Experimental and Numerical Study of Early-age Cracking of Concrete Slabs Reinforced with Steel and GFRP Bars

Robert Sonnenschein1*, Natalia Gazovicova2 and Juraj Bilcik3

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Abstract

Problems due to the corrosion of steel reinforcement in concrete structures can be efficiently and economically solved by using fiber-reinforced polymer (FRP) composites. This paper presents the results of experimental tests and numerical simulations of large-scale concrete slab strips symmetrically reinforced with steel and glass fiber-reinforced polymer (GFRP) bars subjected to axial tension. The results showed that due to their lower modulus of elasticity and bond strength, the slabs reinforced with GFRP bars exhibit wider early-age cracks compared to similar slabs reinforced with conventional steel bars. In order to limit the crack width to a certain value, approximately twice the reinforcement ratio of the GFRP bars as in the case of the steel bars is required. This can particularly be a problem for watertight concrete structures of underground facilities and water tanks.

1. Introduction

In the last three decades fiber-reinforced polymer (FRP) composites have primarily been used in the rehabilitation and retrofitting of existing reinforced concrete (RC) structures. Recently, FRP composites have been used more often in new structures that are subjected to harsh environmental and loading conditions thanks to their advantageous material properties, especially corrosion resistance. The most commonly used FRP composites in new constructions, due to their low cost, are glass fiber-reinforced polymers (GFRP) compared to carbon fiber-reinforced polymers (CFRP). Limitations and problems associated with the corrosion of steel reinforcements have led to use of non-metallic reinforcements, such as GFRP (Myers and Koenigsfeld 2006). A GFRP rebar is a structural ribbed reinforcing bar made of high-strength and corrosion-resistant glass fibers that are bound and protected by a thermosetting polymer resin matrix. The polymer composite derives its mechanical characteristics from those of the fiber and the quality of the fiber/matrix interface. The high tensile strengths, resistance to electrochemical corrosion, lightweight, and nonmagnetic characteristics are the most significant advantages of GFRP compared to steel. On the other hand, some of the common drawbacks include a low modulus of elasticity, no yielding point before brittle ruptures, and a lower bond strength to the concrete. Serviceability covers many aspects of structural performance related to different design parameters. The most commonly encountered serviceability requirements in RC structures are maximum stress, deflection and control of crack widths. Cracking is normal in reinforced concrete structures subject to bending, shear, torsion or tension resulting from either direct loading or restraint of imposed deformations (CEN 2004). The control of cracking in RC structures is particularly important for three reasons, i.e., to ensure their long-term durability, water tightness, and/or aesthetic appearance.

The need for confirmation of the long-term durability of GFRP is perhaps the most critical barrier to its widespread acceptance in field applications. The durability of GFRP bars was affected by environmental effects. The most important are the effects of moist and alkaline environments, the temperature, ultraviolet radiation, polymer matrix corrosion, the thermal expansion coefficient, the presence of chlorides, and freeze-thaw cycles. Accelerated aging tests performed in aqueous solutions with high pH values at elevated temperatures are known to degrade the tensile strength of GFRP bars (Kim 2006; Chen et al. 2007). Tensile strength reductions in GFRP bars ranging from 20 to 50% of the initial values have been reported (see Table 1). The values of the reduction factors in the overestimation of the adverse effects of alkaline solutions on the durability of GFRP bars have led to the conservative design of GFRP-reinforced structures.

Benmokrane and Ali (2019) recommend that the value of the reduction factor according to ACI 440.1R-15 (ACI 2015) should be 0.85. Major studies have been undertaken to provide performance data on GFRP that has been used in several structures across the U.S. (Goora-

