International Journal for Computational Civil and Structural Engineering, 20(2) 177–191(2024)

DOI:10.22337/2587-9618-2024-20-2-177-191

THE COEFFICIENT OF LONGITUDINAL BENDING FOR COMPOSITE STEEL-CONCRETE RODS UNDER SHORT-TERM LOADS

Denis V. Konin

Research Institute of Building Constructions (TSNIISK) Named After V. A. Koucherenko JSC "Research center of Construction", Moscow, RUSSIA

Abstract: The norms (codes) for calculations of compressed composite steel and concrete columns, as well as experimental and theoretical studies by other authors, are considered. An experimental study of compressed columns with small eccentricity at a range of flexibility l_0/h from 6 to 20 was performed. Finite-elements modeling of various variants of structures has been performed: with concrete class from B30 (C25/30) to B100 (C90/105), with I-beam core, with square form core. The experimental and FEM data obtained are compared with theoretical curves of the longitudinal bending coefficient. It was found that the coefficient curves in SP 266.1325800 and Eurocode 4 are not in the correct shape and do not reflect actual work of composite compressed columns. The use of SP 266.1325800 curve, based on the curve for reinforced concrete structures, as well as Eurocode 4 curves, which are accepted as for steel sections by Eurocode 3, does not provide load-bearing capacity reserve and does not ensure reliability. Both codes ignore different work of low- and high-strength concretes. As a result of the research, it was found that class of concrete affects the stability of columns with flexibility l_0/h from 6 to 10, while there is no influence on the shape of the cross-section of the core (type of rigid reinforcement). For flexibility l_0/h from 10 to 20, a uniform coefficient curve can be adopted for all concretes. Curves of the coefficient φ have been developed for calculating compressed elements at small eccentricities and short-term action of loads; a formula for calculating the bearing capacity of compressed racks has been proposed.

Keywords: concrete, high-strength concrete, steel, composite structure, rigid reinforcement, eccentrical compression, stability, buckling, coefficient of longitudinal bending

КОЭФФИЦИЕНТ ПРОДОЛЬНОГО ИЗГИБА СТАЛЕЖЕЛЕЗОБЕТОННЫХ СТОЕК ПРИ КРАТКОВРЕМЕННОМ ДЕЙСТВИИ НАГРУЗОК

Д.В. Конин

ЦНИИСК им. В. А. Кучеренко АО "НИЦ "Строительство", г. Москва, РОССИЯ

Аннотация: Рассмотрены нормы по расчету сжатых сталежелезобетонных стоек, а также экспериментально-теоретические исследования других авторов. Выполнено экспериментальное исследование работы сжатых стоек с малым эксцентриситетом на диапазоне гибкостей l_0/h от 6 до 20. Выполнено численное моделирование различных вариантов конструкций: с бетонами от В30 (С25/30) до В100 (С90/105), с сердечником в виде двутавра, с сердечником в виде квадрата. Полученные данные экспериментов и расчетов сопоставлены с теоретическими кривыми коэффициента продольного изгиба. Установлено, что кривые коэффициента в СП 266.1325800 и Eurocode 4 имеют не корректную форму и не отражают действительной работы сталежелезобетонных сжатых элементов. Использование кривой СП 266.1325800, основанной на кривой для железобетонных конструкций, а также кривых Eurocode 4, которые приняты как для стальных сечений (по Eurocode 3) – не дает нормального запаса конструкций и не обеспечивает надежность. Оба норматива игнорируют различный характер работы бетонов обычной и высокой прочности. В результате исследований установлено, что класс прочности бетона на сжатие влияет на устойчивость стержней с гибкостью от 6 до 10, при этом влияния формы поперечного сечения сердечника (вида жесткого армирования) практически нет. Для гибкостей от 10 до 20 может быть принята единая для всех бетонов кривая коэффициента. Разработаны кривые коэффициента φ для

расчета сжатых элементов при малых эксцентриситетах и кратковременном действии нагрузок; предложена формула расчета несущей способности сжатых стоек.

