EBR FRP STRENGTHENING OF RC STRUCTURES – PROPOSAL OF DESIGN APPROACH TAILORED FOR POLISH REALITY

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Abstract

Technique of strengthening with FRP products has been utilized for almost 20 years. Since that time, several countries prepared national standards regulating design of FRP [1, 2, 3]. Poland is still one of the countries where that kind of document has not been prepared yet. The only document proposing requirements for application of external, glued strengthening of bridges was published by Łagoda in 2002 [4].

The authors intention for this paper is to make an effort of systematization the designing rules of RC elements strengthened with FRP, adjusted to the Polish design practice. It is focused on real situation showing the problem of the strengthenings of cracked and retrofitted concrete, proposed guidelines refer also to problem of strengthenings' effectiveness in different environmental conditions.

Guidelines presented below, based on the actual scientific state of art and code, were prepared mainly to provide to Polish engineers a coherent and user-friendly tool, but also to give a foundation for wider discussion on necessity of regulation of this issue in Poland.

Streszczenie

Technika wzmocnień konstrukcji FRP wykorzystywana jest od niemal 20 lat. W tym czasie w niektórych krajach powstały normatywy regulujące projektowanie tych wzmocnień [1, 2, 3]. Polska jest wciąż jednym z państw, w których nie opracowano takiego dokumentu. Jedynym opracowaniem zawierającym wytyczne do projektowania zewnętrznych, klejonych wzmocnień mostów jest praca Łagody z 2002 [4].

Artykuł stanowi próbę usystematyzowania zasad projektowania wzmocnień konstrukcji żelbetowych nakładkami z kompozytów FRP dla potrzeb polskiej praktyki projektowej. Intencją autorów było opracowanie procedur odzwierciedlających rzeczywiste warunki pracy wzmocnień elementów zarysowanych i naprawianych, dlatego przedstawione wytyczne wskazują również zagrożenia wynikające z wpływu środowiska na pracę wzmocnień.

Przedstawione niżej wytyczne były budowane przede wszystkim w taki sposób by dostarczyć polskiemu projektantowi spójne i przyjazne narzędzie pracy, ale także po to by stworzyć podstawę do szerszej dyskusji o konieczności uregulowania tej kwestii w Polsce.

Keywords: **Externally Bonded Reinforcement (EBR); Fiber Reinforced Polymer (FRP); Design procedure; Reinforced concrete; Strengthening.**

1. INTRODUCTION

Strengthening of reinforced concrete (RC) structures with fiber reinforced polymer (FRP) technique has been commonly utilized in Poland for over decade. The very first application raised significant problem: how to design safely FRP systems fulfilling at the same time requirements of the Polish National Codes. Suppliers of the FRP delivered also some foreign and international documents shaping the frames of the design process, but still the problem was not solved – those documents were barely available for practicing engineers, and often barely applicable to Polish reality. The language was also an obstacle – till 2002

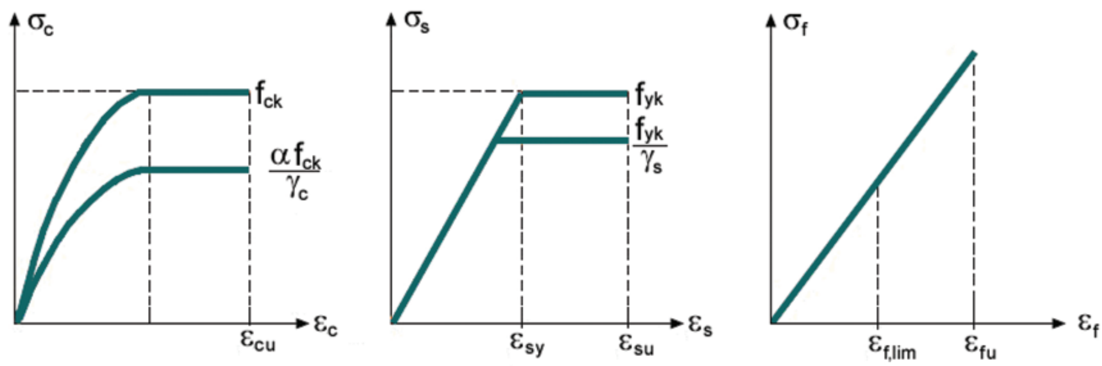

Figure 1.
σ – ε diagrams for material models (concrete, steel, FRP)

designers were not equipped by any document written in Polish [4]. Lack of design tools was paired with the trust in unlimited possibilities of those new systems. In many cases engineers responsible for all of the phases of construction process are refusing to utilize new technologies, preferring to step back and use traditional solutions not charged with hazard of potential lack of compatibility with legal state in Poland.

