

École polytechnique de Louvain

Development and testing of GFRP-SMA rebar connections for reinforced concrete members

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ABSTRACT

This study presents a comprehensive investigation into the potential of Fiber Reinforced Polymer (FRP), particularly Glass Fiber Reinforced Polymer (GFRP), as an effective replacement for steel in Reinforced Concrete (RC) structures. Despite the numerous advantages GFRP offers, including corrosion resistance, high tensile strength, lightweight, non-magnetic properties, and electromagnetic transparency, its low modulus of elasticity and brittle failure mode hinder its application in seismic design.

To overcome these limitations, the replacement of GFRP with Shape Memory Alloy (SMA) in the plastic hinge regions of RC structures is investigated. A significant part of this work revolves around the design, testing, characterization, and validation of mechanical couplers for GFRP-Steel, SMA-Steel, and GFRP-SMA connections. Extensive instrumentation is employed for characterization, incorporating Distributed Fiber Optic Sensors (DFOS) across the three materials and inside the couplers.

Furthermore, a numerical model in Vector2 is proposed to simulate the behavior of RC reinforced with GFRP. The intended future validation of this model involves comparison with experimental tests on two RC beams: one reinforced exclusively with GFRP, and another utilizing a GFRP and SMA combination. The specifications of these beams and associated testing protocol are detailed within the study. The model is subsequently expanded to incorporate SMA and the developed mechanical couplers, providing a robust foundation for future research and development in this area.

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CHAPTER

1

INTRODUCTION

1.1 Motivations

In seismic design, the focus has traditionally been on preventing structural collapse during earthquakes. However, current guidelines recognize the importance of assessing and limiting residual deformations to enhance post-earthquake functionality and minimize repair requirements. This shift towards performance-based design aims to ensure that structures not only withstand seismic events but also maintain their operational capacity.

Earthquakes pose significant economic challenges (See Fig 1.1) due to the reconstruction of damaged buildings. A considerable portion of the economic losses incurred globally can be attributed to these reconstruction efforts. The failure of structural elements and the occurrence of permanent deformations contribute significantly to the high costs associated with earthquakes. Therefore, it is crucial to develop strategies that mitigate these effects and reduce the economic burden.

Modern design standards emphasize achieving ductile failure rather than brittle failure in reinforced concrete (RC) structures. The objective is to avoid situations where buildings become non-repairable after seismic events. By limiting damages and permanent deformations, the costs of repair and recovery can be minimized. Residual deformation levels below a certain threshold, often defined as 0.5% inter-storey drift, have been identified as critical in achieving these cost and life-saving benefits.

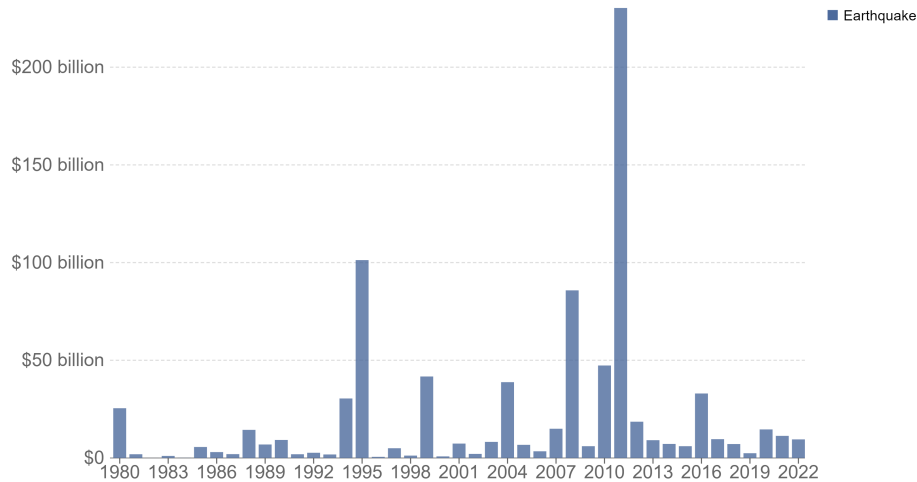
The behavior of RC walls under seismic forces is characterized by the formation of plastic hinges. These hinges are regions where deformations and crack formations concentrate. The residual displacements observed in the structure are primarily influenced by the behavior of these plastic hinges. Consequently, it is essential to develop solutions and implement improved structural technologies that effectively reduce residual displacements. By minimizing the displacements at the plastic hinge regions, the operational integrity of buildings can be maintained.

The limitations of traditional RC construction using steel reinforcement have prompted the exploration of alternative materials to address the challenges mentioned above. One such material is glass fiber reinforced polymer (GFRP)(See Fig 1.2), which offers superior corrosion resistance compared

Global damage costs from natural disasters, 1980 to 2022

Total economic cost of damages as a result of global natural disasters in any given year, measured in current US\$. Includes those from drought, floods, extreme weather, extreme temperature, landslides, dry mass movements, wildfires, volcanic activity and earthquakes.

Our World
in Data



Source: EM-DAT, CRED / Université catholique de Louvain, Brussels (Belgium)
OurWorldInData.org/natural-disasters • CC BY

Figure 1.1: Economical loss due to earthquakes

to traditional steel reinforcement. The use of GFRP rebars can significantly enhance the durability of RC structures by mitigating the effects of corrosion, which leads to deterioration and structural degradation. Furthermore, GFRP rebars possess advantages such as lightweight construction, high tensile strength, non-magnetic properties, and electromagnetic transparency, making them suitable for various applications.



Figure 1.2: GFRP rebar [73]

However, GFRP reinforcement also has limitations that need to be addressed. Its relatively low elastic modulus, which is typically 20% to 25% compared to the one from the steel rebars, results in reduced ductility under loading conditions. In seismic regions where structures are subjected to substantial lateral forces, this limitation may impact the overall performance and resilience of RC structures.

To overcome the limitations of GFRP reinforcement, researchers have turned to shape memory

alloys (SMAs) as a complementary solution. SMAs possess the unique ability to recover their original shape after deformation, allowing them to accommodate large deformations, and reduce residual displacements. Those extremely ductile properties allow SMA to efficiently dissipate energy, especially compared to GFRP. By strategically incorporating SMA rebars, particularly in critical regions such as plastic hinge areas, the seismic performance of RC elements can be significantly enhanced. This hybrid approach combines the corrosion resistance and durability of GFRP rebars with the recovery capabilities of SMAs, resulting in more resilient and sustainable RC structures. It also tackles the problem of the cost of SMA rebars that limits their widespread use.

Conventional concrete construction practices have also raised concerns regarding their sustainability due to the significant consumption of natural resources and the negative environmental impact associated with their production. One of them being the source of the sand used in traditional concrete (See Fig 1.3). While it may seem that sand is abundantly available, not all sand can be used for this purpose. Desert sand, for example, is too smooth for construction purposes. Instead, the demand is high for angular sand found in riverbeds, lake beds, and beaches, causing devastating environmental impacts, such as erosion, habitat destruction, and water pollution, as these locations are stripped to extract sand.[66][12] In response, alternative approaches that minimize resource depletion and reduce environmental harm have gained attention. One such approach is the use of seawater and sea-sand concrete (SSC) in marine and coastal projects, which utilizes locally available resources and reduces the need for extensive material transportation, consequently lowering costs and energy consumption.[86]

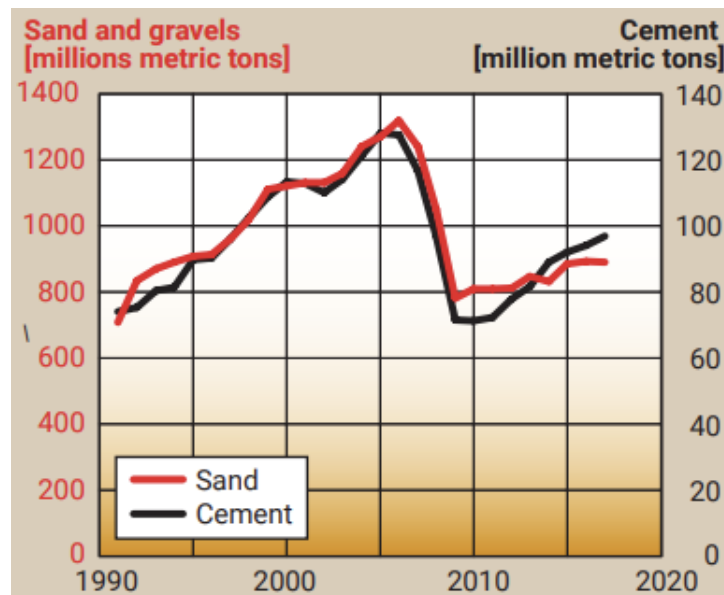


Figure 1.3: Global sand consumption, based on declared cement production [1]

However, the use of seawater and sea-sand in concrete raises the problem of the high chloride content present in these materials. The presence of chloride ions can pose a significant risk of corrosion to steel reinforcement within the concrete structures. To mitigate this issue, the use of fiber-reinforced polymers (FRP) as reinforcement in SSC structures presents a promising solution. FRP rebars exhibit excellent resistance to chloride penetration, eliminating the need for pre-treatment of seawater and sea-sand. By incorporating FRP reinforcement, the risk of corrosion-induced deterioration is significantly reduced, enhancing the durability and longevity of SSC structures.

The incorporation of GFRP rebars in reinforced concrete structures also offers significant advantages in terms of extending the service life of these constructions. One of the main limitations of traditional steel reinforcement is its susceptibility to corrosion, especially in environments with deicing salts, which can accelerate the deterioration process, leading to important cost and repairs requirement.[53] The use of GFRP rebars provides a viable solution to overcome these corrosion problems and enhance the durability of concrete structures as well as further reducing the environmental impact of concrete building construction and maintenance.

1.2 Objectives

One of the main challenge when it comes to combine GFRP rebars and SMA rebars is to ensure a proper connection between the two materials with very distinct physical properties. In order to achieve this connection, this master thesis will focus on mechanical coupler.

The first step is thus to characterize both materials, their mechanical properties and their binding possibilities. After identifying and testing an appropriate mechanical coupler, the aim will be to reduce its length, due to its crucial role as a zone where stirrups cannot be accommodated.

Additionally, there's an aim to develop a numerical model that includes both materials and couplers. This model should be validated through experimental tests on concrete beams, with and without added SMA to the GFRP reinforcement to assess its impact.

1.3 Research novelty

Although it has been much less studied than traditional steel reinforcement, GFRP and SMA-reinforced concrete structures have already been characterized and tested in previous papers. Solutions for mechanical couplers have also already been discussed but they lack precise information and characterization, due to the nature of the papers. Characterizing these couplers is thus the first research novelty of this thesis. With the data that will be obtained during the characterisation, it will also become possible to minimise the length of the coupler to make it more usable in future real-life situations. This will be achieved using extensive instrumentation on experimental tests, including the use of Distributed Fiber Optic Sensing (DFOS), allowing for precise measurement of the strain behavior along the rebars and the couplers.

Secondly, efforts will be directed towards a finite element model using Vector 2 to first predict the behaviour of a full GFRP beam. On this beam will be detailed the influence of the differents models analysis and a focus will be made on developing bond conditons fitting with the GFRP characteristics. The Vector 2 model will then be extended to a second type of beam composed of both GFRP and SMA, as well as mechanical couplers linking the two.

1.4 Thesis organization

The organization of this thesis is mainly centered around three chapters, each focusing on a distinct area of study within this research on Glass Fiber Reinforced Polymer (GFRP).

The first chapter is dedicated to the experimental characterization of GFRP. Given that GFRP is less standardized than traditional steel rebars, it's essential to understand its properties and behavior thoroughly. This chapter not only explores simple GFRP rebars but also delves into their performance

in anchorage situations, and the potential instrumentation to be utilized in mechanical coupler tests.

The following chapter explores various configurations of mechanical couplers. It starts by investigating SMA-Steel and GFRP-Steel couplers, both of which combine either SMA or GFRP with predictable and mechanically compatible steel. This approach allows to start with less complex tests, reducing the number of unknown parameters. The chapter retraces the progression of testing setups, protocols, and results, leading to successful results for these hybrid connections. The conclusion of this chapter is the design and validation of a GFRP-SMA mechanical coupler, marking the achievement of the first objective of this thesis.

In the last chapter, the focus is shifted toward numerical modeling. Using Vector2, a model is developed that simulates the behavior of concrete elements reinforced with various GFRP rebars. This chapter was originally intended to be a strictly experimental one, wherein the mechanical couplers would be validated by comparing two beams under cyclic loading conditions—one reinforced solely with GFRP, and the other with a combination of GFRP and SMA. However, due to unforeseen delays, the physical construction of the beams could not be completed. Instead, a comprehensive design for the setup is provided as well as a numerical prediction of the beams' behavior based on the developed model for RC with GFRP.

CHAPTER

2

LITERATURE REVIEW

2.1 Fiber-Reinforced Polymer (FRP) rebars

FRPs are a class of composite materials that consist of a polymer matrix (resin) and a reinforcing agent, typically in the form of fibers. The term "polymer" refers to large molecules made up of repeating sub-units, while the term "matrix" refers to a continuous phase (in this case, the polymer) that encapsulates the reinforcing agent. The fibers provide strength and stiffness to the composite, while the polymer matrix binds the fibers together and transfers stresses between them. The idea of combining two or more components to produce a material with properties superior to those of the individual components is the fundamental concept of composite materials. In the case of FRPs, the fibers provide strength and stiffness, while the matrix holds the fibers in place and transfers loads between them.

The different types of FRPs are classified based on the type of fiber reinforcement they use. Here is a list of commonly used FRP and their application in the construction industry :

- **Glass Fiber-Reinforced Polymer (GFRP):** This is perhaps the most common and well-known type of FRP. Glass fibers, made from various types of glass depending on the application, are incorporated into a polymer matrix. GFRPs are favored due to their low cost, high strength, and resistance to corrosion.[71] They are less stiff than carbon fibers but are more flexible. GFRPs have a wide range of applications in construction, such as in the reinforcement of concrete and the construction of structural components like beams, panels, and pipes.
- **Carbon Fiber-Reinforced Polymer (CFRP):** CFRP uses carbon fibers as the reinforcement. These fibers have a high strength-to-weight ratio, high stiffness, and high tensile strength, making CFRP an ideal material for many high-performance applications.[16][5] However, CFRPs are more expensive than GFRPs. In the construction industry, CFRPs are often used for strengthening and retrofitting structures, such as reinforcing concrete slabs, columns, and beams, as well as improving the structural performance of bridges and tunnels.
- **Aramid Fiber-Reinforced Polymer (AFRP):** AFRP uses aramid fibers, which are also known as Kevlar. These fibers are extremely strong and tough, with excellent resistance to impact,

heat, and chemicals. They are also lightweight and have good energy absorption characteristics. However, they can be sensitive to UV radiation and moisture.[48] In construction, AFRPs are used in applications that require high strength and toughness, such as explosion-resistant structures or structures that need to resist impact forces.

- **Basalt Fiber-Reinforced Polymer (BFRP):** BFRP is a newer type of FRP and uses basalt fibers, which are made from basalt rock. These fibers offer a balance of performance and cost, with strength and stiffness characteristics between those of glass and carbon fibers. They also have good resistance to heat, chemicals, and UV radiation.[25] In the construction industry, BFRPs are gaining popularity for applications that require good performance and environmental sustainability, as basalt is a naturally abundant material.

Two types of polymer resin can be used : thermoplastic and thermosetting.

Thermosetting resins, once cured, undergo a chemical reaction that results in a hard, infusible product. The curing process forms a highly cross-linked polymer network that is heat resistant and cannot be re-melted or reformed once cured. The benefits of thermosetting resins are their excellent thermal stability, resistance to many chemicals, and high stiffness and strength. Examples of thermosetting resins include polyester, vinyl ester, epoxy, and phenolic resins.

Thermoplastic resins, on the other hand, can be melted and solidified repeatedly without undergoing any chemical changes. The polymer chains in thermoplastic resins do not cross-link during the curing process, allowing the material to be reshaped by heat and pressure multiple times.[13] Common examples of thermoplastic resins include polyethylene, polypropylene, polystyrene, and polyvinyl chloride (PVC).

In terms of their use in fiber-reinforced polymers, thermosetting resins are commonly used in traditional FRP composites due to their high thermal stability and resistance to various environmental conditions. However, the use of thermoplastic resins in FRP composites has been increasing due to their ability to be reprocessed and recycled, although they typically require more advanced processing methods to ensure proper fiber impregnation and bonding.[13]

Fiber-Reinforced Polymer (FRP) applications in the context of concrete structures can be classified as follows[60]:

- **Internal Applications:** This typically involves the use of FRP materials such as bars, rods, and tendons in the initial construction of a structure. These are incorporated within the concrete to improve its tensile strength and overall durability. Examples of this application are seen in the construction of FRP-reinforced and FRP-prestressed concrete structures. Essentially, in this category, the FRP materials become an integral part of the structure during its construction.
- **External Applications:** This category relates to the use of FRP materials like plates, fabrics, wraps, and near-surface mounted (NSM) FRP bars for the purpose of strengthening, retrofitting, and repairing existing structures. This is often applied to structures initially built with steel-reinforced concrete that need to be fortified or restored due to wear, aging, or damage. In the future, even FRP-reinforced concrete structures may require such external reinforcement.

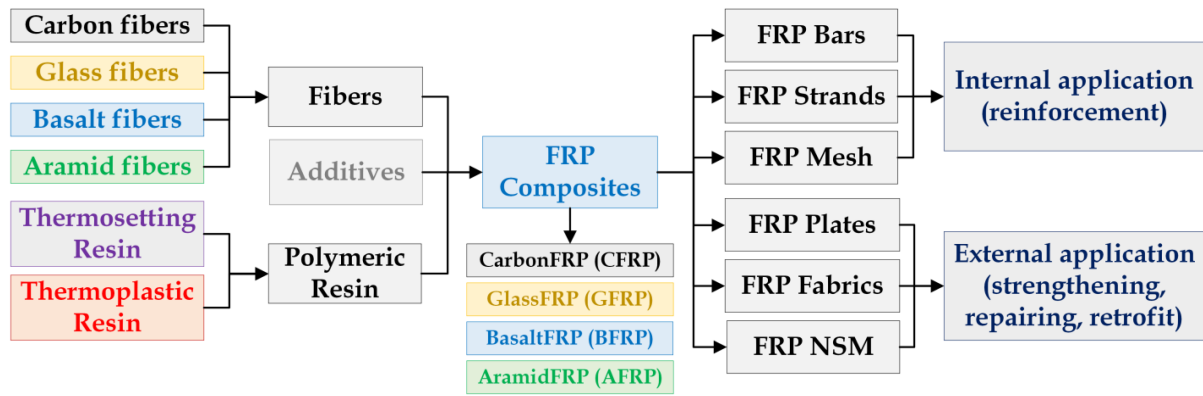


Figure 2.1: FRP composite tree diagram[60]

In the context of this thesis, we'll focus specifically on the GFRP rebars.

The evolution of these materials traces back to the 1930s when the first patented fiberglass was produced by the collaboration between Owens Glass Company and the Corning Company. The ensuing decades, especially during and after World War II, witnessed substantial advancements in FRP technology, predominantly within the military and aerospace industries.[60]

FRP composites, especially GFRP, have several advantages over traditional steel reinforcement, especially in locations where corrosion is a safety and economic issue. The good mechanical performance, excellent durability, and sustainability (lower environmental impact) of FRP composites make them a good fit for such situations. For instance, compared to steel rebars (See Figure 2.2), GFRP bars have higher strength, less weight, and longer service life. However, FRP composites also have their challenges, including differences in thermal expansion coefficients between the fibers and the resin, susceptibility to high temperatures, and a lack of bendability.[60]

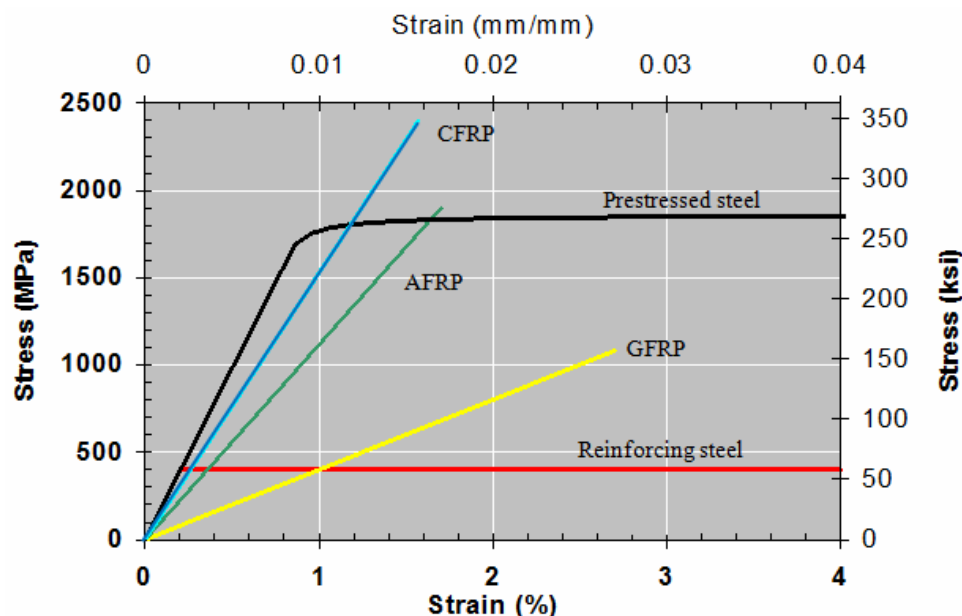


Figure 2.2: Stress-strain plots of FRP (ACI 440R-96)[64]

The initial cost of GFRP rebars, which is often higher than steel rebars, has seen significant

changes in recent years due to price fluctuations in the global metal market and the growth of the GFRP rebar market. However, the long-term cost of FRP-reinforced concrete structures can be lower than traditional reinforcement because of their resistance to corrosion and durability, which can lead to less maintenance and repair costs over the lifetime of the structure.[60]

New construction projects using FRP as the main or auxiliary reinforcement in concrete structures have been carried out all over the world. Figure 2.3 is a perfect example where GFRP was preferred to traditional steel reinforcement to prevent corrosion in a region where deicing salt is abundantly used. Collaborative research between academia, industry, and organizations has yielded important results, providing updated material standards and design specifications that owners, engineers, and contractors can use for the safe construction and inspection of FRP-reinforced concrete structures. Various organizations, such as the American Association of State Highway and Transportation (AASHTO), the American Concrete Institute (ACI), the Canadian Standards Association (CSA), the Japan Prestressed Concrete Institute (JPCI), and the International Organization for Standardization (ISO) have developed guidelines for the design of reinforced concrete (RC) structures with FRP composites. Yet, more work is needed before this technology can be fully implemented.[60]



Figure 2.3: Concrete bridge reinforced with GFRP in Moscow, where deicing salt is abundantly used, to prevent corrosion[29]

2.2 Shape Memory Alloys (SMA)

Shape Memory Alloys (SMAs) are unique materials that possess the ability to remember their original shape, which they can regain when exposed to a certain temperature. This intriguing behavior is due to the phase transformations that occur in the material, specifically between the martensitic phase (low temperature) and the austenitic phase (high temperature) as illustrated in Figure 2.4.

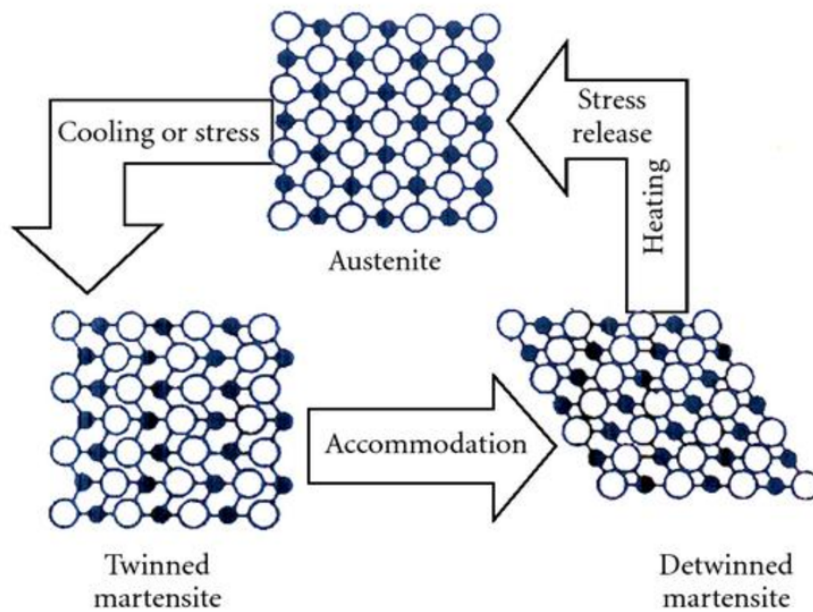


Figure 2.4: Changes in crystal form of SMA which leads to Superelasticity (induced by stress) and SME (induced by heating). Source [28]. Adapted from [37].

The term "shape memory effect (SME)" refers to the effect where the alloy is deformed at a low temperature and then regains its original shape upon being heated. The original shape is set during the manufacturing process, when the SMA is given a "training" shape at high temperatures. If the SMA is deformed at a low temperature (in the martensite phase), it can be returned to its original shape by heating it (causing it to transform back to the austenite phase). SMAs can also exhibit a characteristic known as "superelasticity (SE)", where the material can undergo substantial deformation and then return to its original shape upon removal of stress at higher temperatures.

NiTi (Nickel-Titanium) alloys, often referred to as Nitinol (a combination of Nickel, Titanium, and Naval Ordnance Laboratory where it was first developed), are the most common type of SMAs. They are well-known for their excellent biocompatibility, good wear resistance, corrosion resistance, and, most notably, their shape memory effect and superelasticity. Other alloys, such as Ni-Ti-Nb (Nickel-Titanium-Niobium Alloys), Cu-Al-Ni (Copper-Aluminium-Nickel Alloys), Cu-Zn-Al (Copper-Zinc-Aluminium Alloys) and Fe-Mn-Si (Iron-Manganese-Silicon Alloys) have also been considered for civil engineering application but currently more than half of the studies conducted on SMA focused on NiTi alloys.[65]

The functionality of NiTi SMAs is based on a solid-state phase transformation between the martensitic and austenitic phases. At lower temperatures, NiTi is in a relatively soft and easily deformable martensitic phase. When heated above its transformation temperature, the NiTi alloy transforms to a stiffer, stronger austenitic phase and recovers its original shape, as illustrated in Figure 2.5. The transition between these two phases is driven by a rearrangement of the atoms in the alloy, from a less ordered, lower-symmetry martensitic phase to a more ordered, higher-symmetry austenitic phase. It's important to note that the phase transformation temperatures and the amount of deformation the alloy can recover are not fixed properties, but rather they can be altered within certain limits by changing the alloy's composition (the proportion of Nickel and Titanium) or by

thermomechanical processing.

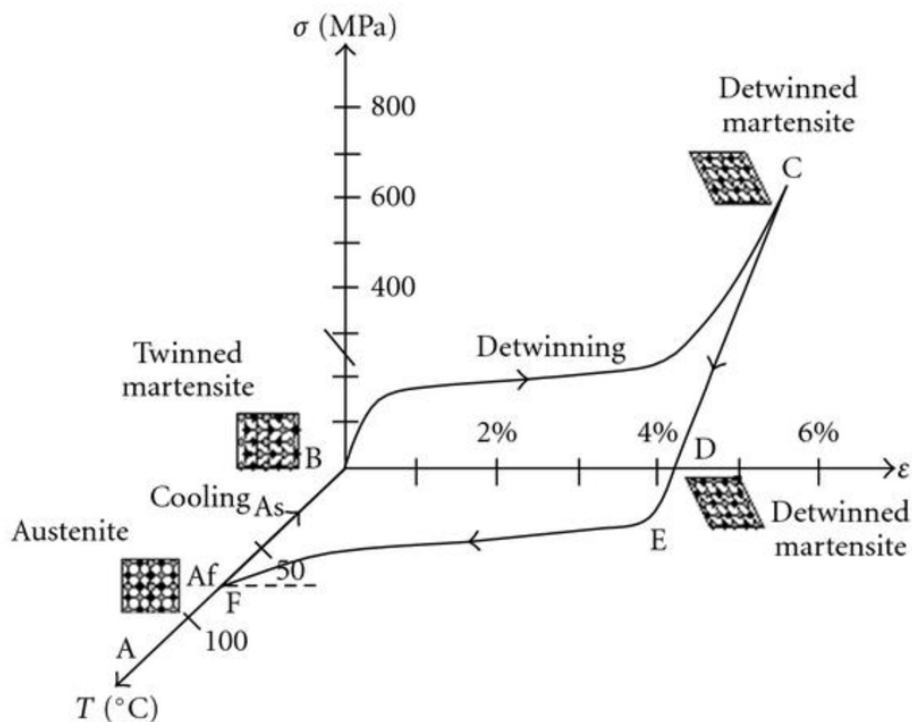


Figure 2.5: Stress-strain-temperature exhibiting the shape memory effect for NiTi alloy. The crystal changes are also related to temperature and stress. Source [28]. Adapted from [47].

Shape Memory Alloys have potential applications in the construction industry due to their unique mechanical properties (SME and superelasticity). The inherent ability of SMAs to remember their pre-deformed shape when subjected to thermal or mechanical loading makes them attractive for various applications.

- **Seismic Damping and Earthquake Mitigation:** SMAs, due to their superelasticity, are capable of undergoing large deformations and then returning to their original shape without permanent deformation. This makes them suitable for use in seismic dampers, which can absorb and dissipate the energy generated during an earthquake, thereby protecting the structure from damage. These dampers can be installed in the structural framework of buildings and bridges.
- **Self-healing Concrete:** SMAs are being explored for use in self-healing concrete. Here, SMA fibers are added to the concrete mix. When the concrete cracks, these SMA fibers can be activated (typically by heating, which can be achieved using electrical currents or even just ambient temperature changes), causing them to return to their original shape and close the crack, effectively 'healing' the concrete.[54]
- **Pre-stressed Concrete:** Pre-stressing is a technique used in concrete construction to overcome concrete's inherent weakness in tension. Traditionally, steel tendons are used to pre-stress the concrete, but these are susceptible to corrosion. SMAs, with their corrosion resistance and the ability to exert a compressive force on the concrete when heated, are being considered as a promising alternative.[70]

- **Structural Health Monitoring:** The unique properties of SMAs can also be used for structural health monitoring. For example, by measuring changes in the electrical resistance of an SMA element under load, it can provide data about the structural health of a building or bridge.[24]

However, while there is much potential, the use of SMAs in the construction industry is still relatively new, and there are still challenges to be overcome, such as cost, the need for heating systems in some applications, and a lack of standardized design codes for their use. More research is ongoing to address these issues and to better understand and utilize the unique properties of SMAs in construction applications.

2.3 Composite SMA-FRP rebars

As mentioned in the introduction of this thesis, while FRP reinforcement offers superior corrosion resistance and high strength-to-weight ratio compared to conventional steel reinforcement, FRP reinforced concrete structures are more flexible, brittle (i.e., less ductile), and have less capability to dissipate energy compared to their steel-reinforced counterparts. This lack of ductility and energy dissipation has hindered FRP reinforcement from being widely used for structural damping or dynamic applications such as reducing the vibration of bridge decks under traffic load or reinforcing structures in areas of moderate to severe seismic demand.[82]

To tackle these issues, some researchers have turned to hybrid FRP composites, also known as SMA Hybrid Composites (SMAHC). SMAHCs have been successful in damage suppression using both active and passive mechanisms. These mechanisms can absorb impact energy, accelerate the closure of fatigue cracks, and reduce crack propagation speed. SMAHCs exhibit higher ultimate strength and failure strain than baseline control materials. The performance of the SMAHCs is dependent on several factors, including the pre-strain, volume fraction, orientation, and position of the SMA fibers within the laminate. Increasing the pre-strain or volume fraction of SMA usually improves damage suppression.[9]

A possible hybridization is proposed by Wierschem, N., & Andrawes, B. (2010)[82], with SMA and FRP wires intertwined in an epoxy matrix (See Figure 2.6). It was found that replacing 25% (by volume) of the fiberglass in a GFRP composite with superelastic SMA wires increases ductility and energy dissipation by 64% and 2.6 times, respectively. Even after fiberglass rupture, the SMA-FRP specimen maintained 28% of its strength up to 3.8% strain, compared to 1.63% strain for GFRP specimens. The paper also suggests that the ductility of SMA-FRP composites is highly dependent on the difference between the SMAs' phase transformation strain and the rupture strain of conventional fibers. SMA-FRP composites reinforced with high rupture strain fibers such as S-Glass and aramid dissipated 180% more energy than composites reinforced with moderate rupture strain E-Glass fibers. However, fibers with a higher modulus of elasticity decreased the composite's ductility.

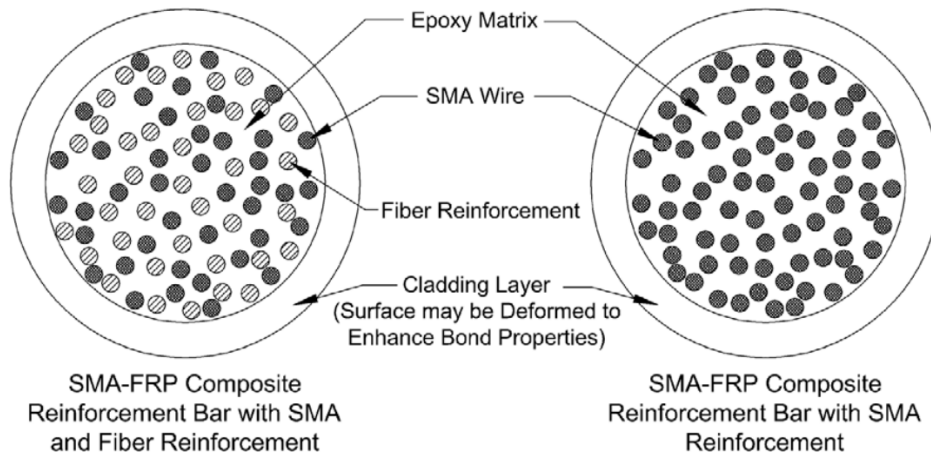


Figure 2.6: Schematic drawing of SMA-FRP rebar cross section. [82]

This is however out of the scope of this thesis that focuses on another way of combining SMA and FRP using mechanical couplers and will therefore not be discussed further, even though it could prove to be a better compromise than the solution developed in the following chapters and should thus not be overlooked in future studies on the same subject.

2.4 Mechanical couplers

A mechanical coupler, often used in the construction industry, is a device used to connect two reinforcing rebars end-to-end. It's an alternative to the traditional method of overlapping rebars and tying them together. They create a joint that is at least as strong as the bars themselves, enabling the force to be transferred between the bars. They come in various types and designs, depending on the manufacturer and specific application, but most work by clamping onto the grooves of the rebars, either by threading the ends of the bars and screwing them into the coupler or by using a crimping or swaging technique to join the bars.

They are usually used to connect two rebars of the same diameter and of the same material to extend their length but they can also be used to connect bar of different diameter, as a transition between different structural elements or when design specification changes along the element. These second types of coupler are often referred to as reducing couplers or transitional couplers. (See Figure 2.7)

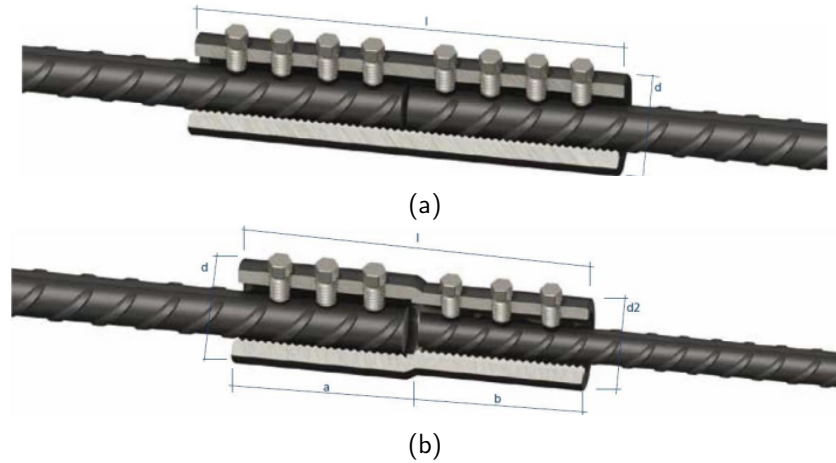


Figure 2.7: Traditional mechanical coupler (a) and reducing coupler (b) from Plaka [62]

Another distinction can be made among these kinds of screw-lock couplers between the regular single barrel coupler (SBC), and the double barrel transition coupler (DBTC), where the bars meet in parallel instead of head to head. (See Figure 2.8)

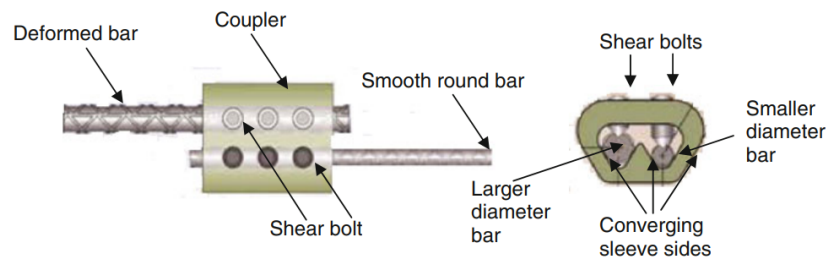


Figure 2.8: Double barrel zap transition screw-lock coupler. Source : Alam et al.[6], Adapted from Barsplice Products Inc.[11]

However, they are not limited to connecting steel rebars and can even be used for connecting different types of rebars, including GFRP and SMA. Previous tests at the Mechanical testing, structures, and civil engineering laboratory (LEMSC) at UCLouvain were conducted with a traditional mechanical coupler from Plaka [62] (See Fig 2.9) to connect steel and SMA. The heads of the screws used to connect the SMA were flattened (Figure 2.10) and 5 of the 6 screws were used for the SMA connection. To ensure a proper connection to the steel rebar despite only having one screw, the couplers were welded to the rebar.

This configuration allowed the mechanism to reach the plastic bearing of the steel as well as the pseudo-elastic bearing of the SMA, ensuring significant energy dissipation before the first sliding of the coupler at 68.45kN (See Figure 2.11). It was judged sufficient to ensure the transfer of longitudinal forces between the steel reinforcements and the SMA reinforcements because of the SMA reaches his first 'plateau'. An unknown still remains regarding its cyclic response.

