terms whereas the contribution of the anchors relatively reduces. This implies that a sustained load condition imposes additional burden on the interface between wedge and strand when compared to a more realistic sustained displacement condition. Nonetheless, the behavior reported in Figure 29 is only representative of the pseudo-relaxation performances of the strand-anchor system and not of the GFRP strand in service conditions for which values of approximately 2% of the guaranteed strength at 1-million-hour are reported in literature for a similar material system (Wolff & Miesseler 1993).

The average 12-hour load loss corresponds to 8.3% of the guaranteed tensile strength of the GFRP strand and may be too high to allow effective pre-stressing. Thus, alternative pulling techniques were investigated.

**Pseudo-relaxation tests with preload**

Four specimens have been subjected to a preload equal to the initial pull for a certain period of time before being engaged in displacement control. The test is labelled pseudo-relaxation with preload (RP). The preload durations selected are 1 minute, 5 minutes, 10 minutes, and 15 minutes. Results are reported in Table 16.
Figure 31 shows the development of load losses in specimens subjected to different preload durations. Losses consistently reduce at increasing preload duration. Furthermore, Figure 32 shows how the 12-hour load loss decrease with a linear trend at increasing preload duration. The $R^2$ is 0.97. The results are aligned to the expected mechanical behavior: to maintain a sustained preload applied on the strand allows the wedges of the anchor to grip into the material and GFRP relaxation to exhaust while load losses are compensated.

The total load loss is reduced to 0.10 of the initial pull by applying a 15 minutes preload. This corresponds to 3.9% of the guaranteed tensile strength of the GFRP strand. However, to maintain a sustained preload during pre-tensioning operations may represent a challenge. Therefore, an alternative that is suitable from the constructability perspective was investigated.

_Pseudo-relaxation tests with re-pulling_

A total of two specimens have been re-pulled after having endured a 12-hour pseudo-relaxation test. The anchors are maintained in the same location. This allows for the wedges to develop optimal gripping during the first 12-hour and for the GFRP relaxation to be exhausted as well. The test is labelled pseudo-relaxation with re-pulling (RR). Results are reported in Table 16.

Results are shown in Figure 33 and compared to loads retained without preload and with a 15 minutes preload. Figure 34 reports the total retained load and the contribution of the slipping between the wedges and the GFRP strand as extrapolated from extensometer readings on the same specimen instrumented during pseudo-relaxation test (STR #1.50-RR). The contribution of slipping is equal to 45% and remains comparable to what
observed before re-pulling. This indicates that re-pulling has an equally mitigative effect on wedge slipping and GFRP relaxation.

The average 12-hour load loss after re-pulling is equal to 0.04 of the initial pull with a coefficient of variation of 25.9%. As previously observed, the coefficient of variation raises as the entity of the quantities measured reduces. However, the standard deviation is measured at 0.01 and remains comparable to what measured for pseudo-creep pseudo-relaxation.

The average 12-hour load loss corresponds to 1.6% of the guaranteed tensile strength of the GFRP strand. The value meets the original target: a strand pulled at 42% of its guaranteed strength would retain a 40% pull after setting. Repulling has been historically performed on HSCS strands before the diffusion of low-relaxation alloys. Thus, it is a procedure that precasters are familiar with.

Figure 35 shows the anchored ends of specimens that have been subject to re-pulling. The damages are localized on the surface of the GFRP strand at the wedge location and do not propagate. During construction, the end portions of the strand will be cut at releasing. Thus, their damaging is not a concern for the structural soundness of the precast element.

**Flexural behavior and coilability**

Traditional HSCS strands are coiled around wooden reels storage. This allows to overcome the length limitations imposed by straight bars that cannot be shipped at extents longer than 12 meters. To verify that the GFRP strand considered in this study can achieve similar bendability characteristics, one flexural test has been conducted on a specimen with a twist of 1.50 per meter.
The strand is tested in a three-point-bending configuration in displacement control over a free span of 610 mm (40 diameters) as shown in Figure 36. Given the composite, orthotropic, and asymmetric nature of GFRP as a material system, to determine the flexural stiffness and the flexural strength of a round bar is a challenge from both the testing and modelling perspective (Zhang et al. 2006). The twisted geometry of a GFRP strand adds complexity. However, assuming that the flexural behavior is predominant over the shearing contribution, it is possible to compute the limit curvature of the GFRP strand based on a simple deflection measure.

The deflection and curvature produced at midspan in a three-point-bending configuration under classical Euler-Bernoulli assumptions are given in Eq. 11 and Eq. 12 respectively. Where: δ is the deflection at midspan; χ is the curvature at midspan; P is the load applied at midspan; L is the free span; and, EJ is the flexural stiffness of the GFRP strand.

\[
\delta = \frac{1}{48} \frac{PL^3}{EJ} \quad (11)
\]

\[
\chi = \frac{1}{4} \frac{PL}{EJ} \quad (12)
\]

From Eq. 11 and Eq. 12 follows Eq. 13.

\[
\chi = \frac{12 \delta}{L^2} \quad (13)
\]

The limit displacement at which the specimen shows the first damaging is 41 mm. The limit radius of curvature can be computed as the inverse of the limit curvature. This
multiplied by two gives 1505 mm corresponding to the minimum diameter of the reel around which the strand can be coiled without damaging.

The smaller the diameter, the more material can be coiled on the reel, and the more efficient the shipping. Traditional HSCS can be coiled around reels of a diameter as low as 610 mm (ASTM 2017b). Coilability of the GFRP strand may be improved by reducing the friction between wires. To validate the performance of coiled strands, a tensile test after bending within the elastic limit is recommended.

**Influence of twisting per meter**

The dispersion of some of the measured quantities have been investigated at varying twisting per meter. Figure 37 shows how the effective area of the GFRP strand remains stable. This validates the use of the same effective area for stress computations across different specimens. Similarly, Figure 38 shows how the jacking strength of the GFRP strand remain stable. This supports the lumping together of measures of jacking strength and load loss performed on strands of different twisting. The twisting may have an influence on other properties of the GFRP strand including its flexural properties and its bond performance that have not been investigated in this study.

**Conclusions**

Within the context of an overview of available solutions and historic experiences in the state-of-the-practice of prestressing technologies with non-conventional tendons, this paper presents the results of an experimental investigation into GFRP strands for pretensioning applications. The study is conducted on a material system at the prototypical stage. Thus, results should be considered indicative of the potential of the technology, but
not representative of industrial grade manufacturing and quality control. Specific findings are listed below.

1. The 7-wire GFRP strand has nominal diameter of 15.2 mm with an effective area of 128 mm², a guaranteed tensile strength of 834 MPa, and an average modulus of elasticity of 44.5 GPa. The values are in line with the state-of-the-practice in GFRP bar manufacturing.

2. The pull in the GFRP strand must be limited at approximately 40% of the guaranteed tensile strength of the GFRP strand to prevent creep rupture over approximately 100 years of service life. The threshold is estimated from archival literature and previous experiences with similar material systems (Benmokrane et al. 2019, Wolff & Miesseler 1993, Zawam et al 2017).

3. The guaranteed jacking strength of the GFRP strand coupled with traditional steel anchors under instantaneous load is measured at 56% of the guaranteed tensile strength of the GFRP strand alone. The value reduces when the strand-anchor system is engaged by a sustained load during pre-casting operations.

4. The guaranteed jacking strength of the GFRP strand coupled with traditional steel anchors under a 12-hour sustained load is measured at 41% of the guaranteed tensile strength of the GFRP strand alone. The 40% target is met. Twelve hours correspond approximately to the period required for concrete to harden before releasing of the strands at the precast yard.

5. The average load losses during anchor setting ranges from 8.3% to 1.6% of the guaranteed tensile strength of the GFRP strand alone depending on the pulling technique deployed. Re-tensioning yields the lowest value of 1.6%. Furthermore, it
represents a procedure precasters are familiar with because it has been historically deployed.

6. The flexural properties of the GFRP strand have been investigated and coilability is possible arounds reels of approximately 1.505 mm diameter. Coilability is critical to allow storage and shipping of the GFRP strand.

7. Based on these findings, the following conclusions can be drawn:

8. GFRP strands can be effectively coupled with traditional steel anchors guaranteeing jacking strengths and load losses that allow for the design of elements that require low levels of prestress equal to approximately 40% of the guaranteed.

9. The ability to couple GFRP strands with traditional steel anchors will minimize impact on traditional precasting operations.

10. GFRP strands show the potential to provide a cost-effective non-corrosive alternative to traditional HSCS strands in applications that require low levels of prestress.
Table 15 – Summary of strands and tendons properties.

