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STRENGTHENING OF BENT REINFORCED CONCRETE BEAMS WITH FRP COMPOSITES – COMPARISON OF ULS DESIGN GUIDELINES

WZMACNIANIE ZGINANYCH BELEK ŻELBETOWYCH MATERIAŁAMI KOMPOZYTOWYMI FRP – PORÓWNANIE PROCEDUR PROJEKTOWANIA Z UWAGI NA NOŚNOŚĆ

A b s t r a c t

The paper raises an issue of designing flexural strengthening of reinforced concrete beams with FRP composite materials. Three different design approaches are presented. The conducted analysis consists of the determination of the load-bearing capacity of a beam, the assumption of loading and the evaluation of a capacity increase after strengthening the beam with a CFRP strip. Results are compared and justified and some conclusions are drawn. Furthermore, an author's computer software for simple verification of ultimate limit state is briefly presented.

Keywords: FRP composite materials, CFRP strips, flexural strengthening, RC beams

S t r e s z c z e n i e

Artykuł porusza kwestię projektowania wzmocnień belek żelbetowych na zginanie przy pomocy materiałów kompozytowych FRP. Opisano procedury projektowania i porównano wyniki obliczeń przykładowej belki według trzech różnych norm, ponadto przedstawiono autorski program komputerowy pozwalający na obliczenie wzmocnienia belki o przekroju prostokątnym taśmami CFRP.

Słowa kluczowe: materiały kompozytowe FRP, taśmy CFRP, wzmacnianie na zginanie, żelbet

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1. Introduction

For more than 20 years, widely known fibre reinforced composite materials have become much more frequently used for strengthening reinforced concrete structures. Their mechanical properties, such as exceptionally high tensile strength, perfect corrosion resistance and low mass make a contribution to the constantly increasing popularity of FRP materials. A considerable advantage is also the lack of influence on not only self-weight of the strengthened construction, but also on the dimensions of the cross-section of elements, which is sometimes a decisive factor during the choice of strengthening method.

The tensile strength of FRP composite strips is within the range of 2000 – over 3000 MPa, while their elasticity modulus is also relatively high, – approximately 160–200 GPa for standard strips and above 300 GPa for high-modulus strips.

FRP composite materials can contribute to an increase of the load-bearing capacity of various construction members, such as columns, beams or slabs. Strips with non-metallic, continuous fibres, arranged in one direction, are used for flexural strengthening, whereas for shear strengthening, a preferable material is FRP sheets with fibres organised orthogonally in the composite matrix. Strengthening reinforced concrete beams subjected to bending is realised by bonding the strip to the tensile, usually bottom, surface of the strengthened member using proper adhesive. The most widely used adhesives are based on epoxy resins. Optionally, there is a possibility to anchor ends of the strip using steel blocks or FRP composite elements (Fig. 1).

Fig. 1. Examples for strengthening concrete beams and slab with anchoring elements [7]

The first European guidelines considering FRP strengthening were published by *fib* (The International Federation for Structural Concrete) in 2001 [1]. Nowadays, there is a variety of design guidelines, among which there are: American *'Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures'* (ACI440, 2R-08, 2008 (first version in 2000), Italian National Research Council (CNR) publication (CNR-DT 200/2004, 2004), Swiss guidelines (SIA166, 2004), Canadian (CAN/CSA-S806-02, 2002) and British Concrete Society document (TR55, 2004) [3]. But on the other hand, in many countries, such as Poland, any formal guidelines for the design of such strengthening have still not been developed. Also, the EC-2 is beyond the scope of modern composite materials used for concrete reinforcement. Model Code 2010 [N5] takes on a non-metallic reinforcement for concrete, determining the material properties and the problem of bonding such a reinforcement for concrete, but does not provide specific design rules for ULS and SLS.

The following paper presents a short description of design procedures for strengthening reinforced concrete beams for flexure and the most important differences between three of the above-mentioned documents: *fib* Bulletin no. 14 [N4], ACI 440, 2R-08 [N2] and the Italian CNR-DT 200/2004 [N3]. Results obtained from these calculations were compared and justified. Furthermore, an author's computer program, written in Delphi environment, is briefly described. It allows for the checking of the ultimate limit state of the strengthened bent RC element, according to *fib* [N4] and the results from the program are also presented.