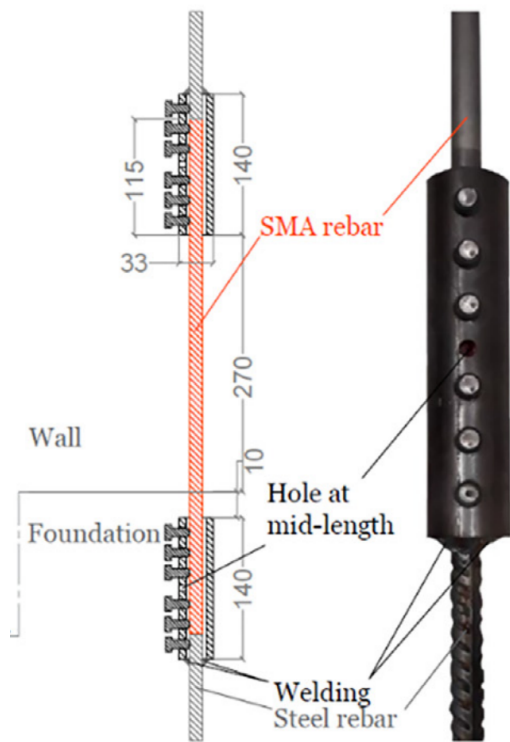


Figure 2.9: SMA-Steel Mechanical coupler with 5 in-line flattened screws for the SMA and 1 screws combined with welding for the steel. From Almeida et Al. [8]



Figure 2.10: Difference between a normal screw and a flattened-head screw. From Herzezel et al.[35]

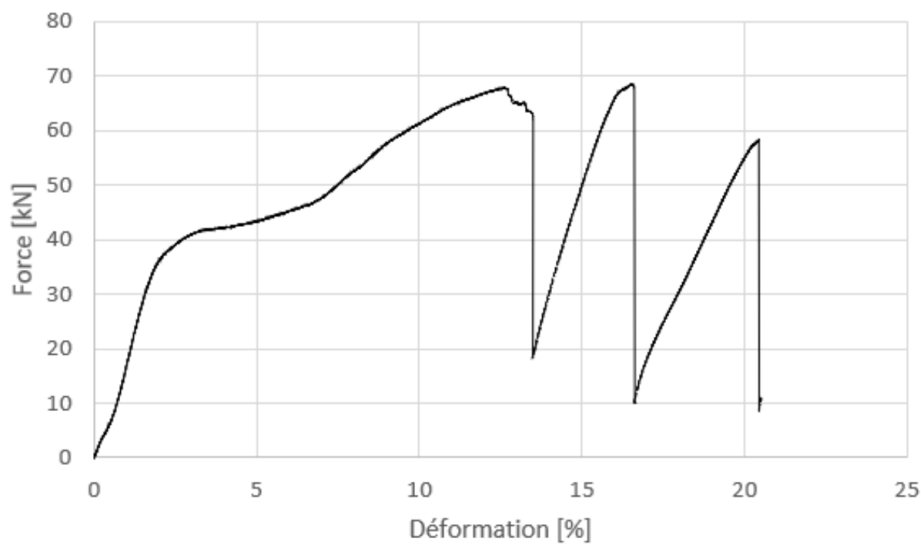


Figure 2.11: Force-Displacement results of the SMA-Steel mechanical coupler (See fig2.9). From Herzezel et Al. [35]

Alam et al.[6] conducted an exploratory investigation on mechanical couplers to connect SMA and FRP rebars. A first exploration of the steel-SMA connection was conducted. 4 cases, illustrated in Figure 2.12, were considered :

- Case 1: The coupler was used without modification. During the test, the shear bolts on the SBC embedded into the SMA bar and created notches, which led to a decrease in the SMA's strength and resulted in premature failure after four load cycles at a strain of 1.48%.
- Case 2: An additional metal strip was added underneath the screw bolts over the SMA bar to prevent direct contact and the creation of notches in the SMA bar. However, this design was ineffective as the SMA bar slid out of the coupler before reaching its yield strength.
- Case 3: Extra rows of holes were drilled into the coupler at a 45-degree angle, with the screw ends being flattened to prevent the creation of notches. This configuration allowed the system to endure eight load cycles before failure, and the SMA bar reached a maximum strain of 3.22% with a residual strain of 0.36%.
- Case 4: Similar to Case 3, but the extra rows of holes were drilled at a 60-degree angle. Additionally, the screw ends were further flattened. This design was the most successful, with the connection enduring eleven load cycles before failure. The SMA bar experienced a maximum strain of 6.38% and a residual strain of 0.73%.

Despite the relative success of Case 4, failure still occurred due to high stress concentration and micro-crack formation at the point of contact of coupler screws with the SMA.

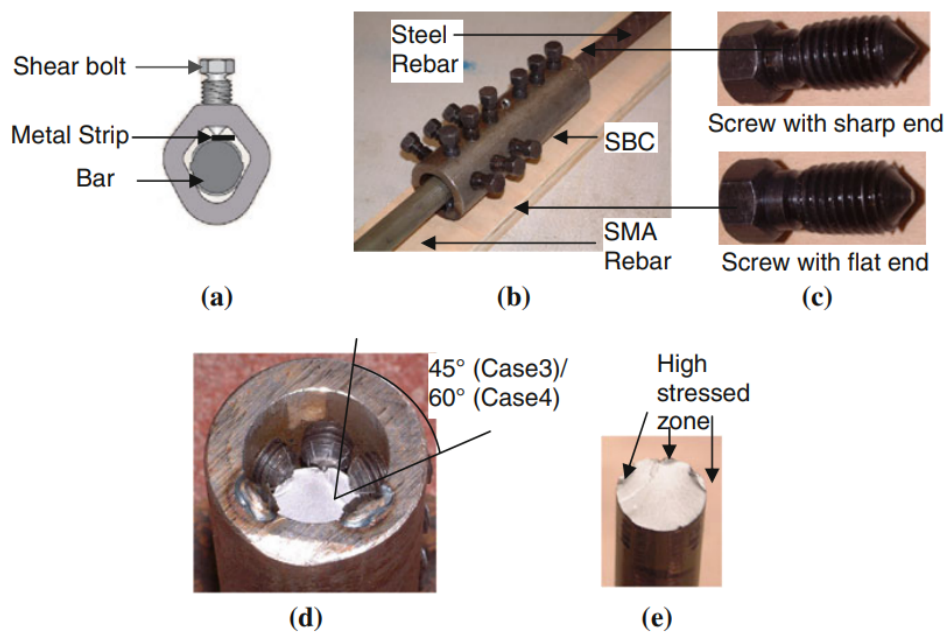


Figure 2.12: (a) Coupler arrangement of SMA bar in the coupler (Case 2), (b) coupler for splicing SMA and steel bar (Case 3), (c) screws with sharp and flat ends used for splicing steel and SMA bar, respectively (Cases 3 and 4), (d) arrangement of screws in SBC at an angle of 45 (Case 3) or 60 (Case 4) from the centre row, also showing the failure of SMA bar at the connection (Case 4), and (e) SMA bar after failure from the coupler (Case 4). From Alam et al. [6].

Finally, Alam et al also tried to connect steel with FRP. However, connecting FRP bars to SMA or steel bars presents a significant challenge. FRP is brittle and cannot handle the normal forces exerted by the screws of the couplers used for steel and SMA bars. Thus, different approaches, illustrated in Figure 2.13, were tested:

- **Epoxy adhesive:** Two types of epoxy were used to connect the FRP and SMA bars. An epoxy-only connection was tested first, and it was found that the second epoxy (Concresive) allowed for higher tensile loads than the first (Epogrip). However, there was a sudden drop in load resistance after reaching the peak load, but 80% of the tensile load could be recovered as the FRP bar continued to slip.
- **Epoxy adhesive with screws:** In the second case, an adhesive connection was used with screws that had been flattened to apply normal forces on the FRP surface without damaging the fibers. When testing this method on sand-coated FRP bars, it was found that the first epoxy (Epogrip) resisted a higher tensile load than the second (Concresive). However, the load resistance dropped and was not recoverable, resulting in the bar pulling out of the coupler. When epoxy and flattened screws were used, the performance improved, with no sudden failures.
- **Increasing the development length:** This involves extending the length of the FRP bar inside the coupler. In tests using two pipes of different lengths (250 mm and 340 mm), the longer pipe significantly improved the pullout resistance of the spliced FRP bar, with a 38% increase in load resistance. The longer pipe also led to a more gradual failure, as opposed to the sudden anchorage failure observed in the shorter pipe.

The results thus indicate that the optimal method for splicing brittle FRP bars with more ductile materials (like SMA or stainless steel) involves using epoxy adhesive, flattened screws, and increasing the development length.



Figure 2.13: Epoxy adhesive connecting FRP bar to (a) mechanical coupler, (b) 250 mm long steel pipe and (c) 340 mm long stainless steel pipe (screwlock-adhesive type coupler) with their ends connected to steel bar. From Alam et al.[6].

2.5 Distributed Fiber Optic Sensors (DFOS)

Distributed Fiber Optic Sensors (DFOS) are sensors that utilize optical fibers to sense a variety of parameters such as strain, temperature, and vibration along the entire length of the fiber. They offer continuous, real-time monitoring, making them suitable for a range of applications in various industries, including construction and research.

DFOS work based on the scattering phenomena that occur naturally in optical fibers. When light, usually from a laser, is sent down an optical fiber, a small amount of it is scattered back toward the source. By analyzing the properties of this backscattered light, it is possible to measure the parameters of interest. The three main types of scattering phenomena that DFOS sensors exploit are Rayleigh scattering, Brillouin scattering, and Raman scattering. (See Figure 2.14)

- Rayleigh-based DFOS: These sensors utilize the naturally occurring imperfections in the fiber to scatter light. By measuring the spectral shift in the scattered light, it is possible to detect strain or temperature changes.
- Brillouin-based DFOS: Brillouin scattering is a result of interactions between the light and sound waves in the fiber. The shift in frequency (Brillouin frequency shift) is dependent on the strain and temperature, which can be used to measure these parameters.
- Raman-based DFOS: In Raman scattering, the scattered light experiences a change in wavelength. This change is dependent on the temperature of the fiber, allowing for temperature measurements.

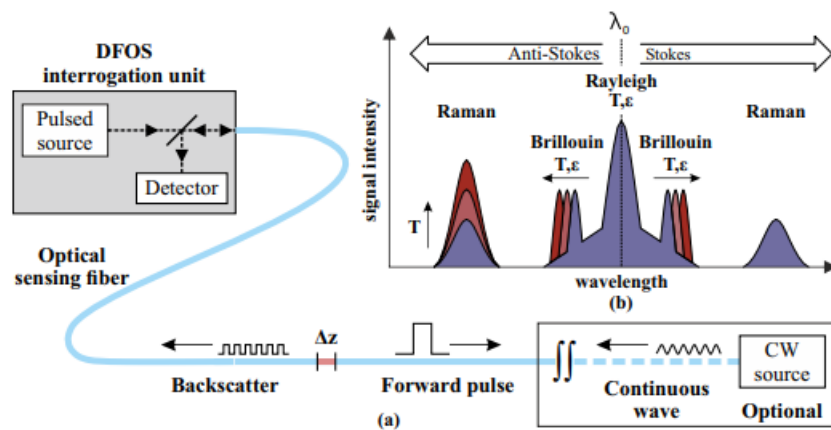


Figure 2.14: Distributed fiber optic sensing techniques : (a) Basic scheme of sensing setup. (b) Different scattering components in optical glass fibers.[56]

Rayleigh scattering utilizing the Optical Frequency Domain Reflectometry (OFDR) approach allows for precise, high-resolution, single-ended measurements, but with a limited sensing range. Brillouin sensors can cover longer distances but with spatial constraints and extended measurement times. These systems can be further categorized into loop setups (Brillouin Optical Time/Frequency Domain Analyzer) or single-ended configurations (Brillouin Optical Time/Frequency Domain Reflectometry), impacting the repeatability of measurements. Therefore, choosing the right Distributed Fiber Optic Sensors (DFOS) technology requires balancing between sensing range, spatial resolution, and strain sensing accuracy, tailored to the specific needs of the civil engineering application.[56]. The characteristics of these different DFOS technology are summarized in Figure 2.15.

In the construction industry, DFOS are extensively used for structural health monitoring of infrastructure such as bridges, tunnels, and buildings. They can provide data on strain, deformation, and temperature over time, allowing engineers to monitor the condition of the structure continuously and identify any potential issues early.

In the research sector, DFOS are used in a variety of innovative ways. For example, they can be embedded into materials and structures during construction to provide real-time data on how these structures behave under various loads. This can provide valuable insights into the material properties and help in the design of more resilient structures. Moreover, DFOS are being used to develop smart infrastructure systems, where real-time monitoring allows for predictive maintenance and proactive management of infrastructure.

These optical fibers are lightweight, flexible, and can be easily attached to structures for monitoring purposes. However, their fragility makes them susceptible to potential damages, requiring protective layers for successful applications. A variety of DFOS cables are commercially available, each designed for specific applications, with coating ranging from simple polyamide or PVC to FRP or even large stranded steel tube (See Figure 2.16). This helps ensure the sensor's integrity during installation and monitoring. These cables' outer layers can be structured to ensure a strong bond with the surrounding material, such as concrete or grout. However, these protective layers can reduce flexibility, making them less suitable for surface applications or instances requiring tight cable routing.[56]

Temperature changes can also affect fiber optic strain readings, therefore, it's advisable to install an additional sensing fiber not influenced by mechanically induced strain as it can be used to compensate for temperature effects and provide more accurate stress behavior readings of the monitored structure.

Given that civil infrastructure is often designed for lifetimes of up to 150 years, the longevity of the DFOS inside or along concrete structures is crucial. However, assessing the durability of DFOS cables is challenging due to insufficient knowledge on long-term stability and the absence of standards. Even more so because the lifespan of the optical fiber not only depends on the mechanical properties but also on external factors such as extreme temperatures or the presence of hydrogen, which can cause major attenuations along the fiber over time. To mitigate this, sensing cable layers can offer a degree of protection, and for harsher environments, carbon-coated fibers have been shown to withstand high hydrogen and radiation exposure.[56]

Scattering	Rayleigh		Brillouin	
Sensing technique	OFDR	TW-COTDR	BOTDA/BOFDA	BOTDR/BOFDR
Configuration	Single ended		Loop	Single Ended
Commercial manufacturers	Luna, Sensuron	Neubrex	fibrisTerre, Omnisens, Febus, OZ Optics, Neubrex	
Sensing range	70 m (2 km)	20 km	up to 80 km	up to 100 km
Spatial resolution	≤ 10 mm	2–20 cm	≥ 20 cm	≥ 100 cm
Typ. strain repeatability	≤ 1 $\mu\text{m}/\text{m}$	≤ 0.5 $\mu\text{m}/\text{m}$	≥ 2 $\mu\text{m}/\text{m}$	≥ 20 $\mu\text{m}/\text{m}$
Typ. measurement duration	<10 s	≤ 10 min	3–60 min	

Figure 2.15: Characteristics of different distributed fiber optic strain sensing systems[56]

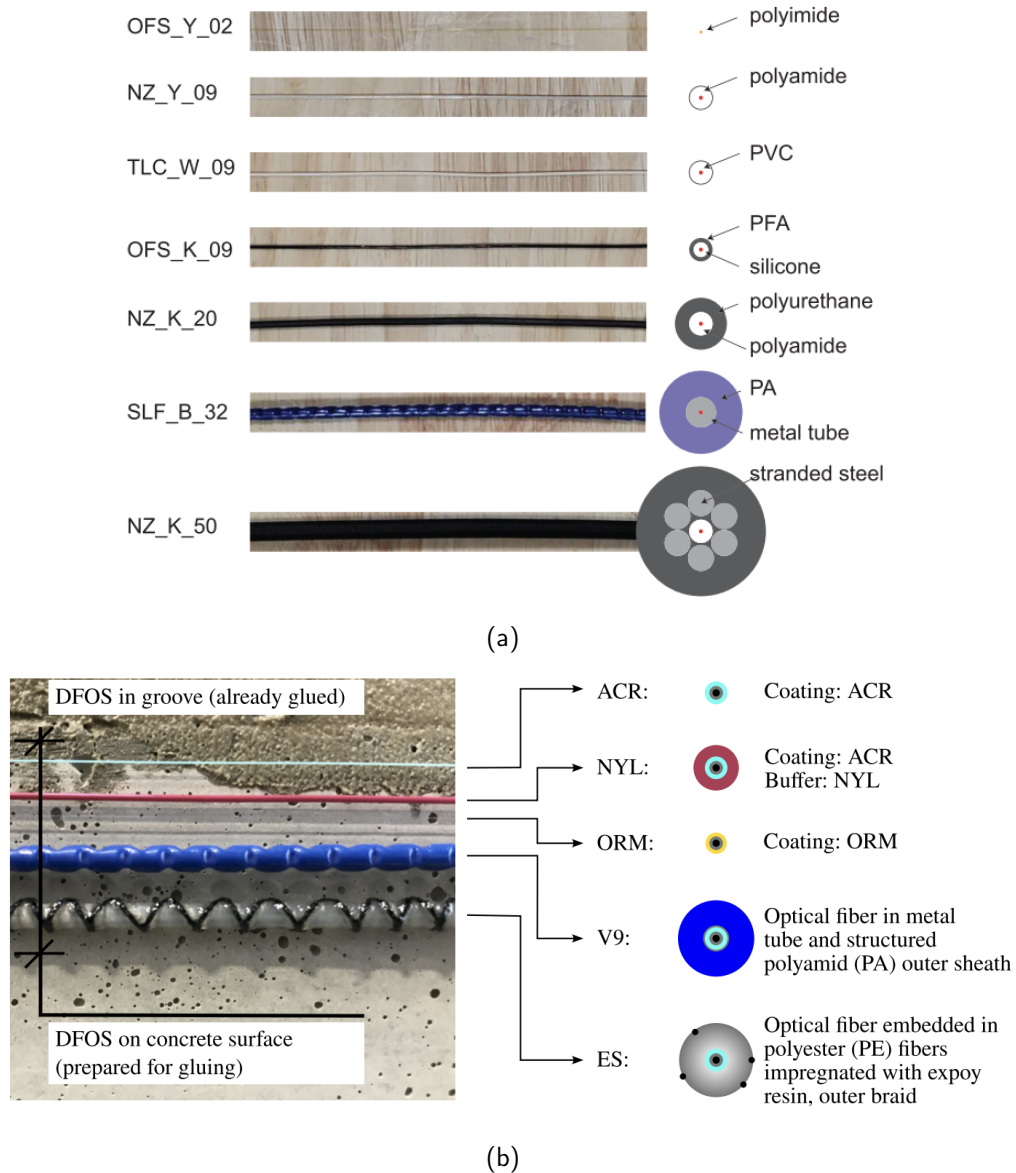


Figure 2.16: Different types of DFOS coating tested in (a) [87] and (b)[34]

2.6 Experimental studies on alternative reinforcement for concrete beams

The primary focus of this thesis being on RC beams, a summary of studies centered on beams reinforced with materials other than steel is provided. However, it's also valuable to highlight research on other types of reinforced concrete members, especially the work of Muntasir et al.[58] which investigates concrete columns reinforced with a combination of GFRP and SMA bars, using a configuration of mechanical couplers similar to the one used in this thesis.

2.6.1 GFRP and hybrid Steel-GFRP RC beams

To the author's knowledge, the first experimental study on GFRP reinforced concrete beams was conducted in 2000 by Toutanji & al.[79]. A few years later, in 2008, Tavares et al.[77] studied the

behaviour under a four-point bending test of beams with GFRP longitudinal reinforcement and steel stirrups compared to a control steel-only reinforced beam.

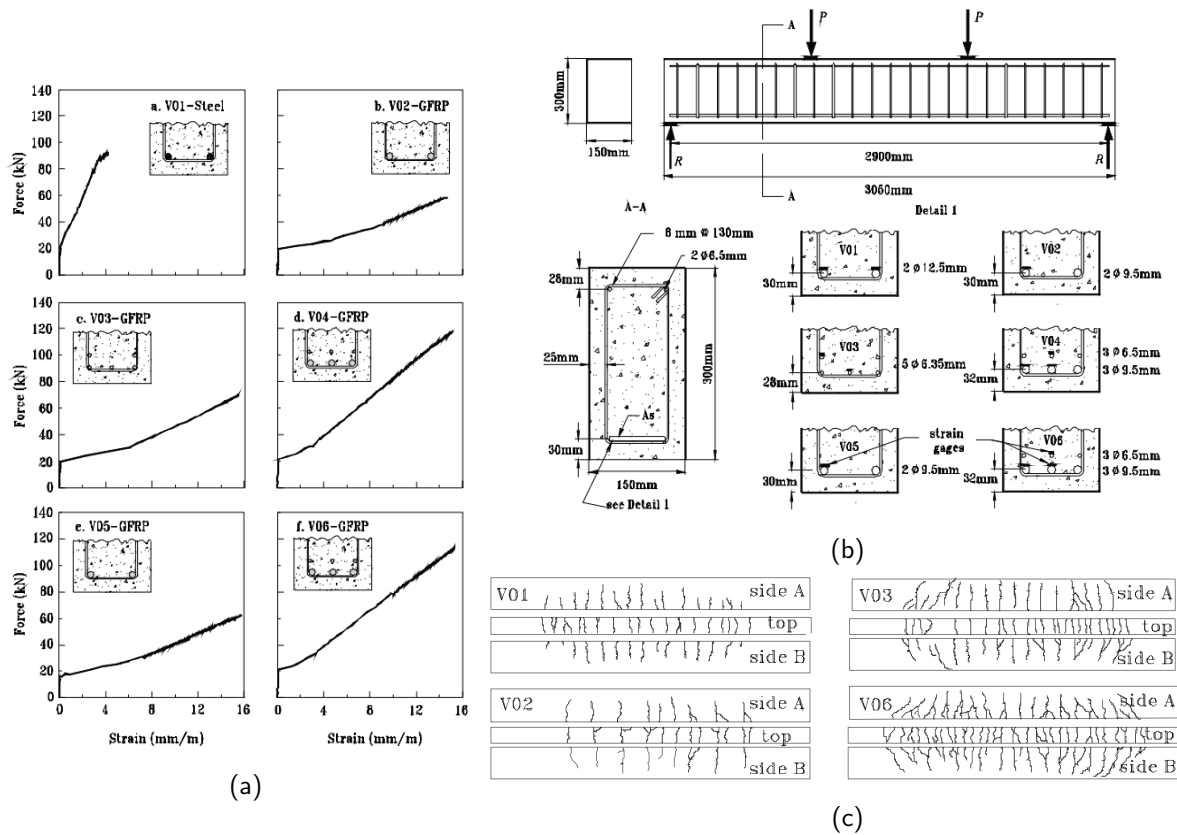


Figure 2.17: (b) Reinforcement setup in the beams tested by Tavares et al.[77], (a) Force-Strain results, (c) Cracking patterns

They tested six specimens (See Fig 2.17b) and measured force versus longitudinal reinforcement strain, as displayed in Fig 2.17a. These tests identified flexural failure in all specimens due to concrete compressive crushing or rupture of GFRP bars. Concrete cracking was identified at a load level of about 20 kN in all specimens. It was noted that GFRP bars showed higher strain at failure than the steel reinforcement bars in the control beam and exhibited a high deformation capacity. The steel-reinforced beam showed less cracking than the GFRP-reinforced beams.(See Fig 2.17c) They observed that the number of bars played an essential role in controlling concrete cracking, with beams designed with smaller diameter GFRP bars showing smaller, more distributed cracks.

The study showed that the GFRP-reinforced beams designed in terms of ultimate limit state did not reach a flexural capacity similar to that of the steel-reinforced beam. The experimental results showed that the GFRP-reinforced beams reached a maximum capacity of between 27.23 kNm to 31.58 kNm, less than the steel-reinforced beam's 42.43 kNm.

The same year, Habeeb & al. [33] conducted similar experiments with 2 simply and three continuously supported GFRP reinforced concrete beams and a continuously supported steel reinforced concrete beam for comparison purposes. Once again, all the beams used steel stirrups. The beams were all 200mm in width and 300mm in depth.

The design of the simply supported beams targeted two different modes of failure: GFRP bar rupture and concrete crushing (See 2.18). The GFRP bar rupture was achieved by using a reinforcement ratio less than the balanced reinforcement ratio as defined in the ACI 440.1R-06 guidelines [32], while the concrete crushing was accomplished by using a reinforcement ratio greater than this balance. Three GFRP-reinforced continuous beams were designed with different top and bottom layer reinforcement combinations. (See 2.19)

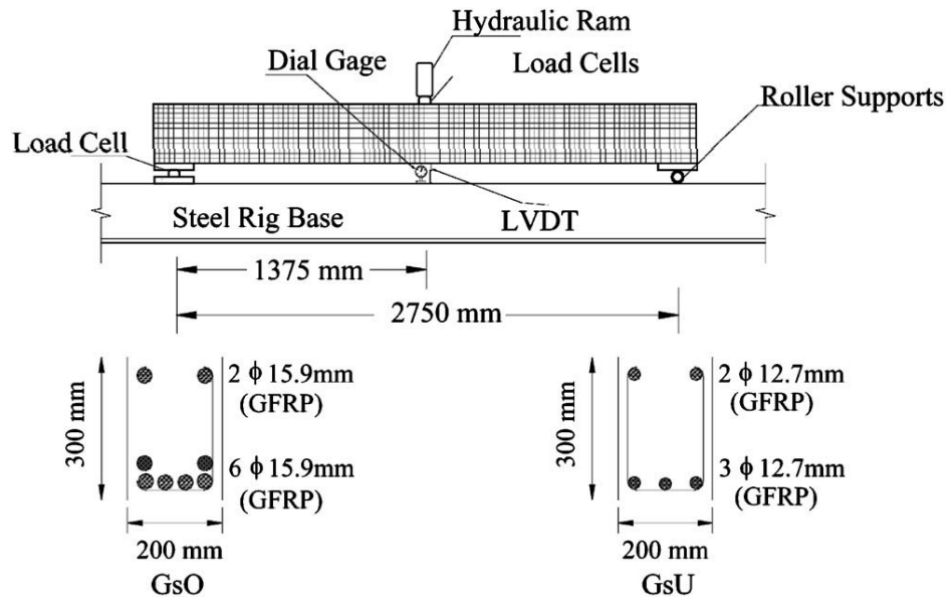


Figure 2.18: Experimental loading system and cross-sectional details of simply supported beams [33]

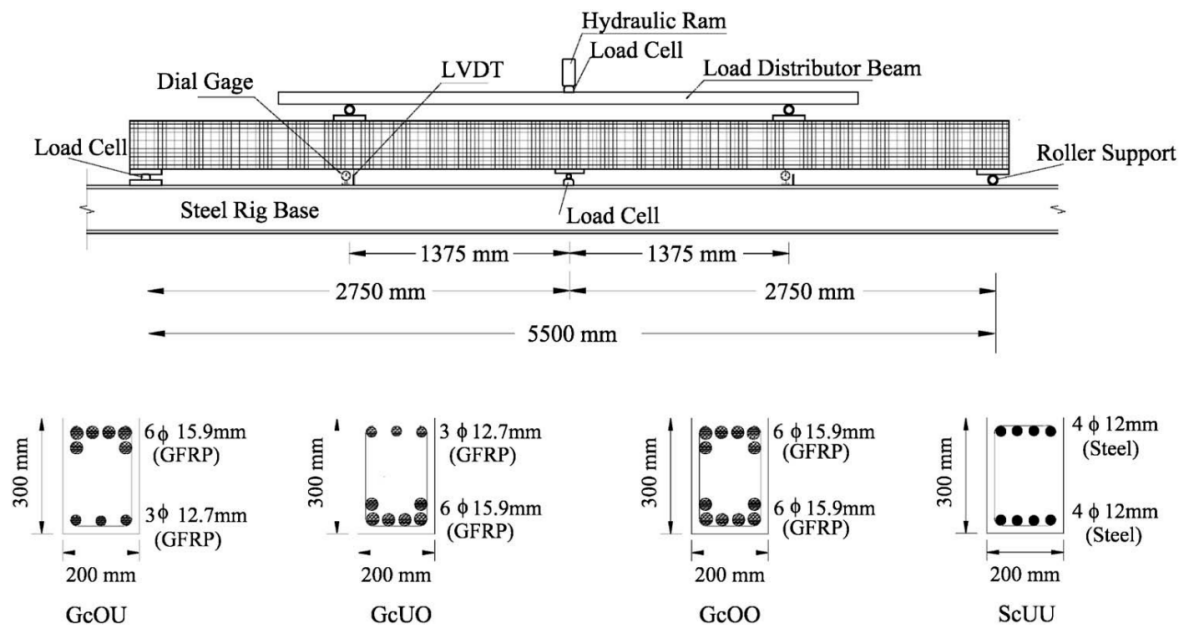


Figure 2.19: Experimental loading system and cross-sectional details of continuous supported beams [33]

Four different modes of failure were observed in the beams.

- **Bar Rupture:** This failure was observed in Beams GcOU and GsU, which were under-reinforced with GFRP bars at the bottom layer. This led to the rupture of GFRP reinforcement at the midspan failed section, before the exhaustion of the ultimate concrete strain.
- **Concrete Crushing:** This was noticed in Beams GcUO and GsO, which were over-reinforced with GFRP bars at the bottom layer. As a result, the concrete at the midspan section crushed before the GFRP reinforcing bars could reach their ultimate strain. In the continuous Beam GcUO, wide cracks appeared over the middle support section before the concrete crushing occurred.
- **Concrete Crushing Combined with Shear Failure:** This was evident in Beam GcOO, which displayed a combination of flexural concrete crushing and diagonal shear cracks that resulted in the sudden collapse of the beam. This beam had a higher amount of GFRP reinforcement, enabling it to resist more compression before the shear force led to failure.
- **Conventional Ductile Flexural Failure Mode:** This was the case for the steel-reinforced concrete Beam ScUU. The tensile steel reinforcement yielded, followed by concrete crushing at both the middle support and midspan sections.

The conclusion about the crack patterns and the strain capacity were similar than those of the previous study. They added that over-reinforcing the bottom layer of simply and continuously supported concrete beams with GFRP bars could enhance load capacity, control deflection, and delay crack propagation at the midspan section of such beams and that increasing the top layer reinforcement of continuously supported GFRP reinforced concrete beams does not significantly improve the load capacity and deflection reduction.

In 2014, Johnson D.[44] conducted a extensive experimental investigation where 24 large-scale GFRP reinforced beams with the same overall dimensions and a longitudinal reinforcement ratio of 1% were constructed and tested. The testing was conducted in a monotonic displacement-controlled manner under a 3-point set-up with a constant shear span. The 24 beams are subdivided into 6 distinct series based on the type of reinforcement used or concrete strength. In addition to keeping the reinforcement ratio and geometry constant for all the beams, the bar location and clear cover to reinforcement was kept constant as well.

The conclusion of those test were :

- **Over-reinforcement in Flexure:** With careful reinforcement detailing, flexural failure due to concrete compression can be a safe and deformable failure mechanism with energy capacities similar to conventionally reinforced beams.
- **Anchored Confining Reinforcement:** Providing suitably anchored confining reinforcement for the flexural compression zone is suggested. This is typically achieved by using closely spaced closed stirrup shear reinforcement.
- **High Tensile Strength GFRP Bars:** Using GFRP bars with high tensile strength reduces the risk of tensile rupture shortly after crushing during the confined response.
- **Normal Strength Concrete:** Using normal-strength concrete instead of high-strength concrete is advisable. This is because confinement tends to be more effective in normal-strength concrete, leading to reduced strains in the transverse reinforcement. As a result, there are fewer wide cracks under service loads.
- **Capacity Design in Shear:** Over-reinforcing in flexure provides a desirable flexural failure mechanism, but it's crucial to also control failure in shear.

- **Combination of Conventional Steel and GFRP Reinforcement:** Considering the combination of conventional steel and GFRP reinforcement can provide an optimal balance of crack and deflection control at service loads, ultimate strength, deformability, warning of failure, and durability.

2.6.2 Hybrid SMA-Steel RC beams

In a 2022 publication, Raza & al.[65] conducted an extensive state of the art review of Shape Memory Alloy reinforcement for strengthening and self-centering of concrete structures, including a review of all the different hybrid SMA-Steel RC beams.

The review cites several studies exploring the use of SMAs for flexural strengthening of RC beams. Rojob and El-Hacha's [68] study revealed that RC beams reinforced with NSM Fe-SMAs demonstrated superior performance under low levels of fatigue load. However, at high fatigue loads, the bond between SMA rebar and grouting material started degrading, leading to rupture after 5.5 million load cycles. Other researchers, such as Hong et al. [39], Strieder et al. [76], and Sinha et al. [72], experimented with different SMA types, activation methods, and strengthening techniques. Their results indicated that the superelasticity and shape memory effect of SMAs could significantly enhance the strength, stiffness, and load-carrying capacity of RC beams. They found that SMA reinforcement ratios between 0.1%–0.5% were effective and suggested further investigations to determine the most effective installation techniques.

As for shear strengthening, the review mentions the work of Mas et al. [50], Soroushian et al. [75], Rius et al. [67], Montoya-Coronado et al. [57], Cladera et al. [17], and Czaderski et al. [21]. These researchers employed various forms of SMAs like spirals, wires, rods, and strips, which were activated at different temperatures. The studies found that SMAs can significantly improve the shear strength and ductility of RC beams, restore damaged RC beams' load-carrying capacity, reduce crack widths, and provide effective retrofits for pre-cracked beams. Despite some drawbacks, such as loss in recovery stresses due to geometric imperfections and underperformance in ductility due to certain anchoring methods, SMAs promise significant enhancements for RC structures.

Overall, the review suggests that SMAs, due to their unique SME, could effectively be used to improve the performance of RC structures, especially in terms of shear and flexural strengthening. However, it underlines the need for more research to determine the most effective reinforcement ratios, activation temperatures, and installation techniques for different types of SMAs.

In a previous master thesis here at UCLouvain De Kerautem B.[46] conducted experiments involving three RC beams, each measuring 2m x 0.19m x 0.15m. The first beam served as a control sample, while the other two were strengthened using a 10cm Nickel-Titanium (NiTi) shape memory alloy (SMA) at the mid-span, likely at the expected location of the plastic hinge. The beams were tested under three-point bending.

The SMA beams exhibited greater initial stiffness compared to the RC control beam. However, for a given imposed displacement, the maximum force exerted by the SMA beams was less than that of the control RC beam. Moreover, in the SMA beams, strains were highly concentrated in the SMA. Therefore, the SMA section will significantly dictate the flexural behavior of the beam. This might indicate that the SMA's properties could potentially dominate the beam's overall performance, and specific attention needs to be paid to this area in the design process.

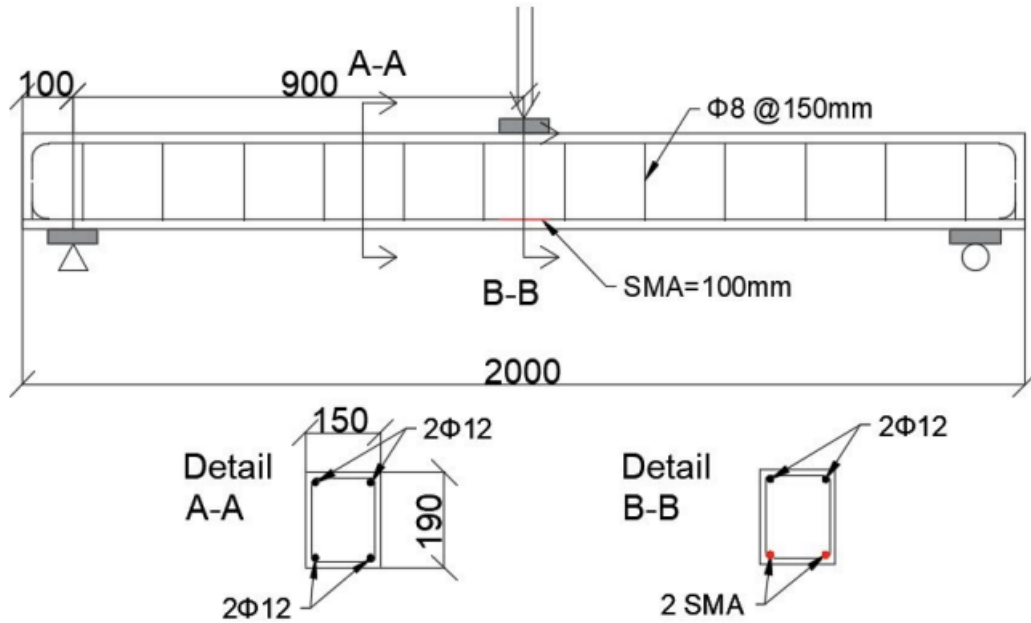


Figure 2.20: Cross section and test setup of the hybrid Steel-SMA beam used as a baseline for this master thesis.[46]

The welded connections between steel and SMAs had a brittle failure. This failure occurred much earlier in the loading protocol than the estimated resistance of the connection predicted. Despite the failure of this connection in these tests, it is believed that with additional research and testing, promising results could be found for this type of linkage. As the material is still available at the lab, this test setup will be the base of this thesis final chapter.

2.6.3 Hybrid SMA-GFRP RC beams

To the author's knowledge, the only experimental research on combined GFRP-SMA rebars for reinforced concrete in beams involved composite SMA-FRP rebars. As mentioned previously, these composite materials are very promising but out of scope for this master thesis.

CHAPTER

3

GFRP REBARS IN TENSION : NORMS AND EXPERIMENTAL TESTS

In this chapter, three tests are conducted to characterize the GFRP rebars for subsequent experiments. Given that GFRP was not well-understood in the laboratory, these tests were essential to gain a comprehensive understanding of its preparation and utilization for various tests. This includes its performance in the testing machine's jaws and the proper method to cut it cleanly. Additionally, the chapter covers a method to install DFOS on various coatings. Details of this procedure are provided in Annex A.

3.1 GFRP rebars

Our tests were conducted on two different GFRP rebar samples, illustrated in Figure 3.1. The first set was provided by the company IRT-M2P [42]. These are 13 mm diameter bars with a fairly smooth surface. As it will be explored later, this coating is a critical factor for the development of mechanical couplers.

The second set of GFRP rebars used in the caraterisation tests were "Glasspree® TP Round Bars" from Sireg [73], with a diameter of 12mm. Their rebars feature a rectangular, ribbed surface profile and are composed of two different materials:

1. Glass : Direct "E-CR" Roving, with a tensile strength exceeding 2000 MPa, elongation of over 4 percent, and an elastic modulus above 70 GPa.
2. Acrylic resin : Tensile strength above 50 MPa and breaking elongation of more than 2 percent.

These round Glass Fiber rebars are created via the pultrusion process. In this method, fibers initially wound on a reel are unwound and impregnated with a thermoplastic resin. This resin is then heated to trigger polymerization, achieving the desired shape. This machining process ensures thorough impregnation of the fiber by the polymer, enabling continuous production of a precise rebar diameter,

which can subsequently be cut to the desired size.

