Numerical Verification of the Use of GFRP Reinforcement in the Ceiling Slab of a Garage House

Stanislav Blaho, Katarína Gajdošová, Róbert Sonnenschein
Faculty of Civil Engineering, Slovak University of Technology in Bratislava, Radlinskeho 11, 810 05 Bratislava, Slovakia

robert.sonnenschein@stuba.sk

Abstract. One of the major advantages of fibre reinforced polymers (FRPs) is their high tensile strength and low weight to strength ratio. Compared to steel reinforcement, this is one of the main and decisive features that could determine the choice of materials used in the design. Since there is no corrosion for this type of reinforcement, it is very suitable for concrete structures in aggressive environments. This point, along with others, gives concrete structures reinforced with FRPs significant advantages over conventional concrete structures with steel reinforcement. Different material properties and behaviour of FRP when compared to steel reinforcement result in the need of the modification of established design procedures, especially in the area of the ultimate strain, because FRP reinforcement has a different \(\sigma-\varepsilon\) diagram than the steel reinforcement. The experimental verification of the new design procedures is necessarily needed. This paper summarizes the results of practical design and calculations of the resistance of alternatively steel and GFRP reinforced concrete ceiling slab of a garage house and experimental results on the beam specimens reinforced with GFRP reinforcement tested under four-points test. It was necessary to adjust the conventional procedures in calculating the required area of GFRP reinforcement in order to avoid brittle failure of the cross-section (rupture of reinforcement). This is ensured by abiding the balance reinforcement level. Another fact to be considered is the creep rupture phenomenon causing immediate rupture of GFRP reinforcement under a high level of sustained loading. The reduction of GFRP strength according to the long-term loading results in rising required area of reinforcement. Finally, the serviceability limit state is checked. According to the low modulus of elasticity of GFRP reinforcement, the deflection should be the limiting factor for the amount of reinforcement designed.

1. Introduction
FRP (Fibre Reinforced Polymer) reinforcement is used extensively in reinforced concrete structures and structures with special environmental class requirements because of its properties and environmental resistance. Due to the presence of chlorides in the structure of a garage house, the steel reinforcement would be threatened by corrosion and the alternative type of reinforcement can be used.

The properties of FRP reinforcement largely depend on factors such as fibre volume, fibre type, resin type, fibre direction, cross-sectional effect and quality control during production [1]. The resin acts as a matrix weaving together the fibres and transfers the load between the individual composite fibres. The resin also acts as a fibre protection against abrasion and environmental conditions (water,
salts, alkalis) that affect the durability of FRP products [2]. Commonly used fibres are carbon (Carbon Fibre Reinforced Polymer-CFRP), glass (GFRP), aramid (AFRP) and basalt (BFRP) fibres.

Glass fibres are the cheapest but less durable due to their chemical sensitivity to the alkaline environment. Carbon fibres have the best resistance and are also resistant to high temperature chlorides and ultraviolet rays. GFRP and CFRP fibres are not absorbent, i.e. they do not absorb water, which results in fatigue strength. The biggest problem of non-metallic reinforcement is its resistance to temperature - mainly because of the resin content.

2. Long-term properties
To date, there is not enough information about the long-term effects of FRP reinforcement as they are not used in structures or reinforcement systems that would reach their service life.

Durability, creep rupture and fatigue are the most important of the long-term properties of FRPs when choosing them to be used in the structure. The access of moisture and the alkaline environment affects the durability of the FRP reinforcement. After a certain period of time, a brittle failure occurs as the FRP reinforcement is subjected to a high level of a constant load. The long-term properties depend on the type of FRP reinforcement [3].

Long-term properties of FRP reinforcement are calculated from short-term properties using reduction factors. However, these reduction factors decrease the mechanical properties to a large extent and therefore it seems to be very inefficient to use FRP reinforcement. These reduction factors are only extrapolated at a time based on short-term tests and it is very needed to provide experimental investigations in real time to confirm or refute them.

3. Short-term experimental tests
Based on the need for actual experience over time to determine the true long-term degradation of FRP materials, extensive experimental verification is prepared. The first part was already realised. Six simply supported concrete beams with the height of 74 mm in average and the width of 150 mm, reinforced with two GFRP bars (f_u = 1000 MPa) with the diameter of 8 mm, were tested under four points bending test and the results were compared with the numerical calculation to verify the behaviour of this members and to prepare an extensive number of specimens for long-term testing. The results of the first part of the experimental study are summarized in Table 1.

There is a very good agreement between calculated and measured values. According to this compliance, the specimens could be studied in more detail. It is very complicated to measure the value of stress in GFRP reinforcement, but it is necessary to know this value for designing long-term load. According to calculations, the stress in GFRP reinforcement in the moment of failure was of about 650 MPa. All beams failed (Figure 1) by concrete crushing and the average value of strain at the compression surface of the concrete was of 3.8 ‰. This value was then used in adapted calculations.

