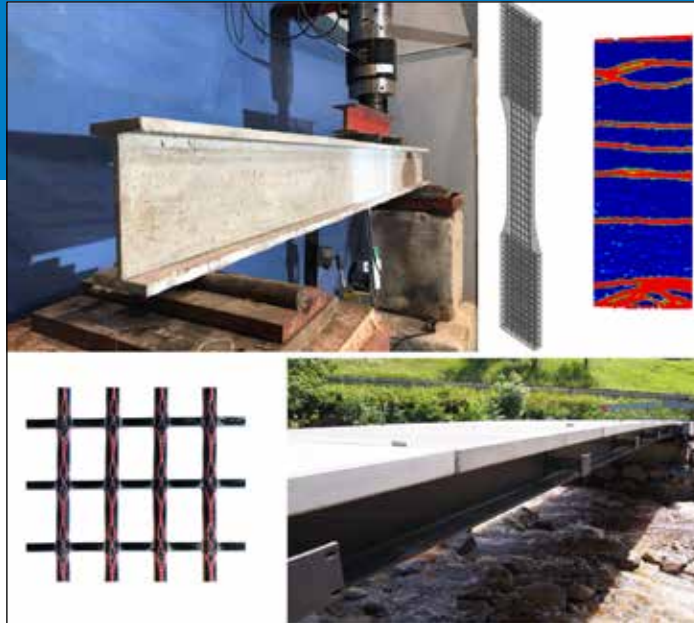


An ACI Technical Publication

SYMPOSIUM VOLUME



Materials, Analysis, Structural Design
and Applications of Textile Reinforced
Concrete/Fabric Reinforced
Cementitious Matrix

SP-345

Editors:
Barzin Mobasher and Flávio de Andrade Silva



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Materials, Analysis, Structural Design and Applications of Textile Reinforced Concrete/Fabric Reinforced Cementitious Matrix

Sponsored by
ACI Committee 549, Rilem-MCC

The Concrete Convention and Exposition
October 20-24, 2019
Cincinnati, OH, USA

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Flávio de Andrade Silva



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First printing, February 2021

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Printed in the United States of America

Editorial production: Gail L. Tatum

ISBN-13: 978-1-64195-133-3

PREFACE

Materials, Analysis, Structural Design and Applications of Textile Reinforced Concrete/Fabric Reinforced Cementitious Matrix

Several state-of-the-art sessions on textile-reinforced concrete/fabric-reinforced cementitious matrix (TRC/FRCM) were organized by ACI Committee 549 in collaboration with RILEM TC MCC during the ACI Fall 2019 Convention in Cincinnati, OH, and the ACI Virtual Technical Presentations in June 2020. The forum provided a unique opportunity to collect information and present knowledge in the field of TRC and FRCM as sustainable construction materials. The term TRC is typically used for new construction applications whereas the term FRCM refers to the repair applications of existing concrete and masonry. Both methods use a textile mesh as reinforcement and a cementitious-based matrix component and, due to high tensile and flexural strength and ductility, can be used to support structural loads. The technical sessions aimed to promote the technology, and document and develop recommendations for testing, design, and analysis, as well as to showcase the key features of these ductile and strong cement composite systems. New methods for characterization of key parameters were presented, and the results were collected towards the development of technical and state-of-the-art papers. Textile types include polymer-based (low and high stiffness), glass, natural, basalt, carbon, steel, and hybrid, whereas the matrix can include cementitious, geopolymers, and lightweight matrix (aggregates). Additives such as short fibers, fillers, and nanomaterials were also considered.

The sessions were attended by researchers, designers, students, and participants from the construction and fiber industries. The presence of people with different expertise and from different regions of the world provided a unique opportunity to share knowledge and promote collaborative efforts. The experience of an online technical forum was a success and may be used for future opportunities.

The workshop technical sessions chairs sincerely thank the ACI staff for doing a wonderful job in organizing the virtual sessions and ACI TC 549 and Rilem TC MCC for the collaboration.

August 2020
Barzin Mobasher
Flávio de Andrade Silva

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CARBON REINFORCED CONCRETE UNDER CYCLIC TENSILE LOADING

Arne Spelter, Juliane Wagner, Manfred Curbach and Josef Hegger

Synopsis: Carbon reinforced concrete (CRC) is a material composed of a high-performance concrete and a carbon reinforcement (textile grids, lamellas, rods). Composite materials with reinforcements of other fiber materials are called textile reinforced concrete (TRC). The investigations of CRC started more than 20 years ago and the continuous development as well as research findings have opened many fields of application. Today, the use of CRC includes the strengthening of reinforced concrete elements as well as the realization of new elements such as facades, shells and even bridges.

Some of these structures require knowledge of the fatigue behavior due to cyclic loading (e. g. bridges). In a collaborative project of the Institute of Structural Concrete of the RWTH Aachen University and the Institute of Concrete Structures of the TU Dresden, the uniaxial tensile fatigue behavior of two carbon textile reinforcement types was systematically investigated. The specimens were subjected up to 107 loading cycles and stress ranges up to 261 ksi (1,800 MPa). The influence of the maximum load and amplitude were investigated as well as fatigue curves for these two reinforcement types derived.

Keywords: Carbon reinforced concrete (CRC), cyclic loading, fatigue curve, S-N curve, tensile fatigue, textile reinforced concrete (TRC)

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Arne Spelter is a research associate at the Institute of Structural Concrete at the RWTH Aachen University, Germany. He received his BS and MS from the RWTH Aachen University in 2014 and 2016. Within the scope of his research work he deals with the static and dynamic long-term behavior of non-metallic reinforcements.

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Josef Hegger is professor of the Institute of Structural Concrete at the RWTH Aachen University since 1993. He is part of several standard committees for the Eurocode 2 – Design of Concrete Structures. In addition, he is a member of several committees of experts of the Deutsches Institut für Bautechnik (DIBt). His research interests include the use of novel reinforcement elements and materials such as carbon in addition to the conventional areas of building and bridge constructions in typical steel reinforced concrete design.

1. Introduction

For more than two decades the innovative composite materials textile and carbon reinforced concrete (TRC and CRC), which comprise reinforcements as textile grids, bars, and lamellas, have been investigated. As a result, extensive knowledge about fibers, impregnation materials and the composite material as well as about their testing methods has been gathered [1-13].

The continuous development of non-metallic reinforcements, especially of grids and bars, paved the way for the realization of several structures made of CRC including facades, shells, slab elements and even bridges [14-21]. Recent research projects show, in addition to the already known designs for foot and cycle path bridges, the applicability of the reinforcements for road bridges [22, 23]. Another innovative field of application is the strengthening of existing structures with thin layers of CRC. Especially the strengthening of the shear capacity is interesting for the future regarding the current shear problems in case of bridges [24, 25]. Here, the fatigue behavior is of great importance and requires the examination of the textile reinforcements for fatigue.

Only a few investigations on the fatigue behavior of textile grids are available, e. g. [26-28]. However, these were primarily intended to provide knowledge of the basic applicability under cyclic loading conditions. Wagner and Curbach recently started systematic investigations on the uniaxial tensile fatigue and bond behavior of CRC [3, 4].

To make CRC marketable in Germany, a huge research project 'C³ - Carbon Concrete Composite' was launched. The research project is divided into various subprojects. The subprojects 'C³ V1.2 – Standardization and Approval' as well as 'C³ V2.1 – Long-term Durability' deal with investigations regarding the static and cyclic long-term behavior of carbon reinforcements, which are partly summarized in [1-4, 28].

The main objective of the conducted fatigue investigations is the derivation of stress-cycle-curves, which are known from steel construction as S N curves (fatigue / Woehler curves). The German standard DIN 50100 'Load controlled fatigue testing - Execution and evaluation of cyclic tests at constant load amplitudes on metallic specimens and components' [29] serves as basis for testing and evaluation of the fatigue tests.

The cyclic load as a sinusoidal load-time-function is illustrated in Fig. 1. The terminology is based on [30].

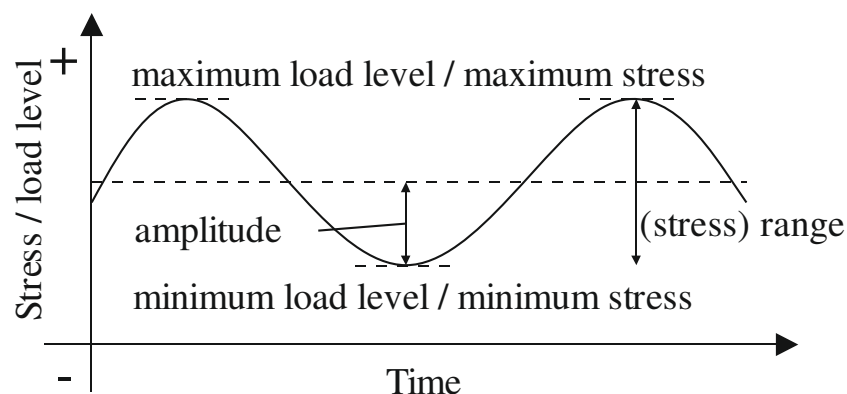


Fig. 1—Terminology used for fatigue (Graphic: Arne Spelter).

For design of cyclically loaded structural elements, S N curves are a mandatory requirement. S N curves are divided into different sections. In case of metallic materials, S N curves are divided into three sections, as shown in Fig. 2. The first section describes the low cycle fatigue up to 10^4 cycles. The fatigue strength under high cyclic loading describes the second section and runs up to the endurance cycle limit which describes the transition from the finite life to the infinite life at which no further loss of strength occurs. The relevant question is therefore inevitably whether this division also applies to CRC.

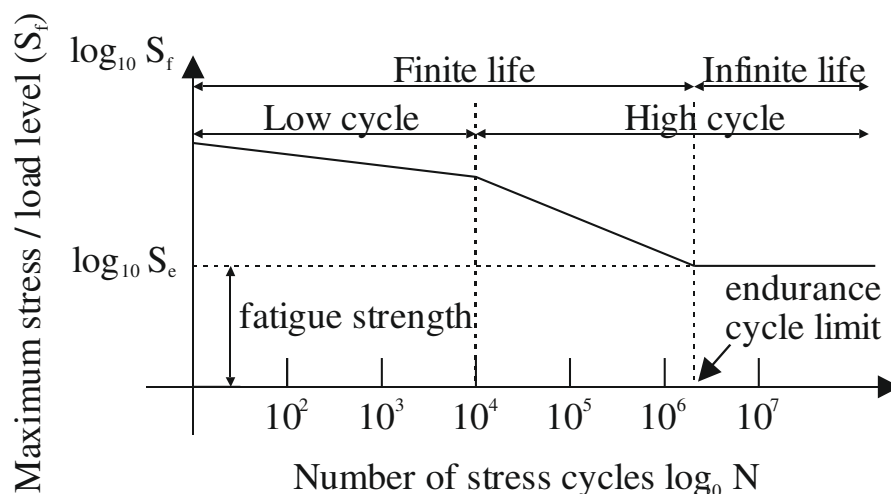


Fig. 2—Example of a typical S N curve (Graphic: Arne Spelter).

The results of uniaxial tensile fatigue tests on CRC specimens with textile grids from the two previously mentioned subprojects are presented in this paper. In the following, a distinction is made between two material combinations (type 1 and type 2). Type 1 consists of an epoxy resin impregnated carbon textile grid in combination with a high-strength concrete. Type 2 is made of a polyacrylate-impregnated carbon textile grid and a sprayable fine-grained concrete. After the examined materials are presented, the differences between the specimens as well as the experimental setups of the two research facilities are outlined. Subsequently, the test results and evaluations for derivation of S N curves for the two material combinations are introduced. Finally, the cyclic material properties are compared.

RESEARCH SIGNIFICANCE

The investigations of TRC and CRC elements were often limited to the short-term load-bearing behavior. However, the extended application possibilities lead to open questions concerning the static and cyclic long-term strength of the textile reinforcements so that creep and fatigue, is increasingly becoming focus of today's material investigations. Since S N curves for textile reinforcements are required for the design of TRC and CRC structures

or reinforced concrete structures strengthened with TRC / CRC, the presented investigations in this paper are very important for the broad application of this innovative composite material.

EXPERIMENTAL INVESTIGATION

The fatigue behavior of two material combinations (type 1 and type 2) with carbon textile grids was systematically investigated. For this purpose, first static reference tests were carried out to determine the stress levels for the fatigue tests (maximum and minimum load / medium load and amplitude). In the following, the materials, the test specimens and set ups as well as the test program is explained. The results of the experimental investigations are published in [18].

Materials

The carbon reinforcement of type 1 is an epoxy resin impregnated textile. In the following it will therefore be referred to 'EP grid'. It was selected as a representative reinforcement for new building components. The cross-sectional area of the symmetrical reinforcement is 0.045 in.² per foot (95 mm² per meter). The characteristics of the warp fiber strand of the symmetrical grid reinforcement are summarized in Table 1.

The textile reinforcement was investigated in combination with a high-strength, self-compacting concrete with a maximum grain-size of 0.2 in. (5 mm) which was developed in another subproject project of C³ [31]. The mix design of the cementitious matrix is for example given in [2]. The concrete compressive and flexural strengths were determined on prisms with dimensions of 6.3 x 1.6 x 1.6 in. (160 x 40 x 40 mm) [32]. At the Institute of Structural Concrete of the RWTH Aachen University, the concrete compressive strength was determined after 28 days to 17,300 psi (120 MPa) and the flexural strength to 2,000 psi (14 MPa) (average values from 21 prisms).

Type 2 is made of a polyacrylate-impregnated carbon-reinforcement and a sprayable fine-grained concrete. In the following, the reinforcement is therefore referred to as 'PA grid'. The textile grid has a cross-sectional area of 0.067 in.² per foot (140 mm² per meter) in warp direction and 0.013 in.² per foot (28 mm² per meter) in weft direction. The characteristics of the warp fiber strand of the biaxial grid are shown in Table 1.

The PA grid was investigated in combination with a sprayable fine-grained concrete (maximum grain size 1 mm). The compressive and flexural strength of this concrete were also determined on small concrete prisms. The testing of 12 prisms resulted in an average compressive strength of 12,900 psi (90 MPa) and a flexural strength of 1,200 psi (8.1 MPa) after 28 days. Fig. 3 shows sections of the two investigated carbon textile reinforcements.

Characteristics	EP-grid (type 1)	PA-grid (type 2)
Effective cross-sectional area, in. ² (mm ²)	0.0056 (3.6)	0.0028 (1.8)
Axial spacing, in. (mm)	1.5 (38)	0.5 (12.7)
Mean ultimate tensile strength, ksi (MPa)	467 (3,200)	513 (3,500)
5%-quantile ultimate tensile strength, ksi (MPa)	397 (2,700)	483 (3,300)
Young's modulus of elasticity, ksi (GPa)	35,530 (240)	33,650 (230)
Mean ultimate elongation, ‰	13.2	15.3

Table 1—Characteristics of the warp fiber strands of the textile grids from [33, 34] according to the methods of [35, 36].

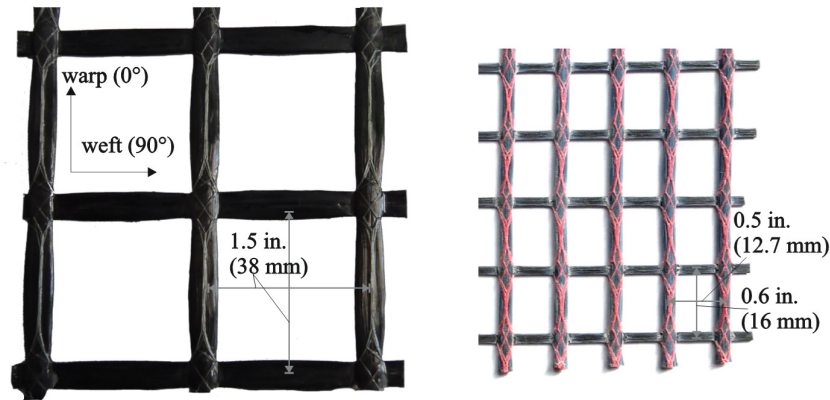


Fig. 3—Sections of the two examined reinforcements (left: EP grid, right: PA grid) (Photos: Arne Spelter and Sarah Bergmann).

