Improving methods of strength design of normal sections of flexural concrete members reinforced with fiber-reinforced polymer bars

Ilshat Mirsayapov, Igor Antakov and Aleksey Antakov

Abstract. Theoretical and experimental studies of the strength of normal sections of flexural concrete members reinforced with FRP bars have been conducted. The test specimens were concrete beams 1810 mm long, with a rectangular section of 120×220 mm, reinforced with two bars in the tensile area. The beams were reinforced longitudinally with steel, glass fiber-reinforced polymer (GFRP) and basalt fiber-reinforced polymer (BFRP) bars. The design methods of guidelines of Russia, the USA, Canada and the European Union have been considered. There are two approaches to the strength design of normal sections – the European and the North American. The approach used in the design method of the Russian guideline SP 295.1325800.2017, when all partial safety factors are used in calculating the design characteristics of materials, causes an overestimation of the boundary values of relative depth of the compressed region ξR. It leads to an inaccurate determination of the failure mode and possible over-reinforcement of the construction. Some corrections have been brought about to the design methods of the Russian guideline SP 295.1325800.2017. Factor β has been introduced, which takes into account prestressing of the reinforcement. As a result, the deviation of the theoretical evidence from the experimental values of failure moments decrease from 30.44 % to 13.2 %. Changes have been made to the approach for the application of safety factors, which allowed increasing the accuracy of determining the failure mode and bring the safety factor for members C to the value of 1.6 adopted by the authors.

Keywords: building, construction, reinforced concrete, fiber-reinforced polymer, non-metallic reinforcement, flexural members.

1 Introduction

Fiber-reinforced polymers (FRP) reinforcement bars have a number of advantages over steel reinforcement: up to 3.5 times more tensile strength, low specific density, high corrosion resistance, and low thermal conductivity. Moreover, the deformation properties of the FRP bars also differ significantly: up to 4 times lower modulus of elasticity, the "stress-strain" diagram for short-term loading is almost straight up to the point of failure.

Studies by many scientists have revealed a number of features of the work of flexural concrete members with FRP reinforcement under load:

- first, the external fibers of the reinforcement bars are subjected to tensile, later the internal ones come into operation [1];

- “bending moment – deflection” diagrams for beams under load are characterized mainly by bilinear dependencies with two behavior phases of the flexural members: without cracks and with cracks [2-9]. At the same time, the deflection values are 3-4 times higher than those of reinforced concrete analogues, and the crack width is correspondingly greater [10-16];

- the compressed region in the normal section significantly, as compared with reinforced concrete members, decreases after the appearance of cracks and then remains almost constant until the failure of the member [17];
the failure of the normal section is of brittle nature with the implementation of two modes of fracture behavior – from the rupture of tensile reinforcement and from the concrete crushing in the compressed region [3, 4, 9, 11, 12, 16, 18]. Simultaneous failure of concrete and tensile reinforcement is also possible [19]. Due to the low modulus of elasticity of the FRP, when the reinforcement ratio is below a defined level, the failure of the beams from concrete crushing is possible, with loads less than design values [20].

- due to the relatively high susceptibility of FRP reinforcement to creep during sustained load, the deflections of the flexural members increase to 90 % of the initial values, depending on the size of load and the type of reinforcement. Moreover, crack formation is possible over time [21]. Depending on the type of FRP, the ultimate strength at sustained load is about 20-65 % of the strength at short duration load [22, 23];

- due to the relatively low modulus of elasticity of the FRP, the requirements for deflections and the crack width can be basic when designing FRP – reinforced constructions [11]. It is necessary to improve existing methods for calculating deflections and crack width of flexural members with FRP [9, 16, 24-27]. The most significant variables in the design for serviceability limit states (SLS) are the reinforcement ratio and modulus of elasticity of FRP [9];

- more cracks are formed in beams reinforced with FRP bars than in reinforced concrete analogues [14]. When the reinforcement ratio increases the crack spacing and their width decreases [15, 28].

