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STRENGTH MODEL FOR CALCULATING CENTRALLY COMPRESSED CONCRETE ELEMENTS WITH COMPOSITE REINFORCEMENT, TAKING INTO ACCOUNT THE SPACING OF STIRRUPS

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Abstract. The article discusses the relevance of developing techniques for confined elements with FRP reinforcement. The method for calculating centrally confined concrete columns with non-metallic GFRP reinforcement (without regard to its compression work) is proposed for the first time in the Russian Federation. The strength model was developed based on the well-known theoretical model of confined concrete. The article considers the effect of strengthening the concrete core of the columns, which is obtained due to the more frequent placement of both transverse and longitudinal reinforcement. The dependence of the bearing capacity of concrete columns on the strength of the transverse reinforcement material is shown. It was proved that with a decrease in the spacing of the longitudinal reinforcement, the area of the effectively confined concrete core inside the reinforcement cage increases. It was shown that, due to the low compressive modulus of elasticity of the FRP reinforcement, the stress in it will be comparable to the concrete stress. Therefore, the compressive strength of the FRP reinforcement can be neglected in the case of determining the bearing capacity of centrally confined concrete elements. As a result, a strength model for calculating confined concrete elements with FRP reinforcement was proposed, considering the spacing of transverse reinforcement, the longitudinal and transverse reinforcement ratio, and the strength of the material of transverse reinforcement. The developed strength model can be applied not only for square, but also for columns of round and rectangular sections.

Keywords: confinement, stirrups, GFRP, spacing, reinforcement ratio.

ПРОЧНОСТНАЯ МОДЕЛЬ ДЛЯ РАСЧЕТА ЦЕНТРАЛЬНО СЖАТЫХ БЕТОННЫХ ЭЛЕМЕНТОВ С КОМПОЗИТНОЙ АРМАТУРОЙ С УЧЕТОМ ШАГА ПОПЕРЕЧНЫХ СТЕРЖНЕЙ

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Аннотация. В статье обсуждается актуальность разработки методики для элементов с эффективно обжатым ядром и армированием стеклокомпозитной арматурой. Впервые в Российской Федерации предложен метод расчета центрально сжатых бетонных колонн с неметаллической стеклокомпозитной арматурой (без учета ее работы на сжатие). Модель прочности была разработана на основе хорошо известной теоретической модели обжатого бетона. В статье рассматривается эффект усиления бетонного ядра колонн, который образуется за счет более частого размещения как поперечной, так и продольной арматуры. Показана зависимость несущей способности бетонных колонн от прочности материала поперечной арматуры. Было доказано, что с уменьшением расстояния между продольной арматурой площадь эффективно обжатого ядра бетона внутри арматурного каркаса увеличивается. Было показано, что из-за низкого модуля упругости при сжатии арматуры из стеклопластика напряжение в ней будет сопоставимо с напряжением в бетоне. Следовательно, прочностью на сжатие стеклокомпозитной арматуры можно пренебречь в случае определения несущей способности центрально сжатых бетонных элементов. В результате была предложена модель прочности для расчета сжатых бетонных элементов с армированием композитной арматурой с учетом расстояния между поперечной арматурой, процента армирования продольной и поперечной арматуры, а также материала поперечной арматуры. Разработанная прочностная модель может быть применена не только для квадратных, но и для колонн круглого и прямоугольного сечения.

Ключевые слова: обжатие, хомуты, стеклокомпозит, шаг, процент армирования.

INTRODUCTION

In recent years, studies of confined concrete are becoming more and more widespread [1,2]. It was noted that concrete works most efficiently under three-dimensional stress, which led to the appearance of structures with both various types of confinement reinforcement and an increased spacing of transverse reinforcement [3]. The advantages of increasing the bearing capacity of columns by creating a three-dimensional stress state with the use of transverse reinforcement are obvious and have great potential [4]. There is also a whole spectrum of compressive members suffering from reinforcement corrosion, where the use of steel reinforcement leads to early failure of the structure due to corrosion and shortens the life cycle. In such harsh conditions (for example, in tanks, silos, bunkers, reservoirs) columns with non-metallic FRP reinforcement can be successfully used, increasing the life cycle of structures, their overhaul intervals and reducing capital repair costs.

