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Composites with Inorganic Matrix for Repair of Concrete and Masonry Structures

Editors:

Gianmarco de Felice, Lesley H. Sneed,
and Antonio Nanni

SP-324



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Preface

This SP is the result of two technical sessions held during the 2017 ACI Spring Convention in Detroit, MI. Via presentations and the resulting collection of papers, it was the intention of the sponsoring committees (ACI Committees 549 and 562 together with Rilem TC 250) to bring to the attention of the technical community the progress being made on a new class of repair/strengthening materials for concrete and masonry structures. These materials are characterized by a cementitious matrix made of hydraulic or lime-based binders, which embeds reinforcement in the form of one or more fabrics also known as textiles. The great variability of fabric architectures (for example, cross sectional area, strand spacing, and fiber impregnation with organic resin) coupled with the types of material used (aramid, basalt, carbon, glass, polyparaphenylene benzobisoxazole (PBO) and coated ultra-high strength steel) makes the characterization, validation, and design of these systems rather challenging. Irrespective of the reinforcement type (synthetic or ultra-high strength steel), the impregnating mortar is applied by trowel or spray-up. It should also be noted that fabric reinforced cementitious matrix and steel reinforced grout, in particular, are very different from other repair technologies such as FRC (fiber reinforced concrete) and UHPC (Ultra High-Performance Concrete) in that they utilize continuous and oriented reinforcement. In a sense FRCM and SRG can be viewed as the modern evolution of ferrocement.

The terminology adopted in ACI to identify this class of new products is FRCM (fabric reinforced cementitious matrix) when the reinforcement is made of man-made fibers and SRG (steel reinforced grout) when the reinforcement is made of ultra-high strength steel. While it is recognized that other organizations and professionals may refer to these systems with different acronyms and names (for example, TRM for textile-reinforced mortar, TRC for textile reinforced concrete), ACI Committee 549 has made the conscious decision to adopt FRCM and SRG to unequivocally identify systems solely intended for repair and strengthening as opposed to new construction.

The two technical sessions and the resulting Special Publication were also the first tangible outcomes of the partnership between ACI Committee 549 and Rilem TC 250 that has as its ultimate objective, jointly producing a design guide for the repair/strengthening of masonry structures using FRCM and SRG. The motivation of the partnership stems first from the desire to maximize efficiency by drawing from the experience and expertise of professionals from around the world. Additionally, the intention is to develop a guide that harmonizes the design procedure based on the approaches followed in the U.S. and European Union so that practitioners can adopt this technology irrespective of the geographical location and with the same outcome.

The editors of this SP are most grateful to the technical sessions presenters and the authors of the papers for having shared their precious knowledge. The editors hope that this SP will be a relevant contribution to the deployment of suitable technologies for the safe preservation and use of existing concrete and masonry structures.

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Determination of Material Properties of TRC Under Cyclic Loads

Karoline Holz, Elisabeth Schütze, Patricia Garibaldi, Manfred Curbach

Synopsis: To enable a widespread industrial application of carbon reinforced concrete, a reliable characterization of material properties, such as the tensile strength and bond behavior of the composite material, has been established using standard test methods. To allow for an even wider range of application, the fatigue behavior and corresponding appropriate test methods were examined in this paper.

A test method along with an experimental setup for determining both static and cyclic tensile behavior is presented. Test results using the proposed methods are shown and discussed. Based on the results of cyclic tests of a reference load combination, load frequency as well as load level were varied and their influence is documented in this paper. Finally, the temperature development during cyclic tests is documented for one test specimen. Based on these results, future research opportunities are presented.

Keywords: Carbon Concrete Composite, Carbon reinforced concrete, cyclic loading, experimental methods, fatigue, tensile strength, textile reinforced concrete, TRC

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INTRODUCTION

Over twenty years ago, researchers in Germany began to investigate a new material known today as textile reinforced concrete (TRC) (Scheerer et al. 2015). TRC consists of a high-performance textile reinforcement embedded in a fine-grained concrete. Today, given the high tensile strength of carbon reinforcement, corrosion-resistant carbon fiber materials are the preferred choice for building efficient TRC structures. This composite material has been successfully used in multiple pilot projects (see e.g. Frenzel et al. 2013); where it was used to construct thin structural elements, façade panels or precast elements for pavilions and for strengthening purposes, e.g. for shell roofs or silos. TRC is either cast in formwork or applied in layers.

Based on extensive groundbreaking research, the new research program *C³ – Carbon Concrete Composite* was launched to investigate the material behavior further and to introduce the material into the construction industry. In this interdisciplinary project, more than 150 partners from research institutions and industry are working together towards this goal.

One of the first steps in this process was to establish reliable test methods for the characterization of the basic material properties. Thus, standard methods for testing tensile strength and bond behavior of the composite under predominantly static loads were defined and validated through multiple tests in the laboratories of the Institute of Concrete Structures at RWTH Aachen, MFPA Leipzig GmbH, and the Institute of Concrete Structures at TU Dresden.

To allow for an even wider range of application for example the strengthening of reinforced concrete bridges, the next step was to examine the fatigue behavior of TRC and the corresponding appropriate test methods. Previous research mainly focused on examining the components of the composite material textile reinforced concrete under fatigue loading. For instance, Marx et al. 2013 and the guidelines of the Model Code for Concrete Structures (fib 2013) showed that the fatigue behavior of concrete depends on the medium stress. Results on carbon fiber rovings or grids were not known. Carbon, in the form of CFRP, is considered unsusceptible to fatigue stress levels up to 80 % of its short-term strength (Nanni et al. 2014; fib 2013).

However, it cannot be simply assumed that the material behavior of the two individual components, is the same as that of the composite material. The interaction not only between concrete and reinforcement but between the individual carbon fibers in a roving also has to be taken into account. Jesse (2012) stated, that the bond between reinforcement and concrete, has a huge influence on the fatigue behavior. Research on the fatigue strength of TRC as a composite material has so far given inconsistent results. While Hansl (2014) found a significant increase in the stiffness of TRC specimens after cyclic preloading, Schütze et al. (2015a) could not confirm this. Instead, they found a slight increase in strain and no decrease in tensile strength due to cyclic loading. However, these investigations had

a limited scope, and only one load level was tested. Another problem was the use of varying test methods, thus making it difficult to compare results among various research.

RESEARCH SIGNIFICANCE

Establishing appropriate test methods for determining the tensile properties of TRC under cyclic loads is of crucial importance in understanding the interaction not only between concrete and reinforcement but also in between the individual carbon fibers of a typical composite material. A suitable test methodology was developed based on the standard test for the tensile strength of TRC under predominantly static loads. It is anticipated that the methodology proposed will assist in determining the effect of test frequency and load level on the fatigue behavior of TRC.

EXPERIMENTAL INVESTIGATIONS

Materials and test specimens

The experimental investigations described in this paper were conducted with a material combination that is typically used for strengthening concrete structures.

For the concrete matrix, a commercially available fine-grained concrete, with a maximum aggregate size of 1 mm (0.04 in.) was used. The ready-mix consists of cement, fly ash, silica fume, quartz sand and concrete plasticizer; only water is to be added before use. The concrete has a soft plastic, thixotropic consistency; thus, it can be applied in both a wet-spraying and a laminating process. The compressive strength of the concrete at 28 days is 80 MPa (11.6 ksi). The flexural strength is 8 MPa (1.2 ksi). The tests were conducted on prismatic specimens measuring 160 mm × 40 mm × 40 mm (6.30 in. × 1.58 in. × 1.58 in.) according to the European standard DIN EN 196-1 (2016).

The textile reinforcement used in the experimental investigations was an open-mesh carbon fabric like the one shown in Figure 1. It consists of carbon rovings with different fineness in the warp and weft direction (see Table 1). The textile reinforcement was impregnated with a styrene-butadiene rubber suspension (SBR). This makes handling of the reinforcement easier, and improves its strength and bond properties while keeping it flexible for easier installation during strengthening applications. For the results shown in this paper, the samples were tested along the warp direction of the textile.

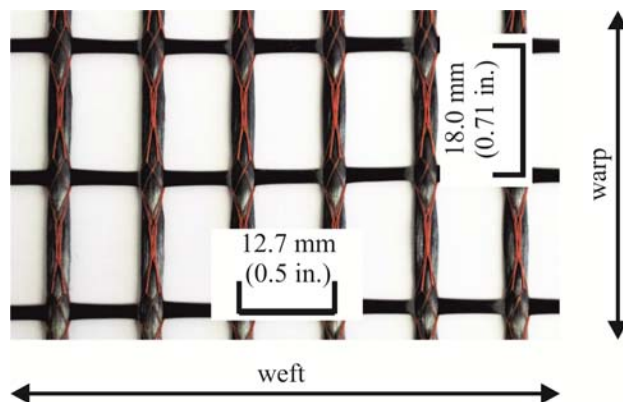


Figure 1 — Carbon textile, approved acc. to (Z-31.10-182 2016)

Table 1 — Properties of the textile reinforcement

characteristics	unit	value
roving fineness		
- warp direction	tex	3200
- weft direction	tex	800
roving cross-sectional area		
- warp direction	mm ² (in ²)	1.83 (0.0028)
- weft direction	mm ² (in ²)	0.45 (0.0007)
distance between the rovings		
- warp direction	mm (in.)	12.7 (0.5)
- weft direction	mm (in.)	18.0 (0.7)

The test specimens were produced from 1.2 m × 0.7 m (47.2 in. × 27.6 in.) and 9 mm (0.35 in.) thin concrete slabs manufactured in a hand-lamination process by alternately layering concrete and textile reinforcement in a smooth, non-absorbing steel formwork, as shown in Figure 2 and Figure 3. The samples consisted of three concrete and two reinforcement layers, arranged symmetrically within the sample thickness, parallel to the slab surface. Each concrete layer had a thickness of 3 mm (0.12 in.), including the thickness of the embedded textile layer. For the first three days, the slabs remained in the formwork at about 20 °C (68 °F), covered by a protective layer while curing. Then, the formwork was removed, and the concrete slabs were stored in water for another four days. Subsequently, the individual specimens with a size of 1100 mm × 100 mm × 9 mm (43.31 in. × 3.94 in. × 0.35 in.), Figure 4, were produced using a water-cooled diamond saw and stored in a climate chamber at 20 °C (68 °F) and 65 % relative humidity until testing. Every specimen consisted of a central test area with a length of 300 mm (11.81 in.) and anchorage areas, at both ends of the specimen, with a length of 400 mm (15.75 in.). To prevent a pull-out failure of the rovings, an additional coating of epoxy resin and sand was applied to the textile, within the end anchorage areas before casting, leaving the central area unaffected. As cyclic tests can take several days, the specimens are stored for at least three months before they are tested under cyclic loads, to prevent a noticeable change in material characteristics due to the aging of the specimens.

The test results described in this paper were obtained from specimens that were more than 250 days old. For these specimen, a total of six slabs were produced. The slabs were each cut into six specimens. Two specimens out of each slab were tested in quasi-static reference tests. The other four underwent fatigue tests.



Figure 2 — Manufacturing of specimen, here: application of the first concrete layer into the formwork (photo: Karoline Holz)



Figure 3 — Manufacturing of specimen, here: application of the first layer of reinforcement (photo: Karoline Holz)

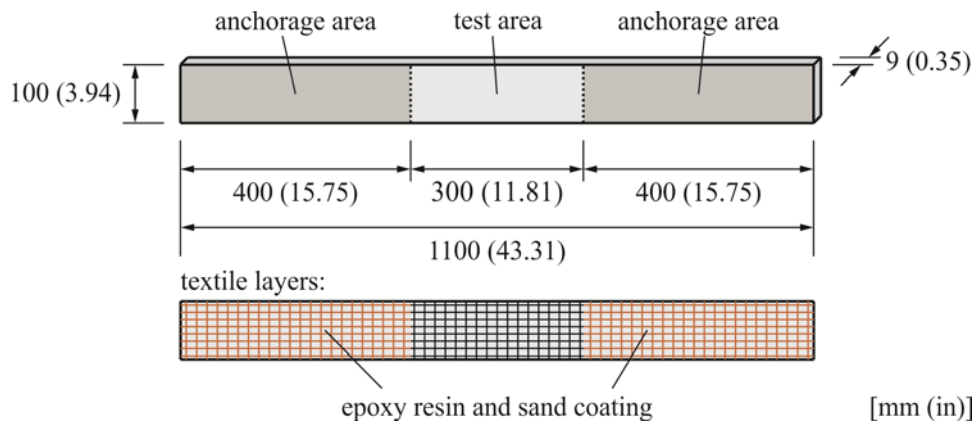


Figure 4 — Test specimen (graphic: Elisabeth Schütze)

Test methods and experimental devices

The tensile strength and the corresponding stress-strain curve of textile reinforced concrete were experimentally determined by uniaxial tensile tests. To carry out these tests, the specimens were clamped between steel plates at the upper and lower anchorage area, as shown in Figure 5. For the clamping construction, various configurations are possible. For instance, the pressure can be applied servo-hydraulically, pneumatically or via screws at both sides of the steel plates. A number of preliminary tests were carried out in the basic project B3 (Schütze et al. 2016) within the project C3 – Carbon Concrete Composite to find a suitable setup for the fatigue tests. It was found that a double-symmetric, compact and stable structure to transfer the load is particularly important to prevent both fatigue of the clamping construction and deviation of the specimens from the loading axis. Particularly, loosely joined connections with too many parts should be avoided. The clamping construction with lateral screws, shown in Figure 5, was specially designed to meet these requirements.

The specimens were clamped thru a length of 300 mm (11.81 in.) at the beginning of the anchorage area of the sample. To provide a larger anchorage length of the textile in the concrete and to prevent bond failure, the clamping constructions were arranged in a way that the specimen protruded over the clamping area by 100 mm (3.94 in.) at both ends. This arrangement resulted in an anchorage length of 400 mm (15.75 in.) at the ends of the specimen. To improve grip and avoid slipping of the specimen, an abrasive cloth layer was placed between the specimen and the steel plates.

After fixing the specimens to the clamping mechanisms, the specimens were mounted in a servo-hydraulic test machine. Rod ends were used to ensure an articulated connection between the clamping construction and the machine and to properly align the specimen while applying the load to prevent bending or torsion. To prevent an off-axis oscillation of the test specimens during a fatigue test, a precise, symmetrical installation was necessary. At first, a pre-load no larger than 0.5 kN (112 lb) was applied to check the specimen's alignment and to make corrections if necessary. The quasi-static reference tests were carried out displacement-controlled with a loading rate of 1 mm/min (0.04 in./min) until the sample failed due to tensile fracture. The tensile strength f_t obtained in this manner was used to define the limits of the alternating stress range during cyclic loading, as a percentage of the tensile strength.

The cyclic tests were carried out force-controlled. At first, the specimens were loaded up to the mean load. Then, the cyclic loading was applied with the chosen test frequency and load amplitude either until the sample failed or up to a set number of load cycles. Finally, specimens that did not fail during the cyclic loading process were tested in a quasi-static tensile test without prior unloading to obtain the residual strength.

The measured values were the applied force F , the piston displacement s and the elongation Δl , at the center of the specimen. The elongation was measured with extensometers with a gauge length of $l = 200$ mm (7.87 in.), arranged at both sides in the center of the specimen at a distance of 50 mm (1.97 in.) between the measuring range and the clamping. As they are particularly resistant to fatigue, inductive displacement transducers with a plunger were used in the cyclic tests. The measuring rate during the quasi-static loading was 5 Hz. During cyclic tests, the measuring rate was at least 20 per cycle. To keep the amount of data manageable, only 10 of every 150 cycles were logged.

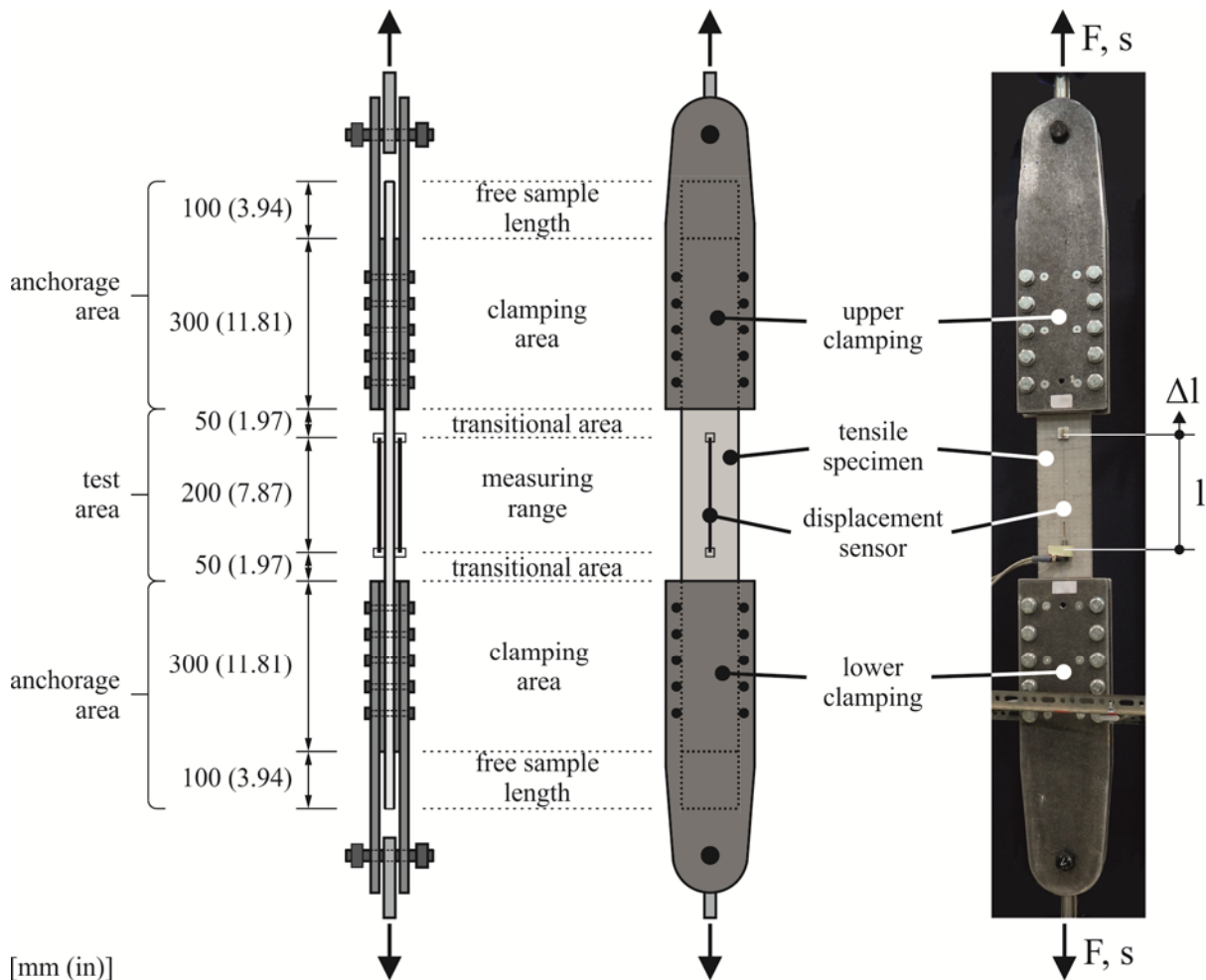


Figure 5 — Experimental setup (graphic and photo: Karoline Holz and Elisabeth Schütze)

From the measured data, the strain ε_{tex} was calculated by relating the elongation Δl to the measured length l , and the textile stress σ_{tex} , by relating the applied force F to the cross section of the textile reinforcement in the specimen A_{tex} . From these the values, the stress-strain curve for TRC was determined.