<table>
<thead>
<tr>
<th>Material</th>
<th>$d_n$</th>
<th>$a_f$</th>
<th>$f_{u,u}^*$</th>
<th>$\varepsilon_{f,u}^*$</th>
<th>$E_f$</th>
<th>$f_{jjs}^<em>/f_{j,u}^</em>$</th>
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<tr>
<td></td>
<td>mm</td>
<td>mm$^2$</td>
<td>MPa</td>
<td>%</td>
<td>GPa</td>
<td>-</td>
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<tr>
<td>HSCS</td>
<td>15.2</td>
<td>140</td>
<td>1725 - 1860$^\dagger$</td>
<td>3.5%$^\dagger$</td>
<td>196</td>
<td>75%$^\dagger$</td>
</tr>
<tr>
<td>HSSS</td>
<td>12.7</td>
<td>93 - 99</td>
<td>1665 - 1724$^\dagger$</td>
<td>1.1% - 1.6%$^\dagger$</td>
<td>162</td>
<td>75%$^\dagger$</td>
</tr>
<tr>
<td>CFRP</td>
<td>15.2</td>
<td>116</td>
<td>2336 - 2990</td>
<td>1.5% - 1.9%</td>
<td>155</td>
<td>70%</td>
</tr>
<tr>
<td>AFRP</td>
<td>various</td>
<td></td>
<td>1200 - 2100</td>
<td>1.5% - 3.8%</td>
<td>54 - 130</td>
<td>40-50%</td>
</tr>
<tr>
<td>BFRP</td>
<td>various</td>
<td></td>
<td>552 - 1442</td>
<td>1.2% - 2.6%</td>
<td>45 - 55</td>
<td>46%</td>
</tr>
<tr>
<td>E-GFRP</td>
<td>various</td>
<td></td>
<td>534 - 1525</td>
<td>1.2% - 3.3%</td>
<td>45 - 51</td>
<td>30-51%</td>
</tr>
<tr>
<td>S-GFRP</td>
<td>7.6</td>
<td>45</td>
<td>1987</td>
<td>2.8%</td>
<td>70</td>
<td>56%</td>
</tr>
<tr>
<td>ECR-GFRP</td>
<td>15.2</td>
<td>128</td>
<td>834</td>
<td>1.9%</td>
<td>44.5</td>
<td>41%</td>
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</table>

$^\dagger$ Ultimate values are reported for HSCS and HSSS in place of the guaranteed.
Table 16 – List of all specimen tested. (*) indicates runout specimen.

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<th>SPECIMEN ID</th>
<th>( f_{f,u} )</th>
<th>( f_{f,ji} )</th>
<th>( f_{f,js} )</th>
<th>( t_{failure} )</th>
<th>( f_{12h} / f_i )</th>
<th>( \tau_{f,u} )</th>
<th>( \delta_F )</th>
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<td>-</td>
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<td>-</td>
</tr>
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<td>-</td>
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<td>-</td>
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</tr>
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Figure 20 – Steel anchors and wedges for steel pre-tensioning. (Rossini & Nanni 2019)

Figure 21 – Coupled CFRP and HSCS strands ready for pulling. (Rossini & Nanni 2019)
Figure 22 – Transverse cracks in concrete after CFRP strands releasing. (Rossini & Nanni 2019)
Figure 23 – GFRP strand ready for pull test (a) with details of steel anchors (b) (c). (Rossini & Nanni 2019)

Figure 24 – Pull test results for representative specimens. (Rossini & Nanni 2019)
Figure 25 – Statistical distribution of pull test results. (Rossini & Nanni 2019)

Figure 26 – Average and guaranteed pseudo-creep-rupture curves for the GFRP strand-anchor system. (Rossini & Nanni 2019)
Figure 27 – Total displacement under sustained load. (Rossini & Nanni 2019)

Figure 28 – Total displacement (frame cross-heads) and creep contribution (extensometer) under sustained load. (Rossini & Nanni 2019)
Figure 29 – Retained load under sustained displacement. (Rossini & Nanni 2019)

Figure 30 – Total retained load (load cell) and slipping contribution (extensometer) under sustained displacement. (Rossini & Nanni 2019)
Figure 31 – Retained load under sustained displacement after preload. (Rossini & Nanni 2019)

Figure 32 – Load loss under sustained displacement as a function of preload duration. (Rossini & Nanni 2019)
Figure 33 – Retained load under sustained displacement after re-pulling. (Rossini & Nanni 2019)

Figure 34 – Total retained load (load cell) and slipping contribution (extensometer) under sustained displacement after re-pulling. (Rossini & Nanni 2019)
Figure 35 – GFRP strand after re-pulling. (Rossini & Nanni 2019)

Figure 36 – Experimental setup for flexural test. (Rossini & Nanni 2019)
Figure 37 – Cross-sectional area at varying twisting per meter. (Rossini & Nanni 2019)

Figure 38 – Jacking strength at varying twisting per meter. (Rossini & Nanni 2019)
Chapter 4

Creep Rupture of GFRP

The experimental portion of this study presents the results of an investigation into the unconditioned creep rupture strength of two different types of 12.7 mm GFRP bar both made with vinylester resin and ECR glass fibers but with different surface enhancements. An overview of the behavior and performance of FRP composites and bars under sustained load is discussed to determine whether a statistically-based empirical approach represents a viable method for the definition of a safe value of creep rupture strength. A refined approach to data handling and the extrapolation of a safe value of creep rupture strength is discussed, including statistical considerations. Results show that the creep rupture knock-down factor prescribed by current design guidelines is conservative when designing using the GFRP bars considered in this study.

Existing guidelines and specifications provide practitioners with the tools they need for the design and construction of FRP-RC structures (ACI 2004, ACI 2015, AASHTO 2018a, AASHTO 2018b, CSA 2012, CSA 2014, fib 2013, Minstroy 2018). Guidelines are periodically updated to reflect advancements in the state-of-the-art and allow for more efficient design where possible. The creep rupture strength of GFRP bars under sustained
load was identified as a limiting factor in some applications (Rossini et al. 2018a) designed according to the first edition of the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings* (AASHTO 2009) and ACI 440.1R (ACI 2015). An investigation into available experimental results allowed for an increment of the exploitable capacity under sustained load from 20% to 30% of the guaranteed strength, reduced by an environmental knock-down factor (Benmokrane et al. 2019). The new coefficient was adopted in the second edition of *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete* (AASHTO 2018b) and, along with a rationalization of the load demands, allowed for a more efficient design of certain bridge elements (Rossini et al. 2018c).

Recent studies into the creep rupture strength of GFRP bars (Benmokrane et al. 2019) capture the improved performance of material systems manufactured according to current material specifications issued by ASTM (ASTM 2017b) and benefit from the adoption of a standardized test method (ASTM 2012b). However, the procedures and safety factors adopted to analyze creep rupture data and extrapolate design parameters remain unchanged since the first edition of ACI 440.1R (ACI 2001) and do not provide a clear indication of the associated safety level.

Investigation into the creep rupture behavior of GFRP bars remains a priority. This includes the addition of new experimental results to the available database and the definition of refined procedures for data analysis and the extrapolation of design parameter. Further advancement in this area would lead to more efficient design of GFRP-RC structures, and may allow for the deployment of GFRP reinforcement in prestressing applications (Rossini et al. 2019b).
State-of-the-art on Creep and Creep Rupture

This section provides an overview of the behavior of FRP bars and composites subject to sustained load. The complex nature of the phenomenon and the inherent uncertainties support the adoption of a statistically-based empirical approach for modelling and data interpretation.

Fundamentals and definitions

Structural materials under sustained load or displacement exhibit time-delayed viscous responses (Findley et al. 1989). Creep deformation gathers in a material subject to sustained load over time. Conversely, stress relaxation occurs in a material subject to imposed displacement over time. Viscous phenomena gain intensity at increasing temperature. Typically, materials start to exhibit significant viscous behavior at temperatures exceeding 40% of their melting temperature or glass transition temperature (fib 2007). Viscous phenomena couples at the micro-scale with damage propagation and progressive cracks opening under sustained load that is referred to as static fatigue (Harris 2003). Static fatigue can be aggravated by aggressive environments where corrosion under sustained load, or stress corrosion, may accelerate crack propagation (Raja & Shoji 2011). The accumulation of creep deformation and progressive crack opening may result in an elapsed collapse under sustained load that is referred to as creep rupture and occurs after a time period called endurance time (fib 2007, ACI 2015). The maximum level of sustained load such that creep rupture does not occur during the service life of a material or structure is defined creep rupture strength. Some materials feature a creep rupture endurance limit, defined as a load level below which a specimen can withstand sustained load indefinitely (Carley 1993). The
existence of an endurance limit for composite materials was speculated but not fully investigated (Harris 2003) and is currently not accepted by standards (JSCE 1997, ASTM 2012b). The service life of FRP-RC/PC structures is usually set at a minimum of 100 years (Cadenazzi et al. 2019) and the creep rupture strength is typically computed at an endurance time of 114 years corresponding to $10^6$ hours for convenience (JSCE 1997, fib 2007, ASTM 2012b). In many structural applications creep rupture and relaxation are coupled: a certain degree of relaxation or other load losses occur within the material, relieving some of the sustained load and delaying the occurrence of creep rupture (Jansen & Nemec 2014).

Viscous phenomena and creep rupture affect any structural material to some degree and must be accounted for during design in some instances. These include the creep and creep rupture of metals at high temperatures (French et al. 1928, Gaibler 1952, Shrestha et al. 2013) and room temperatures (Liu et al. 2001), the relaxation of prestressing steel strands in concrete members (Ghali & Trevino 1985), the stress corrosion of carbon steel and stainless steel in aggressive environments (Li et al. 2016, Fernandez et al 2013), the viscous behavior of concrete and masonry under sustained load (Bazant & Wittmann 1982, Taliercio 2009), and the accumulation of creep deflections in structural elements made of timber (Brokans & Ozola 2014).