Ключевые слова: бетон, высокопрочный бетон, сталь, сталежелезобетонная конструкция, жесткая арматура, внецентренное сжатие, устойчивость, коэффициент продольного изгиба

INTRODUCTION

Consideration of buckling effects for steel reinforced concrete compressed struts is an extremely actual problem. Ignoring the effects of buckling in the design of columns of multi-storey buildings, supports of large-span roofs, bridge piers with large effective lengths can lead to design errors and accidents. Using only idealized design schemes, even taking into account geometric and physical nonlinearities, cannot fully account for the variety of random factors, which in combination can lead to premature failure of a critical building structure. Such random factors include: external eccentricities; internal eccentricities resulting from misalignment of steel and reinforced concrete sections; inaccuracy of manufacturing and assembly of structures; quality and variability of material properties, etc. Some effects can be accounted for by introducing appropriate coefficients in calculations, but the determination of these coefficients must be based on experimental studies of models or full-scale structures. The buckling coefficient simultaneously takes into account the nonlinear behavior of materials in the cross-section. In steel-reinforced concrete structures, this is primarily nonlinear deformation and cracking in concrete. As well as it is possible eccentricities of the load applied to the columns and "internal" eccentricities due to inaccurate assembly.

For long-term loading, the coefficient φ for reinforced concrete elements with rigid reinforcement should be taken from the table of SP 266.1325800.2016 depending on the slenderness ratio of the element and the compressive strength of the concrete. For short-term loading, it should be calculated using the following linear relationship: φ =0,9 at $\frac{l_0}{h} = 10$ ($\lambda = 34.5$) and φ = 0.85 at $\frac{l_0}{h} = 20$ ($\lambda = 69$), where l_0 is the effective length of the element, h is the depth of the section in the plane of determination of the coefficient φ. The values of the coefficient φ are shown in Figure 1 with a red line. The values of the coefficient φ for a rectangular cross-section at central compression taken from [1] are marked with a black dotted line.

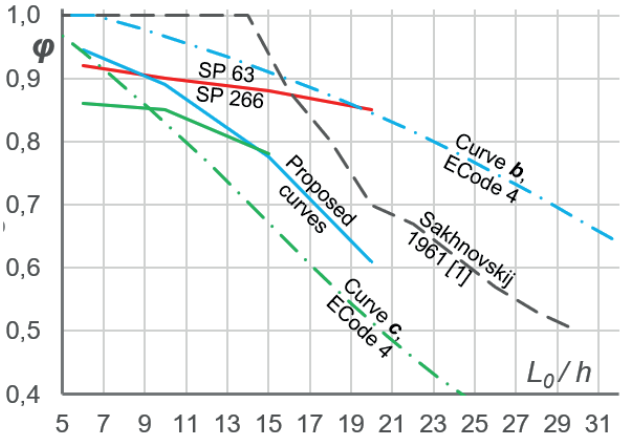


Figure 1. Comparison of coefficients φ

EN 1994-1-1 (2004) "Eurocode 4: Design of composite steel and concrete structures..." (hereinafter referred to as Eurocode 4) requires that the condition 6.44 must be met when designing compressed bars

$$\frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1 \qquad (1),$$

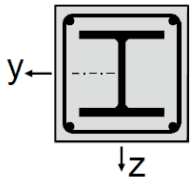
where N_{Ed} is the design load acting on element, and $N_{pl,Rd}$ is the 'plastic' cross-section bearing capacity, defined by formula 2. The nomenclature of parameters in the formula is taken in accordance with SP 266.1325800). Value $N_{pl,Rd}$ is calculated by simply summing up the constituent parts of the cross-section:

$N_{pl,Rd} = R_y A_{st} + 0{,}85 R_{b,c} A + R_{sc} A_{s,tot}$ (2)

where R_y, R_{sc} are the design yield strength for rigid steel profile and for rebars, $R_{b,c}$ is the

design compressive strength of concrete cylinder specimens, A_{st}, A, $A_{s,tot}$ are the cross-section areas of rigid reinforcement, concrete and steel rebars respectively. Also formula 1 utilizes the buckling coefficient χ, which is defined according to the corresponding table 6.5 in Eurocode 4. This table specifies the type of curve of the χ coefficient depending on the type of section and refers to the provisions of clause 6.3.1.2 of EN 1993-1-1: 2005 "Eurocode 3: Design of Steel Structures..." (hereinafter referred to as Eurocode 3). An extract from Table 6.5 of Eurocode 4 and from the corresponding section of Eurocode 3 is given in Table 1. The curve types for the I-beam section, in the corresponding planes (column 5), are also shown there for comparison.