If the reduction factors are used, the maximal allowed stress in GFRP reinforcement for long-term loading is of about 100–150 MPa. That means the tested beams would fail when loaded at the level of one fourth of their resistance.
Figure 1. Beam after failure (author's archive)

Table 1. Results from the numerical calculation and experimental verification of short-term tests

| Experimental measurements | Maximal resistance (kN) | Deflection in the middle (mm) | Ultimate strain of concrete (%) | Height of the cross section (mm) | Thickness of the cover layer (mm) |
|---------------------------|-------------------------|-------------------------------|---------------------------------|---------------------------------|----------------------------------|
| N1                        | 10                      | 44                            | 3.6                             | 76                              | 19                               |
| N2                        | 8.57                    | 50                            | 3.8                             | 73                              | 19                               |
| N3                        | 8.46                    | 41                            | -                               | 73                              | 20                               |
| N4                        | 9.5                     | 41                            | 3.8                             | 75                              | 20                               |
| N5                        | 8.98                    | 35                            | 3.9                             | 73                              | 19                               |
| N6                        | 9.39                    | 39.5                          | -                               | 74                              | 19                               |
| Calculated values         | 9.2                     | 33                            | 3.8                             | 74                              | 19                               |

According to a very good agreement of measured and calculated results, the theoretical assumptions are subsequently used for the design of GFRP reinforcement in the ceiling slab of a garage house. Reinforcement in GFRP alternative is designed according to EN 1992-1-1 with experimentally verified mechanical properties of reinforcement and additional assumptions of balance reinforcement ratio and long-term properties.

4. Design of the structure

GFRP reinforcement was used in the ceiling slab of a garage house (Figure 2). The total dimensions of the structure are of 78.9 m x 34.4 m and the spans in main fields are of 7.1 m x 7.5 m. In addition to the self-weight, the permanent load of 1kN/m² and the variable load of 2.5 kN/m² were considered. According to EN 206-1, the environmental class of the structure was determined as XD2, resulting in C30/37 concrete class and the reinforcement coverage was calculated to be 50 mm for the alternative of steel reinforcement and 25 mm for GFRP reinforcement. The load-bearing system was considered transverse with the following dimensions of structural elements: slab thickness of 250 mm, beams of 650 mm in height and 400 mm in width and columns of 400 x 400 mm at an axial distance of 7.5 m, 7.1 m and 5.1 m (Figure 3). There are two reinforcing cores with wall thicknesses of 250 mm. In the cores, there is a vertical communication system, a staircase and elevator shafts. The intermediate landing is supported in four locations on the opposite sides by the Schock Tronsole Z-type elements and the staircases are considered precast reinforced concrete elements. Other vertical communications are transport ramps, consisting of a slab of the thickness of 250 mm and the perimeter supporting beams of the height of 650 mm and the width of 400 mm, which are supported by columns with dimensions 400 x 400 mm. Around the perimeter of the structure, there are reinforcing walls of 200 mm in thickness, which help the structure to achieve better rigidity.
The reinforcement in a ceiling slab was designed in a few steps and alternatives. The first alternative is represented by a steel reinforcement for ultimate and serviceability limit states. In the second alternative GFRP reinforcement was designed in several steps. At first, the GFRP reinforcement was only designed to transfer tensile forces in a bent cross-section. In this case, the cross-section failure occurs by rupture of the reinforcement. Since the linear behaviour represented by the $\sigma$-$\varepsilon$ diagram of GFRP, the failure by achieving the ultimate tensile force is brittle. To not achieve brittle failure, the concrete crushing failure was chosen for design adjustment and it was necessary to calculate the so-called balance area of the GFRP reinforcement. The enlargement of the reinforcement area according to cross-section failure occurring by the crushing of concrete can be seen in Table 2. Shear resistance of cross-section was also checked using design equations applying for steel reinforcement modified by the ratio of modules of elasticity of GFRP and steel due to the low modulus of elasticity of the GFRP reinforcement ($E_{\text{GFRP}} = 52$ GPa).