However, a strength model that would consider the effect of an increased spacing of transverse reinforcement on the strength of centrally confined elements has not been introduced in the Russian Federation so far. There is still no methodology for calculating centrally confined columns reinforced with longitudinal and transverse FRP reinforcement in our country. In the modern design standards of the Russian Federation [7], the design resistance of FRP reinforcement for compression is taken equal to 0. In the design standards of different countries, FRP reinforcement as compression reinforcement is also not considered [8, 9]. Canadian researchers [10] proposed the following formula (1) for calculating centrally confined circular elements considering confined GFRP reinforcement:

$$P_0 = 0.85 f_c^{'}(A_g - A_F) + \alpha_g f_{fu} A_F, \qquad (1)$$

where 0.85 – coefficient defined as the ratio between the strength of concrete in a structure and the cylindrical strength of concrete;
f'_c – cylinder strength of concrete;
A_g – column cross-section area;
A_F – cross-sectional area of confined FRP reinforcement;
α_g – a new coefficient that considers the compressive strength of FRP reinforcement depending on its tensile strength.
f_{fu} – tensile strength of FRP reinforcement.

From formula (1), it can be established that with the strength of FRP reinforcement of 1000 Mpa and a coefficient of α_g = 0.3, the strength of composite reinforcement considered in the design will be 300 Mpa.

It is known that the design compressive strength of reinforcement is determined to a greater extent by the ultimate compressibility of concrete $\varepsilon_{bu}=2\cdot10^{-3}$. The ultimate compressibility of concrete depends on the strength of the concrete, its grade, composition, and the duration of the load application. With an increase in the grade of concrete, the ultimate deformations decrease, however, with an increase in the duration of the load application, they increase.

Since, due to bond, the reinforcement deforms with the concrete $\varepsilon_{sc}= \varepsilon_{bu}$, the limiting stresses in it are determined by the formula of Hooke's law:

$$\sigma_{sc} = \varepsilon_{sc} E_s \qquad (2)$$

where E_s – modulus of elasticity of the reinforcement. Therefore, the design resistance of the steel reinforcement will be equal to R_{sc}=*400 МПа*, which is accepted in modern design standards for traditional reinforced concrete structures.

Using Hooke's law (2) it can be obtained that the limiting stresses in the FRP reinforcement at the compressive modulus of elasticity 30 000 MPa and standard strain values for concrete 0,2% achieved only around 50-60 *МПа*, which can be comparable to the compressive stresses of concrete. Under such stresses, considering the ratio of reinforcement in confined structures (usually no more than 2-3%), the share of FRP reinforcement in the bearing capacity of the column will be very small, comparable to the computational error, which greatly limits the prospects for its use in confined elements.

Thus, the stresses in confined FRP reinforcement, due to the low modulus of elasticity, cannot reach such high values proposed by Canadian researchers; therefore, this approach is practically inapplicable. In addition, this strength model ignores the influence of the transverse reinforcement parameters (diameter, spacing, strength) on the strength of a confined element, which, as studies show [11, 12], is incorrect. The results of studies carried out with the usual, not frequent spacing of the stirrup [13], show no increase in the strength of the samples. Therefore, it is necessary to develop a methodology that considers the step of the transverse reinforcement, the ratio of longitudinal and transverse reinforcement, and the strength of the stirrups.

METHODS

During the development of the strength model, general scientific research methods were used (analysis and synthesis, methods of generalization, induction, and deduction). To develop the calculation methodology, the well-known model of confined concrete proposed by Mander et al in 1988 [14] was taken and adapted for columns with FRP reinforcement.

A comparative stress-strain diagram is shown in Figure 1. This figure shows two curves. One curve is for unconfined concrete (bottom), the other is for confined (top). The upper curve has 2 branches - ascending and descending. The ascending branch has a variable slope and starts at a point with a value E_b. Further, the slope of the curve decreases until it reaches the value of the maximum limited strength R_{cb}, ε_{cb}. Thereafter, a descending branch begins with a slight negative slope, reflecting plastic behavior. The end of the curve has a point with maximum deformation ε_{bu}, in which the destruction of the first stirrup occurs. The bottom curve reflects the behavior of unconfined concrete. It has the same ascending branch as the confined concrete curve, with a peak value f'_c (R_b), ε_{c0}. Then, the descending branch follows to the value $1,5\ldots2\varepsilon_{c0}$. Further, the dependence is a straight line until zero strength is reached when cracks appear (ε_{sp}). In our case, it is extremely important to find the dependence for determining the stress-strain state of a limited concrete core inside the stirrups.