In 1999, authors of this paper undertook the investigation program concerning complex state of stresses in strengthened RC elements [5]. Results obtained from laboratory tests [6], [7], verified in analytical field [8], [9] allowed to formulate a set of design procedures. They were focused on real life situation showing the problem of strengthenings of cracked and retrofitted concrete, proposed guidelines refer also to problem of strengthenings' effectiveness in different environmental conditions.

Simplified procedures were adjusted to obligatory Polish Code [10], Eurocode [11] and compared with existing international [12] and foreign, national documents of that kind [1-3, 13, 14]. Elaborated material was presented and discussed during major Polish conferences [15, 16], and international congresses [17] concerning FRP. Precious advices by international experts and presented new ways of design allowed next corrections.

As the guidelines are addressed to practicing structural engineers, authors tended to simplify design process. Frame of the guideline was mainly based on solutions proposed by fib Bulletin 14 [12] but some elements were built with tools originally invented by authors (simplified method of design of element subjected to flexure).

2. DESIGN RULES

Design of RC elements strengthened with externally bonded fiber reinforcement (EBR) does not differ much from the design process of traditional RC elements. It starts with determination all of possible combinations of loads and failure modes. Designer should consider additional conditions related to bond behavior of strengthening elements glued to concrete surface.

The standard for EBR FRP strengthening shall establish strict rules for concrete surface quality. Similar rules shall be given for concrete resistance, level of concrete deterioration (corrosion, carbonation, etc). Steel section area should be reduced due to observed range of steel corrosion. Proposed in the codification document design proposals are valid under the condition of fulfilling those demands.

Design procedure follows rules of ultimate (ULS) and service (SLS) limit states defined in Polish Codes [10, 11].

Prediction of work conditions and prevention of failure shall comprise risks coming from typical situations complemented with FRP strengthening failure as accidental case of sudden loosing FRP strengthening (i.e. fire). In case of fire risk, designer shall follow rules of Fire Protection requirements, providing appropriate insulation and/or mechanical anchors of fiber reinforcement.

Adequately to ULS requirements designer is obliged to take into consideration all possible failure modes, in this number all of risks which may occur during construction, transport or assembly.

Concrete and steel properties shall be taken according to the relevant regulations [10] (Fig. 1).

Value of characteristic strength f_{fk}, limit strain ε_{fu} and

Table 1.
FRP material safety factors γ_f

Type of fibers	Application of FRP in conditions allowing complete quality control on every step of realization	Application in particularly difficult conditions
carbon	1.2	1.35
aramid	1.25	1.45
glass	1.3	1.5

Table 2.
Coefficient depending on conditions of exposure η

Type of fibers	inside	outside	aggressive environment
	X0, XC1	XC2, XC3, XC4, XD1, XD2 , XS1, XS2, XF1	XS3, XD3, XF2, XF3, XF4, XA1, XA2, XA3
carbon	0.95	0.85	0.85
aramid	0.85	0.75	0.7
glass	0.75	0.65	0.5

modulus of elasticity of FRP materials may be taken from manufacturer or established on the base of independent laboratory tests. Design values X_d are obtained by multiplying characteristic values X_k by safety factors:

$$X_d = \frac{\eta}{\gamma_f} \cdot X_k \tag{1}$$

Value of material coefficient γ_f depends on the type of fibers utilized in laminate and on conditions of strengthening (tab.1)

Presented in table 2 coefficient of exposure conditions η relates to durability of strengthening with structure's work conditions and it should be assumed according to [10].

Verification of ULS consist in checking level of stresses, deflection and cracking. It is also required to check partial debonding of EBR when mechanical anchorages are predicted.

Due to national regulations ULS analysis may be performed with assumption of complete interaction of fiber reinforcement with concrete in limited, defined range of strains. In different situations mechanic rules of classic RC design shall be implemented.

3. PROPOSALS OF DESIGN PROCEDURES

3.1. Bond behavior

Failure followed by the rupture of FRP reinforcement occurs rather rarely, especially when high strength carbon strips are applied. In general practice failure appears as a result of EBR debonding, where debonding means complete loss of composite action between concrete and FRP plate.

