Fire behaviour of GFRP reinforced concrete slabs

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Abstract: Glass fibre reinforced polymer (GFRP) rebars are emerging as a non-corrodbile alternative to steel reinforcement, especially in sea coast and industrial environments, where steel reinforced concrete (RC) often presents durability issues. However, the mechanical properties and the bond to concrete of GFRP rebars are strongly affected by high temperature, especially when the glass transition temperature of the polymeric matrix is approached or exceeded. The use of GFRP-RC structures in buildings is dependent on a proper fire behaviour, which is still not well understood in the literature. In the present study, experimental and numerical investigations were performed on steel-RC and GFRP-RC slab strips. The main goal was to characterize the fire behaviour of GFRP-RC slabs when simultaneously subjected to the ISO 834 standard curve and a service mechanical load, and to compare such behaviour with that of a steel-RC slab. For the GFRP-RC slabs, the influence of the following two parameters was studied: (i) the concrete cover (2.5 and 3.5 cm), and (ii) the existence of lap splices exposed to heat (with two different development lengths, of 30 and 60 cm). In addition, all RC slabs were subjected to flexural tests at room temperature; the GFRP rebars were also tested in tension from 20 °C up to 300 °C in order to measure their tensile properties as a function of temperature. In the numerical study, three dimensional finite element thermo-mechanical models were developed in order to predict the thermal and mechanical responses of two of the slabs tested at ambient temperature and under fire exposure. The results obtained show that GFRP-RC slabs with continuous reinforcement are able to attain significant fire resistances (about 120 min), provided that the anchorage zones of the rebars remain relatively cold; however, the presence of lap splices directly exposed to heat drastically decreases the slabs’ fire behaviour (for less than 30 min), even if the development length suggested in the design regulation considered is used. Concerning concrete cover thicknesses, for the range of values used, it did not have significant influence on the fire resistance of the GFRP-RC slabs.

Keywords: reinforced concrete, glass fibre reinforced polymers (GFRP), fire, experimental tests, numerical study.

1. Introduction

In the first half of the 20th century, the development of steel reinforced concrete (RC) revolutionized the construction industry, and it rapidly became the most used structural material. However, steel has durability issues, especially when exposed to sea and industrial environments, due to its corrodbility, thus affecting the structural and aesthetic performance of construction elements. Glass fibre reinforced polymer (GFRP) rebar is a non-corrodbile alternative to steel reinforcement, presenting several other advantages, namely the higher tensile strength, the lower weight and the electromagnetic transparency. However, the behaviour of GFRP rebar at elevated temperature is a matter of concern, because when the glass transition temperature of the matrix is reached (usually between 40-120 °C), its mechanical properties are severely deteriorated, thus affecting the rebar's tensile strength and modulus and also the bond to concrete. Consequently, and because few information is available in the literature, most design guides recommend, as a conservative approach, not using GFRP rebar in structures susceptible to fire occurrence, i.e., in buildings.

Bisby et al. [3] conducted experimental tests on two GFRP-RC slabs with different types of coarse aggregates: carbonate and siliceous. The results showed that the slab with carbonate aggregates performed slightly better in fire, reaching temperatures about 10% lower than the slab with siliceous aggregates. Based on these results, the authors developed numerical parametric studies in slabs with those two aggregates, and also with expanded shale aggregates. The results showed that for an indicative cover of 50 mm the slab with expanded shale aggregates would attain a fire resistance of 85 min, followed by the slabs with carbonate (72 min) and siliceous (65 min) aggregates.