The reinforcement areas needed for the tensile force and for balanced conditions were calculated with the assumption of short-term material properties of GFRP reinforcement (design value of tensile strength of 770 MPa). The next step is the consideration of long-term load-bearing capacity. It was also calculated from the force and moment equilibrium conditions and the geometric condition of the similarity of the triangles of the relative strain’s progression. The results in Table 2 show the comparison of the resultant areas of the reinforcement for the individually considered stresses in GFRP. It is worth noting a significant increase in reinforcement areas while limiting the reinforcement in the assessment of long-term load-bearing capacity. The original short-term tensile strength was considered of 770 MPa, whereas at long-term resistance it was reduced to 354 MPa, resulting in a
significant increase in reinforcement areas and thus leading to an unconventional design from the investor's point of view. The long-term reduction factor was assumed according to *fib* bulletin No. 40 [4] as follows:

\[
\gamma_{\text{env}} = \frac{f_{\text{fk}}}{f_{\text{fk100h}}} \frac{1}{(100-R_{10})^\gamma} \tag{1}
\]

\[
\gamma_{\text{f}} = \frac{f_{\text{fk}}}{\gamma_{\text{env}} + \gamma_{\text{f}}} \tag{2}
\]

where \(f_{\text{fk100h}} = 960\) MPa, \(R_{10}=13\), \(f_{\text{fk}} = 1000\) MPa and \(\gamma_{\text{f}} = 1.3\).

**Table 2. Results from the numerical calculation of the reinforcement area**

|                     | Steel reinforcement (mm²) | GFRP reinforcement designed for ultimate tensile force (mm²) | GFRP reinforcement designed for concrete rupture – balanced conditions (mm²) | GFRP reinforcement designed for long-term strength (mm²) |
|---------------------|--------------------------|------------------------------------------------------------|-----------------------------------------------------------------------------|--------------------------------------------------------|
| **Top surface**     | 5Ø12/m (Ø12-200) 565 mm² | 7.4Ø12/m (Ø12-135) 838 mm²                                 | 10Ø12/m (Ø12-100) 1131 mm²                                                 | 10Ø22/m (Ø22-100) 3801 mm²                            |
| **Bottom surface**  | 8Ø10/m (Ø10-125) 628 mm² | 5.7Ø10/m (Ø10-175) 449 mm²                                 | 10Ø12/m (Ø12-100) 1131 mm²                                                 | 12Ø20/m (Ø20-80) 3770 mm²                             |

In the following, the serviceability limit state is verified. The deflection of the structure considered by the three methods and their results are given in Table 3. Deflection according to STN EN 1992-1-1 is calculated according to generally used procedures where the average bending stiffness \(E_{I_r}\) is introduced.

\[
E_{I_r} = \frac{1}{\xi_{F_{II}} + \beta^2 \xi_{F_{I}}} \tag{3}
\]

where \(E_{I_{II}}\) and \(E_{I}\) are the cross-sections bending stiffness with and without cracks, \(\xi\) is the distribution coefficient and \(\beta\) is a coefficient for long-term load assumed to be 0.5. \(M_{c}\) is the moment at the crack formation and \(M_{\text{Eqp}}\) is the moment from the quasi-permanent load combination.

\[
\zeta = 1 - \beta \left( \frac{M_{c}}{M_{\text{Eqp}}} \right)^2 \tag{4}
\]

The deflection calculated as follows represents the final deflection at the end of service life (long-term deflection):

\[
w_{LT} = \frac{1}{3 \eta^4} \frac{E_{I_r}}{384 + \beta^2} \tag{5}
\]

For comparison, another variant of the deflection calculation was the formulation according to Bischoff [5], where the equivalent moment of inertia of \(I_{\text{eqv.Bischoff}}\) was expressed as:
where \( I_i \) is the moment of inertia of the ideal cross-section and \( I_{ir} \) is the moment of inertia of the ideal cracked cross-section to the neutral axis. The short-term deflection for a slab with a uniformly distributed load can be calculated by the following equation:

\[
\omega_{ST} = \frac{1 \cdot q_p \cdot l_{eff}^4}{384 \cdot E_{cm} \cdot E_{ekv} \cdot I_{ir} \cdot I_{ekv \cdot Bischoff}}
\]

Equation (7) was also used for the numerical calculation of experimentally verified slabs (Table 1). The results have a good agreement with measured values that is why the short-term deflection of the designed slab of garage house is assumed \( \omega_{ST} \) according to Bischoff.

According to ACI 440.1R-06 [6], the long-term deflection involving the effect of creep and shrinkage can be calculated as:

\[
\omega_{LT,ACI} = (1 + 0.6 \cdot \zeta) \cdot \omega_{ST}
\]

where 0.6 is the reduction factor derived from the Brown and Bartholomew [7] and Brown [8] trials and \( \zeta \) is the time-dependent constant load factor, which is of 1.0, 1.2, 1.4 and 2.0 for 3, 6, 12 and 60 months.