Bond failure may occur in different zones and as a result of different causes. Usually it is initiated in the cracked zone of concrete. Of course, debonding localized next to the crack still does not mean a failure, but under the growing load it propagates finally inducing separation of the FRP plate from the concrete.

Dominant mode, that affects the flexural strength and ductility is intermediate crack (IC) debonding caused by flexural cracks. Effect of that failure could be taken into consideration by strain limitation. Based on Nidermeier proposal [12] detailed method of calculation is too complex for practical use.

Externally mounted reinforcement will work effectively only when the proper anchoring is ensured. Minimum anchoring length is equal:

$$l_{b,\max} = 1.44\sqrt{\frac{E_f t_f}{\sqrt{f_{ck} f_{ctm}}}} \tag{2}$$

and allowable stress which can be transferred from EBR to the concrete at the end of anchorage zone:

$$\sigma_{fdd,anch} = \frac{0.23}{\sqrt{\gamma_c}}\sqrt{\frac{E_f\sqrt{f_{ck} f_{ctm}}}{t_f}} \tag{3}$$

where E_f is the modulus of elasticity of FRP; t_f thickness of FRP; and f_{ck}, f_{ctm} successively characteristic and mean value of the concrete strength.

Maximum force is still growing beyond the anchorage zone, that phenomenon could be expressed by multiplying force at the end of anchorage by coefficient k_{cr}. Based on Italian proposal [3] k_{cr} =3 could be taken into consideration.

According to (3) simplified formula for maximum strain due to IC (intermediate crack) debonding of FRP reinforcement will take form:

$$\varepsilon_{fdd} = \frac{k_{cr}\sigma_{fdd,anch}}{E_f} \approx \frac{0.7}{\sqrt{\gamma_c}}\sqrt{\frac{\sqrt{f_{ck} f_{ctm}}}{t_f E_f}} \tag{4}$$

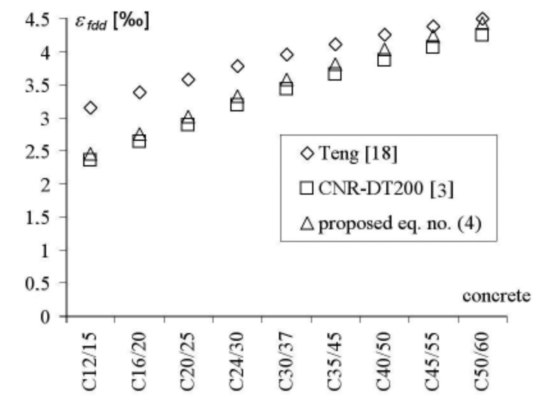

Figure 2.
Exemplary comparison of maximum allowed strain due to IC debonding of FRP plate 1.2 mm thick

Comparison presented in Figure 2 shows, that proposed formula gives results comparable with other well known proposals [3, 18]

Giving consideration to maximum strain allowed by system supplier ε_{fu} and safety coefficients permissible design strain equals:

$$\varepsilon_{fd} = \varepsilon_{fdd} \leq \frac{\eta \varepsilon_{fu}}{\gamma_f} \tag{5}$$

Development of the crack in shear zone could lead to the FRP debonding also called Critical Diagonal Crack debonding. It is particularly dangerous because it usually proceeds within anchoring zone. Excessive cracking of shear zone should be avoided by increasing its shear capacity with additional FRP U-shape jackets fixing flexural FRP plate.

3.2. Flexure

For calculations of elements under bending and longitudinal force following assumptions should be used:

- plane section remains plane after loading;
- all of the elements of cross-section, excluding prestressing tendons, placed in fibers equally distant from strain zero line are deformed identically;
- the strain in steel and FRP reinforcement, whether in tension or compression, is the same as in the surrounding concrete;
- tension in concrete is neglected;
- stresses in reinforcement and FRP are derived from linear dependences in Fig. 1
- there are two main failure models possible, concrete crushing and FRP debonding or rupture.