To obtain reference tensile strength and stress-strain curves, two specimens from each TRC-slab were tested in quasi-static reference tests. Figure 6 shows the individual stress-strain curves from these tests. Curves belonging to the same slab batch are depicted in the same color. The specimens show the characteristic material behavior of TRC when using a heavy-tow carbon textile. As described in detail in Jesse 2004 and Schütze et al. 2015b, the stress-strain curves consist of three sections: the non-cracked state (state I) with a steep course of the curve, the state of multiple crack formation (state IIa) with a very low increase of the curve and the final crack state (state IIb) where the course of the curve steepens again. Brittle failure of the specimen occurs when the tensile strength of the textile reinforcement is reached. The tensile strength measured in the reference tests ranged between 2067 N/mm² (299.8 ksi) and 2607 N/mm² (378.1 ksi). However, it is obvious from Figure 6 that the dispersion of measured strengths reduces drastically if the stress-strain curves are considered separately for each slab. Hence, to calculate a reference tensile strength as a base for defining the load levels for the fatigue tests, a mean strength was calculated for every batch of samples produced from a given slab. Each of these mean strengths was used as the reference to set the load levels when testing the remaining four specimens produced from one slab.

Figure 6 also shows a remarkable dispersion of the measured strain in the tests. One of the reasons for this is the formation of a very small number of cracks within the measuring range. The strain measured on TRC is only slightly larger than the sum of the crack widths in the measured range. Thus, for an ideal strain measurement on the compo-

site material that captures the strain in a reliable manner, the gauge length should be a multiple of the crack spacing. As the crack pattern and crack spacing are slightly different for each specimen, this condition is seldom met in an experiment. For a large number of cracks, the effect of the relation of gauge length to crack spacing on the measured strain is small. However, if there is only a small number of cracks within the measured range, and the crack spacing is relatively big in relation to the gauge length, a slight variation of the crack pattern makes a huge difference.

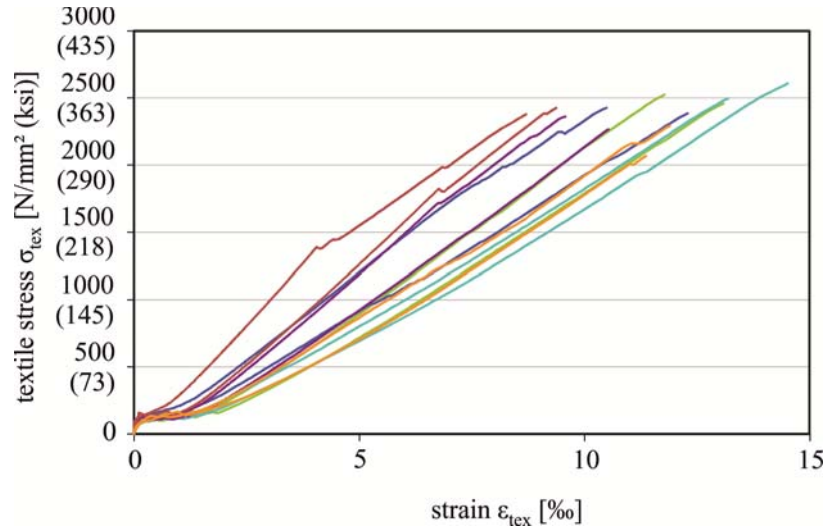


Figure 6 — Stress-strain curves of the reference tests (graphic: Karoline Holz and Elisabeth Schütze)

Cyclic tensile tests

As the authors research's goals is to expand the range of application of TRC to the strengthening of reinforced concrete structures subjected to fatigue, among them bridges, the effect of a high number of cycles on the fatigue behavior of TRC was investigated. The number of load cycles was set to a maximum of two million, following the Eurocode Guidelines for Steel Constructions (DIN EN 1993), as TRC is often compared to steel. In addition, the Eurocode Guideline for Actions on Structures (DIN EN 1991) states that two million cycles is indicative of the number of heavy vehicles expected for roads and motorways with high flow rates per year and per slow lane, and it shall be taken into consideration in the design of concrete bridges. Specimens were tested under variable stress ranges and load frequencies. Specimens that did not fail during the cyclic loading process were tested in static tensile tests to obtain their residual strength. The tests proceeded as follows. First, a reference fatigue loading, also referred as 'reference load level', was chosen with a stress range between 30 and 60 % of the static reference tensile strength f_t and a cyclic loading applied with a frequency of 10 Hz. Subsequently, the load frequency was varied within the stress range established. Due to time constraints, only a small number of tests were performed, as one test with 2 million load cycles and a load frequency of 5 Hz took five days to complete. Afterwards, the load frequency of 10 Hz was kept constant, and the stress range was varied. This was done to identify a dependency of the material behavior on the stress range. A test with a load frequency of 10 Hz and stress levels alternating between 35 and 65 % of f_t was chosen to compare the material behavior. Also, one test specimen was observed with an infrared camera to document the temperature variation during cyclic loading. An overview of all tests parameters follows in Table 2. The maximum, minimum, and mean stresses in Table 2 are shown as a percentage of the reference tensile strength.

Table 2 — Test parameters

Maximum stress σ_{max} [%]	Minimum stress σ_{min} [%]	Mean stress σ_{mean} [%]	Amplitude σ_a [%]	Load frequencies f [Hz]	Number of tests [-]
60	30	45	15	10	4
60	30	45	15	5	2
60	30	45	15	20	2
60	30	45	15	30	2
70	40	55	15	10	2
65	35	50	15	10	3
50	20	35	15	10	2

Material behavior – reference load level — The material behavior of the cyclic loaded reference specimen is illustrated by the aforementioned test with a load frequency of 10 Hz and stress levels alternating between 30 and 60 % of f_t for 2 million load cycles. After the cyclic loading, the residual ultimate tensile stress was determined. In Figure 7, it can be seen that the strain increase over the course of 2 million load cycles is minimal. Once the maximum number of load cycles is reached, the load drops slightly due to the machine stopping. This specimen did not fail during cyclic loading, rather all 16 rovings were broken at maximum tensile stress, see Figure 8 and Figure 9.

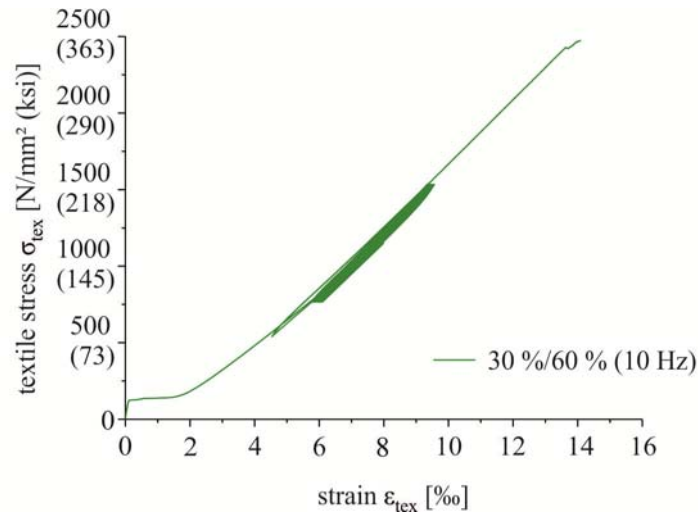
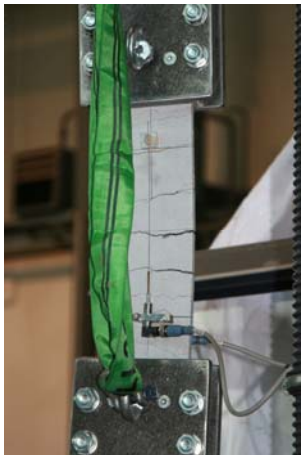
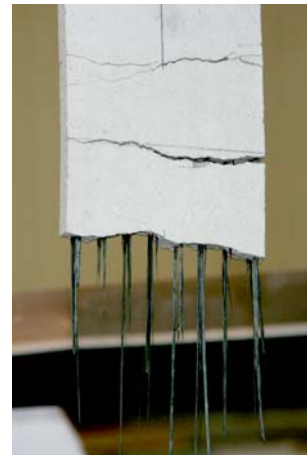
**Figure 7** — Stress-strain diagram over the course of 2 million load cycles (graphic: Karoline Holz)**Figure 8** — Specimen after failure (photo: Karoline Holz)**Figure 9** — Broken rovings after pulling the specimen apart (photo: Karoline Holz)

Figure 10 shows the development of the minimum and maximum of the strains measured on both sides on the test specimens, at the minimum and the maximum stress of the load cycles. It shows no significant increase in strain within the measured range of the inductive displacement transducers over the course of the load cycles. But the increase in the piston displacement is slightly noticeable. This indicates that cracks formed outside the measured range.

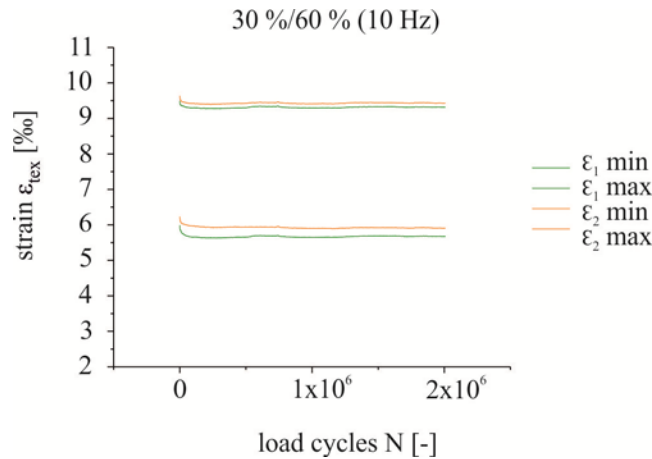


Figure 10 — Strain – load cycle diagram (graphic: Karoline Holz)

Variation of the test frequency — Within the scope of the test program, specimens were tested under different load frequencies. It was shown that at higher frequencies the experimental setup tends towards a resonant oscillation. Due to this, the number of load cycles that can be applied are smaller than those with a lower load frequency. With a load frequency of 5 Hz, the specimen was subjected to 2 million load cycles during a period of five days. With a load frequency of 10 Hz, the scattering of the data was greater (see Figure 11), but the testing time was reduced by half. At a frequency of 10 Hz, the phenomenon of transverse vibration or self-oscillation was not observed. Therefore, a test frequency of 10 Hz was selected for the experiments.

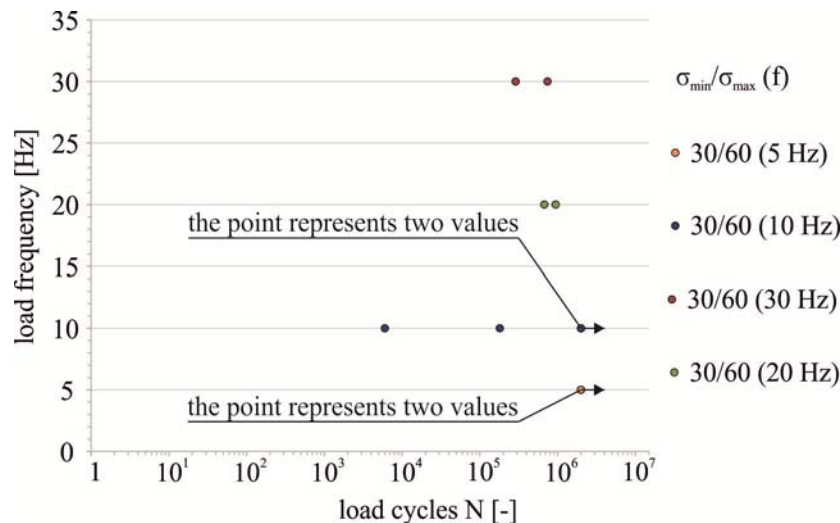


Figure 11 — Load cycle at which failure occurred within a given load frequency (graphic: Karoline Holz)

In Figure 12, the strain of a specimen at σ_{min} and σ_{max} measured during cyclic loading with 30 Hz is compared to that of a specimen tested at reference load level with $\sigma_{min} = 30 \% f_t$ / $\sigma_{max} = 60 \% f_t$, 10 Hz (for this specimen, only the first part of the test with 2 million load cycles is shown in the diagram). The strains in the test at 30 Hz were lower than that of the reference specimen. Cracks developed outside the measuring range at a frequency of 30 Hz, so the measured strain decreases over the course of the experiment.

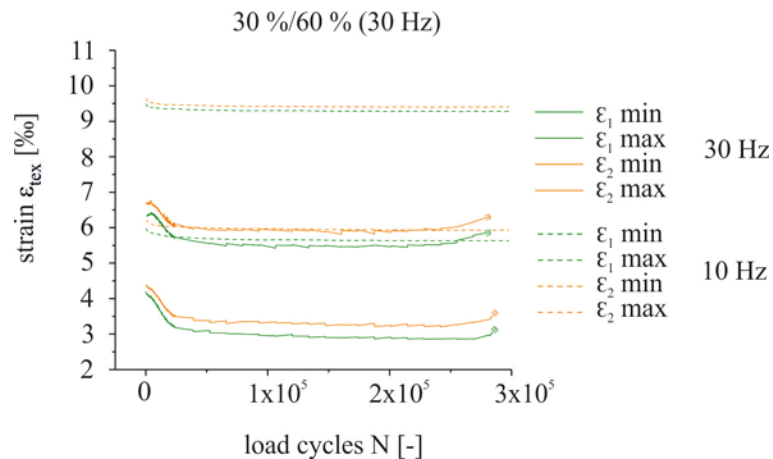


Figure 12 — Strain-load cycle diagram (graphic: Karoline Holz)

Material behavior – variation of the load level — To compare the reference load level with another load level, a test with a maximum stress of 65 % f_t , a minimum stress of 35 % f_t and a load frequency of 10 Hz was selected. The specimen collapsed after 48,000 load reversals. The critical maximum load of this material probably is between 60 and 65 % of the tensile strength. Figure 13 shows the recorded strength on the y-axis and the strain on the x-axis. For this test specimen, the strain is higher, than that of the reference load specimen. There are also discontinuities in the strain because the data acquisition scheme did not capture all the strain increase data during the 150 cycle intervals. The data clearly shows increasing damage to the specimen during the test. Ultimately, the damage observed in the specimen led to the failure of the sample, while subjected to cyclic loading. A clear tensile failure can be observed during the cyclic loading within the stress range, see Figure 14 and Figure 15. Also, spalling of the concrete layer is evident in Figure 14.

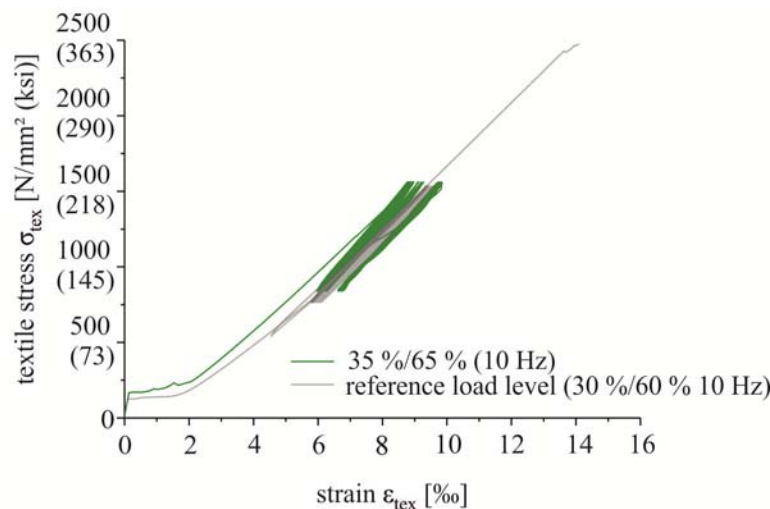


Figure 13 — Stress-strain diagram (graphic: Karoline Holz)



Figure 14 — Specimen after failure
(photo: Karoline Holz)

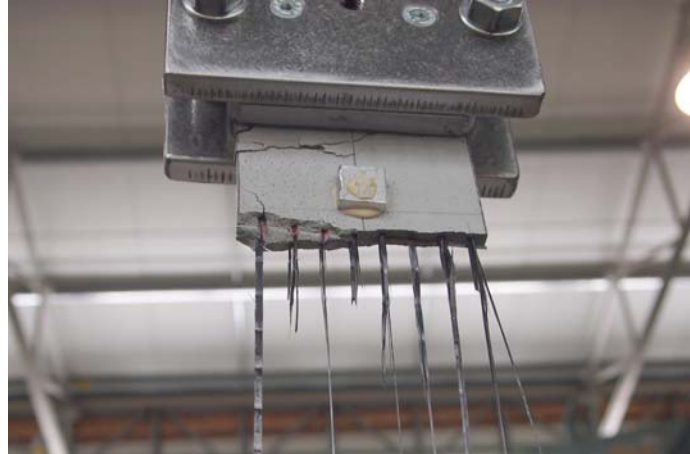


Figure 15 — Broken rovings after the pulling apart of the specimen (photo: Karoline Holz)

Figure 16 shows the development of the strain over the course of the load cycles. In the range of 0 to approximately 40,000 load cycles, the strain increased slightly. After 40,000 load cycles, the strain increase is noticeable, as the sample begins to approach failure. After approximately 48,000 load cycles the strains suddenly increase in a disproportional manner and failure occurs. Compared to the reference load level, which is not completely outlined in the diagram, the strains are nearly identical at the beginning of the load cycles. The strain of the 35/65 % test increases at a higher rate than the strain of the reference test.