Saaëf [4] developed an analytical study about the influence of the reinforcement type on the fire behaviour of RC beams. It was assumed that the beams were subjected to a service load during the tests and that failure would occur when the beams’ resistance was not enough to support the service load. A RC beam with steel rebars failed after 100 min of fire exposure, while a carbon fibre reinforced polymer (CFRP) RC beam attained 60 min of fire resistance. Aramid fibre reinforced polymer (AFRP) and GFRP-RC beams collapsed after 40 min of fire exposure. Bisby et al. [1] performed experimental tests in slabs with three different reinforcements: steel, CFRP and GFRP. The temperature measured on top of the longitudinal rebars was significantly higher in the steel-RC slab, compared with the CFRP-RC and GFRP-RC counterparts. According to the authors, these differences are due to two main reasons: (i) the steel’s higher thermal conductivity, and (ii) the endothermic decomposition reaction of the CFRP and GFRP resin matrices, which slows the rise of temperature in the reinforcement.

Nigro et al. [5, 6] performed fire resistance tests in 6 full scale slabs (3.0 m span and 0.18 m thick), in which two different anchorage schemes of the rebars were analysed: straight and bent. For each scheme, 3 slabs were tested with varying load levels. The failure mode of all slabs with straight...
anchorage was due to the rebars’ pull out, while the slabs with bent anchorage failed due to bar rupture, after a much higher period of exposure. The authors justified the results with the higher bond between rebars and concrete provided by the bent scheme, due to the lower temperatures at the extremities of the slabs. This is the only study available in the literature about this very relevant subject.

McIntyre et al. [7] developed an experimental study in steel, GFRP-RC and CFRP-RC beams, with a single rebar, tested in two different schemes: continuous and with a lap splice at midspan. All three beams with continuous reinforcement reached the limit established for fire exposure (60 min). However, regarding the beams with lap splice, only the steel RC beam reached that limit, while the GFRP-RC and the CFRP-RC beams failed after, respectively, 17 min and 7 min of fire exposure. These results show that this constructive detail is a matter of concern regarding the fire behavior of FRP-RC members. This is the only study available in the literature about the influence of lap splices in the fire resistance behaviour of FRP-RC members; yet, a single overlap length was tested.

Schmitt et al. [8] tested thin concrete panels reinforced with GFRP rebars in flexure, subjected to a high and constant temperature of 210 °C. The panels were reinforced with bars from two different manufacturers presenting very similar thermo-mechanical properties, but different ribbed external surfaces, resulting either from cuts done after the resin’s cure (ComBAR) or directly from the manufacturing process (FireP). According to the authors, the rebars’ external surface has a significant influence on the panel’s high temperature behaviour in terms of failure modes and stiffness. The panels reinforced with ComBAR rebars performed better than those with the FireP ones. McIntyre et al. [7] also exposed to fire beams with a single rebar, from two different manufacturers, with similar thermal properties but slightly different mechanical properties. Additionally, the rebars had different external surfaces, namely fine and coarse sand coatings. The results showed that in these specific tests the beams reinforced with sand coated rebars performed better than those with fine sand coating.

The brief literature review presented above prompts the following main remarks: (i) concrete cover has significant influence on the thermal response of GFRP-RC elements; however, no studies were found regarding its influence on their mechanical behaviour; (ii) concrete elements cast with carbonate aggregates perform slightly better in fire than those with siliceous aggregates; (iii) the slabs’ overall thicknesses does not seem to have significant influence on their fire behaviour; (iv) in the few comparative studies available, CFRP-RC elements performed better than those with GFRP rebars; yet, the results were not fully conclusive; (v) the bond between concrete and FRP reinforcement has high influence on the fire behavior; bent rebars in the far end anchorage may increase significantly the members’ fire resistance; and (vi) lap splices in FRP rebars seems to remarkably affect the fire resistance of FRP-RC beams.

This paper presents further experimental and numerical investigations on the fire behavior of GFRP-RC slab strips. Fire resistance tests were performed in GFRP-RC slabs, in which the influence of the rebars cover and of using lap splices with different overlap lengths was analysed. A reference steel-RC slab was also tested. Additionally, four-point bending tests at room temperature were performed in similar slabs, as a reference. The experimental study comprised also dynamic mechanical analyses (DMA) on the GFRP material, to determine their glass transition temperature, and tensile tests on the GFRP rebars to characterize the influence of high temperature on their tensile strength and elasticity modulus.