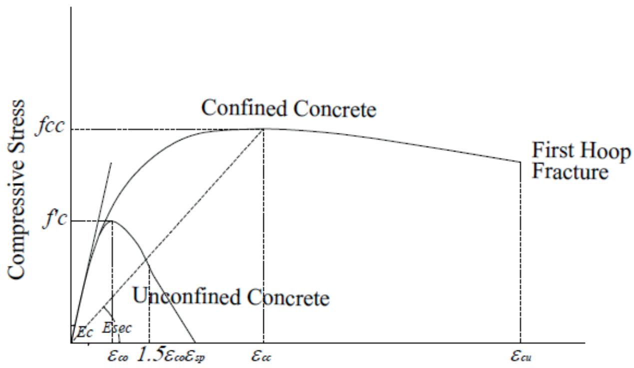

Figure 1. Axial Stress-Strain Model proposed by Mander et al. (1988) for monotonic loading

A distinctive feature of the methods for calculating structures with a triaxial stress state of the inner core is considering the increased strength of concrete in the longitudinal direction.

When comparing the efficiency of structural elements with different methods of reinforcement, the factors of the efficiency of concrete work is η and the efficiency of the structural element as a whole - *m*.

$$\eta = \sigma_b / R_b \quad (3)$$

$$m = N / R_s A_s + R_b A_b \quad (4)$$

where σ_b – longitudinal stresses in concrete at the moment of failure;
N – bearing capacity of the element.

For elements reinforced only with longitudinal reinforcement and the usual standard spacing of transverse reinforcement $\eta = m = 1$, previously conducted experiments [15] found that the efficiency of confined concrete largely depends on the method of reinforcement (Table 1).

Table 1. The dependence of the efficiency factors of concrete μ and the structural element m on the reinforcement method

Type of element, reinforcement	μ	m
Steel tube confined concrete (STCC)	1,5…2,0	3,0…4,2
With reinforcement		
Spiral	1,2…1,4	1,4…1,8
Mesh	1,3…1,5	1,4…1,8
List	1,2…1,4	1,4…1,7
Angle (L-shape)	1,1…1,2	1,2…1,5

The efficiency of a confined concrete core operating in confined conditions can vary significantly depending on the type of concrete. So, for example, for steel tube confined concrete (STCC) concrete η varies from 3.0 for lightweight aggregate concrete to 4.2 for conventional concrete; for elements with mesh and spiral reinforcement - from 1.4 to 1.8, respectively; with corner reinforcement - from 1.2 to 1.5.

Columns of circular cross-section are rarely used in structures for industrial and civil construction therefore, for further research, we will use columns of square and rectangular cross-section.

For the calculation of traditional tube confined concrete structures with a reinforcing cage inside the tube, previously it was proposed to distinguish three characteristic zones in its cross section, which are in different stress-strain state [3].

In our case, due to the frequent setting of transverse reinforcement, only 2 characteristic zones can be considered in the cross section of the element – *A1* inside the reinforcement cage and *A2* from the outer edge of the column to the inner edge of the reinforcing cage (Figure 2). Inside of the A*1* zone concrete works under conditions of a biaxial stress state, while in the zone *A2* the stressed state of concrete can be characterized as uniaxial compression.

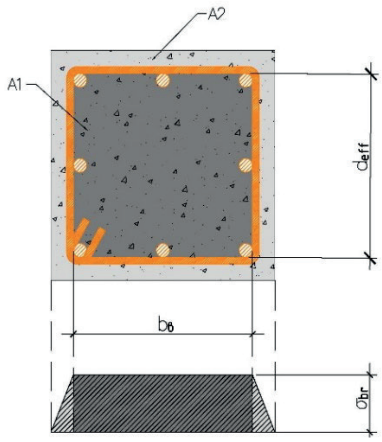

Figure 2. Plot of lateral pressure over the core

For the rest, the calculation is proposed to be carried out by analogy with square-section concrete not taking the steel shell into account.