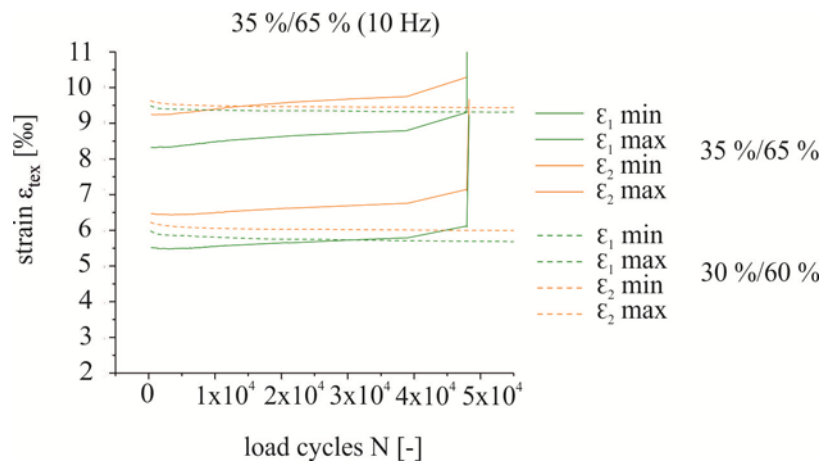


Figure 16 — Strain-load cycle diagram (graphic: Karoline Holz)

Overview of test results — It was not possible to determine the dependence of the material behavior on either the maximum, mean, and minimum stress or the amplitude in these experiments. To visualize the test results, a graph showing the mean stress is used. Figure 17 shows the results of all tensile tests carried out under cyclic loads with a frequency of 10 Hz and an amplitude of 15 %. The mean stress and load cycle relationship is plotted. The x-axis represents the load change number reached before failure, and the y-axis shows mean stress depending on the reference tensile strength. Tests in which the specimen did not fail after 2 million load cycles are identified by an arrow pointing to the right. For these specimens, a residual strength test was performed following the cyclic loading. In principle, it is shown that the maximum number of load cycles that a specimen can sustain decreases with an increase in the mean stress.

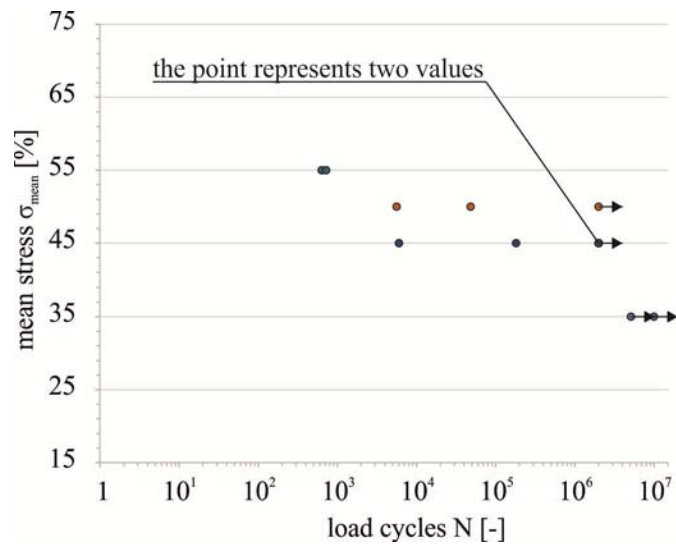


Figure 17 — Mean stress-load cycle diagram (graphic: Karoline Holz)

Figure 18 shows the reference strength on the x-axis and the residual strength on the y-axis. The reference strength used was the average of the values measured for the two tensile strength reference tests for a given batch of specimens from the same slab. Comparing the reference tests and the residual strengths (Figure 18), it can be seen that the residual strengths are only slightly smaller than the reference tests. This means that when the defined maximum load cycles of 2 million are reached, the samples have approximately the same ultimate strength as the static reference tests. No dependence of residual strength on load level, load frequency, load cycles is recognizable.

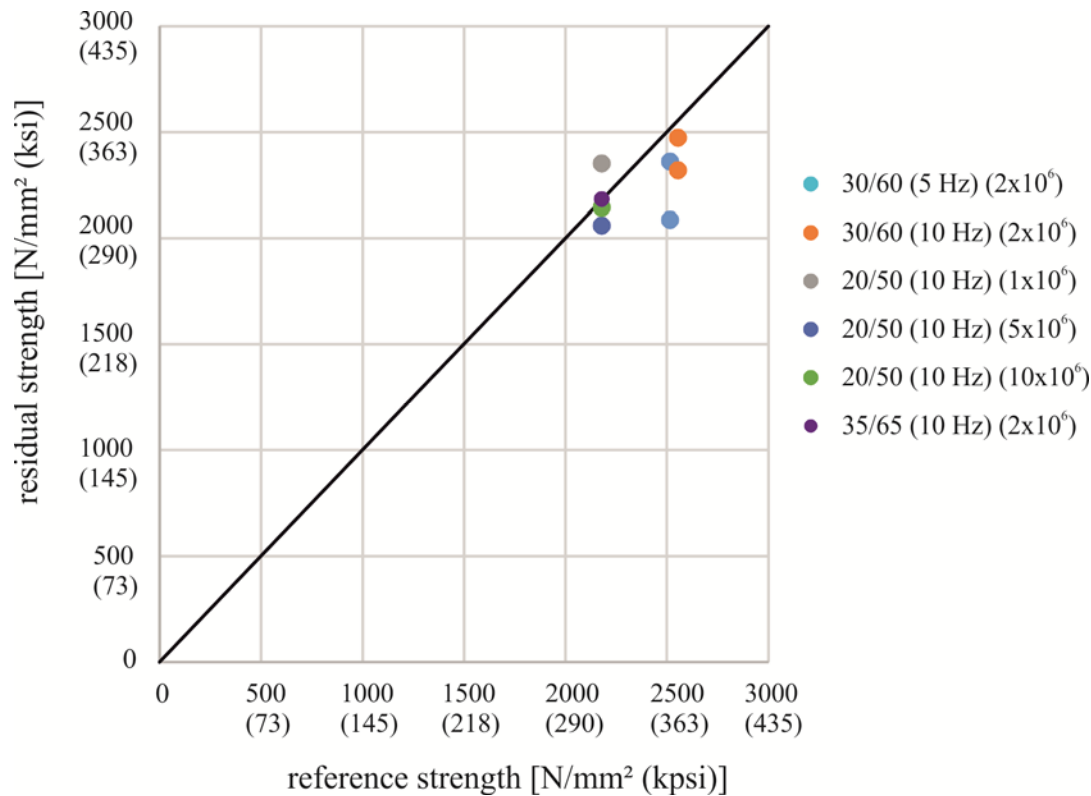


Figure 18 — Residual strength-reference strength diagram (graphic: Karoline Holz)

Temperature development — It is discussed controversially whether the temperature in concrete increases under fatigue stress or not. For TRC such behavior has not yet been quantified. An increase in temperature can affect the

material behavior, especially when using such thin specimens like for TRC. Thus, one test specimen was observed with an infrared camera to document the temperature development during cyclic loading. The test was carried out with a load level between 20 and 50 % of the reference tensile strength, and at a frequency of 10 Hz. The first recognizable sign of a temperature increase in the test specimen occurred at 3,600 load cycles. During the cyclic loading, an image of the specimen was recorded every 5 minutes. There is a clear development of specimen and ambient temperatures due to the temperature fluctuation between day and night. Five distinctive images are presented to illustrate the temperature increase in the sample. In Figure 19, five cracked regions are evident. These areas exhibit a slight temperature increase.

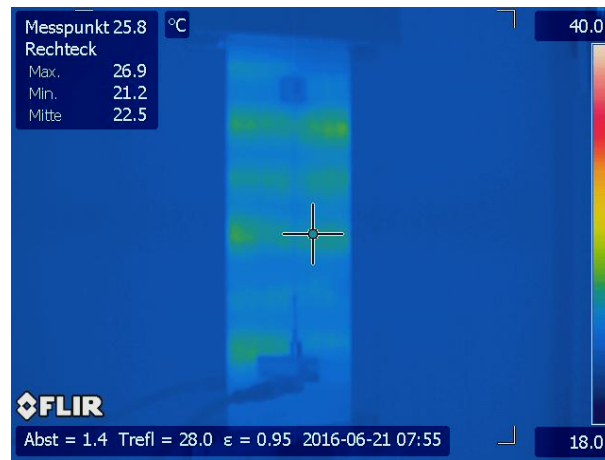


Figure 19 — Temperature increase after 5,800 load cycles

As the testing progressed, the increase in temperature was concentrated around two of the cracks (Figure 20 to Figure 23). As a reference, a crosshair in Figure 20 to Figure 23 approximately reflects the temperature of the test specimen. The temperature difference between material at the crosshair and the maximum temperature is only about 1 to 3 K. The test specimen is approx. 8 to 10 K warmer than the ambient air temperature. The increase in temperature observed during cyclic loading is small. Please note that the measuring error was ± 2 K.

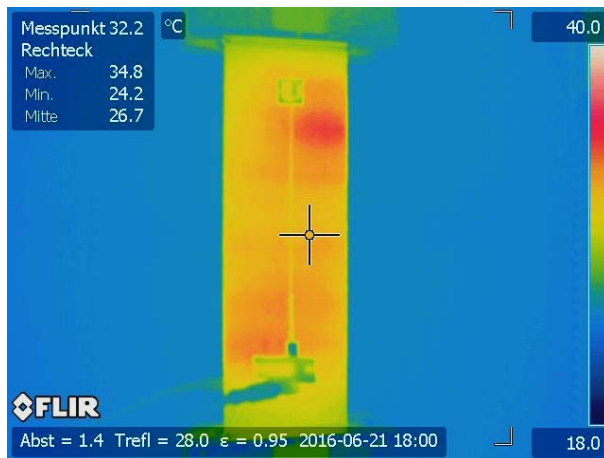


Figure 20 — Temperatures after 369,000 load cycles

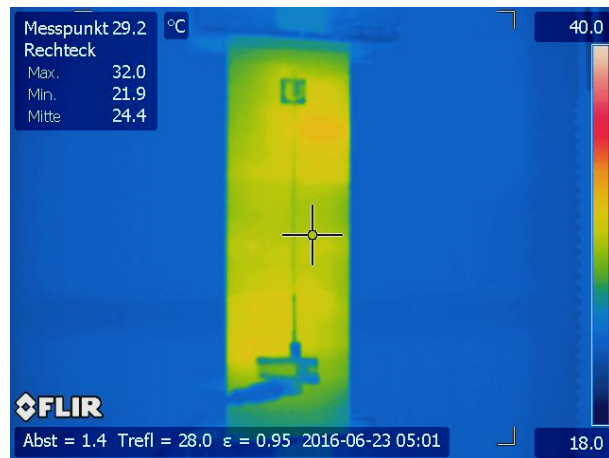


Figure 21 — Temperatures after 1,629,000 load cycles



Figure 22 — Temperatures after 2,061,000 load cycles

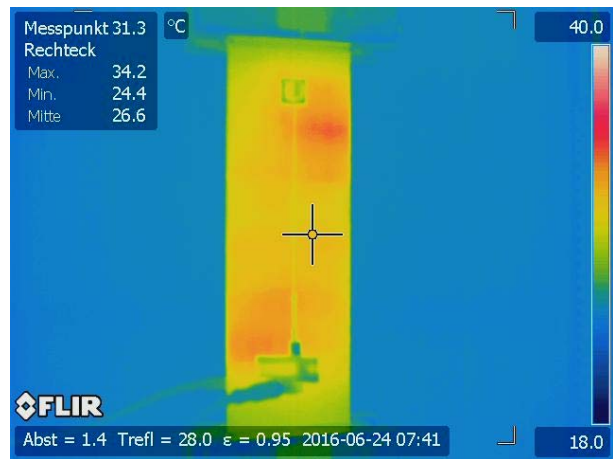


Figure 23 — Temperatures after 2,589,000 load cycles

The documentation of the temperature increase at a load frequency of 10 Hz during cyclic loading shows that only minimal heating occurs. Although the cracks heat up more than the rest of the test specimen, the temperature fluctuations in the surrounding air are higher. Heating due to cyclic loading is considered to be non-critical.

CONCLUSIONS

The designed experimental setup was successful and proved to be suitable for testing TRC under cyclic loading with a load frequency of 10 Hz. First tests to describe the material behavior of TRC were carried out. It is still necessary to precisely define the critical load and a safety level. Further tests with more than 2 million load cycles must also be carried out.

The first cyclic tests on this material combination show no significant dependence of material behavior on either the maximum, mean, and minimum stress or the amplitude. Further tests are recommended to confirm and expand the findings of this research.

The documentation of the temperature development during the cyclic loading shows that the temperature increase within the sample is non-critical.

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The findings presented in this paper have already been documented in an internal report.

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Comparative Assessment of the Tensile Behavior of Steel and Textile Reinforced Mortar Systems

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Synopsis: Steel reinforced grout (SRG) systems consist of high strength steel cords or fabrics embedded in a mortar or grout matrix; these are typically utilized as external reinforcement of existing reinforced concrete (RC) or masonry structures. Despite extensive experimental investigations conducted so far, reliable answers concerning their tensile, bond strength, strain to failure and stiffness properties seem still incomplete mainly due to their heterogeneous nature; this represents a limit for the design, standardization process and applications of such a system. These limiting aspects are also of a major interest in comparison with similar strengthening systems, namely textile reinforced matrix (TRM), in which the reinforcing system is typically made of a fiber reinforced polymer (FRP) bi-directional grid rather than steel cords. This study deals with an experimental investigation on the tensile behavior of SRG and TRM systems, focusing on the mechanical aspects related to the different scales of the strengthening system itself, i.e. reinforcement and composite system. The tensile tests are carried out on two different SRG systems manufactured with two different matrix types. Typical glass TRM systems are also considered in this study for comparison purposes. Based on the experimental outcomes, some analytical interpretations are provided.

Keywords: Glass fibers, steel reinforced grouts (SRGs), steel fabric, tensile behavior, textile reinforced mortars (TRMs).

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INTRODUCTION

Over the last decades, structural rehabilitation of existing buildings has increasingly attracted the attention of scientific community and government institutions. A significant focus has been put on existing reinforced concrete (RC) and masonry structures due to the large portion of the existing building stock they encompass in many European areas, such as Italy. Those structures typically do not comply with current structural codes and, at the same time, suffer from physical/environmental degradation or even structural damage induced by hazardous events.

In this regard, several retrofit/strengthening techniques have been developed so far, often considering innovative technologies and material compatibility as valuable prerequisites for the retrofit/strengthening process itself. Among innovative strengthening strategies, techniques based on Fiber Reinforced Polymers (FRPs) commonly provide a series of advantages, including high strength-to-weight ratios, corrosion and fatigue resistance, negligible influence on global structural mass, easy handling and installation, and low architectural impact (Prota et al. 2008; Triantafillou 1998).

Recently, alternative strengthening techniques have been proposed in substitution of conventional FRP epoxy bonded materials for structural reinforcing of existing RC and masonry elements. The key aspect is the replacement of the polymer-based matrix with an inorganic one (e.g. cement based, geopolymers or lime mortars etc.), resulting in composite systems generally named as fiber reinforced cementitious mortar (FRCM) or textile reinforced mortar (TRM) or, in the case of steel reinforcements, as steel reinforced grouts (SRG) (D’ambrosi et al. 2011; Menna et al. 2013; Ombres 2011). The variety of reinforcement types utilized so far is wide and encompasses glass, carbon, basalt, PBO, hemp fibers or different configurations of steel cords (De Felice et al. 2014; Escrig et al 2015; Menna et al. 2015). The advantages associated with these strengthening techniques are several; among them, the following can

be mentioned: higher compatibility of the matrix with the RC or masonry supports than the epoxy adhesives; adaptability to irregular surfaces (especially in the case of masonry supports); sustainability of the installation and of the materials employed.

Glass, carbon, basalt, PBO and natural fibers used in TRMs in the form of composite reinforcing grids have gained principal interest for externally bonded reinforcement of historical and masonry structures, with emphasis to in-plane retrofitting of masonry walls. On the other hand, great potentialities for the repair and retrofitting of existing masonry buildings have been also reported in the case of SRG systems (De Santis and De Felice 2015). For example, retrofit techniques based on the use of high strength steel cords embedded in either polymer (steel reinforced polymer, SRP, systems) or mortar matrix have been successfully employed for strengthening masonry arches (Borri et al. 2009). With regard to existing RC structures, externally bonded SRP and SRG have proved their efficiency in the improvement of the flexural performance of RC beams (Barton et al. 2005; Huang et al. 2005).