Sireg provides the following characteristics for their 12mm diameter GFRP rebars:

- Characteristic Tensile Strength $f_{fk} = 977$ MPa
- Tensile Modulus of Elasticity $E_f = 53$ GPa
- Characteristic Ultimate Strain $\epsilon_{ft} = 0,0185$ mm/mm

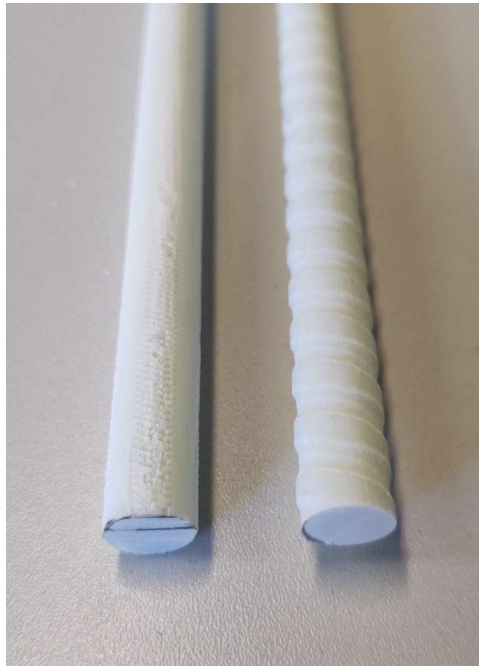
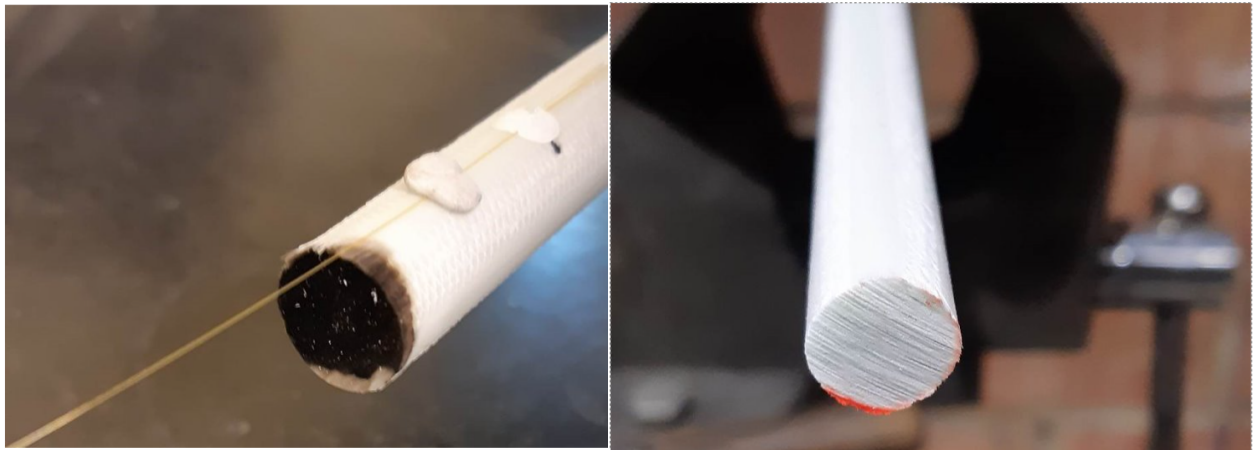


Figure 3.1: Coating of the IRT-M2P GFRP rebar (left) and the SIREG rebar (right)

3.2 GFRP preparation

Several experiments were conducted to identify an effective method for cutting FRP bars. The initial test involved the use of a 'Speed cutting disc for stainless steel' from Würth [84] on an angle grinder. Unfortunately, this approach yielded unsatisfactory results. The cutting process was slow, leading to excessive heat build-up in the GFRP rebar, causing the polymer to melt and the fibers to sustain damage. (See Figure 3.2)

The second trial used an angle grinder equipped with a carbide-tipped disc, also from Würth [84], specifically designed for composite materials. This highly durable disc supports fast rotation speeds, enabling rapid and clean cuts of the GFRP framework. Both the fibers and the polymer matrix remained undamaged after the cutting process. (See Figure 3.2)



(a) Würth Steel cutting disc

(b) Würth Carbide-tipped disc

Figure 3.2: GFRP cutting with different discs

3.3 Norms and Anchors

The whole tensile tests of this thesis were carried out in compliance with the ISO 10406-1 standard [43]. Where this standard lacked specificity (mainly on the design of the anchors itself, their length and width), the American standard ASTM D7205 was used [10], illustrated in Figure 3.3.

To fulfill these standards, specific anchors were constructed to fit into the machine's grips. These were composed of a central steel tube and two 3D-printed components for aligning the reinforcement within the anchor. An epoxy adhesive was then applied to glue the reinforcement to the steel part. The tube's length was sufficient to prevent the reinforcement from slipping, while its thickness allowed it to withstand the machine grip's pressure without damaging the rebar. (See Figure 3.4 and Figure 3.6)

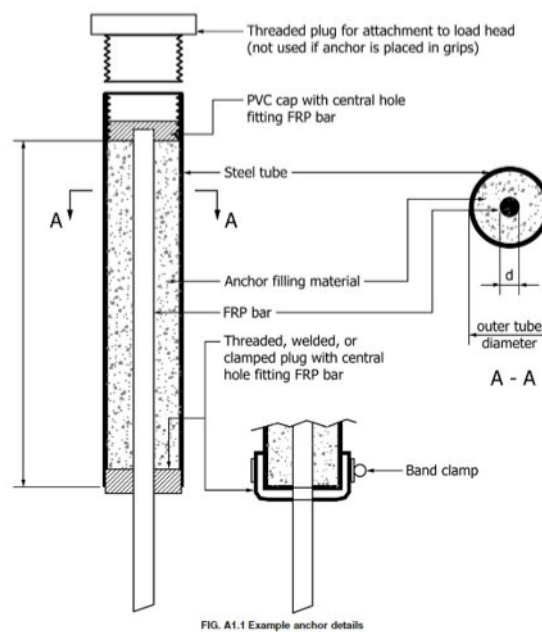


Figure 3.3: GFRP Anchorages for tensile test : ASTM D7205/D7205M06

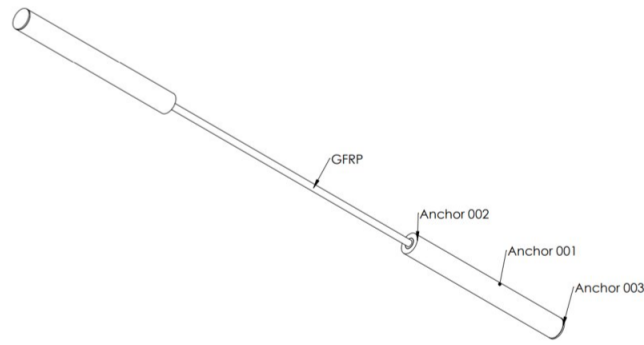


Figure 3.4: GFRP anchoring scheme for tensile test

In Table 3.1, the minimal anchor dimensions for the 13 mm diameter GFRP rebar have been provided. The anchors were made according to those values, as illustrated in Figure 3.5. The rebar's free length however was extended to 700mm in order to allow for enough space for a 500mm extensometer and sufficient distance between the extensometer edges and the anchors to avoid any parasite strain from those interfaces.

	Minimal dimension
Anchorage Length	380 mm
External diameter	42 mm
Internal diameter	21 mm
Rebar free length	520 mm

Table 3.1: Minimal anchorage dimension for a 13mm GFRP rebar



Figure 3.5: Anchorages dimensions for 13mm GFRP rebar

The epoxy adhesive used is the HIT-RE 500 from Hilti [36]. To enhance adhesion, the inner surface of the steel parts was sandblasted prior to applying the glue. The reinforcement was bonded by sequentially injecting epoxy at various heights of the anchorage through holes purposely made in the steel tube, as illustrated in Figure 3.6. This process ensured even distribution of the resin inside the tube along its entire length, a method that is more effective than the traditional approach of filling the tube with epoxy and then inserting the rebar.

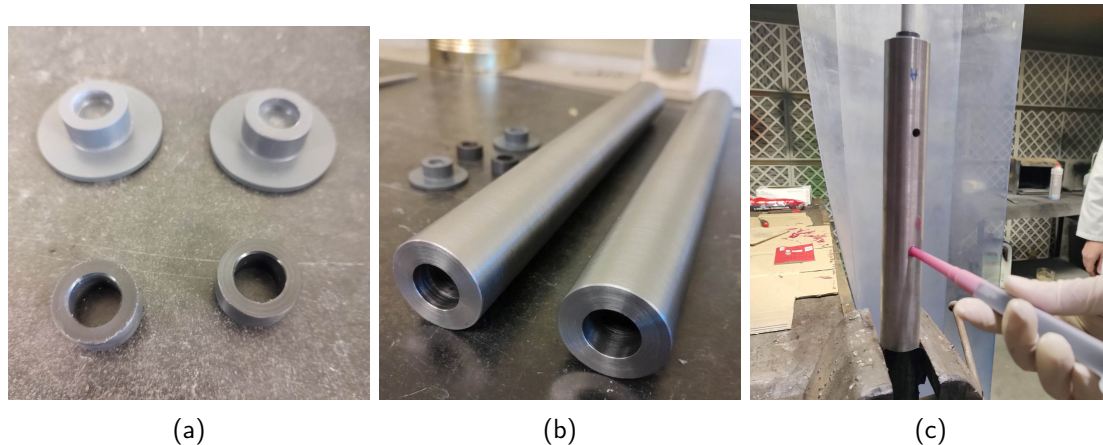


Figure 3.6: (a) 3D components and (b) steel tubes for GFRP anchorages. (c) Epoxy injection in anchorages

3.4 Instrumentation

3.4.1 Extensometer

Various extensometers were utilized across the three tests. For the first test, a 500mm-long extensometer was used, yielding the most accurate possible result on the length of the reinforcement. For the second test, a 100mm extensometer was used. The same 100mm extensometer was used during the third test.

The measuring devices were removed after reaching 30% of the theoretical maximum tensile strength to avoid potential damage. The data from the first 30% will facilitate the extrapolation of the stress-strain curve by adjusting the data measured by the machine.

3.4.2 Distributed Fibre Optic Sensing (DFOS)

An optical fiber was used for the first GFRP tension test with the standard anchors. The purpose of this fiber was to give us information on the behaviour of the GFRP in the free length part, but also within the anchors. To do this, a Distributed Fiber Optic Sensing (DFOS) was glued along the entire length of the reinforcement. To the authors' knowledge, instrumenting FRP rebars with DFOS had not yet been reported in the literature at the time, and therefore these experiments represented a first and emphasize the novelty of the research. Latest publication in May 2023 by Zhou et al.[88] however detailed similar instrumentation and could present interesting installations features.

The Distributed Optical Fiber technology enables almost continuous information gathering over the entire length of a sample. This information may pertain to heat release or stress. The fiber possesses numerous advantages: it's flexible, non-electrical, offers accurate and fast measurements, and is unaffected by external disturbances. However, its fragility necessitates careful handling.

As previously explained in the literature review of this thesis (cf. Section 2.5), this technology relies on the phenomenon of Rayleigh scattering. It is an essential phenomenon in fiber optics which is caused when there are some irregularities or variations in the core of an optical fiber that are smaller than the wavelength of the light traveling through it. Initially, the Rayleigh scattering pattern is recorded and stored as a reference. Subsequently, when the optical fiber is subjected to changes in

stress or temperature, measurements are taken. These measurements capture the alterations in the scattered light. To analyze these changes, the data is transformed from the spatial domain to the frequency domain using the Fast Fourier Transform (FFT). This conversion allows for a more detailed examination of the scattered light signals. By comparing the initial reference state with the disturbed state, the extent of deformation or stress variations can be determined. This working principle is summarized in Figure 3.7.

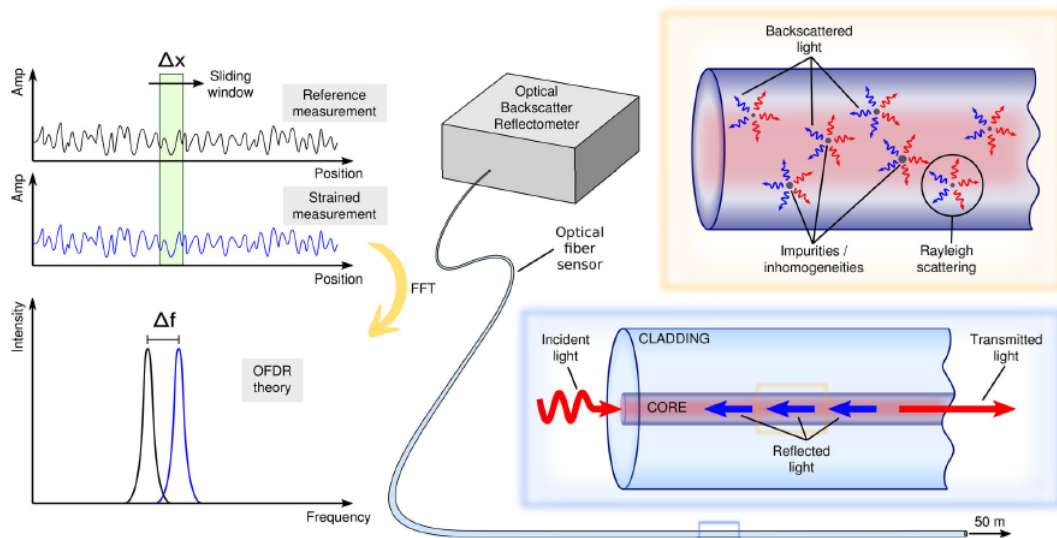


Figure 3.7: Working principle of Rayleigh Backscattering optical fiber sensors based on OFDR[14]

The practical details to glue DFOS on a GFRP are detailed in Annex A. The primary steps for a smooth rebar are as follows:

1. Clean the armature with a degreaser.
2. Fix the fiber to both ends of the framework using cyanoacrylate glue and activator.
3. Secure the fiber in place using a flat-headed wooden pin coated with glue.
4. Check the bond and apply a second coat of glue if necessary.

For the first test setup, the fiber was threaded through the anchors to gather information about the behavior of the GFRP in them. To facilitate this, the centering pieces and the bonding procedure were adapted. (See Fig3.8a) The fragile fiber parts were shielded with a heat-shrinkable polymer or an air-drying polyurethane coating, depending on the zone.

In the third test however, the ribbed surface of the GFRP bar could not allow for a simple gluing of the DFOS. A cutting had to be made along its side to insert and glue the fiber. In order to avoid the problem linked to the fiber inside the anchors that took place during the first test (that will be discussed in the next section), the fiber was glued to the outside of those anchorage instead.(See Fig 3.8b)



(a)



(b)

Figure 3.8: (a) Adapted 3D centering pieces for DFOS measurement and (b) DFOS installation on the side of the anchorage for test 3

3.5 Tension and compression test on IRT-M2P bars with ISO anchorage

For this first test, the objectives were to validate the manufacturing processes of the ISO anchorages, the preparation (cutting and DFOS installation) of a simple smooth bar, and the behavior of the GFRP bars in tension and compression. The choice of ISO anchorages was made to avoid the unknown of the bar's behavior inside the anchors. The test setup is illustrated in Figure 3.10a.

3.5.1 Testing protocol

The testing procedure unfolded as follows :

1. Tension applied until reaching 15% of the theoretical maximum tensile strength.
2. Compression applied until reaching 15% of the theoretical maximum compressive strength.
3. Returned to 0.
4. Tension applied until reaching 30% of the theoretical maximum tensile strength.
5. A 1-minute pause to remove the extensometer before failure.(To avoid damages, see Fig3.10b)
6. Tension applied until failure.

The theoretical maximum tensile and compressive strength employed were respectively 1310 MPa and 550 MPa [85] [22]. The testing speed was 2mm/min.

3.5.2 Results and discussion

In this test, a predictable result was achieved. The GFRP reinforcement underwent brittle failure under a tension load of 137kN, translating to slightly above 1000 MPa, alongside a strain of about 20,000 μ strain. The stress-strain line demonstrates an entirely elastic trend, with no plastic plateau prior to failure. (See Fig3.9)

The first test was interrupted during the compression phase due to a buckling phenomenon. However, the observations from the initial loading-unloading phase confirmed the completely elastic nature of the material, as it exhibited no residual deformation, hypothesis that was kept for the second part of the test, reason why it starts back at a null displacement value.(See Fig3.9)

The tensile moduli of elasticity deduced from the two sections of this test are 52.4 and 53.9 GPa respectively, measured on the entire stage 1 for the first section and on the entire stage 4 for the second section.

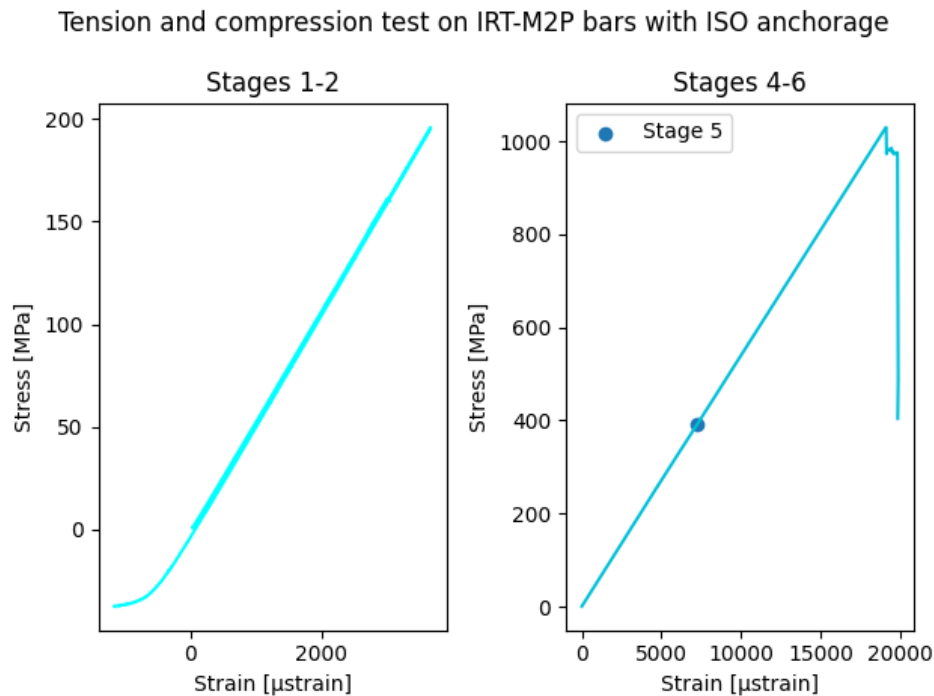


Figure 3.9: Results of the tension and compression test on IRT-M2P bars with ISO anchorage as measured by the extensometer and interpolated from the machine displacement in stages 5-6

The DFOS however did not provide usable data. Indeed, it broke during the installation of the specimen in the test machine's grips. This breakage likely occurred when tightening the anchors, leading to an 'uncapping' effect of the 3D printed centering pieces which probably damaged the fiber.3.10c

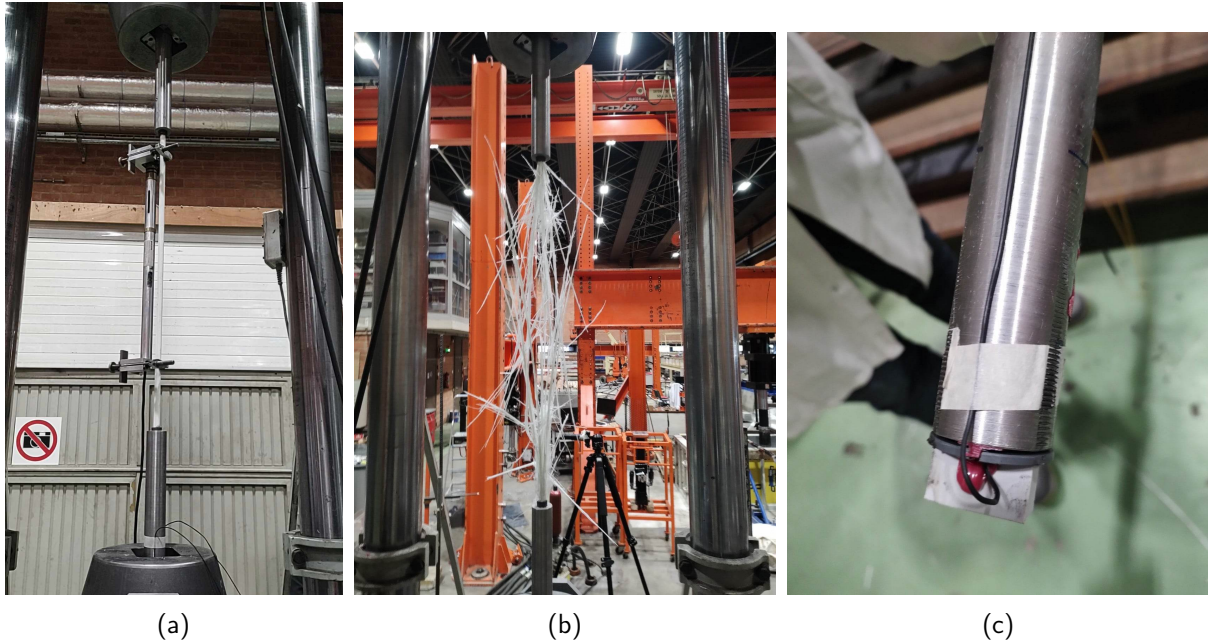


Figure 3.10: (a) Testing setup with visible 500mm extensometer (b) Brittle failure of the GFRP rebar and (c) Uncapping of the 3D printed piece causing fiber damage

In order to examine whether the epoxy was properly distributed within the anchorage, they were longitudinally opened. It was found to be distributed in an optimal and very consistent manner (See Figure 3.11) thus validating the injection procedure with injections holes as previously illustrated in Figure 3.6.



Figure 3.11: Longitudinal cut in the anchorage complying with standards

3.6 Monotonic test on IRT-M2P bars with small aluminium anchorages

In this second test is explored the possibility to conduct the same monotonic test (without the compression part) on the same GFRP bar with smaller aluminium anchors (See Figure 3.12). Indeed, the ISO anchors require extensive manufacturing and a huge quantity of steel whereas those smaller anchors are light, easy to manufacture (as the aluminium tubes are commercially available), and require less space and epoxy.



Figure 3.12: Monotonic test setup of the IRT-M2P bars with small aluminium anchorages and a 100mm extensometer

3.6.1 Testing protocol

The testing procedure unfolded as follows :

1. Tension applied until reaching 30% of the theoretical maximum tensile strength
2. A 1-minute pause to remove the extensometer before failure. (To avoid damages, see Fig3.10b)
3. Tension applied until failure.

The theoretical maximum tensile and compressive strength employed were respectively 1310 MPa and 550 MPa [85] [22]. The testing speed was 2mm/min.

3.6.2 Results and discussion

This second test resulted in failure at 85kN, approximately 640 MPa, due to slippage in an anchor, as illustrated in Figure 3.14. A longitudinal crack was also observed along the reinforcement. The tensile modulus of elasticity deduced from this test was equal to 55.9 GPa. (See Fig3.13)

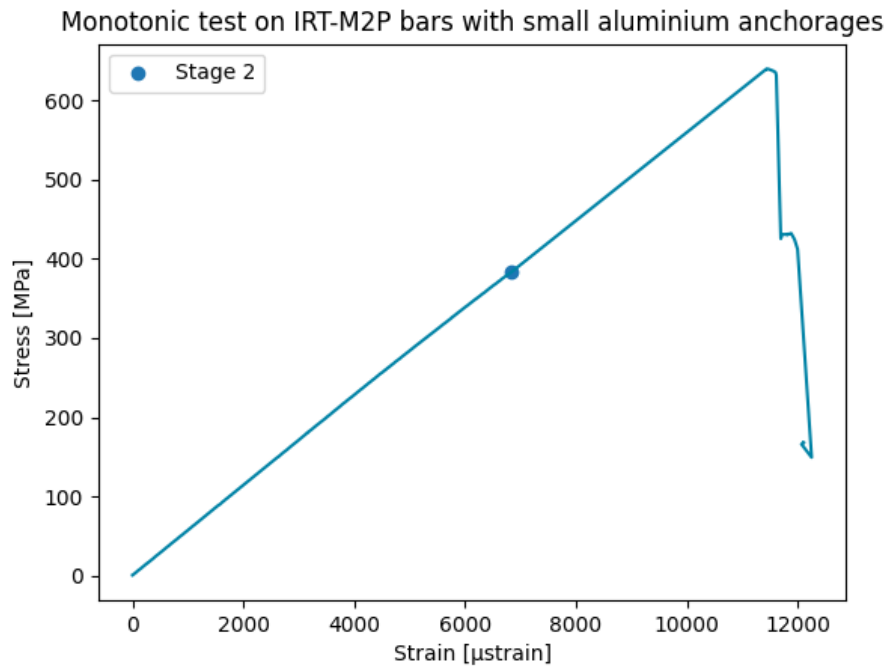


Figure 3.13: Results of the monotonic test on IRT-M2P bars with small aluminium anchorage as measured by the extensometer and interpolated from the machine displacement in stages 2-3

Upon opening the aluminum anchor, a brittle fracture was discovered. This fracture, illustrated in Figure 3.15, was located within the anchor where slippage had previously been identified. This slippage can be attributed to a failure of the GFRP reinforcement within the anchor, which was instigated by the pressure exerted by the machine's jaws. The aluminum tube, which wasn't robust enough, transferred the diametric pressure to the reinforcement. This resulted in a failure due to excessive stress being placed on the fibers in a direction that was not designed to withstand such forces, explaining the early failure compared to the test on the same bars but with ISO anchors.

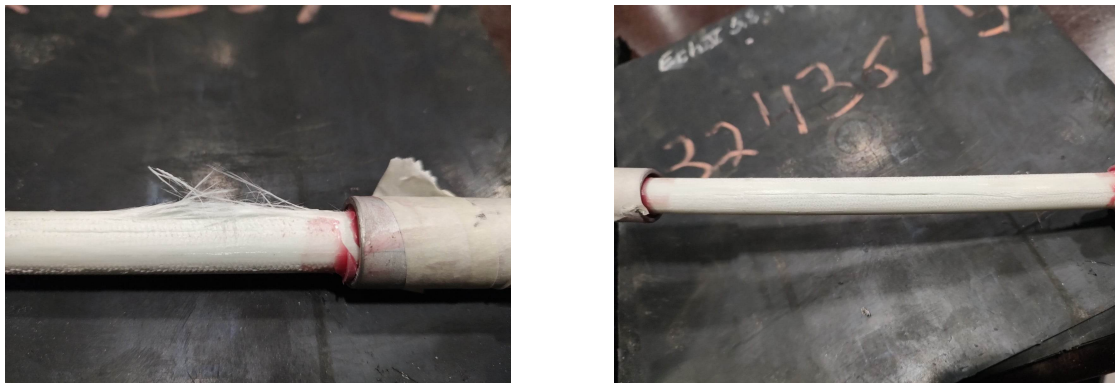


Figure 3.14: Slippage and failure in the GFRP rebar

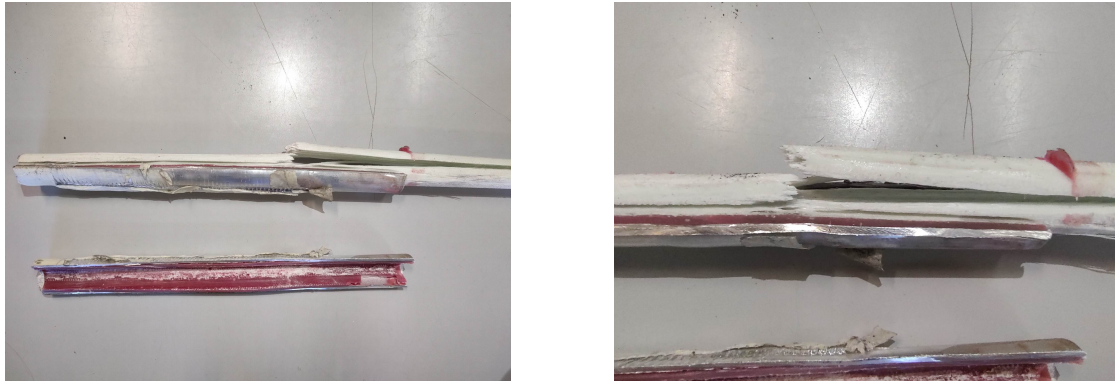


Figure 3.15: Longitudinal cut in the aluminium anchorage presenting slippage

3.7 Cyclic test on Sireg's Glassfree bars with ISO anchorage

For this third and final test, the difference of GFRP bar's coating is explored by using Sireg's Glassfree bars that have a ribbed surface as well as the behavior of GFRP under cyclic loading. ISO anchors were used according to the conclusions of the first two tests, as illustrated in Figure 3.16.

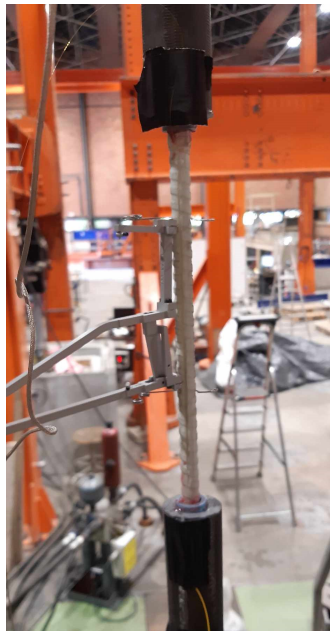


Figure 3.16: Cyclic test setup of the Sireg's Glassfree bars with ISO anchorage, 100mm extensometer and embedded DFOS in a groove along the bar

3.7.1 Testing protocol

The testing protocol was a cyclic loading with steps increasing by 50 MPa at each cycle until reaching 500 MPa and concluded with tension until failure as detailed in Table 3.2. The loading speed remained a constant 2 mm/min.

Cycle	1	2	3	4	5	6	7	8	9	10	11
Stress [MPa]	50	100	150	200	250	300	350	400	450	500	1033**
Strain [μ Strain]*	943	1887	2830	3774	4717	5660	6604	7547	8491	9434	19491

Table 3.2: Loading protocol of the cyclic test on Sireg's Glassfree bars with ISO anchorage

*as expected with $E=53$ GPa

**Average tensile strength provided by the manufacturer[74]

3.7.2 Results and discussion

This third test carried out on the second type of GFRP rebar from SIREG, led to a similar brittle failure as the first test, occurring at 916 MPa. This ultimate tensile stress result is slightly less than the value provided by the supplier.[74] The experimental tensile modulus of elasticity, measured on cycle 11 before the extensometer removal, was found to be 53.4 GPa, which is very similar to the one provided by the supplier and also to the other bars. (See Fig3.17).

The most disturbing difference is the apparent behavior of the bar during the first loading stages. The extensometer's data seems to indicate a negative residual deformation during the 2 first stages, which might be due to a slippage of the extensometer along the rigged surface of the bar as the data from the machine's grip displacement shows no such behavior. The pseudo-residual displacement in the machine displacement is most probably due to the machine's own internal elasticity and internal displacement. However, all the other loading and unloading stages as measured by the extensometer indicate a proper linear behavior with no residual deformation, which will be an assumption for the cyclic behavior of a GFRP rebar.

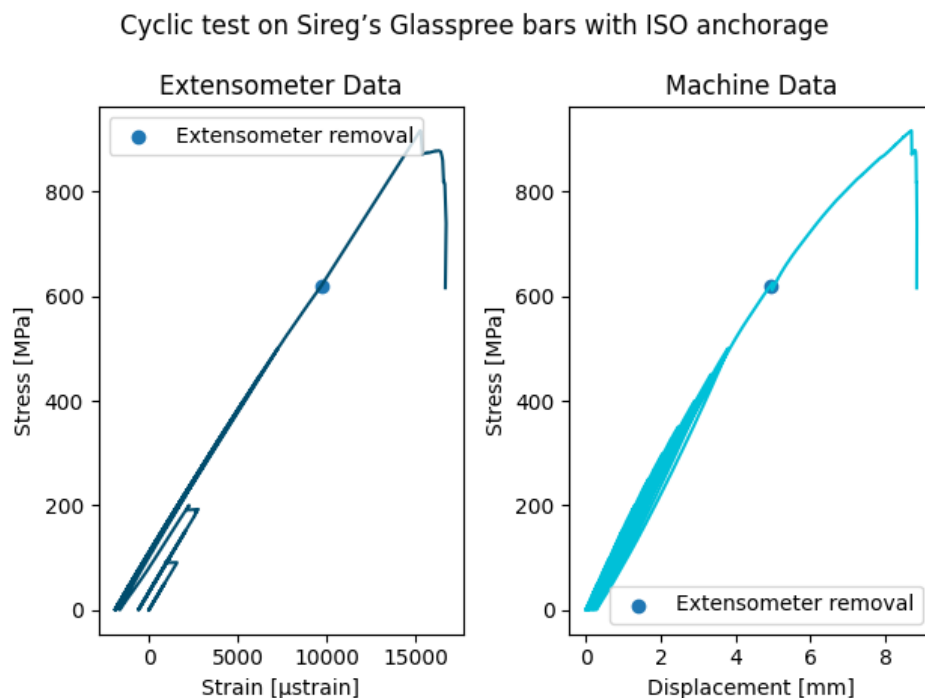


Figure 3.17: Results of the cyclic test on Sireg's Glassfree bars with ISO anchorage as measured by the extensometer and interpolated from the machine displacement after extensometer removal and as measured by the machine displacement

The DFOS data matched the strain extrapolated from the extensometer and the expected strain (See Table 3.3 and related Figure 3.18). However, significant information loss was experienced due to the fiber's angle from the outside radius of the anchorage to the cut in the GFRP rebar. Pinching of the fiber might have occurred due to that angle, which would affect the light transmitting through the fiber optic core. A non-linear behavior can also be observed with the little data remaining, most probably due to the manual grooving of the GFRP bar where the DFOS was embedded being not perfectly straight nor always at the same depth. (See Fig3.19)

Cycle	1	2	3	4	5	6	7	8	9	10
Expected Strain [μ Strain]*	943	1887	2830	3774	4717	5660	6604	7547	8491	9434
Extensometer Strain [μ Strain]**	860	1760	2600	3900	4470	5340	6250	7210	8190	9170
DFOS Strain [μ Strain]***	866	1760	2367	3487	4290	5277	5667	/	/	/

Table 3.3: Strain expected and measured on the cyclic test on Sireg's Glassfree bars

*as expected with $E=53$ GPa

**after removal of the negative residual displacement

***average on the remaining data.

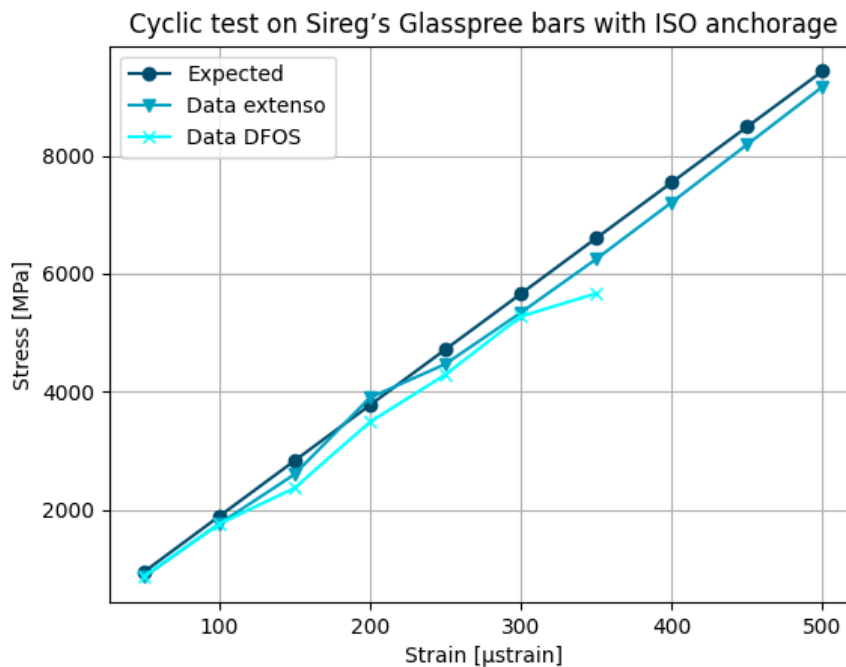


Figure 3.18: Comparison between the expected and measured strain on the cyclic test on Sireg's Glassfree bars

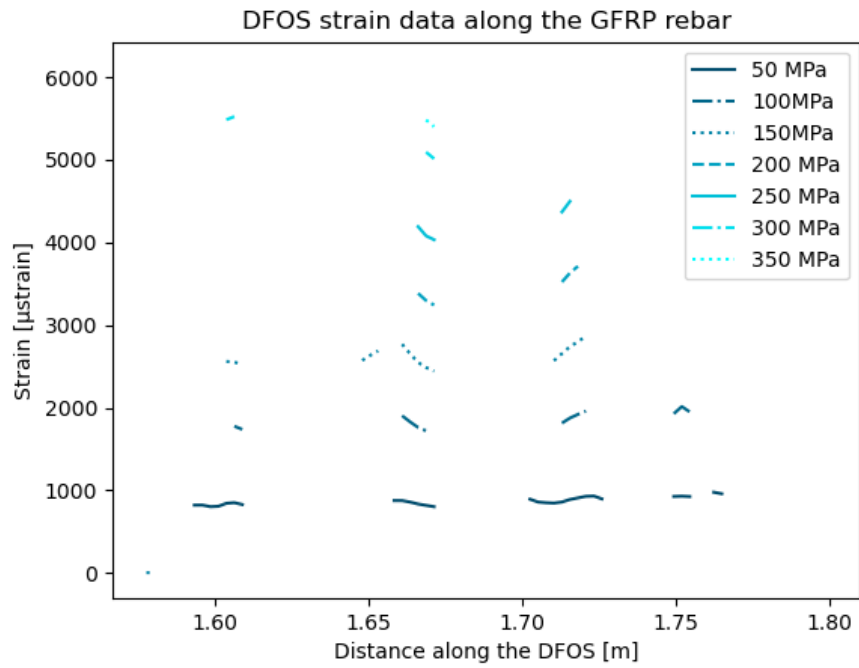


Figure 3.19: Data visualisation of the DFOS strain along the GFRP rebar at different loading stages

3.8 Conclusions

A summary of the different tests results as detailed in the previous section can be found in Fig 3.20.

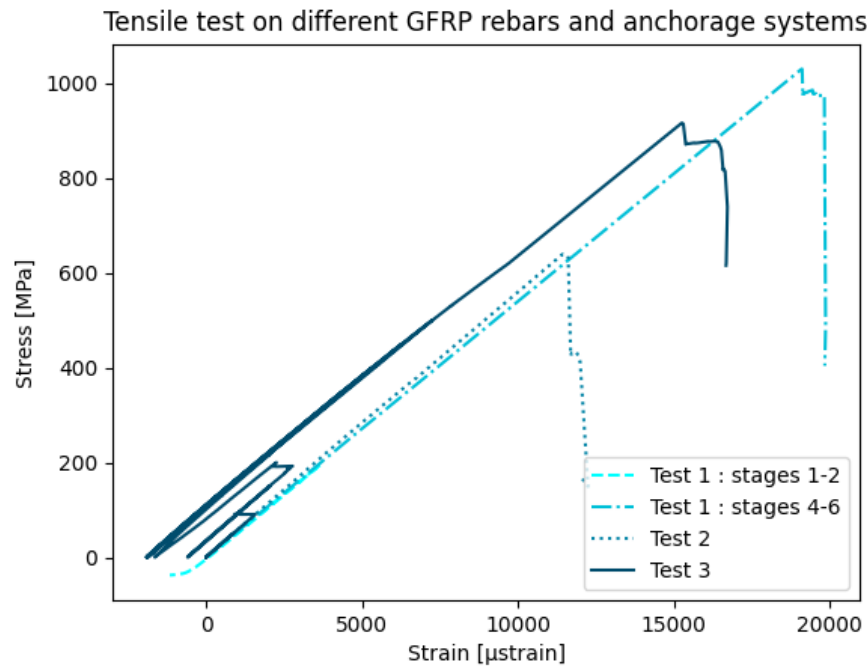


Figure 3.20: Result of the tensile tests conducted for the 2 GFRP rebar characterization

Several interesting conclusions can be drawn from these three tests:

- The anchorages as prescribed by the standard allow for the rupture of the GFRP reinforcement, and will thus be used for all following experiments using GFRP.
- It's not recommended to use an undersized anchor. As observed, the reinforcement does not accept direct confinement stresses if the anchor thickness is too thin.
- The IRT-M2P bars were characterized as having a maximum tensile strength greater than 1000 MPa and a Young's modulus in tension equal to 54 ± 2 GPa. However, these values should be interpreted with caution, particularly for the tensile strength limit, for which only one test was carried out. The standard prescribes a minimum of 5 tests, but the aim of this thesis is not to characterise them precisely, only to make some pre-tests to have an idea of the characteristics of the available materials and to validate the anchorage system of GFRP rebars.
- For the same reasons, the test on the Sireg bars is only conclusive to prove the perfectly linear behaviour of these GFRP bar under cyclic loading with no residual deformation. For the proper mechanical properties, we'll refer to the manufacturer data obtained from proper test according to EAD 260023-00-0301.[74][26]
- The Sireg bars exhibit a very similar behavior to the others. Their surface is thus the only significant difference that will be the focus from now on.
- The application of DFOS in GFRP anchorage still needs refinement. While gluing it to the outside of the anchorage yields better results than encapsulating it inside the anchor and the 3D printed guides, the loss of data is still too significant. For future use of the DFOS, it will be glued up to a few centimeters before the anchorage and safeguarded in a hermetic plastic bag adjacent to the bar.
- If DFOS needs to be attached to a ribbed surface and a groove has to be made, it should be done either by a machine or with extreme care to allow for a proper application of the DFOS inside it.