Specimens

The specimens for type 1 at the RWTH had dimensions of 39.4 x 4.7 x 1.2 in. (100 x 12 x 3 cm) and were reinforced with one layer of the EP grid. The specimens are each reinforced with three warp fiber strands in test direction. The test specimens were produced in individual formworks using the casting method. For this purpose, the reinforcement was slightly prestressed in the formwork and then casted with concrete.

The specimens for type 1 at the TU Dresden differed only slightly. Instead of individual formwork elements, single-layer reinforced plates with dimensions of 39.0 x 15.0 x 1.2 in. (99 x 38 x 3 cm) were produced using the casting method. The plates were cut into three test specimens measuring 39.0 x 4.7 x 1.2 in. (99 x 12 x 3 cm), each reinforced with three warp fiber strands in test direction. The width of the test specimens does not correspond to a multiple of the fiber strand spacing due to the cutting process.

The specimens for type 2 were also reinforced with one carbon reinforcement layer (PA grid). The test specimens were produced by manufacturing CRC plates, which were then cut into individual test strips. The plates produced in Aachen had dimensions of 35.4 x 25.2 in. (90 x 64 cm), while the plates in Dresden measured 46.6 x 27.6 in. (118 x 70 cm). The plates were cut into specimens with four warp fiber strands and a width of 2.5 in. (6.5 cm) each (see Fig. 3 b) and d)). The thickness of the plates varied due to the laminating process, in which the fine-grained concrete and textile reinforcement are placed in layers. The plate thickness is 0.6 in. (16 mm; Dresden) and 0.4 to 0.6 in. (10 to 15 mm; Aachen) depending on the production process, respectively.

The specimens used and their respective clamping areas are shown in Fig. 4.

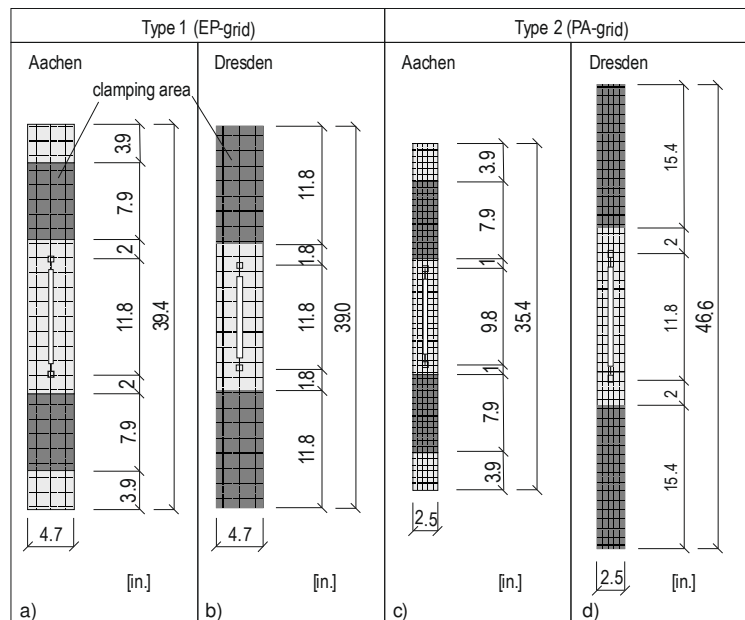


Fig. 4—Specimens for type 1 (a and b); specimens for type 2 (c and d)

(Graphic: Arne Spelter and Juliane Wagner).

Test setup and procedure

At a minimum concrete age of 28 (type 1) and 56 days (type 2), the specimens were tested in a cracked state for tensile fatigue resistance. Besides the specimens, the test setups of the research facilities differ due to the available testing infrastructure.

In Aachen both types were tested with one hydraulic test cylinder. The load from the test cylinder is transferred via clamping plates to the specimens. While the lower clamping device is firmly anchored in the ground, the upper part is connected to the test cylinder, which is attached to a steel frame, and is set into oscillation by it. The clamping pressure for the upper and lower clamping plates was 536 psi (3.7 MPa) for type 1 and 725 psi (5 MPa) for type 2.

Due to the reduced length of the test specimens of type 2 compared to type 1, the measuring range is reduced accordingly from 11.8 in. (30 cm) to 9.8 in. (25 cm). The test frequency is 10 Hz.

At the TU Dresden two test cylinders were used for carrying out the uniaxial cyclic tensile tests. The testing of both types could therefore be performed simultaneously. The test specimens were clamped at the top and bottom with the same pressure as in Aachen. The measuring length for each type is 11.8 in. (30 cm). The test frequency is up to 12 Hz depending on the natural frequency of the testing machines.

At both research facilities, inductive displacement transducers are used to measure the deformations. The displacement transducers are applied to the center of the flat sides of the specimens. The limit of tensile load cycles is set to 2×10^6 and in a few cases up to 10^7 . The tested specimens are classified into two categories. The first category is failing due to cyclic tensile loading and the second category are run-outs, which are classified by reaching the specified number of tensile load cycles. Finally, the residual load capacity of run-outs is investigated in order to determine possible damage due to cyclic tensile loading.

Fig. 5 shows the test setups of both research facilities with the arrangement of displacement transducers.

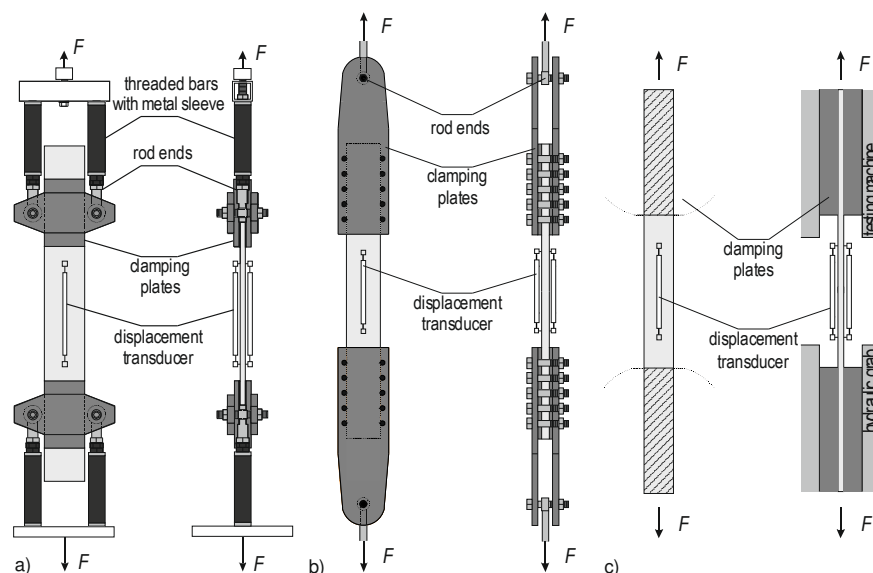


Fig. 5–Test setups for type 1 (a) Aachen, b) Dresden and type 2 a) Aachen c) Dresden (Graphic: Arne Spelter and Juliane Wagner).

Test program

To investigate the fatigue behavior of both types, minimum load levels between 30 and 75% of the respective static reference stresses were selected. The maximum load levels varied to investigate the influence of the maximum load level and amplitude in dependence of the different material properties. For type 1, maximum load levels of 290 to 410 ksi (2,000 to 2,900 MPa) and amplitudes up to 140 ksi (940 MPa) were investigated. For type 2 maximum load levels of 250 to 390 ksi (1,800 to 2,700 MPa) and amplitudes up to 100 ksi (660 MPa) were examined.

EVALUATION OF S-N CURVES

As explained in the beginning, the fatigue tests are evaluated according to the German standard DIN 50100 [29], which provides a few methods for evaluation of the finite life under cyclic loading. The so-called pearl string method was used for analyzation of the test results and is therefore presented in this paper.

Furthermore, the staircase method is presented, which was used to evaluate the fatigue strength of the two types.

Finite life fatigue (pearl string method)

With this method, the S N curve for 50% survival is determined by regression using the least squares method. The pearl string method starts at any load level. Depending on the test result – failure or run-out - the load level is adjusted in the next test. This means that if the specimen does not fail or fails only shortly before a specified limit of loading cycles, the load level is increased in the next test. If the specimen fails very early, the load level is reduced. If enough test data is available, a regression line can be generated from the test result.

As a slight deviation from the S N line in DIN 50100 [29], which is a double logarithmic function, a single-logarithmic line according to Wagner et al. [28] is chosen to describe the finite life fatigue strength, since a single-logarithmic scale for S N diagrams is common in both concrete and polymer construction [37-39].

The S N line is thus given in the form of Eq. 1:

$$\log N = \log C - k \cdot S \quad (1)$$

N = number of cycles [-];

C = constant to describe the position of the S N line [-];

k = inclination of the S N line [-]; and

S = maximum load level or amplitude [%].

By transforming Eq. 1 to N, Eq. 2 can be used to calculate directly the number of cycles for a given load level (maximum load level or amplitude):

$$N = C \cdot 10^{-k \cdot S} \quad (2)$$

Fatigue strength (staircase method)

The last section of the S N curve (see Fig. 2) is the fatigue strength. For the investigation of the fatigue strength, DIN 50100 [29] proposes the staircase method according to Hück [40]. Similar to the pearl string method, the procedure starts with any load level (maximum and minimum load level). The minimum load level is kept constant in each test series, while the maximum load level or amplitude, respectively, is adjusted upwards or downwards depending on the test results. The steps (or load horizons) should always have the same distance to each other. The distance is referred to as step d . The first load level (maximum load or amplitude) should ideally be selected close to the fatigue strength of the minimum load level. If the first test is a run-out, the load level is increased by one step in the following test. If the specimen fails, the following load level should be selected one step lower. The load levels are adjusted in equidistant steps.

For the evaluation of the procedure, each tested load level receives an ordinal number. The lowest load level receives the number 0. Afterwards the performed tests for each load level are counted and multiplied by the corresponding ordinal number.

The endurance stress limit is calculated by Eq. 3:

$$S_{NG} = S_0 + d \cdot \left(\frac{A_t}{F_t}\right) \quad (3)$$

$$A_t = \sum i \cdot f_i \quad (4)$$

$$F_t = \sum f_i \quad (5)$$

S_{NG} = fatigue strength at the endurance cycle limit NG [%]

S_0 = load level of the lowest step [%]

d = size of the equidistant step [%]

i = ordinal number [-]

f_i = number of tests per load level [-]

Fig. 6 shows an example of the procedure and evaluation of the staircase method.

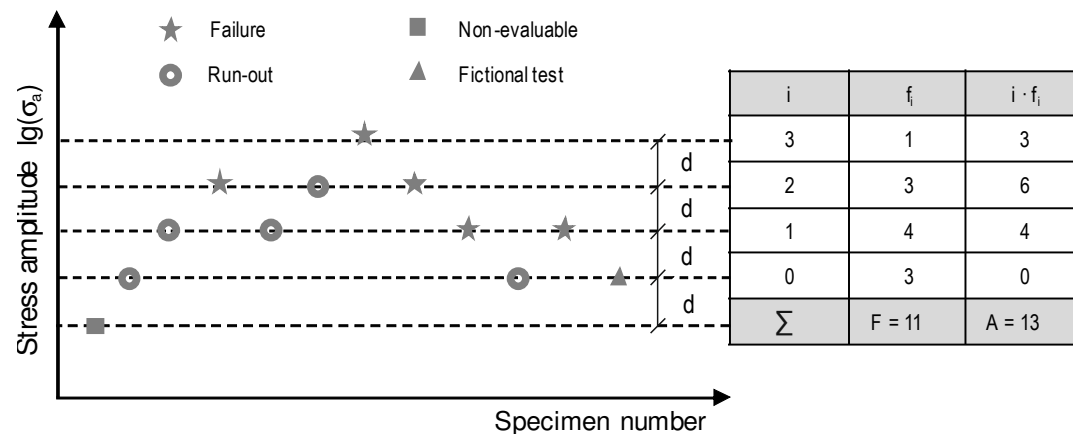


Fig. 6–Evaluation of the staircase method according to Hück [40] (Graphic: Arne Spelter).

Only 'closed' test series will be considered for the evaluation. An interruption of the test series is not permitted. Tests performed at the beginning of the test series may only be evaluated if the corresponding load level is reached again (Fig. 6 'non-evaluable'). Ideally, the first load level is selected as close as possible to the fatigue strength.

The number of tests per load horizon is important for the evaluation. No distinction is made between failures and run-outs. A fictitious test result can therefore be appended to the last physical test. For example, a fictitious test is set one step (or load horizon) higher for a run-out (Fig. 6 'fictitious test').

By calculation A_t and F_t , the fatigue strength can be determined considering the equidistant step d and the lowest load level (Eq. 3).

EXPERIMENTAL RESULTS AND DISCUSSION

In the following the test results are shown and discussed. As the main focus of this paper is the derivation of fatigue curves for the two reinforcement types, the evaluation of the individual tests is not presented. In first step, the results of the different combinations of materials (type 1 and 2) are closely considered.

Type 1 (EP grid)

The results of the cyclic tests of type 1 are shown in Fig. 7. The test results are plotted in relation of the maximum load to the reference load on the ordinate over the number of cycles (x-axis). The different colors indicate the varying values of the lower stress σ_{min} . Each dot represents a single test result. The lower part of the scatter range of the static reference tests is marked by the grey shading. Run-outs are indicated by arrows. All further tests failed due to rupture of the reinforcement.

The different test setups have no influence on the test results, so that all results from Aachen and Dresden are summarized in one diagram.

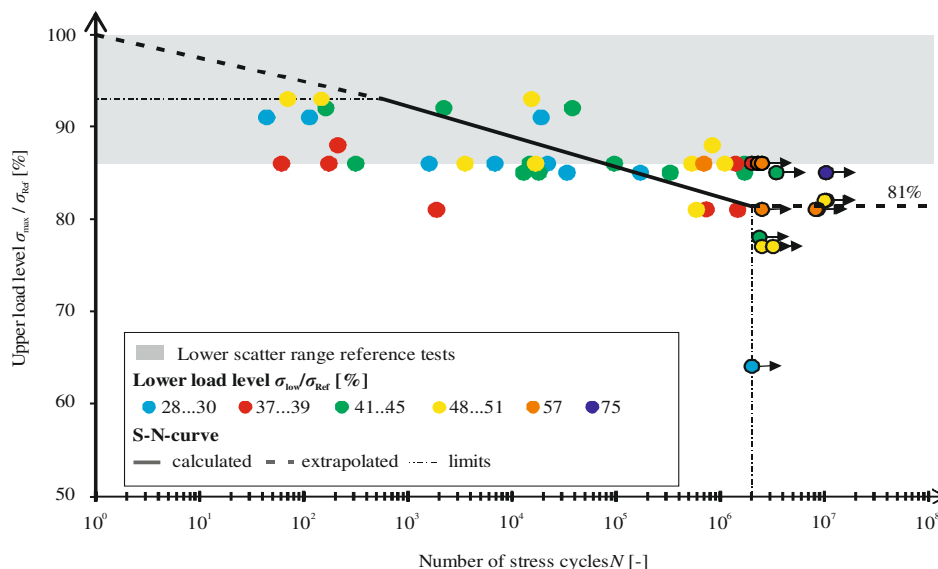


Fig. 7–Test results and S N curve for type 1 (EP grid). (Graphic: Juliane Wagner / Arne Spelter)

Fatigue failure of the specimens was only observed above a maximum load level of 80% of the reference load whereby an increase in the number of cycles is associated with decreasing maximum load level. Otherwise, the achieved number of cycles scatter strongly, which can be recognized in the varying maximum load levels at the limit of the scatter range of the reference load. Due to the scattering, no clear influence of the minimum load level on the number of cycles can be pointed out so that all test results are summarized in a single S N curve. Only the specimens with maximum load levels over 81% are considered (including run-outs) for the S N curve, as this is the lowest maximum load level at which failure occurred.