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1Lecturer, Department of Concrete Structures and Bridges, Faculty of Civil Engineering, Slovak University of Technology, Radlinskeho 11, 810 05 Bratislava, Slovakia.
*Corresponding author, E-mail: robert.sonnenschein@stuba.sk
2Former Ph.D student, Department of Concrete Structures and Bridges, Faculty of Civil Engineering, Slovak University of Technology, Radlinskeho 11, 810 05 Bratislava Slovakia.
3Professor, Department of Concrete Structures and Bridges, Faculty of Civil Engineering, Slovak University of Technology, Radlinskeho 11, 810 05 Bratislava, Slovakia.
norimi et al. 2016) and Canada (Mufiti et al. 2005). The results indicate that no deterioration of the GFRP took place in any of the field demonstration structures included in the studies and that no chemical degradation processes occurred within the GFRP due to the alkalinity of the concrete (Bennokrane and Ali 2019). Research by Robert and Bennokrane (2009) and by Gooranorimi and Nanni (2017) has shown that a concrete environment is generally not as harsh as the alkaline solutions used in accelerated aging tests.

Although the serviceability criteria are usually applied after the strength design, relatively lower elastic moduli mean that serviceability criteria will usually govern the design of GFRP-reinforced structures. Deflections and crack widths are typically larger in GFRP-reinforced concrete than in steel-reinforced concrete due to a lower elastic modulus (Matthys and Taerve 2000; Bakis et al. 2002; Barris 2017; Boila 2018). This is a particular problem for the watertight concrete structures of underground facilities and water tanks. RC tanks, which are among the most important structural facilities in wastewater treatment plants, are usually subjected to a uniquely difficult environment in which corrosion poses exceptional challenges (Mohamed and Benmokrane 2014; Zheng et al. 2020). In terms of the formation and propagation of through cracks, there are three design concepts for watertight concrete structures (GCSC 2017):

1) without through cracks;
2) with through cracks of limited widths (enabling the self-healing of cracks), which are achieved through the design and detailing of the reinforcement; and
3) with through cracks without any control of crack widths, which are subsequently sealed.

For the serviceability limit state of watertight concrete structures, the maximum crack width is between 0.05 and 0.2 mm, depending on the hydraulic gradient (CEN 2006). These narrow cracks may close due to autogenous healing (Edvardsen 1999; Kovler and Bentur 2009; Forth and Martin 2014).

Ghatefar et al. (2014, 2015) have dealt with the effect of the reinforcement ratio on early-age cracks in GFRP-reinforced bridge slabs. Their experiments investigated the effect of a transverse reinforcement on cracking in GFRP-reinforced cement composites from the effects of the shrinkage in different environments for three months. Shrinkage and temperature stress analyses in the GFRP-reinforced concrete slabs were also presented (Chen and Choi 2011). While the properties of steel reinforcements and their interaction with concrete are relatively well understood, the properties and interaction of GFRP bars are not fully resolved. Physical properties of the glass fibers and resin matrix, two constituents of GFRP materials, were published, for example, in fib Bulletin No. 40 (fib 2007). The GFRP properties such as the modulus of elasticity, shape of the surface, reinforcement ratio, diameter of the bar, concrete cover, and concrete strength also have an impact on crack widths (McCallum 2013). The specific properties of GFRP reinforcements and their effect on the limitation of crack widths in concrete are summarized below.

Crack-width calculations, however, include the effect of the bond between FRP bars and concrete. This is taken into account in FRP design codes and guides through the so-called bond-dependent coefficient (ks), while the interpretation of this coefficient remains ambiguous (El-Nemr et al. 2016). Generally, the bond strength of a deformed bar to concrete ensures the adhesion resistance of the interface (the chemical bond), a mechanical interlock due to the irregularity of the interface, and the frictional resistance of the interface against slipping (Yan et al. 2016). The method of the load transfer from the concrete to the reinforcement affects the behavior of the structure, i.e., the width and spacing of the cracks, deformations, anchorage length, and the minimum thickness of the concrete cover (SCCE 2015).