Table 1 – Comparison of Eurocode 3 and Eurocode 4 stability calculation requirements

Type of cross section	Design axes	**Curve type** according to Table 6.5 **Eurocode 4**	Imperfect element	**Curve type** for steel I-beam from steel S355 according to table 6.2 **Eurocode 3**	
				$h/b > 1{,}2$, $t_f \leq 40$ mm	$h/b \leq 1{,}2$, $t_f \leq 100$ mm
1	2	3	4	5	6
	*y-y*	*b*	$L/200$	*a*	*b*
	z-z	*c*	$L/150$	*b*	*c*

The data given in the table shows that curves for hot-rolled column I-beams (column 3) which have approximately the same flange height and width (column 6) are adopted when designing steel reinforced concrete structures of compressed struts in the form of fully concreted I-beams. Note that there is a differentiation in Table 6.2 of Eurocode 3 in the assignment of buckling curves for hot-rolled beam (column 5) and column I-beams, and for welded and hot-rolled sections. Table 6.5 in Eurocode 4 does not take into account the actual design of the I-beam (welded or rolled), its shape (column or beam) or the concrete class. Curve types are taken as the most unfavorable, i.e., in margin. In addition to the one considered in Table 1, there are partially concreted section made of I-beams, for which the curve types are the same as for the one considered. There is also partially concreted section made of two I-beams made in a cross (curve type b), pipe-concrete section with the percentage of reinforcement up to 3% and from 3 to 6% (curve types a and b, respectively), pipe-concrete section with rigid reinforcement in the form of I-beams (curve type b). In Figure 1, the dashed blue and green lines represent curves b and c, respectively. The slenderness ratios are given for the l_0/h ratio used in SP 63.13330 and SP 266.1325800, as for the S355 steel bar.

Steel reinforced concrete structures working in compression have been studied quite well in the USSR and Russia since the second half of the 20th century by A.N. Vasiliev [2], E.L. Zhitnitskaya [3], N.A. Slavinsky, S.K. Mikadze, N.A. Egorov [4], Khandzhi V.V. [8], Travush V.I. with the participation of the author [12] and others. The paper [24] provides justification of the possibility of calculating such structures using a nonlinear deformation model. Among foreign studies in recent years, the researchs of PRC scientists [15, 18, 19], as well as Western scientists [20, 21, 22, 23] have drawn attention. However, they investigate relatively short steel-reinforced concrete bars (l_0/h from 3 to 6), where the stability effects or the influence of deformation of the bar are minimal. Nevertheless, they will be used to analyze the results in the future.

The stability of reinforced concrete bars has been investigated in the studies of E.A. Chistyakov [11], V.M. Bondarenko, V.I. Murashev, as well as

A.D. Beglov and R.S. Sanzharovsky et al [9, 10], V.V. Gichko [7], A.L. Krishan. [17], Teryanik V.V. [16] and others. The studies of E.A. Chistyakov formed the basis of the current norms for the design of reinforced concrete structures SP 63.13330. The stability of reinforced concrete bars taking into account creep phenomena, including under the application of short-term acting loads, is studied in [13, 14]. The performance of modern concretes under load with different aggregates, their deformation diagrams and Poisson's ratio are studied in [25, 26].

2. RESEARCH METHODS

To verify the existing buckling coefficient curves (Figure 1), it is necessary to analyze the available experimental data on the performance of bars of different slenderness with different percentages of reinforcement. Due to the relatively small number of experiments on steel-reinforced concrete structures, test data on reinforced concrete bars of different slenderness are analyzed. This is justified, since the strains of concrete in steel-reinforced concrete bars at the central location of rigid reinforcement elements fully correspond to the strains of reinforced concrete structures. The comparison does not include the results of tests of pipe-concrete structures, since their operation is significantly influenced by the presence of a steel enclosed shell and concrete strains in such structures have certain peculiarities.