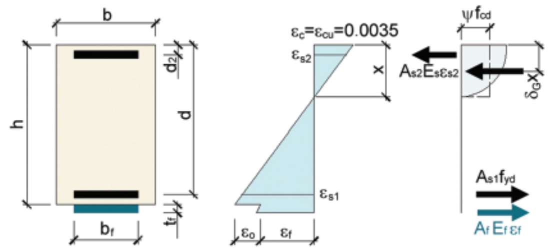

Figure 3.
Assumptions for design procedures of elements under flexure [8, 9]
a) geometry, b) strains, c) stresses

Rules of design bending moment are based on principles of steel reinforced concrete. Location of neutral axis is obtained from force equilibrium and strain compatibility:

$$\psi E_c \varepsilon_c b x = A_{s1} f_{yd} + A_f E_f \varepsilon_f \tag{6}$$

$$\varepsilon_f = \varepsilon_c \frac{h-x}{x} - \varepsilon_0 \tag{7}$$

Then design moment is given by moment equilibrium (Fig. 3):

$$M_{Sd} \leq M_{Rd} = A_{s1} f_{yd}(d - \delta_G x) + A_f E_f \varepsilon_{fd}(h - \delta_G x) \tag{8}$$

where f_{cd} design value of concrete compressive strength; f_{yd} design steel yield strength; b width of section; x depth of the compression zone; A_{s1} area of longitudinal reinforcement; A_f area of FRP reinforcement; ε_0 initial strain in the extreme tensile fiber (when sagged beam is strengthened); ε_f FRP strain.

When concrete crushing is expected $\varepsilon_c = \varepsilon_{cu}$, then stress block coefficients δ_G, ψ are derived from rectangular stress form:

$$\delta_G = 0.4 \tag{9a}$$

$$\psi = 0.8 \tag{9b}$$

and the product of the concrete modulus of elasticity and concrete extreme strain is equal:

$$E_c \varepsilon_c = f_{cd} \tag{10}$$

When failure is followed by the IC debonding compressed concrete does not reach its ultimate strain ε_{cu}. In this case traditional, rectangular stress block cannot be used. To describe new parabolic character of stress block, value of coefficients δ_G, ψ, should be changed. For the parabolic – linear estimation following formulas may be used:

$$\psi = \begin{cases} \dfrac{1000\varepsilon_c\left(6-1000\varepsilon_c\right)}{12} & \text{for } \varepsilon_c \le 0.002 \\ 1-\dfrac{2}{3000\varepsilon_c} & \text{for } 0.002 \le \varepsilon_c \le 0.0035 \end{cases} \tag{11a}$$

$$\delta_G = \begin{cases} \dfrac{8-1000\varepsilon_c}{4\left(6-1000\varepsilon_c\right)} & \text{for } \varepsilon_c \le 0.002 \\ \dfrac{1000\varepsilon_c\left(3000\varepsilon_c-4\right)+2}{2000\varepsilon_c(3000\varepsilon_c-2)} & \text{for } 0.002 \le \varepsilon_c \le 0.0035 \end{cases} \tag{11b}$$

Presented above detailed method leads to cubic equation which should be derived with computer assistance. That inconvenience reduces its practical usability. Adopting Chinese proposals [14] given above coefficients δ_G, ψ could be described by new formula:

$$\delta_G = 0.33+0.07\frac{\varepsilon_{fd}}{\varepsilon_{f,cu}} \tag{12a}$$

$$\psi = 0.35+0.45\frac{\varepsilon_{fd}}{\varepsilon_{f,cu}} \tag{12b}$$

where $\varepsilon_{f,cu}$ is the FRP strain, when the external concrete fiber reaches the level of strains equal to crushing. It ought to be calculated using force and strain equilibrium (6), (8) with δ_G, ψ , coefficients for rectangular stress block (9a), (9b).

Statistical analysis proves effectiveness of that simplified method, especially for practical engineering use. Method was verified by the calculations of dozens strengthened beams. Comparison of received failure bending moments (M_{simpl}) with theoretical load capacities given by classic detailed model (M_{detail}) is shown in Figure 4. Correlation coefficient of presented results is equal 0.99977, whereas standard deviation is equal 0.0131.

3.3. Compression

According to Polish Code [10] triaxial stress state can be taken into account only when column is compressed with eccentricity not exceeding the following value:

$$e_{tot} \le 0.125D \tag{13}$$

Effect of sustained load should be considered if $\frac{l_0}{i} = \frac{4l_0}{D} \le 25$, where l_0 is the design length of the column; i radius of gyration.