The mode of the bond failure of FRP bars is quite different from that of steel deformed bars. The bond strength of FRP bars is predominantly affected by the interlaminar shear strength just below the resin-rich surface layer of the bar. By ribbing or wrapping the surface, it is possible to increase the mechanical interlock and the efficiency of the friction (SCCE 2015). A minimum average height of deformations of 0.75 mm was found to develop satisfactory bond behavior with concrete (Achillides and Pilakoutas 2004). Increasing the rib depth from 0.5 to 1.5 mm, a 55% increase in bond strength is observed (Zhang et al. 2020). The majority of studies investigating the bond behavior of FRP bars in concrete used the direct pull-out test setup according to the RILEM/CEB/fib methodology (RILEM 1994). Chaallal and Bennokrane (1993) found that deformed GFRP bars had an average bond stress of 12.9 MPa, which is 62 to 84% of the bond stress of deformed steel bars. The extensive experimental studies of the effect of surface characteristics on the bond behavior of FRP bars in concrete were presented by Solyom and Balazs (2020) and Kanakubo and Yamato (2014). Baena (2010) compared the bond stress of steel and different types of the surface treatment of GFRP bars (spiral, deformed, grooved, sanded, sandblasted). As a result, the average bond strength of GFRP bars was 70 to 79% of the average bond strength of steel bars of the same diameter with an average concrete strength of 28.6 MPa.
Early-age cracks are defined as cracks that generally develop within the first seven days after the placement of concrete. It is generally accepted that the primary causes of early-age cracking of slabs are internal and external restraints to volume changes (Frosch et al. 2003; Safiuddin et al. 2018). Shrinkage is normally not considered in this context since, in this relatively short term, the drying shrinkage will be small (ACI 2007; Pane and Hansen 2008), and normal concrete has negligible autogenous shrinkage (Holt 2001). The main focus here is on the behavior of materials that result in thermal effects and external restraints from adjoining structural parts. Thermal stresses may induce early-age cracks and may further reduce the serviceability (e.g., water tightness) and durability of the structure. The occurrence of thermal cracking is one of the clear limit states to be assessed in performance-based design (Maekawa et al. 1999).

Many variables influence the width and spacing of early-age cracks, including the quantity, orientation and distribution of the reinforcement crossing the crack, the cover of the reinforcement, and the bond characteristics of the reinforcement (Gilbert 2017). A reinforcement cannot prevent cracks; yet with a proper design, crack widths are smaller and less likely to contribute to serviceability and durability problems. König and Tue (1996), Beeby and Scott (2005), Schlieke and Tue (2015), and others have dealt with the problem of minimum steel reinforcement for the control of early-age crack widths in restrained concrete elements. The reinforcement was designed for structural loading and then was checked to ensure that the steel ratio is adequate both to control any early-age cracking and to limit crack widths.

Although several strategies have been developed to mitigate moisture-linked cracking, fewer options are available to mitigate early-age thermal cracking. The ever-growing number of the massive concrete structures impels a need to establish a strategy to reduce early-age crack widths to values dictated by the autogenous healing process. The objective of the paper is to present the differences between the steel and GFRP reinforcement on early-age cracking behavior of concrete slabs under tensile stresses, simulating the effects of restrained contractions during cooling of concrete after the peak hydration temperature.

2. Experimental program

The purpose of this comparative research study was to examine the effect of different moduli of elasticity and the bond strength of steel and GFRP reinforcements as well as the effects of the reinforcement ratio on the formation and development of cracks. The experimental program was developed to evaluate the effect of the above-mentioned parameters on the spacing and width of early-age cracks in reinforced concrete slab strips subjected to axial tension loading. Tensile stresses build up in concrete when it contracts due to the reduction in concrete temperature from the peak hydration temperature to the ambient temperature.

The tests were carried out at the Slovak University of Technology in Bratislava in 2015 and 2018, respectively. A total of eight large-scale RC slab strips were constructed with two types of reinforcement bars, i.e., steel reinforcement grade BS0 (Sonnenschein 2015) and GFRP (Gazovicova 2018). The mechanical properties of the reinforcement bars, the test set-up and procedures, and the main results are presented in the following sections.

2.1 Mechanical properties of the reinforcement

In order to determine the tensile strength, modulus of elasticity, and bond strength of the steel and GFRP bars, tests were carried out according to the ISO and RILEM specifications (ISO 2012; RILEM 1994), respectively (see Table 2).