In order to supplement the existing test base, a full-scale test of steel-reinforced concrete strut models in the range of slenderness l_0/h from 6 to 20 was carried out. The general cross-sectional view and the view of one of the failed models are shown in Figure 2. The slenderness of the elements was varied by changing the length of the models. The steel of rigid reinforcement was C355 and that of steel rebars was A500C. The concrete had a compressive strength class of B100, which was confirmed by cube specimen tests. The percentage of cross-sectional reinforcement for all models is 10.3%. All models were tested under eccentric compression with an eccentricity of 5 mm along the flanges of the I-beam, which is close in value to random eccentricity (h/30=5.3 mm). During testing, the strains on the surface of steel and concrete, vertical absolute displacements, and the value of compressive load at the beginning and end of the holding time were recorded at each step.

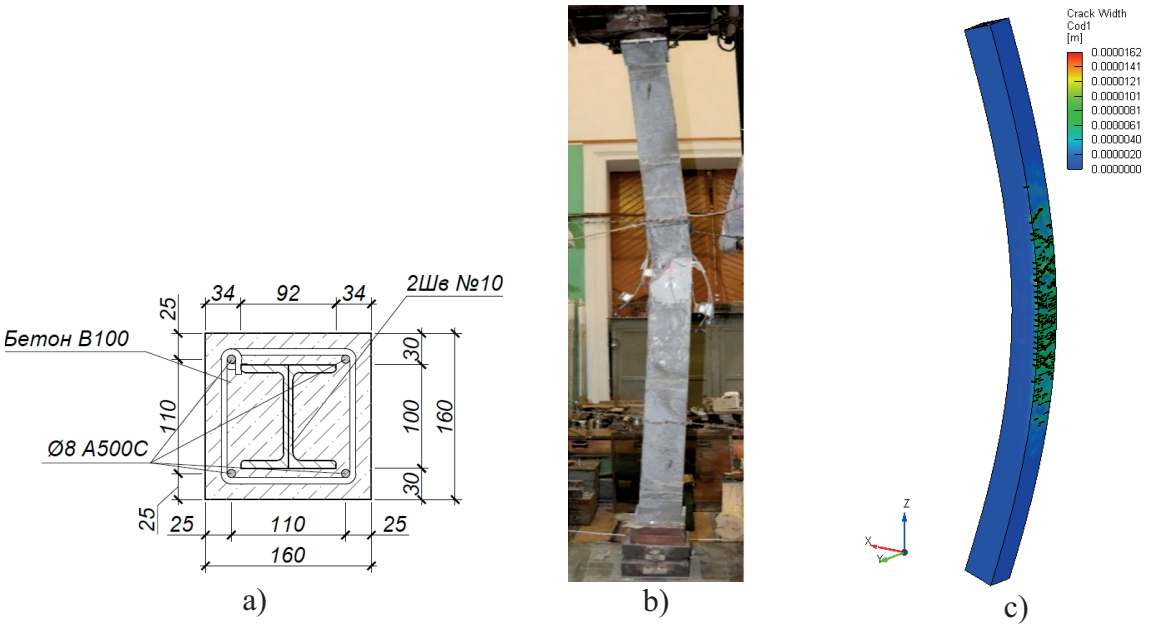


Figure 2. Experimental models: a - cross section, b - fractured model with slenderness ratio of 20, c - ultimate state of the model with slenderness ratio of 20 according to FEA