Despite the above mentioned beneficial features, the low tensile strength of the inorganic matrix of such systems generally leads to cracking phenomena which commonly affect the bond stress transfer between the inner reinforcement, the matrix and the substrate. In addition, possible tension stiffening effects along with the effects of adding short fibers in the matrix composition should be properly correlated to the macroscopic mechanical behavior of such systems. With regard to this aspect, it should be pointed out that the available information on the test methods and mechanical performances of TRMs/SRGs are still limited. Indeed, the mechanical performances of TRM and SRG systems still deserve suitable investigations with the final aim of providing reliable design parameters such as values design strain, strength and stiffness as well as bond strength and design limitations.

In the present study, the tensile behavior of four types of SRG and TRM systems is investigated and compared among the systems themselves. In detail, the experimental program herein presented consists in two levels of mechanical investigation: *i*) assessment of the mechanical properties of the constituent materials of the TRM/SRG composite systems, *ii*) study of macroscopic behavior of the four TRM/SRG composite system by means of tensile tests.

MATERIALS AND EXPERIMENTAL PROGRAM

Strengthening systems

A total of four types of SRG and TRM strengthening systems were considered in the present experimental investigation. The systems result from the combination of two different inorganic matrices, namely M1 and M2, with three different reinforcing types. In detail, the steel reinforcements are made of unidirectional high strength steel cords characterized by different weights (per square meter) and different cord configurations, namely S210 and S444; the other reinforcing type consists in a glass fiber reinforced (GFRP) grid. The three reinforcing constituents are characterized by the following features:

- **G40** is a bi-directional pre-cured glass fiber reinforced (GFRP) grid of weight of 225 g/m² (0.046 lb/ft²) with yarns spaced of 25 mm (0.98 in.) in both direction (40 yarns per meter of width, Fig. 1a). According to the manufacturer's datasheet, the equivalent thickness is 0.035 mm (1.38x10⁻³ in.) while the nominal values of Young's modulus, the tensile strength and the ultimate strain are $E_f = 72$ GPa (10,443 ksi), $f_{tu} = 45$ kN/m (3,084 lb/ft), and $\epsilon_{fu} = 1.8\%$, respectively. Table 1 summarizes the main properties of G40 reinforcing system according to the manufacturer's datasheet.
- **S210** is a unidirectional steel reinforcement made of 210 galvanized steel cords (with zinc coating) per meter of width. Each cord is made up by six small strands twisted together and containing, in turn, a certain number of twisted individual high strength zinc-coated wires of about 250 μ m of diameter. All the cords are gripped together by means of perpendicular woven textile filaments which allows the cords to be aligned along the longitudinal direction and be tightened together during the loading phase (Fig. 1b). According to the manufacturer's datasheet, the steel used for the cords in the SRG strengthening configuration has an elastic tensile behavior up to failure with tensile modulus of elasticity E_f , tensile strength f_{tu} , ultimate rupture strain ϵ_{fu} equal to 210 GPa (30,458 ksi), 735 kN/m (50,171 lb/ft) and 0.026 mm/mm (in./in.), respectively. The equivalent thickness is 0.374 mm (0.0145 in.), leading to a resisting area of 373.80 mm²/m (0.177 in²/ft) and an areal weight of 2100 g/m² (0.43 lb/ft²). Table 1 summarizes the geometrical and mechanical properties of the adopted steel reinforcement according to the supplier data.

- **S444** is a unidirectional steel reinforcement made of 444 zinc coated galvanized steel cords per meter of width. Each cord is made up by six small strands twisted together and containing, in turn, a certain number of twisted individual high strength zinc-coated wires having a smaller diameter than S210 type (Fig. 1c); consequently, the total number of steel cords per meter width is greater than in the case S210 type. As for S210, all the cords are gripped together by means of perpendicular woven textile filaments which allows the cords to be aligned along the longitudinal direction and be tightened together during the loading phase. The steel used for the cords in the SRG strengthening configuration has an elastic tensile behavior up to failure with tensile modulus of elasticity E_f , tensile strength f_{fu} , ultimate rupture strain ε_{fu} equal to 200 GPa (29,000 ksi), 688 kN/m (47,155 lb/ft) and 0.0129 mm/mm (in./in.), respectively. The equivalent thickness is 0.266 mm (0.0105 in.), leading to a resisting area of 266 mm²/m (0.126 in²/ft) and weight of 2000 g/m² (0.41 lb/ft²). Table 1 summarizes the geometrical and mechanical properties of the adopted steel reinforcement according to the manufacturer datasheet.

The above-mentioned reinforcing types were utilized in combination with two different mortars used as matrix system and described as follows.

- **M1** is a bi-component cement-based mortar with short glass fibers spread in the mix, and containing selected aggregates in a granulometric curve, special admixtures and synthetic polymers in water dispersion. The mortar was obtained by Portland cement and free crystalline silica with a diameter greater than 10 μ m (0,39 mil.) in a percentage by weight between 25% and 50%. The short glass fibers had diameter of approximately 45 μ m (1,77 mil.) and an average aspect ratio equal to 75. The synthetic polymers, that provide an high adhesion strength, are mixed to the powder with a mixing ratio 1:4.
- **M2** is a bi-component premixed pozzolana-based mortar made of hydraulic natural lime, sand, polymeric additives and short glass fibers spread in the mix. In details, the pozzolanic mortar contains glass fibers which provide both enhanced mechanical performances (especially with regard to a reduced early shrinkage cracking) and an effective rheological behavior. The pozzolanic mortar was obtained starting from hydraulic lime and sand (ratio by weight 1:3, with a maximum aggregate size of 1.5 mm (0.059 in.)) added with short glass fibers (ratio by total weight 1:10), then mixed together with latex and water (ratio by weight 1:2). The chopped short glass fibers had diameter equal to 45 micron (1,77 mil.) and an average aspect ratio equal to 70.

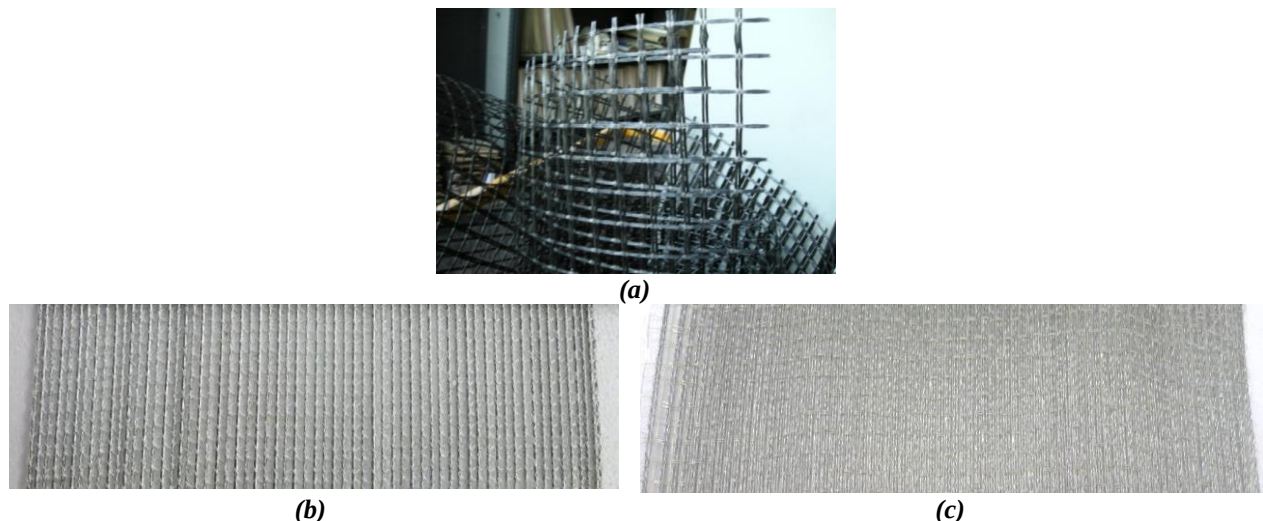


Figure 1—Type of reinforcement: (a) GFRP Grid G40, (b) unidirectional fabric S210, (c) unidirectional fabric S444

Table 1—Properties of the reinforcements investigated in the present study. The data reported refer to the manufacturer's datasheet

	G40	S210	S444
Orientation	Bi-directional	Unidirectional	Unidirectional
Type of fiber	Glass	Galvanized Steel	Galvanized Steel
Weight [g/m²] (lb/ft²)	225 (0.046)	2100 (0.43)	2000 (0.41)
Nr. of yarns/cords per meter	40	210	444
Spacing [mm] (in)	25 (0.98)	4.75 (0.19)	2.25 (0.09)
Area per unit length [mm²/m] (in²/ft)	35.27 (0.0166)	373.8 (0.177)	266.0 (0.126)
Equivalent thickness [mm] (in)	0.035 (0.0014)	0.374 (0.015)	0.266 (0.010)
Tensile strength per meter of width [kN/m] (lb/ft)	45 (3.084)	735 (50.171)	688 (47.155)
Young's Modulus [GPa] (ksi)	72 (10.443)	210 (30.458)	200 (29.000)
Ultimate strain [%]	1.8	2.6	1.29

The four SRG/TRM composite systems investigated in the present study are reported in Table 2 indicating the combination of mortar and reinforcing type employed for each one of them.

Table 2—Type of SRG/TRM tested systems

Specimen name	Mortar type	Reinforcement
SRG-1	M1	S210
SRG-2	M2	S444
TRM-1	M1	G40
TRM-2	M2	G40

The experimental tests were carried out on both reinforcing constituents and SRG/TRM structural systems to be used as external strengthening techniques. In particular, the tests were performed on either the constituent materials, i.e. inorganic matrix and steel/glass reinforcement, and corresponding composite systems of Table 2. The main objective was to highlight the role of different reinforcing types and configurations in the tensile behavior of composite strengthening systems obtained in combination with inorganic mortars. A specific focus was devoted to: the correlation between the mechanical properties of the constituent materials (i.e. mortar and reinforcing types) and the composite SRG or TRM systems, the effects of matrix cracking phenomena, possible transition phases during tensile load conditions, failure modes.

Tests on mortars

The M1 and M2 mortars used to manufacture the strengthening systems indicated in Table 2 were characterized in terms of mechanical properties according to the UNI-EN 1015-11 standard. The mechanical properties were determined after 28 days of curing by means of at least 3 tests on prismatic specimens having dimensions of 40 mm x 40 mm x 160 mm (1.57 in. x 1.57 in. x 6.3 in.). In particular, three points bending tests were firstly performed on prismatic specimens; then, compressive tests were also carried out on the two halves of the tested specimens (with a loading area of 40 mm x 40 mm (1.57 in. x 1.57 in.)). The average values of compressive strength, f_{cm} , and flexural strength, f_{ctm} , Young's modulus in compression, E_c , are reported in Table 3. It is worth noting that both the compressive and flexural strength of M1 (cementitious mortar) were greater than M2 type probably due to the different binder used.

Table 3—Mortar properties

Data	M1	M2
Mortar	Cementitious	Hydraulic-Lime and Eco-Pozzolan
Fiber	Glass	Glass
Compressive strength f_{cm} [MPa] (psi)	24.1 (3.498)	16.05 (2.328)
Flexural strength f_{ctm} [MPa] (psi)	8.01 (1.162)	5.50 (798)
Compressive modulus of elasticity E_c [GPa] (ksi)	10 (1.450)	8 (1.160)

Tests on cords-yarns

Tensile tests were carried out on the fibrous constituents of the SRG or TRM strengthening systems (as indicated in Table 2) by means of a universal servo-hydraulic machine with 500 kN (112,410 lb) of capacity. In particular, single 320 mm (12.60 in.) long steel cords of the S210 and S444 reinforcement type were fixed to the grip system of the testing machine through tubular end-grips (100 mm (4 in.) of length) in which the cord was anchored with an high strength epoxy adhesive (Fig. 2a,b). The actual cross section of each steel cord was approximately 1.78 mm^2 (0.0028 in^2) and 1.18 mm^2 (0.0018 in^2) for S210 and S444 steel reinforcing system, respectively. The tensile test was run under displacement control with a crosshead speed of 0.2 mm/min (0.008 in./min). The tensile axial deformation was derived from the crosshead displacement and referred to a gage length of 120 mm (4.72 in.). In the case of glass fiber reinforced grid belonging to the TRM system, the G40 specimens were cut from the GFRP grid in the form of one single longitudinal yarn with attached half transverse element of the grid (Fig. 2c). From this, single 250 mm (10 in.) long yarn were fixed to the wedge grip system of the testing machine by means of FRP end-grips of 40 mm (1.5 in.) length. The actual cross section of each GFRP yarn was approximately 0.88 mm^2 (0.0014 in^2). The tensile test was run under displacement control with a crosshead speed of 0.2 mm/min (0.008 in./min). As for S210 and S444 cords, the axial deformation was derived from the crosshead displacement and referred to a gage length of 170 mm (6.69 in.). For both specimen types, the following mechanical properties were retrieved: average maximum tensile load, P_{max} , tensile strength, σ_{max} , and average ultimate strain, ε_{fu} .

Tests on composite systems

Tensile tests were carried out on at least five specimens for each of the composite configuration reported in Table 2, i.e. two SRG systems made of steel cord types embedded in the grout type M1 and M2; and two TRM systems with the glass grid embedded in the grout type M1 and M2. All specimens were 400 mm (16 in.) long and 40 mm (1.5 in.) wide and a total thickness of 10 mm (3/8 in.). The specimens were prepared by applying a first layer of mortar of 5 mm (0.19 in.) onto the bottom surface of the mould; then, the reinforcement (G40, S210, and S444) was placed and pressed over the first layer of grout and, afterwards, a second layer of 5 mm (0.19 in.) of grout was arranged above it. It is worth noting that the glass fibers of the TRM-1 and TRM-2 systems were available as pre-coated in contrast to the steel cord reinforcing systems which were dry and not pre-tensioned during the phase of specimen preparation. This circumstance may have led to slight geometrical imperfection in the distribution of the reinforcement within the mortar, having some points of the grid/cords not perfectly co-planar with the mid-plane of the SRG systems. The total number of steel cords or GFRP yarns employed in each SRG or TRM specimen was 8, 9 and 2, for SRG-1, SRG-2 and TRM-1/TRM-2 systems, respectively. At the upper and lower ends of the specimens, a 50x60 mm (2 in. x 2.36 in.) quadri-axial glass fabric was glued with epoxy resin to avoid premature failure of the samples due to local stress concentration near the clamping area of the testing machine.

The tensile tests were carried out by means of a universal testing machine MTS (maximum load 500 kN (112,410 lb)) under displacement control (0.2 mm/min (0.008 in./min)). The specimen was gripped between the clamps of the tensile testing machine by means of hydraulic wedge grips and with a sufficient compressive stress (corresponding to approximately 5 MPa (0.72 ksi) of pressure) to prevent the slippage of the of the reinforcement from the mortar matrix. The local tensile strain was monitored through two linear strain gages (having 4 cm (1.57 in) of gage length) attached onto the mid front and rear surfaces of the mortar matrix while, for global strain estimation, the crosshead displacement was considered. It is worth noting that local strain monitoring by means of linear strain gages aims to characterize the initial slope of the stress-strain curve, i.e. before cracking occurs in the sample, after which the device no longer provides reliable measurements. The outcomes of the tests were elaborated to obtain the maximum tensile load, P_{max} , tensile strength, σ_{max} , and the ultimate strain, ε_{fu} .

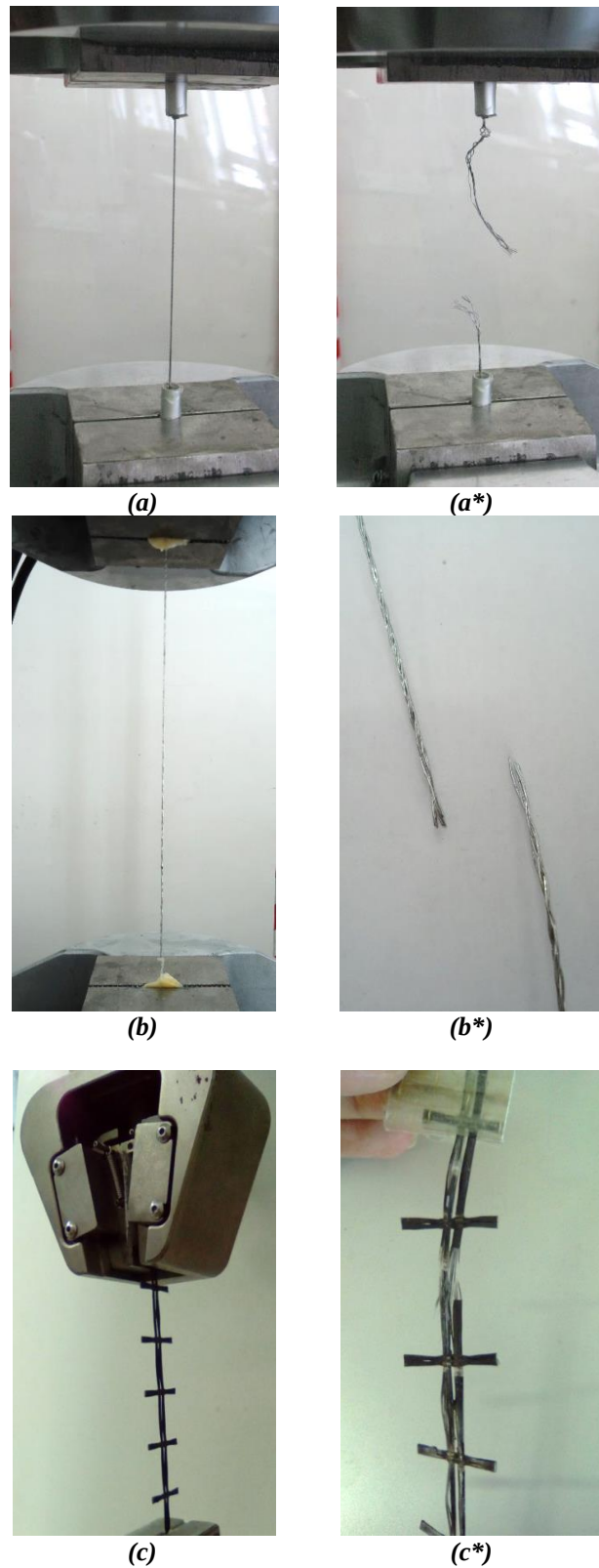


Figure 2—Test on cords-yarns: (a) S210 cord before test, (a*) S210 cord failure; (b) S444 cord before test, (b*) S444 cord failure; (c) G40 yarn before test, (c*) S210 yarn failure.