CSA-S806-02 [9] puts forward a more conservative proposal and suggests the following statement with \( \zeta \) factor equal to 2.0:

\[
\omega_{LT,CSA} = (1 + \zeta) \cdot \omega_{ST}
\]

The third variant of the calculation of long-term deflection was the Mi-as procedure [10], which followed the procedure like the Bischoff method with the difference of the long-term deflection calculation, where \( k_{creep} \) is the fracture coefficient and \( k_{sh} \) is the effect of shrinkage:

\[
\omega_{LT,Mias} = \omega_{ST} \left(1 + K_{creep}\right) + \frac{\varepsilon_{cs} \cdot l_{eff}^2}{8 \cdot \alpha} \cdot k_{sh}
\]

The results of the calculated deflections are shown in Table 3. Results according to ACI and CSA codes seem to show unbelievable low values of a predicted deflection after 50 years of the use of the structure.

| Table 3. Results from the numerical calculation of non-linear deflections |
|-----------------------------------------------|
| Steel STN-EN 8Ø10/m | GFRP STN-EN 10012/m | GFRP Bischoff 10012/m | GFRP Bischoff+ACI 10012/m | GFRP Bischoff+CSA 10012/m | GFRP Bischoff+Mias 10012/m |
|---------------------|----------------------|-----------------------|---------------------------|-------------------------|---------------------------|
| Short-term displacement | -                    | -                     | 1.4 mm                    | 1.4 mm                   | 1.4 mm                    |
| Long-term displacement | 9.8 mm               | 19.9 mm               | -                         | 3.1 mm                   | 4.2 mm                    | 11.3 mm                  |
5. Conclusions

The design of GFRP reinforcement in a reinforced concrete member should be provided in a few steps. At first, the GFRP reinforcement is designed to transfer tensile forces in a bent cross-section. In this case, the failure mode should be verified to avoid rupture of the reinforcement. Since the linear behaviour represented by the $\sigma$-$\varepsilon$ diagram of GFRP, the failure by achieving the ultimate tensile force is brittle. To not achieve brittle failure, the concrete crushing is preferred, and the design should be adjusted by the calculation of the so-called balance area of the GFRP reinforcement. The enlargement of the reinforcement area according to the cross-section failure occurring by the crushing of concrete is of 1.3 (top surface) to 2.5 (bottom surface) times depending on the acting bending moment. The reinforcement areas needed for the tensile force and for balanced conditions are calculated with the assumption of short-term material properties of GFRP reinforcement (design value of tensile strength of about 770 MPa). In the next step, the long-term load-bearing capacity should be considered. GFRP reinforcement strength is reduced to about 350 MPa (to a half of short-term). The value of the reinforcement area when considering long-term strength increases 3.4 times compared to the balanced short-term reinforcement. This leads to an unconventional design from the investor's point of view.

The prediction of deflection at the end of the service life of a concrete member reinforced with GFRP is a complex task. The long-term behaviour of concrete members reinforced with GFRP reinforcement should only be extrapolated in time. The most probable results are given by STN EN European code and Calculations according to Bischoff and Mias. The long-term deflection of a solved slab is predicted to be of about 11 to 20 mm – in average 1.5 times more than for steel reinforced slab with a half reinforcement area.

Acknowledgment(s)

This work was supported by the Slovak Research and Development Agency under the contract No. APVV-15-0658 and by the Scientific Grant Agency VEGA under the contract No. VEGA 1/0645/20.

References

[1] ACI 440.1R-03 “Guide for the Design and Construction of Concrete Reinforced with FRP Bars” 2003.
[2] B. Benmokrane, F. Elgabas, E. Ahmed and P. Cousin, Characterization and Comparative Durability Study of Glass/Vinylester, Basalt/Vinylester, and Basalt/Epoxy FRP Bars. J Compos Const 19(6) 1-12, 2015.
[3] F. Ceroni, E. Cosenza, M. Gaetano and M. Pecce, Durability issues of FRP rebars in reinforced concrete members. Cement & Concrete Composites 28 857-868, 2006.
[4] Fib Bulletin 40: “FRP reinforcement in RC structures”. Technical report. 160 pp, 2007.
[5] PH. Bischoff, Reevaluation of deflection prediction for concrete beams reinforce with steel and fiber reinforced polymer bars. J Struct Eng ASCE; 131(5):752-67 2005.
[6] ACI Committee 440 ACI. Guide for design and construction of structural concrete reinforced with FRP bars. Framington Hills (Michigan, USA): American concrete institute; 2006.
[7] V. Brown and C. Bartholomeu, Long-term deflections of GFRP-reinforced concrete beams. First international conference on composites in infra-structure, Tucson, Arizona, USA, p. 389-00; 1996.
[8] V. Brown, Sustained load deflections in GFRP- reinforced concrete beams. In: 3rd international RILEM symposium on non-metali (FRP) reinforcement for concrete structures (FRPRCS-3), Sapporo, Japan, p. 495-02; 1997.
[9] CSA Standard CAN/CSA-S806-02. Design and construction of building components with fiber-reinforced polymers. Canadian Standard Association, Mississauga, Canada; 2002.
[10] C. Mi-as, L. Torres, A. Turon and C. Barris Composite Structures 96(0) 279-285, 2013.