Similar struts were also studied using finite element modeling in ATENA software, considering the nonlinear performance of the materials: a three-line diagram for concrete and a two-line diagram for steel. Models were created that fully replicate the experimental models in terms of geometry, boundary conditions, eccentricities and materials. In addition, struts with concrete of compressive strength classes B30, B60, and B80 were modeled to extend the results to all material strength ranges. Poisson's ratio was assumed differentially according to the strength class of concrete [26]. The number of steps and mesh size for each model was also selected individually. The minimum number of loading steps was at least 20 and the mesh size was about 20 mm. The Fracture-Plastic Constitutive Material Model (CC3DCementitious2), which is based on a combination of a tensile fracture model (Rankine-Fracturing Model) with a compressive material fracture model (Menétrey-Willam), was used to describe the performance of the concrete. The calculations of all structures were performed considering the formation and opening of cracks in concrete. The limit state of the slender column is shown in Figure 2, c. Also, as part of preliminary finite element modeling, strut models with simulation of contact interaction between steel and concrete were tested. It was found that for struts with small eccentricities, regardless of the slenderness and shape of the core, the presence of a certain contact interaction at the steel-concrete joint practically does not affect the results of calculations. That is, there is no influence of the contact on the value of the ultimate load, the value of the ultimate longitudinal strain of the core. There is influence on the buckling deformation within 3-5%. The difference between forces and deformations for the same type of models with and without contact interaction modeling is less than 0.5%. Therefore, further analysis and modeling were performed for finite element models of struts in which the contact interaction at the steel-concrete joint was not modeled in a special way, and the steel and concrete FEs had common nodes in the design scheme.

3. RESULTS AND DISCUSSION

The main difficulty in analyzing previously conducted studies is that it is not always possible to reliably determine from descriptive materials the design length of elements that should be considered from hinge to hinge of the test bench. In some studies, only the length of the tested structures is given, which introduces a distortion in the results of the analysis. Often tests with small or zero eccentricities were performed in one or two instances, which did not allow to establish reliably the magnitude of the failure load. Therefore, it was possible to analyze 79 tests of reinforced concrete and steel-reinforced concrete struts out of the whole variety of studies based on the works [4, 6, 7, 11, 12, 22, 23], in which the eccentricity value did not exceed the value of $h/30$, as well as the tests described above, conducted by the author. The selected test results fall within the range of slenderness l_0/h from 5 to 32, for concrete on heavy aggregate with compressive strength classes from B20 to B100. The rigid reinforcement elements in almost all experiments are in the form of a single I-beam made of steel similar to C255...C390. In experiments [22, 23] 4 I-beams made of S460 steel were used as a rigid reinforcement. The investigated reinforcement percentage over the whole range of the study is from 0.08 to 46.6%. The cross sections were all rectangular or circular. Thus, a fairly representative array of experimental data was investigated over a wide range of slenderness, percent reinforcement, and compressive strength class of concrete. The longitudinal bending coefficient was calculated from the experimental results using the formula:

$$\varphi = N_{test}/N_{theor.} \quad (3)$$

where N_{test} is the fracture load based on test results, $N_{theor.}$ is the theoretical load-bearing

capacity of the strut, defined as the sum of the compressive load-bearing capacity of concrete, steel, reinforcement according to the actual values of material strengths and their actual dimensions. Figure 3 shows the test results as discrete values, as well as the theoretical values of the coefficient φ according to various sources. Reinforced concrete structures are marked in the figure with round markers, steel-reinforced concrete structures are marked with triangular markers.