By linear correlation and determination of the constants C and k of the S N curve, Eq. 6 for determining the number of cycles as a function of the maximum stress σ_{max} in MPa results from Eq. 2:

$$N_{EP} = 12,6 \cdot 10^{30} \cdot 10^{-30,5 \cdot \sigma_{max}} \quad (6)$$

The highest tested load level $\sigma_{max} / \sigma_{Ref} = 93\%$ represents the maximum limit for Eq. 6 while the lower limit is defined by the intersection of the selected endurance limit of $2 \cdot 10^6$ cycles with the maximum load level of 81%.

The horizontal section of the S N curve at a maximum load level of 81% represents the fatigue strength. As the fatigue strength of this type has not been investigated yet, this assumption must be treated with caution. Furthermore, it should be noted that Eq. 6 was plotted to the median values of fatigue life and thus corresponds to a failure probability of 50%. Due to the large scattering of the test results, the calculation of the number of cycles is subject to a high degree of uncertainty.

Type 2 (PA grid)

The test results of type 2 differ slightly from those of type 1. Apart from an influence of the maximum load level, an influence of the minimum load level was observed so that different S N curves were derived for four different lower load levels minimum stresses (see Fig. 8).

Again, the different test setups of the research facilities have no influence on the test results. Therefore, the results from Aachen and Dresden are considered together.

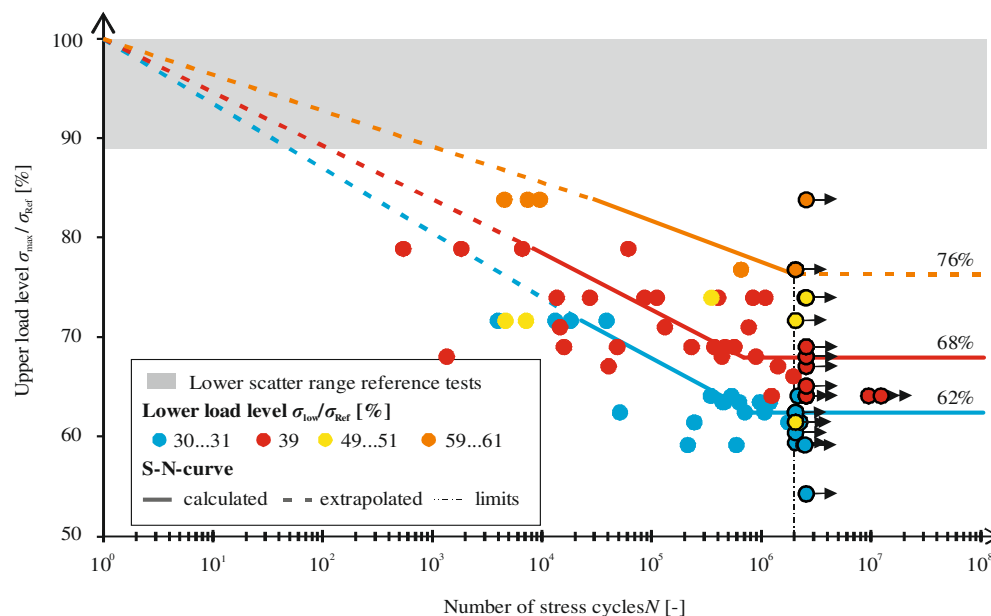


Fig. 8-Test results and S N curves for type 2 (PA grid). (Graphic: Juliane Wagner / Arne Spelter)

The influence of the minimum load level is clearly shown by a decrease of load cycles with decreasing minimum load level and resulting higher amplitude (with the same maximum load level). Consequently, the acknowledged dependency of concrete on both the minimum and the maximum load level is evident. The different S N curves are here generated for the minimum load levels of 30/31, 39 and 59 to 61%. Due to the limited experimental data base for minimum load levels of 49 to 51%, no S N curve was derived. The fatigue strengths were investigated for minimum load levels of 30/31 and 39% using the staircase method according to DIN 50100. For a minimum load level of 30/31% a load step of 1.0% was chosen. For a minimum load level of 39% the load step was 1.2%.

The evaluations of the staircase method can be found in [28]. While a minimum load level of 31% resulted in a maximum load level of 62% (amplitude approx. 16%), a minimum load level of 39% resulted in a maximum load level of 68% or an amplitude of 15% as fatigue strength, respectively.

The fatigue strengths are shown as horizontal lines in Fig. 6 and form the minimum load limits for the corresponding finite life fatigue lines. The point of intersection of the S N curves with the endurance limit of $2 \cdot 10^6$

106 cycles was defined as the lower load limit similar to type 1. The maximum limits were set at the highest examined maximum load levels. In accordance to type 1, the fatigue strengths determined by the staircase method must be critically questioned since the values are based on approx. 20 test values. The equations of the S N curves at mean value of the number of cycles, and thus the number of cycles with a 50% probability of failure.

The S N curves for type 2 are defined as follows (σ_{max} in MPa):

$$N_{PA,30\%} = 3,04 \cdot 10^{16} \cdot 10^{-16,9 \cdot \sigma_{max}} \quad (7)$$

$$N_{PA,40\%} = 5,20 \cdot 10^{17} \cdot 10^{-17,5 \cdot \sigma_{max}} \quad (8)$$

$$N_{PA,60\%} = 6,34 \cdot 10^{24} \cdot 10^{-24,2 \cdot \sigma_{max}} \quad (9)$$

Comparison of the results

The reference loads of the two material combinations (type 1: 449 ksi / 3,100 MPa; type 2: 466 ksi / 3,200 MPa) as well as the investigated upper load levels of the fatigue tests differ only slightly. While for type 1 an upper load level up to 92% of the reference load was applied, for type 2 upper load levels of maximum 84% were investigated.

For type 1, the cyclic loading led to a failure before 104 cycles without influence of the lower load level. According to the current state of investigations, the fatigue strength of type 1 is approx. 80% of the reference load. This value was determined in relation to the run-outs and must be considered with caution. Results for the investigation on the fatigue strength with the staircase method are not available at the present time. An influence of the lower load level is not discernible for type 1 at present. Therefore, the influence of the lower load level on the fatigue strength and possibly also on the finite life fatigue strength should be subject of future investigations.

For type 2, however, an influence of the lower load level can be determined. With a decreasing lower load level, the fatigue strength also decreases. This could be determined in two test series on the fatigue strength and indicates a direct influence of the amplitude on the fatigue strength.

Overall, the investigated load levels of type 1 were slightly above those of type 2. Due to the different areas of application of the two textile reinforcements, the load levels in the structural components also differ. While type 1 ensures the full load-bearing capacity of a structural component, the reinforcing steel in strengthened structural component contributes to a large extent of the load-bearing capacity, so that only a part of the load is taken by the CRC strengthening layer (type 2). In summary, it can be concluded that both types have a very good fatigue behavior and are therefore suitable for the respective applications.

A more precise comparison of the two material combinations is difficult due to the different properties of the textiles (impregnation, fiber strand cross section, fiber strand spacing, symmetrical / non-symmetrical grid) as well as the concrete properties.

Discussion

The investigations carried out within the framework of the collaborative research project on two different material combinations show that fatigue failure only occurs under high cyclic tensile loads. The allowable stresses are reduced by partial safety factors that must be applied for design. Accordingly, the load levels examined in this work will not occur in CRC components.

As described in the beginning of this work, the residual load capacities of the run-out specimens are determined. The investigations show that the residual load capacity of type 1 was at least in the scatter range of the reference load. Damage due to cyclic loading can thus almost be excluded for the investigated number of load cycles.

For the majority of the run-outs of the specimens of type 2, the residual strength was also within the scatter range of the reference strength. Further details can be found in [18]. For a few exceptions of type 2, a decrease of the residual load capacity could be determined with increasing amplitude.

The different material behavior under fatigue is decisively influenced by the impregnation of the two reinforcement types. The impregnation influences the bond between concrete and reinforcement as well as the bond between the individual fibers in the impregnated fiber strand. The specified endurance limit of 2×10^6 load cycles for many of the tests performed in this work should be reconsidered for future studies. The standards for non-metallic reinforcements propose 2×10^6 (ISO 10406-1 [41]), 4×10^6 (CSA S806-12 [42]) or up to more than

10^7 (ASTM D3479 [43]) cycles as the endurance limit. Due to the high fatigue strength of the carbon, it is advisable to test more than 2×10^6 load cycles.

The clamping pressure was chosen to prevent the specimens from slipping during both the static and cyclic tests [5]. According to Valerie et al. [8], the clamping pressure has no influence on the bond in CRC. Alternatively, the test specimens can be glued to the clamping plates for load introduction. In the tests carried out here, however, this would have led to additional loading of the adhesive layer between the test specimen and load application, which was deliberately avoided.

The size of the test specimens and the related different measuring lengths and specimen thicknesses as well as the tested number of fiber strands influence the test results. The length of the measuring range and the specimen thicknesses affect the crack formation and thus the fatigue behavior of the textile reinforcements, since stress peaks occur in the crack area which can cause the reinforcement to fail. Furthermore, the probability of flaws in the reinforcements increases due to larger measuring ranges as well as a growing number of fiber strands in the specimens. On the other hand, a better distribution of the stress on the fiber strands under cyclic loading can be achieved. Since this question cannot be conclusively clarified, it should be part of future investigations.

In addition to the tensile fatigue behavior of the textile reinforcements, the fatigue behavior of the bond between the CRC strengthening layer and the reinforced concrete component is of great importance. Within the scope of the subproject C³ V2.1, investigations are also carried out in this aspect.

It needs to be discussed whether a reduction factor due to fatigue is necessary for the design of cyclically stressed CRC components and if it therefore needs to be considered for a future design guideline.

CONCLUSIONS

Based on the results of the experimental investigation under cyclic tensile loading on two carbon textile reinforcements, the following conclusions are drawn:

- The specimens of type 1 (EP grid) failed only at maximum load levels above 80% and showed the best performance against fatigue. Due to the high maximum load levels close to the range of variation of the reference strength, the failure load cycles scattered.
- The specimens of type 2 (PA grid) failed at lower minimum load levels than type 1. However, the failure load cycles scattered only slightly. However, a direct comparison of the two types is very difficult due to the different textile reinforcements and concretes.
- The investigations showed that further investigations on the influence of the impregnation material as well as the minimum load level and amplitude are necessary. The influence of the concrete on the fatigue behavior should also be investigated to achieve a better comparability between the two reinforcements.
- Even if many specimens of both material combinations failed due to the applied tensile load cycles, the fatigue behavior of CRC can be rated as very good. The investigated loads are far above the permissible loads in components and are used to evaluate the maximum load-bearing behavior under cyclic tensile loading. The S N curves are determined for a 50% survival.
- The residual strength tests of the run outs showed that in most cases there was no reduction due to fatigue and that the residual strength was within the scatter range of the reference strength of the respective material combination.

In future studies the influence of the impregnation material as well as the concrete should be investigated more closely. An influence of the geometrical parameters (fiber strand cross-sectional area, fiber strand spacing etc.) cannot be excluded at the present time. Furthermore, the influence of the amplitude should be the focus of future investigations. The size of the amplitude as well as the number of load cycles depend mainly on the design traffic load of the structure. In addition, further investigations in the high cycle area (over 10×10^6 load cycles) should be carried out to confirm the assumptions made for the fatigue strengths tests.

ACKNOWLEDGMENTS

The results of this paper were obtained in the research projects C³ V1.2 and C³ V2.1 of the project C³. The authors would like to thank the Federal Ministry of Education and Research for the funding. Further thanks goes the staff of the research facilities for their support carrying out the tests.

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Shear Capacity of TRC slabs: Modelling and Examples from Practice

Jan Bielak, Norbert Will, Josef Hegger and Sven Bosbach

Synopsis: Textile-reinforced concrete (TRC) combines high-performance fabrics made of impregnated carbon yarns with state-of-the-art high strength concrete. Due to the corrosion resistance of non-metallic reinforcement, the application of TRC for external components especially with freeze-thaw and de-icing salt exposure is promising. This allows for reduction of concrete cover, to create slender structural elements and to execute thin slabs without additional waterproofing or protective decking. Different existing theoretical models and experience from various research projects were used in design of several pedestrian- and road bridges in Germany. The pedestrian bridges in Rems Valley and Ottenhöfen use TRC slabs without shear reinforcement as transversal load-bearing component. For the road bridges in Gaggenau, skew slabs made of TRC with shear reinforcement were chosen as principal structural system. Prior and during construction, experimental investigations on shear capacity were performed at the Institute of Structural Concrete (IMB) of RWTH Aachen. A comprehensive characterization of the material properties of the non-metallic reinforcement is a prerequisite for transfer and adaption of existing design rules, e.g. the determination of tensile strength of the bent portion of pre-formed shear reinforcement. This paper highlights the application potential and further challenges for the use of textile-reinforced concrete in new engineering constructions.

Keywords: bridges, carbon concrete composite, design provisions, shear, shear span, size effect, textile-reinforced concrete (TRC)

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INTRODUCTION

With the innovative composite material textile-reinforced concrete (TRC), a re-thinking of established construction methods becomes possible [1–3]. The advantages of textile non-metallic fiber-reinforced polymer (FRP) reinforcement especially for external applications such as bridges [4–6], façades [7–9] and high-rise applications with de-icing salt exposition such as park decks [10] for concrete construction is long known. For TRC, the same type of continuous fibers made of carbon or glass as for FRP bars are used. While glass fibers are cost-effective, the more expensive carbon fibers provide higher ultimate stress and better long-term durability [11] in alkaline environment. Rather than by pultrusion, the fibers are bundled and processed on textile machinery to 2D grid-like bi- or multiaxial warp-knitted fabrics or more complex 3D textile structures [12]. Through impregnation with polymeric resin (e.g. epoxy resin) and hardening in the production process, a high efficiency can be achieved regarding fiber strength utilization [13]. This is particularly important as the cost for technical fibers is still high. The yarn cross-sections in the grids are small compared to conventional steel or FRP rebar. Through narrow yarn spacing, a fine distribution of reinforcement is ensured. This makes it perfectly suitable for thin, planar elements such as slabs, façade panels and walls, but also as surface reinforcement or shear reinforcement in linear members. TRC can be used for repair, retrofitting and for new constructions. Internationally, the term fabric-reinforced cementitious matrix (FRCM) for repair and retrofitting is used synonymously to TRC [14]. This article focuses on the application of textile reinforcement as internal reinforcement for new components.