**Behavior of fibers and resins**

Fiber Reinforced Polymers (FRPs) are composite materials made from continuous fibers, typically glass, basalt, carbon, or aramid. Fibers are impregnated with a polymeric thermosetting resin, typically vinyl ester or epoxy (Rossini et al. 2019a). The behavior of FRP under sustained load is a function of fiber properties, resin properties, and of the interaction between the two materials. In addition to the choice of materials, the sizing of
the fibers and their surface preparation play a role in defining the contact surface and guaranteeing mechanical bond between fibers and resin; furthermore, the surface preparation of the fibers affects their protection from aggressive chemicals (Jones 2003).

Carbon fibers feature negligible creep deformation (Huang 2009), do not suffer from static fatigue, and are inert to all but highly oxidizing acids (Jones 2003). Glass fibers feature negligible creep deformation (Franke & Meyer 1992); however, they experience static fatigue that can be worsened by stress corrosion in moist and alkaline environments (Wiederhorn & Boltz 1969, Jones 2003, ACI 2007). Electrical-grade Glass (E-Glass) fibers are widely used in industrial applications thanks to their relatively low cost and good mechanical properties; however, they may be susceptible to stress corrosion in moist and alkaline environment. Therefore, Electrical-grade Corrosion-Resistant Glass (ECR-Glass) fibers are preferred in civil engineering applications thanks to their superior durability (Gooranorimi & Nanni 2017) and their ability to meet the prestational requirements of current material standards for FRP bars (ASTM 2017) without a significant increment in material cost (OC 2011). Basalt fibers have been recently proposed as an alternative to glass fibers. They feature mechanical properties in line with E-Glass; however, their durability and long-term performance are still under investigation (Fiore et al. 2015). Aramid fibers feature creep deformations in reason of their polymeric nature (Giannopoulos & Burgoyne 2012); furthermore, they are susceptible to static fatigue that is aggravated by stress corrosion and hydrolysis in moist and alkaline environments (Jones 2003). Therefore, the use of Aramid FRP (AFRP) is limited to dry conditions (CSA 2014).

Thermosetting resin have been traditionally used in manufacturing FRP composites in reason of their good creep performance and stability at relatively high temperatures (ACI
2007, *fib* 2007). Recently, thermoplastic resins have been developed that may feature comparable performance and be adequate for FRP manufacturing (Sayed Ahmed et al. 2017, Nanni et al. 2019). Nevertheless, the current practice prefers the use of vinyl ester or epoxy in permanent applications in civil engineering (ASTM 2017). Other thermosetting resins like polyester proved not able to guarantee adequate durability in moist and alkaline environments (Benmokrane et al. 2002, Benmokrane et al. 2017).

**Behavior of FRP composites**

Whereas creep deformation in the fibers is typically negligible, resins do creep at ambient temperature (ACI 2007, *fib* 2007). When a unidirectional FRP composite is axially loaded, both resin and fibers are engaged in reason of their relative stiffness. Following creep deformations, the load transitions from the resin to the fibers (*fib* 2007). The accumulation of creep deformation in an FRP unidirectional composite is shown in Figure 39. It develops asymptotically with most of the deformation occurring during the first stage. If the applied load is relatively low, the fibers can withstand the increased demand after load transition without developing static fatigue, and the second stage may extend indefinitely as represented by the long-dashed line. Otherwise, when the sustained stress is too high, crack propagation within the fibers reach a critical threshold and becomes unstable leading the composite to failure in the third stage (ACI 2004, *fib* 2007) as represented by the short-dashed line. There is another possibility in which the applied load is so high to bring the material system to failure within the first stage as represented by the continuous line.

Creep rupture in unidirectional FRP composites is governed respectively by creep deformations in the resin and static fatigue developing within the fibers. However,
secondary factors play a role in determining the creep rupture behavior of FRP bars. These include: the accumulation of creep deformations in some types of fiber (Giannopoulos & Burgoyne 2012); the inherent stress relaxation occurring in some applications (Jansen & Nemec 2014); the straightening of fibers under sustained load (Shi et al. 2015); and, the progressive cracking and damaging of the resin because of static fatigue and delamination at the interface between fibers and resin, that is aggravated by stress corrosion in moist and alkaline environments (Benmokrane et al. 2002, Benmokrane et al. 2017). Other factors such as high temperature, exposure to UV radiation, wet and dry cycles, and freeze-thaw cycles may also decrease the creep strength of an FRP composite (fib 2007). In general, the damaging of composite materials under sustained load is diffused and it is not possible to identify and observe the growth of a dominant crack (Hahn & Kim 1975).

**Guidelines and standards**

Given their complex behavior, simplifying assumptions must be introduced to model FRP composites and bars for design purposes. The approach adopted by ACI 440.1R (2015) and AASHTO (2018a), decouples environmental effects from the definition of a creep rupture strength by introducing two separate knock-down factors that are then applied together to the Guaranteed Tensile Strength (GTS) of the material system. The GTS \( f_{tu}^* \) is defined as the average value of the Ultimate Tensile Strength (UTS) minus three standard deviations (ACI 2015), a design strength \( f_{td} \) is defined by applying an environmental knock-down factor \( C_{E} \) to the GTS as shown in Eq. 14. The value of \( C_{E} \) is given in design guidelines and is based on experimental studies and conditioned tests performed according to ASTM D7705 (ASTM 2012a).
The unconditioned creep rupture strength \( f_{c,ud} \) of the material system can be defined as:

\[
f_{c,ud} = C_{c}f_{fu}^*
\]  

(14)

performing unconditioned tests according to ASTM D7337 (ASTM 2012b) and a creep rupture knock-down factor \( C_{C} \) can be defined as shown in Eq. 15.

\[
f_{c} = C_{c}f_{fu}^*
\]  

(15)

For design purposes, the strength of the material under sustained load in exposed conditions \( f_{s,s} \) can be defined by applying the environmental knock-down factor \( C_{E} \) and the unconditioned creep rupture knock-down factor \( C_{C} \) to the GTS as shown in Eq. 16.

\[
f_{s,s} = C_{E}C_{c}f_{fu}^*
\]  

(16)

The values of \( C_{E} \) and \( C_{C} \) recommended by various design guidelines are reported in Table 17 for Carbon FRP (CFRP), Glass FRP (GFRP), Aramid FRP (AFRP), and Basalt FRP (BFRP) bars and strands for RC and PC structures. Even though test methods are standardized (ASTM 2012a, ASTM 2012b), unified procedures for the elaboration of experimental results and the definition of knock-down factors are not available yet.

**Performance of FRP bars and strands**

Design coefficients agree with experimental results in showing that composites reinforced with more performing fibers under sustained load show higher creep rupture strength (Yamaguchi et al. 1997). Furthermore, at equal fiber content and sustained load, a less stiff fiber like glass will engage a smaller fraction of the total load with respect to a stiffer fiber like carbon. Therefore, the resin will carry a larger fraction of the initial load.
in GFRP composites rather than CFRP and experience more relevant creep strains. This consideration further justifies the better creep rupture performance of composites reinforced with stiffer fibers (Yamaguchi et al. 1997) or characterized by higher fiber content as long as adequate impregnation is provided (Hugo et al. 1993).

CFRP strands are typically applied in demanding prestressing applications where their relatively high material cost is offset by their superior performance. They feature a GTS as high as 2,990 MPa that reduces at increasing cross sectional dimensions because of shear lag, and an elastic modulus of approximately 155 GPa. Their unconditioned creep rupture strength is set at 65% GTS and their excellent durability is reflected in a knock-down environmental factor as high as 1 (AASHTO 2018b). GFRP bars are the current standard in passive reinforcement applications in reason of their good mechanical properties, good durability, and relative cost-effectiveness. They feature a GTS as high as 1,525 MPa that reduces at increasing cross sectional dimensions because of shear lag and a minimum elastic modulus of 45 GPa. Their unconditioned creep rupture strength is currently limited at 30% GTS and the environmental factor is set at 0.7 (AASHTO 2018a). However, recent research suggests raising the environmental knock-down factor as high as 0.9 (Ali et al. 2018) whereas experimental results reported in the experimental portion of this study support a relaxation of the creep rupture knock-down factor. The mechanical properties of BFRP bars align to those of GFRP bars; however, more research is needed to address their long-term performance. AFRP strands have mechanical and creep rupture properties that lay in-between GFRP and CFRP material systems; however, they are sensitive to alkali degradation. A state-of-the-practice on FRP reinforcement is discussed by Rossini & Nanni (2019).
Table 18 reports the mean experimental creep rupture strength ratio at $10^6$ hours (114 years) based on testing of GFRP bars performed by various authors. More recent studies benefit from the standardization of material system (ASTM 2017b) and test method (ASTM 2012b) and provide more consistent results with respect to early investigations. Benmokrane et al. (2019) report 204 creep rupture points collected through testing of 11 different types of vinylester E/ECR-GFRP bars from various manufacturers. The authors extrapolate a mean creep rupture strength equal to 51% UTS at $10^6$ hours (114 years) that can be considered representative of state-of-the-practice GFRP bars. However, the safe value for design purposes is set at only 30% GTS applying the same safety factors adopted since the first edition of ACI 440.1R (ACI 2001). The development of a refined procedure may allow a rationalization of design coefficients.