RESULTS AND DISCUSSION

The results of the tensile tests carried out on coupons made of single cords of steel S210, S444 and single yarns of G40 reinforcement are discussed in terms of tensile stress vs global strain curves which are showed in Fig. 3. All reinforcing types failed in a brittle manner in correspondence of the maximum tensile load carried by the coupon, corresponding to a simultaneous breakage of all single steel filaments or fibers of the steel cord or GFRP yarn, respectively, as depicted in Fig. 2. In the case of steel reinforcements, the tensile behavior was linear elastic up to impending ultimate failure, in correspondence of which, a slightly nonlinear trend (wider in the case of S444 reinforcement type) was exhibited and possibly associated with the twisted configuration of the chords, rather than with the ductility of the material, as found also in previous studies (Ascione et al. 2015). The different configuration of the steel cord types S210 and S444, in terms of total number of cords and inner filament arrangement, was responsible for the different values of maximum tensile load attained and, consequently, for the tensile strength and failure strain measured after the tests (Table 4). In particular, S444 steel type was characterized by average values of maximum tensile load and failure strain of 0.970 kN (216 lb) and 2.08%, respectively, whereas in the case of S210 steel type, the corresponding values resulted equal to 3.77 kN (841 lb) and 3.07%, respectively. The fewer number of steel filaments in each steel cord along with a different twisting arrangement of them were responsible for this discrepancy in the stress-strain and strain to failure behavior of S210 and S444 types. However, on the basis of the values of cord resisting area reported in the previous section, the Young's modulus of the two different steel cord types was almost the same (approximately 200 GPa (29,000 ksi)). Based on the experimental results, the tensile strength of the single cord of S210 and S444 reinforcement type was equal to 2120 MPa (307 ksi) and 822 MPa (119 ksi), respectively whereas the tensile strength per meter of reinforcement width resulted equal to 792 and 431 kN/m (54,284 lb/ft and 29,541 lb/ft), respectively.

On the other side, G40 yarn samples exhibited a linear elastic behavior up to tensile failure, in correspondence of which, a brittle failure was experienced. Failure corresponded to approximately 2.29 % of tensile failure strain and 0.994 kN (223 lb) of maximum tensile load. In Figure 3, it can be observed that the strain to failure is similar for G40 and S444 single yarn/cord; however, G40 exhibited a smoother linear behavior in comparison of S444 system. Based on the test outcomes, the tensile strength of the single yarn of G40 reinforcement type was equal to 1140 MPa (164 ksi), resulting in a tensile strength per meter of reinforcement width equal to 39.7 kN/m (2,721 lb/ft). Even though the maximum tensile load was similar for G40 GFRP yarn and S44 steel cord, the significantly different spacing in the arrangement of the reinforcement, i.e. 25 mm (0.98 in) vs 2.25 mm (0.09 in), led to a remarkable difference of tensile strength per meter of reinforcement width; this aspect should be carefully considered for the design and application of such systems.

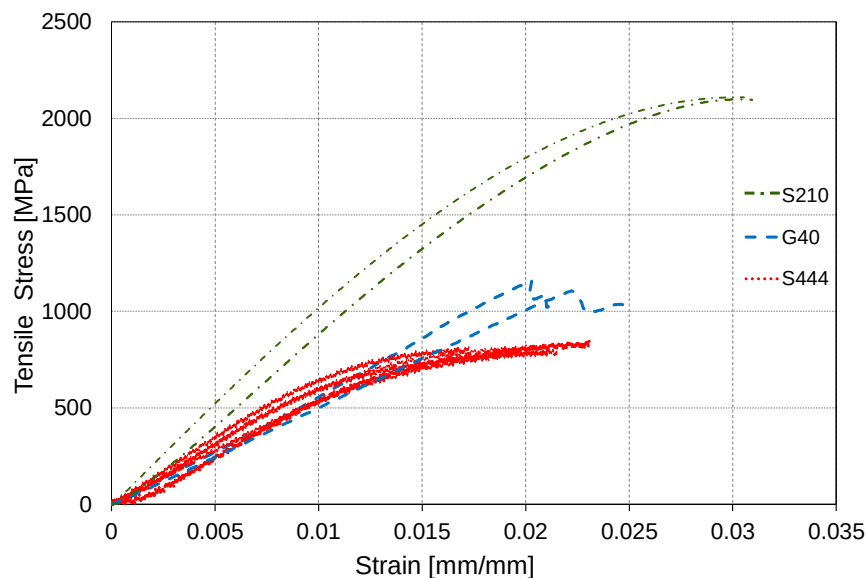


Figure 3—Tensile test on steel cords and glass composite yarns: tensile stress vs strain relationship

Table 4—Results of tensile tests conducted on cords-yarns

Reinforcement type	Average Maximum tensile load P_{max} [N] (lb)	Average tensile strength σ_{max} [MPa] (ksi)	Average ultimate strain ϵ_{fu} [-]	N° of threads/m [-]	Tensile strength of reinforcement per meter of width [kN/m] (lb/ft)
G40	994 (223)	1129 (164)	2.29%	40	39.7 (2.721)
S210	3773 (841)	2120 (307)	3.07%	210	792 (54.284)
S444	970 (216)	822 (119)	2.08%	444	431 (29.541)

Figure 4 reports the experimental tensile stress vs strain curves obtained for the composite systems TRM-1, TRM-2, SRG-1 and SRG-2. The stresses are computed from the tensile load values and divided by the area of the reinforcement used in the sample. It should be pointed out that, reporting the experimental results in terms of tensile stresses may be affected by the dependency on resisting area measurements (or values provided by the manufacturer) since the accurate values are not straightforward to be determined, especially in case of twisted steel filaments. However, the discussion of this issue is not handled in this study but will be investigated by the authors in a different study under progress. The comparison proposed in this section aims to provide a clear identification and direct quantification of the mechanical contribution of the two constituent materials belonging to the SRG or TRM strengthening systems. To this scope, tensile stress vs strain curves obtained for single steel cords and glass composite yarns are also showed in Fig. 6 superimposed to the corresponding curves of the composite systems TRM-1, TRM-2, SRG-1 and SRG-2.

Overall, the mechanical behavior exhibited by the SRG and TRM systems was different in terms of the shape of tensile stress vs strain curves obtained from the tests. In particular, the two TRM systems (i.e. TRM-1 and TRM-2) were characterized by a nonlinear trend of tensile stress as increasing tensile strain. The corresponding curves showed an initial linear branch which ended at approximately 0.1 – 0.2 % of tensile strain. From these values, after a narrow stage up to 0.05 % of strain corresponding to the transition from the uncracked to the cracked regimen, the curves exhibited a change in the slope, leading to a second almost linear branch which was kept roughly constant up to ultimate tensile failure. On the contrary, the two SRG systems did not experience any significant change in the slope of the tensile stress vs strain curves up to ultimate failure. This feature was independent from the steel cord configuration considered (i.e. S210 or S444 in Fig. 1). Global and local strain measurements were also assessed with reference to the initial stage of the stress vs strain curves, i.e. in the uncracked regimen. As a matter of fact, it was not possible to read reliable strain gage measurements in the cases of SRG systems due to early and multiple matrix cracking which affected the integrity of the device. On the contrary, TRM systems allowed for stable strain measurements up to major cracking occurred. Indeed, the stress-strain curves in the initial linear loading stage resulted almost overlapping either using global or local strain measurements (e.g. Fig. 5 for TRM-1 case). This circumstance also highlighted that, for SRG systems, matrix does not have a significant role in terms of initial stiffness, probably due to the very stiff reinforcements which lead to early and multiple matrix cracking.

TRM-1 and TRM-2 achieved an average tensile strength of 1073 MPa (156 ksi) and 1148 MPa (166 ksi), respectively, as reported in Table 5. In terms of tensile loads, the maximum tensile load achieved in TRM-1 and TRM-2 system was equal to 1.50 and 1.61 kN (337 and 362 lb), respectively; these values roughly correspond to the sum of the maximum loads recorded in the single yarn tests. In particular, from Fig. 6a,b it can be recognized that the failure of the two TRM composite systems started nearly in correspondence of yarn breakage with no softening phase in favor of a totally non-ductile failure.

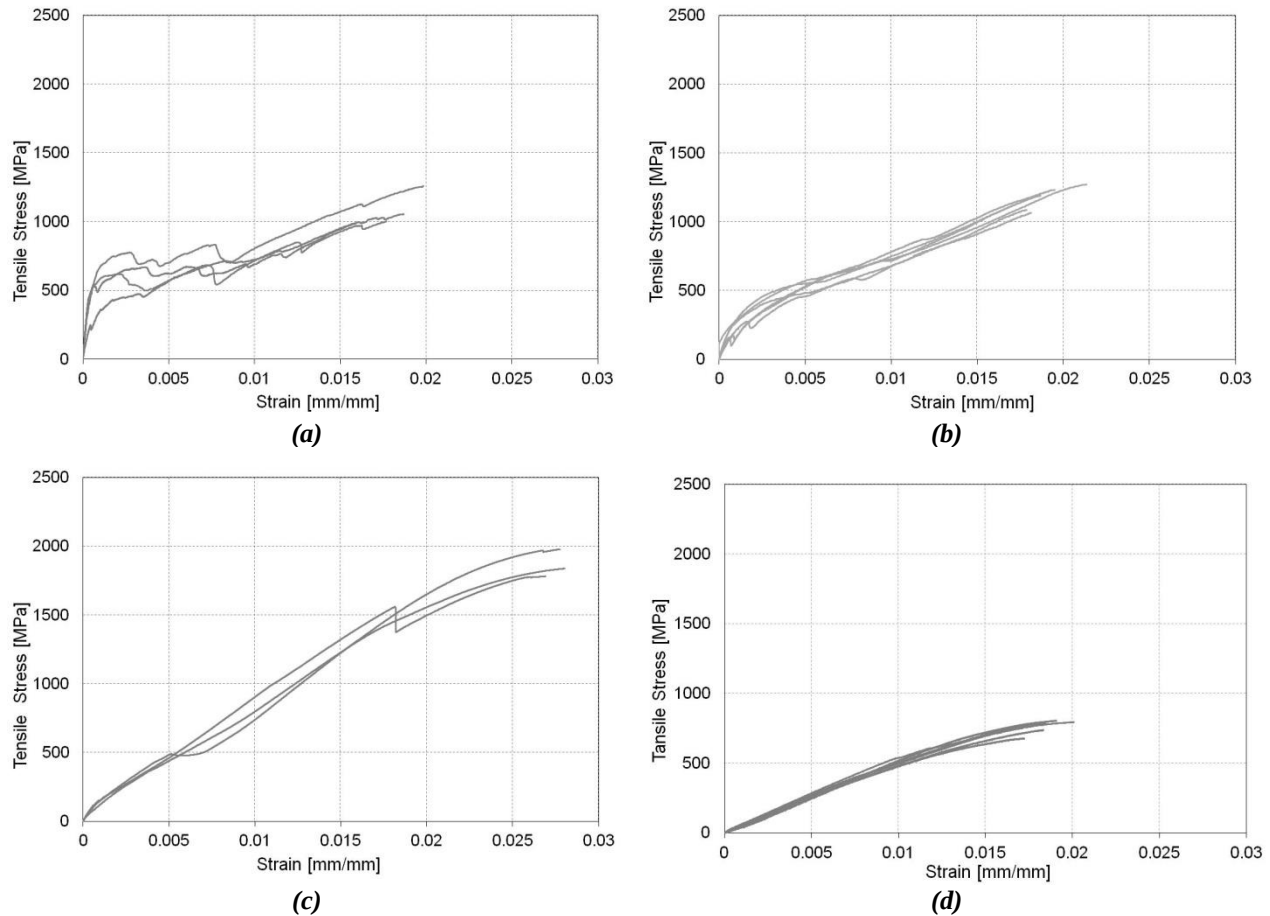


Figure 4—Tensile test on composite systems: tensile stress vs strain relationship: (a) TRM-1; (b) TRM-2; (c) SRG-1; (d) SRG-2

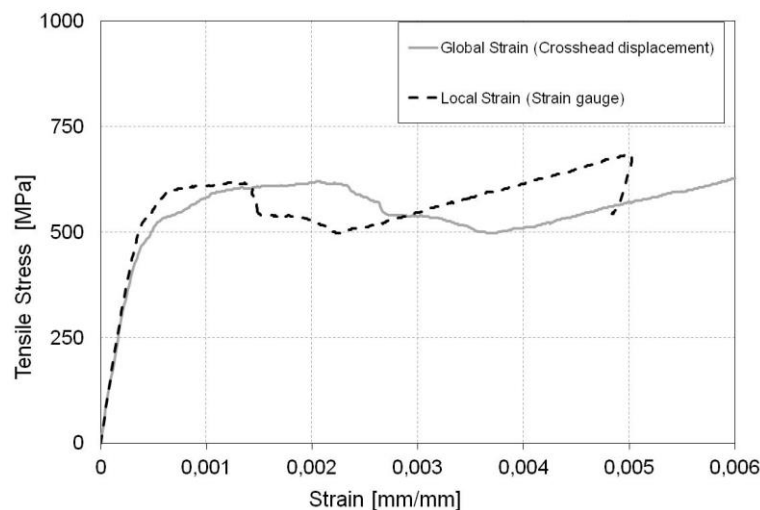


Figure 5—Tensile test on composite systems: different local strain measurements in the uncracked regimen for TRM-1 sample

The same figures also show that the cracking of both mortar M1 and M2 were responsible for a change in the slope of the stress load vs strain curves. In particular, after a transition phase, the tensile strength was attained and the tensile stress-strain curve reproduced a similar mechanical behaviour of the single GFRP yarns. However, slips

between the mortar and the glass fibres probably occurred immediately after the cracking of the matrix, leading to a stiffness of the TRM systems even slightly lower than that of the dry G40 yarns, independently from the type of matrix considered. This phenomenon was probably amplified by the mechanical interaction between the matrix and the GFRP yarn which was characterized by a large free space between the two yarns. In Fig. 7c,d, TRM-1 and TRM-2 specimens are showed, respectively, highlighting the failure of the inner glass reinforced yarns lying on a non-coplanar surface, probably due to the non-simultaneous breakage of the yarns. It is worth noting that yarns failure occurred in correspondence of one large crack in both mortar type M1 and M2, revealing that relative slipping between the mortar and the inner reinforcement probably took place; indeed, strains at failure were reasonably higher than the maximum strain attained in correspondence of tensile breakage of yarns (i.e. 2.29 %).

From the interpretation of the curves obtained, the initial pseudo-linear branch of TRM system can be correlated to the properties of both matrix and reinforcement. Specifically, the resisting load associated with a fully intact matrix (i.e. with a cross section of 40 x 10 mm² (1.5 x 3/8 in.²) corresponds approximately to 900 and 400 N (202 and 90 lb); those values approximately correspond to the data found from direct tensile tests conducted on mortar M1 and M2 (Maddaloni 2017). This consideration is also confirmed by the different flexural strengths obtained for M1 and M2 mortar types and reported in Table 3, which, in such way, affect the first stage of the tensile behaviour of TRM systems.

With regard to SRG-1 and SRG-2 systems, all composite specimens failed mainly due to slippage of the steel cords within the matrix in correspondence of a wide crack opened close to the clamping area (Fig. 7a,b). However, after major slipping phenomena were activated, i.e. in close proximity to the ultimate strain recorded for the two SRG systems, failure of few steel cords was also experienced in both SRG-1 and SRG-2 systems. In the light of the above discussed results regarding TRM-1 and TRM-2 composite systems, major slippage of both steel cord types within the M1 and M2 matrix could be mainly attributed to steel cord-to-mortar mechanical diversity of the constituent materials, being characterized by a high E_f/E_m ratio (i.e. >20), unlike TRM systems for which E_f/E_m ratio ranges between 7 and 9. However, it should be pointed out that the nature of bonding between matrix and reinforcement has an influence on the load carrying capability and, consequently, on the possible slippage at large tensile strains. Also, insufficient clamping force might have played a role in steel cord slippage before attaining tensile rupture. However, considering the significantly high tensile loads achieved in the cases of SRG systems (i.e. between 9 and 30 kN (2023 and 6745 lb)), higher clamping forces may have resulted in a clamping stress greater than the compressive strength of the matrix M1 or M2 employed in the SRG systems. Hence, it is plausible that, for high tensile loads, cracking of the matrix between the clamping wedges may have led to a reduced contribution of the matrix itself to the applied load, promoting slippage of the cords.