For bending design of TRC with fully impregnated fabrics, design models exist [15–17] and a reliability analysis with derivation of partial safety factors was performed [18]. Bond and anchorage of yarns and fabrics has been addressed in several studies in the recent years, e.g. [19–23], but the proposal of a design model for anchorage based on fundamental characteristics of the reinforcement and concrete including safety factors is pending. For shear design of TRC, engineers as well as researchers are still confronted with even more fundamental questions. Despite numerous applications for thin façade panels and slabs without shear reinforcement in recent years [7, 24–26], there is still need for research into the technical and economic application of TRC components with and without shear reinforcement made of non-metallic textiles, especially for larger constructions and higher loads. Within the coordinated research project C³ - Carbon Concrete Composite [27], IMB of RWTH Aachen University covers this specific aspect in the subproject C³-V-I.14. Reliable design models including partial safety factors for a variety of material combinations are the aim of current studies. Essential questions here are the transferability of shear design approaches from steel-reinforced concrete which rely on the plasticity of the cracked reinforced concrete as well as the limitation of usable ultimate stress and strain for the fiber reinforcement materials. Background to this problem is the linear-elastic material behavior up to the point of brittle failure of impregnated textiles in tension. This is closely linked to the determination of permissible reinforcement shapes and their type of anchorage in the compression and tension chord of members subjected to bending. The fundamental problem of the intersection of textile grids for longitudinal and transverse reinforcement is well known (see e.g. [4, 6]) but unsolved satisfactorily. This point of conflict does not occur with separate stirrups and bar-shaped longitudinal reinforcement usually used in steel- and FRP reinforced concrete. In addition to technical aspects, economic aspects need to be considered, which could lead to different reinforcement types for beams and slabs. A simple

transfer of established reinforcement strategies and design models from reinforced concrete construction is short-sighted. The article gives an overview on current discussions on shear capacity without and with shear reinforcement. The challenges of shear design and the requirements of practice are illustrated with three bridge project examples from Germany.

SHEAR CAPACITY WITHOUT SHEAR REINFORCEMENT

Two-dimensional textile reinforcement grids are ideally suitable for reinforcing planar elements as for example one- or two-way slabs, panels, and shells. In such components the placement of large textile mats is easier than arranging a multitude of individual small-scaled FRP bars. While for very thin panels typically used in facades (< 5 cm (2 in)) often limitation of deflection and bending capacity due to the low effective depth governs design, for slabs between 5 cm and 20 cm (2 in and 7.9 in), shear capacity might become a decisive factor. Typical applications of this depth are secondary structural elements such as slabs spanning between principal beams, compressive chords of T-beams, or slabs for small precast elements such as garages. TRC distinctly differs from concrete with fiber reinforced polymer (FRP) bars as primary reinforcement. The latter is much more comparable to conventional steel reinforced concrete in component size, reinforcement diameter, stiffness in transversal direction, shape, and bond mechanisms. However, as recent investigations showed, existing models for FRP reinforcement to predict shear capacity without shear reinforcement seem suitable for TRC [28], as well. The model currently in discussion for new Eurocode 2 [29] (simplified critical shear crack theory, CSCT, e.g. [30–32]) was able to predict ultimate shear capacities of TRC slab segments with impregnated carbon textile reinforcement with satisfying accuracy. The model considers different modulus of elasticity of the longitudinal reinforcement but is unable to account for different bond characteristics of reinforcement. Other models, e.g. [33], are as well suitable for prediction of shear capacity. However, the fundamentally different crack patterns and the high scatter observed in the tests described in [28] indicate that for TRC, longitudinal cracking, concrete splitting and delamination resulting from bond requires additional, fundamental research. A larger database with specimen utilizing different reinforcement types (especially by variation of modulus of elasticity, and variation of bond strength) is a prerequisite for transfer and final adaption of existing models and derivation of partial safety factors. The differences between bar-shaped FRP reinforcement for beams and slabs and non-metallic textile FRP reinforcement for thin slabs should be well understood to avoid fallacies in transfer of design approaches. The following list offers some crucial points to consider for evaluation of TRC shear capacity:

- Textile grids usually provide less cross-sectional area per yarn than FRP bars and thus have a smaller bending stiffness (smaller area moment of inertia). This is relevant for shear stress transfer by dowel action of longitudinal reinforcement.
- The effect of in-plane longitudinal cracking with splitting and delamination of concrete is more relevant for 2-D grids than for individual bars, where the cross-sectional area of longitudinal reinforcement is concentrated on fewer, often circular shaped elements. Typical available grids cover a larger in-plane area where no concrete tensile stress can be transferred (lower effective area). The individual yarns are typically elliptic or flat rather than circular, with the larger elliptical diameter oriented in-plane [23, 34, 35]. The remaining concrete in the grid openings is subjected to higher tensile stress perpendicular to the reinforcement axis compared to the concrete between FRP-bar reinforcement.
- Compression zones become disproportionately small in thin slabs with TRC reinforcement (especially using high-strength concrete) compared to beams with larger effective depth. This influences the relative contribution of the uncracked compression zone to shear resistance of members without shear reinforcement and the ability to form direct compression struts or compression arches.
- For TRC, usually concrete with small aggregate size (< 8 mm (0.31 in)) is used to enable compaction and concrete flow through the small grid openings. This has an influence on the often assumed shear transfer mechanism by aggregate interlock for lower concrete strength. For high-strength concrete, which is typically employed for TRC, aggregate size (and aggregate interlock) is less important, because shear cracks pass through the grains. Thus, only mechanical interlock of rough crack surfaces remains.

Three major fields of study regarding shear capacity of TRC slabs without shear reinforcement have not yet been addressed. First, the effect of longitudinal axial stress (compression and tension) on shear capacity of TRC needs further investigation. A small number of tests with prestressing rods [36] or tendons [37] combined with textile shear reinforcement were performed previously. Prestressing of slab panels with bonded carbon micro-strand reinforcement is performed by a swiss manufacturer [38, 39]. For those panels, shear failure could no longer be achieved in bending tests due to positive influence of prestressing. However, the presumed negative effect of axial tension on shear capacity of TRC slabs has not been studied in experiments, yet. This is relevant in restraint systems with centric tension due to shrinkage or temperature loads, and for superposition of bracing loads (e.g. in frame systems) with shear from vertical loads. Second, the influence of the effective shear span and shear slenderness in continuous, statically indeterminate systems needs to be addressed. Similar studies have recently

been performed for steel-reinforced concrete (see [40, 41]) at IMB of RWTH Aachen. Last, the effect of distributed loads and the proportion of directly transferable loads both for concentrated loads and for distributed loads needs clarification. One major result would be the applicability of reduction factors for loads near supports, which eventually leads to safe and more economic design for shear.

ENHANCING SHEAR CAPACITY WITH DISTRIBUTED TEXTILE REINFORCEMENT

Usually, engineers try to avoid placement of transversal reinforcement in thin slabs to minimize construction time and to reduce costs. However, in regions with concentrated loads, at edges or near supports, higher-than-usual loads which surpass the shear capacity of those simply reinforced slabs might occur. If local increase of longitudinal reinforcement does not provide enough shear strength and an increase of depth or concrete strength are no option, the application of shear reinforcement is typically the only solution. 2-dimensional (bi- or multiaxial) reinforcement grids made of non-metallic impregnated yarns are a promising alternative to localized shear connector pins or closed stirrups. Those reinforcements can either be planar or pre-formed in the desired shape prior to hardening of the impregnation resin (Figure 1, right). They provide a distributed reinforcing function with multiple yarns which cross emerging shear cracks. The arrangement in regular intervals in transversal and in longitudinal direction activates larger regions of concrete for anchorage and stress transfer. An even distribution of shear reinforcement elements over the width of the component promises a more favorable stress distribution in the concrete (Figure 1, left). When high-performance textiles with short anchorage lengths are used, the concrete is subjected to comparatively higher local stress than in steel-reinforced concrete. Through distribution, local stress concentrations at the ends or at bends, as are typical for single reinforcement elements such as stirrups or headed bars, can be reduced. This also minimizes the local stress on the anisotropic fiber material caused by transverse pressure. Additionally, less internal transversal forces are required to transmit shear forces to the reinforcement elements.

For TRC slabs with grid-type longitudinal reinforcement, closed stirrup-cages known from beam elements cannot be used economically. Possible reinforcement elements in this situation are for example planar vertical grids, C-profiles, double C-profiles, or S-profiles (Figure 1, left). The question of proper anchorage of those elements in tension and compression chord is most relevant for evaluation of efficiency. However, the aspect of constructability and ease of use on site or in precast element production should not be neglected. In all cases, an intersection of longitudinal and shear reinforcement planes in the tension chord is required. If the yarn spacing and yarn orientation of both reinforcements are matched, the elements may be inserted into each other. The manufacturing technique in textile machines allows to omit specific yarns during production. Thus, the legs of S- or C-profiles can be produced without longitudinal yarns which would otherwise prevent the insertion. An element with high efficiency (known from TRC sandwich-wall panels, [42]) is shown in Figure 1, bottom right. The 45° inclined yarns cross typical shear cracks at a favorable angle, and they are provided with longer anchorage length. However, the longitudinal reinforcement needs to be cut at the intersection point or be placed on top or below this element. Higher effort for production, higher waste and subsequently higher cost counteract the greater material efficiency. In consequence, 90°-oriented transversal yarns are often preferred.

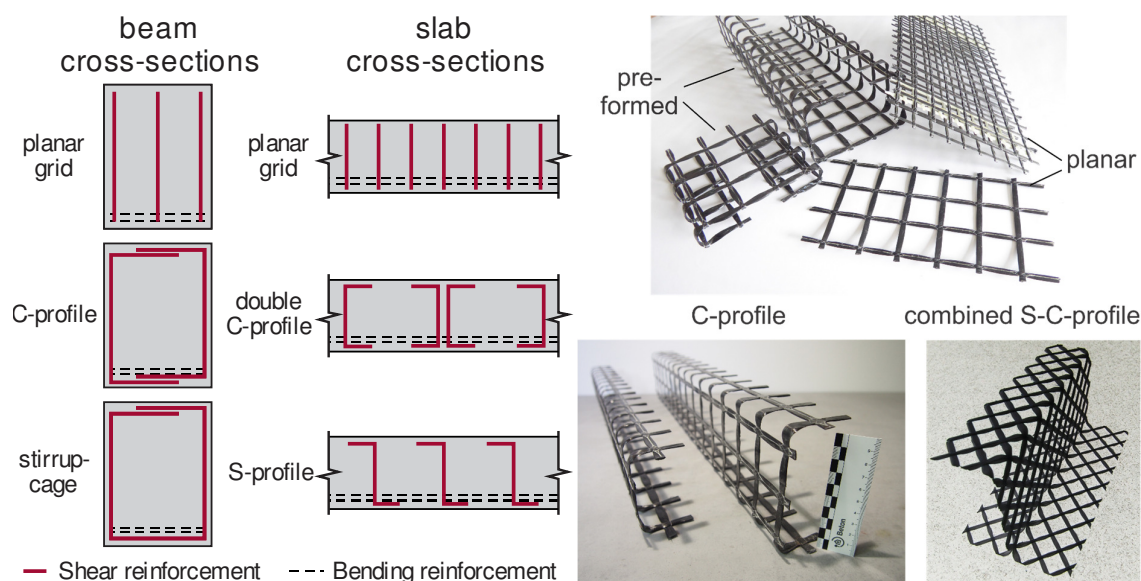


Figure 1–Typology of distributed textile reinforcement for beam and slab cross-sections (left) and photos of pre-formed and planar carbon-textiles with epoxy impregnation (right) (figure and photos: J. Bielak)

To date, no validated models for prediction of shear capacity of TRC slabs with distributed shear reinforcement types as shown in Figure 1 exist. Experimental investigations on slab segments with planar and C-shaped reinforcement indicate that both types are suitable to effectively increase shear capacity of slabs [43]. *Kulas* investigated the effect of impregnated, pre-formed double-C-profiles made of AR-glass in thin double-T-beams [34]. In his experiments local pull-out and concrete pry-out at the bend of the reinforcement near the flange led to premature (anchorage) failure. Consequently, the model he proposes, incorporating those failure modes, is of limited use for flat slabs. *Kromoser* and *Preinstorfer* tested thin elements with bar-shaped longitudinal and pre-formed shear reinforcement in bending and shear and observed similar behavior [2, 36]. Additionally, delamination in the webs occurred and eventually governed the compressive strut capacity. Those two failure modes, local pry-out and splitting in the web are less relevant for slabs and slab segments. Shear compression failure with reduction of compressive chord and failure of the reinforcement in tension are the two decisive failure mechanisms. As first step towards a reliable calculation procedure, the transfer of existing models (e.g. [33, 44–48]) or code provisions [29, 49–51] for FRP shear reinforced elements to TRC slabs is a promising approach. Nevertheless, some fundamental differences between FRP shear reinforcement and textile reinforcement should be highlighted:

- As mentioned for longitudinal textile reinforcement, also the vertical yarns (serving as shear reinforcement) provide less transversal stiffness compared to FRP bars. They are easier sheared off when shear crack faces are displaced parallel to the crack, and a reduction in strength might be necessary due to this effect.
- The transversal yarns available in biaxial grid-like shear reinforcement may contribute to shear resistance in 90°-configuration of reinforcement elements.
- Depending on the type of impregnation material, different anchorage mechanisms govern for textile shear reinforcement. This needs to be considered at some point of shear design, either directly or implicitly by detailing provisions.
- In thin slabs, anchorage of vertical reinforcement elements becomes a relevant factor for shear capacity. As the anchorage length does not scale down together with the effective depth, the same shear reinforcement materials are less effective in thin slabs due to their lower anchorage length.

Considering those fundamental questions, a simple transfer of existing models to (thin) TRC slabs is not advisable. Especially the well-known tie-and strut model with 45° inclined compression fields should not be utilized for thin slabs. The actual strain state of the vertical reinforcement needs to be considered. Typical impregnated non-metallic textile reinforcements with glass or carbon filament material would require strains larger than 10 ‰ for rupture. Shear compression failure occurs before a necessary shear crack opening would be reached, and the reinforcement is not fully utilized. This effect is known from FRP-bars as shear reinforcement, where upper strain limits are given in several models and codes. However, the effect is even more pronounced for TRC elements with smaller effective depth, larger deflections and larger rotation of the strain distribution. Additionally, the anchorage of the vertical elements in the nodes (compression- and tension chord) is less effective, especially for straight elements, but also for pre-formed reinforcements. On the one hand, ongoing research at IMB RWTH Aachen aims at investigation of the individual effects contributing to shear resistance separately. On the other hand, experimental insight based on actual projects from practice guarantee that economic aspects and aspects of constructability and ease of use for textile shear reinforcement are equally considered.

REQUIREMENTS FOR TRC SHEAR DESIGN: EXAMPLES FROM PRACTICE

Pedestrian bridges in Rems Valley

On the occasion of a regional horticultural show in 2019, three new pedestrian and cyclist bridges were erected in Rems Valley, Germany. All three bridges are of type ‘Integral Stuttgart Timber Bridge’ [52, 53], with two longer spans of 38.2 m (125 ft 4 in) and one shorter span with 16.6 m (54 ft 5.5 in) length. The principal structural system consists of glued laminated timber beams (Figure 2). Glued-in reinforcement bars connect the clamping moment at both ends to a steel reinforced concrete support. The thin large-scale deck slabs, however, are made of TRC without additional water-proofing or protective decking. The upper, sandblasted side is directly driven upon. Those elements have a maximum dimension of 5.09 m · 3.20 m (200 in · 129 in) and rest on transversal hollow steel profiles, which also connect the steel railings on both sides of the bridge. As bending reinforcement on top and bottom, one layer of epoxy-impregnated carbon grid was utilized. The slabs have no shear reinforcement, but contain non-structural, preformed spacers and C-profiles at free edges made of the same reinforcement material (Figure 3, left).