**Experimental Investigation**

In the experimental portion of this study, two types of GFRP bars were tested to evaluate their creep rupture strength. The two types of bars have the same nominal diameter of 12.7 mm and are both made of ECR glass fibers and vinylester resin. The first bar type is manufactured through the pultrusion process to produce a constant cross section that is later carved to produce ribs that will enhance the mechanical bond between the bar and the surrounding concrete. This bar type is named M13(1) in this study and is shown in Figure 40a. The second type of bar is also produced through the pultrusion process; the bar is then winded with a helical wrap of fibers at approximately 25 mm spacing to enhance the bond between the bar and the concrete. This bar type is named M13(2) in this study and is shown in Figure 40b.
Material characterization

The first stage of the experimental program involved conducting quasi-static tensile tests on the two types of bars in order to establish their ultimate tensile strengths and elastic modulus. Twenty five (25) specimens from bar type M13(1) and twenty four (24) specimens from bar type M13(2) were tested in tension according to ASTM D7205 (ASTM 2006). Area measures were performed according to ASTM D7205 subsection 11.2.5.1 (ASTM 2006). The geometry and the mechanical properties of the two types of GFRP bars tested are shown in Table 19. The tensile strength results for type M13(1) bars and type M13(2) bars are shown in Table 20 and Table 21, respectively.

Bar type M13(1) has a nominal diameter \(d_n\) of 12.7 mm, a nominal area \(a_n\) of 129 mm\(^2\) and an average effective area \(a_f\) of 146 mm\(^2\) with a coefficient of variation equal to 1.1%. It has an average UTS \(F_{fu,m}\) of 144 kN with a coefficient of variation equal to 4.1%, a GTS \(F_{fu,*}\) of 126 kN, and an average elastic modulus \(E_f\) of 56 GPa computed on the effective area. Bar type M13(2) has a nominal diameter \(d_n\) of 12.7 mm, a nominal area \(a_n\) of 129 mm\(^2\), and an average effective area \(a_f\) of 149 mm\(^2\) with a coefficient of variation equal to 1.1%. It has an average UTS \(F_{fu,m}\) of 165 kN with a coefficient of variation equal to 4.9%, a GTS \(F_{fu,*}\) of 140 kN, and an average elastic modulus \(E_f\) of 57 GPa computed on the effective area.

The two bar types have similar geometries with a 2% difference on the effective area, and the same basic constituents in terms of fibers and resin; however, M13(2) shows an average UTS 15% higher than M13(1). This can be explained considering the different surface preparations. The grooves in the surface of M13(1) cut through the external layers of fibers preventing them from providing mechanical contribution. Conversely, the helical
wrapping in M13(2) only deviates some of the external fibers allowing for some contribution in the longitudinal direction. The difference cannot be appreciated performing area measurements but has a contribution on the tensile strength of the two types of bar.

Creep rupture tests

The second stage of the experimental program entailed conducting creep rupture tests on the two types of bars. Tests were performed following the principles of ASTM D7337 (ASTM 2012b).

Specimen preparation

Figure 41 shows a schematic view of the dimensions and details of a typical creep rupture specimen with the anchoring system needed for connection to the testing frame. Each bar was potted into an epoxy-filled steel tube at each end to serve as an anchor during testing. The steel tubes were 500 mm in length with an outside diameter of 33.4 mm and a wall thickness of 3.4 mm. A threaded steel rod was welded to the steel tube at one end of the specimen for anchoring to the test frame by means of a nut. Two parallel plates were welded to the other end. The plates had holes at their ends to insert a steel chain which was then connected to the loading system, as shown on the right-end-side of Figure 42.

Test frame

A customized test frame was designed and manufactured for the purpose of loading and sustaining the load on the tested bars until creep rupture occurred. Figure 42 shows a schematic view of the testing frame and Figure 43 shows a general view of the frame and the specimens under load. The frame resembles a prestressing bed with two parallel
abutments. The left-end side of the specimen in Figure 42 is fixed to the frame using a nut resting against a 25.4 mm steel plate. Conversely, on the right-end side, load was applied using concrete blocks. An adjustable loading system was designed to magnify the weight of the concrete blocks by a factor of 100 using a turn-buckle and a system of levers as shown in Figure 44. The arrangement was alternated on both sides of the frame to save space, as shown in Figure 43.

**Loading protocol**

A hollow hydraulic jack and a load cell were used to load each specimen to the required level. Load was applied on the threaded bar side while the other side of the specimen was engaged by the adjustable loading system. Load was monitored using a display unit until the load cell indicated reaching the desired level. Then, while monitoring the load and keeping it constant, the strain in the steel tubes was recorded using a strain gage which was attached for that purpose as shown on the left-end-side of Figure 41. The recorded strain corresponds to the desired level of load. While the hydraulic jack is engaged, the nut at the end of the trenched bar is turned until it bears against the steel plate to prevent movement. Then, the load in the jack is slowly released thus leaving the bar resting against the frame. Due to some inevitable sitting of the anchoring system, the load in the bar relaxes and the strain in the steel tubes slightly drops. To compensate for this relaxation, the turnbuckle in the adjustable loading system is turned until the strain in the steel tubes matches the desired value. The strain gage was periodically monitored on each specimen and the load level was adjusted to compensate for sitting losses. The time-to-failure was recorded for each specimen.
Test results

Forty (40) specimens for bar type M13(1) and twenty six (26) specimens for bar type M13(2) were tested. Of these, only twenty four (24) specimens from bar type M13(1) reached creep rupture whereas the others were terminated after approximately 25000 hours (1041 days) before failure occurred (runout). Creep rupture points were collected on both bar types over six different load levels ranging from 89% UTS to 73% UTS. Runout points were collected on bar type M13(1) at load levels ranging from 78% UTS to 63% UTS. No runout points were collected on bar type M13(2) because only relatively high sustained loads were deliberately imposed. Results are reported in Table 22 for bar type M13(1) and Table 23 for bar type M13(2).

Data Analysis

In this section, the analysis of creep rupture test results is discussed. The procedure includes statistical considerations and allows to define a safe value of creep rupture strength that can be used for design purposes. The procedure is first outlined for clarity and then applied to the data set collected in the experimental portion of this study.

Mean creep rupture curve

The creep rupture strength of GFRP bars decreases logarithmically at increasing endurance time as reported by various authors (Seki et al. 1997, Yamaguchi et al. 1997, Keller et al. 2017, Benmokrane et al. 2019). Creep rupture points can be plotted in a logarithmic diagram where the x-axis reports the logarithmic time-to-failure and the y-axis reports the level of sustained load as shown in Figure 45. A mean creep rupture curve can be defined as reported in Eq. 17 and represented by a dotted line in Figure 45.
\[
\frac{F_{fc,m}}{F_{fu,m}} = 1 - B_m \log_{10}\left(\frac{t}{t_0}\right)
\]  

(17)

Where: \(F_{fc,m}\) is the mean creep rupture strength at endurance time \(t\), \(F_{fu,m}\) is the average UTS at time zero \((t_0)\) equal to 0.0001 hours, and \(B_m\) is an adimensional regression parameter equal to 0.031 for bar type M13(1) and 0.035 for bar type M13(2). For consistency with tensile test results, when \(t\) equals \(t_0\), the mean creep rupture strength must equal the average UTS of the material system as shown in Figure 45. A quasi-static tensile test has a total duration of approximately 10 minutes (ASTM 2006); however, the maximum load is maintained on the specimen only for one instant, corresponding to the final step in the load ramp. Therefore, a quasi-static tensile test can be seen as a case-limit creep rupture test with a load duration \(t_0\) equal to a fraction of a second.

**Statistical distributions**

Both tensile test results and creep rupture test results are statistically variable. In tensile tests, the load duration is set, and results are scattered along the load axis. Conversely, in creep rupture tests the sustained load is set, and rupture points are scattered along the time axis. These variabilities are represented by the two bell-shaped frequency distributions shown in Figure 45. Under simplifying assumptions, the statistical distribution of creep rupture points along the time axis is proportional to the statistical distribution of tensile test results along the load axis (Franke & Meyer 1992). This holds true if the creep rupture of a GFRP bar is solely governed by static fatigue developing within glass fibers with negligible accumulation of creep strains (Franke & Meyer 1992). This stance is challenged by experimental results showing creep strains as high as 12% of the initial value after \(10^4\)
hours (417 days) under a sustained load ranging from 25% to 30% of UTS (Youssef & Benmokrane 2011, Can et al. 2017). Furthermore, because of the complex nature of the phenomenon, creep rupture tests show an inherent variability that is not present in tensile tests (Budelmann & Rostasy 1993). Therefore, the two statistical distributions must be treated separately to account for the higher degree of variability in creep rupture test results.

Table 19 summarizes the statistics of the tensile test results reported in Table 20 and Table 21. The results align along a normal distribution with a correlation coefficient of 0.98 for both bar types M13(1) and M13(2), as shown in Figure 46. Alignment of tensile test results on GFRP bars to a normal distribution is confirmed by various authors (He & Qiu 2010, Ribeiro & Diniz 2013, Nasrollahzadeh & Aghamohammadi 2018) and is in line with traditional assumption on the statistical distribution of the experimental strength of construction materials (CEN 2002, CEN 2004). The characteristic tensile strength ($F_{u,k}$) reported in Table 19 can be computed using tabled values for the normal distribution. It represents the 95th strength percentiles, meaning that 95% of the tested specimen showed a higher strength.