The strength and modulus of elasticity of the GFRP bars were determined by performing a tensile test with three 16 mm diameter bars (Fig. 1).
2.1.1 Bond strength test of the GFRP bars

The bond-slip relationship of the GFRP bars was determined by using a direct pull-out test (RILEM 1994). The GFRP bars had a nominal diameter of 16 mm, and the spiral bar deformation was 7 mm wide and 0.5 mm deep. Only a part of the bar with a length of 5 times the diameter was bonded to the concrete to minimize the beneficial effect of the confinement of the support on the bar anchorage. The rest of the bar was encased in a PVC hose to prevent the bond of the GFRP bars with concrete.

After 3 days, the average concrete compressive strength was 19.8 MPa, and the average maximum bond strength of the GFRP bars was 11.8 MPa as seen in Fig. 2(a). The bond of the 3-day-old concrete was mainly formed by the mechanical interlock of the spiral deformation. The bond failure occurred due to the gradual pulling out of the GFRP bar, which resulted in the deformations and resin completely peeling off with a slip of 5 mm as in Fig. 2(b).

After 28 days, the average compressive strength of the concrete was 45.9 MPa, and the average maximum bond strength of the GFRP bars was 15.3 MPa [Fig. 3(a)]. As can be seen from Figs. 2(a) and 3(a), the bond strength of the GFRP bars governed either by the adhesion resistance (33% and 53% of the highest bond strength of the 3 and 28-day-old concrete, respectively) or by the mechanical interlock of the bar deformations (100% of the highest bond strength of the 3 and 28-day-old concrete, respectively) and after the peak value by the frictional resistance to slipping (93% and 68% of the highest bond strength of the 3 and 28-day-old concrete, respectively). The bond failure was much more ductile than the GFRP strength failure.

2.1.2 Bond strength test of the steel bars

The direct pull-out test was also used to determine the bond strength of the deformed steel bars with a nominal diameter of 12 mm in the 3-day-old concrete. After 3 days, the average compressive strength of the concrete was 22.1 MPa, and the average maximum bond strength of the steel bars was 13.3 MPa [Fig. 4(a)].

The results of the direct pull-out tests of the steel reinforcement in the 28-day-old concrete is shown in Fig. 5(a). After 28 days, the average compressive strength of the concrete was 51.1 MPa, and the average maximum bond strength of the steel bars was 16.4 MPa [Fig. 5(b)].

| Material | Yield strength (MPa) | Tensile strength (MPa) | Ultimate tensile strain (%) | Modulus of elasticity (GPa) |
|----------|----------------------|------------------------|-----------------------------|----------------------------|
| Steel    | 520.17               | 618.0                  | 14.03                       | 200.34                     |
| GFRP     | –                    | 1020 to 1080           | 2.4 to 2.8                  | 54.74                      |

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![Fig. 2 (a) Bond-slip relation of the GFRP bars in the 3-day concrete, (b) detail of the GFRP bar’s surface after bond failure with completely sheared spiral deformations.](image)

![Fig. 3 (a) Bond-slip relation of the GFRP bars in the 28-day concrete, (b) detail of the GFRP bar’s surface after bond failure with partially sheared spiral deformations.](image)
the concrete was 39.37 MPa, and the average maximum bond strength of the steel bars was 18.1 MPa.

From a comparison of Figs. 4(b) and 5(b), it is obvious that the bond failure of the steel bars in the 3-day-old and 28-day-old concrete was similar. The concrete keys between the ribs were crushed, and the ribs of the steel reinforcement were partially damaged during both test times.

The maximum bond strength could be quite significant, depending on the geometry of the bar deformations, the radial stiffness of the bar, and the amount of concrete confinement provided. At 3 and 28 days, the bond strength of the GFRP bars was 11.8 and 15.3 MPa, respectively, which is approximately 85 and 88% of the bond strength of the steel bars, depending on the concrete strength.

2.2 Test of the large-scale RC slab strips

The test specimens for the experimental program consisted of RC slab strips that were 0.7 m wide, 0.3 m tall and 4.0 m in their effective length. Four slab strips were reinforced with steel bars and four with GFRP bars. The longitudinal bars were anchored to the enlarged parts at the ends of the slab strips.