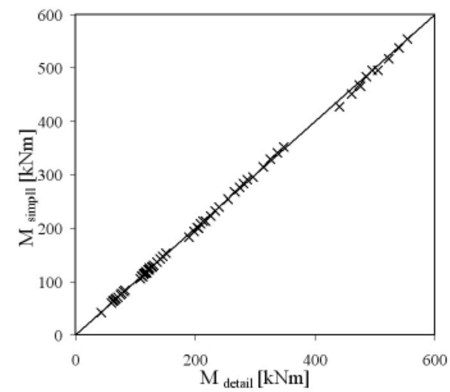

Figure 4.
Comparison of failure moments given by detailed and simplified method

Condition expressing load capacity of confined column can be written as:

$$N_{Sd} \le N_{Rd} = A_c\left(f_{cd}+1.25\rho_f E_f \varepsilon_{fd} k_e k_s k_d\right)+A_s f_{yd} \tag{14}$$

where:

A_c effectively confined concrete core;

$k_e = 1-\dfrac{8e_{tot}}{D}$ – effect of eccentricity;

$k_s = \left(1-\dfrac{s'}{2D}\right)^2$ – effect of partial confinement;

$k_d = \dfrac{1}{1+\left(\dfrac{s}{\pi D}\right)^2}$ – effect of fibers orientation (helical wrapping).

Reinforcement ratio of column confined with strips equals:

$$\rho_f = \frac{4t_f}{D} \quad \text{(for fully confined),} \tag{15a}$$

$$\rho_f = \frac{4t_f b_f}{Ds_f} \quad \text{(for partially confined).} \tag{15b}$$

Effect of partial confinement and used geometrical symbols are presented in Fig. 5.

3.4. Tension

Eccentric tension

Design of such an element is similar in its approach to design of element subjected to flexure. It bases on the assumption of flatness of the section before and after the deformation. It bases also on equations of inner forces equilibrium.

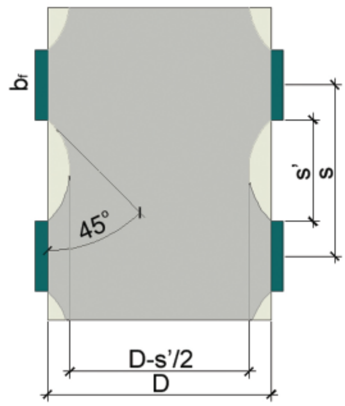

Figure 5.
Effectiveness of partial confinement

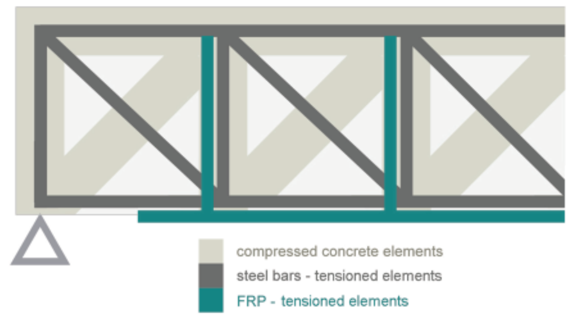

Figure 6.
Ideogram of truss analogy

In this design proposal two schemes of failure should be taken into consideration:

– Concrete crush in compressive zone connected with yielding of main reinforcement of tensile zone.

– FRP strengthening rupture and yielding of tensile steel rebars.

First scheme is typical for large eccentricity and significant appearance of bending moment. Equilibrium equations for longitudinal forces and bending moment are as follows:

$$\psi f_{cd} bx + A_{s2} E_s \varepsilon_{s2} + N_{Sd} = A_{s1} f_{yd} + A_f E_f \varepsilon_f \tag{16}$$

$$N_{Sd} e_{cc} \leq A_{s1} f_{yd} (d - \delta_G x) + A_f E_f \varepsilon_f (h - \delta_G x) + A_s E_s \varepsilon_{s2} (\delta_G x - d_2) \tag{17}$$

where eccentricity (by the center of gravity of compressive zone) is equal:

$$e_{cc} = e_{tot} + 0,5h - \delta_G x \tag{18}$$

According the rule of section flatness the strains may be derived as in the section of element subjected to flexure.

When FRP rupture occurs, equilibrium equation for axial forces and moments takes form:

$$\psi E_c \varepsilon_c bx + A_{s2} E_s \varepsilon_{s2} + N_{Sd} = A_{s1} f_{yd} + A_f f_{fd} \tag{19}$$

$$N_{Sd} e_{cc} \leq A_{s1} f_{yd} (d - \delta_G x) + A_f f_{fd} (h - \delta_G x) + A_s E_s \varepsilon_{s2} (\delta_G x - d_2) \tag{20}$$

Condition of deformations is equal to those in equation (7), symbols as defined in point 3.2.