- in the previous study [29], it was established that the SLS of beams reinforced with FRP bars occur at 26.1-52.9 % of the ultimate breaking load, for the reinforced beams with prestressed FRP bars – at 42.3-70.3% of the ultimate failure load. Using bars of smaller size proves to be more effective.

Thereby, the behavior of flexural members with FRP bars under load is essentially different from that of reinforced concrete constructions, due to physical and mechanical properties of FRP reinforcement.

In July 2015, Amendment No. 1 to SP 63.13330.2012 came into operation, which contains Appendix L with recommendations for the calculation and design of structures with FRP reinforcement. In 2017, SP 295.1325800.2017 was released which dealt exclusively with the design of structures with FRP reinforcement. The design methods of SP 63.13330.2012 and SP 295.1325800.2017 are identical, except for the calculation of the crack width. There are also guidelines on the use and design of structures with FRP reinforcement in a number of other countries. Methods of the USA, the European Union (EU) and Canada have been considered to compare the calculation results.

A common feature of all considered guidelines and recommendations is that calculation methods are based on existing approaches for reinforced concrete structures. In both Russian and foreign methods, the calculations are performed according to two groups of limit states: 1 - Ultimate Limit States (ULS), 2 Serviceability Limit States (SLS).

There are two approaches to the strength design of normal sections - these are European and North American [30]. In the first, the strength state condition of the normal section is presented as

\[ R \geq L, \]  

(1)

where \( R \) – is the design resistance of the cross section, determined taking into account the design characteristics of the materials, \( L \) – is the stress from the external design load.

In the second approach,

\[ R_n \cdot \phi \geq L, \]  

(2)

where \( R_n \) – is the nominal resistance of the cross section, determined taking into account the normative characteristics of materials, that consider only the environmental reduction factor for exposure conditions, \( \phi \) – is the strength reduction factor, taking into account the failure mode.

Thus, the fundamental difference in the described approaches are the methods of applying the safety factors. The approach used in the design method of the Russian guideline SP
295.1325800.2017, as expected, causes an overestimation of the boundary values relative depth of the compressed region $\xi_R$, — criterial value, which characterizes the type of failure mode of the members. As a result, a distortion occurs in the balanced reinforcement ratio, in which the failure of the member is possible simultaneously from the rupture of tensile reinforcement and from the concrete crushing in the compressed region.

In North American guidelines, the design characteristics of materials are determined only using a partial safety factor that takes into account the environmental reduction factor for exposure conditions of the construction. That is, the value of the design tensile strength of FRP is approximately equal to the value at failure from rupture, and the strength reserve is provided by a safety factor $\phi$, applied to the value of the nominal moment capacity.

The value $\xi_R$ of the Russian guidelines corresponds to that of $\rho_{fb}$ in the US ACI 440.1R-06 and the EU – fib Bulletin 40 methods. For the analysis of the considered methods, the values of the balanced reinforcement ratio $\rho_{fb}$ are given for beams with a rectangular section of 120x220 mm, reinforced with two bars of 8 mm in diameter, with varying classes of concrete (Figure 1).

The difference between the $\rho_{fb}$ values, determined by the methods of SP 295.1325800.2017, fib Bulletin 40 and ACI 440.1R-06, in the calculations taking into account the safety factors, reaches 100%. In this case, the SP data exceeds ACI by 50-60%. When calculating without taking into account the safety factors, the results according to the SP and ACI methods are quite close, the maximum difference is 5.08%. Therefore, the principle of using safety factors affects the value of the balanced reinforcement ratio.

**Figure 1.** Diagrams of changes in $\rho_{fb}$ values calculated by the SP295.1325800.2017, fib Bulletin 40 and ACI 440.1R-06 depending on the concrete class, with and without the account for partial safety factors.