Table 5—Results of tensile tests conducted on composite systems

Composite system	Mortar	Reinforcement	E_f/E_m	Average maximum tensile load P_{max} [N] (lb)	Area of reinforcement [mm ²] (in ²)	Average tensile strength σ_{max} [MPa] (ksi)	Average ultimate strain ϵ_{fu} [-]
TRM-1	M1	G40	7.2	1503 (338)	1,4 (0,0022)	1073 (156)	1.80%
TRM-2	M2	G40	9.0	1608 (362)	1,4 (0,0022)	1148 (166)	1.89%
SRG-1	M1	S210	21	27915 (6,276)	14.9 (0,0230)	1865 (270)	2.75%
SRG-2	M2	S444	25	8138 (1,830)	10,6 (0,0164)	765 (111)	1.85%

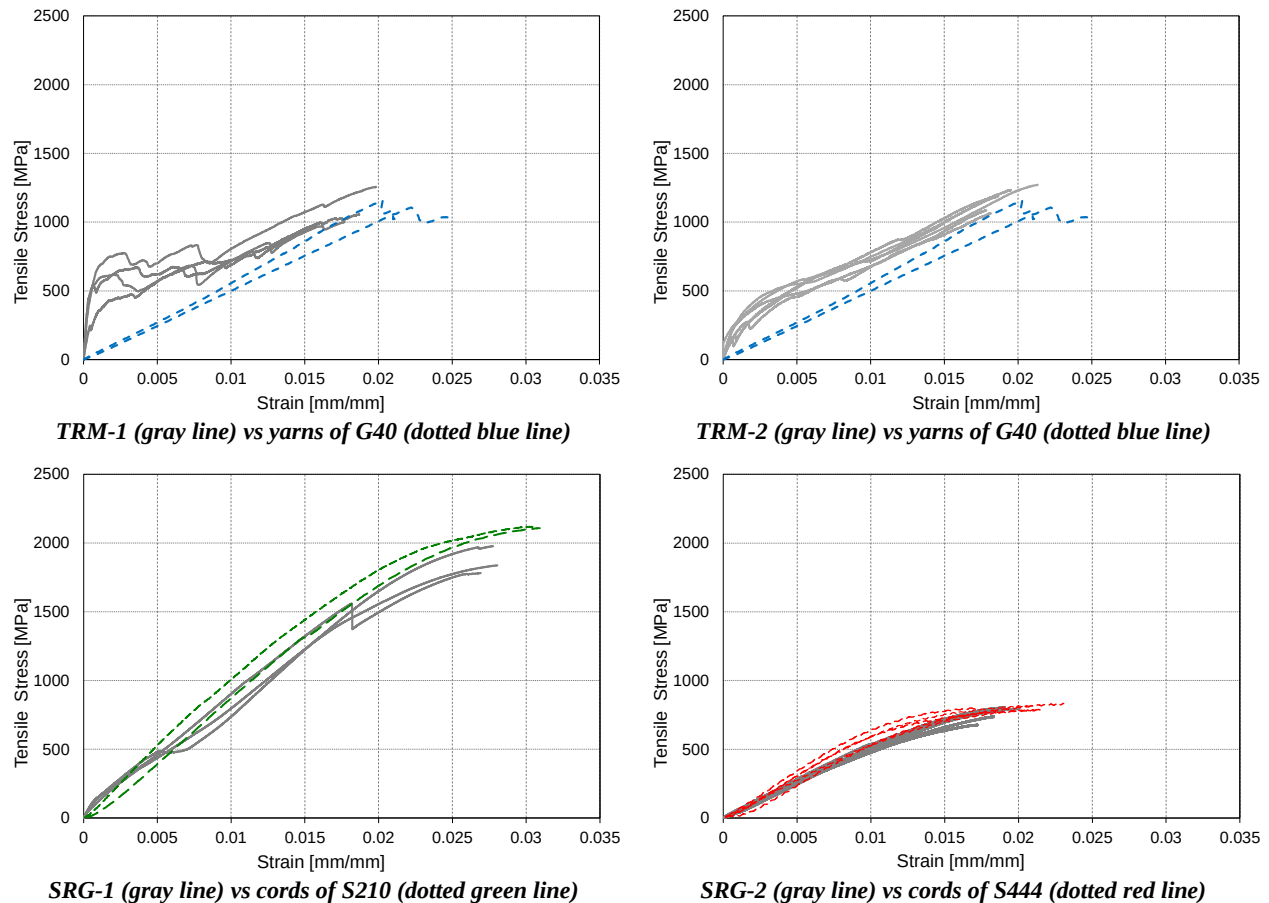


Figure 6—Comparison between tensile stress-strain curves of composite systems and corresponding reinforcement elements in the specimens

As previously mentioned, the tensile stress vs strain curves obtained for SRG-1 and SRG-2 systems were characterized by a negligible initial stage that is associated with an entire resisting area of the composite system. Indeed, irrespective of the S210 or S444 type employed, no clear drops in the tensile stress were recorded, being possibly assimilated to the matrix cracking. Probably, multiple matrix cracks already occurred at the very beginning of the test due to the very different deformation capability of matrix and steel cords. However, it is pointed out that the SRG obtained with matrix type M1 experienced some small drops in the curve at higher strains, irrespective of SRG-2 system. In addition, although the initial part of the tensile stress vs strain curves of SRG-2 samples was perfectly similar to that corresponding to the single S444 steel cords, a slight inner slippage may have occurred for higher loads since a minor change in the slope of the same curves was recorded for strains > 0.005 . It is plausible that the different diameter and arrangement of S444 type along with a weaker matrix M2 had prevented an effective matrix bonding; indeed, the strain before final slippage in M2/SRG-2 systems was lower than in the case of M1/SRG1.

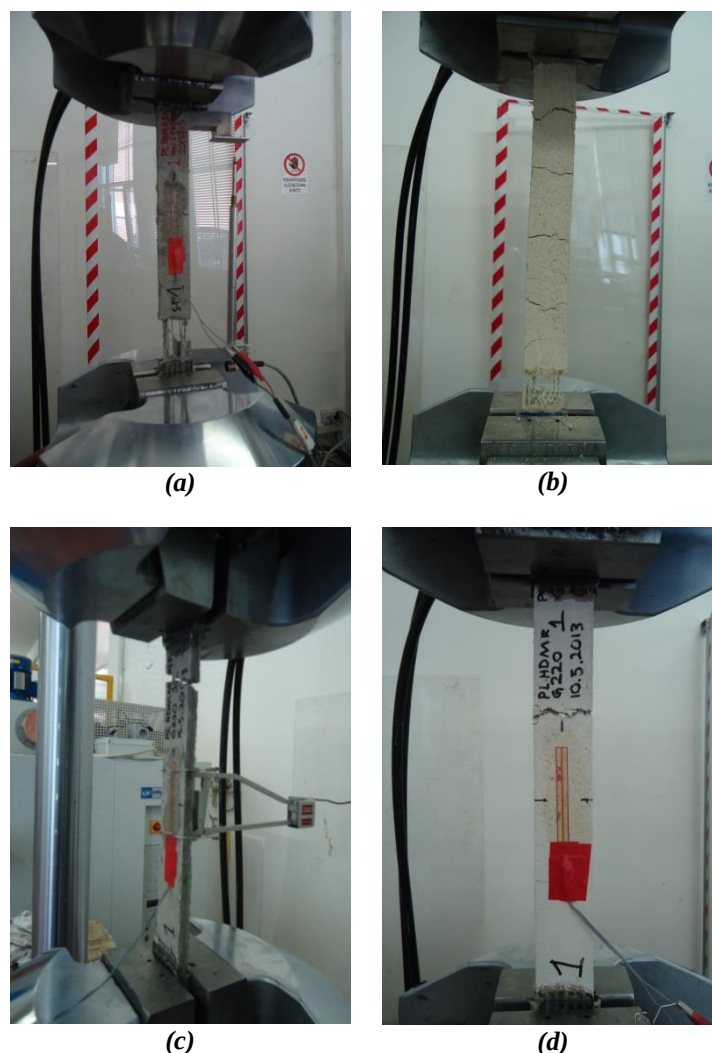


Figure 7—Failure mode of composite systems: (a) SRG-1; (b) SRG-2; (c) TRM-1; (d) TRM-2.

CONCLUSION

The tensile behavior of four different composite systems made of high strength steel cord and glass fiber reinforced grids embedded in two different inorganic matrices was investigated in this study. The influence of the different reinforcing type on the tensile stress-strain response was investigated along with the role of the matrix type.

The overall mechanical behavior exhibited by the SRG and TRM systems was found different in terms of the shape of tensile stress vs strain curves and failure modes. In particular, the two TRM systems exhibited a nonlinear trend of tensile stress as increasing tensile strain with a clear change in the slope of the curve attributable to the matrix cracking. The load corresponding to this transition phase is related to the tensile strength of the matrix itself. In TRM systems, although fiber failure was achieved, yarn-matrix slippage was detected only after the opening of a major crack. On the contrary, the tensile stress vs strain curves obtained for SRG-1 and SRG-2 systems did not show a clear initial stage in which the matrix contributes to the tensile load. Multiple matrix cracks characterizing the deformation pattern along with the large difference between the stiffness of the two materials (steel and mortar) may have played a major role in this initial phase. The failure mode of SRG systems was mainly related to the slippage of the steel cords within the matrix in correspondence of a wide crack being opened close to the clamping area.

In the light of the results obtained, it appears that the prediction of failure due to slippage or steel cord/yarn breakage should be related to the mechanical diversity of the constituent materials. For instance, the stiffness ratio of the two constituent materials could be taken into account into analytical formulations for design criteria.

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Experimental Characterization of Glass and Carbon FRCMs for Masonry Retrofitting

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Synopsis: In the last decade, Fibre Reinforced Cementitious Matrix (FRCMs) became an interesting inorganic alternative to the widespread organic-based Fibre Reinforced Polymers (FRPs). FRCMs are more appealing as retrofitting materials for masonry structures thanks to their generally better compatibility to existing masonry substrates, especially when lime-based matrices are used. In this framework, two FRCMs were tested to evaluate their tensile and shear bond mechanical behaviours, when applied to brick masonry prisms. One FRCM was made of a hydraulic lime mortar coupled with an alkali-resistant glass fibre mesh, while the other was made of a latex-modified cement mortar coupled with a carbon mesh. Tensile tests and single lap shear tests were performed to characterize the relevant strengths of the two FRCMs. The objective was the definition of the basic design parameters for the appraisal of the two FRCMs as strengthening inorganic composite materials for masonry structures. In this paper, the experimental results will be presented and discussed with the definition of the tensile and bond design strengths of the two FRCMs. An example of application with related normalized cost estimations is also provided; it showed that the best trade-off of mechanical performance and cost-effectiveness was given by the carbon mesh FRCM.

Keywords: Fabric Reinforced Cementitious Matrix (FRCM), Textile Reinforced Mortar (TRM), Masonry, Strengthening, Glass mesh, Carbon mesh.

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INTRODUCTION

A continuous maintenance process for existing masonry buildings and Built Heritage (BH) is required to endure the effects of hazardous natural events and mitigate their effects. This can be achieved by promoting activities of restoration and structural repair adopting traditional and innovative materials and intervention techniques (Modena et al. 2011; Tomaževič 2011).

In the last couple of decades, the application of Externally Bonded Fibre Reinforced Polymers (EB-FRPs) has become one of the reference interventions for existing buildings, also in the case of historical structures where dedicated requirements for restoration need to be met (Triantafillou 1998; Shrive 2006; Valluzzi et al. 2014a). The increasing use of epoxy-based FRPs, in the field of civil engineering structural retrofitting, dates back to the Eighties (Meier 1995; Hollaway 2010) when their potential was studied in the case of reinforced concrete elements, whereas the application of FRPs on masonry structures started in the Nineties (Shrive 2006; Valluzzi 2014a). The main advantages of FRPs that fostered their widespread application on existing structures, including also those belonging to the BH, are for instance: their ease and flexibility of application, their fast curing and the related ability in carrying tensile stresses in few hours or days, low weight in comparison to many traditional materials (e.g. steel rods/plates), the high tensile strength and stiffness-to-weight ratio, enhanced fatigue and endurance in aggressive environments (Shrive 2006; Hollaway 2010; Valluzzi et al. 2014a). Nonetheless, FRPs are still showing some compatibility and removability issues with existing and BH masonry structures, mainly due to the adoption of organic matrices (Valluzzi et al. 2014a).

In the last decade, research has moved towards the substitution of organic (e.g. epoxy) with inorganic matrices, like cement or lime-based mortars (Papanicolaou et al. 2007, 2008; Garmendia et al. 2014a; 2014b; de Felice et al. 2014; Triantafillou 2016). More recently, geopolymers or alkali activated materials are also under investigation as possible inorganic matrices for masonry substrates (Garbin et al. 2014; Tamburini et al. 2017). The combination of fibres, usually in the form of meshes, with inorganic matrices is often designated with the acronym of Fibre Reinforced Cementitious Matrix (FRCM), acronym that also covers for cement-free matrices (AC434; ACI 549.4R-13 2013). These composites materials are also often referred to as Textile Reinforced Mortars (TRM) in several publications (Papanicolaou et al. 2007, 2008; Garmendia et al. 2014a; De Santis et al. 2017a; Wang et al. 2017).

The studies focused on the use of FRCMs or TRMs are aimed at limiting some of the shortcomings brought by organic matrices (Katz et al. 2000, Shrive 2006, Hollaway 2010, Tedeschi et al. 2014, Valluzzi et al. 2014a, CNR-DT200 2014), which are related to their lack of vapour permeability (a major concern for built heritage), to their sensitivity to environmental agents (e.g. ultraviolet radiation), to their limited compatibility and removability with existing masonry substrates, to their typically brittle tensile and bond failure and their limited fire performance.

Inorganic matrices have lower bond strength with respect to organic counterparts, but they are generally a more compatible and durable choice for masonry substrates, conform with traditional craftsmanship, are better removable, are water-vapour permeable, are applicable over moist substrates, lack toxic vapours in case of fire, have an improved fire resistance (Papanicolaou et al. 2007, 2008; Orosz et al. 2010; de Felice et al. 2014, Garmendia et al. 2014a, 2014b; Triantafillou 2016). Additionally, inorganic matrices appear particularly suitable for applications to masonry structures, where the bond strength of epoxy matrices can hardly be entirely used due to the inherent low mechanical characteristics of the substrate, and for a better compliance with restoration criteria, especially when cement-free matrices are adopted (de Felice et al. 2014, Valluzzi et al. 2014a).

FRCMs or TRMs systems have been already used for strengthening masonry elements in compression (Bernat et al. 2013; Fossetti and Minafò 2017), for improving the out-of-plane bending strength (Valluzzi et al. 2014b) and the in-plane shear strength (Lignola et al. 2009; Gattesco and Boem 2015), and to increase the capacity of masonry arches and vaults (Garmendia et al. 2011, 2014a, 2015). Nonetheless, few guidelines and publications are currently available for the testing in tension and bond shear of these systems (Larrinaga et al. 2013; Ascione et al. 2015; Ghiassi et al. 2016; De Santis et al. 2017a), for the design of the strengthened elements (ACI 549.4R-13 2013; Garmendia et al. 2014b; Gattesco and Boem 2015) and for the structural modelling (Triantafillou 2016; Wang et al. 2017) of this emerging class of FRCM/TRM retrofitting systems. In this framework, and with the aim of studying the mechanical properties of several FRCM systems to support the drafting of design guidelines, the RILEM technical committee TC 250-CSM “Composites for sustainable strengthening of masonry”, in collaboration with the Italian Industry Association for Composite Materials (ASSOCOMPOSITI), promoted an extensive Round-Robin Test (RRT) able to cover the large majority of the FRCM/TRM systems available in the market and made with glass (Leone et al. 2017), basalt (Lignola et al. 2017), steel (De Santis et al. 2017b), carbon (Carozzi et al. 2017), aramid and PBO (Caggegi et al. 2017) nets or meshes. Within the Round-Robin the influence of several parameters, as the meshes geometry, meshes and matrix mechanical and adhesive properties, test set-up, sample preparation and curing procedure (De Santis et al. 2017a), were evaluated.

Two FRCMs made with glass and carbon meshes were tested at the University of Padova within the scopes of the RILEM TC 250-CSM Round-Robin campaign. Tests aimed at the definition of their tensile and bond shear strengths when applied to masonry prisms made of soft mud clay bricks. The Alkali-Resistant (AR) glass fibre mesh was coupled with a hydraulic lime mortar, whereas the carbon mesh was coupled with a latex-modified cement mortar. The initial test matrix for the Round-Robin campaign was afterwards expanded with additional tensile tests and single lap shear tests with different clamping methods and bonded lengths, respectively. These later tests were performed to obtain clearer tensile curves, suitable for analytical design models, and the definition of a possible minimum effective lengths assuring the tensile failure of the fibre meshes of each FRCMs. The objective was defining the basic design parameters for the appraisal of the two FRCMs as strengthening materials for masonry structures.

In this paper, the experimental results will be presented and discussed with the definition of tensile and bond design strengths of the two FRCM reinforcements. An example of application with related normalized cost estimations is also provided; it showed that the best trade-off of mechanical performance and cost-effectiveness was given by the carbon net FRCM system.

MOTIVATION AND METHODS

Context

The present experimentation moves from the abovementioned Round-Robin Test (RRT) that was aimed at supporting the drafting of the Italian guidelines for the qualification of FRCMs for structural strengthening (De Santis et al. 2017a; Carozzi et al. 2017; Leone et al. 2017), on the basis of a database of tensile tests and bond shear tests of FRCMs applied to masonry elements. Tests herein reported concern an extension of that former experimental program. Several features were kept consistent with the RRT, i.e. materials, test setups, specimen width and thickness, and amount of reinforcement. Conversely, the specimen length in tensile specimens and the bonded length in shear tests were varied to obtain more refined tensile curves, suitable for design models, and the definition of a possible minimum effective length, for both Glass and Carbon FRCMs, able to assure the tensile failure of the fibre meshes.

Experimental plan

The experimental phase consisted of: (i) tensile tests on FRCM coupons; and (ii) shear tests on reinforcements applied to soft-mud brick masonry prisms. Two reinforcements were taken into consideration, one made of a carbon fibre mesh coupled with a cementitious mortar that was also charged with latex, and the second made of a glass fibre mesh

with a hydraulic lime-based mortar. These materials are in accordance with current market trends and also allow to investigate a range of strength and stiffness significantly wide and able to cover the current practice of retrofitting interventions with FRCM systems.

The reference specimen for tensile and shear tests replicated in both cases those of the RRT program, with the only minor difference that specimens were entirely prepared in the lab of the University of Padova. Additional tensile specimens with a reduced length of mortar matrix impregnation and a different clamping system were tested. The different clamping system was aimed at potentially improving the grip between the fibre mesh and the test set-up, thus obtaining more refined tension-stiffening curves from these specimens. Concerning shear tests, the bonded length of 260 mm (10.2 in) adopted for the RRT, corresponding to the thickness of 4 bricks and 4 mortar joints, was progressively and modularly reduced up to 65 mm (2.6 in), equal to the thickness of one brick and one mortar joint (Table 1), in order to empirically estimate the minimum anchorage length. The test plan reported in Table 1 concerned both Carbon and Glass FRCMs, thus resulting in overall 18 tensile tests and 30 shear tests.