CHAPTER

4

DEVELOPMENT OF MECHANICAL COUPLER GFRP/SMA/STEEL

In this chapter will be developed different mechanical connections between SMA and steel, GFRP and steel, and finally, SMA and GFRP. The first section summarizes the properties of the different rebars used in these tests. Each section describes the mechanical coupler configuration and the different test setups and instrumentation, before discussing the results.

4.1 Material Properties

This section provides a detailed examination of the materials utilized in this study, namely GFRP rebars, steel rebars, and SMA. The comparative tensile test results for these different materials are illustrated in Figure 4.3.

4.1.1 GFRP rebars

As presented in the previous section, two different GFRP rebars were used for the different tests (See Fig3.20). In the Table 4.1 are listed the characteristics of both of them. A separate comparison of how different materials behave under tensile tests is represented in Figure4.3, with the Glasspree 12mm's curve extrapolated from Table 4.1.

	Φ [mm]	f_{ft0} [MPa]	f_{ftk0} [MPa]	E_f [GPa]	ϵ_{ft0} [mm/mm]	ϵ_{ftk0} [mm/mm]
GFRP, IRT-M2P	13	>1000*	/	54*	0.0199*	/
Glasspree® TP Round Bars" SIREG	12	1033**	977**	53** 53.4*	0.0196**	0.0185**

Table 4.1: GFRP rebars properties

*Obtained experimentally

**From manufacturer[74]

4.1.2 Steel rebars

High-resistance BE500S steel rebars were utilized for the coupler tests. Table 4.2 provides a detailed account of their physical characteristics. The behavior of different materials under tensile tests, including steel rebars, can be compared in Figure 4.3.

Diametre [mm]	f_y [MPa]	f_u [MPa]	E_s [GPa]	ϵ_{sy} [mm/mm]	ϵ_{su} [mm/mm]
12	500	635*	210	0.014	0.178*

Table 4.2: Steel rebars properties
*Obtained experimentally

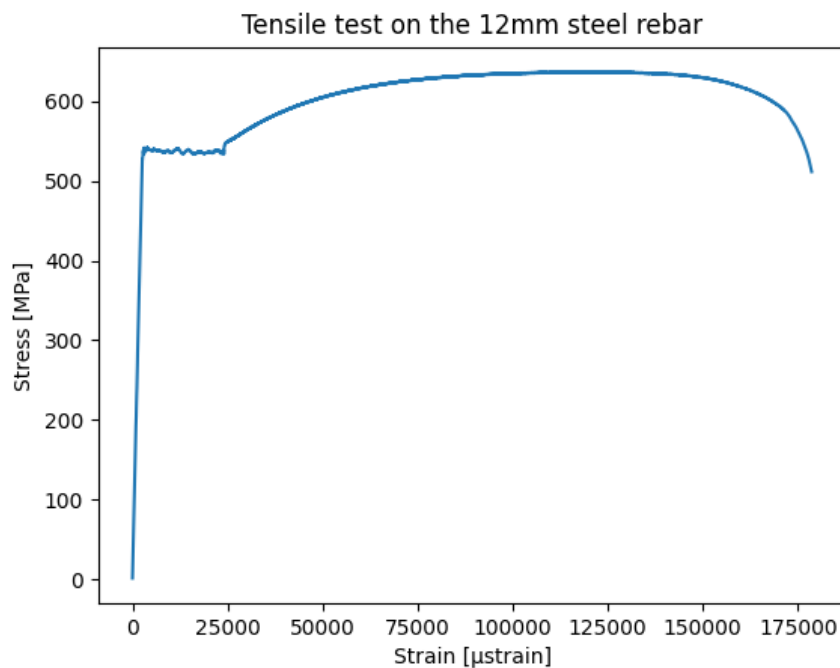


Figure 4.1: Tensile test result for the characterization of the BE 500S steel bars

4.1.3 SMA rebars

The values in Table 4.3 have been adopted from the study by Almeida et al.[8]. Figure 4.2 shows the experimental Stress-Strain curve for steel (monotonic) and SMA rebars (cyclic). A comparison of the behavior of different materials, including SMA, under tensile tests is also provided in Figure 4.3.

Diametre [mm]	σ_{AMs} [MPa]	σ_{AMf} [MPa]	σ_{MA_s} [MPa]	σ_{MAf} [MPa]	σ_{nu} [MPa]	E [GPa]	ϵ_{nu} [mm/mm]
12.7	319	349	73	32	1053	31	0.013

Table 4.3: SMA rebars properties

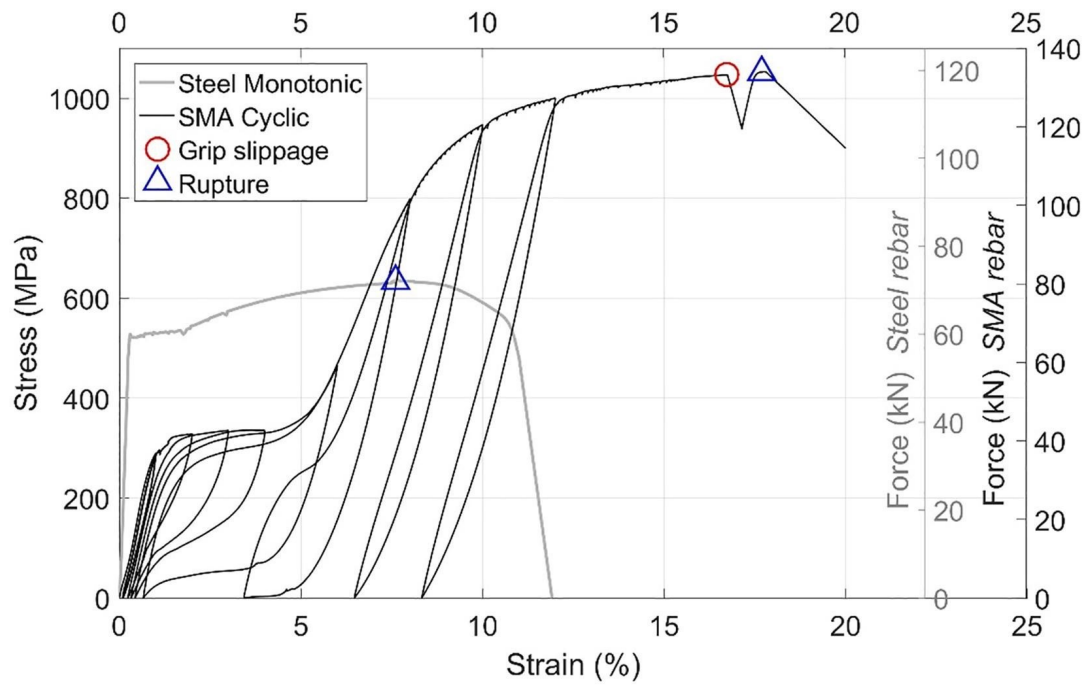


Figure 4.2: Experimental Stress-Strain curve for steel (monotonic) and SMA rebars (cyclic)[8]

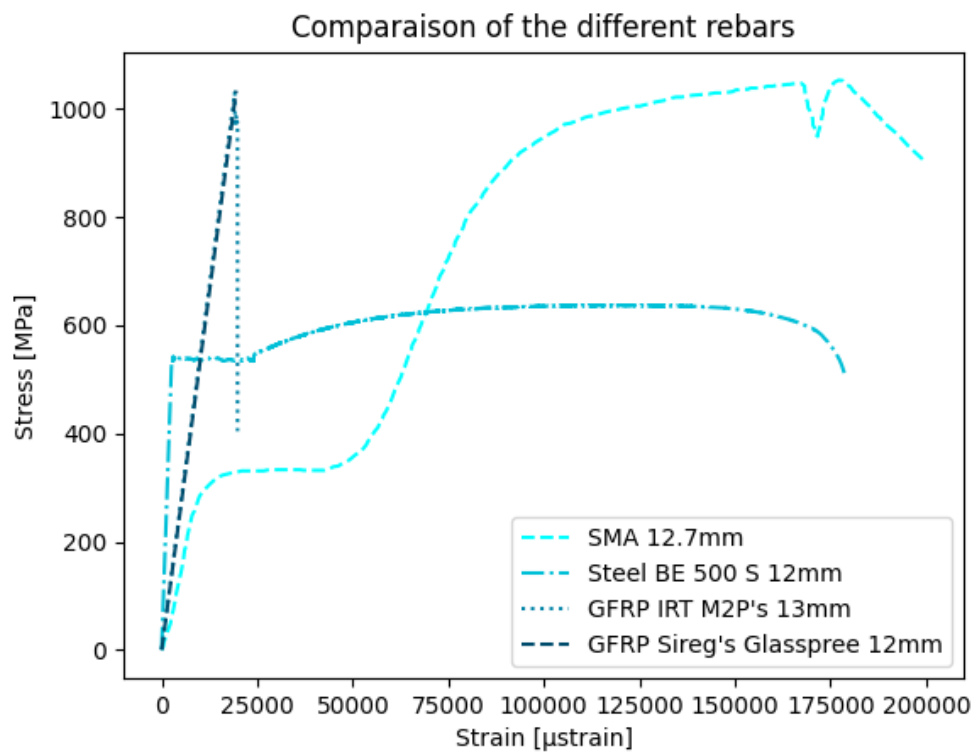


Figure 4.3: Tensile test results of the different materials used in this thesis, SMA curve referenced from Almeida et al. [8]

4.2 SMA-Steel Coupler

4.2.1 Coupler description

To develop a GFRP-SMA mechanical coupler, initial efforts focused on the SMA part of the coupler, basing the work primarily on the research of Almeida et Al.[8] and Alam et Al. [6]. The disadvantage of SMA is its sensitivity to punching, preventing it from being connected using conventional mechanical couplers for steel, as represented in Figure 4.4. These couplers comprise a tube in which steel rebars are connected using three or four pointed screws. The screws are tightened to press the rebar against the two racks welded at the bottom of the tube. This 3-point fixing is extensively used for steel reinforcing bars, providing a reliable connection. However, the punching issue with SMA prevents their direct use.

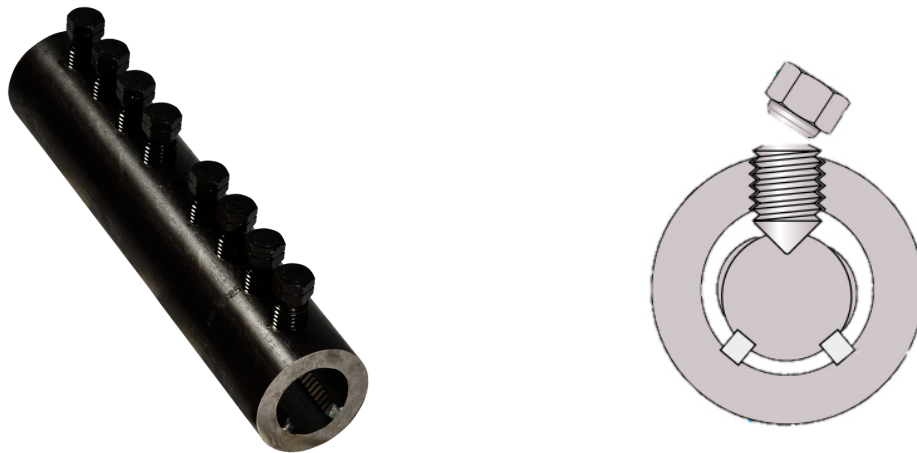


Figure 4.4: Steel mechanical couplers PLAKA [62]

Another limitation is the difficulty to weld SMA, necessitating a different approach. In their research, Alam et Al. showed that two main failure modes can appear when developing SMA couplers:

1. Punching of screws in the SMA
2. SMA slippage in the coupler

Almeida et Al. proposed a solution with five screws aligned longitudinally, allowing forces to be transmitted through the coupler. However, due to practical constraints, the design proposed by Alam et Al. was utilized, which is half as long. It involves a classic half steel coupler with two additional rows of three holes, inclined at 60° , as shown in Figure 4.5 and 4.6. The initial steel couplers were ordered from PLAKA and were modified to suit the required geometry. The original screws were kept, which are fusible screws that break when a torque of around 70 kNm is applied. They were flattened to limit punching problems. The entire manufacturing procedure to create the SMA part of the coupler is available in Annex B.2.

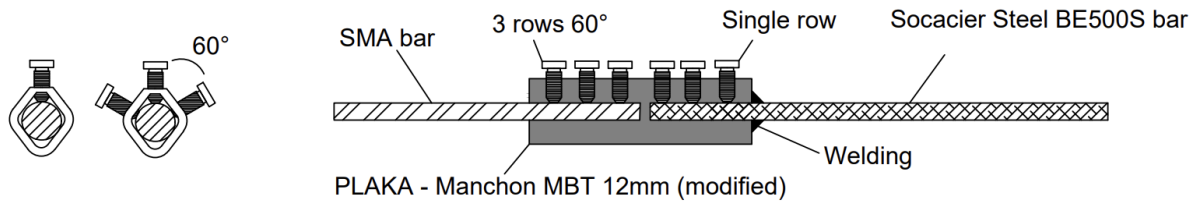


Figure 4.5: Schematic drawing of the SMA-Steel mechanical coupler

For the first test on the SMA coupler, high-resistance steel was utilized for the second half of the coupler, fixed with a classical three screws mechanical coupler. The extremity of the coupler was welded to the steel rebar for additional safety on the steel-coupler connection.



Figure 4.6: SMA-Steel Coupler

A significant risk of this type of coupler is the appearance of a moment concentration on the first screw of the SMA coupler due to the imperfect geometry of the centering racks or a slight misalignment of the rebars. This can lead to significant punching at this point, causing premature failure of the SMA, as described by Herrezeel and Rigot [35]. The coupler can not be tested by clamping the SMA directly in the jaws of the test machine as it would increase bending, which would be taken up in the form of a moment at the coupler inlet. More degrees of freedom must be left to the SMA for the coupler under test to approximate a real-life configuration, achieved by using the configuration shown in Figure 4.7 with two couplers. Here, the SMA is coupled on each side by high-strength steel, which can be clamped tightly in the machine's jaws.

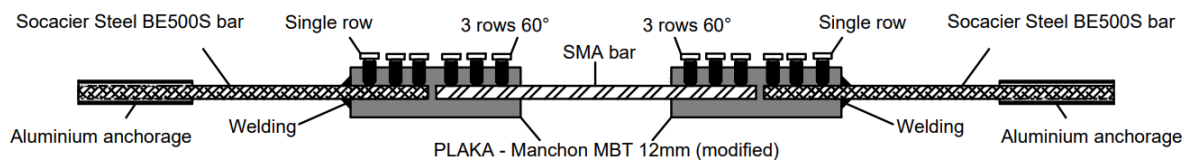


Figure 4.7: Schematic drawing of the testing assembly for the SMA-Steel connection

The objective of the coupler design is to reach the SMA plastic stage. Results from Almeida et Al. served as a reference, given they used the identical SMA rebar, with the configuration of five screws in series. They achieved a resistance of 67.8 kN (+515MPa) before the slippage of the SMA.

Alam et Al. reached a resistance of around 500 MPa with the same coupler configuration as the one designed in this section, before a brittle failure due to the punching of the screws on the rebar.

4.2.2 Testing procedure and instrumentation

The validation test of this coupler was a simple tension test at 3mm/min. The testing process employed two extensometers, as illustrated on Figure 4.8:

1. A 100 mm length extensometer placed on the free length of the SMA
2. A 500 mm length extensometer placed over the two couplers

A thermocouple was also placed on the SMA rebar. A thermocouple [38] [15] operates based on the principle of the Seebeck effect. It consists of two dissimilar metal wires, typically joined at one end to form a junction. When a temperature gradient is present along the wires, a voltage proportional to the temperature difference is generated at the junction.

The Seebeck effect occurs because various metals have different electron configurations and conduct electricity differently. If one junction of the thermocouple is exposed to a higher temperature compared to the other junction, it triggers a flow of electrons from the hotter junction to the colder one. This flow of electrons produces a voltage across the junction, known as the thermoelectric voltage. The magnitude of this voltage is directly proportional to the temperature difference between the two junctions. By measuring this voltage, the temperature at the hot junction can be determined using a reference temperature at the cold junction.



Figure 4.8: Test setup of the SMA-Steel configuration as in Fig4.7, with two extensometers and a thermocouple on the SMA (right)

4.2.3 Results of the tests

As anticipated, the first stage of the test demonstrated an almost linear behavior, corresponding to the elastic phase of both the steel and the SMA. The SMA then reached its "Strain plateau" or superelasticity (SE) phase where it underwent much more strain (See Fig 4.9). The strain continued to accumulate primarily in the SMA part of the mechanism until a brittle failure of the SMA in the mechanical coupler occurred, due to punching of the first row of screws. (See Fig 4.13). Without DFOS equipped on the mechanism, the best representation of its strain behavior is Fig 4.10, where a clear change in the linear behavior (due to a constant displacement applied by the machine) of the strain can be observed when the SMA started to change its phase, altering the strain distribution along the mechanism.

The maximum force of the connection was 64.3 kN, which is a bit less than the configuration of Almeida et Al.. This might be due to a slight misalignment of the bars in the coupler. Future tests (on a GFRP-SMA configuration) will thus focus on ensuring as aligned a connection as possible to increase this value. However, having reached the strain plateau of the SMA, this test is considered sufficient for the SMA connection.

The thermocouple demonstrated an apparent almost perfect relation between strain in the SMA and temperature increase (See Fig 4.11). However, when representing the Stress-Strain-Temperature relation of the SMA during the test, illustrated in Figure 4.12, it shows that this relation is actually not correlated throughout the entire test and that the temperature stops to increase when the SMA enters its SE phase. This relation can be explained by the thermomechanical properties of the SMA and in this case a relaxation from a twinned martensitic phase to an untwined martensitic phase (cf. Fig 2.5).

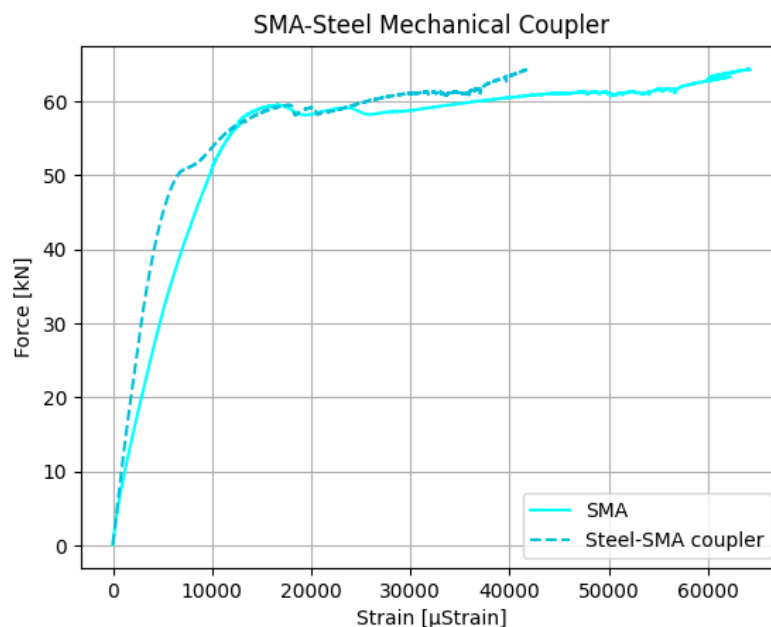


Figure 4.9: Force-Strain measured along the SMA and along the two couplers for an SMA-Steel connection

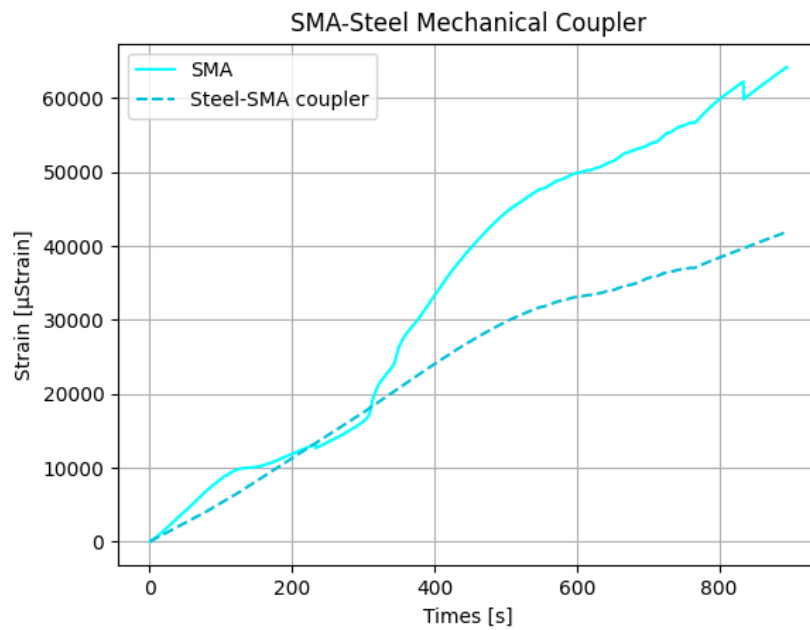


Figure 4.10: Strain increase during the test, with clear strain increase of the SMA when reaching the strain plateau

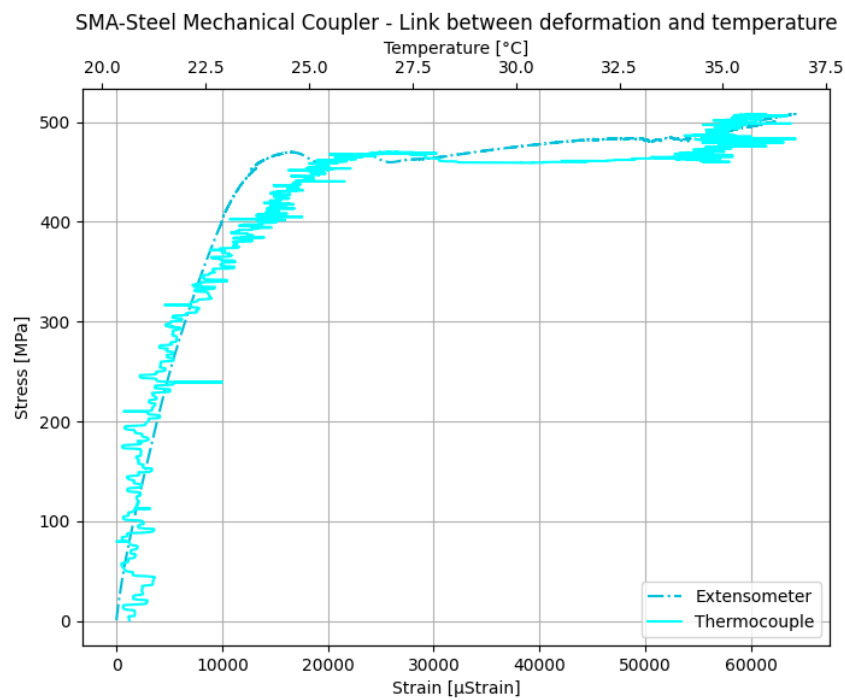


Figure 4.11: Strain and Temperature increase in the SMA during the SMA-Steel mechanical coupler test

Stress-Strain-Temperature relation in the SMA

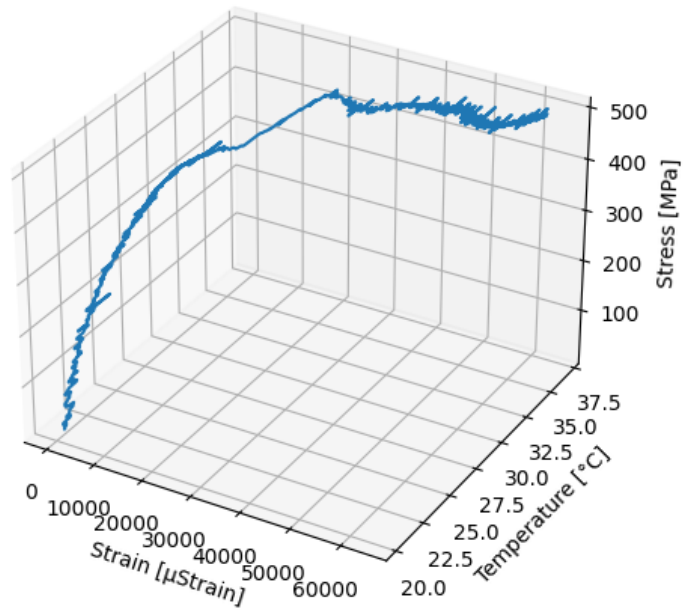


Figure 4.12: Stress-Strain-Temperature relation in the SMA from the extensometer and the thermocouple data, with an accommodation from a twinned martensitic phase to an untwined martensitic phase

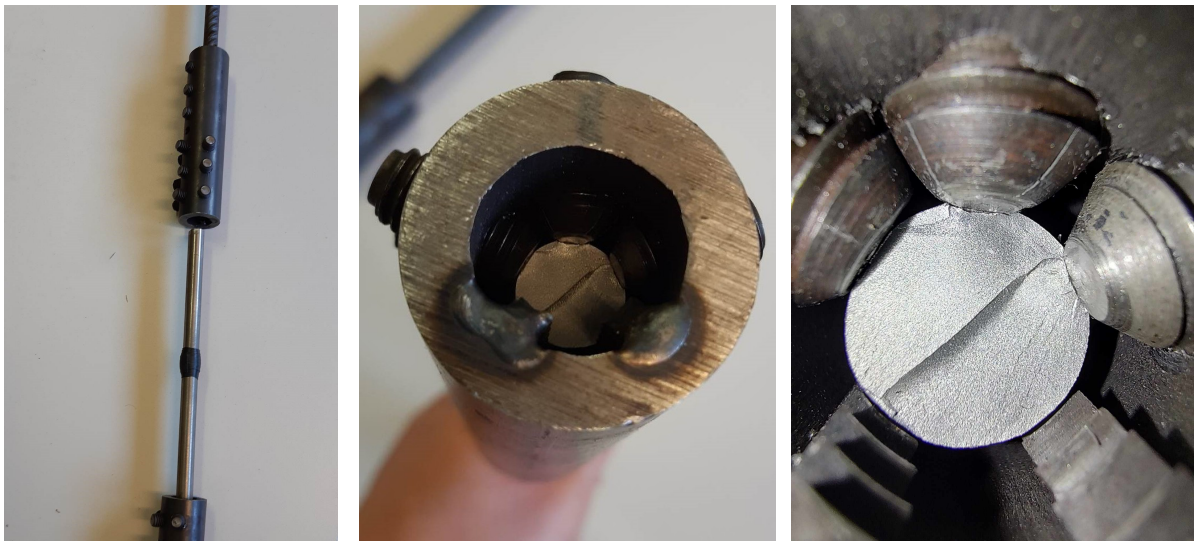


Figure 4.13: Brittle failure of the SMA-Steel coupler in the SMA due to a punching of the screws

4.3 GFRP-Steel Coupler

4.3.1 Coupler description and objectives

The objective of the GFRP part of the coupler is to ensure a resistance greater than the ultimate resistance reached with the SMA coupler configuration developed before (i.e. 65 kN, cfr. section 4.2). This will allow the SMA to be the limiting part in the SMA-GFRP coupler.

The second limiting point for the development of the coupler is that it must be as short as possible. This consideration is carried out in order to make the solution as applicable as possible on a worksite.

Alaou et Al. [7] described GFRP couplers and testing procedure. Their results are not comparable to the results that have to be reached in this thesis because they try to reach the failure of the GFRP, not a specific resistance capacity. Their couplers have to be 491 mm long to reach the full GFRP capacity, that gives a first order of size for the design.

Alam et Al.[6] tried different connections for GFRP couplers. The most suitable geometry regarding the objectives previously mentioned is a 340 mm steel tube. This will guide the first iteration of the experimental research, to then reduce the size as much as possible.

The GFRP part of the coupler is a steel tube in which the GFRP rebar will be centered using 3D printed pieces and glued by an epoxy resin, the HIT-RE 500 from Hilti [36]. The internal diameter of the tube complies with the ASTM standard for a GFRP tensile test anchorage. For a 13mm reinforcement, the internal diameter is 21mm. The cross-section of the steel tube has been dimensioned to resist a maximal normal force of 100 kN, which is way above the resistance of the different rebar to make sure that the coupler never breaks. In the case of a 13mm rebar, the external diameter of the GFRP part of the coupler must be at least 32mm. As made for the anchorages in Chapter 3, injection holes are drilled in the steel tube to ensure a good distribution of the epoxy when injecting it.

The second half of the GFRP-Steel coupler is, as for the SMA-Steel coupler, a half simple steel coupler, welded to the GFRP half coupler. A security welding is made at the junction steel-coupler to ensure the resistance of the steel part of the coupler. An iterative process was then made to find the minimal steel tube length ensuring the required resistance of the coupler. The steps are developed in the testing procedure section. The schematic drawing of the coupler and its realisation are presented on Figures 4.14 and 4.15.

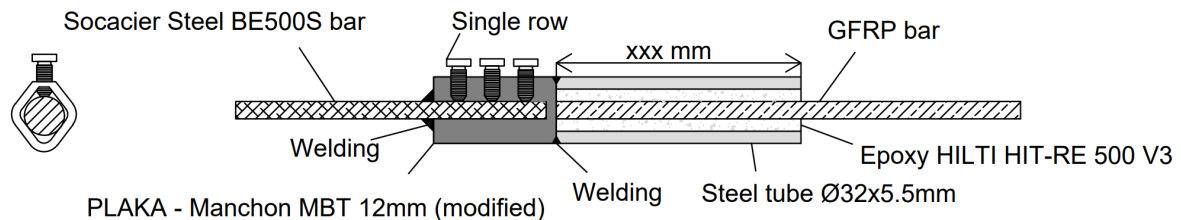


Figure 4.14: Schematic drawing of the GFRP-Steel mechanical coupler



Figure 4.15: GFRP-Steel Coupler

The complete configuration represented on Figure 4.16 used an ISO steel anchorage for the GFRP side of the coupler as described in section 3.3 and a simple aluminium tube for the Steel side.

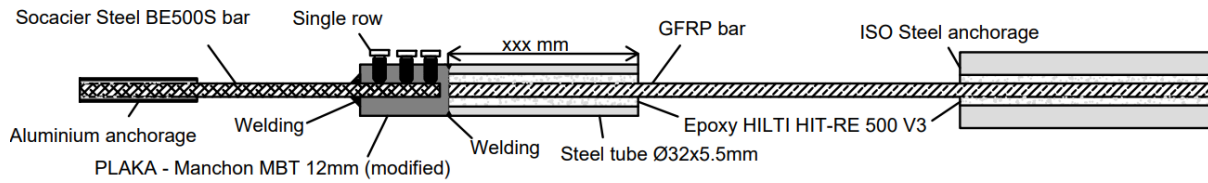


Figure 4.16: Schematic drawing of the testing assembly for the GFRP-Steel connection

4.3.2 Testing porcedure and instrumentation

The different test setups are reported in table 4.4. All tests were conducted at a testing speed of 3mm/min, using GFRP anchorages compliant with the norms as developed in section 3.3. [10][43]

	Rebar Type	Rebar Coating	Coupler length [mm]
SGFRP-I125	IRT-M2P GFRP rebar	Smooth surface	125
SGFRP-I200	IRT-M2P GFRP rebar	Smooth surface	200
SGFRP-I250	IRT-M2P GFRP rebar	Smooth surface	250
SGFRP-S200	SIREG Glasspre®	Rectangular ribbed surface profile	200
SGFRP-S250	SIREG Glasspre®	Rectangular ribbed surface profile	250

Table 4.4: GFRP couplers samples

Unlike the SMA-Steel coupler test in section 4.2, only one 500mm extensometer was used on the samples, to measure the displacement around the whole coupler. Furthermore, a distributed optical fiber was placed on the SGFRP-I200. (See Fig 4.17)

The DFOS was installed continuously on the GFRP and the steel, passing through the coupler. Here are the main stages of the installation of the fiber on such a coupler:

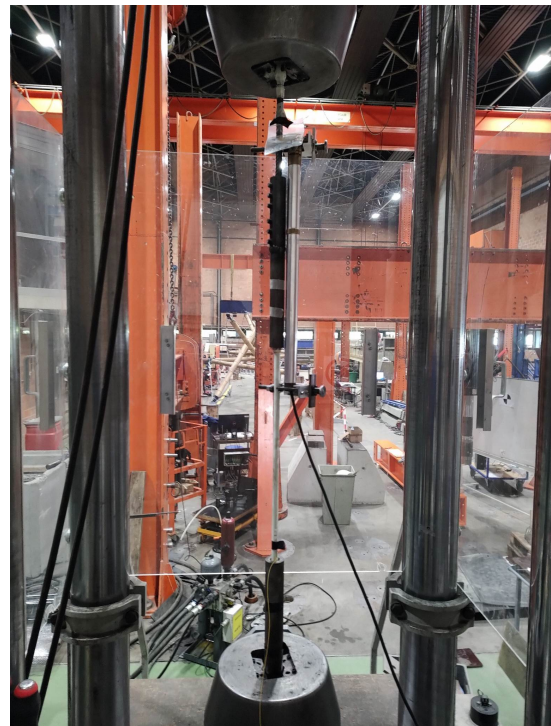
1. Temporary gluing of the two reinforcing bars using a two-component glue. This will allow work to be carried out on the complete bond without there being any displacement between the two bars at the intersection.
2. Gluing the fiber to the GFRP reinforcement following the instructions in the appendix A.

3. In the continuity, gluing the fiber to the steel reinforcement, by making a groove in the reinforcement beforehand to accommodate the fiber and fill it with glue [40].
4. Protect the free ends of the fiber with a sheath.
5. Insert the rebars through the coupler, using 3D centering pieces, and taking care to pass the fiber between the racks and the clamping screws to keep it safe.
6. Tighten the clamping screws and inject the epoxy into the coupler.

For this DFOS-equipped coupler, a complete 6-screw coupler was used to connect the steel part instead of a half one. This allows to not have to make an additional welding at the coupler outlet, which was impossible in a configuration with fiber.



(a)



(b)

Figure 4.17: Test setup for the GFRP-Steel mechanical coupler (a) equipped with DFOS (b)

4.3.3 Results of the tests

For each test, two possible failure mechanisms can appear:

- GFRP slippage [GS] inside the coupler tube due to an insufficient length. (See Fig4.18a)
- Plastification and failure of the steel [SF] in its free length. (See Fig4.18b) This failure mechanism validates the test, because the ultimate steel strength is higher than the coupler resistance objective described in the section 4.2.1.



(a)



(b)

Figure 4.18: (a) GFRP slippage and (b) Steel plastification failure modes in the GFRP-Steel connection

	Type of Failure	Coupler resistance	Strain in the Steel at failure
SGFRP-I125	GS	35.5 kN	314 MPa
SGFRP-I200	GS	60.1 kN	532 MPa
SGFRP-I250	SF	>72 kN	635 MPa
SGFRP-S200	SF	>72 kN	635 MPa
SGFRP-S250	SF	>72 kN	635 MPa

Table 4.5: GFRP-Steel couplers results and type of failure. SF: Steel failure. GS: GFRP slippage in the coupler

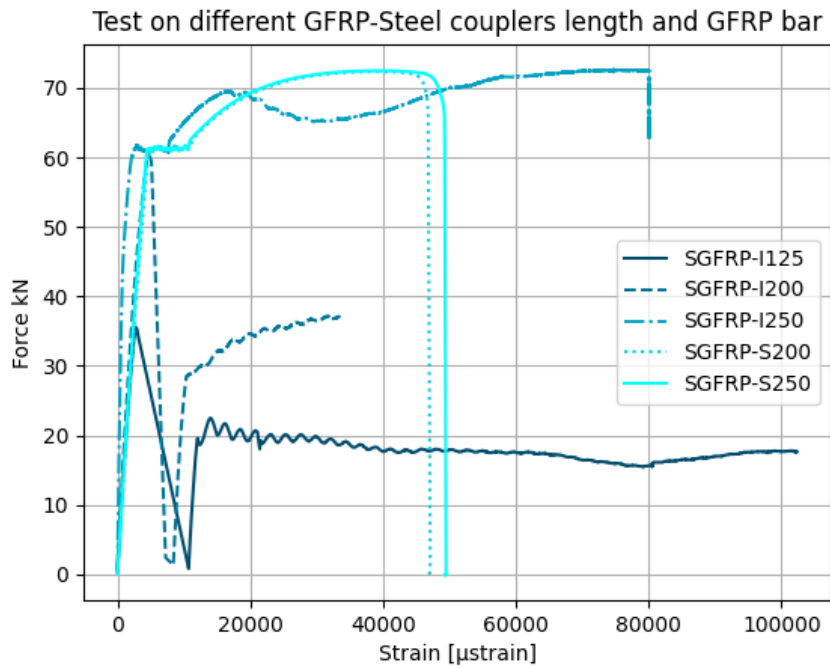


Figure 4.19: Experimental results of different GFRP-Steel mechanical couplers

As it can be seen in Table 4.5 and Fig 4.19, the two most important parameters impacting the performance are the tube length and the GFRP coating. Indeed, this coating is the first to fail in shear due to the tensile force that is applied perpendicularly to the GFRP surface. Table 4.5 shows that a 250 mm tube-length is necessary for the IRT-M2P rebar to reach the steel plastification at 635 MPa whereas for the ribbed surface Glasspreex rebar from Sireg, only a 200 mm tube-length is already enough. In terms of force, those coupler configuration are reaching a capacity above 72 kN which is large enough regarding the objectives.

A linear increase can be even observed in the coupler resistance to slippage when the tube length increases when comparing the different tests where such a failure happened. (See Fig 4.20) If an interpolation of those values is made for 70 kN (slightly higher than the goal), a minimal tube length of 235 mm is determined for the IRT M2P rebar, that validates the 250 mm test and might even explain the composite behavior of the SGFRP-I250 test, different from the ordinary steel-plastification behavior like in tests SGFRP-S200 and SGFRP-S250. A Bond Stress Peak $\sigma_{bond,peak} = 7.4 MPa$ can be defined for this type of coating in the Hilti epoxy. Unfortunately, there is no similar failure modes results for the Sireg's rebar so it is not possible with the actual data to find their optimal length value.