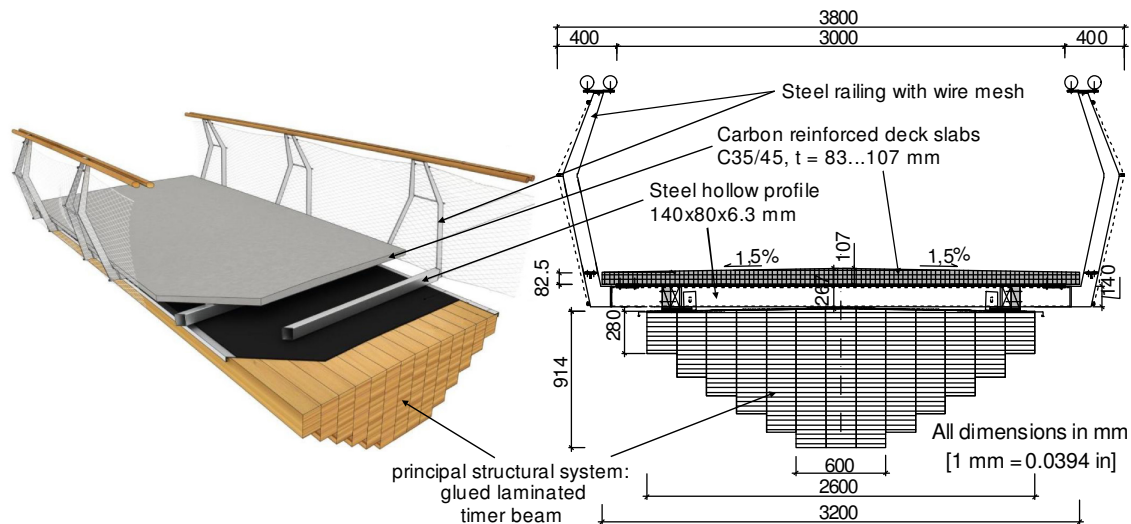


Figure 2–Pedestrian bridges in Rems Valley, Germany, with TRC deck slabs (bridge type “Integral Stuttgart Timber Bridge” [53], rendering and cross-section by Knippers Helbig GmbH)

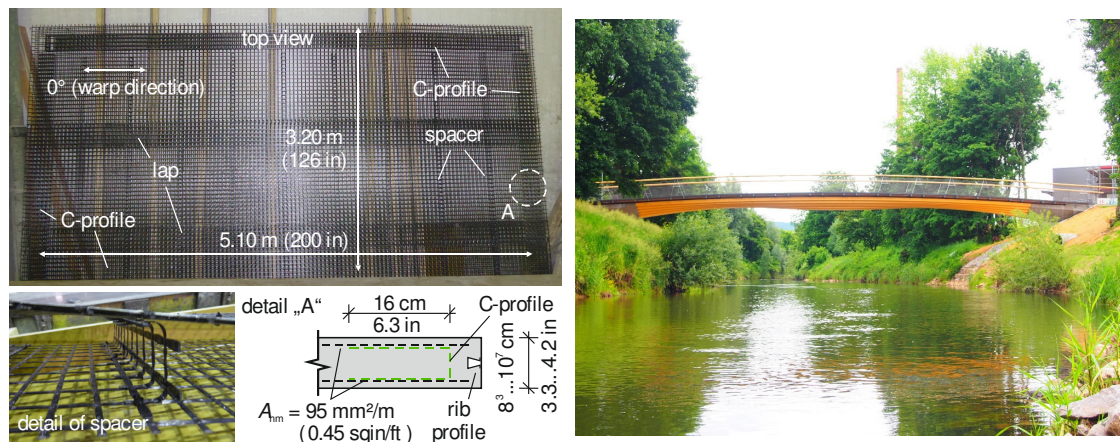


Figure 3–Reinforcement cage for typical slab element (left) and finished bridge in Weinstadt, Germany (right)
(Photos: Jan Bielak)

Non-standardized construction techniques and construction products such as TRC require approval by federal authorities in Germany if they are utilized in areas with public access. In design for construction performed by Knippers Helbig GmbH, IMB of RWTH Aachen University was assigned with evaluation of structural safety and load bearing capacity of the TRC parts. Both bending and anchorage verification was performed with existing models and based on previous experimental experience. However, for shear design a new approach following the simplified critical shear crack theory [29, 30] was utilized. Due to pending validation of this model for TRC and limited experimental experience for shear failure in thin TRC slabs, the federal authorities called for extended quality control during production of the slabs. Therefore, six additional slab segments were cast together with the actual deck, utilizing the same production methods. Their thickness and reinforcement were the same as for the critical end-slabs of the bridges, which had a larger thickness (120 mm (4.7 in)) compared to the standard slab (83 mm (3.3 in)). Bending reinforcement consisted of two layers in warp direction (0°) with a filament cross-sectional area of 95 mm²/m (0.45 sqin/ft) each. Those 50 cm (19.7 in) wide slab segments were tested in three-point bending with a shear span of 400 mm (15.7 in). Two tests per specimen were performed. 11 specimens failed in shear, as intended. One test failed in bending with rupture of the reinforcement, because the corresponding specimen had one layer of reinforcement incorrectly placed on the wrong side of the slab. Figure 4 compares the ultimate shear loads obtained in the experiments to prediction by two models. Mean values for compressive strength (47.4 MPa (6875 psi)) and modulus of elasticity of the concrete (27303 MPa (3960,000 psi)) at the day of testing were used. Mean values for textile were determined in uniaxial tensile tests according to [54] and [55] accounting to 3221 MPa (467167 psi) for ultimate stress, and 244835 MPa (35,510,314 psi) for modulus of elasticity. Further characteristics of the reinforcement are given in [28]. The results show clearly that the

CSCT/EC2 model [29] utilized during design can predict the ultimate shear loads with satisfying accuracy on mean value level ($\gamma_c = 1.0$). The coefficient of variation of 14.6 % is typical for shear tests. Note that in this case the scatter results almost exclusively from the experimental side and not from the model uncertainty, because the specimens were nearly identical. To transfer from mean value level to design level, model uncertainty and scatter of random variables need to be considered either in the empirical factor (here, 0.6 was utilized as per draft 3 for steel-reinforced concrete) or the material safety factor for concrete, $\gamma_c = 1.5$, to reach the desired reliability level. In contrast to the CSCT model, the current ACI model [50] is more conservative with its prediction without further modification, using mean value of concrete compressive strength and applying no strength reduction factor for shear ($\phi = 1.0$). The different reliability concept behind both design codes and the different stages where safety factors are incorporated in the models needs to be considered when directly comparing prediction quality on mean value level.

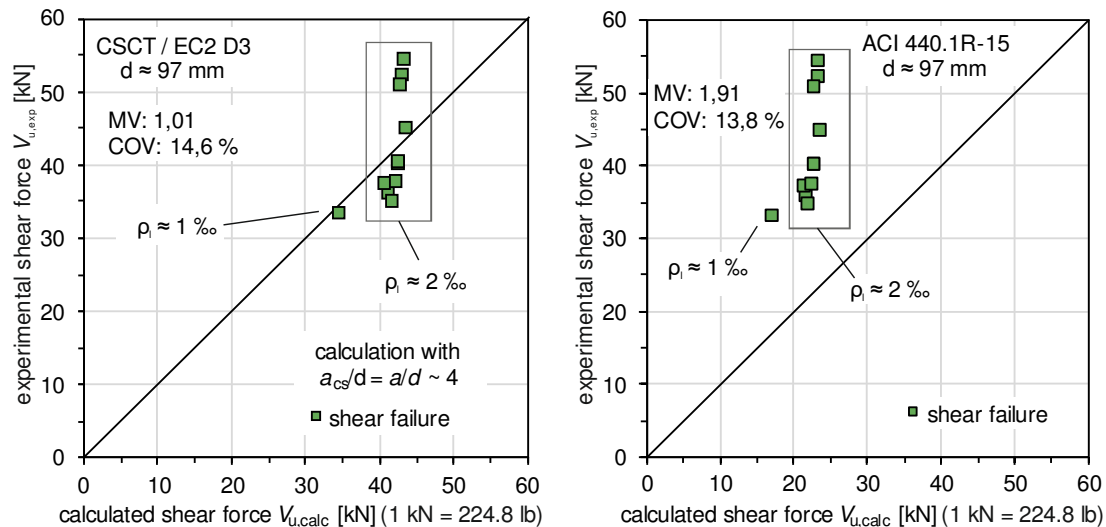


Figure 4—Comparison of experimental results from shear tests during quality control of Rems Valley bridges with prediction. Left: Eurocode 2 draft 3; right: ACI code.

The bridges in Rems Valley show a clever combination of materials that makes the best possible use of the strengths of each component, while maintaining an outstanding architectural quality. This was recognized by nomination for the German Bridge Construction Prize in 2020 [56]. The innovative material TRC with its non-corrosive reinforcement is perfectly suitable as outermost protective cover for the timber [52]. The predecessor of Rems Valley bridges in Neckartenzlingen, Germany [57], used conventional steel-reinforced concrete with additional protective coating and more joints and the “Schillersteg” in Schwäbisch Gmünd, Germany, had natural stone panels with significantly more joints and larger thickness. In contrast, TRC slabs require no coating. Due to their reduced thickness and adjustable cross-sectional resistance, larger panels can be prefabricated maintaining the same lifting weight, which might reduce the number of maintenance-intensive joints. Thus, maintenance costs, which have a large influence on life-cycle cost of bridges [58], could be lowered. Alternative deckings such as full FRP elements or the use of non-corrosive steel reinforcement were not considered in this project.

Pedestrian bridge in Ottenhöfen

The small community of Ottenhöfen in the Black Forest, Germany, commissioned the replacement of an old pedestrian and bicycle bridge over the river Acher (Figure 5) in 2018. The principal structural system of the 13.55 m (44 ft 5 1/2 in) long bridge consists of conventional double-T steel beams in single span. In transversal direction, 10 cm (4 in) thin carbon-reinforced concrete elements transfer all vertical loads as one-way three-span slab (Figure 6). As the bridge is intended as emergency relief for local traffic, high concentrated loads from 16 t trucks (35,274 lb) with an area of $40 \cdot 40$ cm² (15.7 in · 15.7 in) per tire were considered. Eventually, a smaller 12 t service vehicle (26,456 lb), e.g. for snow removal, was decisive for shear design due to the smaller load area per tire of $20 \cdot 20$ cm² (7.9 in · 7.9 in).



Figure 5—Site view prior to construction of new bridge; left: construction pit for new foundation (photo: RS Ingenieure GmbH & Co. KG); right: temporary pedestrian bridge (photo: Sergej Rempel)

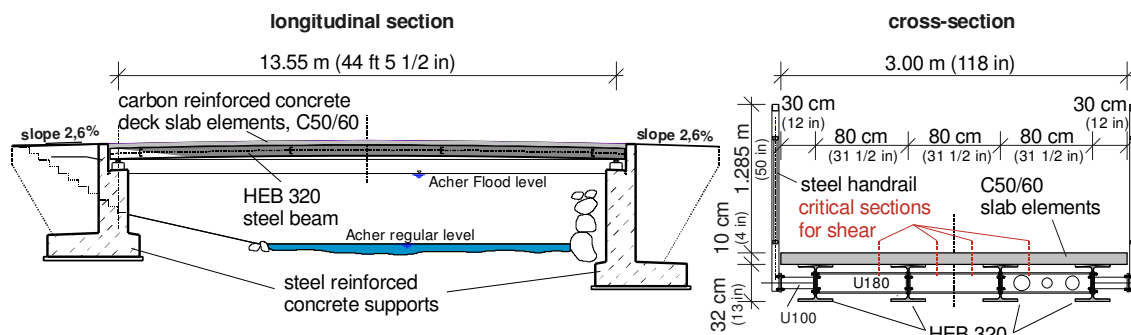


Figure 6—Longitudinal and cross-section of new pedestrian bridge in Ottenhöfen, Germany, with TRC slabs as three-span continuous beam in transversal direction (drawings adapted from RS Ingenieure GmbH & Co. KG)

The $2.00 \cdot 3.00$ large precast concrete elements contained bending reinforcement, but no shear reinforcement (Figure 7). The single-layered epoxy-impregnated carbon grids had a filament cross-sectional area of $142 \text{ mm}^2/\text{m}$ (0.67 sqin/ft) in both directions, with the warp direction perpendicular to the steel beams. Again, non-structural C-profiles along the free edges and spacers made from the same material were utilized. The concrete is of grade C50/60 with maximum aggregate size 8 mm (0.3 in) designed by the contractor to meet the requirements of freeze/thaw and de-icing salt resistance without utilization of air-entraining agents. Each plate is secured against uplift at the flanges of the two outer beams with cast-in threaded sleeves and screws (Figure 7, right). IMB of RWTH Aachen University produced the expert report necessary for utilization of the non-standardized construction material TRC. This included verification of bending, shear, anchorage and longitudinal crack formation in ultimate limit state. The latter was of interest because of the small concrete cover of 1.5 cm (0.6 in), and therefore evaluated by using results from uniaxial tensile specimens according to the test setup in [59]. Shear was critical for design in this example from practice. In the center of the plates, the high concentrated loads can be distributed on a larger length at the critical sections for shear (Figure 6, right). However, at the free edges adjacent to the next plate, no transfer of vertical loads is intended from one plate to another. Thus, the concentrated loads of one tire right at the edge of the plate cannot be distributed in two directions, and the linear shear loads are considerably higher at the edges than in the center of the plate. There, theoretical shear verification to obtain permission for construction with existing models was not possible. The potential positive influence of the continuous system in this case could not be predicted in advance. One reason for this is that previous shear tests were performed solely on single-span slab segments. An experimental campaign for verification of shear capacity of the exclusively carbon-reinforced slabs could not be finished prior to erection of the bridge because of time constraints. In consequence, the client decided to insert additional stainless-steel rebar at both edges (Figure 7, left). For those bars, a general building approval in Germany enables verification of shear loads according to standard Eurocode 2 provisions. Apart from the edges, the shear verification was performed using the same approach as for Rems Valley bridges, with [29]. Bending and anchorage of the reinforcement was not decisive for choice of reinforcement.

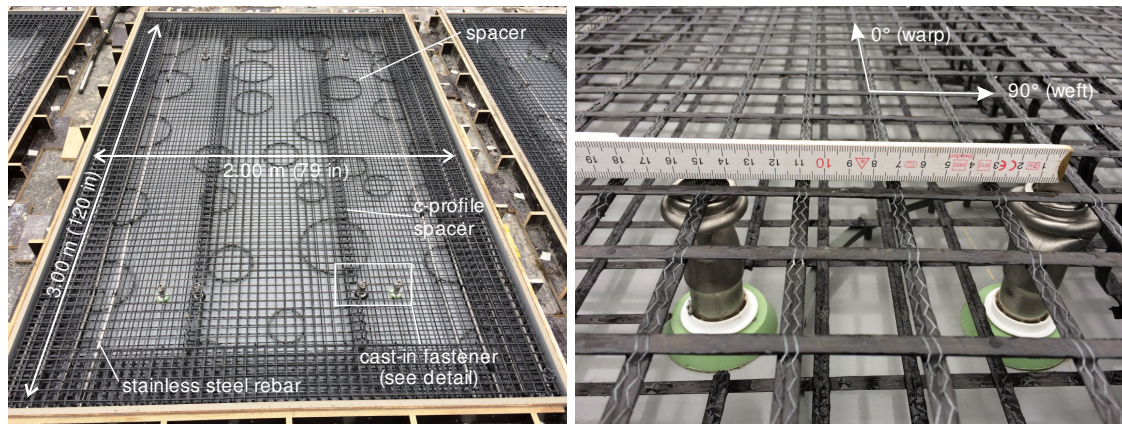


Figure 7–Reinforcement for precast deck slabs of Ottenhöfen bridge: overview (left) and detail (right)
(photos: Sergej Rempel)

To verify the models used in the expert report and for quality control of the contractor, 6 slab segments of the same depth, but with the half of the real length, (150 cm (59 in)), and a width of 40 cm (15.7 in) were produced together with the deck slabs. In the experiments, bending tests simulating the continuous system aiming at shear failure were performed for the first time for TRC. The goal of this approach was to investigate whether the shear capacity compared to single-span beams is higher or not. In the tests, a cantilever with a concentrated load created a hogging moment, while the system remained statically determinate (Figure 8). Both concentrated loads were introduced by a rigid steel crossbeam and were increased proportionally. The moment at support B amounted to 50 % of the span moment. This relates to the critical load case in the actual bridge, where one tire of the service truck is in the center of the outermost span.