Table 1 — Tests matrix for carbon and glass reinforcements.

Dimension	Tensile tests: specimen length L			Shear tests: bonded length L _b		
	L = 400 mm (15.7 in)	L = 260 mm (10.2 in)	L _b = 260 mm (10.2 in)	L _b = 195 mm (7.7 in)	L _b = 130 mm (5.1 in)	L _b = 65 mm (2.6 in)
No. of samples	6	3	6	3	3	3

Specimens preparation and test setups

Test samples were prepared according to good practice principles (Fig. 1 - Fig. 4). The two-component latex-cementitious matrix was mixed in the proportion given by its technical datasheet, without water, whereas the lime matrix was mixed with 24% in weight of water (this was the upper limit provided by the technical datasheet and was adopted to improve workability, thus fostering the impregnation of the glass mesh). Blends were prepared by 1 kg (2.2 lb) powder per time, since it was assumed that keeping constant both quantity and time of mixing would have improved the reproducibility of the mechanical properties. In all specimens, a first layer of matrix 5 mm (0.2 in) thick was applied, paying attention not to smooth excessively the surface to avoid alteration of the interlaminar bond of the mortar. Then the reinforcement mesh was placed and covered with a second layer of matrix, keeping the same thickness of the first one. Demountable steel moulds (Fig. 1) were used to respect the planned sizes of specimens, which were cured for at least one month in a laboratory room at 21°C (70°F) with about 70% of relative humidity.

Tensile test — Reference specimens complied with the AC434 (2013) standard – Annex A. They were prismatic coupons of FRCM 400 mm (15.7 in) long, 40 mm (1.6 in) wide and 10 mm (0.4 in) thick, incorporating one layer of fibre mesh at the middle plane (Fig. 3 left). The carbon mesh included 4 strands, whereas the glass mesh had 5 strands sampled along the warp direction. Before testing and according to ACI 549.4R-13 (2013), two couples of steel plates were bonded with a quick-setting resin at their ends, and tightened with bolts. Plates were carefully glued over an assembly template made on purpose to ensure a correct alignment (Fig. 5). During this operation part of the quick-setting resin was systematically used to glue also the protruding strands from the mortar matrix (Fig. 5), thus promoting as much as possible an efficient anchoring of the FRCMs to the steel plates. Specimens were then connected to the test machine by two Ø20 mm (Ø0.8 in) steel pins passing through the plates and clevis eyes, which provided some out-of-plane and in-plane adjustments for a correct alignment with the universal machine axis (Fig. 5 centre and left). The additional specimens differed in the length impregnated by the matrix (Fig. 3 right) that was of 260 mm, shorter than the 400 mm used for the specimens of the RRT (Fig. 3 left). They were produced with the bare mesh protruding at both ends, which was directly glued to the steel plates of the tensile test set-up. The goal behind those shorter specimens was to clamp directly with resin the protruding bare fibres, in the attempt of overcoming issues related to the development of the anchorage at the ends, thus promoting as much as possible a tension-stiffening like behaviour.

The test machine was a 25 kN (5600 lbf) universal electro-mechanic frame. Tests were performed in displacement control, with a rate of the movable loading beam of 0.3 mm/min (0.012 in/min) for the first phases, then progressively increased up to 1.2 mm/min (0.047 in/min) in the last phases when number and location of crack were stable. Load and displacements were monitored by the machine load cell and by two potentiometric transducers placed with a gauge length of 200 mm (7.9 in) along the axis on both the wider surfaces of the specimen. Data were recorded by an external electronic acquisition system with a rate of 10 Hz.

Shear test — Specimens (Fig. 2 and 4) were prepared according to the RRT directions, i.e., by applying the FRCM reinforcement to masonry prisms made of 5 solid soft-mud bricks and 4 mortar bed joints. The matrix covered the fibre reinforcement only in correspondence of the bonded area. The mesh outside the bonded area was left bare (not impregnated with the inorganic matrix) and an extra length of fibre mesh was secured to connect the specimens to the testing universal machine. The bond started at a distance of 30 mm (1.2 in) from the edge of the prism, as usually performed to prevent possible wedge failures (Mazzotti et al. 2008). Thickness and width of the FRCM, as well as the number of fibre strands, were the same of tensile specimens. The bonded length varied from the reference one, 260 mm (10.2 in), to the shortest one, 65 mm (2.6 in).

The adopted single lap shear test set-up (Fig. 6 left), already used in former RRT campaigns (Valluzzi et al. 2012; Panizza et al. 2012) was designed for a universal electro-mechanic and software-controlled machine, coupled with an electronic data acquisition system. Two steel plates 50 mm thick (2.0 in) were mutually connected by four threaded steel bars, two of them fastened to the lower machine's head. Specimens were inserted between those plates, the upper working as a contrast. The contact between masonry prisms and steel plates was improved with thin high-strength steel plates forced into interstices. The bare mesh was glued with a quick-setting resin to the loading steel element, which was connected to the upper machine's head through a ball joint. Three potentiometers, two for the loaded end (LE), at the beginning of the bare mesh and one for the unbonded end (UE) were applied to monitor displacements between the substrate and the FRCMs. For the bonded length of 260 mm (10.2 in) and 195 mm (7.7 in) (Fig. 4), a fourth potentiometer was placed at the middle length for the specimens to possibly record intermediate relative displacements. The potentiometers configuration for a bonded length of 260 mm (10.2 in) is shown in Fig. 6 right.

Tests were controlled by the displacement rate of the transverse movable beam, set to 0.3 mm/min (0.012 in/min) at the beginning, then progressively increased up to 1.2 mm/min (0.047 in/min) as for tensile tests. The data acquisition rate was 10 Hz.

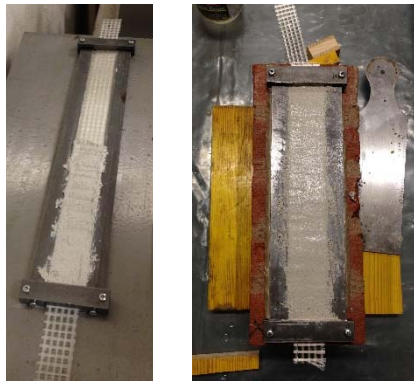


Figure 1 — Preparation of specimens for tensile test (left) and shear test (right) with the glass mesh

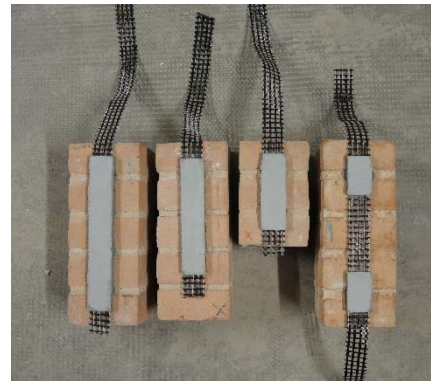


Figure 2 — Comparison of some shear test specimens with various bonded lengths with the carbon mesh

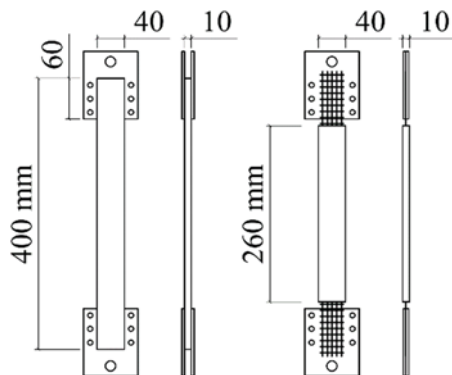


Figure 3 — Specimens for tensile test.

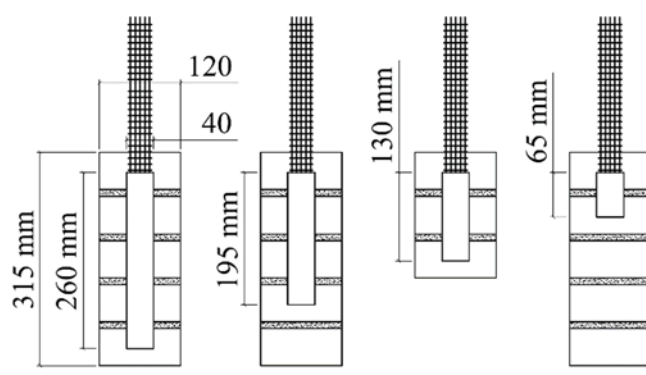


Figure 4 — Specimens for shear test.



Figure 5 — Detail of the assembly template for tensile tests (left), detail of the bonding with quick-setting resin (centre-left) and mounted test set-up (centre-right and right)

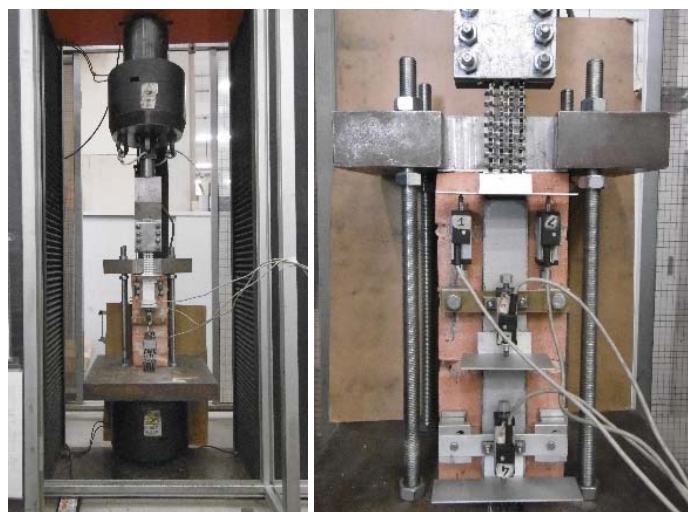


Figure 6 — View of the shear test set-up (left) and detail of the displacement transducers (right)

PROPERTIES OF THE MATERIALS

Reinforcements

Fibre meshes — Two bidirectional balanced meshes were used, one made of carbon (CR), and the second made of alkali-resistant glass (GL), both composed of dry strands of fibre without any partial-impregnation with polymers and without any coating. The average measured centre-to-centre spacing of CR strands was about 9.3 mm (0.37 in) in both directions, while GL mesh, although balanced as well, presented strands spaced about 7.7 mm (0.30 in) in the weft direction and about 6.7 mm (0.26 in) in the warp one, so that the area of a single strand was different according to its direction. Meshes came without any declared value of equivalent thickness, t_f , which was estimated (Eq. 1, where ρ_A is the declared net area density and ρ_V is the fibre density) on the basis of the average density value of carbon and glass fibres collected by a literature (Callister 1994; Mazumdar 2001; CNR-DT200 2013) and market survey on several similar products (10 for glass and 20 for carbon). An experimental validation of density values was carried out according to the main principles of ASTM D792-13 (2013), resulting in an underestimation of about 10% that can be reasonably explained by an excess of air entrapped among fibres, which should have reduced the weight measured by immersion due to the increased buoyancy. The main properties of fibre mesh, declared by the manufacturer or experimentally measured, are listed in Table 2. Tensile tests were carried out on two-strands samples impregnated

with epoxy resin, in order to avoid premature failures and promote an even stress distribution among the two strands. The contribution of resin was detracted from the obtained results via linear calculations by us the linear behaviour of both materials, having measured the average cross-section of the resin and imposed strain compatibility of the two materials up to failure. The mechanical properties of the epoxy resin were measured in Valluzzi et al. (2012).

$$t_f = \frac{1}{2} \cdot \frac{\rho_A}{\rho_V} \quad (1)$$

Table 2 — Properties of bidirectional balanced fibre meshes.

Mesh	declared area density		assumed density		calculated eq. thickness		Declared tensile strength ($f_{t,decl}$)		measured tensile strength ($f_{t,exp}$)			declared tensile elastic modulus	
	g/m ²	lb/ft ²	g/cm ³	lb/in ³	mm	in	N/mm ²	ksi	N/mm ²	ksi	CoV	kN/mm ²	ksi
CR	200	0.041	1.80	0.065	0.056	0.0022	2500	362.6	2854	413.9	7.9%	230	33.4·10 ³
GL	220	0.045	2.60	0.094	0.042	0.0017	1300	188.5	1260	182.7	4.7%	65	9.4·10 ³

Matrices — The carbon mesh was coupled with a bi-component premixed latex-modified cement-based medium-grade mortar containing polyacrylonitrile (PAN) short fibres, herein labelled CM, while the glass mesh was embedded in a premixed cement-free pozzolanic lime-based mortar (LM), which presented a certain amount of short fibres, likely PAN, to reduce shrinkage cracking. Their main properties, provided by the manufacturer or experimentally measured according to EN 1015-11 (2006), are reported in Table 3. LM compressive strength was found lower than the declared value, probably due to the slightly greater amount of water (yet allowed by the mortar datasheet) added to increase its workability in the preparation phase.

Table 3 — Properties of matrices.

Matrix	declared compressive strength		measured compressive strength			measured bending strength			declared elastic modulus	
	N/mm ²	ksi	N/mm ²	ksi	CoV	N/mm ²	ksi	CoV	kN/mm ²	ksi
CM	> 38	> 5.51	41.3	5.99	1.9%	9.8	1.42	4.1%	≥ 15	≥ 2.2·10 ³
LM	18	2.61	14.6	2.12	1.8%	2.2	0.32	3.4%	≥ 16	≥ 2.3·10 ³

Masonry

The reinforcement of shear bond test specimens was applied to masonry prisms made by 5 overlaid solid soft-mud clay bricks, with sizes (length × width × thickness) 250 mm×120 mm×55 mm (9.84 in×4.72 in×2.17 in), having 4 bed joints of hydraulic lime-based mortar, about 15 mm (0.59 in) thick. Prisms were prepared according to the good practice principles: bricks were saturated in advance with water, but avoiding any superficial water film, to prevent absorption from mortar joints; proper moulds were used to assure, to a feasible extent, a regular and uniform shape; curing was carried out in a moist environment for at least one month. Before the reinforcement application, each prism was cut in two equal parts along the vertical axis, so as to double the available samples and keep geometrical proportions consistent with previous experimental campaigns (Valluzzi et al. 2012; Panizza et al. 2012), which involved the same substrate materials. Bricks were tested in compression (EN 772-1 2015), bending and splitting (UNI 8942-3 1986). Mortar was characterized according to EN 1015-11 (2006). Additional masonry prisms, with the same geometry and the same materials used in the present campaign, were tested in monotonic compression and in indirect tension. The measured properties are listed in Table 4; the elastic moduli in compression refer to secant values between 0-30% of the strength, for bricks and mortar, and between 0-40% of the strength, for masonry prisms. Further details can be found in Panizza et al. (2012).

Table 4 — Measured properties of masonry materials.

material	compressive strength			bending strength			splitting strength			elastic modulus		
	N/mm ²	ksi	CoV	N/mm ²	ksi	CoV	N/mm ²	ksi	CoV	kN/mm ²	ksi	CoV
bricks	19.8	2.87	3%	3.7	0.54	4%	2.5	0.36	11%	5.8	0.84·10 ³	7%
mortar	2.6	0.38	11%	0.8	0.12	25%	0.4	0.06	7%	5.5	0.80·10 ³	5%
masonry	8.2	1.19	21%	—	—	—	1.1	0.16	4%	2.1	0.30·10 ³	18%

RESULTS AND DISCUSSION

Tensile tests

Experimental results — During test execution, the applied load and the displacements at the two opposite wider surfaces of the sample were recorded by an electronic data acquisition system, while number and position of subsequent cracks were annotated. According to AC434 (2013), RILEM TC 232-TDT (2016) and De Santis et al. (2017a) displacements were referred to their gauge length – about 200 mm (7.9 in) – and the average value was assumed as the strain, whereas the load was divided by the fibre area of the mesh, which was 2.08 mm^2 ($3.2 \cdot 10^{-3} \text{ in}^2$) for CR and 1.41 mm^2 ($2.2 \cdot 10^{-3} \text{ in}^2$) for GL, to derive the nominal tensile stress. As reported by several authors (Garmendia et al. 2011; Larrinaga et al. 2013; de Felice et al. 2014; Carozzi et al. 2017; Leone et al. 2017), stress-strain curves showed a wide variability with frequent load drops in correspondence to cracking. Nevertheless, they can be approximately reduced to a tri-linear trend, typical of a tension-stiffening like behaviour, having a first branch (uncracked mortar matrix) almost vertical (the contribution of the mortar is not explicitly accounted for) then a second branch with a negligible slope (development of cracks) and, finally, a third branch (stable crack pattern) with slope comparable to the fibre elastic modulus that lasted up to failure. Various failure modes were observed and classified according to external visible damages. Cracking on the mortar matrix (types A-C) often entailed the inner pull-out/partial tensile rupture of the strands as shown in Figure 7 and Figure 8. Failure modes E and F were related only to the additional shorter specimens with steel plates directly bonded to the mesh:

- A) cracking of the matrix close to the anchoring plates;
- B) major cracks of the matrix close to the anchoring plates, with minor cracks distributed along the specimen;
- C) cracks distributed along the specimen;
- D) cracks distributed along the specimen, with a neat tensile failure of the mesh;
- E) failure of the fibres outside the matrix, with pull-out and/or tensile breaking of the mesh;
- F) cracks distributed along the specimen, with pull-out and/or tensile failure of the fibres outside the matrix.

Experimental values, in terms of maximum load (P_{max}), maximum average strength (f_{FRCM}) and characteristic strength ($f_{FRCM,k}$) – i.e. 5-percentile according to EN 1990 (2010), strain in correspondence to P_{max} (ϵ_{Pmax}), and coefficient of effectiveness, herein defined as the ratio between f_{FRCM} and either the declared ($C.E._{decl}$) or the experimental ($C.E._{exp}$) fibre strength, are listed in Table 5. Failure modes, average number of cracks and average crack distance, referred to the length between the two extreme fissures, are given in Table 6. The type of FRCM is reported using the abbreviation of the fibre mesh plus the abbreviation of the matrix, as in the previous sections.