2. Design guidelines for FRP strengthening

2.1. American ACI 440, 2R-08

The *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*, which was developed in the United States, widely describes the strengthening and retrofitting of concrete structures with FRP materials. Procedures given in this document start from calculating existing strains in extreme concrete fibres of the strengthened member and then continue to the estimation of the neutral axis depth. The β_1 coefficient for rectangular shape of stress distribution varies from 0.65 to 0.85, depending on concrete compressive strength.

Nominal capacity is decreased by two coefficients, the first of which is a strength reduction factor Φ [cf. N1 p. 9.3.1], dependent on the character of cross-section work (flexure, compression, shear etc.). A value of Φ proper for flexure (tension-controlled sections) is assumed as 0.9 and hence such a value was used during calculations. The second one is an additional strength reduction factor, ψ_f, which is equal to 0.85 and its aim is to decrease the FRP contribution to total flexural capacity.

2.2. *fib* Bulletin no. 14

The technical report, Bulletin no. 14 of *fib*, entitled '*Externally bonded FRP reinforcement for RC structures'*, published in 2001, presents FRP strengthening materials and techniques, design assumptions and description of strengthening for various actions (flexure, shear and torsion, confinement). The guidelines present a set of criteria essential to fulfilling the aim of ensuring a proper bond between composite materials and the strengthened member concrete surface.

The document also highlights the importance of the relevant requirements for the structure to be strengthened. Firstly, the element should be of a good quality – all wide cracks should be formerly injected to protect the member from problems caused by the penetration of water, such as steel reinforcement corrosion, and to avoid weakening of bond strength in places of horizontal cracking. The injection should be made with a low-viscosity resin, which allows for the connection of composite material with the concrete surface properly. Furthermore, the minimum concrete tensile strength should exceed 1.5 MPa and the recommended minimal concrete grade is C15/20 [N4].