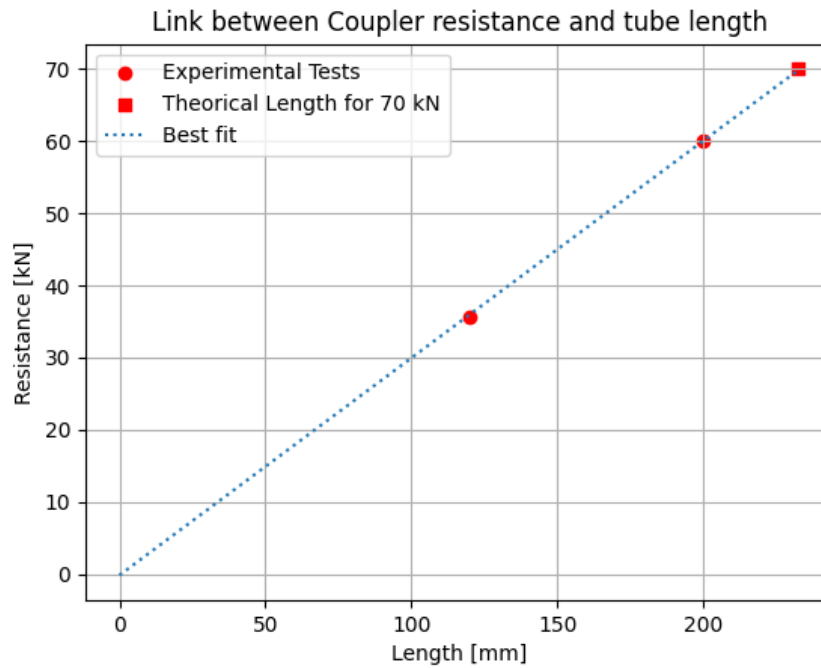


Figure 4.20: Evolution of the IRT-M2P GFRP-Steel coupler maximal resistance with the steel tube's length

As mentioned before, a DFOS was installed along the whole specimen, going from the steel extremity to the GFRP one, passing through the coupler. Fig 4.21 represents a sample of this DFOS data at certain loading stages. The strain behavior of the different parts of the mechanism can be observed on it. The first thing that can be seen is the proper strain distribution in the GFRP bar, with value similar to the theoretical one obtained with $E_{GFRP} = 54GPa$ and a slightly less linear behavior along the steel bar but with still very similar values than those obtained with $E_s = 210GPa$. (See Table 4.6)

Force [kN]	Strain GFRP [μ Strain]	Strain Steel [μ Strain]
10	1396	421
20	2791	842
30	4187	1263
40	5582	1684
50	6978	2105

Table 4.6: Theoretical Strain in the GFRP and the Steel rebar along SGFRP-I200 at different loading stages

As the load increases, it takes more and more GFRP-epoxy bond length to transmit the stress, which can clearly seen in the strain propagation from the GFRP-Coupler interface to the GFRP-Steel interface inside the coupler, initially at 0. The same behavior can be seen in the steel section of the coupler, meaning that the screws act in a similar way to the epoxy to transmit stress. Finally, a strain peak of 0.4% can be observed at the GFRP-Steel interface inside the coupler, which can be explained by the total lack of bond between the two rebars. At this particular point, the fiber is

glued on each extremity of both rebars and a small gap may appear between the two rebars which are working independently, creating a local stress concentration for the fiber.

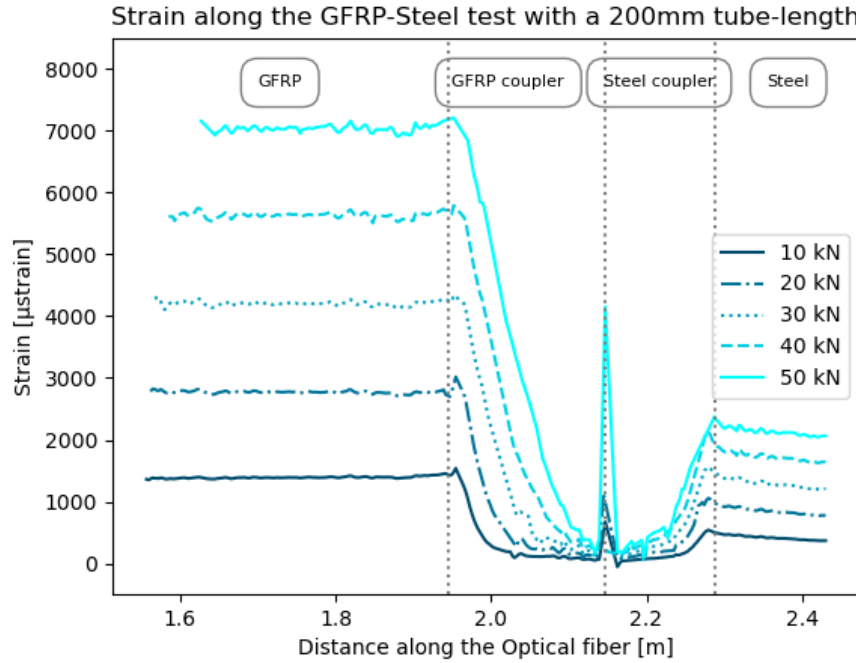


Figure 4.21: Sample DFOS's data of the strain along SGFRP-I200 at different loading stages

Using the Bond Stress Peak $\sigma_{bond,peak}$ calculated earlier, the strain behavior in the GFRP, in the Steel and inside the GFRP side of the coupler can be approximated under any load F :

$$1. \text{ In the GFRP rebar: } \epsilon = \frac{F}{E_{GFRP}A_{GFRP}} \quad (4.1)$$

$$2. \text{ In the GFRP coupler: } \epsilon(x) = \max\left(\frac{F}{E_{GFRP}A_{GFRP}} - \frac{4\sigma_{bond,peak}}{d_{GFRP}E_{GFRP}}x, 0\right) \quad (4.2)$$

$$3. \text{ In the Steel rebar: } \epsilon = \frac{F}{E_sA_s} \quad (4.3)$$

$$\text{With : } \sigma_{bond,peak} = \frac{F_{max}}{P_{GFRP}L_{steel \text{ tube}}} \quad (4.4)$$

An equation similar to Eq4.2 could be approximated for the steel part of the coupler but the behavior is much more complex due to the different screws and the purpose of this thesis is mainly to design a GFRP-SMA coupler, the steel part is thus not that important.

These simple equations proved to be quite successful, especially for higher loads, as visible on Fig4.22, even though it underestimates the strain around the interfaces.

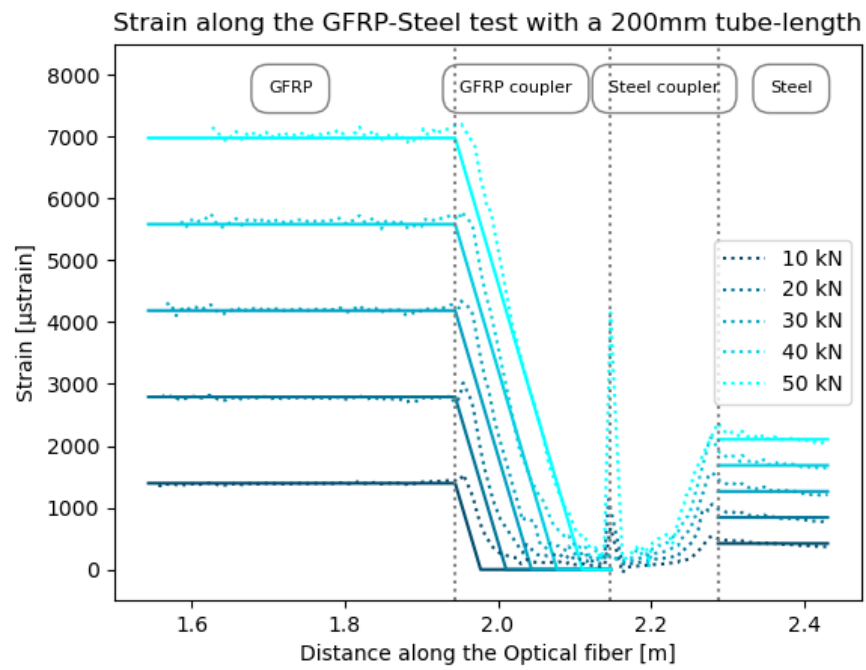


Figure 4.22: DFOS sample data along SGFRP-I200 (dotted lines) compared to model equations (straight lines)

4.4 GFRP-SMA coupler

4.4.1 Coupler description

The aim of this test is to validate a GFRP-SMA coupler configuration ensuring enough tensile resistance to develop the SE properties of the SMA, corresponding to a strain above the pseudo-yield strain $\epsilon_{nu} = 0.013[mm/mm] = 13000\mu Strain$, where the "strain plateau" begins, as determined by Almeida et al.[8]. The coupler is a fusion between the half SMA coupler detailed in section 4.2 and the SGFRP-I250 coupler configuration from section 4.3. Figures 4.23 and 4.24 illustrate the configuration of the GFRP-SMA coupler, specifically designed for the IRT-M2P rebar based on the results from section 4.3.

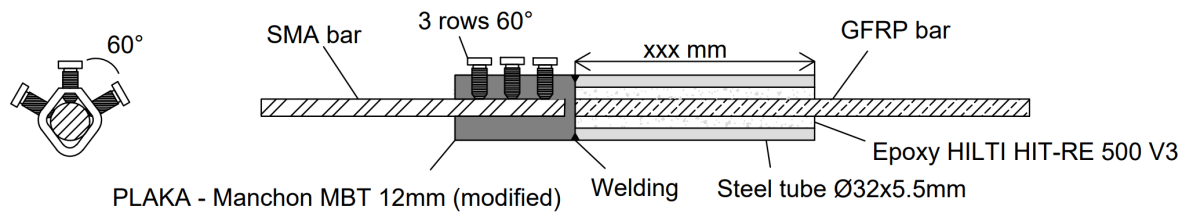


Figure 4.23: Schematic drawing of the GFRP-SMA mechanical coupler



Figure 4.24: Mechanical Coupler for the GFRP-SMA conection

4.4.2 Testing procedure and instrumentation

To minimize the moment concentration [35] on the first screw of the SMA coupler described in section 4.2, a GFRP-SMA-GFRP configuration was used with two couplers as illustrated in Figure 4.25. The anchorages of the GFRP were made according to the norms [43][10].

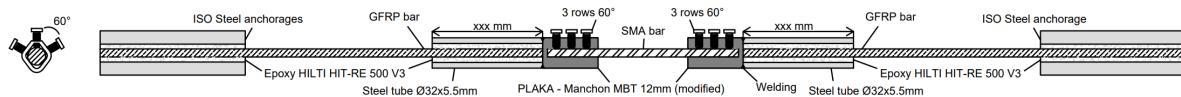


Figure 4.25: Schematic drawing of the testing assembly for the GFRP-SMA connection

The first test is a monotonic tension test using an IRT-M2P GFRP bar, instrumented with the two usual extensometers. An attempt was made to use DFOS along the whole specimen, through

both couplers. During the installation, the fiber was damaged by sharp elements in the coupler geometry. Running this fiber through two couplers is a highly technical and delicate operation. This initial attempt was an opportunity to appreciate the difficulty of the manipulation for the second test and unfortunately proved to be unsuccessful. The results of this test are thus purely based on the data from the different extensometers.

The second test is a reverse cyclic test on the same configuration, with a Sireg's Glasspreer GFRP rebar, and successfully equipped with DFOS. To prevent the data loss of the fiber in the GFRP sections, a new grooving method using a fixed angle grinder and a sliding rail for the bar to be grooved (See Fig 4.26) was adopted. The procedure is detailed in Annex A. For this test, the SMA part of the coupler was also filled with Epoxy. If this coupler was embedded in concrete, this would also have to be done to prevent any gravel or sand from entering the coupler and causing unexpected disturbances in its behavior. As this is the final configuration of this thesis, it was decided to test if it was feasible. To ensure a proper connection between the screws and the SMA bar, they were tightened to rupture before injecting the epoxy.



Figure 4.26: Grooving of the GFRP bar

For the tension parts of the loading protocol, the reference value of the pseudo-yield strain of the SMA $\epsilon_{nu} = 0.013[mm/mm]$ was used and decomposed from 25% to 100% (beginning of the SE phase) and then multiplied at each cycle to demonstrate the SE properties. The aim was to reach cycles 5-7 (See Table 4.7), where the SMA undergoes significant elastic deformations ("Strain plateau"). In cycle 7 and above the SMA would enter its plastic deformation phase but they seem unrealistic compared to the values reached in the previous tests on SMA connection. These tensile phases are easily drivable with a 25mm extensometer installed on the SMA section of the test.

For the compressive parts of the protocol, the assumption had to be made that the entire assembly was a GFRP bar. The calculations were based on its theoretical maximal compression loading. The compressive behavior of GFRP can be separated into 3 sections, depending on its free length. For short sections, the bar will crush whereas for long sections, buckling will prevent crushing. For mid-length sections, a hybrid crushing-buckling behavior occurs. This behavior can be modeled using the equations from Gao et al. [30]:

(1) Crushing

$$\sigma_1 = \sigma_C (0 \leq \lambda \leq 30) \quad (4.5)$$

(2) Buckling & Crushing

$$\sigma_2 = \alpha \times (\lambda - 30) + \sigma_C (30 \leq \lambda \leq 50) \quad (4.6)$$

(3) Buckling

$$\sigma_3 = \beta \frac{\pi^2 EI}{\left(k \frac{\lambda d}{4}\right)^2} \times \frac{4}{\pi d^2} (50 \leq \lambda) \quad (4.7)$$

For the Sireg's rebar, the theoretical maximal load $F_{c,max}$ for this assembly of 840mm is 1kN (See Fig 4.27). Because this assumption of a simple GFRP bar instead of the actual complex assembly is really safe to model the global buckling behavior of the assembly, the compression will go up to 2kN (See Table 4.7)

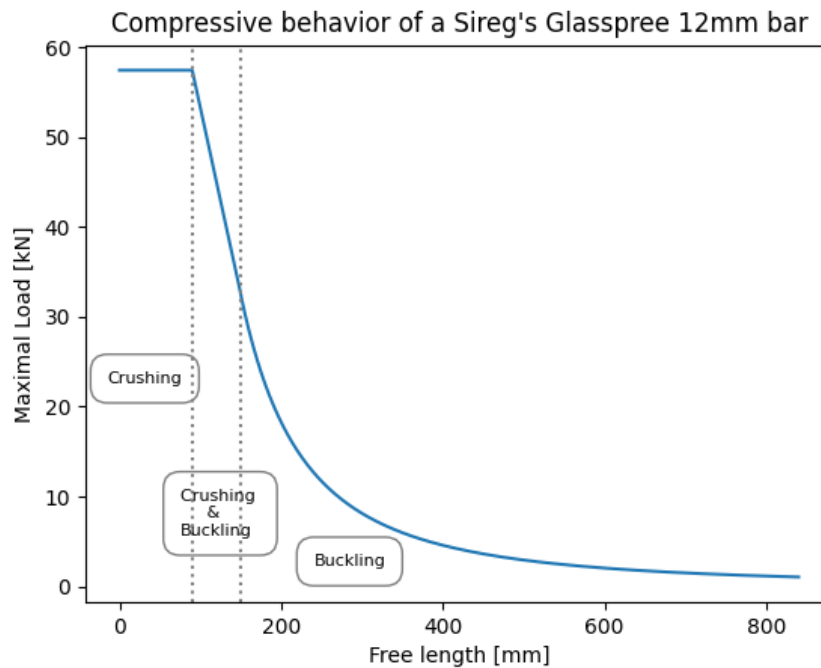


Figure 4.27: Compressive behavior of a Sireg's Glassfree 12mm bar

Table 4.7 and Figure 4.28 summarize the loading protocols and the different cycles, emphasizing on the huge difference between the values reached in tension compared to those in compression.

	Cycle 1	Cycle 2	Cycle 3	Cycle 4	Cycle 5	Cycle 6	Cycle 7	Cycle 8	Cycle 9
% ϵ_{nu}	25	50	75	100	200	300	400	500	600
% $F_{c,max}$	25	50	75	100	200	200	200	200	200

Table 4.7: Loading protocol of the GFRP-SMA cyclic test

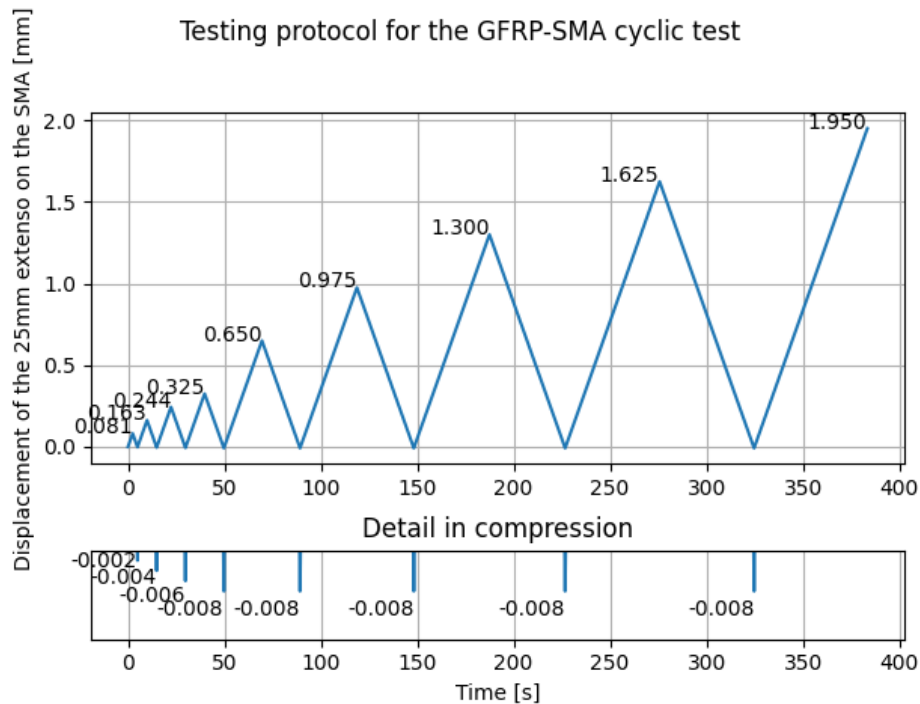


Figure 4.28: Tensile Loading protocol of the GFRP-SMA cyclic test

4.4.3 Results of the tests

Tensile test of the GFRP-SMA coupler

The simple tensile test turned out to be conclusive, with the same mode of rupture than the SMA-Steel coupler (brittle failure of the SMA in the first row of screws of the coupler, see Fig 4.13) and a maximal tensile strength of 66.7kN, corresponding to a strain of 526.8 MPa in the SMA.

The Strain-force graph of this test presented on Fig 4.29 shows a first slippage of the SMA in the coupler which caused the fall of the extensometer. However clear strain plateau in the SMA can be observed before the slippage. The graph also represents a succession of loading and slippage in the coupler after this first slip and before the brittle failure of the SMA.

Cyclic test of the GFRP-SMA coupler equipped with DFOS

As presented on Fig4.30, the cyclic test reached the pseudo-yield value of the SMA but unfortunately slipped during the 5th tension cycle, at 56,2 kN (i.e. 442 MPa in the SMA). This difference of performance compared to the tension test can be explained by the loading and unloading which might have damaged the couplers, especially the compressive phases where some buckling of the mechanism was observed (mainly around the SMA). This problem of reduced performance is problematic if such a coupler is used in buildings subject to earthquakes, synonym of important tension-compression cycles. However, the risk is in fact less significant because the buckling that caused the problem will be much more limited in a real case due to the confinement offered by the concrete in which the coupler will be placed.

Coming back to the force-displacement result of the cyclic test, some residual displacement can be observed after the 4th cycle. After the first slip, the behavior was similar to that of a tensile

test with consecutive loading and slippage phases. However, contrary to the other test involving SMA, this time the SMA slipped significantly more in the coupler and never broke (See Fig 4.34). This might be due to the presence of the epoxy in that part of the coupler that allowed for a better distribution of the load of the screws or to the heads of the screws that had been perfectly flattened in a machine, thus preventing local punching.

The installation of the DFOS on this sample was a success and it provided valuable information. It validated the strain in the SMA measured by the extensometer, but only up to 12000 μ Strain, after which it failed to measure the strain further than the GFRP-SMA interface. As stated by Hoult et al. [40], this value is consistent with other experimental studies [87] [51] where data losses appeared as soon as the yield point was reached. This is because the interrogator unit, similar to the one used in this test, was unable to analyze and detect peaks in the frequency spectra. The comparison between the extensometer and the DFOS data can be found in Fig 4.31 and the strain measured along the entire mechanism is represented in Fig 4.32 at the different peak values. The last curve illustrates the loss in data further than the GFRP-SMA interface after a certain maximal strain was reached. As predicted, the highest strain concentrated around the SMA.

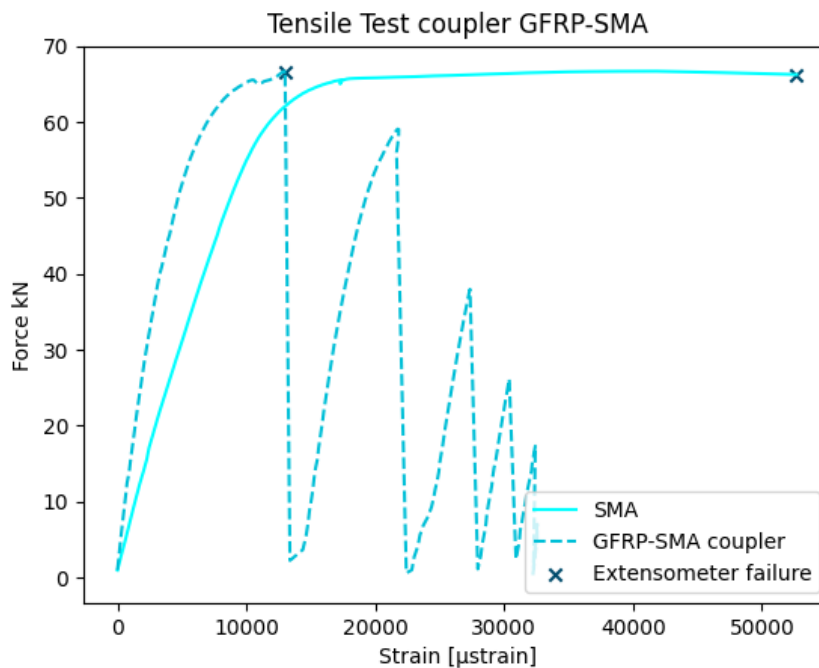


Figure 4.29: Tensile test GFRP-SMA : Force-Strain

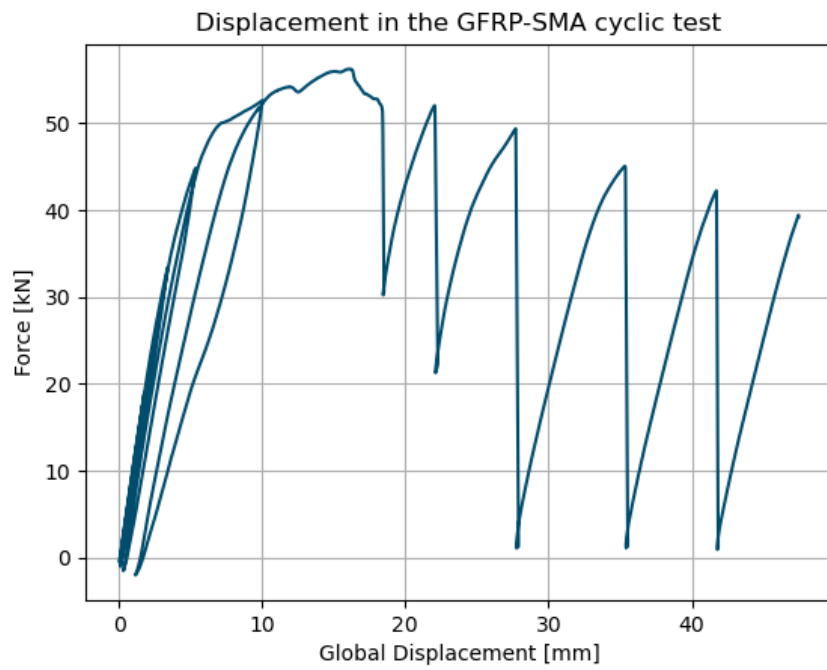


Figure 4.30: Displacement of the GFRP-SMA mechanism in the cyclic loading

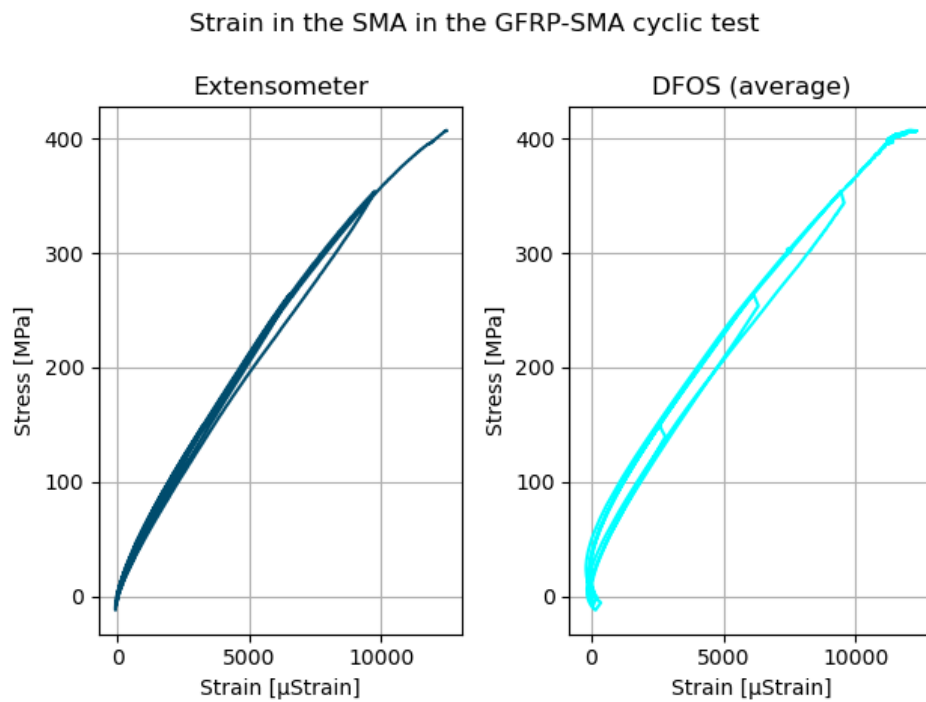


Figure 4.31: GFRP-SMA cyclic elastic phase DFOS vs extensometer

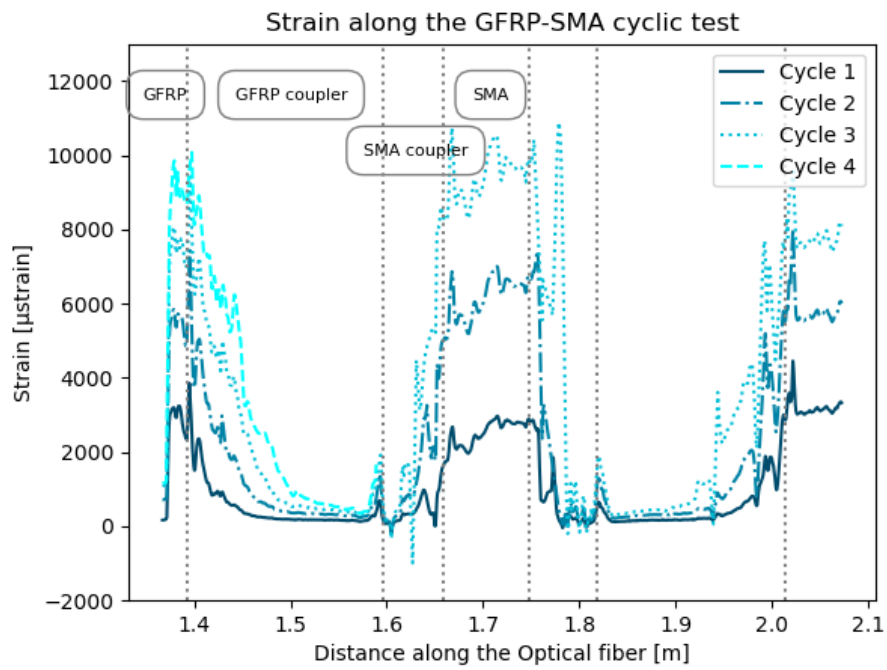


Figure 4.32: Strain along the GFRP-SMA cyclic test measured by the DFOS

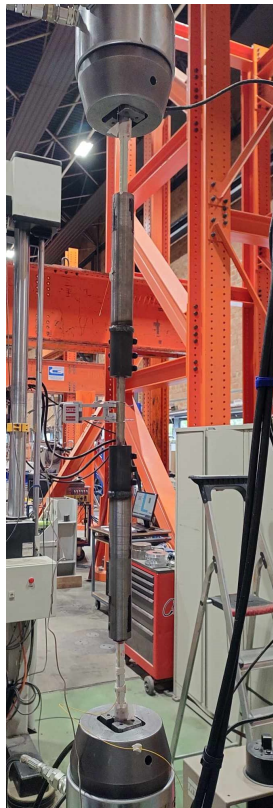


Figure 4.33: GFRP-SMA cyclic test setup as in Fig 4.25 instrumented with a 25mm extensometer on the SMA rebar and DFOS



Figure 4.34: Slippage of the SMA during the GFRP-SMA cyclic test

4.5 Conclusions

This series of experiments led us to these conclusions regarding the different coupler configurations:

- The Steel-SMA coupler is adequate for most of its applications as it reaches significant strain in the SMA before failure. However, it requires a lot of manufacturing to produce and other simpler solutions can provide similar results.
- The GFRP-Steel coupler is more than sufficient for any application. The steel plasticizes and breaks before any failure in the coupler, and the length of the steel tube can be adapted to any load and GFRP bar. However, this length is still significant and could prove to be a problem in RC structures.
- The main limitation of the GFRP connection with the steel tube is the bond strength of the GFRP coating in shear.
- The GFRP-SMA coupler proved to be sufficient in the monotonic loading test, but the performance was lower in the cyclic test.

CHAPTER

5

NUMERICAL ANALYSIS OF GFRP AND GFRP-SMA RC BEAMS

Due to a series of unforeseen delays, the focus of this thesis shifted from the pure experimental validation of RC structures combining GFRP and SMA to a more detailed numerical analysis of these materials behavior in a concrete beam under loading.

This chapter describes the modelling assumptions and validation of GFRP rebar embedded in concrete using Vector2. The objective is to predict the behaviour of future experimental beam tests, starting with small-scale beams, which will be designed in this paper, and then moving on to high-rise walls, which were the focus of a joint thesis here at UCLouvain[78].

All these experiments will use the different materials and mechanical coupler configurations described in Chapter 4.

5.1 Introduction to Vector 2

In order to predict the response of a future GFRP RC beam test, a nonlinear finite element analysis was performed. This was accomplished using a program named Vector 2, developed by the University of Toronto[31]. It was designed to provide two-dimensional nonlinear finite element analysis on concrete beam sections, based on the Modified Compression Field Theory from Vecchio and Collins [52]. The functionality of the program and the different parameters available are thoroughly detailed in the User Manual [83].

Many literature papers describe Vector 2 analyses of RC structures reinforced with steel, but limited documentation is available regarding the modelling of GFRP reinforcements with Vector 2 software. The most extensive study on this subject was conducted by Sheikh et al [71]. Therefore, some of the assumptions in this chapter are based on their research, as well as the research of Vint et al.[81] who have examined the boundary conditions for GFRP.

In the following sections are studied the different model parameters that have a significant influence on the GFRP-reinforced concrete section response prediction. Firstly, an analyse is carried out on the bond conditions model for GFRP. Next, the model parameters for GFRP RC sections are studied and validated, based on reference beams from the literature. This validated model will be the basis for the development of a numerical prediction of future experimental beams setups.

5.2 Bond models

The characteristics of GFRP reinforcements differ in many ways from those of conventional steel. The main point of attention concerns the coating of the reinforcements affecting the bond-stress relation between the GFRP and the concrete. As noticed in Chapter 4, different GFRP coating surfaces are available leading to significant differences in the performances results. Due to this bond property, GFRP rebars cannot be considered as perfectly bonded in concrete, contrary to what is usually modelled for steel rebars. A bond-slip model must therefore be studied in Vector 2 for each type of GFRP coating, in order to approximate as closely as possible the behaviour at the interface between the coating and the concrete.

Before going any further into the bond-conditions analysis, it is important to specify the general physical properties used to model the GFRP material with the Vector 2 software. Two possibilities are available to consider the perfect elastic behaviour of the GFRP in tension and compression:

1. The first option is to use a ductile steel reinforcement in the reinforcement properties window. Subsequently, apply a linear hysteretic response in the Job Models of the software. Beware that with this option, the program will use the Ultimate Strength assigned to the reinforcement to create the linear response of the rebar, ignoring the Yield Strength.
2. The second option is to use the same ductile steel reinforcement but assign an Ultimate Strength value very close to that of the Yield Strength as represented on Fig. 5.1. Then, run the simulation with any of the available reinforcement models. This option can be useful when another type of reinforcement besides GFRP is on the same model.

Define Reinforcement Properties

Reinforcement Type

Type: Reinforcement 1

Add

Update

Delete

Reinforcement Properties

Reference Type: Ductile Steel Reinforcement

Cross-Sectional Area: 132.66 mm²

Reinforcement Diameter, Db: 13 mm

Yield Strength, Fy: 1100 MPa

Ultimate Strength, Fu: 1100 MPa

Elastic Modulus, Es: 54000 MPa

Strain Hardening Strain, esh: 20.37 me

Ultimate Strain, eu: 20.37 me

Thermal Expansion Coefficient, Cs: * 0 ue/°C

Prestrain, Dep: 0 me

Unsupported Length Ratio, b/t: 0

Color

Reinforcement material types to be used for truss elements only.

OK Cancel

Figure 5.1: Typical parameters for GFRP rebars in Vector 2[31]

Once the physical properties have been implemented, the focus can shift to the bound conditions of the GFRP rebars in concrete. As explained, their surface differs from standard steel and the interaction between rebars and concrete cannot be considered as perfect.

To model the bond-slip behavior, bond-slip elements were used in Vector 2. Without them, reinforced concrete elements and discrete reinforcement elements share common nodes, ensuring compatible displacements and a perfect bond between them.

Bond-slip elements act as flexible interfaces between these elements. They are defined by paired nodes with the same coordinates, where one node connects to a reinforced concrete element and the other to a discrete reinforcement element. As bond stresses increase, bond slip occurs, causing relative displacement between the paired nodes and the connected elements.

In Vector2, two types of bond-slip elements exist: the link element and the contact element. For this model, link elements were used, which are designed to be the most accurate for models with constant bond slips. As explained in the Vector 2 User Manual[83] and presented in Figure 5.2, the link element can be seen as two orthogonal springs, linking each node from the reinforced concrete element to the one from the discrete reinforcement element as explained before. The first spring deforms tangentially to the rebar in the concrete, representing bond slip and bond stresses. The second one deforms radially and represents radial displacements and stresses.

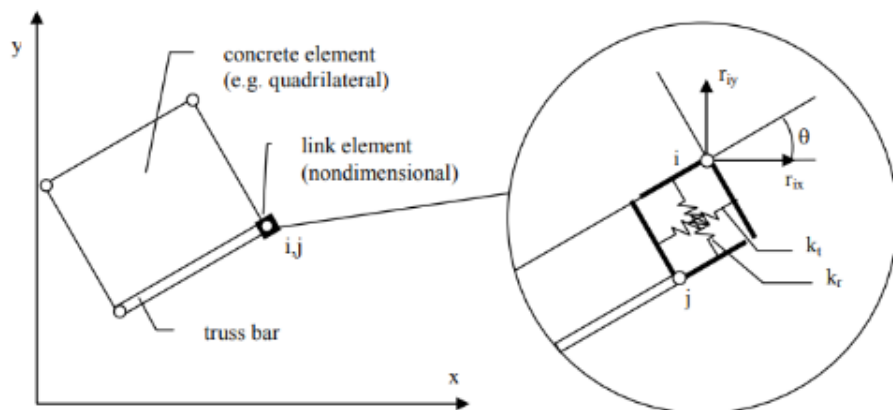


Figure 5.2: Link element theory[83]

As discussed in Chapter 3, GFRP bars can have very different coatings. Consequently, their bond-slip strength in concrete varies significantly from one bar to another. In this numerical simulation will be modeled the bond-slip of two different types of coating: a relatively smooth coating, similar to the M2P bars described in Chapter 3, and a ribbed surface profile like the Sireg's bars in that same chapter.

The bond-slip experimental results that will be replicated with this model are provided by Christophe Dandois[23] who is conducting a parallel master thesis about anchorage-slip effects on reinforced concrete members. His averaged post-processed data from normalised pull-out tests were used.

In this document are described pull-out tests carried out on samples of smooth GFRP reinforcement, named 'Calendering GFRP'. The same tests were also carried out with ribbed bars, called 'Machined GFRP'. The free-end slip/bond-stress relationships from these tests will serve as the basis for the numerical model. To validate the boundary conditions of GFRP in Vector 2, pull-out tests

were performed in the software for these two types of rebars using different bond conditions and analysis models [81].

The pull-out simulation setup is represented in Figure 5.3. The input properties are the same as the experimental ones: 38 MPa concrete, typical GFRP 13 mm characteristics, and an anchorage length in the concrete equal to 4 times the nominal diameter of the rebar.

The GFRP rebar is bonded with link elements as explained before. A small mesh size of rectangular 5x5 mm is used for the concrete to obtain the most accurate results.

The superior anchorage is made of high-resistance steel on which no slippage will occur due to perfect bond conditions. The mesh size (20x20 mm) is larger than the one used for concrete because there is not need of highly precise results in this area. The thickness perpendicular to the page was considered the same as the length, resulting in a squared cross-section. The displacement is then applied at the top of the steel piece in 0.1 mm increments to a final displacement of 10mm. It must be kept in mind that the displacement result given by the program will be an overall displacement of the superior steel anchorage, from which the deformations of the free-length of GFRP will have to be subtracted to determine the slippage in the concrete. Vint et al. used the same setup and concluded that the deformation of the steel anchorage can be neglected compared to the total displacement[81]. It is important to note that the geometry of the entire configuration is critical to the reliability of the results. The mesh size in the concrete is another important factor. The user Manual advises to use a small mesh size for better performances using link elements. While a mesh sensitivity analysis was not conducted here, as the focus was on a very local effect, such an analysis was conducted for the RC beam simulations in Section 5.3.3

Finally, the pre and post-peak concrete models used for the analysis are the ones recommended by Vint et al. for a pull-out model: the Smith-Young and the Saenz/Spacone models [81].

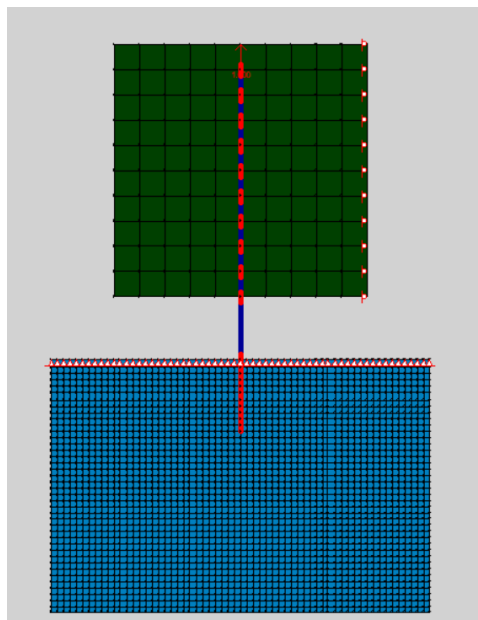


Figure 5.3: Pull-out setup in Vector 2

Different anchor lengths in concrete are possible depending on the pull-out test procedures. It is also interesting to observe the conditions at the end of the reinforcement. In this case, the reinforcement was modeled with a free end. However, there are other configurations possible such as

a U-shape rebar or an anchor fixed to the end of the reinforcement. These different configurations presented on Figure 5.4 have been studied by Vint et al. and provide the best possible approach to the various possible configurations.