2. Experimental investigations

2.1. Test programme

Fire resistance tests and flexural tests at ambient temperature were carried out to evaluate, respectively, the fire and flexural behaviour at room temperature of GFRP-RC and steel-RC slabs. For both types of tests, 5 slabs with the following characteristics were tested: (i) reinforced with straight continuous GFRP bars and 2.5 cm of cover (GFRP 2.5); (ii) similar to GFRP 2.5, but with 3.5 cm of cover (GFRP 3.5); (iii) similar to GFRP 2.5, but with lap spliced rebars at midspan with development length of 30 cm (GFRP E30); (iv) similar to GFRP E30, but with 60 cm of development length (GFRP E60), and (v) RC 2.5, similar to GFRP 2.5, but with steel rebars bent at 90° in both far ends.

2.2. Materials

Sand coated GFRP rebars with 10 mm of diameter produced by Hughes Brothers (model Asian 100) were used in this study. Tensile tests were performed in the GFRP rebars, according to [9, 10], at the following temperatures: 20, 50, 100, 150, 200, 250 and 300 °C. The results in terms of tensile strength and elasticity modulus are presented in Fig. 1. The glass transition temperature of the GFRP rebars was determined through DMA, developed according to [11], indicating a glass transition temperature of 98 °C (based on the onset of the storage modulus).

Steel rebars type A500 were used in slabs RC 2.5. No tests were performed to determine their properties. However, both the elasticity modulus (200 GPa) and the yielding tensile strength (585 MPa) often present reduced scatter and hence were assumed according to [12].

Concrete class C25/30 with cement type CEM II/A-L 42.5R and calcareous aggregates was used. The concrete compressive and tensile properties were determined at the age of 135 days, when the flexural and fire resistance tests were carried out. Compressive and splitting tensile strength tests were performed according to [13] and [14], respectively, providing the following average values: cubic compressive strength of 53.3 MPa and splitting tensile strength of 2.8 MPa. It is worth mentioning that during the first 50 days of age, the test specimens were subjected to
different curing conditions than the slabs; in fact, they were kept in a controlled environmental chamber (20°C of temperature, 100% of relative humidity), while the slabs were kept at the lab’s facilities during the whole period, i.e., at room temperature and relative humidity (not controlled).

2.3. Geometry of slab strips

The slab strips tested were 1.50 m long, 0.25 wide and 0.11 m thick. The internal reinforcement consisted of 3 longitudinal rebars with 10 mm and 6 mm of diameter, respectively, for the lower and upper layers. Reinforcement was also applied in the transverse direction, in both lower and upper layers. Fig. 2 illustrates the reinforcement schemes for the different slabs.

2.4. Flexural tests at room temperature

2.4.1. Setup and instrumentation

The flexural tests were performed in a four-point bending simply supported configuration, with two concentrated loads applied at thirds of the 1.40 m span. Regarding the instrumentation, an electrical TML displacement transducer (model CDP-100, 100 mm of stroke) was used at midspan to measure the vertical deflection of the slabs. A Novatech load cell (200 kN of capacity) was used to measure the applied load. TML strain gauges (model FLK-6-11-3L) allowed monitoring the axial strains in the longitudinal (upper and lower layers) rebars at midspan section during the tests.