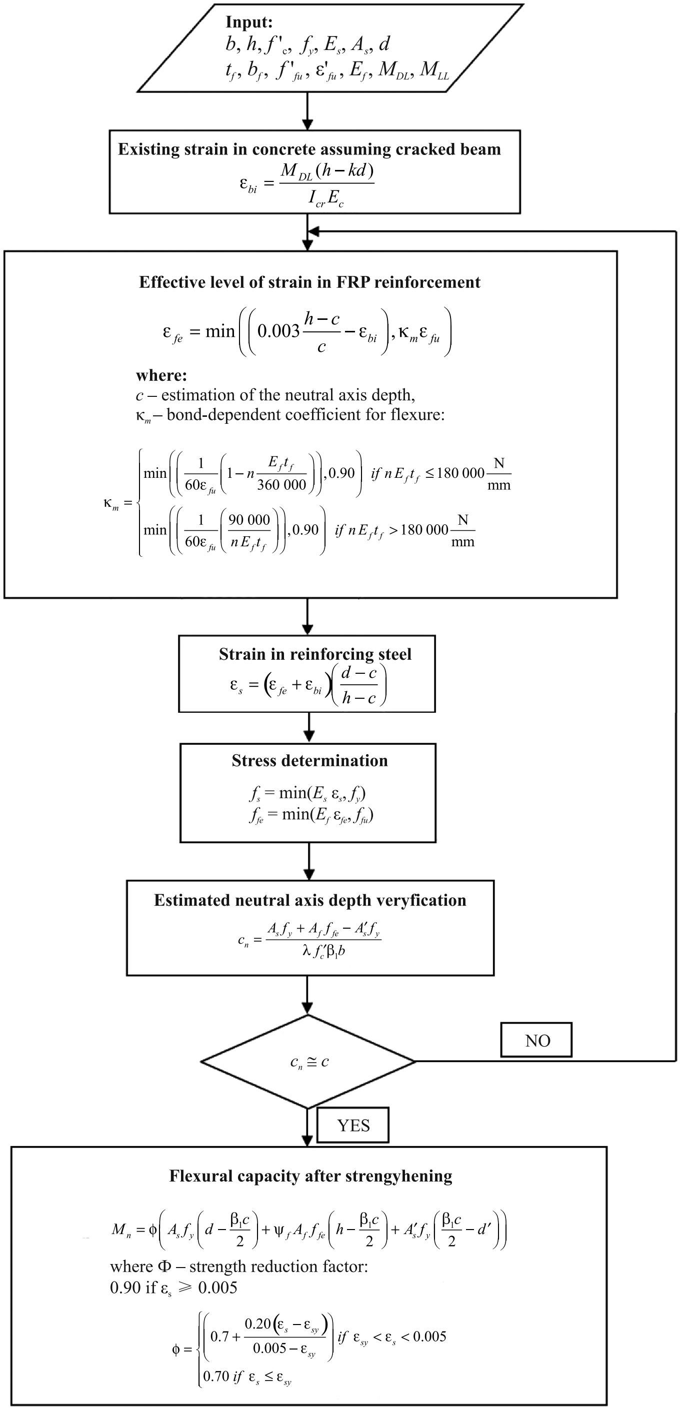

Fig. 2. ACI 440.2R-08 design procedure flowchart

The *fib* guidelines distinguish two groups of failure modes – assuming full composite action or loss of composite action. The former is divided into two cases – concrete crushing in compressed section due to exceeding maximum strains and FRP rupture caused by failure in tension. The latter group consist of various types of debonding, which means that the local deformation of the FRP strip in the critical cross-section is greater than can be carried by the bond between the strip (or laminate) and concrete substrate. There are two possible modes of failure: debonding of the laminate from its end and mid-span shear debonding.

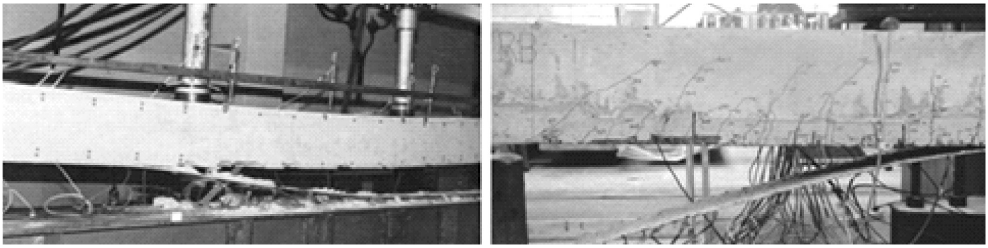

Fig. 3. Failure examples of concrete beams strengthened with EBR FRP [3]

The most probable failure model should be analysed checking ultimate limit state. Considering the strain distribution in the strengthened cross-section a verification whether the limit strain values are exceeded in FRP ($\varepsilon_f = \varepsilon_{\text{flim}}$, but $\varepsilon_c < \varepsilon_{cu}$) or in concrete ($\varepsilon_c = \varepsilon_{cu}$, but $\varepsilon_f < \varepsilon_{\text{flim}}$) is possible.

If failure is due to FRP rupture real strains in the most compressed concrete fibre should be determined and, furthermore, the constant values of ψ and δ_G coefficients are replaced by equations dependent on previously calculated concrete strains:

$$\psi = \begin{cases} 1000\,\varepsilon_c \left(0.5 - \dfrac{1000}{12}\varepsilon_c\right) & \text{for} \quad \varepsilon_c \leq 0.002 \\ 1 - \dfrac{2}{3000\,\varepsilon_c} & \text{for} \quad 0.002 \leq \varepsilon_c \leq 0.0035 \end{cases}$$

$$\delta_G = \begin{cases} \dfrac{8 - 1000\,\varepsilon_c}{4\,(6 - 1000\,\varepsilon_c)} & \text{for} \quad \varepsilon_c \leq 0.002 \\ \dfrac{1000\,\varepsilon_c\,(3000\,\varepsilon_c - 4) + 2}{2000\,\varepsilon_c (3000\varepsilon_c - 2)} & \text{for} \quad 0.002 \leq \varepsilon_c \leq 0.0035 \end{cases}$$

FRP material safety factors (Table 1), which are necessary for ULS calculations, are assumed depending on the type of application and on-site working conditions (type A – normal quality control conditions, prefab systems, type B – difficult conditions, wet lay-up conditions).