Important features of the considered methods are the differences in the values of the safety factors. In Table 1 there is a comparison of the given safety factors of ACI 440.1R-06 and SP 63.13330.2012, depending on the type of load.
Table 1. Comparison of the general safety factors of ACI 440.1R-06 and SP 295.1325800.2017 methods depending on the type of load

| Guidelines       | Type of load | Short duration load | Sustained load |
|------------------|--------------|---------------------|----------------|
| SP 295.1325800.2017 | 0.46-0.6     | 0.14-0.24           |
| ACI 440.1R-06    | 0.39-0.52    | 0.14-0.16           |

With a short duration load, the design tensile strength of the reinforcement is from 39% to 60% of the characteristic value of tensile strength of FRP reinforcement, with a long-term - from 14% to 24%. Thus, the use of safety factors, leads to a significant underestimation of the values of the design tensile strength of FRP reinforcement and, accordingly, the values of the ultimate bending moment by the cross sections.

The validity of using the presented values of partial safety factors can be assessed by determining the theoretical safety factor for members $C$, which characterizes the ratio of the failure bending moment to the theoretical design ultimate bending moment. Figure 2 shows diagrams of changes in the safety factor $C$ for various values of the reinforcement ratio for the methods of SP 295.1325800.2017 and ACI 440.1R-06.

![Figure 2. Diagrams of changes in the value of the safety factor for members $C$ for various values of the Ratio of reinforcement of a flexural members with a section of 120x220 (h) mm, made of concrete C30/37 (B35), with reinforcement for GFRP, for the methods SP 295.1325800.2017 and ACI 440.1R-06.](image)

The greatest strength reserve is provided by the ACI method, which applies the corresponding largest safety factors. In contrast to the safety factors for members $C$ presented in GOST 8829-94, the strength reserve in the methods under consideration in the first failure mode (FRP rupture) is significantly higher than in the second (concrete crushing). This is a consequence of the use of the partial safety factors shown. Changes in the value of the safety factor $C$ in the SP method at values of the reinforcement ratio of 0.22% are caused by the inaccuracy in determining the design failure mode. The contradiction is that when determining the failure bending moment, the assumed failure mode was the FRP bars rupture, and when determining the design ultimate bending moment – the concrete crushing in the compressed region.

Thus, the issues of determining reasonable values of the safety factors and updating the principles of their use are relevant for the study and improvement of the SP method.
2 Materials and methods
In order to obtain experimental data on the behavior of flexible members with FRP reinforcement, experimental studies of 7 series of beam specimens were carried out. In Figure 3 the scheme of support and loading of the studied beams is presented. A series of beams 1-5 were tested under short duration load, a series of 6 and 7 – under sustained load.

![Figure 3. The scheme of support and loading of the beam specimens.](image)

Table 2 shows the characteristics of the series of beam specimens. The results of the study of beams of series 6 and 7 are presented in the article [31].

| Series No. | Strength classes for concrete | Number of bar, type, prestressing value | Design diameter, mm | Ratio of reinforcement μ, % | Modulus of elasticity Еs(f), МПа | Ultimate tensile strength R$_{s(f)}$, МПа |
|------------|-------------------------------|----------------------------------------|---------------------|-----------------------------|-------------------------------|----------------------------------|
| 1          | C25/30 (B30)                  | 2 Ø12 A400                             | 12                  | 0.942                       | 200000                        | 400                             |
|            |                               | 2 Ø8 A400                              | 8                   | 0.421                       |                               |                                 |
|            |                               | 2 Ø6 A400                              | 6                   | 0.238                       |                               |                                 |
| 2          | C30/37 (B40)                  | 2 Ø10 GFRP                             | 8.6                 | 0.484                       | 51500                         | 1200                            |
|            |                               | 2 Ø8 GFRP                              | 7                   | 0.321                       | 51500                         | 1200                            |
|            |                               | 2 Ø6 GFRP                              | 5                   | 0.164                       | 51770                         | 1250                            |
| 3          | C30/37 (B35)                  | 2 Ø7 BFRP                              | 7                   | 0.321                       | 50000                         | 1000                            |
|            |                               | 2 Ø5 BFRP                              | 5.3                 | 0.184                       | 51000                         | 1100                            |
|            |                               | 2 Ø4 BFRP                              | 4                   | 0.105                       | 51000                         | 1200                            |
| 4          | C25/30 (B30)                  | 2 Ø10 GFRP                             | 8.6                 | 0.484                       | 51500                         | 1200                            |
| 5          | C12/15 (B15)                  | 2 Ø6 GFRP, G$_{sp}$=250; 400 MPa       | 6.3                 | 0.260                       | 52000                         | 1280                            |
|            | C16/20 (B20)                  | 2 Ø8 GFRP, G$_{sp}$=250; 400 MPa       | 8                   | 0.419                       | 50800                         | 1120                            |
| 6          | C30/37 (B40)                  | 2 Ø6 GFRP                              | 5                   | 0.164                       | 51770                         | 1250                            |
| 7          | C30/37 (B35)                  | 2 Ø5 BFRP                              | 5.3                 | 0.184                       | 51000                         | 1100                            |
3 Results