<u>Axial tension</u>

ULS of elements under axial load, in which steel rebars capacity was not exceeded may be checked by the following condition:

$$N_{Sd} \leq N_{Rd} = f_{yd} (A_{s1} + A_{s2}) + \sum f_{fd} A_f \tag{21}$$

3.5. Shear forces

Design method for elements subjected to shear forces is based on truss analogy in form similar to Mörsch method (Fig. 6). According to Polish Code [10] designer is allowed to choose independently the inclination of cross braces in defined range. Inclination of main compressive stresses corresponds with inclination of compressed cross brace of the truss. It is defined by angle Θ. Value of cotangent shall be limited to the range: $1 \leq \cot\Theta \leq 2$.

Tensile stresses, responsible for inclined cracking are to be carried by transversal reinforcement (bent steel bars, stirrups and FRP strengthening). In this case total shear reinforcement should fulfill the rule:

$$V_{Sd} \leq V_{Rd3} = V_{Rd31} + V_{Rd32} + V_{Rd3f} \tag{22}$$

where capacity of stirrups V_{Rd31} and inclined (with angle α_1) rebars V_{Rd32} are calculated due to the Polish Code [10]. Capacity of FRP strengthening is described as:

$$V_{Rd3f} = \varepsilon_{fd} E_f \rho_f b_w z (\cot\Theta + \cot\alpha_2) \sin\alpha_2 \tag{23}$$

where ρ_f is the FRP reinforcement ratio; z arm of internal forces, approximately could be assumed $z = 0.9d$; α_2 inclination angle of FRP fibres.

Total strength of FRP reinforcement allowed by producer ε_{fu} could be taken into consideration only when

FRP is overwrapped. In other cases maximum strain ε_{fd} should be limited appropriately to the rules presented in chapter 3.1.

According to assumption of truss model, longitudinal rebars are participating in carriage of normal forces. Except resulting from flexure, internal force F_{td} has to bear additional force ΔF_{td}:

$$\Delta F_{td} \leq 0,5 V_{Sd}\left(\cot\Theta - \frac{V_{Rd32}}{V_{Rd3}}\cot\alpha_1 - \frac{V_{Rd3f}}{V_{Rd3}}\cot\alpha_2\right) \quad (24)$$

It is possible to use additional longitudinal FRP elements. Its area may be estimated with the use of following formula:

$$A_{sf} = \frac{F_{td} - A_{sL} f_{yd}}{\varepsilon_{fd} E_f} \quad (25)$$

where A_{sL} means area of steel reinforcement.

3.6. Torsion

Design of element subjected to torque is based on spatial Rausch truss, in version modified by Leonhardt (Fig. 7).

Torque capacity due to tensed strut is derived as a sum of existing longitudinal reinforcement capacity and capacity of fiber reinforcement:

$$T_{Sd} \leq T_{Rd2} = T_{Rd2,w} + T_{Rd2,f} \quad (26)$$

where:

$T_{Rd2,w}$ – capacity of longitudinal steel reinforcement

$T_{Rd2,f}$ – capacity of externally bonded FRP, that shall be derived as:

$$T_{Rd2,f} = 2A_c \varepsilon_{fd} E_f \frac{t_f b_f}{s_f}\left(\cot\Theta + \cot\alpha\right)\sin\alpha \quad (27)$$

Where t_f, b_f, s_f thickness, width and spacing of FRP wrapping. Value of cotangent Θ is defined as above (point 3.5), α is an angle of inclination of circumferential reinforcement.

Similarly to elements subjected to shear forces for non wrapped elements, the strains of FRP should be limited correspondingly to chapter 3.1.

The strength $T_{Rd2,sl}$ transferred thru longitudinal reinforcement:

$$T_{Rd2,sl} = \frac{A_{sl} 2 f_{yd} A_k}{u_k\left(\cot\Theta - \cot\alpha\right)} \quad (28)$$

3.7. Design approach for deteriorated and repaired elements

For successful use of FRP strengthening it is crucial to define the actual state of the structure to be reinforced with. Given above (points 3.1 to 3.6) are valid for initial strengthening of non-deformed and uncracked elements.

Strengthening of deformed and still not cracked elements is connected with necessity of estimating actual level of strains in concrete. Results of those analyses should be taken into consideration.