2.4.2. Results and discussion

The results in terms of load vs. midspan displacement are presented in Fig. 4 for the 5 slabs tested. Up to 7 kN of load, all slabs presented similar stiffness, as expected. For higher loads, their stiffness was reduced considerably, due to cracking. In this phase, the behaviour was different in steel-RC and GFRP-RC slabs. In the steel-RC slab, up to 32 kN, the stiffness was approximately constant and then, for higher loads, due to steel yielding, the stiffness was strongly reduced and the displacement increased until flexural failure occurred, due to concrete crushing, as presented in Fig. 3 a). Regarding the behaviour of the GFRP-RC slabs after cracking, all presented approximately constant stiffness up to failure, much lower than in slab RC 2.5. These differences between steel-RC and GFRP-RC slabs were expected, due to (i) the lower elasticity modulus of the GFRP rebars compared to that of the steel rebars; and (ii) the rebars’ constitutive laws (while steel yields, GFRP has linear-elastic behaviour up to failure). The slabs GFRP 2.5, GFRP 3.5 and GFRP E60 failed due to shear (Fig. 3 b)), while slab GFRP E30 failed due to the pull-out of the rebars in the lap splices, with local spalling of part of the concrete cover (Fig. 3 c)). The displacements at failure were approximately the same for all 4 GFRP-RC slabs, even if for different loads, and were much lower than in slab RC 2.5, which exhibited considerable softening after the maximum load was attained.

![Fig. 2 – Reinforcement distribution – a) longitudinal view of slab RC 2.5; b) longitudinal view of slabs reinforced with GFRP rebars; c) cross section of slab RC 2.5; d) cross section of slabs reinforced with GFRP rebars.](image1)

![Fig. 3 - Failure modes – a) slab RC 2.5; b) slab GFRP 2.5; c) slab GFRP E30.](image2)
2.5. Fire resistance tests

2.5.1. Test setup and procedure

Fig. 5 presents a scheme of the test setup used in the fire resistance tests. The slabs (C) were placed at the top of a furnace (A) over two supports (D and E). The supports were placed on top of steel plates, connected to a steel reaction frame (H) with steel rods. Concrete weights (F) were suspended at both extremities of a load transmission beam (G), thus applying the mechanical (fire) load to the slabs. As the slabs were 0.25 m wide, the remaining area of the furnace top opening had to be covered with a thermal insulation system (B). In the following sections each component of the setup mentioned above is described in more detail.

2.5.1.1. Furnace

The fire resistance tests were performed in an intermediate scale furnace with the following exterior dimensions: 2.10 m high, 1.35 m wide and 1.20 m deep. The furnace is fuelled by propane gas, with 6 burners (3 in each wall). The walls are lined with ceramic wool, delimiting a top area of 0.95 m x 0.80 m. Fig. 6 a) illustrates the furnace without the frontal opening.

2.5.1.2. Loading system

Concrete weights were suspended in both extremities of a load transmission beam, applying two concentrated loads in the slab at quarters of its span. The weights were suspended using pulley blocks. Fig. 6 b) presents the loading system on one side of the furnace. The fire loads were calculated according to the expression (2.4) of the Eurocode 2, part 1-2 [15], resulting in the following values: (i) 20.8 kN for slab RC 2.5; (ii) 16.9 kN for slabs GFRP 2.5, GFRP E30 and GFRP E60, and (iii) 13.6 kN for slab GFRP 3.5.

2.5.1.3. Thermal insulation system

Six metallic steel modulus lined with ceramic wool were placed over the furnace’s top opening, adjacent to the slab strip being tested, as depicted in Fig. 7. Ceramic wool was also placed in front of the supports to avoid them being damaged by the heat and also to insulate the furnace.

2.5.1.4. Slab’s supporting system

The slab’s supporting system had the following parts: (i) reaction frame; (ii) steel rods; (iii) steel plates, and (iv) supports (one sliding, one fixed). The supports were fixed to the steel plates, which were suspended in the beam of the reaction frame with 4 steel rods (in each support). Another 2 steel rods (in each support) were welded to the steel plates and connected to the columns of the reaction frame. Fig. 8 illustrates the system used for the fixed support.