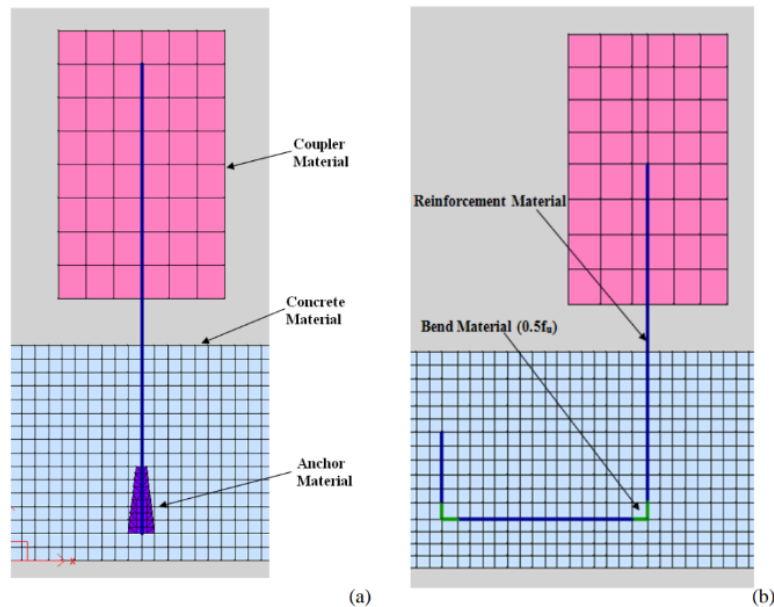


Figure 5.4: Different end-conditions analysed by Vint et al.[81]

The first GFRP bond-slip modelled in Vector 2 is the one from the Calendering GFRP rebars. As these rebars are quite smooth, a first attempt was made by running different model analyses using the pre-defined model 'Embedded Smooth Rebars'. As noted by Vint et al., the best confinement factor for this type of pull-out test is equal to zero. For reminder, this factor can vary between 0 and 1. A confinement pressure factor of zero corresponds to the unconfined case of the rebar in the concrete. On the contrary, a factor equal to 1 is the case of a very high pressure factor confinement, confining the reinforcement in the concrete. The main consequence of a large factor is an increase in the stress at which failure-peak occurs, as well as a higher residual post-peak stress. All the plots of pre-defined models in the next sections are made with this confinement factor equal to 0. The GFRP rebar parameters are those described at the beginning of this section.

Due to the imperfect bond-slip results obtained with the pre-defined models, a new bond-stress model was implemented with the custom input option in Vector 2, based on a trial-and-error approach. The final parameters are presented in Figure 5.6. Note that the bond model analysis applied to this custom input does not influence the results. These new boundary conditions are represented in Figure 5.7, compared to two different analyses (Eligehausen and Gan-Vecchio) applied on a smooth embedded rebar model.

The two different analyses, Eligehausen and Gan-Vecchio, are each modeling a different bond stress-slip using different equations available in the Vector 2 Manual [83]. Those relations are depending on the confinement factor as mentioned before. Both theoretical models are presented on Figure 5.5, on which the influence of the confinement factor on the responses can clearly be seen.

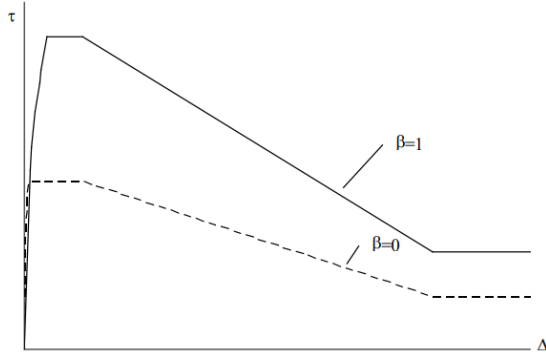


Figure 76: Eligehausen bond stress-slip response

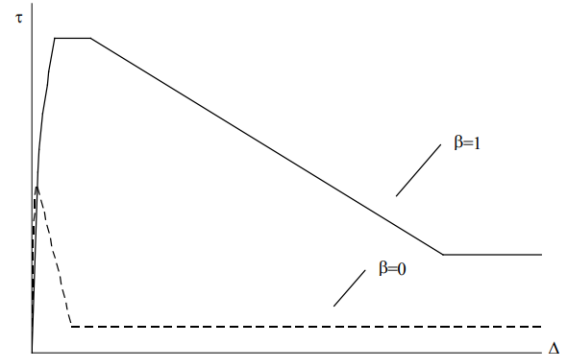


Figure 77: Gan bond stress-slip response

Figure 5.5: Eligehausen and Gan bond Stress-Slip models from Vector 2 User Manual [83]

As mentioned previously, a second type of GFRP rebar named 'Machining GFRP' has to be modeled, with a ribbed surface. Once again, because no existing bond model suited the analysis, a custom input option was used to find the parameters that best matched the experimental results (See Figure 5.8). The numerical pull-out setting was the same as the one for the calandering GFRP rebar. Figure 5.9 shows the numerical bond-stress results compared to the experimental ones.

The 3 values used for the custom input are respectively the ones for the pre-peak curve, the peak point and the residual stress plateau. The 3 values used in each model seem strange in view of the Stress-slip curves that have to be obtained. It seems to be an unexplained scaling effect in between the values encoded in the software and the results. However, a trial-and-error exercise confirmed that these values are the closest to the experimental behaviour. The efficiency of the bond models will be confirmed later on GFRP beam (cfr. Section 5.3.4).

Define Bond Properties

Bond Type
Type:

Bond 1
Bond 2

Add
Update
Delete

Reference Type:

Embedded Bars - Custom Input

Bond Properties for Embedded Bars

Confinement Pressure Factor:

0

Min (Bar Clear Cover, Spacing), CMin:

0

mm

No. of Reinforcement Layers thru Depth:

0

Hooked Bar:
☐

Bond Properties for Externally Bonded Plates or Sheets

Bonded Surface Area, A_o:

0

mm²/m

Bond Stress-Slip Curve Reference Points

Point 1, U1

0.67

MPa

S1

0.03

mm

Point 2, U2

0.74

MPa

S2

0.22

mm

Point 3, U3

0.4

MPa

S3

1.4

mm

Bond Stress-Slip Compression Enhancement Factor

Comp. Stress

0

MPa

Factor

1

Bond Stress-Slip Curve Tension Reduction Factor

Tens. Stress

0

MPa

Factor

1

Color

Bond material types to be used for interior or exterior bonded elements.

OK
Cancel

Figure 5.6: Bond model analysis for GFRP Calandered rebar

Bond model Calandering GFRP

Free-end slip [mm]	Experimental [Mpa]	Custom Input [Mpa]	Embedded Smooth Rebar - Eligehausen [Mpa]	Embedded Smooth Rebar - Gan-Vecchio [Mpa]
0	0.0	0.0	0.0	0.0
0.5	3.0	3.0	4.2	4.2
1.0	2.5	2.5	4.2	4.2
2.0	1.8	1.8	3.5	0.6
3.0	1.6	1.6	3.0	0.6
4.0	1.6	1.6	3.0	0.6
5.0	1.6	1.6	3.0	0.6
6.0	1.6	1.6	3.0	0.6
7.0	1.6	1.6	3.0	0.6
8.0	1.6	1.6	3.0	0.6

Figure 5.7: Bond model analysis on Calandered GFRP Rebar

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Define Bond Properties

Bond Type
Type:

Bond 1
Bond 2

Add
Update
Delete

Reference Type:

Embedded Bars - Custom Input

Bond Properties for Embedded Bars

Confinement Pressure Factor:

0

Min (Bar Clear Cover, Spacing), CMin:

0

mm

No. of Reinforcement Layers thru Depth:

0

Hooked Bar:
☐

Bond Properties for Externally Bonded Plates or Sheets

Bonded Surface Area, A_o:

0

mm²/m

Bond Stress-Slip Curve Reference Points

Point 1, U1

1.3

MPa

S1

0.2

mm

Point 2, U2

1.65

MPa

S2

0.7

mm

Point 3, U3

0.3

MPa

S3

6.4

mm

Bond Stress-Slip Compression Enhancement Factor

Comp. Stress

-300

MPa

Factor

1

Bond Stress-Slip Curve Tension Reduction Factor

Tens. Stress

0

MPa

Factor

1

Color

Bond material types to be used for interior or exterior bonded elements.

OK
Cancel

Figure 5.8: Bond model analysis for GFRP Machined rebar

Bond model Machined GFRP

Free-end slip [mm]	Experimental [Mpa]	Custom Input [Mpa]	Embedded Smooth Rebar - Elgehausen [Mpa]	Embedded Smooth Rebar - Gan-Vecchio [Mpa]
0	0	0	0	0
0.5	5.5	5.5	4.2	4.0
1.0	6.5	6.5	4.2	2.5
2.0	5.5	5.5	3.5	0.6
3.0	4.5	4.5	3.0	0.6
4.0	3.5	3.5	3.0	0.6
5.0	2.5	2.5	3.0	0.6
6.0	1.5	1.5	3.0	0.6
7.0	1.2	1.2	3.0	0.6
8.0	1.2	1.2	3.0	0.6

Figure 5.9: Bond model analysis on Machined GFRP Rebar

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5.3 GFRP-RC Beam Model

5.3.1 Beams specimens

The validation of a GFRP reinforced concrete beam was made using three different experimental beams from the literature in order to work on the different material parameters and model analysis.

The two first (B2-OC and B3-OC) are experimental beams tested by Falah Hassan et al. [27] (fig 5.10). The longitudinal reinforcements are made of GFRP and the stirrups of steel. Material properties are summarized in table 5.1 . The testing procedure is a monotonic 4-points bending. The beam is enough shear reinforced to have a failure mode due to tensile failure caused by GFRP bar fracturing or by crushing of concrete.

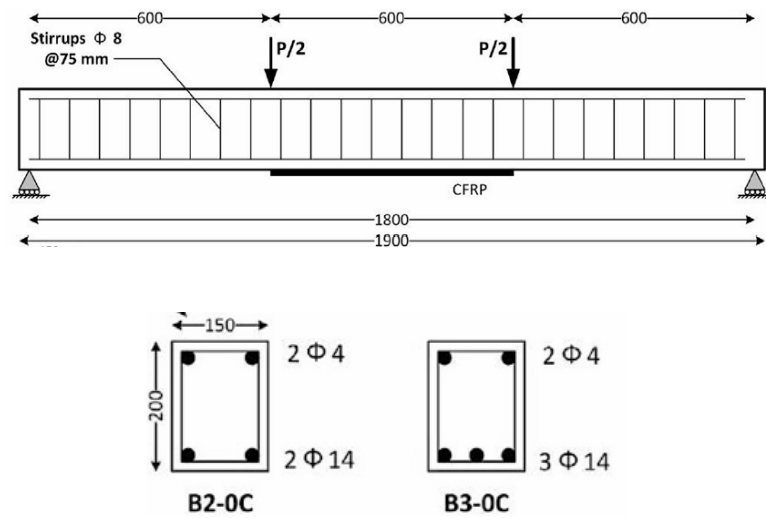


Figure 5.10: B2-OC and B3-OC beams configurations [27]

	Concrete	GFRP
Ultimate Strength (MPa)	32	1200
Modulus of Elasticity (GPa)	37	55
Ultimate Strain ($\mu\epsilon$)		1950

Table 5.1: B2-OC and B3-OC material properties

The third beam used as a validation beam is the JSC32-NT from Sheikh et al. [71]. This beam is represented in Figure 5.11 but has no transversal reinforcement in the JSC32-NT configuration. The test configuration was a 3-points bending. Concerning the material properties, no detailed informations were available in the literature other than the 32 MPa concrete Ultimate Strength. The hypothesis of classical GFRP properties presented in table 5.2 were used for the numerical simulation.

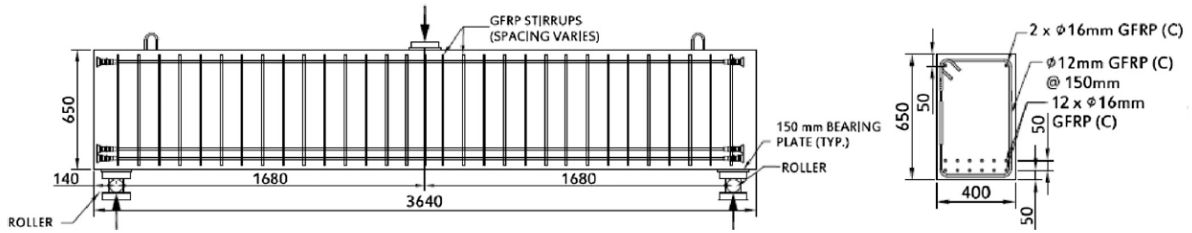


Figure 5.11: JSC32-NT beam configuration [71]

Ultimate Strength	Modulus of Elasticity	Ultimate Strain
1210 MPa	50 GPa	2420 $\mu\epsilon$

Table 5.2: JSC32NT GFRP rebars properties hypothesis

5.3.2 Material Models Analysis

Different concrete models were studied to simulate a GFRP reinforced beam. The first two parameters are the compression pre- and post-peak models. These models characterise the behaviour of concrete in its compressed state before and after crushing. The simulation respectively used a Normal-Strength-Concrete model (Popovics) [63] and a Modified Park-Kent model[45][69], commonly used in simple beam configurations. These models are the most relevant for the validation beams because of the properties of the concretes used in the experimental tests. Indeed, this model from Popovics is used for strenghts up to 45 MPa, according to the conclusions from Palermo and Vecchio[61]. The tension softening model used for the concrete is the one from Vecchio (default) [80] as suggested by Mohr [55][41]

Sheikh et al. [71] developed the critical importance of the Tension Stiffening model used for GFRP beam model analysis. Indeed, GFRPs have very low stiffness, leading to non-realistic deflection prediction if not well modeled. Their conclusion is that an accurate model, combined with good bound conditions, can provide interesting results.

The study of the influence of the Tension Stiffening model was done on the JSC32-NT beam using the different available models : Collins and Mitchel, Bentz, and a model without Tension Stiffening.

A perfect bond condition between concrete and rebars was used, combined to a Normal-Strength-Concrete model (Popovics) [63] and a Modified Park-Kent model as concrete models. The other models available on Vector 2 seem to have a negligible influence compared to the Tension Stiffening models and will therefore be set up with the default values for this study case.

The beam model in Vector 2 complies with the geometry, supports and material physical properties described before. A mesh size of 20 mm was discretized and a monotonic loading was applied with 0.5 mm steps at the middle length of the beam.

Figure 5.12 shows the force-displacement results using the different Tension-Stiffening models. As predicted by Sheikh et al., the model developed by Collins and Mitchell [18] is the one that fits best with the experimental curve. The final models analysis used for the GFRP-RC beams numerical simulations are summarized on Figure 5.13.

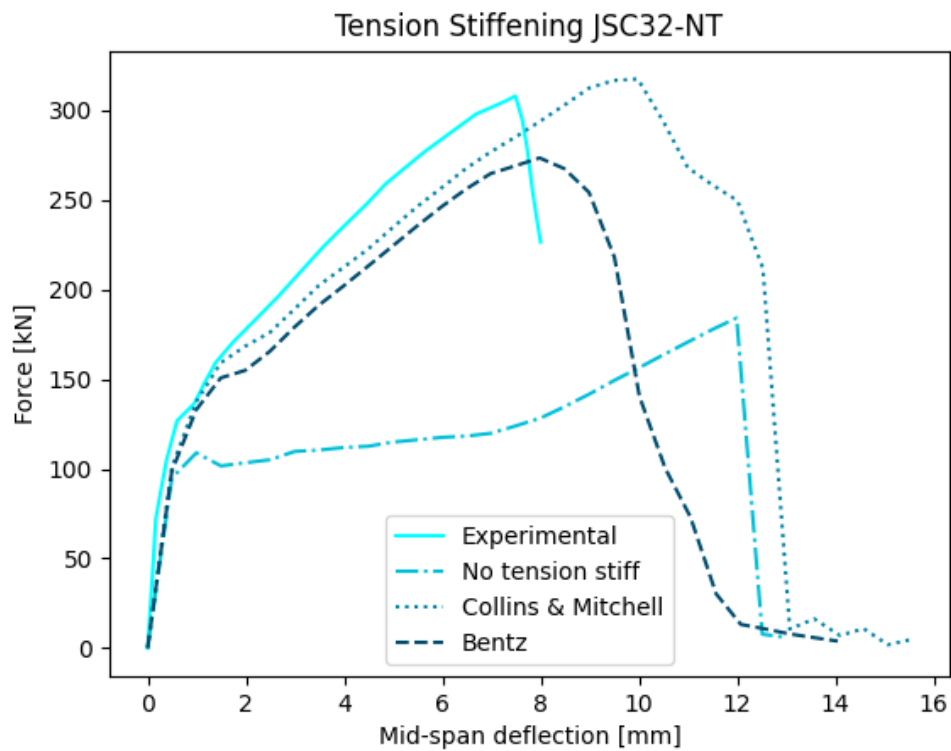


Figure 5.12: Tension Stiffening Models analysis

Define Job

Job Control | Models | Auxiliary | Special

Concrete Models

Compression Pre-Peak: Popovics (NSC) | Confined Strength: Kupfer / Richart

Compression Post-Peak: Modified Park-Kent | Dilation: Variable - Isotropic

Compression Softening: Vecchio 1992-A (e1/e2-Form) | Cracking Criterion: Mohr-Coulomb (Stress)

Tension Stiffening: Collins-Mitchell 1987 | Crack Stress Calc: Basic (DSFM/MCFT)

Tension Softening: Nonlinear (Hordijk) | Crack Width Check: Agg/2.5 Max Crack Width

FRC Tension: SDEM - Monotonic | Crack Slip Calc: Walraven

Creep and Relaxation: | Hysteretic Response: Nonlinear w/ Plastic Offsets

Reinforcement Models

Hysteretic Response: Linear

Dowel Action: Tassios (Crack Slip)

Buckling: Akkaya 2012 (Modified Dhakal-Ms)

Bond Models

Concrete Bond: Harji

Analysis Models

Strain History: Previous Loading Considered

Strain Rate Effects: C: n/c S: n/c

Structural Damping: Not Considered

Geometric Nonlinearity: Considered

Cracking Spacing: CEB-FIP 1978 - Deformed

Reset Options: Basic, Advanced

OK Annuler

Figure 5.13: Final models used for GFRP Beam analysis

5.3.3 Mesh Size Analysis

The last variable that was studied is the influence of mesh size on the numerically modelled beam. Several mesh refinement levels were analyzed to determine the best mesh size. This analysis was carried out on the B20C beam (Results in Figure 5.14) and JSC32-NT beams (Results in Figure 5.15), and validated on the B3OC. The mesh size noted Dx is expressed in mm. The parameters used during this analysis are those determined to be the most reliable in the previous sections, summarized in Figure 5.13. The geometry and material properties are the one from section 5.3.1. The loading protocol is a monotonic load with 0.5 mm increments and applied according to the experimental test.

Mesh size analysis gives results that align with the predictions. In fact, the more refined the mesh is, the better the results of the model will be[83]. This can be explained by the use of link elements, for which the bond conditions work better with greater discretization of the elements.

The validity of this conclusion can be confirmed once again in the numerical analysis of the B3-OC beam with a very large mesh and a second more refined mesh. (See Figure 5.16)

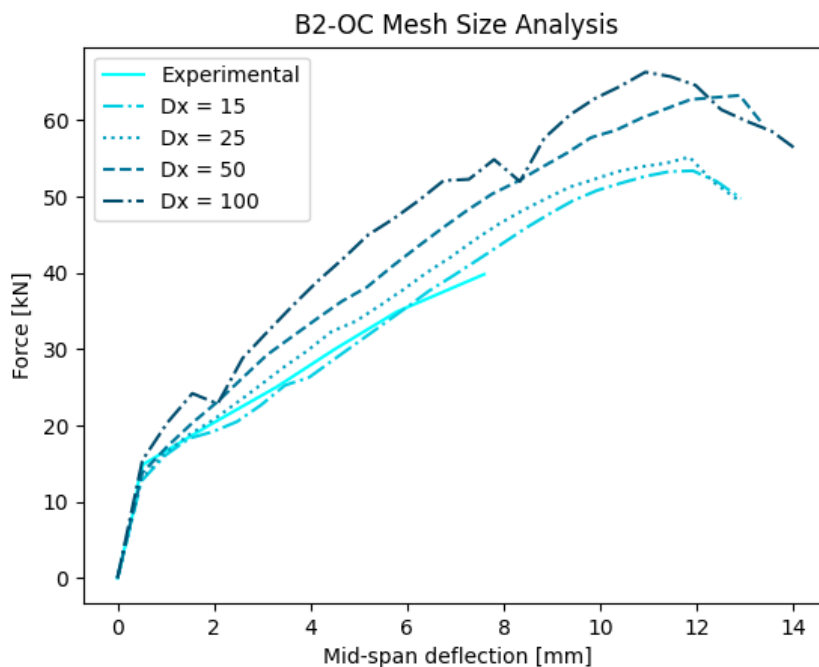


Figure 5.14: B2-OC Mesh Size Analysis

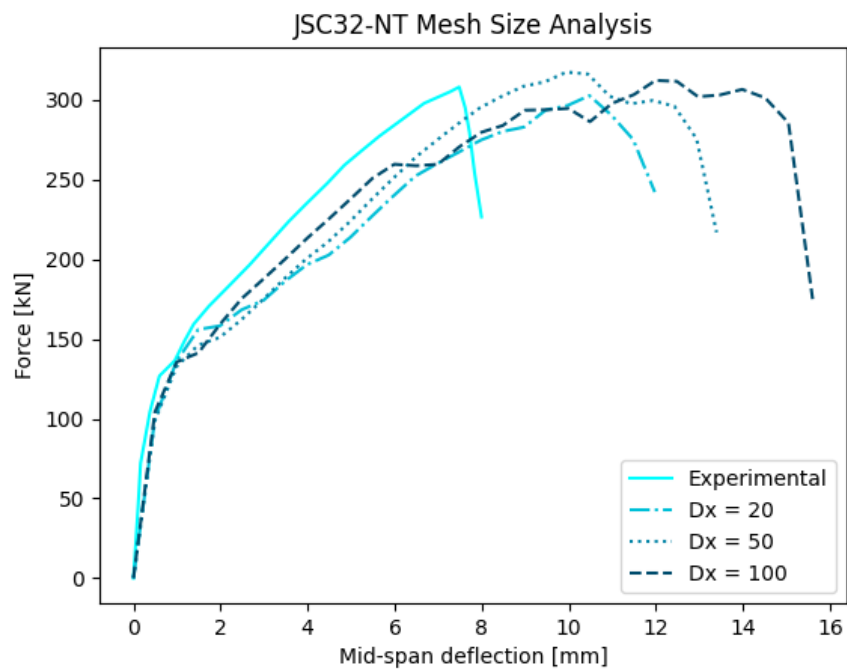


Figure 5.15: JSC32-NT Mesh Size Analysis

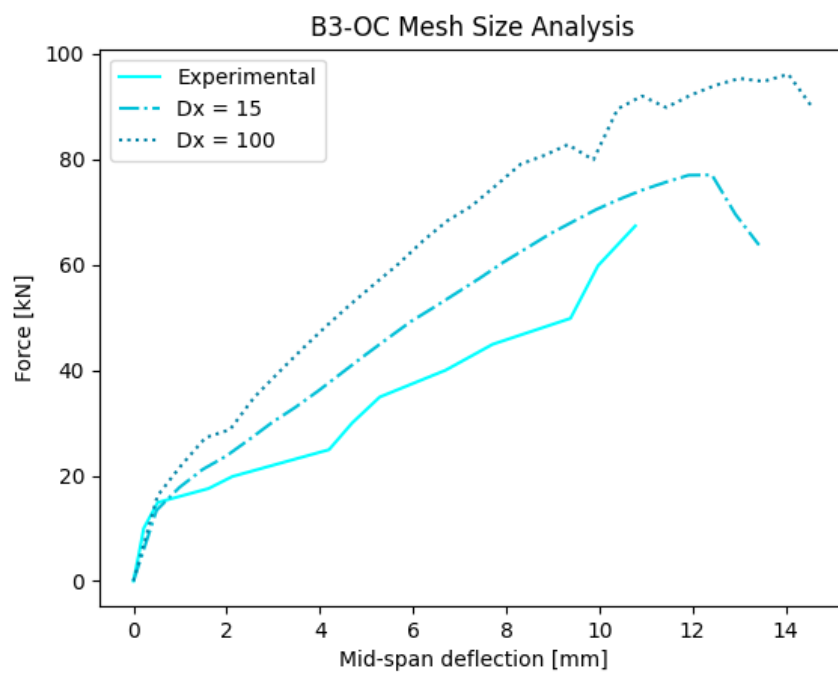


Figure 5.16: B3-OC Mesh Size Analysis

5.3.4 Force-displacement results

Once the best concrete models determined and the mesh size analysis done, the bond model from section 5.2 could be added to the numerical simulation. The following graphs (Figures 5.17, 5.18 and 5.19) represent the three different validation beams, with the concrete parameters, GFRP parameters, and model analysis described before. Each graph represents the force-displacement differences between a perfect bond condition and a customized bond model input. The bond conditions applied on the discrete GFRP rebars are those for a Machined rebar, described in Section 5.2. The type of coating of the rebars used in the experiment isn't mentioned in the papers. The use of the 'Machined' model seems to be a realistic hypothesis because it is the most widely used type of GFRP rebar.

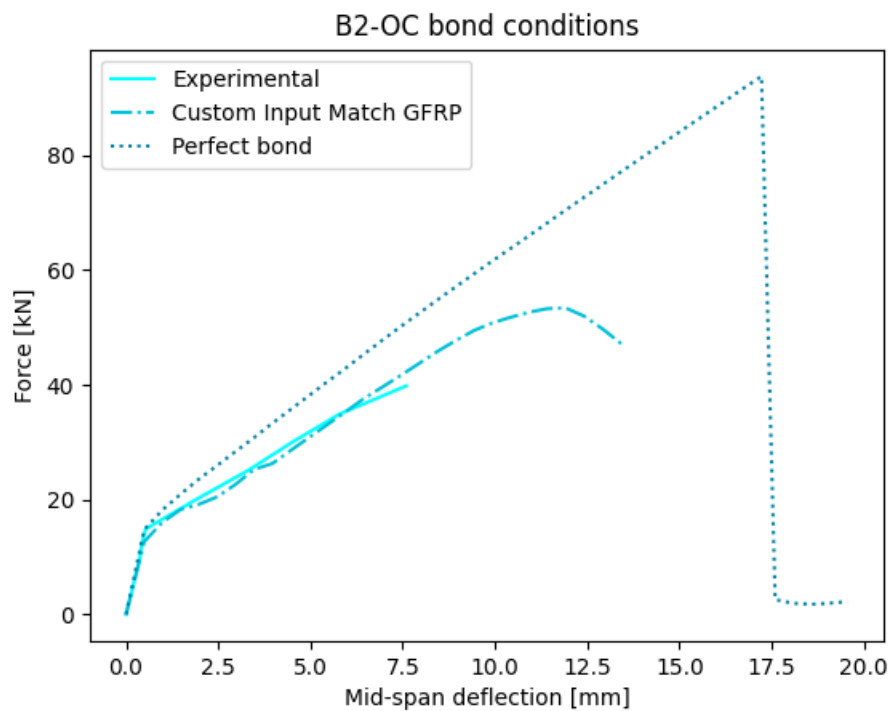


Figure 5.17: B2-OC beam : influence of numerical bond-slip model

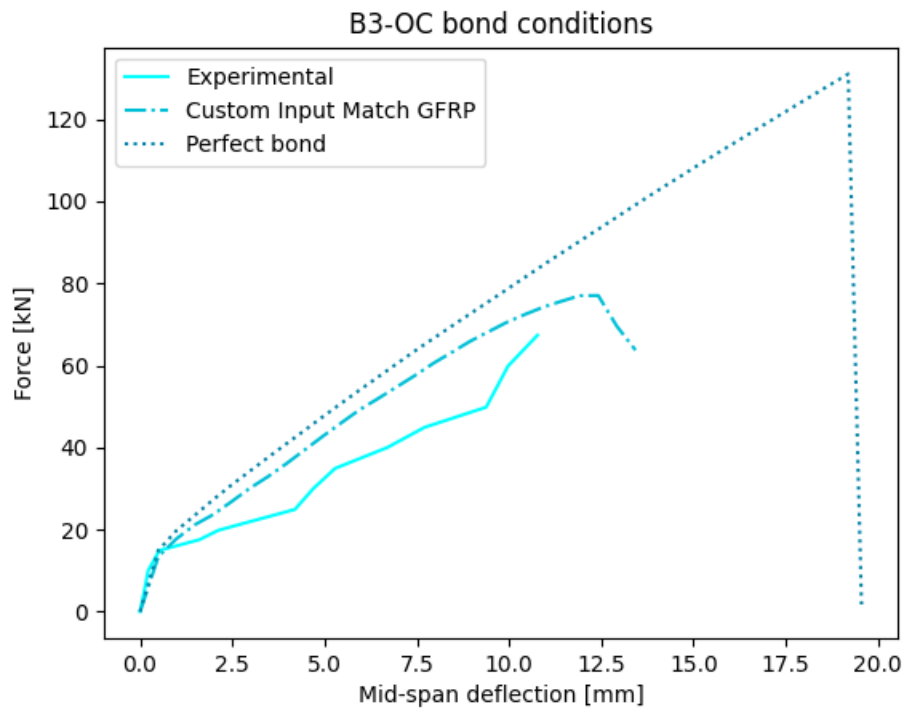


Figure 5.18: B3-OC beam : influence of numerical bond-slip model

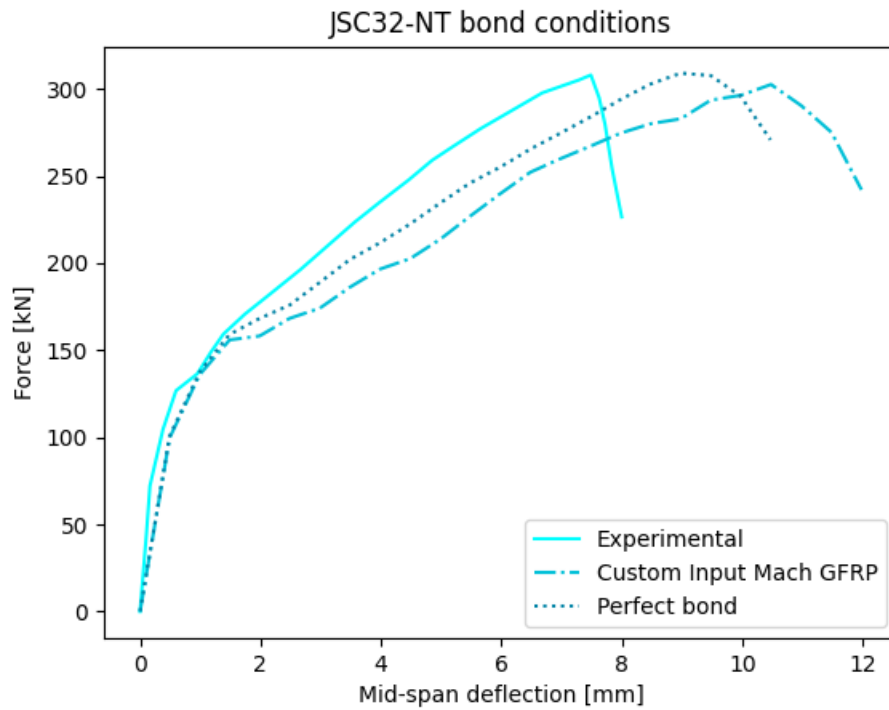


Figure 5.19: JSC32-NT beam : influence of numerical bond-slip model

The analysis models and input conditions combined and developed for analyzing GFRP beams subjected to 3- or 4-points bending yield satisfactory results. The general appearance of the stress-strain graphs is similar between the experimental and numerical results. The type of failure that occurs in the beams is also comparable. The timing of failure is in the same order of magnitude between the results obtained with Vector 2 and the experiments described in the different papers. However, some differences can be noticed in the failure values, which can be explained by several factors:

- The conditions at the end of the GFRP rebars in the concrete beam may be studied with more accuracy. As explained before and illustrated in Figure 5.4 [81], these conditions can have a real impact on the results. This difference impacted the JSC32-NT validation model, in which a free-end was modeled instead of a small anchorage. The future models have to take care about this point to have a good approach to the real end bond conditions.
- The GFRP and concrete characteristics used as inputs for the numerical model were not always precisely described in the papers from which they came. Numerous assumptions had to be made for some parameters, which may have led to inaccuracies. These parameters can be, for example, Young's moduli, maximal strengths, or type of rebar coating.
- Some model analysis could have been developed further to be better adapted to the situation of a GFRP beam. For example, the tension stiffening can be improved to better fit the real behavior. Some research is being conducted by Sheikh et al. [71] and the University of Toronto to develop this point. Other variations of concrete models analysis can also be studied to improve the predictions.

5.4 GFRP-RC Beam Simulation

As explained in the introduction, the initial aim of this Master's Thesis was to experimentally validate two beams: one entirely in GFRP and a second one combining GFRP and SMA. To do this, two beams were designed and numerically simulated in Vector 2. Those beams were inspired by the previous master's thesis of De Kerautom [46]. To facilitate comparison, the general dimensions of the beams as well as the testing protocol have been kept similar. The choice of testing beams, as opposed to walls or other RC structures, is linked to the availability of the test setup and molds in the lab. This decision also allows us to reduce costs in case of premature failure, primarily of the SMA rebar, which is quite expensive.

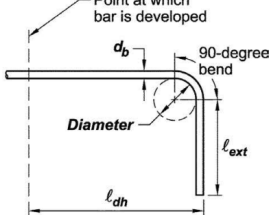
Unfortunately, due to a series of delays in the design, fabrication, and delivery process of the rebars, these tests could not be completed before the conclusion of this thesis.

5.4.1 Beam design

The design of the first full GFRP control beam SG12 (Fig 5.22) is almost identical to the steel control one of De Kerautom, for comparison purposes. In order to highlight the mechanical properties of the GFRP compared to traditional steel rebar, the diameter of the bar are kept the same, even though it is way above the minimal longitudinal reinforcement value recommended by the norms. However, due to the nature of the reinforcement, some adaptation still have to be made.

The first one is the height of the beam. The general dimensions of 2000x150 mm are kept but the height was increased from 190 to 200 mm to allow for enough anchorage length of the longitudinal rebar in concrete, according to ACI CODE-440.11-22 [3] (See Fig5.20). This difference in anchorage length to traditional RC beams is mainly due to the lower bond-strength between GFRP and concrete that between steel and concrete.

Table 25.3.1—Standard hook geometry for development of GFRP bars in tension

Type of standard hook	GFRP bar size	Minimum inside bend diameter, in.	Straight extension* ℓ_{ext} in.	Type of standard hook
90-degree hook	No. 2 through No. 8	Refer to ASTM D7957	$12d_b$	

*A standard hook for bars in tension includes the inside bend diameter specified by ASTM D7957 and straight extension length defined in Table 25.3.1. It shall be permitted to use a longer straight extension at the end of a hook. A longer extension shall not be considered to increase the anchorage capacity of the hook.

Figure 5.20: ACI CODE-440.11-22 Design recommendation for GFRP longitudinal reinforcement anchorage [3]

The second one is the stirrup's spacing. As discussed earlier, the mechanical properties of GFRP, mainly its lower modulus elasticity and brittle behavior, make it much less appropriate for shear reinforcement, reasons why their number have to be increased. The detailed calculations can be found in Annex D.

Another limitation that has to be overcome is the geometrical limitation of the rebars bending, depending of the manufacturer. These limitations evolve quickly as the manufacturers develop new

bending and production techniques. In the case of Sireg, they had to create new mandrels for the GFRP rebar in order to reach a bending radius of 3 time the bar diameter, which helped limiting the height and the width of the beam. The other limitation of Sireg's bending process is that they could not bend the bars more than twice, meaning only straight, L-shaped or U-shaped rebars. For these reason, the 2 U-shaped configuration has been chosen for the GFRP stirrups, as proposed in ACI CODE-440.11-22. (See Fig 5.21)

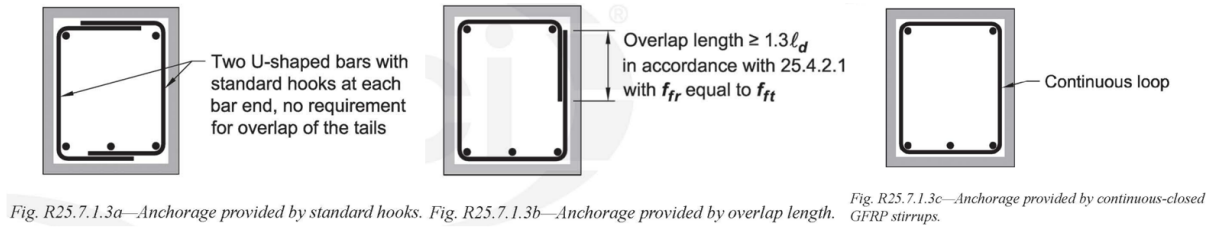


Figure 5.21: ACI CODE-440.11-22 Classical GFRP stirrups configuration recommendations [3]

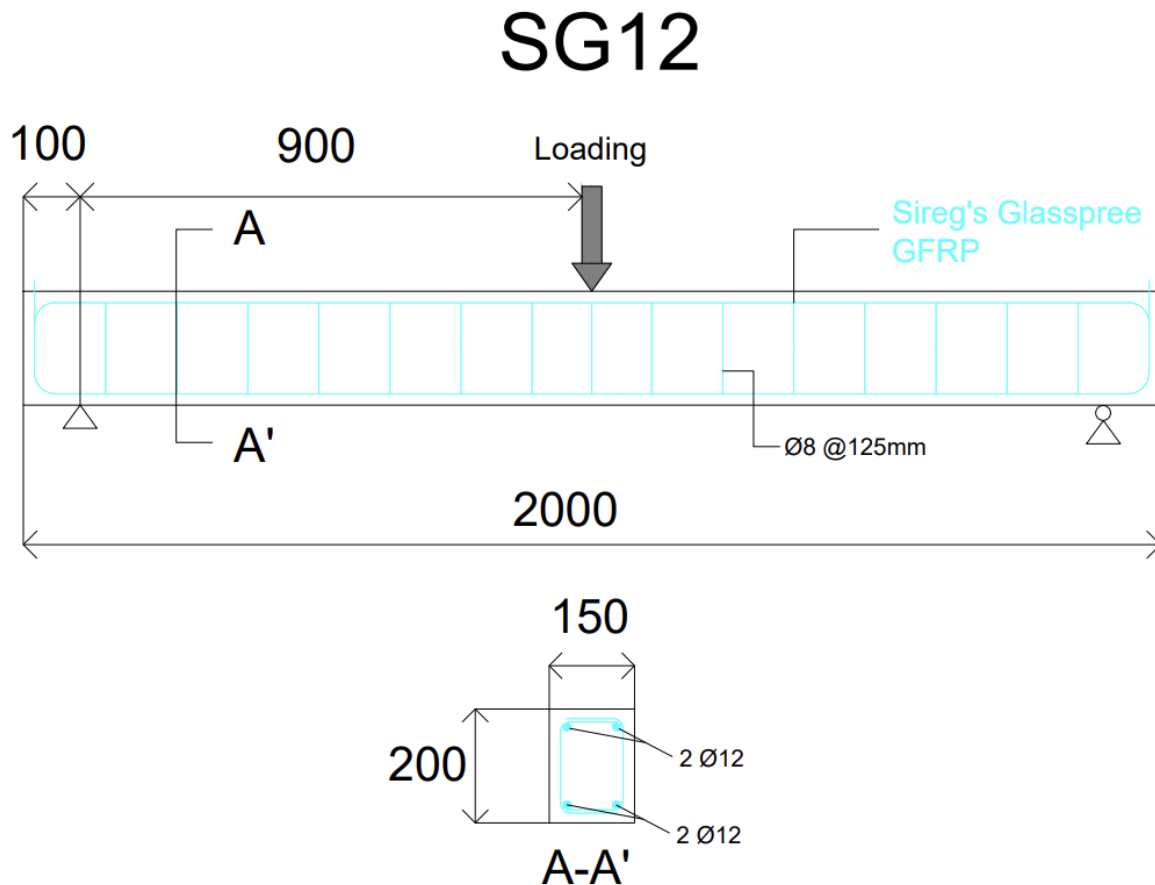


Figure 5.22: Configuration of the SG12 Beam

5.4.2 Numerical Prediction

The analysis of the full SG12 beam was conducted using optimal input parameters studied in section 5.2 and 5.3.2 on the beam geometry introduced in section 5.4.1. This is a plausible assumption, given the beam configuration is similar to that of the validation beams studied earlier, both in terms of dimensions and of three-point bending testing protocol. The numerical setup adopted a 10x10mm

mesh while utilizing the analysis models presented in Figure 5.13. The test setup is a 3-point bending test with a loading point at the middle of the beam length. The bond conditions, modeled with link elements, were based on 'machining' type GFRP rebars (Fig 5.9), from tests conducted by Christophe Dandois [23]. This rebar coating is the closest to the Sireg's Glasspreere rebars [73].