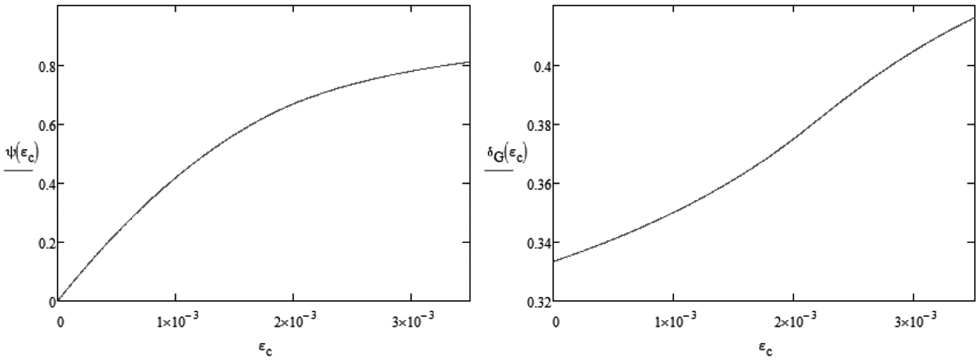

Fig. 4. Coefficients ψ and δ_G in dependence on ε_c

Table 1

Partial safety material coefficients for FRP [N4]

FRP type	Application type A	Application type B
CFRP	1.2	1.35
AFRP	1.25	1.45
GFRP	1.3	1.5

According to statements included in the *fib* guidelines, there is a necessity to check not only the Ultimate Limit State, but also the Serviceability Limit State conditions, for instance to prevent the strip from peeling-off phenomena, described above. Verification whether bond failure at the end anchorage and along the FRP occurs is essential for complete design and a good assessment of reliability. Broadly conceived research considering this problem was conducted in Poland by R. Kotynia [6]. SLS conditions not rarely govern the design, hence maximum acceptable strains in FRP should be reduced to a certain limit, for example as identified in the research [6]. However, in this paper authors focused only on the procedure given for ULS verification. Finally, after determining the failure mode, a calculation of the neutral axis depth of strengthened members can be evaluated based on the balance of forces equation, and therefore a value of increased flexural capacity is obtained from the equilibrium of bending moments. The *fib* Bulletin No. 14 design procedure flowchart is presented in Fig. 5.

2.3. CNR-DT 200/2004 Italian code

The procedure of strengthening design according to Italian code is quite different than the two previously mentioned examples. At the beginning, a calculation of maximum allowable strains in CFRP must be made, and the obtained value is later used in following calculations. This strain is determined to prevent the structure from debonding failure mode – the maximum