RESULTS

The strength of the concrete core of a composite reinforcing cage element operating under volumetric compression is determined as the average value of the strength of the peripheral and central zones:

$$R_{b3,m} = \frac{R_{b3,f} A_{s,z} + R_b (A - A_{s,z})}{A}, \quad (5)$$

where $R_{b3,f}$ – the compressive strength of the central zone of a concrete core having indirect reinforcement in the form of frequently installed transverse FRP reinforcement. $A_{s,z}$ – area of the zone inside the stirrups.

Strength of concrete reinforced core $R_{b3,f}$ is determined by the formula (6) with replacement of R_b by $R_{b,s}$ and $\overline{\sigma}$ by $\overline{\sigma}_s$ (or $\overline{\sigma}_f$). In our case $\overline{\sigma}_s$ - the relative value of the lateral pressure in the limiting state from the side of the reinforcing cage on the concrete core and $R_{b,s}$ – compressive strength of concrete with indirect reinforcement with stirrups.

$R_{b,3f}$ is calculated by the formula:

$$R_{b,3f} = R_b \left[1 + \left(0.5\overline{\sigma_{fc}} + \frac{\overline{\sigma_{fc}} - 2}{4} + \sqrt{\left(\frac{\overline{\sigma_{fc}} - 2}{4} \right)^2 + \frac{\sigma_{fc}}{b_1}} \right) \right] \quad (6)$$

where $\overline{\sigma}_{sc}$ - relative radial stresses inside the reinforcement cage.

The value of $\overline{\sigma}_s$ is calculated by the following formula [6,13]:

$$\overline{\sigma} = 0{,}48 e^{-1{,}5b1} \rho^{0{,}8} \quad (7)$$

In the formula (7) the structural element ρ is replaced by ρ_f, defined as follows:

$$\rho_f = \frac{\sigma_{y,p} A_p}{R_{b,s} A} \quad (8)$$

The value of $\overline{\sigma}_{fc}$ is calculated by the following formula:

$$\overline{\sigma_{fc}} = \frac{\mu_{fc} \sigma_{y,sc}}{R_b}, \quad (9)$$

where $\sigma_{y,sc}$ - spiral yield strength;

μ_{fc} - transverse reinforcement ratio with stirrups.

The transverse reinforcement ratio with stirrups depends on the area of the stirrup A_{fc}, the area limited by the diameter of the spiral d_{eff} and spacing of stirrups s. μ_{fc} is calculated according to the formula:

$$\mu_{fc} = \frac{2A_{fc}}{d_{eff} s} \quad (10)$$

Formulas (9) and (10) reflect the previously made assumption that the smaller the spacing of transverse reinforcement, the higher the relative radial stresses inside the reinforcement cage and, therefore, the higher the strength of the concrete core.

In our case, in formula (9) instead of the yield point of steel $\sigma_{y,sc}$ it is necessary to use the tensile strength of the material from which the stirrups are made, namely GFRP reinforcement.

In current construction practice, bent steel rods are already shipped to site prefabricated or bent on site. Unlike the FRP reinforcement, steel has an elastoplastic nature and, therefore, can be easily fixed to “cold”. Current design codes specify bending radius for steel reinforcement from 2,5d_s for mild steel, which corresponds to a maximum deformation of 20%. In the case of FRP reinforcement, there may be problems with potential buckling of the fibers on the inner (compressed) side. Besides that, the typical ultimate strain of composites ranges from 1% to 2.5%, therefore strains in the fibers must be controlled to avoid early failure of the entire rebar. As a result, the cold bending of composite rebar requires larger radius than steel rebar.

It is believed that the bending strength of FRP reinforcement bars is lower than the strength of a straight bar [16, 17]. The key factor is the parallelism of the fibers. So, for fibers, the decrease in strength begins for angles from 5º. For laminates with an angle of inclination of the fibers from the b-axis 30º strength is only 10% of the initial (Figure 3).

The compromise between mechanical properties, ease of manufacture and use of composite stirrups is a bend diameter 7 times larger than the stirrup diameter [18]. In addition, some researchers [19] obtained graphs of the dependence of the strength of FRP reinforcement rods on the angle of inclination of the fibers (Figure 4).