2.5.1.5. Instrumentation

In order to measure the temperature during the tests, the slabs were instrumented with 0.25 mm diameter type K
thermocouples. The distribution of the thermocouples was different in the steel-RC and GFRP-RC slabs, being represented, respectively, in Figs. 9 and 10. Additionally, two thermocouples were placed above the slab’s unexposed surface, in order to measure the air temperature. The midspan displacement was monitored during the fire tests with a TML DP500-E wire transducer (500 mm of stroke).

2.5.1.6. Test procedure

The fire resistance tests comprised two phases: (i) at ambient temperature, each slab was first subjected to the mechanical load, which was kept constant until the end of the test; this phase ended when the displacement stabilized (a few minutes later); and (ii) the slab was then exposed to the time-temperature fire curve defined in ISO 834 [16]; this phase ended when failure occurred or after 240 min of fire exposure (maximum limit established).

2.5.2. Results and discussion

In the following sections the results of the fire resistance tests are presented and discussed. Regarding the slab RC 2.5, it is worth mentioning that: (i) the test was interrupted after 127 min of fire exposure, due to problems in the test setup (due to the the magnitude of the slab’s displacement, the load transmission beam became in contact with the extremities of the slab, thus changing the load configuration); (ii) the transducer stopped monitoring the displacement after 115 min of fire exposure, and (iii) during the whole test, the sliding support was unable to move, due to a problem in the preparation of the setup. Concerning the test of slab GFRP 2.5, after 108 min of fire exposure the sliding support was prevented from sliding to the furnace’s opposite direction.

2.5.2.1. Temperature profiles

The temperature profiles obtained during the fire resistance tests are presented in Figs. 11 to 15. For the sake of clarity, these figures do not include the measurements obtained with several thermocouples that are not essential for the analysis. Regarding the temperature profiles measured in the slabs, the following overall remarks are prompted: (i) as expected, the temperatures decreased from the bottom part of the section to the top part; (ii) the temperatures progressively increased in all thermocouples, even if at different rates, and (iii) the thermocouples located in the slabs’ lower face (T12 in slab RC 2.5 and T10 in the remaining) followed closely the ISO 834 [16] standard curve. These results validate the temperature measurements made in the tests.

In the tests of slabs RC 2.5, GFRP 2.5 and GFRP 3.5, there were significant fluctuations in the temperatures measured above the slabs’ upper surfaces (air temperature); this was
related with the convective effect of the cooler air circulating at the furnace’s top.

In all tests, when temperature approached 100 °C, the heating rates considerably decreased (increasing again few minutes later), due to the moisture evaporation, an endothermic process. It is worth mentioning that no spalling was detected at the end of the tests (except for the slab GFRP E30, in which local spalling occurred, due to the splices and not to the vapour pressure).

The thermocouples T11, T13 and T14 for slab RC 2.5, and the thermocouples T9, T11 and T12 for the remaining slabs presented very similar temperature profiles, which confirms the effectiveness of the lateral insulation of the slabs (otherwise, the thermocouples located closer to the lateral faces would have presented higher temperatures than those located in the centre).

The thermocouples T4 and T8 for slab RC 2.5, and thermocouples T3 and T6 for the remaining slabs provided significantly different temperature profiles (reaching 100 °C in the longer tests), possibly due to: (i) different (non-uniform) temperature distributions inside the furnace; (ii) different thickness of thermal insulation between the slabs’ lower surfaces (in extremities), and (iii) inaccuracies in the positioning of the thermocouples.

The temperatures at the reinforcement level varied significantly from the directly exposed zones (centre) to the unexposed zones (supports). For example, when slab GFRP 3.5 failed, the reinforcement temperatures at midspan section and at the extremities were, respectively, 600 °C and 70 °C (below glass transition temperature). However, 10 cm away from the latter thermocouples (located in unexposed zone, but closer to the furnace), the temperature was 145 °C. This means that even though the unexposed zone was 22.5 cm long (on each side), less than half of this distance was below the glass transition temperature of the GFRP rebars.