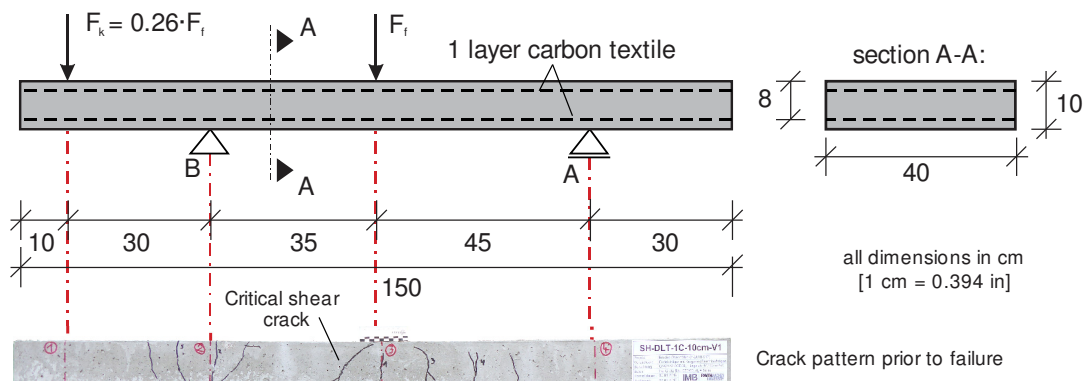


Figure 8–Test setup for simulation of continuous system and respective crack pattern

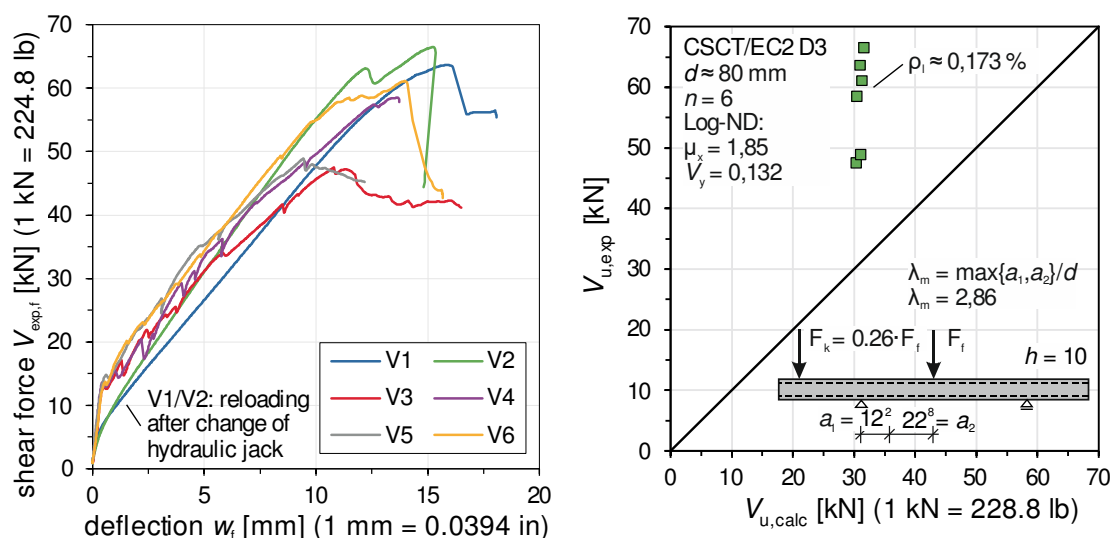


Figure 9—Experimental response of single-span slab segments with cantilever and comparison with calculation approach EC2 D3 [29]

The results indicate that the ultimate shear capacity in the continuous system with an equivalent moment to shear ratio $a/d \approx 2.86$ is larger than anticipated by results from single-span tests which had a shear span of $4 \cdot d$ (Figure 9). The crack pattern observed for the specimen in Figure 8 shows two shear cracks, one inclined bending crack near support B and one near the concentrated load in mid-span which eventually turned out to be the critical shear crack at maximum load. At this point, a general statement on the positive influence of statically overdetermined unidirectional multi-span systems with their smaller moment to shear ratio and the presumed higher shear capacity at inner supports for reinforcement with brittle failure and no yield capacity cannot be made. A larger test program focusing on this aspect is planned. Still, based on the experimental results, the additional stainless steel rebars could have been omitted. Furthermore, based on the most recent results on C-profiles as shear reinforcement [43], the non-structural C-profiles right at the edges (Figure 7, left) could have been utilized for shear verification. Yet, despite the additional costs for the stainless steel, the client decided to put TRC to use for this bridge. The result is a slender appearance of the deck slabs in the finished bridge (Figure 10).



Figure 10—Site view after installation of TRC deck panels prior to fastening of steel handrail (left, photo: Jan Bielak) and with service vehicle after inauguration (right, photo: RS Ingenieure GmbH & Co. KG)

Alternative solutions such as alkali-resistant glass rebars, a reinforcement cage made completely of stainless steel or even to use full panels of glass fiber-reinforced polymers (GFRP) were discussed in advance. The reduction of maintenance cost by omitting the normally necessary polymeric coating of conventional steel-reinforced concrete was a strong argument for use of non-corrosive carbon reinforcement. The UV-resistance and the creep performance of concrete is superior to that of full-GFRP-panels. The biggest obstacle to the application of TRC in this example was the time required for the expert report and the approval and the uncertainty whether the

capacity will be sufficient. Reliable design models for TRC could remove this barrier and facilitate the use of this innovative composite material.

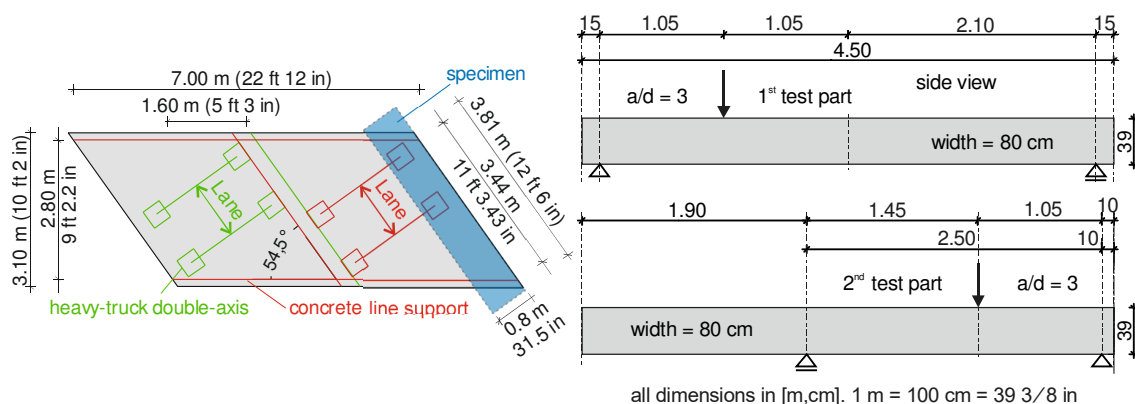
Road bridges in Gaggenau

In Gaggenau-Michelbach, Germany, two damaged bridges made of reinforced concrete and steel girders were to be replaced by new constructions (Figure 11). They are part of the subordinate street network and used for car, truck and pedestrian traffic. Both bridges cross a small creek in skew angle. To increase the maximum rate of water flow for the creek during heavy rain, the new bridges needed a larger span. Repair and retrofitting were therefore out of the question. Given this situation, the municipality decided to use the innovative material TRC without additional waterproofing and asphalt decking as it would be required for steel reinforced concrete in this situation, hoping to reduce maintenance cost.



Figure 11—Situation prior to bridge replacement in Gaggenau, Germany: a) Siedlungsstrasse; b) and c) Heilweg (photos: Jan Bielak)

Figure 12 (left) shows the structural system of that bridge which was used for determination internal forces and moments by means of a linear-elastic finite-element model for dimensioning of both bridges. The supposedly simple structural system of a single span slab is turned into a statically indeterminate skew slab by the rotation of the lane axis. Originally, five elastomer point-supports on either side were planned to transfer the loads to conventionally reinforced abutment walls, which are not discussed in the following. Prior to construction, the support was changed to a simpler concrete line bedding. As for the last two bridge examples, IMB of RWTH Aachen was assigned with verification of structural capacity in ultimate limit state with the aim of a project-related approval for use of TRC. In this case, there was enough time for experimental investigations with three-point bending tests prior to construction. Thus, slab segments of 80 cm (31.5 in) width, the same depth and approximately the same span as the actual bridges were investigated regarding their ultimate shear capacity (Figure 12, right). Each specimen could be tested twice with a shear slenderness of $a/d = 3$. Due to biweekly trespassing of the bridges by heavy waste disposal trucks in the community, parts of the tests were conducted with cyclic loading.



all dimensions in [m,cm]. 1 m = 100 cm = 39 3/8 in

Figure 12—Structural System of skew bridge (left, adapted from [60]) and test concept for one-way shear (right)

High loads of a 40 t (88,185 lb) truck lead to a shear load of $v_{Ed} = 206 \text{ kN/m}$ (14.1 kips/ft) in the control section located at a distance d (effective depth) to the face of the support. Even with a depth of 39 cm (15.4 in), this load could not be transferred without shear reinforcement. Based on previous experience with TRC as distributed shear reinforcement, preformed C-shaped textile grids made of the same material (filament cross-sectional area of $95 \text{ mm}^2/\text{m}$ (0.45 sqin/ft)) as the longitudinal reinforcement were proposed (Figure 13). The preliminary dimensioning by the reinforcement manufacturer followed the approach of [33] which led to double C-profiles

with a transversal spacing of 40 cm (15.75 in). As matrix, a conventional concrete class C50/60 with a maximum aggregate size of 8 mm (0.3 in) was used.

The specimen contained two upper and four lower layers of epoxy-impregnated bi-axial carbon grid as bending reinforcement and two double C-profiles with a leg length of 50 mm (2 in) in compression and tension chord as shear reinforcement. To enable the intersection of longitudinal and transversal reinforcement, the grid openings were matched, and the manufacturer omitted the longitudinal yarns in the lower leg of the shear reinforcement. In the compression zone, the two-layered longitudinal reinforcement was placed on top of the horizontal legs of the double-C-profiles, which thus also act as spacers (Figure 13, c).

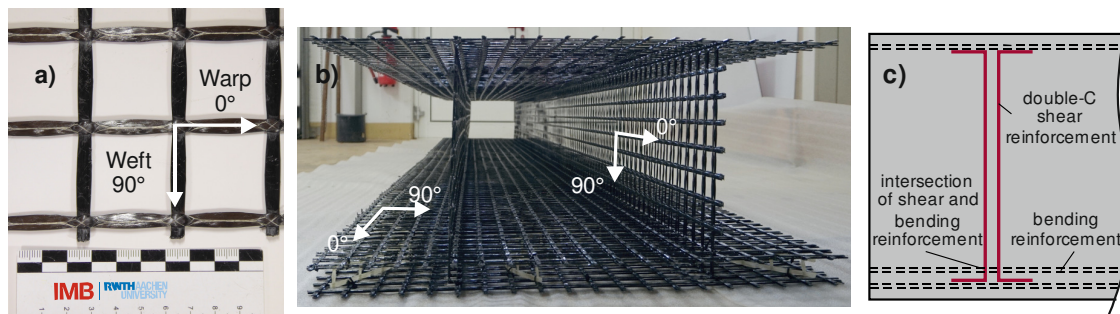


Figure 13–Reinforcement for test specimen: a) planar epoxy-impregnated carbon grid (photo: M. Schmidt); b) reinforcement cage (photo: S. Rempel) c) Schematic of shear reinforcement [60]

Figure 14 shows the shear force-deflection curves for six static and two cyclic tests without self-weight. The loading rate in the deflection controlled static tests was 1 mm/min (0.0394 in/min), and cyclic tests were performed with a frequency of 2 Hertz. The number of load cycles in the first cyclic test of 155.000 was based on the estimated number of truck crossings during the life-span of the bridges ($n \geq 100.000$) but was increased in the second test to one million cycles because no premature failure or influence on the ultimate capacity could be observed. The upper limit for cyclic shear load of 115 kN/m (0.67 kips/in) including self-weight was chosen based on the stress in the reinforcement at serviceability limit state during truck trespassing. The lower limit was set to 30 % of this load, which corresponds to the contribution of dead load to shear load.

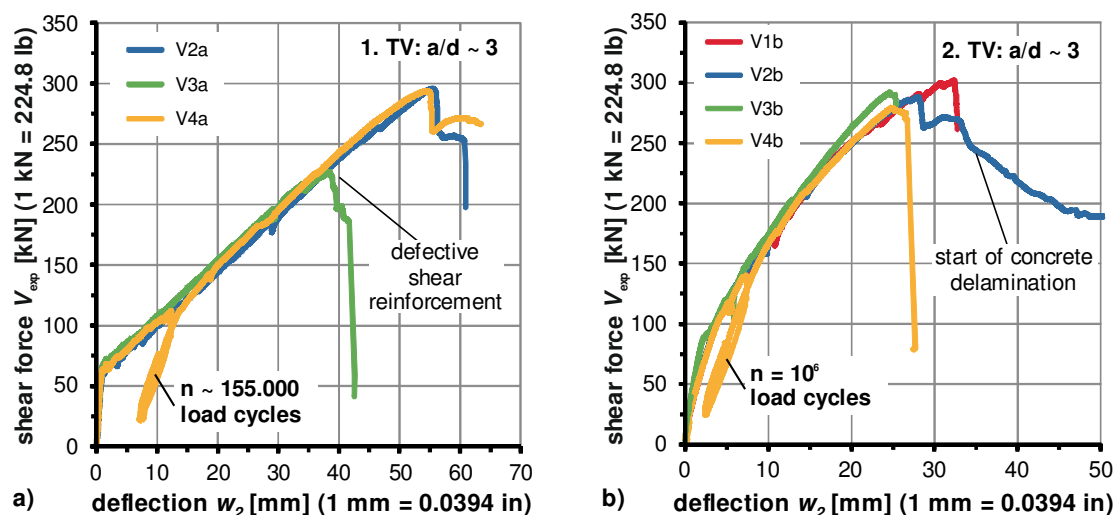


Figure 14–Experimental results from shear tests with shear reinforcement for Gaggenau bridges [60]

In seven tests, shear-compression failure with visible reduction of the size of the compression block above the diagonal tension crack could be observed (constriction of compressive chord, see for example Figure 15, definition given e.g. in [61]). A premature rupture of vertical reinforcement was neither visible nor audible. Typical for high-strength impregnated yarns would be a sound at rupture of individual yarns. One specimen was tested with a shear slenderness of 3.85 and failed in bending with rupture of the longitudinal reinforcement due to the higher bending moment.



Figure 15–Shear-compression failure of specimen V3a [60]

The ultimate capacity showed surprisingly low scatter (COV: 3.1%) for shear tests. Since no tests on the same member depth without shear reinforcement were carried out, the contribution of shear reinforcement to shear resistance cannot be quantified. However, during production of specimen V3a, one of the two double-C profiles was damaged and buckled under the pressure of concrete. The lower ultimate shear load in this test (green curve in Figure 14 a)) indicates the significant contribution. This is in accordance with results from [43], where shear reinforcement in slab segments with larger depth was more effective.