2.2.1 Concrete mixture and strength

Table 3 shows the mix proportions of the concrete used in the experiment. In Table 4, the concrete strength of the

| Component                          | Specification   | Weight (kg) |
|------------------------------------|-----------------|-------------|
| Gravel aggregate                   | Fraction 0/4    | 770         |
|                                    | Fraction 4/8    | 290         |
|                                    | Fraction 8/16   | 810         |
| Cement (CEN 2011)                  | CEM II/A–LH 42.5 R | 275       |
| Pozzolanic additive based on fly ash | Stachesil P     | 60          |
| Plasticizing admixture             | Stacheplast 110 | 2.9        |
| Water                              |                 | 150         |

The k-value concept according to European Standard (CEN 2016).

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\frac{w}{(c + k \cdot \text{addition})} = \frac{150}{(275 + 0.2 \cdot 60)}. 
\]

\[0.52\]

Fig. 4 (a) Bond-slip relation of the steel bars in the 3-day-old concrete, (b) detail of the steel bar's surface due to bond failure after the concrete keys between the ribs were crushed.

Fig. 5 (a) Bond-slip relation of the steel bars in the 28-day-old concrete, (b) detail of the steel bar’s surface due to bond failure after the concrete keys between the ribs were crushed.
slab strips at the time of the test (3 days after concreting) as well as the reinforcement ratio are reported. The longitudinal steel bars had a diameter of 12 mm and the GFRP bars a diameter of 16 mm, respectively.

All the slab strips contained additional transverse reinforcing bars with spacing of 250 mm and a diameter of 12 mm. The transverse reinforcement was the same type as those used in the longitudinal direction. Figure 6 shows the geometry and dimensions of the slab strips, and Fig. 7 shows the configuration of the reinforcement.

The experimental set-up consisted of a steel loading frame that was mounted around the concrete slab strip. Inside the frame two hydraulic jacks were mounted for the generation of axial tensile stress in the slab strip (Fig. 8) to simulate the restrained contraction during cooling of concrete after the peak hydration temperature.

### 2.2.2 Test set-up and procedure

The aim of the present experimental study was to evaluate the effect of different types and amounts of reinforcement on early-age cracks that develop in concrete slabs. The mold was thermally insulated in order to

#### Table 4 Test program for the slab strips: concrete strength and reinforcement ratio.

| Slab strip | Type of the reinforcement | Average concrete compress. strength at the time of the test (MPa) | Average concrete tensile strength at the time of the test (MPa) | Longitudinal reinforcement in slab strips | Reinforcement ratio ρ (%) |
|------------|---------------------------|---------------------------------------------------------------|---------------------------------------------------------------|----------------------------------------|------------------------|
| S1         | Steel                     | 16.49                                                         | 1.68                                                          | 10 ø 12                                | 0.54                   |
| S2         |                           | 21.75                                                         | 1.72                                                          | 14 ø 12                                | 0.75                   |
| S3         |                           | 16.49                                                         | 1.68                                                          | 16 ø 12                                | 0.86                   |
| S4         |                           | 21.75                                                         | 1.72                                                          | 18 ø 12                                | 0.97                   |
| G1         | GFRP                      | 16.96                                                         | 1.31                                                          | 14 ø 16                                | 1.18                   |
| G2         |                           | 16.96                                                         | 1.31                                                          | 16 ø 16                                | 1.35                   |
| G3         |                           | 21.50                                                         | 1.61                                                          | 18 ø 16                                | 1.51                   |
| G4         |                           | 21.50                                                         | 1.61                                                          | 22 ø 16                                | 1.85                   |

Fig. 6 Plan view of the geometry and dimensions of the slab strip.

Fig. 7 Configuration of the reinforcement of the slab strip: (a) steel bars in specimen S3, (b) GFRP bars in specimen G2.
simulate the effect of the hydration temperature on a wide cross-section of the slab and to prevent any increase in the temperature of the steel frame. The slab strips were covered by a plastic sheet to reduce water evaporation.