Failure was eventually related to the fibre mesh, but in most cases with a combination of internal slippage (fibre pull-out) and partial tensile rupture. With reference to the maximum stress, shorter samples showed results similar to those of longer ones in the case of CR+CM. The performance of shorter samples was sensibly lower and more scattered in the case of GL+LM. This can be reasonably related to the weaker and less tough glass fibre mesh that was not able to undergo the likely uneven load distribution among the strands, outside the matrix. The tests showed that dimensions and clamping systems have a marked influence on the experimental outcomes. The best tensile test results were obtained with the longer specimens fully impregnated on the entire length. Reasonably, a longer load introduction area as per RILEM TC 232-TDT (2016) might enhance the experimental outcomes.

Apart the short GL+LM specimens, the average tensile strength was in between 40-43% of the measured strength of the mesh alone (see Tab. 2). The measured strain was comprised between 0.5 and 1.1%, with a moderate Coefficient of Variation (CoV), equal to 17%, as exhibited by longer samples (400 mm), differently than the shortest (260 mm). The 200 mm gauge length for the potentiometers was sufficiently long to generally assure the presence of at least 3 or 4 cracks within the monitored area, thus providing a sufficient representativeness of the cracking process during the second branch of the tri-linear trend.

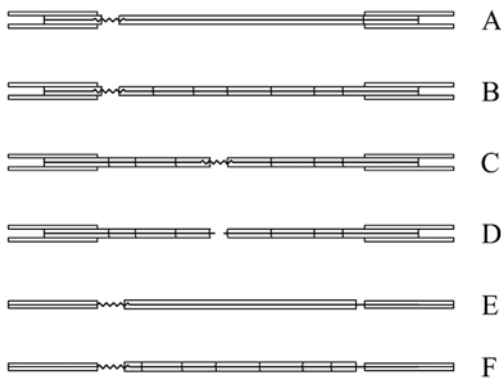


Figure 7 — Failure modes of tensile tests

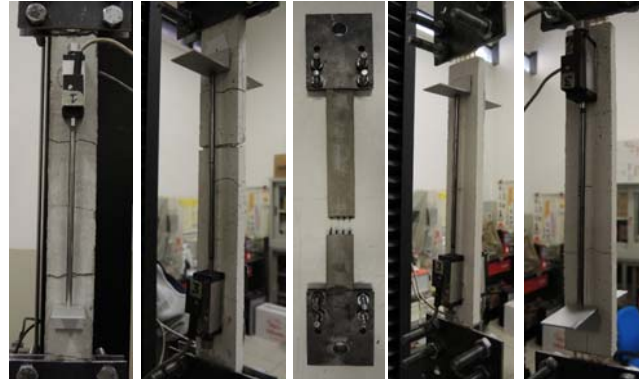


Figure 8 — Observed failure modes for tensile tests

Table 5 — Main results of tensile tests.

FRCM	length L		P _{max}			f _{FRCM}		f _{FRCM,k}		ε _{Pmax}		effectiveness	
	mm	in	N	lbf	CoV	N/mm ²	ksi	N/mm ²	ksi	%	CoV	C.E. _{decl}	C.E. _{exp}
CR+CM	400	15.7	2360	531	9.8%	1135	165	938	136	0.8%	17%	45%	40%
CR+CM	260	10.2	2527	568	11.3%	1215	176	972	141	0.5%	74%	49%	43%
GL+LM	400	15.7	721	162	9.6%	513	74	424	61	1.1%	17%	39%	41%
GL+LM	260	10.2	502	113	39.9%	357	52	105	15	0.7%	36%	27%	29%

Table 6 — Failure modes and crack spacing for tensile tests.

FRCM	length L		failure modes	avg no. of cracks		avg. crack distance		
	mm	in		n _{cr}	CoV	mm	in	CoV
CR+CM	400	15.7	B (2); D (4)	5	34%	77	3.0	9.8%
CR+CM	260	10.2	C (all)	5	0%	41	1.6	11.3%
GL+LM	400	15.7	C (all)	7	35%	56	2.2	9.6%
GL+LM	260	10.2	E (1); F (2)	1	43%	—	—	39.9%

Analysis and application of the ACK predictive model — The model presented by Aveston and Kelly (1973), adopted also by Larrinaga et al. (2013), was applied to analyse the performed tensile tests. The predictive model assumes an elastic homogenization between the mechanical properties of the mortar matrix and of the fibre mesh for the first branch (uncracked phase), then it provides an estimation of the length of the second branch with a negligible slope (representing the simplified development of cracks), and lastly it gives an estimation of the third branch (stable crack pattern). For this last branch, a reduction coefficient for the apparent modulus of elasticity of the fibres of 0.61 and a reduction coefficient for the mesh strength of 0.43 were adopted, respectively. These reduction coefficients were derived from the current literature (Soranakom and Mobasher 2010; Larrinaga et al. 2013) covering the tensile behaviour of FRCM/TRM systems made by inorganic matrices and dry fibre meshes without any partial-impregnation with polymers and without any coating, and of the present tests (Tab. 5). In fact, this type of mesh typically shows a partial interaction with the inorganic matrix and a telescopic-like failure type in tension, which can be converted into an apparent modulus of elasticity and tensile strength smaller than those of the sole fibres (Häußler-Combe and Hartig 2006; Hegger et al. 2007; Soranakom and Mobasher 2010; Larrinaga et al. 2013). In fact, the tensile test showed that the theoretical values of the modulus of elasticity, ultimate deformation and tensile strength for the third branch of both FRCMs are smaller than those of the dry fibres.

The comparison between the experimental data and the analytical model are presented graphically in Figure 9 and Figure 10, and numerically in Table 7, where σ_1 and σ_3 indicate the tensile stress of the horizontal branch (plateau) and the ultimate tensile stress, respectively; ϵ_1 , ϵ_2 and ϵ_3 stand for the strain at the beginning and the end of the plateau, and the ultimate strain in correspondence of σ_3 , respectively; E_1 and E_3 are the slope of the first and the last branch,

respectively. For the Glass mesh FRCM the comparison is fairly good, whereas for the Carbon reinforcement the ACK model can properly describe the trend of the experimental data, but with an overestimation of the performance for the second branch, which later provides the offset between the experimental and model data for the third branch. The difference noted for the Carbon mesh FRCM can be reasonably due to the latex additive in the cement mortar, which influenced too positively the mechanical performance of the mortar samples, but not the tensile specimens. In fact, the calculation of σ_1 relies on the bending tensile strength of CM mortar (Table 3), which is very likely and overestimation of the its direct tensile strength.

Table 7 — Comparison of experimental and analytical parameters for tensile behaviour.

	σ_1		ϵ_1	ϵ_2	σ_3		ϵ_3	E_1		E_3	
	N/mm ²	ksi	%	%	N/mm ²	ksi	%	kN/mm ²	ksi	kN/mm ²	ksi
CR+CM (experimental)	531	76.9	0.02%	-	1148	166.5	0.74%	2414	350·10 ³	-	
CR+CM (ACK model)	684	99.1	0.04%	0.21%	1082	157.0	0.49%	1952	283·10 ³	141.0	20.5·10 ³
GL+LM (experimental)	216	31.3	0.01%	0.37%	513	74.3	1.10%	1976	287·10 ³	39.9	5.8·10 ³
GL+LM (ACK model)	204	29.6	0.01%	0.21%	563	81.6	1.11%	2616	379·10 ³	39.9	5.8·10 ³

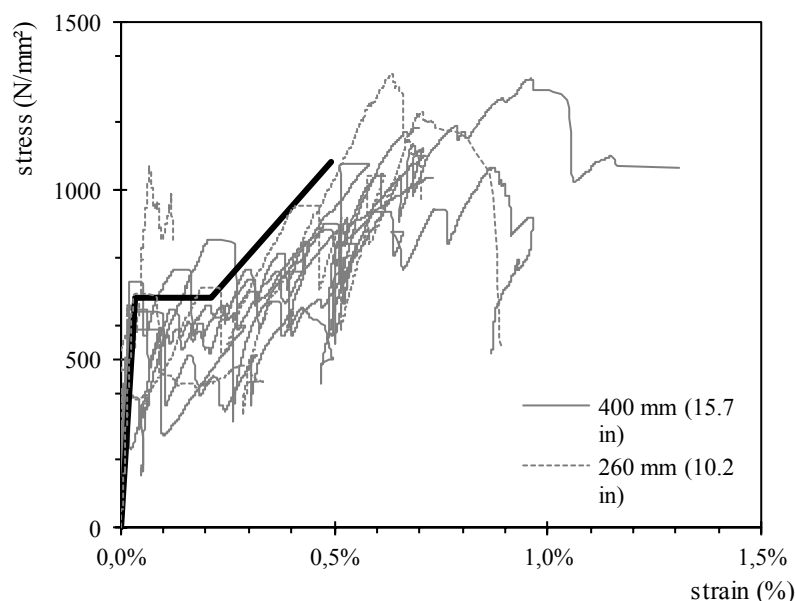


Figure 9 — Experimental and analytical stress-strain curve for carbon CR+CM FRCMs

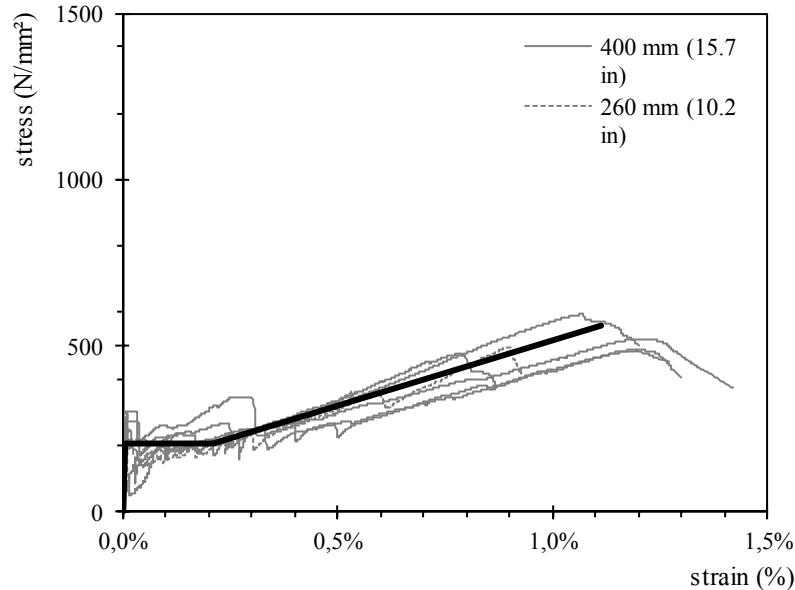


Figure 10 — Experimental and analytical stress-strain curve for glass GL+LM FRCMs

Shear Tests

The applied load and the relative displacements between substrate and reinforcement were monitored with an external data acquisition system, including either 3 or 4 potentiometers, namely at the loaded end in correspondence of the beginning of the bare mesh (2 transducers), one at the unloaded end and one halfway of the bonded length. This last potentiometer was used only for the bonded length of 260 mm (10.2 in) and 195 mm (7.7 in). As expected for a rigid quasi-frangible material, displacements of the matrix upper layer were imponderable, therefore only those at the loaded end of the bare mesh were subsequently taken into account. Despite the precautions in positioning both the transducers and the contact plate fixed to the mesh, and the care in keeping the measuring position as close as possible to the plane of the mesh, any slight variation of the alignment of the fibre bundles during the test caused issues in the load-displacement curve, especially in the first phases with low levels of load. For this reason, values of displacements were considered not significant and, therefore, they are not reported.

Various failure modes were observed, as classified in Figure 11:

- A) failure at the interface between substrate and matrix;
- B) failure at the interface between matrix and fibre mesh, with spalling of portions of the upper layer of mortar;
- C) spalling of the upper layer of mortar close to the loaded end, with pull-out and/or tensile failure of the mesh;
- D) tensile failure of the mesh, at the beginning of the bare strands;
- E) tensile failure of the mesh, outside the matrix;
- F) pull-out of the fibre mesh inside the matrix.

A selection of representative failure modes is shown in Figure 12, while Figure 13 displays some examples of the obtained load-displacement curves. The average maximum strength was plotted against the bonded length, for both reinforcements, in Figure 14, and compared to that obtained from tensile tests. The failure mode (P_{max}), the average bond strength of FRCM reinforcement ($f_{ST} = P_{max} / \text{cross section area of the mesh}$), the characteristic bond strength of FRCM reinforcement ($f_{ST,k}$), and the effectiveness ratios for the declared tensile values of the mesh ($C.E._{decl} = f_{f,decl} / f_{ST}$), for the average experimental tensile strength of the mesh ($C.E._{exp} = f_{f,exp} / f_{ST}$) and the average tensile strength of the FRCM system ($C.E._{FRCM} = f_{FRCM} / f_{ST}$) are given in Table 8.

It is worth noting that, although the detachment of the whole reinforcement from the substrate occurred twice (only with GL+LM, Fig. 11 – Type A) the predominant failure mode was the fibre mesh tensile rupture. The Type A failures can be reasonably attributed to a gentle application of the mortar that was not sufficiently pressed against the masonry substrate. Type A failure was not observed for CR+CM specimens, neither in shorter bonded lengths, where the pull-out of the strands was obtained. Unfortunately, two CR+CM samples with $L_b = 130$ mm (5.1 in) failed before the test,

during the handling for applying transducers and positioning them into the test machine. Nonetheless, based on the overall results, the effective length (intended as the minimum bonded length able to mainly force the fibre mesh tensile failure) can be reasonably placed in between 100 and 150 mm (4 and 6 in) for CR+CM. Whereas, for GL+LM an effective length between 50 and 100 mm (2 and 4 in) can be assumed, due to the lower strength of glass fibres. The tensile stress within the glass mesh for the shear specimens was in average 27% smaller than the tensile strength of the glass mesh used in the tensile specimens. This difference can be reasonably related to a different performance of the glass mesh sample used to produce the GL+LM specimens. In fact, also for the shortest bonded length the tensile failure of the glass mesh was obtained. Both effective length for CR+CM and GL+LM FRCMs are longer than the anchoring area of the tensile tests (Fig. 3). This apparent discrepancy is explainable with the extra positive effect of the quick-setting resin grabbing the protruding strands at the end of the tensile coupons (Fig. 5). Consequently, the bond strength of FRCMs systems can be directly compared with their tensile strengths. Nonetheless, this aspect further confirmed the importance of the appropriate geometrical definition of the load introduction area, as also stressed in the RILEM TC 232-TDT (2016).

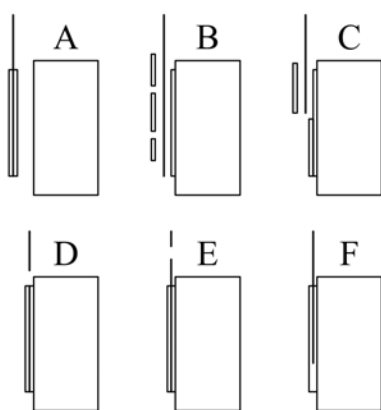


Figure 11 — Failure modes of shear tests

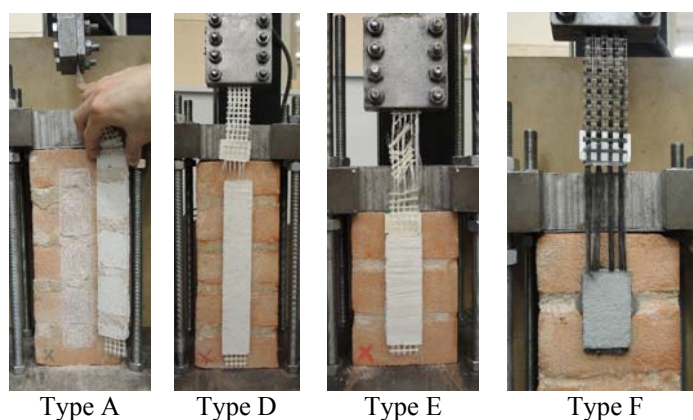


Figure 12 — Failure modes of tensile tests

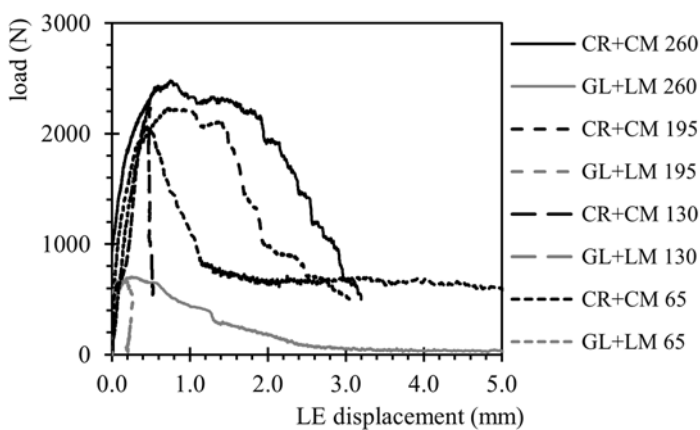


Figure 13 — Representative load-displacement curves

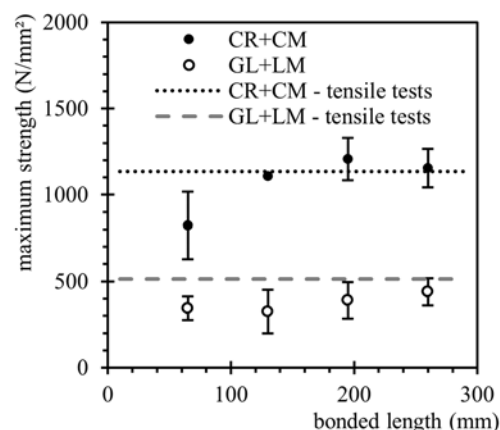


Figure 14 — Shear strength vs. bonded length