Figure 4 shows the moment-deflection curves obtained as a result of testing beam specimens of the series 1-5. The behavior of beams specimens with FRP reinforcement under load is characterized by a relatively high deformability, compared with the corresponding beams with steel reinforcement and, mainly, by the linear “moment – deflection” dependence after cracking up to failure. The maximum deflections of beams reinforced with FRP were \((1/29 \div 1/46) l_o\), beams with prestressed FRP reinforcement – \((1/41 \div 1/67) l_o\), where the \(l_o\) – beam span.

![Figure 4. Moment-deflection curves obtained as a result of testing beam specimens of series 1-5.](image)

The failure mode of beam was recorded: 1 – rupture of tensile reinforcement; 2 – concrete crushing in the compressed region; 3 – simultaneously from the rupture of tensile reinforcement and from the concrete crushing; 4 – slipping of tensile reinforcement (due to failure bond between concrete and FRP reinforcing bars).

To assess the accuracy, reliability of the considered methods Table 3 and Figure 5 presents a comparison of the values of theoretical and experimental failure moments for specimens.

| Series No. | Reinforcement of beam specimens | Strength classes for concrete | SP 295.1325800.2017 | ACI 440.1R-06, ACI 440.4R-04 | ISIS Canada | fib Bulletin 40 | Experiment | Beam failure mode |
|------------|--------------------------------|------------------------------|----------------------|-----------------------------|-------------|----------------|------------|-----------------|
| 2          | 2 Ø10 GFRP                    | C30/37 (B40)                 | 21.025               | 21.067                       | 21.483       | 22.419         | 22.582     | 2              |
|            | 2 Ø8 GFRP                     |                              | 17.24                | 17.133                       | 17.588       | 18.14          | 18.88      | 4 / 2           |
|            | 2 Ø6 GFRP                     |                              | 9.436                | 9.169                        | 9.397        | 9.69           | 9.41       | 1              |
| 3          | 2 Ø7 BFRP                     | C30/37 (B35)                 | 14.429               | 14.165                       | 14.663       | 15.122         | 16.451     | 3              |
|            | 2 Ø5 BFRP                     |                              | 9.298                | 9.001                        | 9.284        | 9.573          | 11.283     | 1              |
|            | 2 Ø4 BFRP                     |                              | 5.862                | 5.64                         | 5.788        | 5.968          | 5.95       | 1              |
| 4          | 2 Ø10 GFRP                    | C25/30 (B30)                 | 18.211               | 18.05                        | 18.602       | 19.237         | 20.766     | 4              |
|            |                                | C30/37 (B40)                 | 21.025               | 21.067                       | 21.483       | 22.419         | 22.582     | 2              |
|            |                                | C35/45                       | 21.851               | 21.971                       | 22.326       | 23.371         | 22.959     | 4 / 2           |
Figure 5. Comparison of theoretical and experimental failure moments for specimens of series 2-5.

Deviations of theoretical evidence from the experimental values of failure moments for the considered methods are presented in Table 4.

Table 4. Ultimate deviation of theoretical evidence from the experimental values of failure moments, %

| Series No. (type of reinforcement) | SP 295.1325800.2017 | ACI 440.1R-06, ACI 440.4R-04 | ISIS Canada | fib Bulletin 40 |
|-----------------------------------|----------------------|-------------------------------|-------------|-----------------|
| 2, 3, 4 (AKII)                    | 17.6                 | 20.23                         | 17.72       | 15.16           |
| 5 (ACKP)                          | 30.44                | 21.8                          | -           | -               |

Certain deviations of theoretical evidence from experimental values indicate the possibility and need to improve the accuracy of calculation methods.