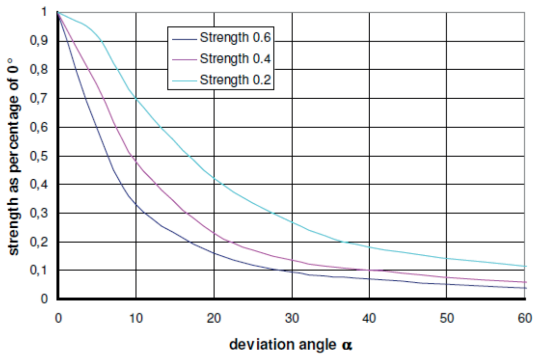

Figure 3. The influence of deviation angle of fibers to strength of UD laminates for different fiber contents [19]

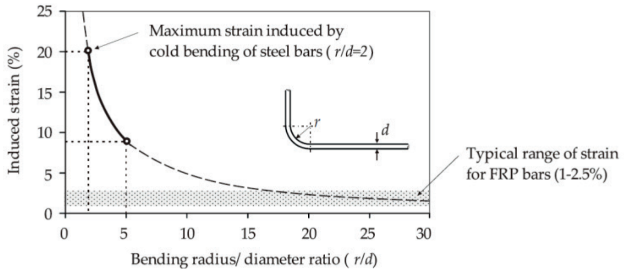

Figure 4. Induced strain values in cold bent bars [18]

Considering the above, we can conclude that for composite stirrups the strength of the rebars on the bends critically depends on the angle of inclination of the fibers. This fact is reflected in the design standard of the Russian Federation for structures reinforced with FRP reinforcement [7]. So, according to clause 5.2.10 [7], the design value R_{fw} of the resistance of FRP reinforcement to tensile at the bend radius of stirrups made of rebars with a diameter d_{fw}, equal to at least $6d_{fw}$ should be defined by the formula:

$$R_{fw} = 0.004E_f \le 0{,}5R_f \tag{11}$$

Thus, at the most common value of the tensile elastic modulus of GFRP reinforcement *50 000 МПа* it is necessary to substitute values no more than R_{fw}=*200 MPa* in the formula (11) instead of $\sigma_{y,sc}$.

It is also important to determine the area of the effectively confined core of the section. According to [4], the area of an effectively limited concrete core A_e is less than the area of the core inside the center lines of the stirrups, excluding the area of the longitudinal reinforcement A_{fc}. The confined concrete zone is assumed to be the area within the center lines of the stirrup's perimeter (Figure 5).

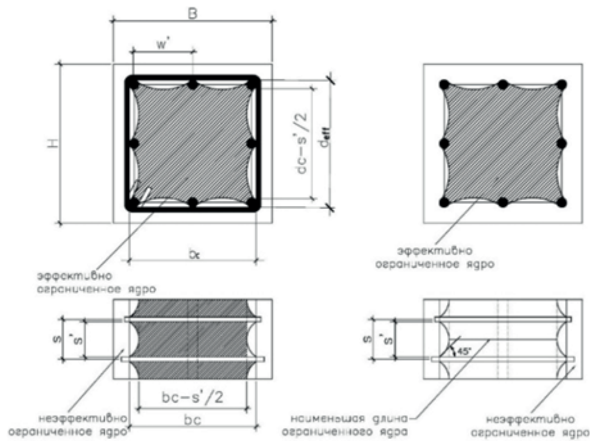

Figure 5. Zone of confined concrete for square columns

The total ineffective area of the confined core at the level of the stirrups, taken that their number is n:

$$A_i = \sum_{i=1}^{n} \frac{\left(w_i^{'}\right)^2}{6} \tag{12}$$

Considering that the boundary of effectively confined concrete between two adjacent stirrups has the shape of a square parabola with an initial angle of inclination 45º, the ratio of the area of effectively confined concrete to the area of the core will be as follows:

$$\lambda = \frac{\left(A_{et} - \sum_{i=1}^{n} \frac{\left(w_i^{'}\right)^2}{6} \right)}{A_{et}} \quad (13)$$

As can be seen from Fig. 5, the contour of an effectively limited core is not square or rectangular, as it was assumed earlier [20]. From the equation of the area of a square parabola, the ordinate of the parabola depends on the abscissa (the spacing of the longitudinal reinforcement). That is, with a decrease in the distance between the bars of the longitudinal reinforcement, the ordinate of the parabola also decreases. At the same time, the area of unconfined concrete is reduced, and the area of efficiently confined concrete is increased. Based on the above, we can conclude that saturation with longitudinal reinforcement, i.e., an increase in its spacing, directly affects the area of an effectively confined core enclosed inside the stirrups and, as a result, affects an increase in the bearing capacity of the centrally compressed element.