2.5.2.2. Mechanical behaviour

The variation of the midspan deflection (measured in the second phase of the tests) for the 5 slabs tested is presented in Fig. 16.

The displacements in slabs RC 2.5 and GFRP 2.5 presented a similar variation in the first 50 min of fire exposure. Then, the displacements in slab GFRP 2.5 increased more than in slab RC 2.5, most likely due to the higher susceptibility of GFRP rebars to high temperatures; after 115 min of exposure, the midspan displacements in those slabs had increased 28 mm and 54 mm, respectively.

From a qualitative point of view, the midspan displacement vs. time curves of slabs GFRP 2.5 and GFRP 3.5 presented the same phases, characterized by different rates of displacement increase due to: (i) the varying thermal gradients throughout the slabs’ thicknesses, and (ii) the (non-linear) variation with temperature of the mechanical properties of the materials (and possibly, of the concrete-rebars bond). It is worth mentioning that, in both tests, the displacement rates increased significantly when the rebars reached 300 °C. The fire resistances of slabs GFRP 2.5 and GFRP 3.5 were similar, respectively, 148 min and 158 min, with maximum displacement increases (at failure) of 81 mm and 86 mm (L/172 and L/162, respectively, L being the span). For the range of geometries tested, the cover had little influence in the fire resistance performance.

Regarding slabs GFRP E30 and GFRP E60, both presented continuous deflection increase up to failure, which occurred after 12 min and 21 min of fire exposure, respectively, with
maximum displacement increases (at failure) of 12 mm and 28 mm.

It is worth pointing out the significant differences in the fire resistance behaviour (both in terms of time of exposure and displacement increase at failure) of the GFRP-RC slabs with continuous and lap spliced rebars. This constructive detail remarkably affected the fire resistance behaviour of the slabs.

### 2.5.2.3. Failure modes and post-fire assessment

As mentioned above, spalling of the concrete cover was not observed, with the exception of the slab GFRP E30, where local spalling occurred in the splice zones (detailed ahead). The heat exposed surface of the slabs with continuous reinforcement exhibited considerable disaggregation of the superficial concrete.

Fig. 17 a) presents the longitudinal reinforcement of slab GFRP 2.5 after removal of the concrete cover. It was possible to observe that the rebars’ resin was completely decomposed and the fibres were covered with a black carbon layer, resulting from the resin’s decomposition. Additionally, after removing the concrete at both extremities of the slab it was possible to confirm that no rebars’ slippage took place at the anchorage zones. Fig 17 b) depicts a detail of a rebar in the central part of the slab, showing that failure occurred due to tensile rupture of the longitudinal reinforcement, i.e., of the glass rovings.

![Fig. 17 – Slab GFRP 2.5 post fire assessment – a) longitudinal reinforcement after mincing; b) rebar’s failure detail.](image)

The failure mode of slab GFRP 3.5 (Fig. 18) was similar to that of slab GFRP 2.5, yet the decomposition degree of the resin seems to have been more complete than in slab GFRP 2.5 (at least, in the area inspected); indeed, the fibres had turned into white, indicating a more complete decomposition of the resin, including of the carbon black residue layer. When failure occurred, a very wide crack developed (across the entire cross section depth) and the fibres of the rebars were directly exposed to the furnace temperature.

For slabs GFRP E30 and GFRP E60 failure occurred in a similar way, suddenly, with extensive cracking in the lap splice zones (Fig. 19 a)) and massive concrete spalling in slab GFRP E30. In both tests, the post-fire assessment showed that failure occurred due to rebar slippage in the lap splice zones (Fig. 19 b)); regarding slab GFRP E60, at least one rebar was broken in the transverse direction, possibly due to the sudden displacement increase when failure occurred. Failure of the slabs GFRP E30 and GFRP E60 occurred after only 12 min and 21 min, showing the high influence of lap splices in GFRP rebars in areas directly exposed to heat (note that in the latter slab, the development length was that provided by the guidelines used for design at ambient temperature [17]).