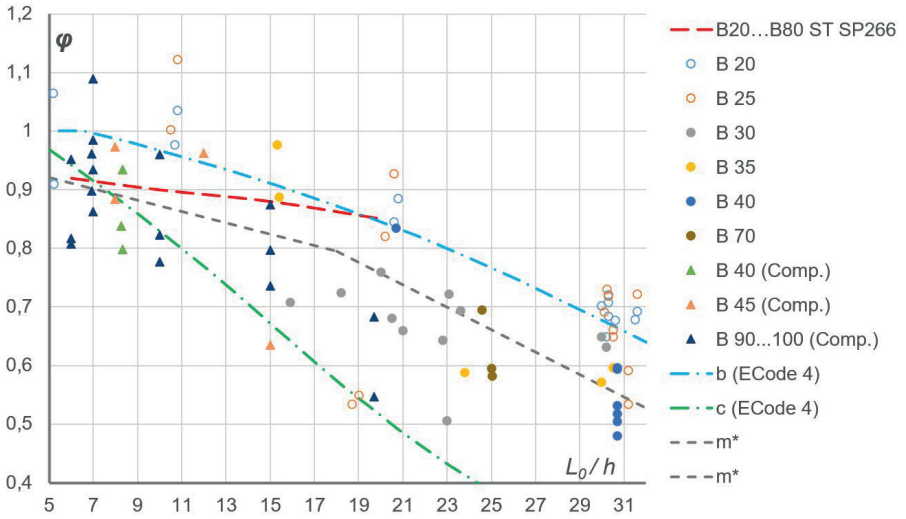


Figure 3. Comparison of theoretical curves of the coefficient φ with experimental data

While considering the data of the experiments, a rather large variation in the values of the coefficient φ attracts attention. At slenderness 6 the variation is from 0.81 to 1.0 (Δφ=0.19), at slenderness 10 - from 0.78 to 0.96 (Δφ=0.18), at slenderness 15 - from 0.64 to 0.98 (Δφ=0.34), at slenderness 20 - from 0.55 to 0.84 (Δφ=0.29), at slenderness 30 - from 0.48 to 0.65 (Δφ=0.17). The minimum squares approximation by a linear function leads to the following system of equations for the coefficient φ (shown in Figure 3 by the gray dashed line):

$$\begin{cases} \varphi = 0{,}97 - \frac{0{,}001 l_0}{h}, for \;\; 5 < l_0/h \leq 18 \\ \varphi = 1{,}14 - \frac{0{,}019 l_0}{h}, for \; 18 < l_0/h \leq 32 \end{cases} \quad (4)$$

Significant variation of test results is related to the complexity of the modeled structures. First of all, it is steel and reinforced concrete structures, in which the influence of internal eccentricities is high. In addition, it is difficult to take correct readings from load cells and displacement sensors in case of sudden brittle failure of the structure at the last stages of testing. These features are typical for all analyzed studies. A significant variation of data within one study is representative if model fabrication and measurements are performed by the same groups of specialists. The line of the characteristic coefficient according to SP 266.1325800 (red dotted line) lies above a significant number of tests and has a smaller slope with increasing slenderness than expessions (4).

Eurocode 4 lines b and c (blue and green dashed lines) also have an unsatisfactory correlation with lines plotted by expressions (4). Line b lies above a significant number of tests, i.e., calculations that use it will not give a margin. Line c lies substantially below almost all tests, which will lead to unjustified overconsumption of materials. At the same time, both of these lines at the value of slenderness of 5 are practically equal to 1, which does not ensure the reliability of structures and does not take into account the peculiarities of nonlinear operation of the combined steel-reinforced concrete section.

Expressions (4) are constructed for all concretes and slenderness ratios. If other smaller samples

by concrete strength classes, reinforcement percentages or other design parameters were to be separated in the investigated sample. The size would be unsatisfactory for approximation (n=3...5) and statistical analysis of the test results. Therefore, the mean function of the coefficient φ presented in expressions (4) over the entire range of concretes, reinforcement percentages and slenderness ratios can only be a guide. For example, the results of tests of steel-reinforced concrete structures with rigid reinforcement and concrete of strength class B90...B100 by different groups of specialists show that mainly at slenderness ratios from 10 to 20 the values of the coefficient φ lie below the approximation lines (4). This indicates the peculiarities of the performance of such structures and the need for a separate analysis. The peculiarities of the performance of structures with different types of cores can be investigated in detail using verified finite element models.

Finite element analysis of models with different slenderness ratios and concretes allowed us to construct curves of the coefficient φ, which was calculated using Formula 3. In this formula, the value of N_{FEM} is the ultimate compressive force based on the results of the finite element modeling that was utilized used instead of the failure load during the tests N_{test}. Extremes of the force-displacemt graphs for strut shortening and buckling were taken as the ultimate state (Figure 4, a). The plots of the coefficient φ for the whole considered range of slenderness ratios and B30, B60, B80 and B100 concretes are shown in Figure 4, b. The curves for all concretes at slenderness greater than 15 follow practically the same trajectory, and at slenderness from 6 to 15 they lie one under the another. The least strong concrete (B30) has the maximum coefficient φ=0.95 at slenderness of 6 (blue line). A core of the stronger concrete (B100) at the same slenderness has a value of φ=0.85 (green line).