Table 24 and Table 25 report the statistics of the times-to-failure recorded at each load level for bar type M13(1) and M13(2) respectively. The coefficient of variation ranges from 60% to 195% averaging at approximately 100% or two order of magnitudes higher than the values of 4.1% and 4.9% recorded on tensile tests. Similar variability can be appreciated in studies reporting multiple creep rupture points at the same load level (Budelmann & Rostasy 1993, Seki et al. 1997, Benmokrane et al. 2019). Chiao et al. (1974) also report coefficients of variation equal to approximately 100%. The normal distribution is not suited for modelling such a dispersion, whereas the Weibull distribution provide a good matching
with a correlation coefficient of 0.92 for bar type M13(1) and 0.99 for bar type M13(2) as shown in Figure 47. Alignment of times-to-failure in creep rupture tests to a Weibull distribution is confirmed by various authors (Chiao et al. 1974, Hahn & Kim 1975, Yamaguchi et al. 1997). The characteristic times-to-failure ($t_{\text{failure},k}$) reported in Table 24 and Table 25 can be computed using tabled values for the Weibull distribution. It represents the 95\textsuperscript{th} time-to-failure percentile, meaning that 95\% of the specimens failed later. Average times-to-failures ($t_{\text{failure},m}$) are also reported.

**Statistical testing**

To validate the models selected to describe the stochastic variability of tensile strengths and times-to-failure, statistical testing is performed. In the specific context considered, statistical testing allows a user to verify whether a model is accurate at a certain significance level. The higher the significance level, the closer the model to the actual behavior of the empirical sample. Testing of the four statistical sample at hand is performed applying both the goodness-of-fit test and the Kolmogorov-Smirnoff test.

According to the goodness-of-fit test, the tensile strengths of bar type M13(1) and M13(2) can be modelled using a normal distribution at a significance level above 10\%. The discrete cumulative distribution function plotted over four intervals and compared to an ideal normal distribution for the tensile strengths of bar type M13(1) and M13(2) is shown in Figure 48 and Figure 49. Similarly, the Kolmogorov-Smirnoff test yields a maximum error of 0.081 for bar type M13(1) and 0.115 for bar type M13(2) corresponding to a significance level above 20\%. A significance level of 5\% is typically considered a threshold for acceptance. Therefore, tensile strengths can be modelled using normal distributions for the purposes of this study.
According to the goodness-of-fit test, the times-to-failure of bar type M13(1) and M13(2) can be modelled using a Weibull distribution at a significance level above 10%. The discrete cumulative distribution function plotted over six intervals and compared to an ideal Weibull distribution for the times-to-failure of bar type M13(1) and M13(2) is shown in Figure 50 and Figure 51. Similarly, the Kolmogorov-Smirnoff test yields a maximum error of 0.192 for bar type M13(1) and 0.128 for bar type M13(2) corresponding to a significance level above 20%. A significance level of 5% is typically considered a threshold for acceptance. Therefore, times-to-failure can be modelled using Weibull distributions for the purposes of this study.

**Characteristic and guaranteed creep rupture curves**

A characteristic creep rupture curve can be traced interpolating characteristic creep rupture points reported in Table 24 and Table 25 instead of plain experimental values reported in Table 22 and Table 23. The curve is defined in Eq. 18 and reported in Figure 45 as a dashed line.

\[
\frac{F_{fc,k}}{F_{fu,k}} = 1 - B_k \log_{10} \left( \frac{t}{t_0} \right)
\]

Where \( F_{fc,k} \) is the characteristic creep rupture strength at endurance time \( t \). For consistency with tensile test results, when \( t \) equals \( t_0 \), the characteristic creep rupture curve must intersect the characteristic tensile strength (\( F_{fu,k} \)) of the material system as shown in Figure 45. The regression parameter (\( B_k \)) equals 0.047 for bar type M13(1) and 0.046 for bar type M13(2). The characteristic curve is translated downwards and rotated leftwards with respect to the mean curve to account for variabilities along the time and load axis.
The characteristic creep rupture curve represents a lower-bound propriety of the material but does not define a safe threshold for design purposes yet. To that aim, further conservatism can be introduced following the design-assisted-by-testing procedure detailed by EN 1990 (CEN 2002). According to the guideline, the 99.9th strength percentile can be used as a safe value for design purposes with only a 0.001 probability of the material not meeting the target. Under the assumption of normal strength distribution, a safe value for design purposes can be computed applying the safety factor defined in Eq. 19 to the characteristic value.

\[
\frac{1}{\gamma} = \left(1 - 3.04 \text{COV}\right) F_{\text{fu,m}} = \frac{F_{\text{fu}}^{*}}{F_{\text{fu,k}}}
\]  

(19)

The quantity at the numerator corresponds to the definition of a guaranteed strength according to ACI 440.1R (ACI 2015) with a negligible difference of 0.04 standard deviations. Therefore, a guaranteed creep rupture curve can be defined by multiplying Eq. 18 by the safety factor defined in Eq. 19 and simplifying as reported in Eq. 20.

\[
\frac{F_{\text{fc}}^{*}}{F_{\text{fu}}} = 1 - B_k \log_{10}\left(\frac{t}{t_0}\right)
\]  

(20)

Where \(F_{\text{fc}}^{*}\) is the guaranteed creep rupture strength at endurance time \(t\). For consistency with tensile test results, when \(t\) equals \(t_0\), the guaranteed creep rupture curve must intersect the guaranteed tensile strength \(F_{\text{fu}}^{*}\) of the material system as shown in Figure 45. The guaranteed curve is translated downwards with respect to the characteristic curve to account for the additional conservatism required on the load axis.
Discussion of test results

The procedure discussed beforehand was applied to the results collected on bar type M13(1) as shown in Figure 52 and bar type M13(2) as show in Figure 53. Note that the scale on the right-end-side of the diagrams is a function of the ratio between UTS and GTS that is equal to 1.14 for bar type M13(1) and 1.17 for bar type M13(2). Runout points were not considered in the analysis and are only reported for reference.

The average, characteristic, and guaranteed unconditioned creep rupture strengths at an endurance time of $10^6$ hours (114 years) were extrapolated. Bar type M13(1) has a mean creep rupture strength ($F_{Fc,m}$) equal to 69% UTS, a characteristic creep rupture strength ($F_{Fc,k}$) equal to 46% UTS, and a guaranteed creep rupture strength ($F_{Fc,*}$) equal to 41% UTS. Bar type M13(2) has a mean creep rupture strength ($F_{Fc,m}$) equal to 65% UTS, a characteristic creep rupture strength ($F_{Fc,k}$) equal to 46% UTS, and a guaranteed creep rupture strength ($F_{Fc,*}$) equal to 39% UTS. For design purposes, the creep rupture strength is expressed as a function of the GTS of the bar applying the creep rupture knock-down factor ($C_C$) defined in Eq. 15. As reported on the right-end side of Figure 52 and Figure 53, for both bar types M13(1) and M13(2) the guaranteed creep rupture strength ($F_{Fc,*}$) equals 46% GTS. Therefore, a creep rupture knock-down factor ($C_C$) equal to 0.46 can be used when designing using the GFRP bars tested in this study. The value is approximately 50% higher than the 0.30 coefficient currently recommended by AASHTO (2018a).

The surface preparation of the two types of bar have no additional detrimental effect on their creep rupture performance. This is because the creep rupture strength is defined as a function of the tensile properties of the bars that are already affected by their different surface preparations.
Conclusions

This paper presents the results of an experimental investigation into the unconditioned creep rupture strength of GFRP bars made with vinylester resin and ECR glass fibers. A refined approach to data handling and the extrapolation of a safe value for design purposes is discussed. Results show that the creep rupture knock-down factor prescribed by current design guidelines is conservative when designing using the GFRP bars considered in this study. Specific findings are listed below.

1. An overview of the behavior and performance of composite materials and bars under sustained load is discussed. The complex nature of the viscous and fatigue phenomena developing within the constituents and the composite, and the inherent variabilities in the material properties, support the choice of a statistically-based empirical approach for the definition of a safe value of creep rupture strength.

2. Two different types of bar named M13(1) and M13(2) are considered in this study. M13(1) has a carved surface whereas M13(2) has a winded surface with a helical wrap. The two types of bar have approximately the same effective area with a difference of 2%. However, M13(2) shows an average tensile strength that is 15% higher with respect to M13(1). This may be caused by a negative influence of carving as opposed to winding on tensile strength. No additional detrimental effects are noticed on creep rupture properties as long as the creep rupture strength is expressed as a function of the tensile strength.

3. Tensile strength results statistically align to a normal distribution with a coefficient of correlation of 0.98 for both bar type M13(1) and M13(2). Times-to-failure align to
Weibull distributions with a correlation coefficient of 0.92 for bar type M13(1) and 0.99 for bar type M13(2).