Different situation occurs when the element to be strengthened is pre-cracked. In this case usually repairing and retrofitting systems are implemented. The systems may improve pre-cracked element's load carrying capacity, as it was experimentally proved in research [5], but not always this situation takes place. Increase of failed element's capacity is visible in elements subjected to shear and flexure, repaired with epoxy injection. In elements under torque there is no visible improvement.

Increase of capacity depends also on the type of repair and the quality of its performance.

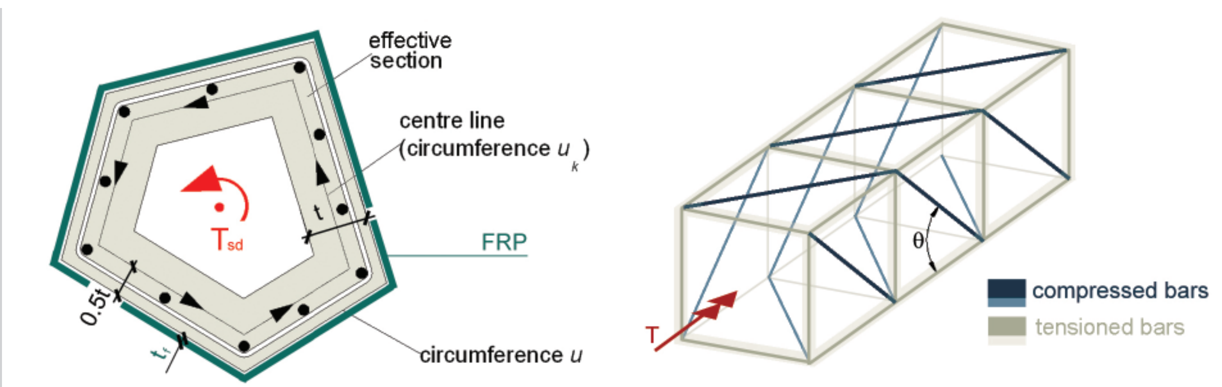

Figure 7.
Strut and tie model utilized for torsion

Designer is then recommended to take this important factor into account, especially when designing strengthening of elements under torsion, or under complex load scheme (i.e. edge beams, where due to Polish Code and tradition torsion is not taken into account). It is suggested to decrease capacity of repaired RC element subjected to torque up to 80% of original load carrying capacity.

3.8. Influence of fire and high temperatures

Temperatures may negatively influence components of FRP system. High temperature significantly change mechanical features of the glue layer between concrete surface and FRP, negative temperatures affect mainly the resin matrix of FRP stripes.

Designers aware of the risk of increased temperatures long term activity on the FRP strengthening (insulation, industrial environment of steelyards, power stations etc) shall take this factor into consideration during the process of designing.

One of the possibilities is predicting thermal insulation. In other cases it is recommended to reduce loading capacity of FRP strengthening or design it with mechanical anchorage. Vaz et al [19] indicates that normal conditions of work of FRP strengthening with standard epoxy glue may be consider up to 40ºC. Between 40°C and 60°C capacity of the strengthening shall be reduced at least by 40%.

Gamage et al [20] research results show that the epoxy adhesive temperature should not exceed 70°C in order to maintain integrity between the FRP and concrete at high temperature.

4. EXAMPLE

Beam shown in Figure 8 should be strengthened due to the new service load requirements. Total unloading during strengthening works is possible – we can neglect strain   in the tensed zone.

Geometry:

dimensions: $b = 0.3$ m; $h = 0.6$ m; $a_1 = 0.03$ m;

$d = 0.57$m;

main reinforcement: $A_{s1} = 4.52 \cdot 10^{-4}$ m^2;

stirrups $A_{sw1} = 0.565 \cdot 10^{-4}$ m^2; $s_1 = 0.3$ m

Material properties

steel A1 (main reinforcement and stirrups):

$f_{yd} = 210$ MPa; $E_s = 200$ GPa

concrete C16/20: $f_{ck} = 16$ MPa; $f_{cd} = 10.6$ MPa;

$f_{ctm} = 1.9$ MPa; $f_{ctd} = 0.8$ MPa

FRP plate: $\varepsilon_{fu} = 0.6\%$, $E_f = 165$ GPa,

FRP sheet: $\varepsilon_{fuw} = 0.8\%$, $E_{fw} = 230$ GPa

Maximum internal forces including beams deadweight (live load $q_k = 30$ kN/m; live load coefficient $\gamma_f = 1.2$):

$M_{Sd} = 81.9$ kNm; $V_{Sd} = 81.9$ kN

Beams bearing capacity in accordance with [10] (without strengthening):

$M_{Rd} = 52.7$ kNm; $V_{Rd1} = 70.03$ kN; $V_{Rd2} = 458$ kN;

$V_{Rd31} = 20.3$ kN

Element ought to be strengthened in the flexure and shear zone.