Comparison of theoretical and experimental evidence on the fracture behavior of beams showed that the criteria adopted in the method of SP 295.1325800.2017, which determine failure mode of flexural members, are inaccurate. In contrast to the calculation results without taking into account the safety factors, the accuracy of determining the failure mode in the calculation taking into account the safety factors decreased. One of the main reasons is the underestimation of the values of the design tensile strength of FRP reinforcement.

The influence of the sequence of using the reliability coefficients in the method of SP can be visually assessed by comparing the values of \( \xi \) and \( \xi_R \) in the calculations with and without safety factors.

The quantity \( \xi_R \) determines the transitional point of the first failure mode of flexural members in the second. It can be assumed, that at \( \xi / \xi_R = 1.0 \) a third failure mode should occur – the simultaneously from the rupture of tensile reinforcement and from the concrete crushing. In Table 5 a comparison of
the values $\xi$ and $\xi_R$ using the method of SP 295.1325800.2017 for calculating beams of series 1, 2, 3, with and without safety factors is presented. When determining the design ultimate bending moment of beams with FRP, where the calculation is performed taking into account the safety factors, the value $\xi_R$ increased to 40.9%, compared with the calculation without taking into account the safety factors, and the ratio $\xi/\xi_R$ decreased by 34.95%. For beams with steel reinforcement, the $\xi/\xi_R$ ratio increased by 7.9%. Therefore, in the method of SP 295.1325800.2017, when determining the design ultimate bending moment of normal sections, the values of $\xi_R$ are overestimated. It leads to an inaccurate determination of the failure mode and the possible over-reinforced of construction.

Table 5. Comparison of the values of $\xi$ and $\xi_R$ according to the method of SP 295.1325800.2017 when design beams of series 1, 2, 3, with and without the account for safety factors

| Series No. (type of reinforcement) | Reinforcement of beam specimens | Values $\xi$ and $\xi_R$ of the method of SP 295.1325800.2017 |
|-----------------------------------|-------------------------------|----------------------------------------------------------|
|                                   |                               | Excluding safety factors | Taking account of safety factors |
|                                   |                               | $\xi$ | $\xi_R$ | $\xi/\xi_R$ | $\xi$ | $\xi_R$ | $\xi/\xi_R$ |
| 1 (A400)                          | 2 Ø12 A400                    | 0.16 | 0.509 | 0.314 | 0.182 | 0.533 | 0.3409 |
|                                   | 2 Ø8 A400                     | 0.0707 | 0.509 | 0.1389 | 0.0804 | 0.533 | 0.1508 |
|                                   | 2 Ø6 A400                     | 0.0397 | 0.509 | 0.078 | 0.0452 | 0.533 | 0.0847 |
|                                   | 2 Ø10 GFRP                    | 0.15 | 0.105 | 1.44 | 0.1317 | 0.176 | 0.7492 |
| 2 (GFRP)                          | 2 Ø8 GFRP                     | 0.099 | 0.105 | 0.95 | 0.0869 | 0.176 | 0.4943 |
|                                   | 2 Ø6 GFRP                     | 0.0525 | 0.101 | 0.518 | 0.046 | 0.171 | 0.2688 |
|                                   | 2 Ø7 BFRP                     | 0.0917 | 0.119 | 0.77 | 0.0905 | 0.181 | 0.5009 |
| 3 (BFRP)                          | 2 Ø5 BFRP                     | 0.0575 | 0.112 | 0.515 | 0.0568 | 0.17 | 0.3334 |
|                                   | 2 Ø4 BFRP                     | 0.0356 | 0.104 | 0.344 | 0.0352 | 0.159 | 0.2213 |

Figure 6. Proposed changes in the method of SP for determining the ultimate limit state of normal sections flexural members.
Taking into account the evidence obtained changes were made to the calculation equations of the SP295.1325800.2017 method for strength design: in the expression for determining the height of the compressed region of concrete for $\xi > \xi_R$, the coefficient $\omega$ is replaced by the coefficient $\beta$, the values of which are determined as a result of processing the experimental evidence. With a prestressing reinforcement $\beta$ it is taken equal to 1.25, without prestressing – 0.8. Taking into account this change, the deviation of theoretical evidence from the experimental values of failure moments decreased from 30.44 % to 13.2 %.