Under slow cooling conditions, concrete can undergo a 20°C drop in temperature without cracking (ACI 2016; Neville 2011; Shi et al. 2014; Bobko et al. 2015). The temperature of the tested concrete was monitored by a thermocouple situated on the surface and inside the concrete slab strip. Figures 9(a) and 9(b) show that due to the heat from the hydration of the concrete, the temperature in the slab strips exceeded the ambient temperature by a maximum of 13°C at 12 to 15 hours and 12°C at 24 to 26 hours after the concreting, respectively.

The recorded time shift of the maximum temperature time was caused by the different ambient temperatures. After the cooling of the concrete, the difference in temperature of the concrete did not exceed 20°C, and no early-age cracks were observed on the concrete's surface.

In order to simulate the formation of early-age cracks, tensile stresses were produced by two hydraulic jacks at the mid-span of both sides of the steel loading frame (Fig. 8). The load frame transferred the force to the enlarged ends of the slab strips, which distributed a uniform tensile stress across the width of the central part of the strips. Three days after the concreting, the load was increased in steps from 20 to 60 kN, depending on the load level. The experimental test was terminated when the stabilized
cracking stage was reached. At this stage, the number of the cracks was stable, and only their width changed as the tensile force was increased.

2.3 Results of the experimental program
The resulting axial forces and corresponding tensile stresses leading to the formation of the first crack in the concrete slab strips are presented in Table 5.

The calculated stresses were verified by comparing them with the experimentally measured values of the concrete strains. During the testing, the specimens were examined for cracks at each load increment up to the stabilized cracking stage. The growth of each crack was examined, and the spacing of the crack was determined. The crack patterns at the stabilized cracking stage after the testing for two of the eight slab strips are presented in Figs. 10 and 11.

The stresses, the number of cracks and their widths at the stabilized cracking stage in the slab strips are presented in Table 6.

Figures 12 and 13 shows the experimental stress-strain diagrams in tension of the slab strips with steel and GFRP bars, respectively. The zigzag line was formed by the stress losses under constant elongation. Each time a new crack was observed on the concrete's surface, the elongation was sustained during its measurement. The measured stress dropped because the stress that causes crack propagation is less than the stress that initiates cracking (Carreira and Chu 1986).

It can be observed from Table 6 and Figs. 12 and 13 that due to being approximately twice the reinforcement ratio of the steel bars, the GFRP-reinforced slab strips produced more cracks than the steel-reinforced strips, thus eliminating the effect of the lower bond strength of the GFRP bars. In the GFRP-reinforced slab strips, the crack widths were roughly twice as large as in the steel-reinforced strips, due to a 3.7-fold smaller modulus of elasticity of the GFRP bars compared to the steel bars (Table 2).

2.4 Numerical analysis
A nonlinear finite element analysis (FEA) was conducted
using ATENA 3D software to validate the results of the experimental program that was reported in the preceding section. The material properties represent the stress-strain response of the constituent materials. The numerical analysis consists of 16 models of the slab strips reinforced with steel bars and 16 models reinforced with the GFRP bars. An analysis of the significance of the relationship between the input parameters (the steel and GFRP reinforcement ratio and bar diameter) and the output parameter (the average crack width) of the parametric study is summarized in Tables 7 and 8.

The boundary conditions of the concrete slab strips with enlarged ends were modeled as rigid steel plates at the end edges of the strips (Fig. 14). The size of the finite-element mesh was set at 5 mm. In addition to the monitoring points shown in Fig. 14, a global monitoring point was used to measure the crack widths. The loading of the slab strips was controlled by displacements with an

![Fig. 12 The complete force-strain diagram in tension of the concrete slab strips with steel bars.](image1)