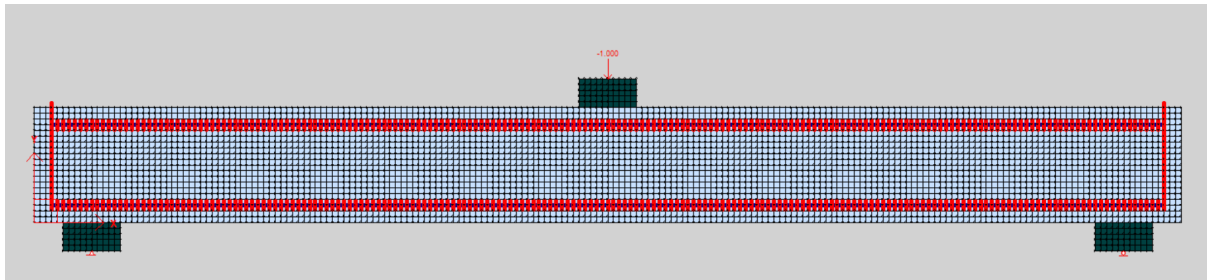


Figure 5.23: Vector 2 model for SG12 Beam

According to the finite element model results on Figure 5.24, this SG12 beam has a maximum capacity of 29.9 kN, achievable at a 17.5 midspan deflection. Concrete drives the failure mode. The distribution of cracks along the beam length can be compared to the ones from the N1 experimental test of Baudouin de Kerautem Master Thesis [46]. The N1 beam's crack configuration (Fig. 5.26) is comparable to that of the SG12 specimen in Vector 2 (Fig 5.25). There are two main cracks on either side of the load support, with smaller cracks along the beam.

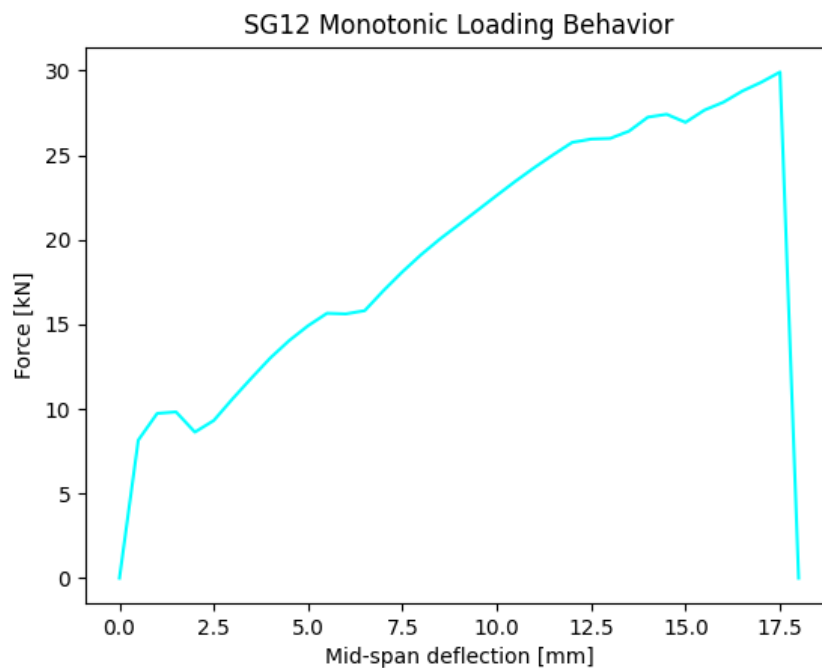


Figure 5.24: Force-displacement behaviour of the SN12

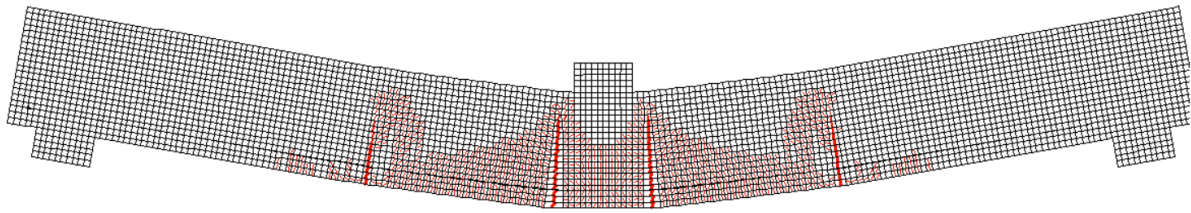


Figure 5.25: Numerical Behaviour Prediction for SG12 Beam



Figure 5.26: Cracks repartition on N1 steel beam[46]

This numerical model was produced by modelling a curved section at the end of the lower horizontal reinforcement as a boundary condition to respect the real geometry. As mentioned by Vint et al. [81] in his paper on boundary conditions models, such a curved reinforcement must be modelled as a section in which the reinforcement resistance f_u is equal to 0.5 times the initial reinforcement resistance. Since the reinforcement has been modelled as a single-section rebar, a simple check is made to ensure that the stress in the bend is not greater than half its maximum capacity, which is verified in this case as can be seen in Figure 5.27.

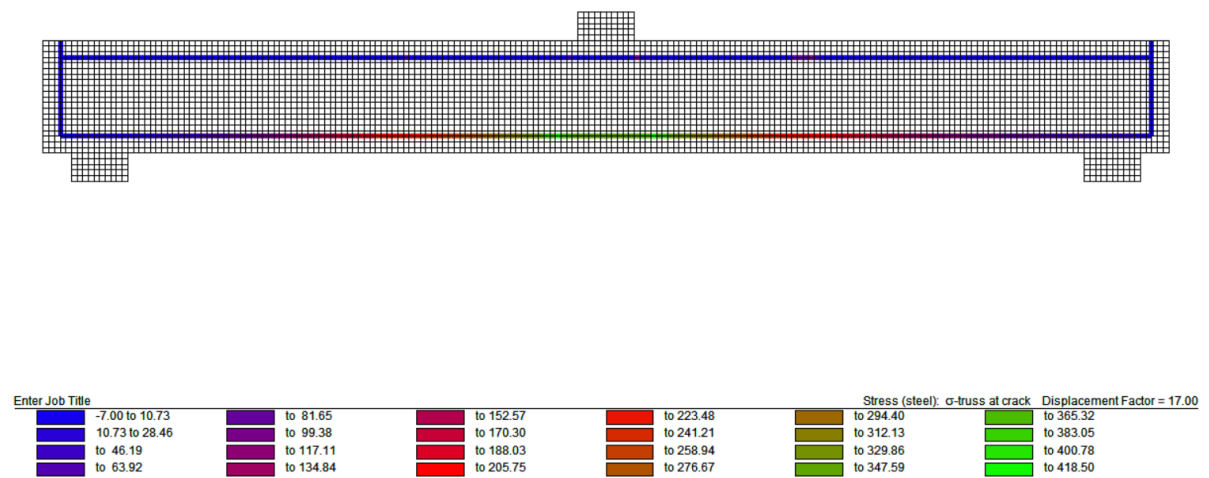


Figure 5.27: Stresses in the reinforcements of the SG12 GFRP beam

5.5 GFRP-SMA-RC Beam Simulation

5.5.1 Beam design

The design of the second beam, the SG12N, is very similar to SG12, except for the 100mm SMA reinforcement at mid-span of the bottom longitudinal reinforcement in tension, where the plastic hinge is expected to happen. This is again the same disposition and free-length than De Kerautom [46], in order to characterize the different impact of SMA in a steel or in a GFRP hybrid configuration.

This hybrid configuration implies the use of mechanical coupler as detailed in Section 4. Unfortunately, those couplers prevent the use of stirrup in their section, explaining the different spacing of the stirrups in the center of the beam. (See Fig 5.28)

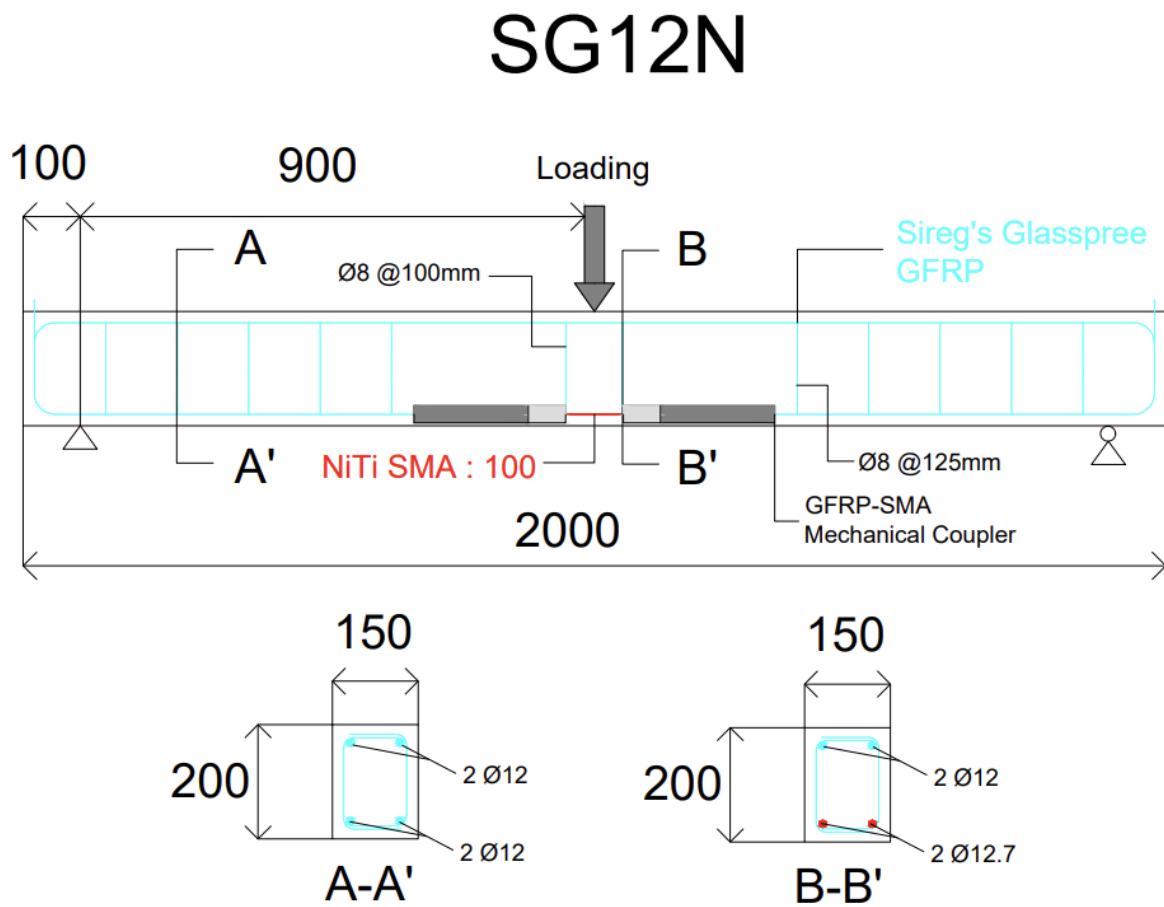


Figure 5.28: Configuration of the SG12N Beam

5.5.2 Numerical Prediction

SMA Material Properties

Vector 2 software provides several SMA models. Type 1 SMA is modeled with a perfect theoretical shape, without offsets, and displays a flag-shape hysteresis. Type 2 SMA has a more suitable stress-strain response for what has to be modeled. It incorporates strain-hardening and small strain offsets, giving it a more realistic behavior. To represent the smooth coating of the SMA reinforcement, an 'embedded smooth rebars' bond model was used along the SMA's length in the beam, with link elements in Vector 2 (cfr Section 5.2)[83].

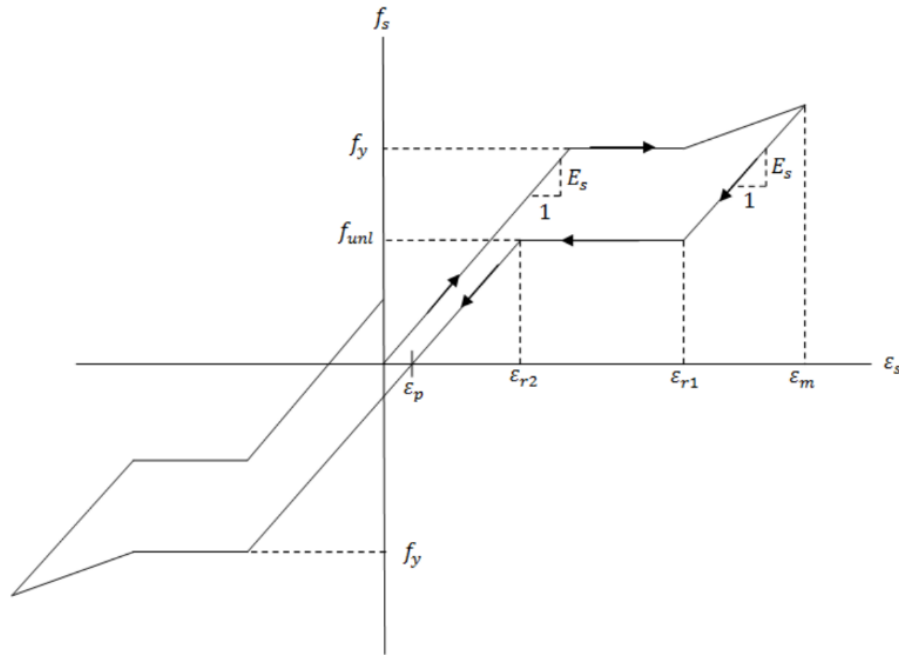


Figure 5.29: Stress-strain response of Type 2 SMA in Vector 2[83]

Coupler model

The mechanical coupler between the GFRP cage and the SMA central part is represented in the numerical model. This was achieved by using a high-strength steel section with high stiffness, in a full section of the same diameter as the coupler tube, i.e., 32mm. Abdulridha et al. [2] recommend considering the bond condition along the length to be perfectly bonded..

Beam SG12N

The SG12N beam underwent analysis using the same models as the SG12 beam. The mesh used was also similar, involving link elements with custom bond conditions on the GFRP reinforcement and the parameters previously mentioned for the couplers and SMA reinforcement. To generate realistic results from the model, the intersections between the various types of reinforcement are assumed to be perfectly bonded without link elements, as depicted in figure 5.31.

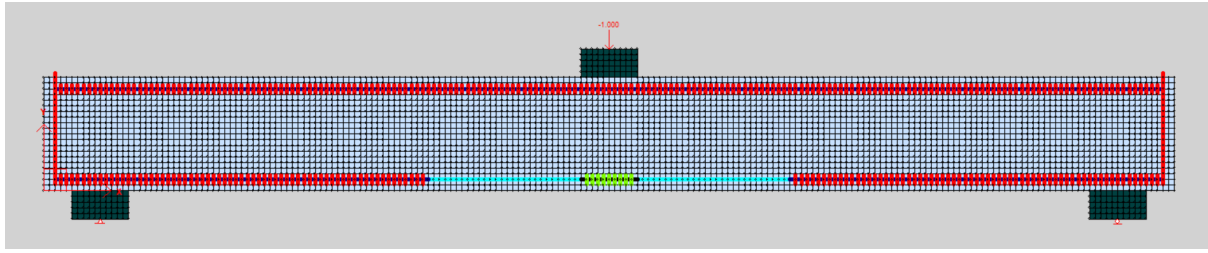


Figure 5.30: Vector 2 model for SG12N Beam

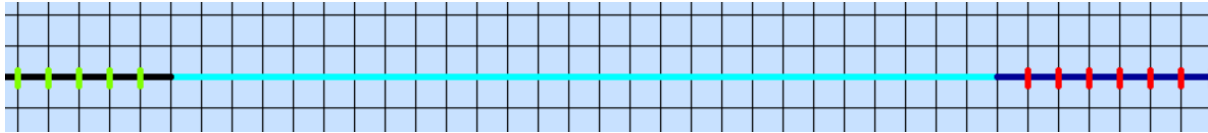


Figure 5.31: Zoom on the bond conditions at the rebars connexions (SMA-Steel-GFRP)

The force-displacement relationship of the beam is displayed in Figure 5.32. The maximum force is observed to be 35.7 kN, occurring at a displacement of 12.75 mm. The graph shows a 'plateau' before failure, indicating effective operation of the SMA. The distribution of cracks and the failure mode resemble those observed in the SG12 beam. The maximum stress in the hooked section is also within the acceptable range.

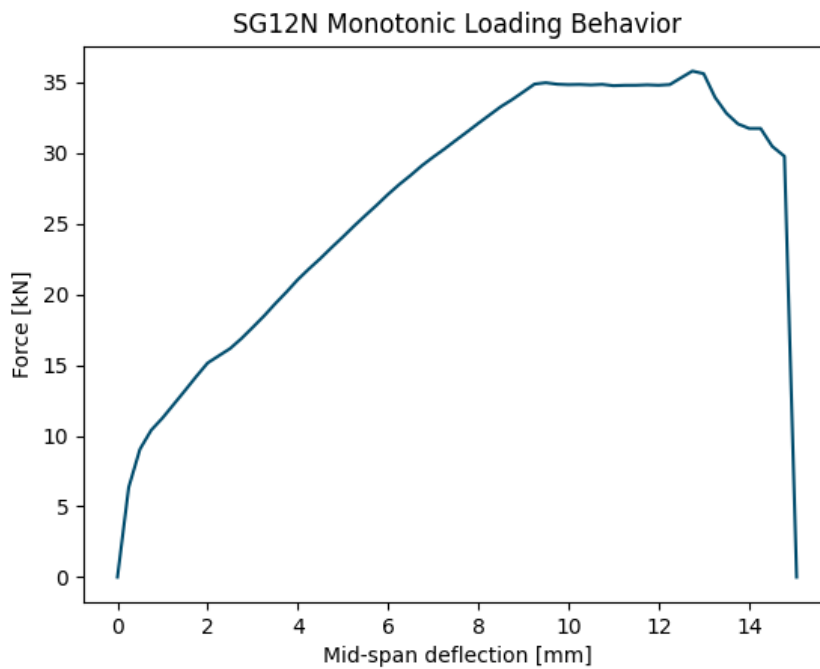


Figure 5.32: Force-displacement behaviour of the SN12N

The final point to validate is the maximum force in the mechanical coupler. As established in Chapter 4, the developed couplers can withstand a maximum force of 67 kN, equivalent to a maximum

strain in the SMA of 529 MPa. This constraint is satisfied, as the maximum stress in the SMA at beam failure is 336 MPa. (cfr Fig 5.33).

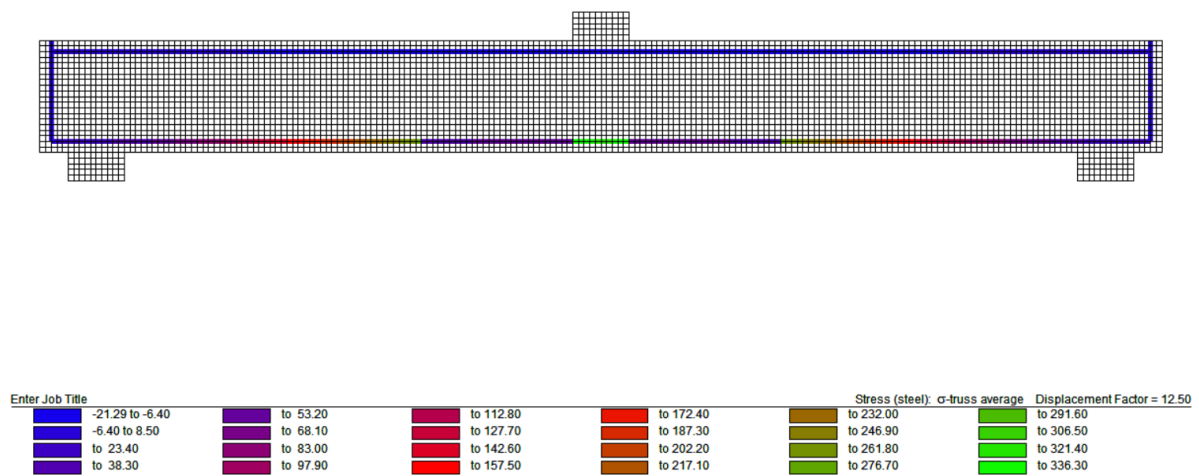


Figure 5.33: Stresses in the reinforcements of the SG12N beam

Additional analysis using the SG12N numerical model with cyclic loading can be conducted. This simple loading protocol involving 4 cycles incremented by 3 mm mid-span displacement each, permits us to visualize the 'memory shape' of the SMA and the minor residual displacements resulting from cyclic loading of the beam. (Fig. 5.34).

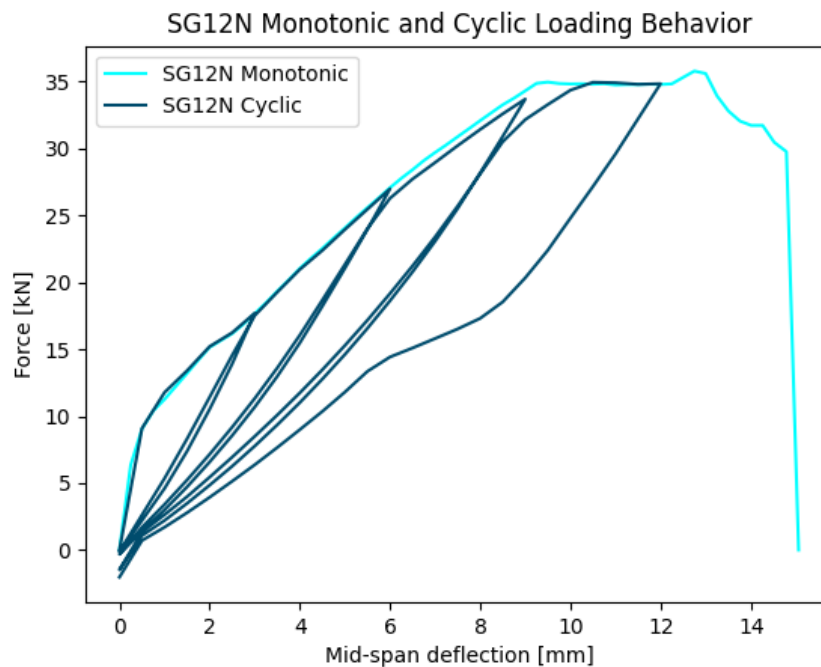


Figure 5.34: Cyclic Loading of the SG12N beam

5.6 Conclusion

To conclude this numerical approach, GFRP beam modeling requires a good model analysis, especially for the bond model of the rebars, which can't be considered as perfectly bonded unlike steel, and for the tension stiffening model, which is an important limiting factor for GFRP reinforced beams. Moreover, good initial hypotheses are crucial to provide a result as reliable as possible.

The numerical prediction of both SG12 and SG12N beams appear to be consistent. The graph presented on figure 5.35 shows the expected behaviour of the two beams. The failure occurs at a higher level in the GFRP-SMA beam than in the full GFRP specimen. This is attributable to the extremely stiff steel couplers that are perfectly bonded, enhancing the overall performance of the beam by minimizing cracking in the concrete. As expected, the main difference is in the shape of the force-displacement response. The first beam have a linear response due to the GFRP properties, where the second one present a pseudo-plastic plateau, attributable to the SMA, which is the objective of the beam conception.

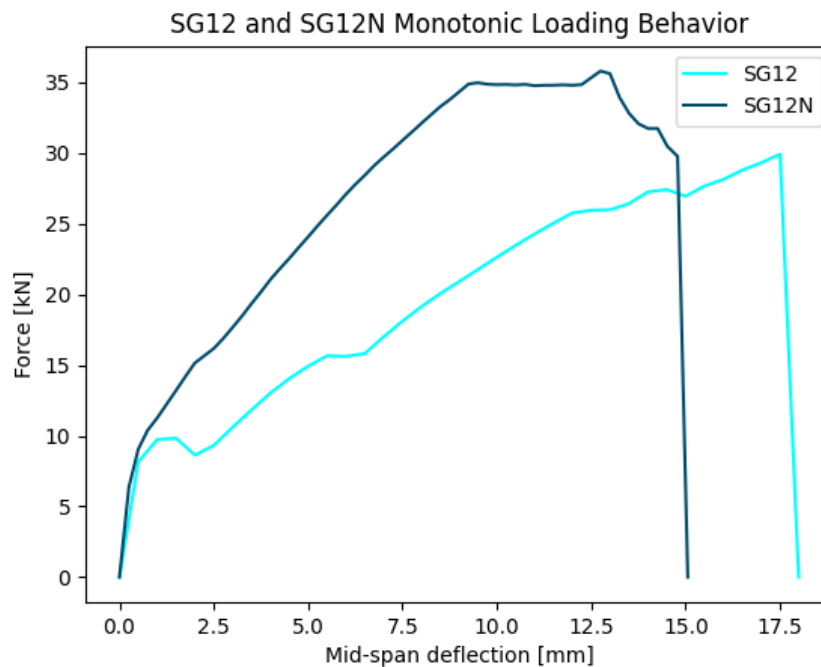


Figure 5.35: Force-displacement behaviour of the SG12 and SN12N

The two numerical beams can be compared to full steel one from B. De Kerautem [46] which has the same geometry. The figure 5.36 makes this comparison. Some important differences are observed between the two beams with GFRP and the full-steel one.

- The ultimate displacement at failure of the beam is higher for the full-steel beam. This behaviour was predictable due to the low Young Modulus of GFRP and SMA compared to the steel.
- The maximal force reached by the S1 steel beam is 20 percent higher than the SG12N beam predicted value.
- The GFRP beam including SMA rebar in the plastic hinge region has very low residual deformation during cyclic loading, unlike steel which presents large residual displacements.

SG12 and SG12N prediction compared to S1 experimental results

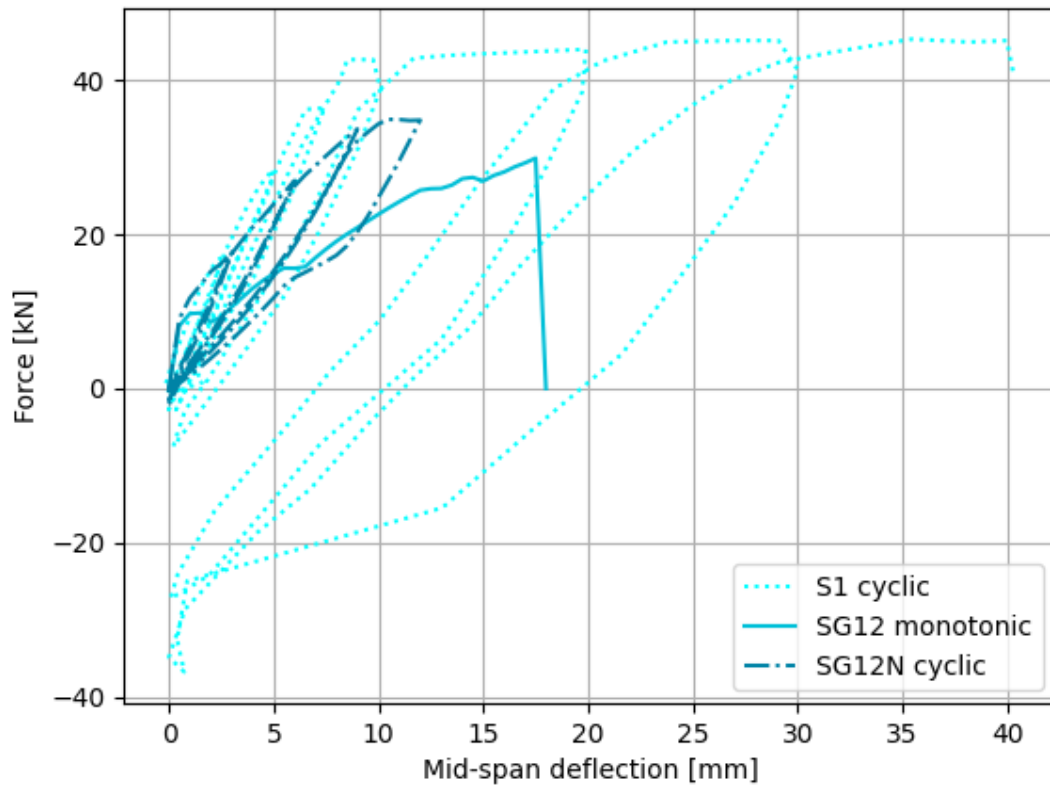


Figure 5.36: Predicted behaviour of the SG12 and SN12N compared to the experimental response of the S1 beam

The conclusion of these results is that the numerical models give hope for the performance of a beam combining GFRP and SMA in the plastic hinge section. In fact, although the ultimate strength predictions are 20 percent lower than with full-steel RC, the force-displacement response during cyclic loading offers a fairly similar performance while offering some ductility with the SMA and avoiding residual displacements. This behaviour is promising for the implementation of this type of solution in seismic applications, as it would allow energy to be dissipated through the ductile properties of the SMA while avoiding residual displacements.

The SG12 and SG12N setups still need to be tested to validate the models predictions. It will be interesting in the future to make a better study about the GFRP reinforced beam geometry to take better advantage of GFRP characteristics and better understand its weaknesses, such as its poor bending behaviour, in order to obtain even better results like a better use of the SMA plateau. In any case, this initial approach made it easy to compare the predictions with the results for a steel beam, and is already providing some answers to the problem of the ductility in GFRP RC structures.

CHAPTER

6

CONCLUSION AND FUTURE DEVELOPMENTS

6.1 Conclusion

Glass Fiber Reinforced Polymer (GFRP) has proven to be an efficient replacement for steel in Reinforced Concrete (RC) structures. Its high tensile strength and low weight-to-strength ratio allow for efficient concrete sections. Its resistance to corrosion makes it very useful and long-term cost-efficient in many applications. Moreover, its non-magnetic properties and electromagnetic transparency make it suitable for various applications where steel could not be used. However, its low modulus of elasticity, brittle failure behavior, and poor resistance to shear prevent it from being as efficient as steel in seismic design. To increase these performances, we've studied its possible combination with Shape Memory Alloy (SMA), which presents the advantages of Superelasticity (SE), where the material can undergo substantial deformation and then return to its original shape, and Shape Memory Effect (SME), where the alloy can return to its original shape upon being heated.

After an extensive literature review, it emerged that many studies on RC structures with GFRP, GFRP and Steel, or Steel and SMA had already been conducted. These studies have proven the abilities of GFRP to achieve similar performances to traditional steel RC structures upon careful capacity design in shear and over-reinforcement in flexure. Furthermore, SMA reinforcement ratios between 0.1%-0.5% could significantly improve the shear strength and ductility of steel RC structures, restore damaged RC structures' load-carrying capacity, reduce crack widths, and provide effective retrofits for pre-cracked RC structures. However, no study had been conducted on the effect of combining GFRP and SMA in beams.

Based on previous mechanical coupler investigations, we've designed and tested coupler configurations for SMA-Steel connections and GFRP-Steel connections in order to finally design and test a configuration for a GFRP-SMA connection. To do so, a first series of tests was carried out on two different GFRP rebars to characterize them and their behavior in tensile tests. These tests also allowed us to develop an installation procedure for Distributed Fiber Optic Sensors (DFOS) along

the different materials and inside the couplers. The conclusions of this experimental campaign are as follows:

- GFRP is sensitive to crushing in the testing machine's jaws and thus requires proper anchorages as prescribed by the standard. Therefore, it is not recommended to use undersized anchors.
- As expected, both GFRP bars, IRT-M2P and Sireg's Glasspree, exhibited perfectly linear elastic behavior. They had similar elasticity modulus of around 54 GPa and tensile strength of over 900 MPa.
- The Steel-SMA coupler proved sufficient to develop significant strain in the SMA before a brittle failure of the SMA due to the punching of the first row of screws. However, the extensive manufacturing procedure required makes it less appropriate than other simpler solutions with similar results. Therefore, this configuration is not recommended.
- Analysis of the behavior of GFRP in its section of the coupler highlighted the predominant limitation of its coating shear resistance in a configuration using contact binding like epoxy. This bond stress peak, $\sigma_{bond,peak}$ has been characterized for the IRT-M2P bar and has proven to be a key factor in the characterization of the strain distribution inside the GFRP section of the coupler. Ribbed surfaces of the GFRP bar, like the one of the Sireg's Glasspree bar, proved to have higher value of $\sigma_{bond,peak}$ and thus require less surface of contact to efficiently connect GFRP. It should thus be preferred for structures requiring the use of mechanical couplers as it reduces the couplers' length and thus reduces the length of the area where no shear reinforcement can be connected to the bars.
- The GFRP-Steel coupler has proven to be very efficient for any application. The steel plasticizes and breaks before any failure in the coupler if properly designed, and the length of the steel tube can be adapted to any load and GFRP bar with respect to $\sigma_{bond,peak}$.
- The SMA-GFRP coupler proved sufficient in monotonic loading as it allowed the SMA to reach its strain "plateau" and undergo significant deformation before a similar failure as the Steel-SMA configuration. However, the performance under cyclic loading proved to be lower and the SMA couldn't develop its superelasticity properties. The values reached are still good enough for some applications but performances of the SMA part of the coupler could be improved with other configurations and research should continue on this subject.
- An installation procedure of DFOS on different GFRP bars was developed and is described in Annex A. For non-smooth rebars, a grooving method also had to be developed and is also described in Annex A.

In order to validate this GFRP-SMA connection, two beams were designed. The first one, SG12, is purely reinforced with GFRP and serves as a control whereas the second, SG12N, incorporates SMA at mid-span of the longitudinal reinforcement in tension to characterize its impact.

A numerical model using Vector2 has also been developed and trained on other experimental tests on RC structure with GFRP to predict the performances of the beam SG12 and was interpolated to include the SMA and SMA-GFRP couplers to model the SG12N beam. The conclusions of this modelling are as follows:

- The model parameters for a GFRP reinforced beam need to be much different from those for a traditional steel RC beam. The main differences are located in the tension stiffening model and in the bond-conditions of the GFRP rebars. A customized bond-stress input needs to be used, based on pull-out tests, to approximate the best numerical behavior as possible.

- Using the most adapted models, bond-conditions, and setup configurations, some interesting behavior predictions for full GFRP beams can be obtained, matching the training dataset of experimental tests on GFRP RC beams.
- The GFRP-SMA beam force-displacements prediction shows us an expected curve divided into different parts: a first deformation driven by the elastic response from the GFRP and the elastic part of the hysteresis curve of the SMA, followed by a typical 'plateau' due to the SMA superelasticity properties. The cyclic loading confirms this explanation of a plateau driven by the SMA, with very low residual deformation despite the pseudo-plastic plateau specific to superelasticity property.

However, due to a series of delays, these beams were not constructed before the conclusion of this thesis and the numerical predictions could therefore not been compared to experimental results.

6.2 Future developments

Several lines of development can be explored to complement the research conducted during this master's thesis. First, the conclusions of this Master Thesis could serve as a foundation for future research and experimental theses at LEMSC, starting with the testing of the beams that were designed and simulated in this work. This would validate the behavior of the coupler configuration proposed in this thesis in a real scenario and test the accuracy of the numerical model predictions. Additionally, other tests could be considered, such as a high wall with a GFRP-SMA combination, utilizing the mechanical couplers developed for this purpose. One interesting point to develop before these tests would be to establish the exact minimum length required for the GFRP part of the coupler in the case of a Sireg's Glasspreet bar. Indeed, during the experiments were only tested enough couplers with IRT-MTP GFRP bars to assert that the minimum coupler length had been reached with those rebars. This was not the case with the Sireg's Glasspreet bars. Therefore, it would be advisable to conduct pull-outs of these reinforcements in epoxy anchors to accurately characterize the bond-stress relationship, particularly the peak values.

Second, concerning the mechanical couplers, several further developments could be envisaged. If the current configuration is still used, the first point of development would be, as mentioned above, to conduct additional tests on the bond-stress behavior between the epoxy and the GFRP reinforcement to determine the minimum coupler length specific to the characteristics of the used rebars. This was briefly studied during this master's thesis with smooth-coated bars, but as concluded, the coating has a strong influence on the results, hence better characterization is necessary. The second point pertains to the number of samples tested, which should be greater to obtain more representative results. Third, a cyclic test at a higher frequency should be carried out to study the coupler's response to this type of loading.

More broadly, the proposed coupler configuration may quickly become limiting in terms of the load capacities it can support. Other types of mechanical coupling systems could be considered in future research. Furthermore, new technologies are emerging on the market, such as the development of GFRP-SMA composite materials, as highlighted in the literature review. Such materials could eliminate the need for complex connections with mechanical couplers, and their development seems promising.

Lastly, regarding the numerical model, the experimental tests on the two beams will allow us to confirm the validity of the numerical model for the GFRP and to validate the model combining SMA and GFRP. Some differences might emerge between numerical predictions and reality, potentially stemming from three particular points that could be further improved. The first point involves the stiffening model, which is not yet optimal. The development of a model specific to GFRP seems necessary to achieve an even better characterization of GFRP structures, due to their low tension stiffness. The second point that could be improved pertains to the bond-condition parameters. The bond-stress curves on which the numerical model is based were produced with samples made with a single pull-out configuration and a single anchor length. A more representative sample set would be beneficial to refine the model. Third, modeling the coupler in the finite element model requires additional work. Currently modeled as a highly rigid solid steel section, it would be interesting to compare this approach with another type of model. This model can be even more precise as the behavior of the reinforcement and the slip that occurs inside it is now understood from the measurements taken by the DFOS. Such an analysis seems complex to achieve using Vector 2 due to the two-dimensional aspect of the analysis, but other finite element models might be better suited to a study on this subject.