Figure 6 shows a flow chart with the changes to the method of SP 63.13330.2012 for design ultimate limit state of normal sections of flexural members.

The sequence of using safety factors is changed. The safety factor $\gamma$ is removed from the equation for determining the calculated design tensile strength of FRP reinforcement. The ultimate bending moment $M_{ult}$ is replaced by the nominal bending moment $M_{nom}$. An equation is introduced to determine the ultimate bending moment $M_{ult}$, using the safety factors $\gamma$, taking into account the failure mode.

Depending on the ratio of $\xi$ and $\xi_R$, the failure mode is determined:
- when $\xi \leq \xi_R$ the member is failure by rupture of tensile reinforcement;
- when $\xi > \xi_R$ there is concrete crushing in the compressed region.

Since the FRP deformation curve has a linear shape without a yield strength, the failure of members from rupture of the reinforcement is “brittle nature”. In this regard, the values of $\gamma$ are taken in order to bring the safety factors $C$ to a value of 1.6. The safety factor $\gamma$ is taken depending on the ratio of $\xi$ and $\xi_R$:
- when $\xi \leq \xi_R$ – $\gamma = 1.3$;
- when $\xi_R < \xi < 1.2\xi_R$ – $\gamma = 1.2$, when $\xi > 1.2\xi_R$ – $\gamma = 1.15$.

These changes in the design method increased the accuracy and reliability of determining the failure mode of members, which is confirmed by the results of experiments.

Figure 7 compares theoretical values of the design bending moments and the experimental evidence for beams of series 2-5 in order to determine the values of the safety factor for members $C$ regulated by GOST 8829-94.

Deviations of the obtained safety factors from the value of $C = 1.6$ amounted to: SP 295.1325800.2017 – up to 27.5 %; ACI 440.1R-06 – up to 77.9 %; ISIS Canada – up to 21.7 %; fib Bulletin 40 – up to 29.4 %; improved method of SP – up to 12.4 %.

4 Discussions
As a result of the conducted research, the shortcomings of the existing methods of FRP – reinforced flexural members design have been discovered. It is established that the use of safety factors in the considered guidelines methods leads to a significant underestimation of the values of the design tensile
strength of FRP, in the range of 14-60 % of the ultimate tensile strength, and the accepted the section values of bending moments. Approach to safety factors application, used in the design method of the guideline SP 295.1325800.2017, leads to overestimation of the boundary values of relative depth of the compressed region $\xi_a$. As a result, there is a distortion of the value of the balanced reinforcement ratio, at $\xi/\xi_a = 1.0$, that leads to inaccurate determination of the failure mode and the possible over-reinforcement of the constructions.

Taking into account the experimental evidence, alterations have been made in the design methods of SP 295.1325800.2017 of the strength of normal sections of flexural concrete members with GFRP and BFRP, according to ultimate limit states:

1. To determine the compression area, the concrete height was introduced into the calculation formula, where $\xi/\xi_b$ empirically determined coefficient $\beta$ has been introduced which takes into account occurrence of prestress of the reinforcement. Due to this alteration, the deviation of theoretical formula has been determined, which leads to inaccurate determination of the failure mode and the possible over-reinforcement of the constructions.

2. Alterations in the approach to safety factors use have been made to increase the accuracy of determination of fracture mode. After the analysis of FRP reinforcement behavior under load, a safety coefficient factor for members $C$ has been determined, which equals 1.6. In the guideline SP 295.1325800.2017 this parameter (value) is measured as 2.04. In the modified guideline the difference between the design value and the rate value of the safety factor $C=1.6$ doesn’t exceed 12.4%.