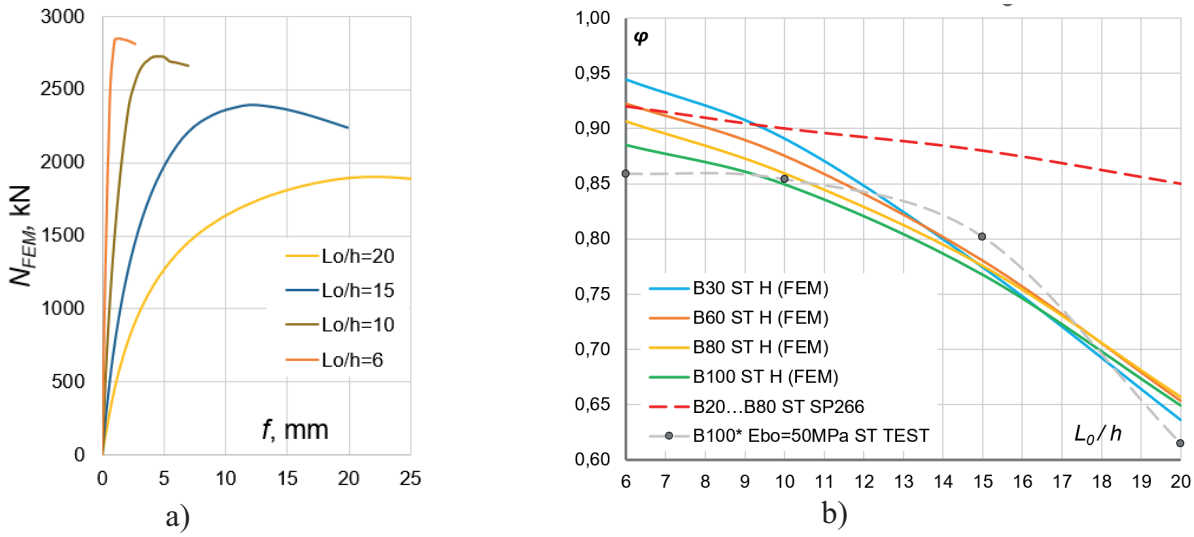


Figure 4. Comparison of finite element analysis results for different concretes: a - force-displacement graphs for identifying the buckling of struts made of B100 concrete for different slenderness ratios; b - diagrams of the coefficient φ for different slenderness ratios and concretes

Compared to the normative line of the coefficient φ (red dashed line, Figure 4), all obtained lines are lower, especially for slenderness ratio of 20 and more. At a slenderness of 20, the difference is more than 30%. Based on the obtained data from the analysis of validated finite element models, the refined graphs of the coefficient φ can be plotted.

In addition, we evaluate the influence of the cross-sectional shape of the rigid reinforcement on the performance of a bar compressed with a small eccentricity. For this purpose, finite element models of bars of different slenderness were created with a core in the form of a solid square section that is equivalent in area to the previously

studied bars with I-beams (Figure 5). The same options for creating and for analysis of models in ATENA were used for simulation as presented above. According to the results of the simulations, we can plot the buckling coefficients and compare them with those previously obtained for the bars with I-beam reinforcement (Figure 6). Solid lines in the graphs show the curves of the coefficient φ for bars with I-beam core, dotted lines indicate the curves of the coefficient φ for bars with square core. Despite the insignificant difference in the coefficient values (about 5-10%), the curves run parallel and there is almost no significant influence of the shape of the core cross-section. Unlike steel bars in compression, where the influence of cross-sectional shape on the type of φ coefficient curve has been established, here the effect is minimal and can be ignored for most common cross-sections with central and symmetrical arrangement of rigid reinforcement elements. This is primarily due to the fact that the value of the coefficient φ in steel-reinforced concrete structures is primarily influenced by the concrete and its deformation diagram. Therefore, the different character of deformation of concretes of different strengths should be accounted by the new curves of the φ coefficient, also taking into account the studies [25, 26].