CHAPTER

7

APPENDICES

APPENDIX

A

DFOS INSTALLATION ON GFRP

This section aims to describe the procedure of the optical fiber installation (DFOS) along a GFRP rebars to test it. Here is the list of the material needed:

Material	References
Rebars	GFRP Diam 13 IRT-M2P ref : 221016
Fibre optique polyimide	SMF-12-P-125-1 ; 125/155um Polyimide
High-Definition Fiber Optic Strain Sensors	HD6S
Optical Distributed Sensor Interrogator	ODISI 6000 SERIES[49]
Funnel	https://www.action.com/fr-be/p/lot-dentonnoirs-/
Heat-shrinkable polymer	RS Heat Shrink Tubing Black 1.8mm, 2:1 ratio
Compressed air duster	HG nettoyant à air comprimé
Degreaser	CSM-3 Micro-Measurements accessories
Cyanoacrylate	Loctite 401
Cyanoacrylate Adhesive Activator	Loctite SF 7457
Air-Drying Polyurethane Coating	M-Coat A Micro-Measurements accessories
Brushes	All kind
Paper towels	All kind
Adhesive tape	All kind
Flat-headed wooden stick	All kind
Angle grinder disc	Würth, Carbided-Tipped cutting Disk

Table A.1: Material required for the DFOS installation on a GFRP rebar

A.1 Rebar preparation

Before beginning the installation of the optical fiber on the GFRP, the first step in the process is to cut the rebar to the desired size for testing. Several angle grinder discs are studied to obtain the desired cut but the best results were obtained with a carbided-tipped cutting disk. The cutting is

carried out in a ventilated environment and with the recommended safety equipment (gloves, glasses and dust mask).

Once cut, the rebar is cleaned in order to receive the optical fiber. To do this, it is placed on a support in its centre to avoid contact with the table. The cleaning treatment is carried out using a degreasing agent applied firstly on a paper towel to rub the bar. In a second time, the degreasing spray is putted on a brush and brushed along the bar. Once these two steps done, the degreaser is dried with a compressed air duster or just with time.(FIG A.1)



Figure A.1: GFRP rebar preparation

A.2 Optical fiber installation:

Now that the GFRP is completely clean, the next step is to glue the optical fiber along it. For this test, we decided to run the fiber back and forth on both sides along the armature.

Cyanoacrylate will be used to glue the fiber but also to protect it. Great caution should be taken as it is very fragile once there is an angle in it. Here are the different steps:

- Unroll the HD6S fiber for the length needed.
- Protect with heat-shrinkable polymer the beginning of the fiber which will not be glued between the interrogator and the GFRP. (FIG A.2)
- Start with the first side of the rebar. A first cyanoacrylate dot is made at one end and immediately followed by a shot of spray of the activator. (FIG A.4) Thanks to that it will instantly polymerized. The fiber is held taut to the other end. A second cyanoacrylate dot is applied, with the activator, to hold the fiber in this position of tension. An additional adhesive tape can be used to support the glue point during work.
- Apply the cyanoacrylate along the entire length with a flat-headed wooden stick. Wait polymerisation of the cyanoacrylate. (FIG A.3)

- Connect the Optical Distributed Sensor Interrogator to the beginning of the fiber inside the funnel and tare the software
- Check that the fiber is properly glued and apply a second layer of cyanoacrylate if necessary. Be careful there is a termination at the end of the fiber that needs to be immaculate. Wait polymerisation of the cyanoacrylate.
- Make a 180 degree loop with the fiber to get it to the other side of the GFRP. Be careful to respect the minimum bending radius prescribed for the fiber. Apply only 2 more local glue dots and activator at 90 and 180° to make this loop. Protect the fiber at this point only with an Air-Drying Polyurethane Coating. (FIG A.5)
- Repeat the same actions as for the first side : apply a dot of glue and activator to tighten the fiber on the second side of the GFRP. Then glue twice the whole length with a flat-headed wooden stick. Wait polymerisation of the cyanoacrylate.
- Put the unused length of fiber on a spool.



Figure A.2: Heat-shrinkable polymer



Figure A.3: Cyanoacrylate application



Figure A.4: Cyanoacrylate dot with activator



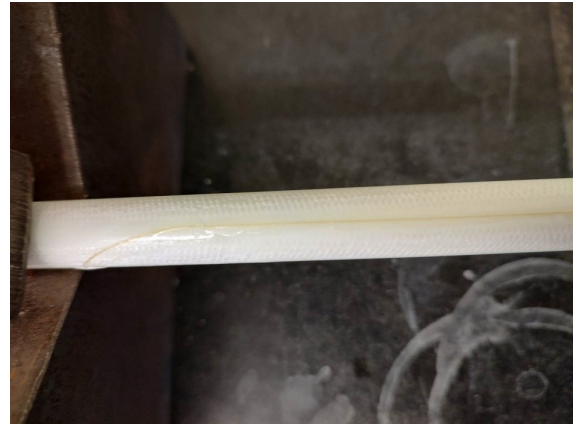
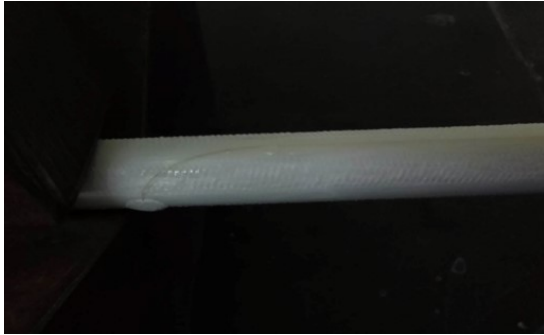


Figure A.5: 180° loop at the end of the rebars

A.3 Alternative for non-smooth bars

Some GFRP rebars coating are not smooth and will thus require a specific preparation, involving digging an incision along the bar. Once incised, the procedure of installation of the DFOS is the same, with the fiber and the cyanoacrylate applied in the crack.

The best way of digging that incision is to create a setup with an angle grinder fixed to a plank and adjustable with different screws to be able to cut precisely at some depth on a GFRP bar glued to a moving rail guided by static rails. The setup is detailed in Figure A.6.

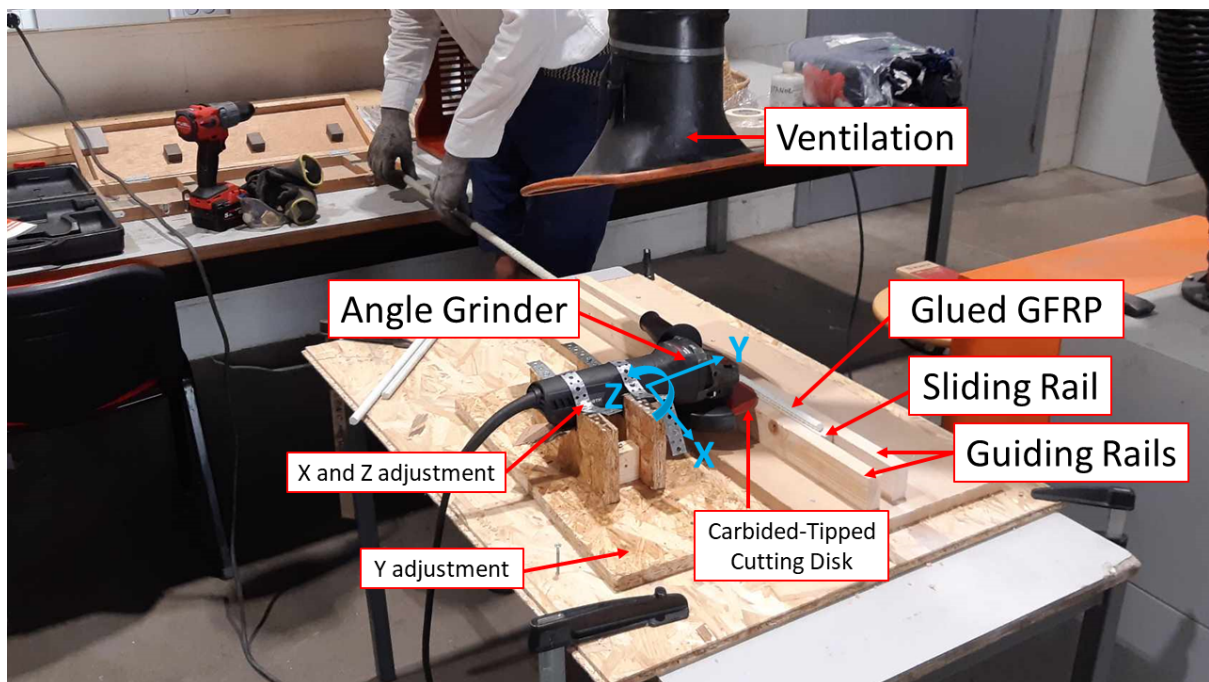


Figure A.6: Setup for the grooving of a GFRP rebar

APPENDIX

B

GFRP-SMA MECHANICAL COUPLER FABRICATION

This section describes the fabrication method of the GFRP-SMA coupler as used in this thesis. The material required for 1 coupler is as follow :

Material	References
Steel tube	Enough for your dimensions
Steel mechanical coupler	PLAKA – MANCHONS MBT - BRMBT12
Injection resin	HILTI Résine HIT-RE 500 V3

Table B.1: Material required for the GFRP-SMA mechanical coupler (machines not included)

B.1 GFRP steel tube

Depending on the length of the steel tube that you need and the radius of your GFRP rebar, the dimensions of this section may change a lot. Here's how to create them yourself if you cannot find ready-made tube with the desired dimension in the market :

- Cut your tube to the desired length. A rotary saw is preferable as it will cut cleaner and flatter faces. (FIG B.1)
- Drill the hole in a metal lathe. You should use different drill bit, with at least one smaller diameter before your goal diameter. Pay attention that depending on the length of your tube you might need longer drill bit than those usually available. (FIG B.2)
- Drill the injections holes, at least one every 7-10 cm. Be carefull to drill perpendiculary to the direction of the tube (toward it's center) to allow a proper distribution of the injection resin later on.
- Sandblast the inside of the tube to allow for good adhesion of the injection resin.



Figure B.1: Cutting of the steel tube



Figure B.2: Drilling with the metal lathe



Figure B.3: Steel tube (without the injection holes)

B.2 SMA Coupler

The SMA part of the GFRP-SMA mechanical coupler is the most critical and needs to be as precise and aligned as possible. To do so, we created a paper guide for the added holes on the sleeve, at 60° from the others :

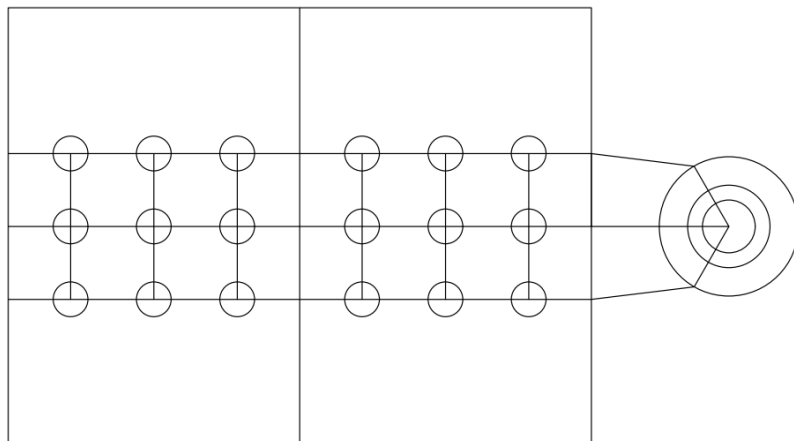


Figure B.4: Guide for drilling the additional holes

The procedure is then as follow :

- Enroll the paper around the sleeve and align it by inserting the screws in the holes already present.

- Pin the center of the additional holes using the paper guide. (FIG B.5)
- Drill the additional holes. The alignment with the center of the sleeve is of the utmost importance to avoid parasite stress in the SMA due to unaligned screws. (FIG B.6)
- Tap the additional holes. (FIG B.7)
- Cut the sleeve in two using the rotary saw. To do so, insert a steel rebar and tighten it using some of the screws. It is recommended to cut the sleeve with a very slow speed to avoid braking the saw on the rack and pinion. (FIG B.8)



Figure B.5: Pinning the center of the holes



Figure B.6: Drilling the holes



Figure B.7: Tapping the holes

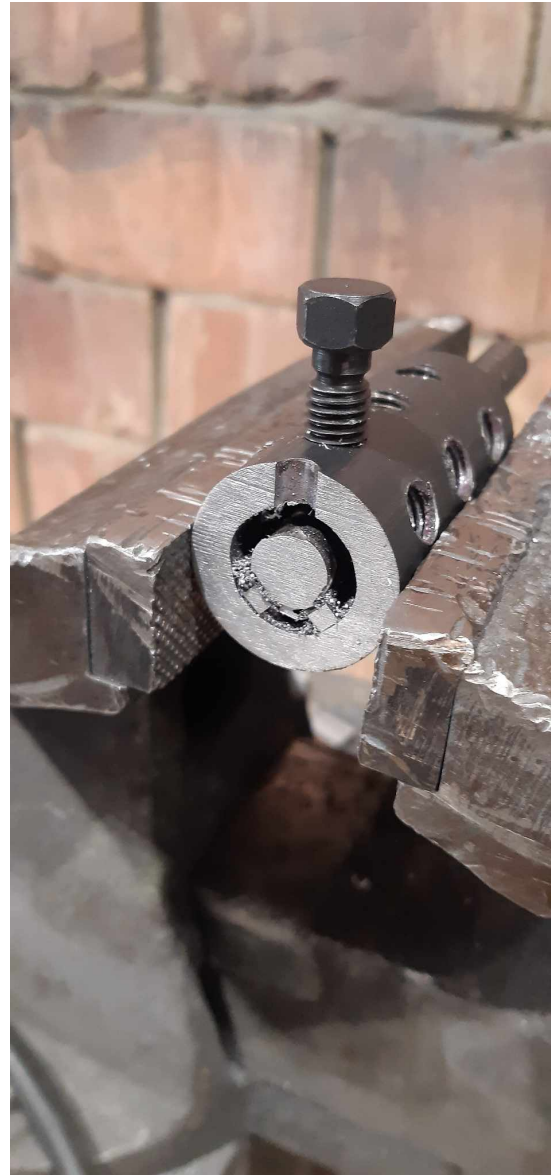


Figure B.8: Cutting the sleeve in two

B.3 Assembling

To ensure the best alignment possible, the procedure to weld the two parts of the coupler is as following :

- Rectify the faces of the two part in the metal lathe to have two perfectly plane faces for the junction.
- Use a standing sander to dig a 45° trench around these two faces to allow for a deep welding (better welding resistance and gain of space) (FIG B.9)
- Create a guide using a threaded rod and bolts to tighten the two parts. The SMA part can be centered on the rod using its screws whereas the GFRP part should be centered using 3D-printed rings to the required dimension. (FIG B.10)

- Once the two parts are centered, weld them together.



Figure B.9: 45° trench for the in-depth welding



Figure B.10: Centering of the GFRP part with 3D-printed rings



Figure B.11: Centered coupler before welding

Once the mechanical coupler created, it's time to add the SMA and the GFRP. To do so, it is important to follow that order :

1. Flatten the heads of the screws. To do so, a standing sander can be used but better results were showed when rectifying them directly on a metal lathe. (FIG B.12)
2. Tighten the SMA in its part of the coupler. Tighten the screws until their head break, one after the other, like when tightening a car's wheel. It is important to do this before injecting the resin to avoid resin between the SMA and the screws.
3. Center the GFRP using the 3D printed rings. (FIG B.13)
4. Maintain the assembly vertical and inject the resin in the GFRP part of the coupler, one injection hole at a times, carefully avoiding air bubbles, and let it dry for a few hours. If the coupler is made to be imbeded in concrete, also fill the SMA part to avoid having unknown actions of the concrete bewteen the SMA and the coupler.



Figure B.12: Flattened screws, provided with the sleeves



Figure B.13: 3D printed rings for the centering of the GFRP



Figure B.14: GFRP-SMA mechanical coupler



Figure B.15: Detail of the GFRP part of the coupler



Figure B.16: Detail of the SMA part of the coupler



Figure B.17: Assembly for a tensile test

APPENDIX

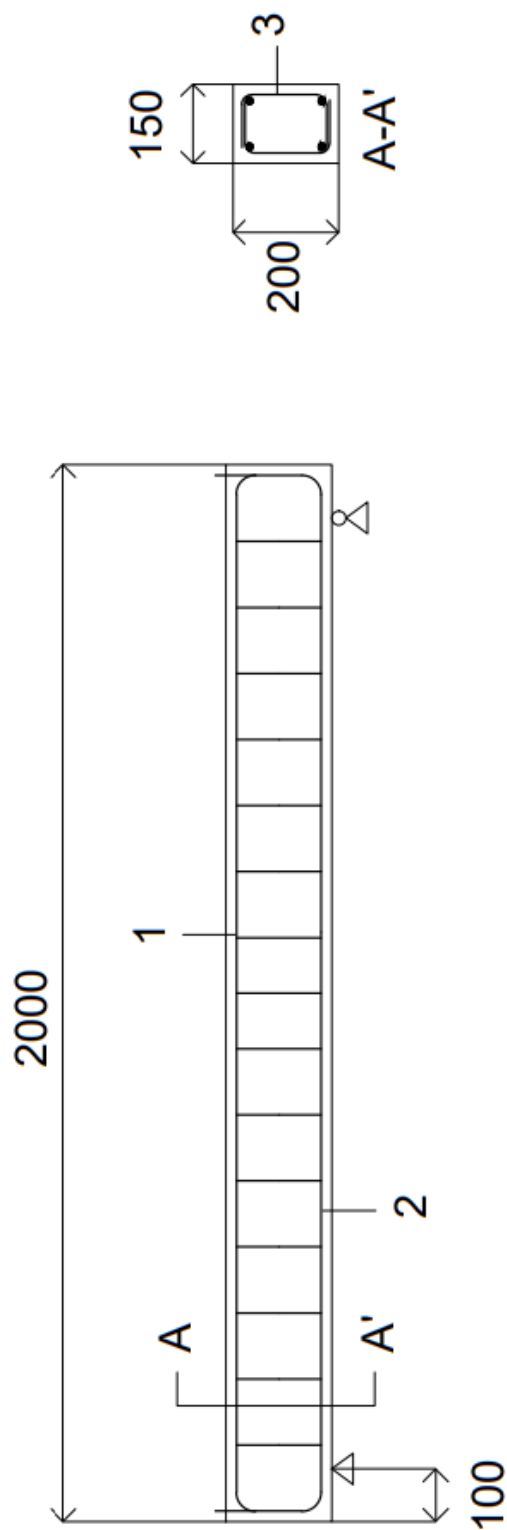
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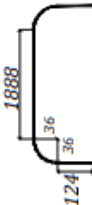
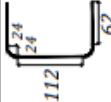
REINFORCEMENT PLANS OF THE BEAMS

This section includes the reinforcement plans and corresponding comments as sent to Sireg for the creation of the 2 beams SG12 and SG12N.

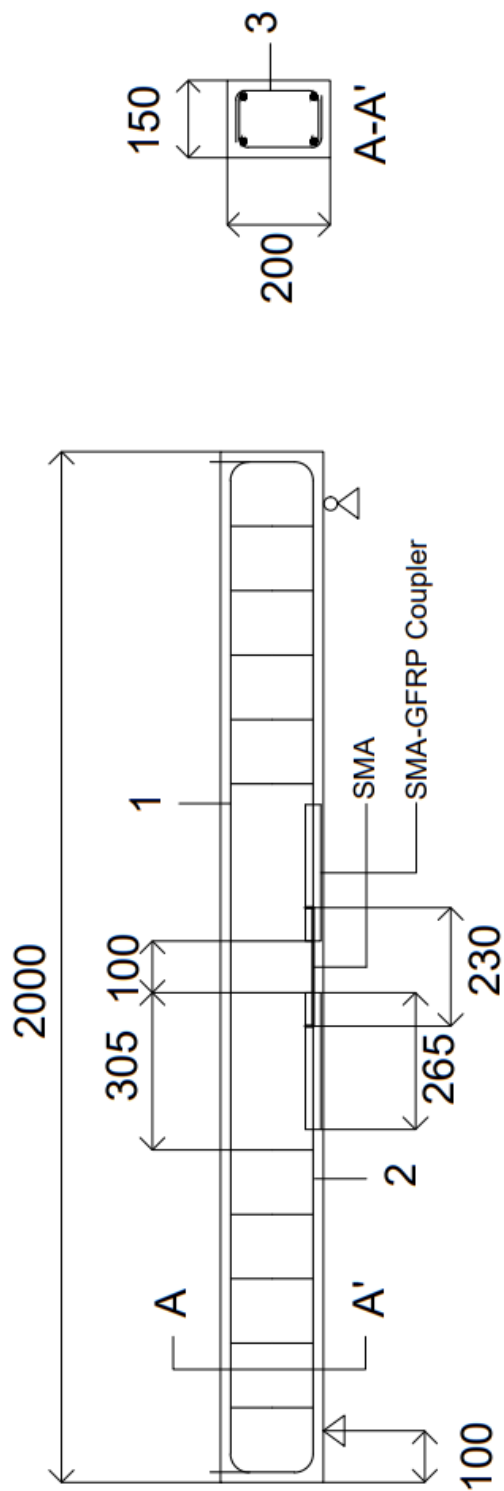
The choice of geometry is discussed in Chapter5 and the detailed calculations can be found in Annex D.

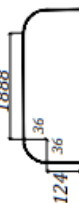
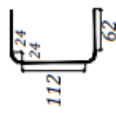
C.1 Plan SG12



Summary table of GFRP rebars - Beam SG12					
N°	Ø (mm)	Scheme (mm)	Quantity	Spacing (mm)	Notes
1	12		2		
2	12		2		Rebar extends outside of concrete for optical fiber measurement
3	8		30	125	2 U-shaped bars bonded together to form a stirrup

C.2 Plan SG12N



Summary table of GFRP rebars - Beam SG12N					
N°	Ø (mm)	Scheme (mm)	Quantity	Spacing (mm)	Notes
1	12		2		
2	12		4		Rebar extends outside of concrete for optical fiber measurement
3	8		28	From the edges : 125 Around the coupler : 305 At the center : 100	2 U-shaped bars bonded together to form a stirrup. The spacing is different at the center because the mechanical coupler can't be connected to stirrups

APPENDIX

D

DETAILED CALCULATION OF THE BEAMS

This annex details the calculations of the beam SG12's geometry and reinforcement. Beam SG12N only differs by the removal of some stirrups where the mechanical coupler prevent their installation. Those calculation are based on ACI CODE-440.11-22 [3], thereafter reffered as "ACI" and AFGC guide to use of composite reinforcement in RC [4], thereafter reffered to as "AFGC".

Height of the beam h_{tot} :

$$h_{tot} = l_{ext} + e + r_{curv} = 200mm \quad (D.1)$$

With :

- d_b : nominal diameter of the longitudinal reinforcement [mm], here 12mm.
- l_{ext} : straight extension of a standart hook geometry for development of GFRP bars in tension (ACI Table 25.3.1): $l_{ext} = 12d_b = 144mm$
- e : Specified concrete cover for concrete not exposed to weather or cast against the ground, for a beam, column, pedestals or tension ties with GFRP reinforcement $e = 1$ to $1/2$ in = 25.4 to 12.7 mm. (ACI Table 20.5.1.3.1) Here we'll take 20 mm.
- r_{curv} : minimal radius of curvature of the GFRP bar : $r_{curv} = 3d_b = 36mm$ (limitation from the manufacturer Sireg).

Minimal flexural reinforcement $A_{f,min}$ (ACI Eq. 9.6.1.2):

$$A_{f,min} = \frac{4.9\sqrt{f'_c}}{f_{fu}} b_w d = 81.1mm^2 \quad (D.2)$$

$$A_{f,min} = \frac{330}{f_{fu}} b_w d = 74mm^2 \quad (D.3)$$

With (ACI Eq. 20.2.2.3) :

$$f_{fu} = C_E f_{fu*} \quad (D.4)$$

$$d = h_{tot} - e - d_s - \frac{d_b}{2} = 166mm \quad (D.5)$$

- f'_c : specified concrete compressive strength [PSI]. Here we'll take the average strength of the concrete as tested by De Kerautom [46] as the concrete formula will be the same here at the LEMSC, which is 37.5 MPa = 5437.5 PSI.
- f_{fu*} : guaranteed concrete strength of GFRP reinforcement [PSI], as indicated by the manufacturer Sireg = 900 MPa = 13500 PSI.
- f_{fu} : design tensile strength of GFRP longitudinal reinforcement [PSI].
- C_E : environmental reduction factor, which shall be 0.85 for concrete both exposed and not exposed to earth or weather.
- b_w : smallest width of the cross section in the tensioned zone [mm], here 150 mm.
- d_s : nominal diameter of the shear reinforcement [mm], here 8 mm.
- d : effective depth which is the distance between the extreme fiber of compressed concrete and the center of gravity of the tensioned longitudinal reinforcements.

We'll take 2 $\Phi 12$ rebars as longitudinal reinforcement :

$$A_f = 2 * 113 = 226mm^2 > A_{f,min} \quad (D.6)$$

Now we have to make sure that we have no rupture in the stirrups :

Maximal spacing of the stirrups s_{max} (AFGC Section 3.6.3.2) :

$$s_{max} = \min(0.75d; 600) = 125mm \quad (D.7)$$

Resistant shear force in the absence of shear reinforcement $V_{Rd,c}$ (AFGC : Eq. 3-35):

$$V_{Rd,c} = C_{Rd,c} \cdot k \cdot \left(100 \cdot \frac{E_f}{E_s} \cdot \frac{A_f}{b_w \cdot d} \cdot f_{ck} \right)^{\frac{1}{3}} \cdot b_w \cdot d = 12.24kN \quad (D.8)$$

With :

$$C_{Rd,c} = 0,18/\gamma_c; \quad (D.9)$$

$$k = \min \left\{ 2, 0; 1 + \sqrt{\frac{200}{d}} \right\} \text{ with } d \text{ in mm}; \quad (D.10)$$

- E_s : reference modulus of elasticity of steel [MPa], taken as 210 GPa.
- f_{ck} : 28-day characteristic compression strength of concrete [MPa], = $f'_c = 37.5MPa$.

Mid-span stresses V_{Ed} :

$$V_{Ed} = 1,35V_g + 1.5V_q = 43.28kN \quad (D.11)$$

With :

$$V_g = +S_c \cdot \rho_c \cdot l/2 \quad (D.12)$$

$$V_q = \frac{Q}{2} \quad (D.13)$$

- V_g : permanent loads.
- V_q : variable loads.
- S_c : Area of the concrete = $b_w h_{tot}$.
- ρ_c : Force density of concrete $235.44kN/m^3$
- l : length of the beam = $2m$.
- Q : maximal load applied at mid-span during the test procedure. Here we'll take the maximal force reached by De Kerautom = $45kN$.

Because $V_{Rd,c} < V_{ED}$, we need to add stirrups to the minimal transversal reinforcement.

Shear strength of stirrups $V_{Rd,f}$ (AFGC Eq. 3.37):

$$V_{Rd,f} = V_{Ed} \quad (D.14)$$

$$\frac{A_{f,v}}{s} = \frac{V_{Rd,f}}{z \cdot f_{f,w,lim} \cdot \cot \theta} = 6,83cm^2/m = 2\Phi 8/146mm \quad (D.15)$$

With :

$$f_{f,w,lim} = \min \{ \alpha_{bend} \cdot f_{f,w,d}; E_{f,w} \cdot \epsilon_{f,w,lim} \} \quad (D.16)$$

$$f_{f,w,d} = \alpha_f \cdot \frac{f_{f,w,k}}{\gamma_f} \quad (D.17)$$

$$\alpha_{bend} = \min \left\{ 1; 0,05 \cdot \frac{r_{curv}}{\phi_{f,w}} + 0,3 \right\} \quad (D.18)$$

- z : lever arms of internal forces = $0.9d$
- $f_{f,w,lim}$: maximum allowable stress in the reinforcements [MPa]
- α_{bend} : coefficient taking into account the reduced strength in the bending zone of curved FRP reinforcements.
- $f_{f,w,d}$: design strength of the straight part of the shear reinforcement [MPa].
- $E_{f,w}$: elastic modulus of the shear reinforcement = E_f .
- $\epsilon_{f,w,lim} = 0.004$.
- α_f : coefficient accounting for the long-term effects of the environment on the tensile strength of the FRP reinforcement, = 0.8 for GFRP.
- γ_f : Partial coefficient related to the FRP reinforcement = 1.3.
- $\phi_{f,w}$: diameter of the transversal reinforcement, here = $8mm$.

The required spacing being over the maximal allowed spacing, we'll take $s = s_{max} = 125mm$.

APPENDIX

E

CODE TO READ DFOS DATA

In this annex, we'll detailed some python code snippet to efficiently treat data from the DFOS.

Because of its high level of discretisation in terms of spatial and time intervals, we often find ourselves dealing with data files of over 10^8 strain or temperature data points. To save on computational time each time we want to read and process these, we follow the steps below:

- We open the data file in Excel using Excel's built-in functions. Note that the decimal and thousand separators can vary and corrupt your data if not handled properly.
- We read the Excel file in a Jupyter Notebook using the Python library pandas. This library is very efficient with large datasets and can be seamlessly integrated with other data processing libraries, such as scipy. The Notebook allows us to read the Excel file only once and then run the other cells to process it, which is significantly faster.

Reading the file is straightforward with pandas as it allows us to skip the first rows that aren't strain or temperature data points. Here is a Python script to perform this operation:

```
import math
import matplotlib as mpl
import matplotlib.pyplot as plt
import pandas as pd
import numpy as np
from scipy.signal import argrelextrema

dir = "C:/Dir of the file containing the Excel/"
df = pd.read_excel(dir+'NameOfTheExcel.xlsx', sheet_name="NameOfTheSheet",
                  skiprows=32)
```

After reading the file, it's important to work on a copy of the dataframe to avoid overwriting it and consequently needing to read it again. If some gauges were used, this is also when we should strip the dataframe of the data outside of these gauges if it's unneeded:

```

df2=df.copy()

time=df2.iloc[:,0] #might be usefull later
df2.drop(df2.iloc[:,0:3],inplace=True, axis=1) #drop the first rows that
                                                are useless

x = df2.columns      #x vector

#Find the index of the gages
x_start = 1.5776 #value of the first usefull data point
x_end = 1.7986 #value of the last usefull data point
iloc_start = np.where(x == x_start)[0][0]
iloc_end = np.where(x == x_end)[0][0]

df2=df2.iloc[:,iloc_start:iloc_end+1]

```

Next, we have to deal with the numerous NaN values. Three possible solutions are considered here. When not much data is lost, masking the NaN is the best solution. When a lot of data is lost, it might be better to keep the NaN to avoid having straight lines from points that are quite distant from each other. Alternatively, more advanced interpolations could be used, but with the default interpolation, the noise produced is too high. Here's an example using data from a GFRP-Steel coupler test:

```

df3=df2.copy()
df4=df2.copy()
df5=df2.copy()

fig, axs = plt.subplots(3)

# keeping the Nan
y3 = df3.iloc[9280].values
axs[0].plot(x, y3)
axs[0].set_title('Keeping the Nan')

# masking the Nan
y4 = df4.iloc[9280].values
mask = ~np.isnan(y4)
xclean = x[mask]
yclean = y4[mask]
axs[1].plot(xclean, yclean)
axs[1].set_title('Masking the Nan')

# interpolating the Nan
df5=df5.interpolate()
y5 = df5.iloc[9280].values
axs[2].plot(x, y5)
axs[2].set_title('Interpolating the Nan')

fig.tight_layout(pad=2.0)

```

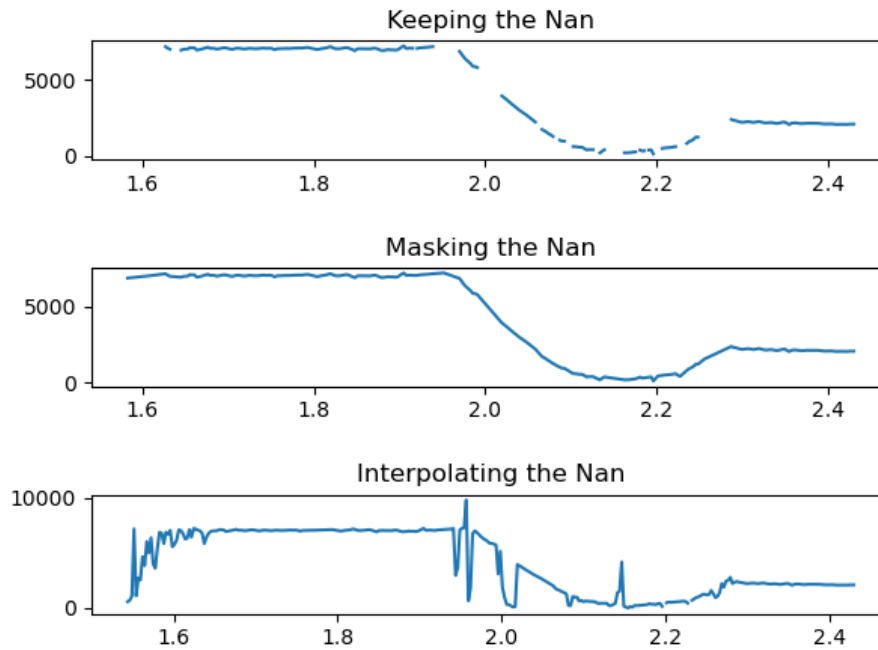


Figure E.1: Graphical representation of the different Nan processing procedures

The data can then be easily processed and plotted depending of the needs.

Unfortunately, sometimes the data is heavily corrupted and it becomes difficult to process. One way to do this on simple tests, where some portions of the sample are expected to have linear strain, is to plot its average strain against time. Even then, it can be useful to try and find the peaks of this plot. Here's an example with our data from the corrupted DFOS on a GFRP ribbed rebar:

```
# Create the y values (average of each row, excluding NaNs)
df4=df3.copy()
df4['average'] = df4.mean(axis=1)
time = pd.to_datetime(time)
df4['time_diff'] = (time - time[0]).dt.total_seconds()

# First iteration
maxima_indices = argrelextrema(df4['average'].values, np.greater)
maxima_df = pd.DataFrame({
    'time_diff': df4['time_diff'].iloc[maxima_indices[0]],
    'average': df4['average'].iloc[maxima_indices[0]]
})
maxima_df = maxima_df[maxima_df['average'] >= 0] # Only positive

# Iterations up to <10 peaks
while len(maxima_indices[0])>10 :
    maxima_indices_2 = argrelextrema(maxima_df['average'].values,np.greater
    )
    maxima_df_2 = pd.DataFrame({
        'time_diff': maxima_df['time_diff'].iloc[maxima_indices_2[0]],
        'average': maxima_df['average'].iloc[maxima_indices_2[0]]
    })
    maxima_df = maxima_df_2
    maxima_indices = maxima_indices_2
```

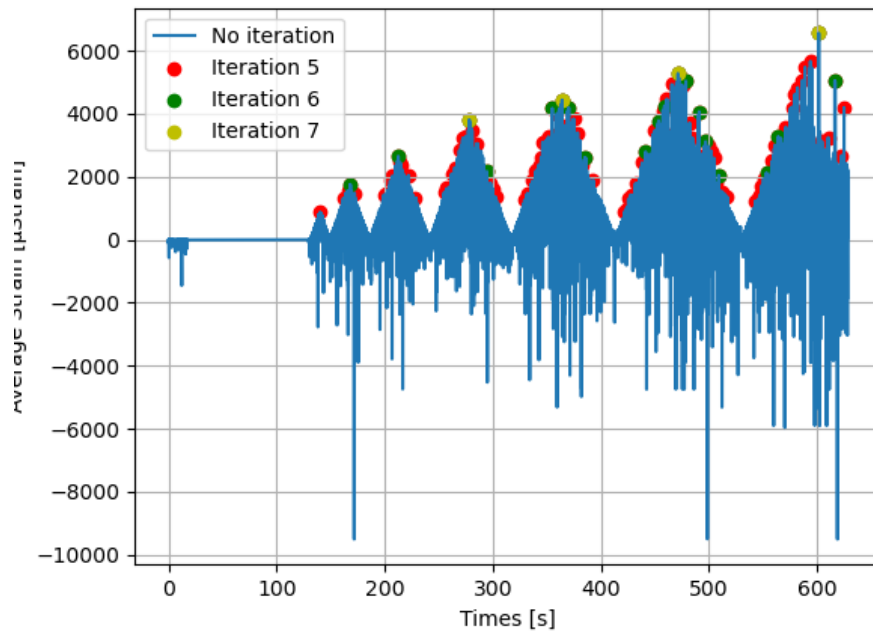


Figure E.2: Average strain peak on a corrupted DFOS

Finally, once all this data is processed, we can create an animation in a GIF format by using the FuncAnimation from the matplotlib library:

```
# Change matplotlib backend
\%matplotlib notebook

# Animation function
def animate(i):
    plt.cla()
    y = df3.iloc[i].values
    plt.plot(x,y)
    plt.xlabel("Distance on the Optical fiber [m]")
    plt.ylabel("Strain [*10**-6]")

# Create animation
ani = FuncAnimation(plt.gcf(), func=animate, frames= len(df3.index) ,
                    interval=10, repeat=False)

# Save animation
ani.save('AnimatedPlot2.gif', writer='imagemagick', fps=100)
```


METODOLOGY

In this section, we acknowledge the use of various AI-powered tools in the composition of this thesis. Primarily, these tools aided in navigating the extensive online literature. We utilized Connected Papers [19] to locate relevant papers on our topic and Consensus [20] to find pertinent research statements for our work.

Connected Papers is a tool that assists researchers in discovering and exploring academic papers relevant to their field of study. It generates a visual graph based on the analysis of approximately 50,000 papers, choosing the ones most closely related to the origin paper. The graph isn't a citation tree, but instead places papers based on their similarity in terms of co-citation and bibliographic coupling. This implies that papers with overlapping citations and references are likely to be closely connected. Connected Papers employs a force-directed graph algorithm that clusters similar papers together and separates dissimilar ones, providing a visual path from each paper to the origin. It utilizes the Semantic Scholar Paper Corpus database.

Consensus is an AI-powered search engine designed to identify and synthesize claims from scholarly research papers. Despite sharing technology with chatbots, Consensus differs in its function—it responds to research queries with relevant answers derived from academic papers. The platform draws from the Semantic Scholar database. At its core, Consensus operates through a proprietary "claim extraction" model applied to its database. This model identifies sentences, or "claims," where authors present their findings based on experimental evidence. The model has been trained on tens of thousands of annotated papers by PhD scholars, resulting in an expansive database of scientific claims ready for search and analysis.

Lastly, we extensively employed the language model **ChatGPT** from OpenAI [59], particularly the GPT3.5 and GPT4 datasets for the following functions:

- Drafting initial versions of some sections
- Summarizing scientific papers
- Translation, spelling, and grammar checking
- Text reformulation and harmonization

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