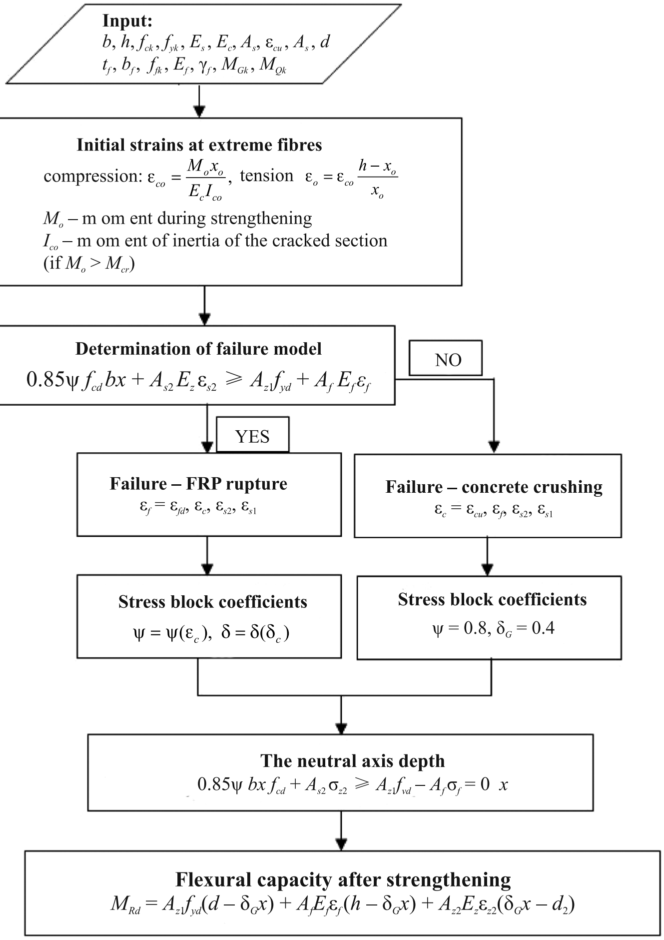

Fig. 5. *fib* Bulletin No. 14 design procedure flowchart

force that may be transferred from concrete to FRP material is evaluated with the specific fracture energy of the FRP-concrete interface is taken into account.

Then the determination of the failure mode is made, similar to the *fib* approach, however in Italian design codes, it is determined by comparing mechanical ratios: μ_s and μ_f – related to tension steel reinforcement and FRP system, respectively, with the balanced mechanical ratio μ_{f12}, calculated as follows (u is the ratio between compression and tension reinforcement area):

$$\mu_{f12} = \frac{0.8\,\varepsilon_{cu} \dfrac{h}{d}}{\varepsilon_{cu} + \varepsilon_{fd} + \varepsilon_0} - \mu_s (1 - u)$$

Once the failure mode is known, it is possible to evaluate the existing strains in concrete, FRP and steel reinforcement, determine the neutral axis depth and then calculate the flexural capacity (Fig. 6).