![Fig. 13 The complete force-strain diagram in tension of the concrete slab strips with GFRP bars.](image2)

| Slab strip | Longitudinal reinforcement in slab strips | Reinforcement ratio (%) | Actual load (kN) | Stress in reinforcement (MPa) | Average crack width (mm) | Maximum crack width (mm) | Number of cracks (pcs) |
|------------|------------------------------------------|-------------------------|-----------------|-------------------------------|--------------------------|--------------------------|------------------------|
| S1         | 10 ø 12 steel                            | 0.54                    | 630             | 492.5                         | 0.319                    | 0.45                     | 17                     |
| S2         | 14 ø 12 steel                            | 0.75                    | 655             | 386.7                         | 0.248                    | 0.40                     | 20                     |
| S3         | 16 ø 12 steel                            | 0.86                    | 640             | 341.5                         | 0.229                    | 0.35                     | 21                     |
| S4         | 18 ø 12 steel                            | 0.97                    | 650             | 282.1                         | 0.212                    | 0.25                     | 21                     |
| G1         | 14 ø 16 GFRP                             | 1.18                    | 470             | 183.5                         | 0.326                    | 0.80                     | 30                     |
| G2         | 16 ø 16 GFRP                             | 1.35                    | 500             | 165.2                         | 0.305                    | 0.80                     | 26                     |
| G3         | 18 ø 16 GFRP                             | 1.51                    | 540             | 197.8                         | 0.290                    | 0.70                     | 26                     |
| G4         | 22 ø 16 GFRP                             | 1.85                    | 640             | 209.3                         | 0.277                    | 0.60                     | 27                     |
cracks with a width of w \leq 0.05 \text{ mm}, whereas the solid line represents the through reinforcement bond material. According to the results of the pull-out tests using the steel and GFRP bars with diameters of 12, 14, 16 and 20 mm, respectively, after sixty loading steps. The dashed line presents narrow surface cracks with a width of w < 0.05 \text{ mm}, whereas the solid line represents the through cracks with a width of w \geq 0.05 \text{ mm}.

The bond of the GFRP reinforcement was determined according to the results of the pull-out tests using the reinforcement bond material.

3. Results and discussion

The ability of concrete to carry tension between cracks is defined as the tension stiffening effect of concrete. As shown in Fig. 17, there are three distinct gradients to the response to tension of GFRP reinforced concrete:

- pre-cracking stage
- crack development stage
- post-cracking stage.

The post-cracking stage was not observed during the experimental testing due to the fact that the tests were terminated after the stabilized cracking stage was reached (Figs. 12 and 13).

The comparisons of the experimental and numerical results of the crack widths at the stabilized cracking stage for the slab strips reinforced with the steel and GFRP bars with diameters of 12, 14, 16 and 20 mm, respectively, are presented in Tables 7 and 8, and in Fig. 18.

The effects of the different reinforcement ratios and the steel and GFRP bar diameters on the average crack width at the stabilized cracking stage are graphically summarized in Fig. 18.

Figure 19 indicates that a steel reinforcement with a
diameter of 12 mm results in smaller crack widths during the experimental measurements than the calculations according to European Standard EC2 (CEN 2004). The smaller crack widths were obtained with diameters from 14 to 20 mm according to EC2 than by the numerical analysis at the same reinforcement ratio. Since the calibration of the numerical models was performed by adjusting the experimental measurements with the steel reinforcement with a diameter of 12 mm, it is not possible to verify the results for the other diameters.

The diagrams in Fig. 20 show a comparison of the average crack widths according to EC2 and the numerical and experimental analyses of the slab strips reinforced with a GFRP reinforcement. The conformity of the results with the GFRP reinforcement is better than the results with the steel reinforcement.

The calibration of the numerical models was performed based on the results of the experimental measurements with the GFRP reinforcement with a diameter of 16 mm. The calculations according to EC2 were for concrete structures reinforced with a steel reinforcement; therefore, it was necessary to adjust the coefficients in the calculations by taking into account the bond between the concrete and the GFRP reinforcement. The adjustment of these coefficients was based on the results of the pull-out tests.