As visible in Figure 14, the cyclic loading at the selected load levels had neither a significant effect on the ultimate shear capacity nor on maximum deflection at failure. No additional longitudinal cracking in the layer of longitudinal reinforcement occurred during 1 million load cycles. For the bridges, it can be concluded that the repeated heavy truck traffic does not result in a reduction of load capacity or significant increase in deflection. This is in accordance with experimental investigations on the fatigue behavior of the same impregnated carbon yarns in uniaxial tension [62, 63].

Based on the experimental results, the applicability of TRC for short slab bridges, as alternative to reinforced concrete, without waterproofing and protective asphalt could be demonstrated. The results led to a project-related approval of TRC for the two Gaggenau bridges.

FURTHER RESEARCH

Further research should aim at providing validated shear design models for TRC, both for situations without and with shear reinforcement. Investigation of direct load introduction for concentrated and distributed loads could lead to adaption or derivation of reduction factors for shear loads near supports. The internal load distribution in transversal direction in slabs, and the width contributing to shear resistance for concentrated loads needs to be determined, depending on transversal reinforcement type and ratio. This eventually leads to more economic design of TRC slabs and enables further reduction of member thickness.

The compilation of a database allows for statistical and probabilistic analysis and eventually for derivation of partial safety factors, providing an acceptable level of reliability throughout the planned service life of the components. For this database, experiments with different reinforcement materials (glass, carbon), reinforcement shape (grid size, yarn cross-sectional area), reinforcement ratio, concrete strength and concrete aggregate size, member size, type of loading (concentrated load, distributed load) and type of structural system (single span, continuous) need to be considered. Crucial for prediction of capacity for shear-reinforced slabs is the characterization of material properties of pre-formed non-metallic reinforcement. Appropriate test methods are required to determine the strength and modulus of elasticity, taking into account the reduction of ultimate stress at bends. Additionally, the reduction of efficiency of shear reinforcement due to partial anchorage in tension and compression chord will be investigated in further research. The definition of permissible reinforcement shapes together with detailing specifications is an integral part of this study and will be addressed, as well.

SUMMARY AND CONCLUSIONS

In contrast to FRP shear design, major questions for TRC shear design for new constructions remain subject of current research, e.g. the influence of (too) good bond characteristics resulting in longitudinal splitting and delamination. For slabs without shear reinforcement, existing models such as simplified CSCT for steel- or FRP reinforced concrete, seem transferable. A systematic study on shear-reinforced slab segments indicates that planar textile grids and pre-formed C-profiles are equally suitable as shear reinforcement. They increase shear capacity despite small slab thickness through their distributed reinforcing function aligned to member direction. However, for derivation of generally valid design rules, further experiments with larger variation of material characteristics are required.

The three bridge examples from practice shown in this paper highlight the application potential for TRC both for thin slabs without shear reinforcement as secondary structural system as well as for thicker slabs as principal structural system with shear reinforcement. The use of TRC in external, weathered conditions with de-icing salt

and freeze/thaw exposition fully exploits the potential of the highly durable non-corrosive reinforcement. The successful application of the pre-impregnated textile grids during production in precast plants and on site underlines that the innovative non-metallic reinforcement for use in TRC is market ready. However, the currently necessary approvals in Germany with case-specific experimental investigations represent a substantial barrier to innovation. They are time- and cost-intensive and often prevent potential users from implementation. As a conclusion, one primary objective of research in TRC needs to be the derivation and provision of user-friendly design guidelines and standards which ensure safe, reliable, and economic construction.

ACKNOWLEDGEMENTS

This research was funded by the German Federal Ministry of Education and Research (BMBF), project C³-V-I.14, grant number 03ZZ0380. This financial support is gratefully acknowledged. The authors wish to express their thanks to Knippers Helbig GmbH, RS Ingenieure GmbH & Co. KG, solidian GmbH and the city Weinstadt for permission to use the figures, photos and experimental results from the three bridge projects.

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Performance of TRM/TRC systems under elevated temperatures and fire conditions

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Synopsis: In this paper, authors draw information from previous work aiming at contributing to: (i) a summary of the existing knowledge on the mechanical performance of TRM/TRC systems under elevated temperatures and fire conditions; and (ii) highlighting the most important issues of testing procedures and protocols. The paper refers to both TRM- and TRC-related data. The former include experimental assessments of the: (a) TRM-to-substrate residual bond characteristics as a function of temperature; and (b) structural performance of fired masonry elements furnished with a new type of TRM-based overlay for combined structural and energy upgrading. TRC-related data focus on the structural degradation of thin TRC plates under both standard and non-standard fire conditions.

Keywords: TRM, TRC, elevated/high temperatures, fire

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INTRODUCTION

During the past fifteen years, Textile Reinforced Mortar (TRM) systems have increasingly been gaining ground as a means of strengthening damaged or seismically deficient masonry structures. This is in addition to the repair of reinforced concrete structures (yet, to a lesser extent). On the other hand, the use of Textile Reinforced Concrete (TRC) has produced a wide spectrum of prefabricated products encompassing both architectural (non-loadbearing) and structural (loadbearing) elements. Promotion of these materials is based on arguments regarding their superior performance under elevated/high temperatures when compared to more "traditional materials" such as Fiber Reinforced Polymers (FRP) and concrete for strengthening/rehabilitation works or new construction, respectively. It is not uncommon for some practitioners to erroneously perceive TRM/TRC systems as fire-resistant ones. Although properties possessed by TRM/TRC such as nonflammability and incombustibility comprise an obvious advantage in regards to composites with organic matrices little is known on the actual behavior of these materials under elevated/high temperatures and fire conditions as pertinent experimental results are scarce.

Elevated versus high temperatures

Currently, no clear distinction between "elevated temperatures" and "high temperatures" is defined, although, the available literature uses 300°C (572°F) as a provisional boundary. On the other hand, some researchers use the terms indistinguishably. Exposure to "elevated" temperatures may account for a plethora of practical situations – not necessarily including fire scenarios – such as: (i) hot weather exposure (where concrete surface temperatures close to 50°C (122°F) or higher can be recorded [1], (ii) exposure to climatic conditions inside nuclear reactor containments where the maximum allowed (accident-inflicted or short-term) surface temperatures on concrete elements is set to approx. 175°C (347°F) [2] and (iii) chemical and metallurgical industrial applications in which the (non-refractory) concrete is in close proximity to furnaces [3]. Realistic exposure conditions to higher (or much higher) temperatures – also, under higher temperature rise rates – are frequently the result of semi- or fully-developed fire or blast events.

Test protocol and material-related parameters in thermal testing

In order to understand the behavior of cement-based TRM/TRC under thermal load (either as a standalone material or in combination with others) several applicable points regarding heated concrete should be considered. These points are the main source of data scatter and difficulty in combining results from different experimental campaigns concerning the same (nominal) material. More specifically, the following factors should be taken into consideration (see also [4]):

- testing regime:
 - loading-heating-time sequence
 - boundary conditions

- initial moisture content and moisture exchange conditions (for all materials when TRM-strengthened elements are tested)
- geometry and size of specimens.

Tests may be carried out under different thermal modes; for example, under transient temperatures during heating (or re-heating) and cooling (“thermomechanical testing”); or steady state temperatures (after exposure to elevated/high or ambient ones). The testing may take place either immediately after heating-cooling or after a longer post-cooling period. The rate of heating (and/or cooling and in conjunction with the size of the specimen) also plays an important role in simulating realistic heating conditions with the aim to extract meaningful results. Moisture content and moisture exchange conditions between specimens and the environment comprise additional parameters that should be taken into account during comparative studies of different test campaigns. Finally, loading conditions present during heating/cooling alter the deformation response of heated specimens by introducing load induced thermal strains.

- types of TRM/TRC, and substrates for TRM-strengthened elements:
Matrix formulation as affected by binder(s), chemical admixtures, mineral and/or polymeric additives as well as aggregates, mix proportions, and curing regime are important parameters. Types of textiles used depending on fiber material(s), fiber sizing, yarns’ treatment, fabrication method, geometry, and production/application method (wet lay-up, sprayed, extruded). Hence, the term “TRM/TRC” is admittedly a generic one capable of describing a multitude of inorganic-matrix composites with a degree of complexity larger than the one of concrete. The temperature-dependent physico-chemical alterations of the paste, aggregates and fibers and the interactions between them are unique for each type of TRM/TRC and define the thermal response of the composite.

Exposure of TRM/TRC systems at elevated/high temperatures and fire conditions: current map of test campaigns

Although there exist some areas of rather limited knowledge regarding the mechanical response of TRM/TRC systems (e.g. bond efficiency under cyclic loading conditions for TRM or TRC/substrate interfaces), researchers’ interest has started to turn to the investigation of the mechanical performance of such systems under or after exposure to harsh environments (e.g. RILEM TC IMC – “Durability of Inorganic Matrix Composites used for Strengthening of Masonry Constructions”). In line with this trend, some investigations are relevant to exposure at elevated/high temperatures and fire conditions. These include material characterization tests (on tensile and flexure coupons for the derivation of critical mechanical properties under both transient and steady-state thermal conditions) [5-22, 24, 26], bond tests [23-28] and loading tests on TRM-strengthened structural elements [29-38]. (NB: Bond tests can also be regarded as material characterization ones; nevertheless, a distinction is made herewith.) The bar chart in Fig. 1a provides an annual overview of published data as per the previous classification spanning over the last ten years. Most of the experimental campaigns (~55% of total, see Fig. 1b) concern material characterization tests. These tests can provide meaningful data for the design of new TRC structures; in the case of designing a structural strengthening intervention with TRMs, though, such information is qualitative since the thermal inertia of the assemblage (TRM plus existing substrate on which the TRM product is externally bonded) is quite different from that of a standalone coupon (see also [22]). Furthermore, moisture boundary conditions are distinctively different. TRM-to-substrate bond tests (and, provisionally, single-yarn pull-out ones) during or after exposure to elevated/high temperatures or fire provide useful information for the design of structural strengthening interventions with TRMs under these conditions.

Material characterization tests

Review of the published data on material characterization tests reveals the large variability in test protocol parameters. The employed thermal modes – that is whether testing takes place “during heating/while hot” or “after heating/cooling” – provides an obvious categorization criterion. Sound comparisons between results from different test campaigns should be made at least on the basis of the same thermal mode adopted. Table 1 lists such (and other) test protocol parameters for the case of material characterization and bond tests found in literature. For example, in [6] and [24] the post-heating/cooling residual tensile properties of TRM coupons were assessed. The types of fiber material (AR glass) and mortar (cement-based) were common between the aforementioned studies. Differences in test protocol parameters pertained to (see Table 1): heating/cooling regimes, threshold temperatures, textile geometry/areal weight, mortar strengths and (more importantly) tensile test configurations employed (clamping grip and clevis-type for [6] and [24], respectively). These differences hinder the derivation of safe conclusions in respect to strength losses as a function of temperature. Furthermore, that clamping grip pressure along heat-stricken end parts of specimens might result in premature cracking and failure.

As a general remark, it can be said that the residual tensile strength of AR glass TRM coupons previously heated at 400°C (752°F) is less than half of the tensile strength of unheated specimens. The same temperature threshold (400°C/752°F) seems to signify an appreciable material degradation in heated/cooled carbon fiber TRC double-sided pullout test specimens [7]. Although the strength loss in single-yarn pull-out load for TRCs furnished with dry (uncoated) carbon yarns was moderate (25%), the respective loss for polymer-coated carbon ones was almost complete; according to the authors, this was attributed to the polymer coating's thermal decomposition. An interesting finding of the investigation was that after exposure to relatively moderate temperatures [up to 200°C (392°F)] post-heating/cooling pull-out strength of polymer-coated carbon yarns was either substantially increased [by approx. 30% at 100°C-150°C (212°F-302°F)] or minimally reduced [at 200°C (392°F)]. Nevertheless, results' variability in [7], for an exposure temperature range of 100°C-150°C (212°F-302°F), increased to a level comparable to the reported pull-out strength increase ratios. The beneficial thermal effect of organic coating to the post-heating/cooling mechanical response of carbon fiber TRCs under uniaxial tensile loading was confirmed by [26] for temperatures up to 120°C (248°F). It was postulated by the authors [7] that this effect was owed to "matrix-polymer interlocking mechanism of molten and re-stiffened polymer, which formed a high-performing bond between filaments and matrix".

The influence of fiber yarns' coating on the residual post-heating/cooling tensile properties of TRMs has also been investigated by [14]. In this study, AR glass textiles were coated using two types of epoxy polymers; matrix comprised a lime-based mortar. Exposure temperatures up to 250°C (482°F) were opted for. For the coatings employed, the authors conclude that the resin-degradation mechanism at the textile-to-matrix interface is the governing process during heating as opposed to further cross-linking. Generally, the extent to which each mechanism is activated is polymer-dependent. Under transient or constant temperature exposure conditions, the influence of fiber yarns' coating on the tensile properties of TRMs is completely different. As reported by [10], TRCs with latex-coated basalt textiles exposed and tested in hot conditions show a progressive loss of the load-bearing capacity. As temperature increases, a significant reduction of the tensile strength, elastic modulus and strain capacity is observed. Values for the ultimate tensile strength obtained under hot conditions were, on average, about 50% lower than the residual ones. The detrimental effect of organic coatings/impregnation means on the during-heating tensile properties of textile-reinforced mortars is also verified by [26] documenting that epoxy-impregnated carbon textiles resulted in an ultimate tensile capacity reduction of the composite in the order of 70% when tested at 80°C (176°F) and 120°C (248°F). Thermal decomposition of the organic coating/impregnation means and matrix dehydration and conversion reactions are substantiated in the literature by thermogravimetry, X-ray diffraction, SEM and XRD analyses.

As a general remark, in both conditions, hot and residual, textile-reinforced inorganic matrices exhibit strain hardening behavior up to approximately 400°C (752°F) [10, 13]. When tested during heating, these materials show an increase in deformation (see [10]) the heating rate dependency of which is a factor of exposure temperature.

The next sections present a more elaborate presentation of experimental assessments of the: (a) TRM-to-substrate bond characteristics as a function of temperature; and (b) structural performance of fired masonry elements furnished with a new type of TRM-based overlay for combined structural and energy upgrading. Emphasis is given on masonry substrates as TRM-based strengthening solutions are best suited for this type of structures. For (a), both clay brick and concrete substrates are considered as they represent common masonry block types.

TRM-TO-SUBSTRATE RESIDUAL BOND CHARACTERISTICS AS A FUNCTION OF TEMPERATURE

Bond capacities of both the TRM-to-substrate and the textile-to-matrix interfaces often comprise an upper limit strength affecting the efficient use of this composite material. For this reason, the investigation of the effect of individual parameters on bond behavior has received appreciable attention from the academia through a number of experimental studies. Nevertheless, there are only a limited number of studies addressing the effect of elevated or high temperatures on the bond characteristics of TRM systems.

TRM-to-masonry interfaces

In Maroudas and Papanicolaou [24], a TRM system (of various bond lengths) consisting of a single-layer dry AR glass fiber textile embedded in a polypropylene fiber-reinforced cementitious matrix was applied on masonry wallettes made of solid clay fair-faced bricks. Single-lap/single-prism shear bond tests were conducted after specimens' exposure at 100°C, 200°C, and 300°C (212°F, 392°F and 572°F) for 1 hour [heating rate from ambient to target temperature being equal to 75°C/min (167°F/min)]. Prior to subjecting the specimens to the heating regime each specimen was dried in an electrical oven for a duration of 30 min at 40°C (104°F). After completion of the heating process specimens remained in the switched-off oven (with a small side aperture open to prevent condensation of any

moisture left in it); oven's door remained closed until 50°C (122°F) were reached. The authors noticed a temperature-dependent shift in failure modes from fibers' slippage at 20°C (68°F) and 100°C (212°F) to strip debonding and fibers' rupture at 200°C (392°F) and 300°C (572°F). It was concluded that (for adequate bond lengths) the residual shear bond load that can be undertaken by this type of TRM/masonry interfaces after exposure at 100°C, 200°C, and 300°C (212°F, 392°F and 572°F) amounts, respectively, to 65%, 60%, and 50% of the respective load reached in ambient conditions without prior heating/cooling.