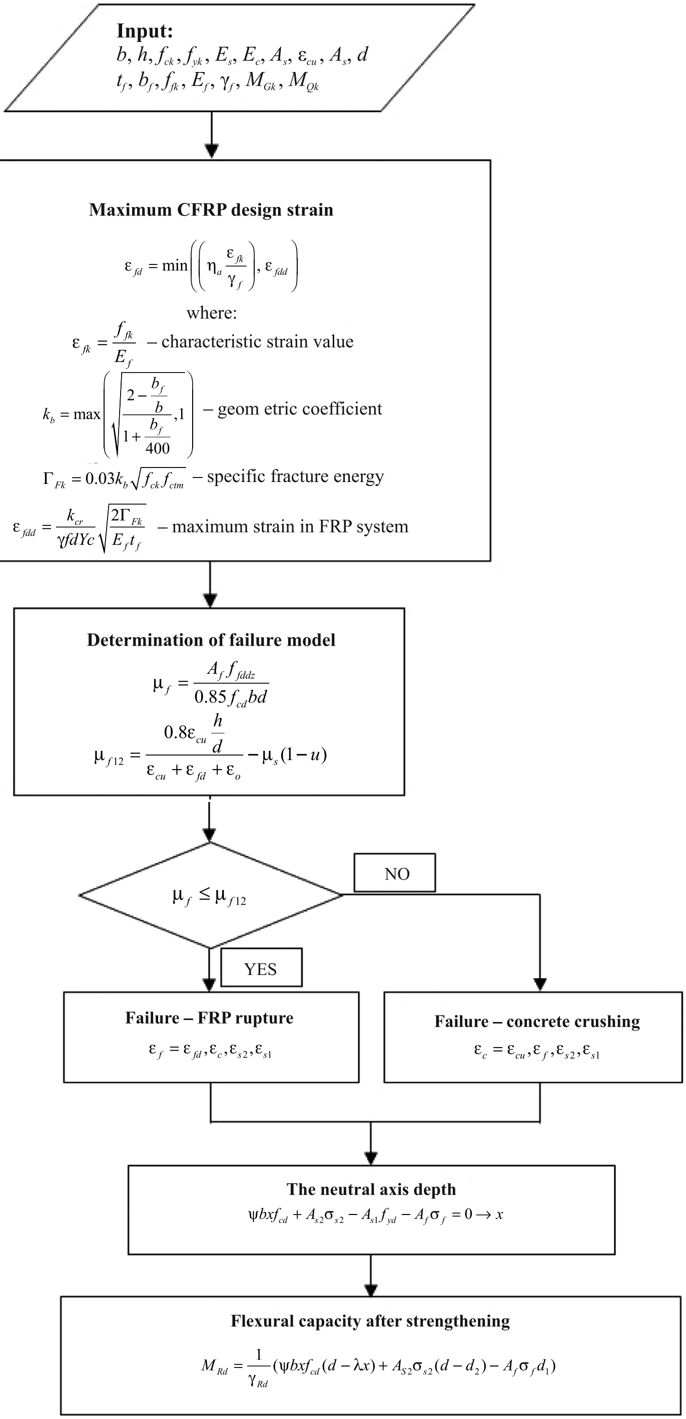

Fig. 6. CNR-DT 200/2004 design procedure flowchart

During the flexural capacity evaluation, the partial safety factor γ_{Rd} for resistance models is assumed as 1.0 (for bending), so there is no reduction of the final moment value, as it will be in case of shear, torsion strengthening or confinement (values of γ_{Rd} equal to 1.20 and 1.10, respectively) [N3].

3. Design example

3.1. Assumptions and the subject of analysis

A rectangular RC beam of height equal to 50 cm and width of 30 cm, with tensile reinforcement of 4 bars Φ = 16 mm was subjected to a simple comparative analysis. The assumed material parameters are as follows: concrete grade C30/37 and steel of tensile strength equal to 500 MPa. The beam was analysed as one, simple-supported, 5-metre long span.

The analysis was conducted in two steps. The first step was based on the determination of the load-bearing capacity of the beam according to each guideline, then loading was assumed to exceed the capacity in the middle cross-section of the beam span. The second step of the analysis was to determine an increase of capacity after strengthening the beam with the same CFRP strip.

3.2. Capacity and loading

Loading was the same for each case in the analysis, nevertheless there are differences in moment values. This issue is mainly due to various load partial safety factors for each of the considered guidelines (cf. Table 2).

Table 2

Total loading and maximum moment values for different guidelines

Guidelines & Combination scheme		Load safety factor	Loading (characteristic)	Loading (design)	Total loading (design)	Total moment (design)
		[–]	[kN/m]	[kN/m]	[kN/m]	[kNm]
ACI (1.4DL + 1.7LL)	DL	1.4	28.75	40.25	61.50	192.19
	LL	1.7	12.50	21.25		
FIB (1.35Gk + 1.5Qk)	Gk	1.35	28.75	38.81	57.56	179.88
	Qk	1.5	12.50	18.75		
CNR (1.4Gk + 1.5Qk)	Gk	1.4	28.75	40.25	59.00	184.38
	Qk	1.5	12.50	18.75		

The values of bending moment capacities were different in dependence on the guidelines, which is a result of various factors.

ACI318 [N1] defines no partial safety material coefficients as do FIB [N4] and CNR [N3], instead, a global strength reduction factor is determined dependent upon the character of cross-section work (flexure, compression, shear etc.). A slight difference also occurs in coefficients for rectangular shapes with regard to stress distribution – in ACI β_1 vary from 0.65 to 0.85, depending similarly on the concrete compressive strength. Furthermore, there is a difference in defining the modulus of elasticity of concrete E_c, which, according to ACI, is calculated in relation to concrete compressive strength and is distinctly lower.

Differences between the fib guidelines and CNR code are not major and they are mainly as a result of different partial safety material factors for concrete – γc = 1.6 for CNR and 1.5 for FIB.

All of these issues have an impact on the calculated capacities and caused the value for this beam according to ACI to be higher in comparison to FIB or CNR (cf. Table 3).

However, a set-up of calculated design capacities and the required capacity (due to the loading assumed above) proves that the deficiency of the bending moment capacity is approximately 20 per cent in each case, and the value for ACI is placed between those for FIB and CNR despite higher values of capacities calculated formerly.

Table 3

Total flexural capacities for different guidelines and values required due to loading

Guidelines	Capacity	Required capacity	Deficiency
	[kNm]	[kNm]	[%]
ACI	157.69	192.19	21.9
FIB	151.36	179.88	18.8
CNR	150.68	184.38	22.4

The second step of the conducted analysis was to determine the increase in capacity after strengthening the beam with a non-stressed, single CFRP strip.