![Fig. 18 – Failure mode of slab GFRP 3.5.](image)

![Fig. 19 – a) Failure mode of slab GFRP E30; b) Detail of rebars slippage in slab GFRP E60.](image)

### 3. Numerical investigations

#### 3.1. Modelling

Three-dimensional (3D) finite element (FE) models of two of the slabs tested (slabs RC 2.5 and GFRP 2.5) were developed using commercial package ABAQUS. Three types of analyses were performed: (i) mechanical, in order to simulate the flexural behaviour at room temperature (section 2.4); (ii) thermal, to simulate the evolution of temperatures in the slabs during the fire resistance tests, and (iii) thermomechanical, to simulate the mechanical response under fire exposure (section 2.5). The FE models had the same geometry of the slabs tested; the single exception was concerned with the bent extremities of the rebars of
slab RC 2.5, which were considered straight. In order to reduce the computational effort, only one half and one sixth of, respectively, the slabs’ length and width were modelled (symmetry simplifications). The geometry of the FE mesh was similar for both slabs (Fig. 20): eight-node brick elements (type C3D8R) were used for all materials (concrete, steel and GFRP).

Concerning the boundary conditions, the lateral faces of the slabs were considered adiabatic, since in the experiments they were insulated with ceramic wool. Heat transfer through radiation and convection were considered in the upper and lower faces of the slabs; a convection coefficient of 25 W/m²·°C was considered, according to [15]. The mechanical loads adopted in the thermomechanical analysis were the ones used in the tests, 20.8 kN and 16.9 kN, respectively, for slabs RC 2.5 and GFRP 2.5.

The temperature dependence of the properties of all constituent materials was incorporated in the models due its significant influence on the thermal and structural responses of the slabs when subjected to fire. For concrete, a classical damaged plasticity model was adopted; a perfect elastic-plastic model was adopted for the steel rebars; and the GFRP rebars were considered as linear elastic isotropic. The material properties of concrete at ambient temperature were defined based on the tests performed in the present study. The variation with temperature of both thermo-physical and mechanical properties of concrete were assumed according to Eurocode 2 [15]. Regarding the steel rebars, their thermo-physical properties were assumed according to Eurocode 3 [18] and their mechanical properties were defined according to previous tensile tests on similar rebars (which often present low scatter), while reduction with temperature was defined according to Eurocode 2. The thermo-physical properties of the GFRP rebars were assumed according to [19]. Their mechanical properties (tensile strength and elasticity modulus) as a function of temperature (up to 300 °C) were defined according to the tensile tests performed in the experimental study; for higher temperatures, the elasticity modulus was assumed to vary according to [20] (the only study available in the literature, where results present high scatter), while the tensile strength was calibrated based on the agreement between the experimental and numerical results in terms of fire resistance (the tensile strengths obtained based on such calibration procedure were similar to those reported in [20]). Due to the lack of information in the literature, a simplifying assumption was made about the bond between the different rebars and concrete: perfect bond was considered regardless of the temperature.

To simulate the flexural tests at ambient temperature, the mechanical analyses were performed under displacement control. To simulate the fire behaviour of the slabs a sequentially uncoupled thermo-mechanical procedure was performed: a heat transfer analysis (with a maximum time step of 10 seconds) was first performed to obtain the temperature distributions for a similar duration to that of the tests; next, a mechanical analysis was carried out, considering the thermal data determined in the previous step.

3.2. Comparative analysis with experimental results
3.2.1. Mechanical analysis at room temperature

Fig. 21 presents the load vs. midspan deflection behaviour of the different slabs at ambient temperature. It can be seen that there is a close agreement between experimental and numerical data, in terms of failure loads and displacements, which were generally slightly higher in the models (with the exception of slab RC 2.5). The most significant difference is concerned with the cracking loads, which were considerably higher in the numerical models (12 kN vs. 7 kN in the tests); the relative difference is probably due to the different curing procedure of the slabs and concrete specimens used for mechanical characterization – indeed, the latter were cured in a moist chamber during the first 50 days of age, while the slabs were mostly subjected to room temperature; this may have resulted in an overestimation of the mechanical properties of the slabs, namely of their cracking load.