Thus, the area of confined concrete in the section is determined in the middle between two adjacent stirrups:

$$A_{emt} = \left(b_c - \frac{s'}{2} \right)\left(d_c - \frac{s'}{2} \right) = b_c d_c \left(1 - \frac{s'}{2b_c} \right)\left(1 - \frac{s'}{2d_c} \right) \quad (14)$$

Accordingly, the effective area in the middle section is calculated as follows:

$$A_e = \lambda b_c d_c \left(1 - \frac{s'}{2b_c} \right)\left(1 - \frac{s'}{2d_c} \right) = \\ = \frac{1}{b_c d_c}\left(b_c d_c - \sum_{i=1}^{n} \frac{\left(w_i^{'}\right)^2}{6} \right) b_c d_c \left(1 - \frac{s'}{2b_c} \right)\left(1 - \frac{s'}{2d_c} \right) \quad (15)$$

$$A_e = \left(b_c d_c - \sum_{i=1}^{n} \frac{\left(w_i^{'}\right)^2}{6} \right)\left(1 - \frac{s'}{2b_c} \right)\left(1 - \frac{s'}{2d_c} \right) \quad (16)$$

It can be seen from the formula above that the smaller the spacing of the transverse reinforcement, the larger the area of effectively confined concrete in the core of the section will be and, therefore, the higher the bearing capacity of the centrally compressed element will be. This proves the previously stated assumption that with a decrease in the spacing of the transverse reinforcement, the bearing capacity of a confined concrete element increases with FRP reinforcement. In addition, the spacing of the longitudinal reinforcement also affects the area of the effectively compressed core of the section. The more often the longitudinal reinforcement is installed in the section, the smaller the variable w' will be, the larger the area of the effectively compressed core and, therefore, the greater the bearing capacity of the element will be.

DISCUSSION

As a result, the bearing capacity of a compressed element with FRP reinforcement can be represented as following:

$$N_{ult,3f} = R_{b3,f} A_e + R_b (A - A_e) \quad (17)$$

This formula takes into account the increased strength of an effectively confined core $R_{b3,f}$ whereas the compressive strength of the FRP reinforcement is not taken into account.

Generally, the method takes into account the spacing of the transverse reinforcement, the saturation of the longitudinal and transverse reinforcement, the strength of the material of the transverse reinforcement (stirrups).

It can be noted that the developed model can be adapted for round and rectangular columns as well. Only in the case of columns with a circular cross-section, the efficiency of using FRP reinforcement should be higher due to the greater strength of the transverse reinforcement material without bends.For comparison with the experimental data, a comparison was made with the research results obtained in [11,12,21]. The comparison results (table 2 and figure 6) shows good correlation between theoretical and experimental values.

<u>Table 2</u>. Experimental results and model prediction for GFRP reinforced compressed members