4. Creep rupture points can be plotted in a logarithmic diagram and can be interpolated by a mean creep rupture curve. Accounting for the variability along both the time and load axis, a characteristic creep rupture curve can be defined. Introducing additional conservatism, a guaranteed creep rupture curve associated to the 99.9th strength percentile can be defined, and a guaranteed creep rupture strength can be extrapolated at an endurance time of 106 hours (114 years). The ratio of the guaranteed creep rupture strength to the guaranteed tensile strength defines the unconditioned creep rupture knock-down factor.

5. Bar type M13(1) has a mean creep rupture strength equal to 69% UTS, and a guaranteed creep rupture strength equal to 41% UTS (46% GTS). Bar type M13(2) has a mean creep rupture strength equal to 65% UTS, and a guaranteed creep rupture strength equal to 39% UTS (46% GTS).

6. A creep rupture knock-down factor \((C_C)\) equal to 0.46 can be used when designing using both bar type M13(1) and M13(2). The value is approximately 50% higher than the 0.30 coefficient currently recommended by AASHTO (2018a).

7. The creep rupture strength of the material system must be reduced to account for environmental exposure by applying a knock-down factor \((C_E)\) ranging from 0.7 as currently prescribed by ACI 440.1R (ACI 2015) and AASHTO (2018a) to 0.9 as recent studies suggest (Ali et al. 2019). Further assessment of the durability performance of GFRP reinforcement falls beyond the purposes of this study.
Table 17 – Values of $C_E$ and $C_C$ recommended by various guidelines.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Date</th>
<th>CFRP $C_E$</th>
<th>CFRP $C_C$</th>
<th>AFRP $C_E$</th>
<th>AFRP $C_C$</th>
<th>GFRP $C_E$</th>
<th>GFRP $C_C$</th>
<th>BFRP $C_E$</th>
<th>BFRP $C_C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>JSCE CES23</td>
<td>1997</td>
<td>1.00</td>
<td>0.70</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CNR DT203</td>
<td>2007</td>
<td>0.90</td>
<td>0.90(x)</td>
<td>0.80</td>
<td>0.50(x)</td>
<td>0.70</td>
<td>0.30(x)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$fib$ MC</td>
<td>2013</td>
<td>^</td>
<td>0.80</td>
<td>^</td>
<td>0.50</td>
<td>^</td>
<td>0.30</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CSA S806</td>
<td>2012</td>
<td>1.00</td>
<td>0.65</td>
<td>1.00</td>
<td>0.35</td>
<td>1.00</td>
<td>†</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CSA S6</td>
<td>2014</td>
<td>1.00</td>
<td>0.65</td>
<td>1.00</td>
<td>0.35</td>
<td>1.00</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ACI 440.4R</td>
<td>2004</td>
<td>*</td>
<td>0.60</td>
<td>*</td>
<td>0.40</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ACI 440.1R</td>
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<td>0.90</td>
<td>0.55</td>
<td>0.80</td>
<td>0.30</td>
<td>0.70</td>
<td>0.20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ICC AC454</td>
<td>2016</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.70</td>
<td>0.20</td>
<td>0.70</td>
<td>0.20</td>
</tr>
<tr>
<td>AASHTO</td>
<td>2018</td>
<td>1.00</td>
<td>0.65</td>
<td>-</td>
<td>-</td>
<td>0.70</td>
<td>0.30</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Minstroy</td>
<td>2018</td>
<td>1.00</td>
<td>0.60(x)</td>
<td>0.80</td>
<td>0.60(x)</td>
<td>0.70</td>
<td>0.30(x)</td>
<td>0.80</td>
<td>0.40(x)</td>
</tr>
</tbody>
</table>

- Not available.
^ A refined approach to durability is discussed in fib bulletin 40 (fib 2007).
† CSA S806 limits to 0.002 the sustained strain on GFRP bars (CSA 2012).
* ACI 440.4R does not explicitly define a design strength (ACI 2004).
x Applied without environmental reduction factor.

Table 18 – Mean experimental creep rupture strength ratios at $10^6$ hours (114 years) reported by various authors.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Date</th>
<th>Note</th>
<th>Creep rupture strength ratio as a fraction of UTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wolff &amp; Miesseler</td>
<td>1993</td>
<td></td>
<td>0.70</td>
</tr>
<tr>
<td>Budelman &amp; Rostasy</td>
<td>1993</td>
<td></td>
<td>0.64</td>
</tr>
<tr>
<td>Yamaguchi et al.</td>
<td>1997</td>
<td></td>
<td>0.27</td>
</tr>
<tr>
<td>Seki et al.</td>
<td>1997</td>
<td></td>
<td>0.53</td>
</tr>
<tr>
<td>Perigny et al.</td>
<td>2012</td>
<td></td>
<td>0.45</td>
</tr>
<tr>
<td>Johal</td>
<td>2016</td>
<td>Tests on stirrups</td>
<td>0.40-0.63</td>
</tr>
<tr>
<td>Keller et al.</td>
<td>2017</td>
<td>Exposed to alkali</td>
<td>0.45</td>
</tr>
<tr>
<td>Sayed-Ahmed et al.</td>
<td>2017</td>
<td></td>
<td>0.49</td>
</tr>
<tr>
<td>Jeremic</td>
<td>2018</td>
<td>Tests on stirrups</td>
<td>0.50</td>
</tr>
<tr>
<td>Benmokrane et al.</td>
<td>2019</td>
<td>Various manufacturers</td>
<td>0.51</td>
</tr>
</tbody>
</table>
Table 19 – Geometry and mechanical properties of the GFRP bars tested.

<table>
<thead>
<tr>
<th>Bar type</th>
<th>Surface</th>
<th>$d_n$</th>
<th>$a_n$</th>
<th>$a_f$</th>
<th>COV</th>
<th>$E_f$</th>
<th>$F_{fu,m}$</th>
<th>COV</th>
<th>$F_{fu,*}$</th>
<th>$F_{fu,k}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>M13(1)</td>
<td>carved</td>
<td>12.7</td>
<td>129</td>
<td>146</td>
<td>1.1%</td>
<td>56</td>
<td>144</td>
<td>4.1%</td>
<td>126</td>
<td>134</td>
</tr>
<tr>
<td>M13(2)</td>
<td>helical</td>
<td>12.7</td>
<td>129</td>
<td>149</td>
<td>1.1%</td>
<td>57</td>
<td>165</td>
<td>4.9%</td>
<td>140</td>
<td>152</td>
</tr>
</tbody>
</table>

Table 20 – Tensile strength results for bar type M13(1).

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>$F_{fu}$ kN</th>
<th>Specimen ID</th>
<th>$F_{fu}$ kN</th>
<th>Specimen ID</th>
<th>$F_{fu}$ kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>M13(1)-T1</td>
<td>135</td>
<td>M13(1)-T10</td>
<td>141</td>
<td>M13(1)-T19</td>
<td>147</td>
</tr>
<tr>
<td>M13(1)-T2</td>
<td>135</td>
<td>M13(1)-T11</td>
<td>141</td>
<td>M13(1)-T20</td>
<td>149</td>
</tr>
<tr>
<td>M13(1)-T3</td>
<td>136</td>
<td>M13(1)-T12</td>
<td>143</td>
<td>M13(1)-T21</td>
<td>149</td>
</tr>
<tr>
<td>M13(1)-T4</td>
<td>138</td>
<td>M13(1)-T13</td>
<td>144</td>
<td>M13(1)-T22</td>
<td>152</td>
</tr>
<tr>
<td>M13(1)-T5</td>
<td>138</td>
<td>M13(1)-T14</td>
<td>144</td>
<td>M13(1)-T23</td>
<td>153</td>
</tr>
<tr>
<td>M13(1)-T6</td>
<td>138</td>
<td>M13(1)-T15</td>
<td>144</td>
<td>M13(1)-T24</td>
<td>154</td>
</tr>
<tr>
<td>M13(1)-T7</td>
<td>139</td>
<td>M13(1)-T16</td>
<td>145</td>
<td>M13(1)-T25</td>
<td>155</td>
</tr>
<tr>
<td>M13(1)-T8</td>
<td>140</td>
<td>M13(1)-T17</td>
<td>146</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M13(1)-T9</td>
<td>140</td>
<td>M13(1)-T18</td>
<td>147</td>
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<td></td>
</tr>
</tbody>
</table>

Table 21 – Tensile strength results for bar type M13(2).

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>$F_{fu}$ kN</th>
<th>Specimen ID</th>
<th>$F_{fu}$ kN</th>
<th>Specimen ID</th>
<th>$F_{fu}$ kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>M13(2)-T1</td>
<td>144</td>
<td>M13(2)-T10</td>
<td>164</td>
<td>M13(2)-T19</td>
<td>171</td>
</tr>
<tr>
<td>M13(2)-T2</td>
<td>150</td>
<td>M13(2)-T11</td>
<td>164</td>
<td>M13(2)-T20</td>
<td>173</td>
</tr>
<tr>
<td>M13(2)-T3</td>
<td>156</td>
<td>M13(2)-T12</td>
<td>164</td>
<td>M13(2)-T21</td>
<td>174</td>
</tr>
<tr>
<td>M13(2)-T4</td>
<td>156</td>
<td>M13(2)-T13</td>
<td>165</td>
<td>M13(2)-T22</td>
<td>174</td>
</tr>
<tr>
<td>M13(2)-T5</td>
<td>157</td>
<td>M13(2)-T14</td>
<td>165</td>
<td>M13(2)-T23</td>
<td>178</td>
</tr>
<tr>
<td>M13(2)-T6</td>
<td>162</td>
<td>M13(2)-T15</td>
<td>165</td>
<td>M13(2)-T24</td>
<td>178</td>
</tr>
<tr>
<td>M13(2)-T7</td>
<td>162</td>
<td>M13(2)-T16</td>
<td>168</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M13(2)-T8</td>
<td>162</td>
<td>M13(2)-T17</td>
<td>169</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M13(2)-T9</td>
<td>163</td>
<td>M13(2)-T18</td>
<td>171</td>
<td></td>
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</table>
Table 22 – Creep rupture test results for batch M13(1).