Fig. 22 illustrates the simulated failure modes of slabs RC2.5 and GFRP 2.5 at ambient temperature, which were coincident with those observed in the experiments, i.e., concrete crushing with steel yielding and shear failure, respectively.

3.2.2. Thermal analysis

Figs. 23 a) and b) illustrate the temperature profiles obtained for both slabs, comparing numerical and experimental data. It can be seen that a general good agreement was obtained in both cases. Yet, it is worth mentioning that the slight heating rate decrease measured in the tests when temperatures approached 100 °C (namely, in thermocouple T9, due to moisture evaporation) was not captured by the numerical models, which retrieve a relatively continuous temperature increase for that range.
3.2.3. Thermo-mechanical analysis

Fig. 24 plots the variation of the midspan displacement as a function of time for slabs RC 2.5 and GFRP 2.5, comparing experimental and numerical results. Although some simplifying assumptions were made, a relatively good agreement was obtained between numerical and experimental results. Concerning slab RC 2.5, the numerical displacements were slightly higher than the experimental ones, during the whole test; the numerical model was stopped after 115 min of fire exposure without failure of the slab, as in the experiments. Regarding slab GFRP 2.5, the predicted fire resistance (130 min) was similar to that measured in the test (148 min), and so were the displacements at failure. The failure mode predicted by the numerical model, tensile failure of the rebars also matched that observed in the test.

Fig. 25 plots the variation of the ratio between the tensile stress in the reinforcement (at a given temperature) and the corresponding strength (for the same temperature). It can be seen that both slabs presented continuous increase of such ratio up to failure, except from 15 to 60 minutes in slab RC 2.5, and from 50 to 70 minutes in slab GFRP 2.5, where a very slight reduction occurred, probably due to the decrease underwent by the elasticity modulus of the GFRP rebars. Additionally, when the test of slab RC 2.5 was interrupted, the normalized strength was 0.91, meaning that it was close to failure. Regarding slab GFRP 2.5, the normalized strength at failure was already higher than 1.0 (1.22), which is consistent with the failure mode observed in the experiments; the fact that the ratio was significantly higher than 1.0 may be due to the fact that temperatures and stresses in the reinforcement were calculated assuming a linear distribution through the rebars’ thickness, which may not be a fully accurate assumption.
4. Conclusions

The experimental and numerical investigations presented in this paper about the fire response of concrete slabs reinforced with GFRP rebars allow drawing the following main conclusions:

1. For the temperature range studied (20 °C to 300 °C), the tensile strength of GFRP rebars is considerably reduced when the glass transition temperature is reached (retentions of 60% and 57% respectively at 150 °C and 300 °C), while the elasticity modulus is much less affected (87% retention at 300 °C);
2. At ambient temperature, GFRP-RC slabs present much wider cracks than steel-RC slabs, yet within the permissible values at service limit states;
3. GFRP-RC slabs can attain significant fire resistance (above 90 min) provided that the anchorage zones of the rebars remain relatively cold;
4. For the range of values studied, increasing the concrete cover in GFRP-RC slabs (from 2.5 to 3.5 cm) provided only a minor increase in fire resistance;
5. In GFRP-RC slabs, even if the development lengths recommended in design guides are used, lap splices can remarkably reduce the fire resistance (to less than 30 min), if they are directly exposed to heat (i.e., in the centre of the slabs);
6. The thermo-mechanical models developed in this paper, even though featuring several simplifying assumptions, in general were able to predict the thermal and mechanical responses of the slabs tests, at both room temperature and under fire exposure.

5. References