Flexure

Optimizing the strengthening IC delamination (or FRP rupture) model should be taken into consideration. According to formula (4) and (5) for plate $t_f = 1.2$ mm thick:

$\varepsilon_{fd} = \varepsilon_{fdd} = 0.302\%$

Solving the equations (6), (7), (8) and (11) we receive system of three equations with three unknowns:

$$265 \cdot 10^3 \varepsilon_c^2 (6 - 1000\varepsilon_c) x - 94.95 - 498 \cdot 10^3 A_f = 0$$

$$94.95\left(0.57 - \frac{8 - 1000\varepsilon_c}{4(6 - 1000\varepsilon_c)} x\right) + 498 \cdot 10^3 A_f \left(0.6 - \frac{8 - 1000\varepsilon_c}{4(6 - 1000\varepsilon_c)} x\right) = 81$$

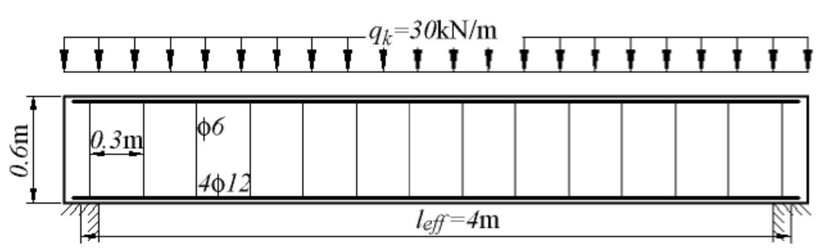

Figure 8.
View of calculated beam

$$\varepsilon_c(0.6-x)-0.004x=0$$

and its solution: $A_f = 1.16 \cdot 10^{-4}\,\text{m}^2$ for $x = 0.132$ m and $\varepsilon_c = 0.085\% < 1.2$ mm

We can strengthen beam using one CFRP plate 100×1.2 mm or two plates 50×1.2 mm.

Shear

We will use overwraping CFRP sheet with transverse direction of wrapping ($\alpha_2 = 90°$C), maximum allowed strain that could be taken into consideration, due to (5) and application inside in normal condition:

$$\varepsilon_{fdw} = \frac{\varepsilon_{fuw} \cdot 0.95}{1.2} = 0.633\%$$

Considering the force taken by stirrup reinforcement we can find required capacity of overwrapped strengthening:

$$V_{Rd3f} \geq V_{Sd} - V_{Rd31} = 61.6\text{kN}$$

Using formula (23) and reinforcement ratic $\rho_f = 2t_{fw}/b$ we can find required thickness of wrapped CFRP sheet:

$$t_{fw} \geq \frac{V_{Rd3f}}{2\varepsilon_{fdw}E_{fw}0.9d(1+\cot\alpha_2)\sin\alpha_2} = 0.041\text{mm}$$

Shear zone can be strengthened using single wrapped sheet 0.045 mm thick.

5. CONCLUSIONS

This paper presented some proposals of FRP design adjusted to existing Polish standardization for RC structures. Specifity of presented set of design proposals is based on the following assumptions:

– design values are determined by material factor and exposure coefficient,
– flexural capacity is limited by maximum concrete strain and FRP Intermediate Crack debonding,
– rectangular stress block of compressed concrete for analyses of flexural capacity caused by concrete crushing and FRP debonding with simplified model,
– parabolic stress block of compressed concrete for detailed analyses of flexural capacity caused by FRP rupture or debonding,
– effect of confinement limited by eccentricity,
– truss models for shear and torsion with limited FRP strain in accordance with debonding and system supplier recommendations,
– strong impact on elements with history of load, cracked and repaired with retrofitting systems,
– predicted elevated temperatures long term influence on the FRP strengthening shall be taken into consideration during the process of calculation.

Authors were trying to propose coherent and user-friendly tool for practicing designers answering for majority of questions which were usually appearing during design process. Set of proposals was demonstrated in practical way in design example. However, some of the answers were just sketched and, it is certain, that proper formulation of FRP designing guidelines still demands more work supported by producers of FRP systems and experts discussion.

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