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Sust. load % UTS</th>
<th>tfailure hour</th>
<th>Specimen ID</th>
<th>Sust. load % UTS</th>
<th>tfailure hour</th>
</tr>
</thead>
<tbody>
<tr>
<td>M13(1)-C1</td>
<td>0.87</td>
<td>0.12</td>
<td>M13(1)-C21</td>
<td>0.78</td>
<td>25113†</td>
</tr>
<tr>
<td>M13(1)-C2</td>
<td>0.87</td>
<td>0.33</td>
<td>M13(1)-C22</td>
<td>0.74</td>
<td>24992†</td>
</tr>
<tr>
<td>M13(1)-C3</td>
<td>0.87</td>
<td>1.42</td>
<td>M13(1)-C23</td>
<td>0.74</td>
<td>24993†</td>
</tr>
<tr>
<td>M13(1)-C4</td>
<td>0.87</td>
<td>2.50</td>
<td>M13(1)-C24</td>
<td>0.74</td>
<td>25110†</td>
</tr>
<tr>
<td>M13(1)-C5</td>
<td>0.87</td>
<td>16.0</td>
<td>M13(1)-C25</td>
<td>0.74</td>
<td>25113†</td>
</tr>
<tr>
<td>M13(1)-C6</td>
<td>0.87</td>
<td>23.8</td>
<td>M13(1)-C26</td>
<td>0.73</td>
<td>18.1</td>
</tr>
<tr>
<td>M13(1)-C7</td>
<td>0.85</td>
<td>0.13</td>
<td>M13(1)-C27</td>
<td>0.73</td>
<td>229</td>
</tr>
<tr>
<td>M13(1)-C8</td>
<td>0.83</td>
<td>0.60</td>
<td>M13(1)-C28</td>
<td>0.73</td>
<td>2568</td>
</tr>
<tr>
<td>M13(1)-C9</td>
<td>0.83</td>
<td>1.30</td>
<td>M13(1)-C29</td>
<td>0.73</td>
<td>11012</td>
</tr>
<tr>
<td>M13(1)-C10</td>
<td>0.83</td>
<td>1.80</td>
<td>M13(1)-C30</td>
<td>0.68</td>
<td>25187†</td>
</tr>
<tr>
<td>M13(1)-C11</td>
<td>0.83</td>
<td>2.00</td>
<td>M13(1)-C31</td>
<td>0.68</td>
<td>25186†</td>
</tr>
<tr>
<td>M13(1)-C12</td>
<td>0.83</td>
<td>290</td>
<td>M13(1)-C32</td>
<td>0.68</td>
<td>25185†</td>
</tr>
<tr>
<td>M13(1)-C13</td>
<td>0.81</td>
<td>980</td>
<td>M13(1)-C33</td>
<td>0.68</td>
<td>25184†</td>
</tr>
<tr>
<td>M13(1)-C14</td>
<td>0.81</td>
<td>980</td>
<td>M13(1)-C34</td>
<td>0.68</td>
<td>25183†</td>
</tr>
<tr>
<td>M13(1)-C15</td>
<td>0.81</td>
<td>5512</td>
<td>M13(1)-C35</td>
<td>0.63</td>
<td>25114†</td>
</tr>
<tr>
<td>M13(1)-C16</td>
<td>0.81</td>
<td>6518</td>
<td>M13(1)-C36</td>
<td>0.63</td>
<td>25113†</td>
</tr>
<tr>
<td>M13(1)-C17</td>
<td>0.81</td>
<td>7694</td>
<td>M13(1)-C37</td>
<td>0.63</td>
<td>25112†</td>
</tr>
<tr>
<td>M13(1)-C18</td>
<td>0.78</td>
<td>11.6</td>
<td>M13(1)-C38</td>
<td>0.63</td>
<td>25116†</td>
</tr>
<tr>
<td>M13(1)-C19</td>
<td>0.78</td>
<td>5685</td>
<td>M13(1)-C39</td>
<td>0.63</td>
<td>25115†</td>
</tr>
<tr>
<td>M13(1)-C20</td>
<td>0.78</td>
<td>7259</td>
<td>M13(1)-C40</td>
<td>0.63</td>
<td>25111†</td>
</tr>
</tbody>
</table>

† Runout points
**Table 23 – Creep rupture results for batch M13(2)**

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Sust. load % UTS</th>
<th>( t_{\text{failure}} ) hour</th>
<th>Specimen ID</th>
<th>Sust. load % UTS</th>
<th>( t_{\text{failure}} ) hour</th>
</tr>
</thead>
<tbody>
<tr>
<td>M13(2)-C1</td>
<td>0.89</td>
<td>0.25</td>
<td>M13(2)-C14</td>
<td>0.80</td>
<td>40.1</td>
</tr>
<tr>
<td>M13(2)-C3</td>
<td>0.86</td>
<td>0.38</td>
<td>M13(2)-C15</td>
<td>0.80</td>
<td>41.2</td>
</tr>
<tr>
<td>M13(2)-C4</td>
<td>0.86</td>
<td>3.63</td>
<td>M13(2)-C16</td>
<td>0.80</td>
<td>254</td>
</tr>
<tr>
<td>M13(2)-C2</td>
<td>0.86</td>
<td>4.00</td>
<td>M13(2)-C17</td>
<td>0.80</td>
<td>361</td>
</tr>
<tr>
<td>M13(2)-C5</td>
<td>0.86</td>
<td>6.22</td>
<td>M13(2)-C18</td>
<td>0.77</td>
<td>13.4</td>
</tr>
<tr>
<td>M13(2)-C6</td>
<td>0.86</td>
<td>11.2</td>
<td>M13(2)-C19</td>
<td>0.77</td>
<td>101</td>
</tr>
<tr>
<td>M13(2)-C7</td>
<td>0.86</td>
<td>30.8</td>
<td>M13(2)-C20</td>
<td>0.77</td>
<td>198</td>
</tr>
<tr>
<td>M13(2)-C8</td>
<td>0.83</td>
<td>5.47</td>
<td>M13(2)-C21</td>
<td>0.77</td>
<td>210</td>
</tr>
<tr>
<td>M13(2)-C9</td>
<td>0.83</td>
<td>21.6</td>
<td>M13(2)-C22</td>
<td>0.77</td>
<td>296</td>
</tr>
<tr>
<td>M13(2)-C10</td>
<td>0.80</td>
<td>2.05</td>
<td>M13(2)-C23</td>
<td>0.75</td>
<td>220</td>
</tr>
<tr>
<td>M13(2)-C11</td>
<td>0.80</td>
<td>4.10</td>
<td>M13(2)-C24</td>
<td>0.75</td>
<td>317</td>
</tr>
<tr>
<td>M13(2)-C12</td>
<td>0.80</td>
<td>13.6</td>
<td>M13(2)-C25</td>
<td>0.75</td>
<td>2520</td>
</tr>
<tr>
<td>M13(2)-C13</td>
<td>0.80</td>
<td>21.3</td>
<td>M13(2)-C26</td>
<td>0.75</td>
<td>4249</td>
</tr>
</tbody>
</table>

**Table 24 – Statistics of the times-to-failure for each load level for bar M13(1).**

<table>
<thead>
<tr>
<th>Load level</th>
<th>( t_{\text{failure,m}} ) % UTS</th>
<th>( t_{\text{failure,k}} ) hour</th>
<th>COV /</th>
</tr>
</thead>
<tbody>
<tr>
<td>87% UTS</td>
<td>7.35</td>
<td>0.02</td>
<td>125%</td>
</tr>
<tr>
<td>85% UTS</td>
<td>0.13</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>83% UTS</td>
<td>59.1</td>
<td>0.01</td>
<td>195%</td>
</tr>
<tr>
<td>81% UTS</td>
<td>4337</td>
<td>274.4</td>
<td>65%</td>
</tr>
<tr>
<td>78% UTS</td>
<td>4318</td>
<td>0.16</td>
<td>72%</td>
</tr>
<tr>
<td>73% UTS</td>
<td>3457</td>
<td>1.54</td>
<td>129%</td>
</tr>
</tbody>
</table>

* Single creep rupture point.
Table 25 – Statistics of the times-to-failure for each load level for bar M13(2).