- Assumed CFRP strip properties are: width – 80 mm,
- thickness – 1.2 mm (cross-section area – 96 mm^2),
- tensile strength – 3100 MPa,
- modulus of elasticity – 165 GPa.

The results of these calculations are presented in Table 4 and show that the value of capacity after strengthening was the highest according to FIB guidelines and caused an increase of 69.5% in comparison to ACI (25.3%) and CNR (16.5%).

The large difference between *fib* and the other guidelines is mainly due to not taking into account the assumptions of serviceability limit state (SLS) and verification of possible bond failure, which restrict maximum allowable FRP strains because of debonding failure mode (cf. p. 2.2). In ACI and CNR, this phenomenon is considered by various limiting factors included in the design procedures: κ_m (bond-dependent coefficient for flexure) in ACI and $\varepsilon_{\mathrm{fdd}}$ (maximum allowable strain in FRP system) in CNR.

Table 4

Flexural capacities after strengthening for different guidelines

Guidelines	Initial capacity	Capacity after strengthening	Strengthening ratio	Required capacity	*Is* strengthening sufficient?
	[kNm]	[kNm]	[%]	[kNm]	
ACI	157.69	197.62	25.3	192.19	YES
FIB	151.36	256.54 (210.38*)	69.5 (38.9%*)	179.88	YES
CNR	150.68	175.53	16.5	184.38	NO

Assuming that characteristic FRP strain limit value preventing from debonding is 0.85% instead of 1.7% alternative calculations were conducted and their results are presented in brackets in Table 4. A capacity of 210.38 kNm is obtained and a strengthening ratio is decreased to 38.9%.

Considering CNR results, strengthening was not sufficient for loadings assumed before, as obtained capacity does not exceed the required one. One of the possibilities for achieving the necessary capacity according to CNR is by using a wider strip – verification shows that a strip of width equal to 150 mm is sufficient (capacity raises to 186.31 kNm and strengthening ratio up to 24%).

4. Author's computer software

The author's program, which was written in the Delphi environment, allows for easy and fast checking of the conditions of the ultimate limit state for reinforced concrete elements with rectangular cross-section, strengthened with FRP strips for flexure (Fig. 7).

There is a possibility for the user to choose concrete grade from the list as well as to define material parameters manually; a similar situation occurs for FRP material characteristics. After determining the geometry of the element, reinforcement and materials calculations are made according to the *fib* procedure (as described in p. 2.2, of this article). The initial capacity and capacity after strengthening is evaluated.

A failure model is determined and illustrated with an appropriate graph of strains – real strains are calculated both in FRP strip and in concrete most compressed fibre and therefore ψ i δ coefficients are determined. It is necessary to give information about the bending moment during strengthening (for instance, in the case of possible partial unloading).

Finally, a strengthening ratio (defined as a ratio of the bending moment capacity after strengthening to the initial capacity) is calculated and presented (Fig. 8).