In a recent work by [28], lightweight matrices were introduced for the production of TRMs in order to investigate whether a heat-shielding effect to the fibrous grid could be established combined with an improved post-heating/cooling residual bond strength of TRM/masonry interfaces. To this purpose, masonry wallettes (comprising stack-bonded, ridge-faced, perforated fired clay bricks) were unilaterally furnished with TRM strips comprising two systems; both shared the same type of textile (a dry AR glass fiber textile – different from [24] – either in a single-layer or in a double-layer configuration) and different matrices: one normal weight (TRNM) and another lightweight (TRLM, with pumice sand) of equal compressive strengths. Prior to subjecting the specimens to the prescribed heating regime, each one was dried in an electrical oven for a duration of 24 hours at a constant temperature of 40°C (104°F). Until testing, all specimens were stored in a way that ensured zero moisture exchange with the atmosphere. Single-lap/single-prism shear bond tests were conducted after specimens' exposure at 120°C and 200°C (248°F and 392°F) for 1 hour [heating rate from ambient to target temperature being equal to 75°C/min (167°F/min)]. In this case, the failure mode (fibers' slippage from within the mortar) was common for all exposure temperatures and for both types of matrices. It was shown that at control conditions (non-heated specimens) and after exposure at a nominal air temperature of 120 °C (248°F), both single-layer TRM systems (TRNM and TRLM) exhibit similar bond capacities. Nevertheless, after exposure at a nominal air temperature of 200°C (392°F), both single-layer and double-layer TRNM overlays outperformed the respective TRLM ones (by 15%-20%, in terms of maximum axial textile stress). This was mainly attributed to larger quantities of evaporable moisture in the lightweight matrix as opposed to the normal weight one buffered in the pumice sand which was used in a saturated state during mixing. Moisture evaporation by heating caused a stress buildup in the dense cement paste due to high vapor pressure. Exposure temperature increase prior to testing had a minor effect on the residual bond capacity of single- and double-layer TRNM overlays. The same did not apply for TRLM overlays. In this case, the residual textile axial stress at maximum was found to be equal to 65% and 75% of the respective value reached in ambient conditions for single- and double- layer overlays, respectively.

Various TRM systems were applied on solid clay brick masonry wallettes by [27]: one made of PBO fiber textile and a cement-based matrix while two other were made of dry basalt fiber textiles (of different areal weight and grid spacing: a 'heavy' and a 'light' one) and a lime-based mortar. Single-lap/single-prism tests were conducted after specimens' exposure at 100°C, 150°C, and 200°C (212°F, 302°F and 392°F) for 3 hours [heating rate from ambient to target temperature being equal to 2°C/min (35.6°F/min)]. After heating, specimens were left to cool down freely to ambient temperature. Results indicated that the bond response of previously heated specimens depended on the fibers' type and textiles' density which in turn resulted in different failure modes. For basalt-based TRMs, the failure mode was system-dependent ('heavy' or 'light' textile) and temperature-independent. More specifically, specimens furnished with the 'heavy' basalt textile failed due to interlaminar shear (shearing-off of the top mortar layer at the textile/top mortar layer interface) whereas specimens with the 'light' basalt textile failed due to fibers' rupture along the unbonded length of the textile. For PBO-based TRMs, the failure mode was temperature-dependent: from interfacial shear failure at control conditions and after thermal treatment at 100°C (212°F) to textile rupture after thermal treatment at higher temperatures. The residual shear bond load decreased by up to 50% and 90% in the case of PBO-TRM and 'heavy' basalt-TRM, respectively, while it remained almost unaffected in the case of the 'light' basalt-TRM ('light' having half the areal weight of 'heavy').

Furthermore, a numerical model for the simulation of TRMs' bond behavior under different temperatures was proposed by [26]. For the calibration of the model, double-lap/single-prism shear bond tests were conducted (prisms, in this case, standing for single clay bricks). The TRM systems comprised textiles with either uncoated (dry) or epoxy-impregnated and sanded carbon fiber yarns embedded in a cement-based mortar. Specimens were tested under two different regimes: (i) while, at 120°C (248°F), being conditioned to the same temperature for a duration of 100 min, and (ii) after exposure at 120°C (248°F) for 60 minutes and subsequent cooling at ambient temperature. It was shown that – when tested under the former heating regime – TRMs with impregnated yarns lost more than half of their bond capacity while TRMs with dry yarns remained almost unaffected. However, TRMs with impregnated yarns retained almost all of their initial bond capacity when tested under the second heating regime (which is in line with the findings of [7]). Slippage of the fibers from within the mortar was the common failure mode for all specimens.

From the studies referenced above the following general remarks can be made:

- Thermal history (“after heating/cooling” thermal mode) typically consists of a heating ramp [heating rates ranging from 2 to 75°C/min (35.6 to 176°F/min)], a thermal plateau [max air temperature examined = 300°C (572°F), steady-state duration ranging from 60 min to 180 min] and an uncontrolled cooling branch.
- Each TRM/masonry joint type examined thus far is unique in terms of constituent materials and thermal response.
- Change of a single material-related parameter (e.g. mortar type or textile weight of the inorganic matrix composite) - while keeping the rest of test protocol parameters unchanged - may lead to different residual bond strength ratios (= post-heating/cooling bond strength / bond strength of unheated system at ambient temperature conditions).
- Failure modes can be temperature-dependent but there is no clear indication of the factors controlling this failure mode shift. Whenever fibers’ rupture along the unbonded part of the TRM strip is evidenced the efficiency of the thermal protection means applied on the bare textile while heating the specimen should be questioned.

TRM-to-concrete interfaces

The first study discussing the thermal effects on the shear bond performance of TRM/concrete interfaces was that of [23]. In this work, single-layered and double-layered TRM overlays consisting of PBO textile and a cement-based matrix were applied on concrete prisms. To ensure a good bonding laitance was removed by sandblasting. Single-lap/single-prism shear bond tests were conducted after specimens’ exposure at 50°C and 100°C (122°F and 212°F) for 8 hours and subsequent uncontrolled cooling. Results indicated that the bond response of unheated specimens depended on the number of textile layers (fiber/matrix slippage and interlaminar shear failure or debonding at composite/concrete interface for single- and double-layered configurations, respectively). Thermal conditioning induced a switch in failure mode for single-layered systems only. This finding (i.e. the thermal dependency of the failure mode of single-layered PBO TRM systems) is in agreement with [27]; nevertheless, the temperature-dependent order of reported failure modes differs between [23] and [27] probably also owing to the different substrates employed in these studies. The bond capacity of the double-layered TRM strips was reduced for both exposure temperatures [by 27% and 38% at 50°C (122°F) and 100°C (212°F), respectively]. The capacity of single-layered strips was negatively affected (by 26%) only for the higher exposure temperature.

Both steady-state and transient thermal conditions were applied in [25]. In this case, TRM strips with three or four dry carbon textile layers were applied on concrete prisms through a polymer-modified cementitious mortar for the realization of single-lap/double-prism tests. As in [23], a special concrete surface treatment preceded TRM application. In this campaign, composite-to-substrate bond enhancement was achieved by laitance removal in combination to the creation of a 50-mm (1 ³¹/₃₂-in) mesh of grooves with a depth of approximately 2-3 mm (⁵/₆₄ - ¹/₈ in). Two testing procedures were followed. According to the first one, specimens were heated at a target temperature [50°C, 75°C, 100°C, 150°C, 200°C, 300°C, 400°C, and 500°C (122°F, 167°F, 212°F, 302°F, 392°F, 572°F, 752°F, 932°F)] for 60 minutes and they were then tested under the same temperature. These specimens were not seriously affected in terms of TRM bond capacity up to a temperature of 400°C (752°F) while their bond load was less than or equal to half of the untreated specimens’ load for a temperature of 500°C (932°F). According to the second procedure, specimens were loaded up to a certain percentage of their bond strength (25%, 50%, and 75%) at ambient conditions and they were then heated up to failure (temperature at failure: approximately 300°C (572°F) when stressed to 25% and 50% of their bond strength and approximately 75°C (167°F) when stressed to 75% of their bond strength). For both thermal modes adopted and irrespective of exposure temperature, failure was due to the cohesive debonding at the TRM/concrete interface. Same with the findings of [23], failure of high-rigidity TRM/concrete joints due to in-plane loading results due to a temperature-independent debonding of the overlay from the substrate. Finally, [25] conclude that the bond behavior of TRM/concrete interfaces is sensitive to the thermal mode they are subjected to: steady-state conditions result to considerably higher bond strengths (for nearly the same temperature) in respect with the transient ones.

Residual bond strength versus temperature

Fig. 2 is a scatter plot of all residual bond strengths (RBS) of TRM overlays referenced in literature after heating/cooling regimes versus the target air temperature at which the TRM/substrate assemblages were exposed. RBS values are given as a percentage of the bond strength reached by these systems at ambient temperatures without any prior heating/cooling. Different combinations of TRM/substrate joints are included (namely, those investigated in

[23], [24], [26], [27] & [28]). Failure types of the majority of specimens involved fibers' slippage from within the matrix. In an attempt to provide an indicative (and rather crude) lower bound prediction of RBS as a function of temperature a decay law is proposed (Fig. 2). It must be noted, however, that should this "material-blind" function be adopted bond strength reduction predictions might well be overestimated. Furthermore, each data point in Fig. 2 is the result from either a single specimen or represents the mean value of a very limited number of specimens (2 specimens). Taking into account the inherent variability of shear bond tests, more specimens per test group should be tested in order to increase the reliability of data produced. Finally, differences in the heating/cooling history and shear bond test configurations adopted by the different experimental campaigns increase the level of uncertainty in estimating a "worst case scenario" RBS. Until an elaborate theoretical prediction is proposed, designers are encouraged to rely on material-specific experimental data taking into account all specificities related to the experimental campaigns producing the relevant data.

FIRE TESTS ON MASONRY ELEMENTS FURNISHED WITH A NEW TYPE OF TRM-BASED OVERLAY FOR COMBINED STRUCTURAL AND ENERGY UPGRADING

Fire tests on TRM-strengthened medium- or large-scale masonry elements are lacking. In [33] the authors introduce, for the first time, a combination of structural (seismic included) and energy retrofitting means into one system. The new system (Fig. 3) comprises a TRM composite combined with an insulating material in a single unit, either in the form of a prefabricated board or constructed in situ. Several configurations (schemes of TRM and insulation alternating courses) may be formulated subjected to structural, constructional, functional and aesthetical requirements; in this sense, the TRM may be placed outside the insulating material, inside, that is between the insulation and the masonry wall, or on both faces (outside and inside) of the insulation. Unilateral or bi-lateral application of the system to masonry-infilled frames and unreinforced masonry (URM) walls is possible. The mechanical response of fired URM walls furnished with several alternatives of the proposed system was also investigated in [33]. Specimens underwent one-sided standard fire exposure as per [39] for 90 min in a fit-for-purpose fire resistance furnace and were subsequently tested under cyclic out-of-plane loading in ambient conditions (cooling taking place in the furnace after the end of the heating regime).

Thermally unprotected TRMs (that is combined with a layer of insulating material sandwiched between the TRM and the wall) reached 870°C (1598°F) at an exposure (furnace) temperature of 1006°C (1843°F) and were severely damaged. Compared to the unfired counterpart specimens fired ones of this type exhibited zero or minimal flexure strengths (less than half of the unfired). Heat shielding of the TRM achieved when the insulation comprised the external (exposed) layer of the system. The degree of heat shielding and the level of TRM damage relied on the thickness of the insulating material. Walls receiving systems that provided adequate thermal protection to the TRMs retained most of their unfired flexural strengths.

CONCLUDING REMARKS

Although there exists a rather limited number of publications discussing the mechanical performance of TRM/TRC systems under elevated temperatures and fire conditions – in respect to those regarding the mechanical response of TRM-strengthened structural members or new ones made of TRC under normal service conditions – the knowledge produced thus far is considerable. Nevertheless, the lack of commonly accepted testing protocols for the investigation of the heating effects on the composites in question renders comparisons between different experimental campaigns difficult or even invalid. Therefore, drawing of generalized conclusions on the topic (which is multi-parametric in nature) is not without risk.

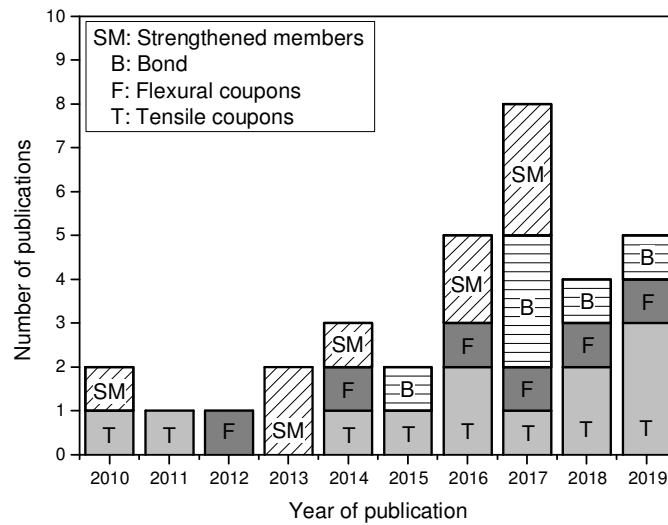
Thermomechanical material characterization tests should be revisited on the basis of intended applications of the composites under investigation. Time-temperature-loading sequences should be designed so as to correlate with realistic exposure conditions. Moisture content and moisture exchange paths is yet another parameter that deserves special attention should rational testing procedures be drafted. More specimens have to be tested per group of identical specimens in order to increase experimental data reliability. More information derived from large-scale fire testing on load-bearing elements furnished with TRM/TRC systems is needed. International scientific initiatives (such as collaborative research projects) on the matter (and especially on the issue of homogenizing testing procedures) should be encouraged as they will provide fertile ground for technological synergies and cross-pollination of ideas.

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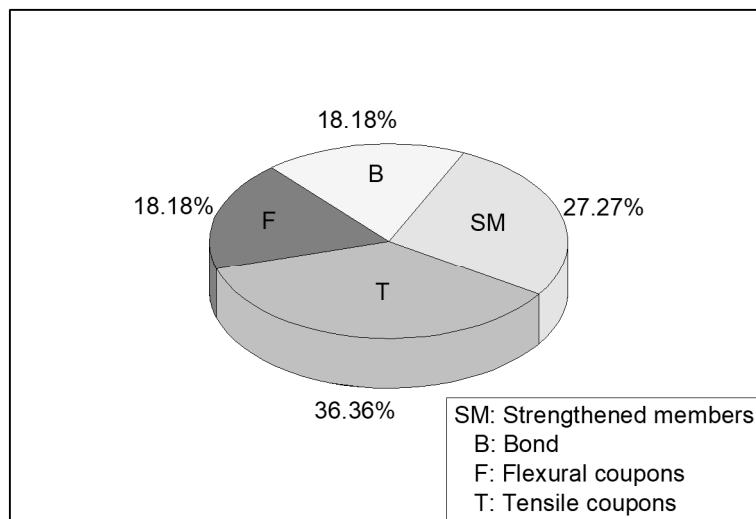
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(a)



(b)

Fig.1 – (a) Annual overview of published data relevant to TRM/TRC systems under elevated temperatures and fire conditions spanning over the last ten years; (b) Breakdown of total tests included in (a), by type of test.