<table>
<thead>
<tr>
<th>Load level</th>
<th>$t_{\text{failure}, m}$</th>
<th>$t_{\text{failure}, k}$</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>89% UTS</td>
<td>0.25</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>86% UTS</td>
<td>9.37</td>
<td>0.22</td>
<td>108%</td>
</tr>
<tr>
<td>83% UTS</td>
<td>13.5</td>
<td>1.21</td>
<td>60%</td>
</tr>
<tr>
<td>80% UTS</td>
<td>92.1</td>
<td>0.66</td>
<td>139%</td>
</tr>
<tr>
<td>77% UTS</td>
<td>163.6</td>
<td>7.38</td>
<td>60%</td>
</tr>
<tr>
<td>75% UTS</td>
<td>1826</td>
<td>32.9</td>
<td>92%</td>
</tr>
</tbody>
</table>

* Single creep rupture point.

Figure 39 - Creep stages in an FRP unidirectional composite. (Rossini et al. 2019c)
Figure 40 – Cross section and surface of bar M13(1) [a] and M13(2) [b]. (Rossini et al. 2019c)

Figure 41 – Schematic diagram of a typical creep rupture specimen. (Rossini et al. 2019c)
Figure 42 – Schematic diagram of the creep rupture test frame and setup. (Rossini et al. 2019c)

Figure 43 – General view of the creep rupture test setup. (Rossini et al. 2019c)
Figure 44 – Detail of the adjustable loading system. (Rossini et al. 2019c)

Figure 45 – Creep rupture curves in a logarithmic diagram. (Rossini et al. 2019c)
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Figure 48 – Probability density function for the normalized tensile strengths of bar M13(1).

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Figure 51 – Probability density function for the normalized times-to-failure of bar M13(2).
Figure 52 – Sustained load versus logarithmic time-to-failure for batch M13(1). (Rossini et al. 2019c)

Figure 53 – Sustained load versus logarithmic time-to-failure for batch M13(2). (Rossini et al. 2019c)
Chapter 5

Conclusions

This dissertation tackles the challenges related to state-of-the-practice FRP prestressing by proposing an alternative solution and investigating its feasibility. The first component of this study addresses the opportunities and challenges related to Carbon FRP (CFRP) prestressing while developing the design, construction, and load testing of a short-span bridge entirely reinforced and prestressed with FRPs. The lack of design guidance was identified as a limiting factor for wider applicability of FRP prestressing. To address this knowledge gap, a unified framework was developed for the design of FRP reinforced and prestressed structures that was later formalized in the second edition of the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete* (AASHTO 2018b) and is consistent with the first edition of the *AASHTO Guide Specifications for the Design of Concrete Bridge Beams Prestressed with CFRP Systems* (AASHTO 2018a).

The experience gathered highlighted some limitations of CFRP prestressing including the inherent complexity of the tensioning operations, the brittleness at pull, the tendency to cause concrete splitting, and the relevant material cost. Therefore, in the second study,
mild pre-tensioning using GFRP reinforcement was proposed as a novel approach to the design and construction of those elements that require a relatively low level of prestress equal to approximately 40% of the guaranteed strength of GFRP material systems and are most exposed to environmental weathering and chloride penetration in coastal areas. To limit the level of prestress greatly eases tensioning operations and allows to use traditional steel anchors available at any precast yard. It also prevents failures at pull and concrete splitting. The use of a cost-efficient material system that is also less prone to prestress losses offsets the need for a larger number of strands. Experimental evidences to support this innovative approach were gathered for the first time on a prototype GFRP strand specifically developed through a federally-funded partnership with industries.

To be effectively used in prestressing, a material system must maintain its initial pull without delayed failures. Historically, the main limitation to GFRP prestressing laid in the relatively low creep-rupture strength reported in codes and standards because of the lack of experimental evidence and reliable predictive models in archival literature. To address this gap, the third study collects and analyzes a large number of creep-rupture and tensile test results to develop a rational predictive model based on statistical considerations. This novel approach allows for a reliable assessment of the long term properties of GFRP reinforcement and shows how previous limitations may be overly conservative and GFRP can be effectively used in prestressing applications.

**Implications on the State-of-the-practice**

The experience gathered during the design, construction, and load testing of the Innovation Bridge was instrumental in developing the second edition of the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete* (AASHTO
that constitute the current state of the practice for GFRP deployment in infrastructural applications. The author was part of the task group of practitioners, academics, and transportation officials that developed the document (Rossini et al. 2018c). The level of prestress applied to the CFRP strands in the Innovation bridge was deliberately higher than the limitations imposed by ACI 440.4R (ACI 2004) at the time of design and close to values later adopted in *AASHTO Guide Specifications for the Design of Concrete Bridge Beams Prestressed with CFRP Systems* (AASHTO 2018a) that constitute the current state of the practice for CFRP deployment in infrastructural applications. The Innovation Bridge was taken as a case-study to validate the performance of a design software specifically developed by the Florida Department of Transportation (FDOT) for the design of bridges prestressed with CFRP in the State of Florida. The author contributed to the validation as part of the University of Miami (UM) collaboration with FDOT (Rossini et al. 2018d). The software will be made available for practitioners on FDOT website along with standard specifications.

Even if at its early stage of development, a GFRP strand for prestressing applications shows the potential to be a game changer in easing the deployment of FRP prestressed solutions, as demonstrated by the interest and investments of various stakeholders. The development of the GFRP strand prototype discussed in the second study of this dissertation is carried on by a joint partnership of academic institutions, industries, and governing agencies. These include Arkema as a resin manufactures, SIREG as an FRP manufacturer, Owens Corning as a glass fiber manufacturer, and the National Cooperative Highway Research Program (NCHRP) (Nanni et al. 2019). The Florida Department of Transportation (FDOT) has committed to lead pre-standardization and deployment of the
technology pending its successful development to the commercial stage. The drafting of developmental standard specifications is currently undergoing. Per recent policies, FDOT is actively promoting the use of FRP reinforcement in aggressive exposures.

The third component of this dissertation provides insight on the creep rupture performance of GFRP through adding a relevant number of data points to the existing experimental database, and, more importantly, by developing a statistically-based methodology for analyzing the data and extrapolate a safe value for the creep rupture reduction factor. This effort will have a direct implication on the development of the Building Code provisions for GFRP-reinforced concrete within ACI committee 440-H to which the author contributes. Creep rupture was identified as one of the limiting design requirements in some applications (Rossini et al. 2018a).

**Recognition**

In view of its unique features, the Innovation Bridge was awarded: (a) the 2018 *Engineering Excellence and the Grand Conceptor Award* of the Florida Engineering Society and the American Council of Engineering Companies of Florida; (b) the 2017 *Project of the Year Category I Award* of the American Society of Civil Engineers Miami-Dade Branch; and, (c) a nomination for the 2018 *fib Awards for Outstanding Concrete Structures* of the *fédération internationale du béton*. The innovative features associated to the development of GFRP reinforcement for prestressing applications were recognized by (d) the 2019 *JEC Innovation Award in Construction and Infrastructures* of the JEC Group presented to a joint partnership of Arkema, NCHRP, SIREG, and UM for the work developed within the MILDGLASS project.
Future Work

Carbon FRP (CFRP) prestressing is currently a well-established technology and the lack of design guidelines was recently addressed by AASHTO (2018a, 2018b). Future refinement is expected following the assembling of larger experimental databases based on standardized test procedures, but no major advancements are foreseeable. Conversely, Glass FRP (GFRP) prestressing is a potential game changer that has only recently been brought to the table.

The GFRP strand investigated in the second study of this dissertation is an early stage prototype non representative of industrial grade production. At the time the study was conducted, the goal was to assess the feasibility of the technology. Now, the prototype technology is being refined through the deployment of a specific type of thermoplastic resin aimed to ease manufacturing of the complex twisted shape. Research and development efforts will continue to refine the technology and a full mechanical and physical characterization of the final product will be required before moving to commercial applications. This should include long-term tests under sustained load and durability tests. Furthermore, structural tests and demonstrative applications are required to assess the performance of the material system in prestressed members.

The use of thermoplastic resin can also have relevant implications on the GFRP bar supply chain. In the steel industry, a steel mill is not responsible for shaping and cutting bars to order. Instead, a bar fabricator stocks large quantity of coiled straight bars that are readily shaped and cut once orders are received from the construction site. In the FRP industry, the bar manufacturer must bend and shape FRP bars and stirrups when the bar is pultruded. Therefore, field operations may be forced to schedule according to the speed of
A pultrusion line and adjustments during construction may represent a challenge for bar suppliers. The use of thermoplastic resin would allow the production of stocks of coiled FRP bars that can be subsequently heated, shaped, and cut to length as orders are received. This solution would sensibly speed up the delivery of complex shapes and stirrups at the construction field. Before this option becomes viable further Research and Development is required as carried on within the MILDGLASS research effort (Nanni et al. 2019) and a full characterization of the new material system is required.

The proposed approach to the characterization of the creep rupture strength of GFRP bars was proved to be a viable statistically based method. However, the results gathered are specific to the two types of GFRP bars investigated. The approach needs validation over a larger database including different manufacturers, bar sizes, and surface preparations.

Basalt FRP (BFRP) bars are recently gaining attention as an alternative to Glass FRP (GFRP) bars. Basalt fibers, when manufactured in an adequately controlled environment, feature mechanical performance and chemical stability comparable or potentially superior to ECR glass fibers (Fiore et al. 2015). The development of BFRP strands for prestressing applications is theoretically possible achieving slightly better mechanical performance at a similar cost. However, the basalt fiber industry should be able to deliver on consistent performance to make this possible.
References


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