№	Specimen	b(d), mm	w', mm	s, mm	R_b, MPa	$R_{b,3f}$, MPa	N^{exp}	N^{theor}	$\frac{N^{exp}}{N^{theor}}$
1	**2**		**3**	**4**	**5**	**6**	**7**	**8**	**9**
Lapshinov, Madatyan [12]	CB 1-1	200	75	500	14,9	15,65	615,1	646	0,95
	CB 2-1	200	75	500	14,9	15,22	504,5	624,9	0,81
	CB 1-2	200	75	250	14,9	16,36	600,1	640	0,94
	CB 2-2	200	75	250	14,9	15,53	532,2	626,8	0,85
	CB 1-3	200	75	167	15,9	18,05	738,3	689,6	1,07
	CB 1-4	200	75	100	15,2	18,59	746,1	688,6	1,08
	CB 2-4	200	75	100	15,9	16,67	619,8	680,1	0,91
	CB 1-1	200	75	500	14,9	15,63	568,4	650,4	0,87
	CG 2-1	200	75	500	14,9	15,22	561,1	628	0,89
	CG 1-2	200	75	250	14,9	16,35	583,3	643	0,91
	CG 2-2	200	75	250	14,9	15,52	491,2	629,8	0,78
	CG 1-3	200	75	167	14,9	17,05	586,1	641,4	0,91
	CG 1-4	200	75	100	15,2	18,64	798,4	678,6	1,18
	CG 2-4	200	75	100	15,2	16,74	726,2	646,4	1,12
Lapshinov, Tamrazyan [11]	CG 2-3	200	74	167	19,73	21,9	849,6	818,6	1,04
	CG 2-4-6	200	74	100	19,73	23,18	901,3	839,1	1,07
	CG 2-4-3	200	74	100	19,73	23,18	901,3	839,1	1,07
	CG 2-4-4	200	148	100	19,73	23,18	849,6	779,1	1,09
	CC 1-4	200	74	100	19,73	23,79	1042,3	848	1,23
	CS 1-4	200	74	100	19,73	23,79	1091	1210	0,9
	CS 1-5	200	74	50	19,73	27,14	1140	1270,7	0,9
	CS 1-5	200	74	50	19,73	27,14	1042,2	908,7	1,15
	CS 2-5	200	74	50	19,73	26,08	993,6	891,5	1,11
	CS 2-5-4	200	74	50	19,73	26,08	901,3	891,5	1,01
	CS 2-5-6	200	75	50	19,73	26,08	936,5	890,4	1,05
Lapshinov, et al [21]	C.4Ø10-50-1	150	100	50	22,4	35,95	670,5	541,4	1,24
	C.4Ø10-50-2	150	100	50	27,8	41,97	652,3	664,6	0,98
	C.4Ø10-100-1	150	100	100	20,1	27,68	501,5	471,2	1,06
	C.4Ø10-100-2	150	100	100	20,1	27,68	538,9	471,2	1,14
	C.8Ø10-50-1	150	50	50	26,2	40,20	721,6	666,8	1,08

In the table 2: CB – columns with BFRP rebars, CG – columns with GFRP rebars, CC – columns with CFRP rebars, CS – columns with steel rebars, C.4Ø – columns with GFRP rebars.

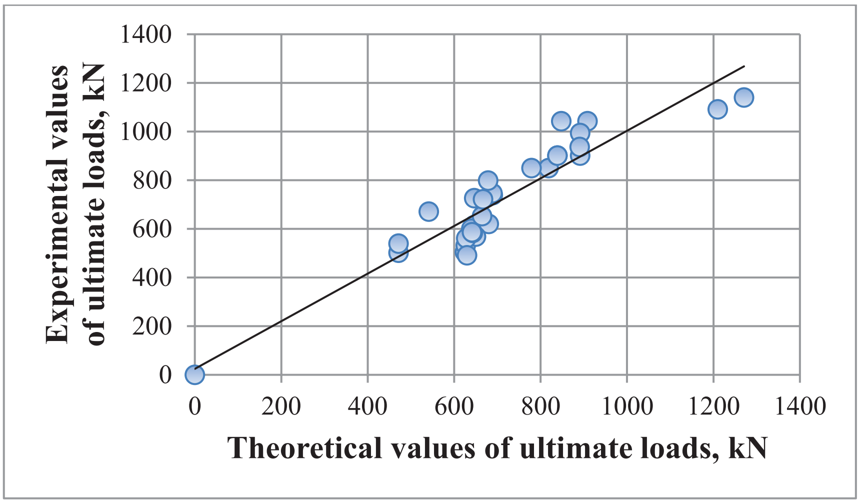

Figure 6. Comparison of calculated and experimental strength data of the author's tests [11,12,21]

CONCLUSION

1. Based on the above-described model of confined concrete, a model for calculating the strength of a centrally compressed concrete element with FRP reinforcement has been adapted and proposed for the first time in RF.
2. The proposed method considers the effect of the frequent spacing of the transverse reinforcement and saturation of longitudinal reinforcement on the strength of centrally confined concrete elements with FRP reinforcement.
3. The influence of the strength of the transverse reinforcement material on the bearing capacity of the centrally compressed concrete element has been justified.
4. An increase in the bearing capacity of confined elements with a decrease in the spacing of transverse reinforcement and saturation of the amount of longitudinal reinforcement is theoretically justified.

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