References

[1] Rimshin V I and Merkulov S I 2015 About Normalization of Characteristics of Rod Non-Metallic Composite Reinforcement. Promyshlennoe i grazhdanskie stroitel'stvo 5 pp 22-26
[2] Attia K, El Refai A and Alnahhal W 2020 Flexural behavior of basalt fiber-reinforced concrete slab strips with BFRP bars: experimental testing and numerical simulation Journal of composites for construction 24(2) DOI: 10.1061/(ASCE)CC.1943-5614.0001002
[3] Salih R and Zhou F Y 2019 Numerical investigation of the behavior of reinforced concrete beams reinforced with FRP bars Civil Engineering Journal 5(11) pp 2296-2308 DOI: 10.28991/cej-2019-03091412
[4] Pawłowska D and Szumigala M 2015 Flexural behaviour of full-scale basalt FRP RC beams – experimental and numerical studies 7th Scientific-Technical Conference Material Problems in Civil Engineering (MATBUD’2015). Procedia Engineering 108 pp 518-525 DOI: 10.1016/j.proeng.2015.06.114
[5] Acciai A, D’Ambris A, De Stefano M, Feo L, Focacci F and Nudo R 2016 Experimental response of FRP reinforced members without transverse reinforcement: Failure modes and design issues Composites part B-engineering 89 pp 397-407 DOI: 10.1016/j.compositesb.2016.01.002
[6] Adam M A, Said M, Mahmoud A A and Shanour A S 2015 Analytical and experimental flexural behavior of concrete beams reinforced with glass fiber reinforced polymers bars Construction and building materials 84 pp 354-366 DOI: 10.1016/j.conbuildmat.2015.03.057
[7] El-Nemr A, Ahmed E A, El-Safta A, Benmokrane B 2018 Evaluation of the flexural strength and serviceability of concrete beams reinforced with different types of GFRP bars. Engineering Structures 173 pp 606-619 DOI: 10.1016/j.engstruct.2018.06.089
[8] Ju M, Park Y and Park C 2017 Cracking control comparison in the specifications of serviceability in cracking for FRP reinforced concrete beams Composite Structures 182 pp 674-685 DOI: 10.1016/j.compstruct.2017.09.016
[9] Adam M A, Said M, Mahmoud A A and Shanour A S 2015 Analytical and experimental flexural behavior of concrete beams reinforced with glass fiber reinforced polymers bars Construction and building materials 84 pp 354-366 DOI: 10.1016/j.conbuildmat.2015.03.057
[10] Al-Sunna R, Pilakoutas K, Hajirasouliha I and Guadagnini M 2012 Deflection behavior of FRP reinforced concrete beams and slabs: An experimental investigation Composites Part B: Engineering 43(5) p 23 DOI: 10.1016/j.compositesb.2012.03.007

[11] Urbanski M, Garbacz A and Lapko A 2013 Investigation on concrete beams reinforced with basalt rebars as an effective alternative of conventional R/C structures Proceedings of the 11th International Conference on Modern Building Materials, Structures and Techniques. Procedia Engineering 57 pp 1183-91 DOI: 10.1016/j.proeng.2013.04.149

[12] Ruan XJ, Lu CH, Xu K, Xuan GY and Ni MZ 2020 Flexural behavior and serviceability of concrete beams hybrid-reinforced with GFRP bars and steel bars Composite structures pp 235 DOI: 10.1016/j.compstruct.2019.111772

[13] Barris C, Torres L, Vilanova I, Miás C, Llorens M 2017 Experimental study on crack width and crack spacing for Glass-FRP reinforced concrete beams Engineering Structures 131 pp 231-242 DOI: 10.1016/j.engstruct.2016.11.007

[14] Kim S and Kim S 2019 Flexural behavior of concrete beams with steel bar and FRP reinforcement Journal of Asian architecture and building engineering 18(2) pp 94-100 DOI: 10.1080/13467581.2019.1596814