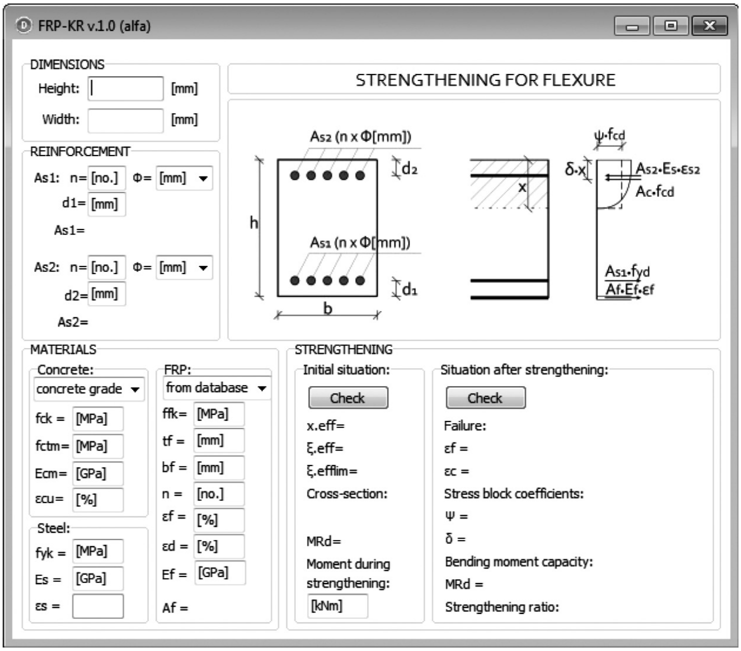

Fig. 7. Layout of the described program

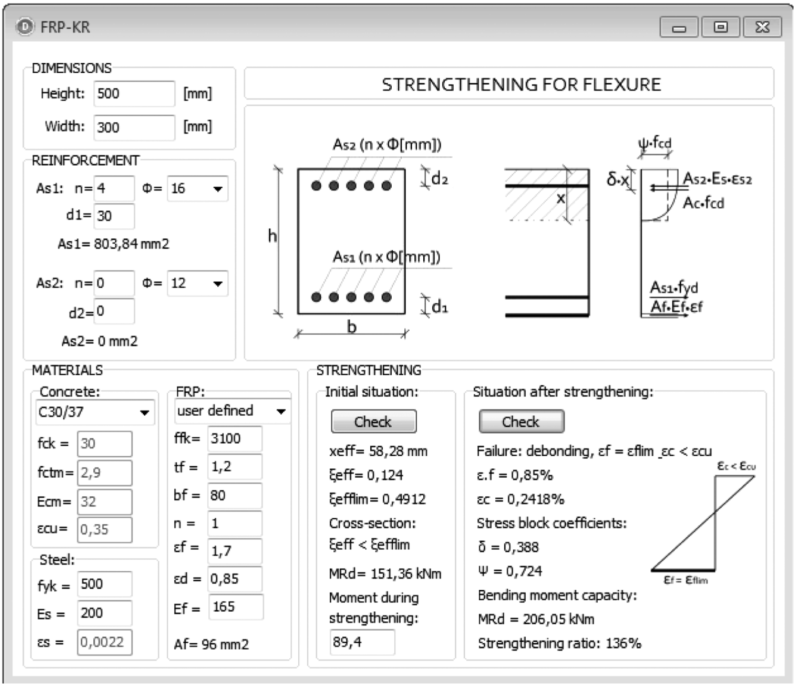

Fig. 8. Calculations and the way of presenting results

5. Conclusions

In Poland, there is no standard dedicated to strengthening reinforced concrete structures with FRP materials but, on the other hand, there are many different guidelines on this subject in the world, so it was necessary to perform a comparative analysis that will identify the most important differences between them.

This paper describes different design procedures for the flexural strengthening of reinforced concrete beams with FRP composite materials, however, only the ultimate limit state was taken into consideration. We analyzed the American and Italian standards as well as fib guidelines which, although are the oldest, are considered to be the basic document in many countries. It should be noted that in the analyzed strengthening technology, the issue of the debonding of the FRP laminate is essential. All the analyzed guidelines address this issue, but in different ways.

Comparing the results of performed calculations is not easy, because the final results are affected by a number of small differences in many factors. The highest initial bending moment capacity was obtained from the ACI318 [N1] design model. The main difference is due to a lack of partial material safety factors, which appear in Eurocodes, and an existence of the global strength reduction factor which decreases the value of the nominal bending moment.

Load partial safety factors are different in each set of guidelines, what contributes to different values of required strength although the same load values were assumed. The bending moment required by ACI is the highest, and values of moments obtained for CNR and FIB do not differ significantly.

The most conservative values of strengthened cross-section capacity were obtained for the CNR code, and the least from the fib recommendations.

Finally, the strengthening ratio (calculated as the ratio of the capacity increase to the initial capacity) differs significantly between three analysed approaches. It is mainly due to the lack of straight verification of debonding failure mode in the ULS fib procedure. In ACI and CNR, factors which aim is to prevent debonding, are included in the design ULS procedures, whereas in fib, these calculations are separate.

The article also presents the concept of the author's program for quick verification of bearing capacity for the bent RC cross-section strengthened with FRP laminates. For now, this is the start of work on a program for engineering use – it needs to be significantly expanded.

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