Figure 20 indicates that it is not possible to achieve a crack width of 0.2 mm, even with a GFRP reinforcement ratio of 3%. The steel reinforcement ratio of about 1%, depending on the diameter of the reinforcement, can limit the crack widths from 0.2 to 0.25 mm (Fig. 19). Based on the above results, it can be concluded that:

1) in order to limit the crack width to a certain value, approximately twice the reinforcement ratio of the
GFRP bars as in the case of the steel bars is required. More precisely:

i. for diameter 12 mm: GRFP reinforcement ratio/steel reinforcement ratio = 1.0/0.54 = 1.85, determined at crack width $w = 0.318$ mm.

ii. for diameter 14 mm: 1.39/0.73 = 1.90, at crack width $w = 0.292$ mm.

iii. for diameter 16 mm: 1.87/0.9 = 2.07, at crack width $w = 0.275$ mm.

iv. for diameter 20 mm: 2.97/1.4 = 1.98, at crack width $w = 0.245$ mm.

2) at the same reinforcement ratio, smaller bar diameters resulted in smaller crack widths, e.g., for the reinforcement ratio $\rho = 1\%$:

![Graph showing comparison of average crack width for different steel and GFRP reinforcement ratios and bar diameters at the stabilized cracking stage.](image1)

![Graph showing comparison of average crack widths according to EC2 with the experimentally and numerically obtained values for the steel reinforcement.](image2)

![Graph showing comparison of average crack widths according to EC2 with the experimentally and numerically obtained values for the GFRP reinforcement.](image3)
i. steel diameter 12 mm, the crack width $w = 0.21$ mm, while for diameter 16 mm, $w = 0.27$ mm.

ii. GFRP diameter 12 mm, the crack width $w = 0.32$ mm, while for diameter 14 mm, $w = 0.34$ mm.

4. Conclusions

This paper presents a summary of work dealing with the problem of early-age cracking in concrete slabs reinforced with steel and GFRP bars. The main purpose of this research was to investigate the effects of restrained volume changes on the early-age cracking behavior in concrete slab strips reinforced with steel and GFRP bars. Emphasis must be placed on the fact that the findings of this study are limited, due to the special loading conditions (axial tension) and a single GFRP rebar type; however, based on the results of the experimental program and numerical analysis, the following conclusions can be drawn:

(1) The pull-out tests of the steel and GFRP bars indicate a moderate increase in bond strength with the increasing compressive strength of the concrete. At the age of 3 days, the average maximum bond strength of the concrete was 13.3 MPa for the steel and 11.8 MPa for the GFRP bars. At the age of 28 days, the average bond strength was 18.1 MPa for the steel and 15.3 MPa for the GFRP bars.

(2) The comparison of the direct pull-out test of the steel and GFRP bars for the 3-day-old concrete suggests different patterns of bond failure. The steel reinforcement had sheared the concrete keys between the bar deformations, while the GFRP had completely sheared the spiral deformation. The GFRP had two peaks, because with the first sheared part of the spiral deformation, the bond stress shifted to the undamaged part of the spiral. The bond behavior of the steel reinforcement was different; it had just one bond peak, and then the bond gradually decreased. The average maximum bond strength of the GFRP bars was approximately 85 to 88% of the bond strength of the steel bars, depending on the concrete's strength.

(3) The comparison of the experimental and numerical results of the crack widths at the stabilized cracking stage for the slab strips reinforced with the steel and GFRP bars with diameters of 12, 14, 16 and 20 mm, respectively, are presented in Tables 7 and 8, and in Fig. 18. In order to limit the early-age crack width to a certain value, approximately twice the reinforcement ratio of the GFRP bars as in the case of the steel bars is required.

(4) The width of the early-age cracks in the slab strips tested with the GFRP bars at the stabilized cracking stage was always wider than 0.2 mm, i.e., the width that can enable the self-healing of cracks (Design concept 2). As a result of this experience, it is obvious that GFRP can be used in watertight structures without through cracks (Design concept 1) or in structures with through cracks that are subsequently sealed (Design concept 3).

(5) GFRP bars offer excellent resistance to electrochemical corrosion in concrete and are an attractive option, especially for RC structures in aggressive environments. GFRP-reinforced structures extend their design life and significantly reduce operating costs, compared to steel-reinforced structures. On the other hand, the experimental results confirm that due to their lower modulus of elasticity and bond strength, the use of GFRP bars increased the width of the cracks. This could be a problem, especially for watertight concrete structures designed according to Design concept 2.

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