[15] Pan M X and Xu X S 2017 Study on crack development of concrete beams in bending reinforced with FRP bars. 3rd international conference on energy materials and environment engineering (ICEEMEE) (Thailand: Bangkok) DOI: 10.1088/1755-1315/61/1/01203

[16] Lapko A and Urbanski M 2015 Experimental and theoretical analysis of deflections of concrete beams reinforced with basalt rebar Archives of Civil and Mechanical Engineering 15(1) pp 223-230 DOI: 10.1016/j.acme.2014.03.008

[17] Barris C, Torres L Turon A, Baena M and Mias C 2008 Experimental study of flexural behaviour of GFRP reinforced Fourth International Conference on FRP Composites in Civil Engineering (CICE2008) (Switzerland: Zurich)

[18] Fei Peng and Weichen Xue 2018 Design approach for flexural capacity of concrete T-beams with bonded prestressed and nonprestressed FRP reinforcements Composite Structures 204 pp 333-341 DOI: 10.1016/j.compstruct.2018.07.091

[19] Sun Y, Liu Y, Wu T, Liu X and Lu H 2019 Numerical analysis on flexural behavior of steel fiber-reinforced LWAC beams reinforced with GFRP bars APPLIED SCIENCES-BASEL 23(9) DOI: 10.3390/app9235128

[20] Imomnazarov T S, Al-Sabri S A M and Dirie M H 2018 The use of composite reinforcement System technologies 2(27) pp 24-29

[21] Gross S, Yost J and Kevgas G 2003 Time-dependent behavior of normal and high strength concrete beams reinforced with GFRP bars under sustained loads High Performance Materials in Bridges, American Society of Civil Engineers pp 451-462

[22] Bennmokrane B, Brown V L, Mohamed K, Nanni A, Rossini M and Shield C 2019 Creep-rupture limit for GFRP bars subjected to sustained loads Journal of composites for construction 23(6) DOI: 10.1061/(ASCE)CC.1943-5614.0000971

[23] D'Antonio T and Pisani M A 2019 Long-term behavior of GFRP reinforcing bars Composite Structures pp 227 DOI: 10.1016/j.compstruct.2019.111283

[24] Barris C, Torres L, Comas J and Mias C 2013 Cracking and deflections in GFRP RC beams: an experimental study Composites: Part B 55 pp 580-590

[25] Mahdi Feizbahr, Jayaprakash, Mortezza Jamshidi, Choong Kok Keong 2013 Review on Various Types and Failures of Fibre Reinforcement Polymer Middle-East Journal of Scientific Research 13(10) pp 1312-18 DOI: 10.5829/idosi.mejr.2013.13.10.1180

[26] Begunova N V, Grahov V P, Vozmishchev V N and Kislyakova J G 2019 Comparative evaluation of results on test of concrete beams with fiberglass rebar and calculated data Science and Technique 18(2) pp 155-163 DOI: 10.21122/2227-1031-2019-18-2-155-163
[27] Ng P L, Barros J A O, Kaklauskas G and Lam J Y K 2020 Deformation analysis of fibre-reinforced polymer reinforced concrete beams by tension-stiffening approach Composite Structures 234 DOI: 10.1016/j.compstruct.2019.111664

[28] Miàs C, Torres L, Guadagnini M and Turon A 2015 Short and long-term cracking behaviour of GFRP reinforced concrete beams Composites Part B: Engineering 77 pp 223-231 DOI: 10.1016/j.compositesb.2015.03.024

[29] Antakov I A 2018 Features of behavior of flexural members with composite polymeric reinforcement under load Zhilishchnoe Stroit’stvo 5 pp 15-18

[30] Kuzevanov D V 2012 Scientific technical report «Constructions with fiber-reinforced polymer reinforcement. Review and analysis of foreign and domestic guidelines». [Electronic resource]. NIIZHB named after A. A. Gvozdev Laboratory No 2 URL: http://www.niizhb2.ru/Article/nka2012.pdf

[31] Antakov A B and Antakov I A 2016 Experimental studies of flexural members with fiber-reinforced polymer reinforcement under sustained load application International conference on new architecture, design construction and renovation NASKR (Cheboksary) pp 67-72