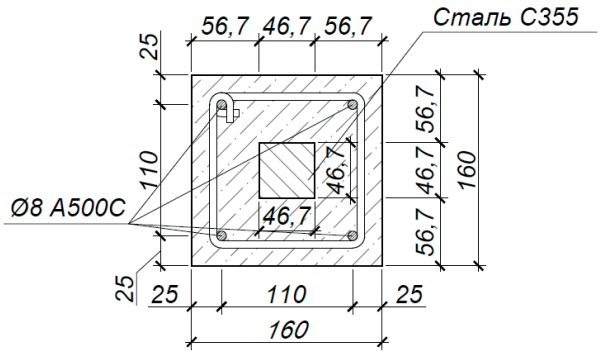


Figure 5. Cross-section of a strut with a square core

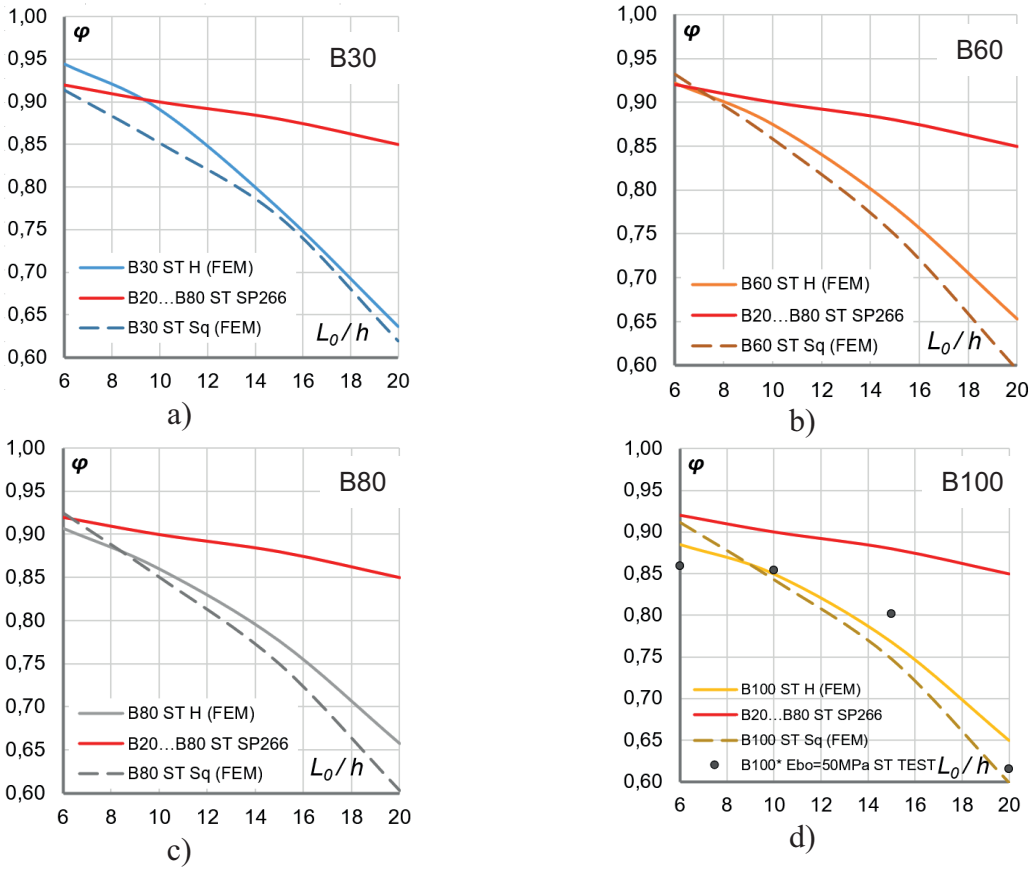
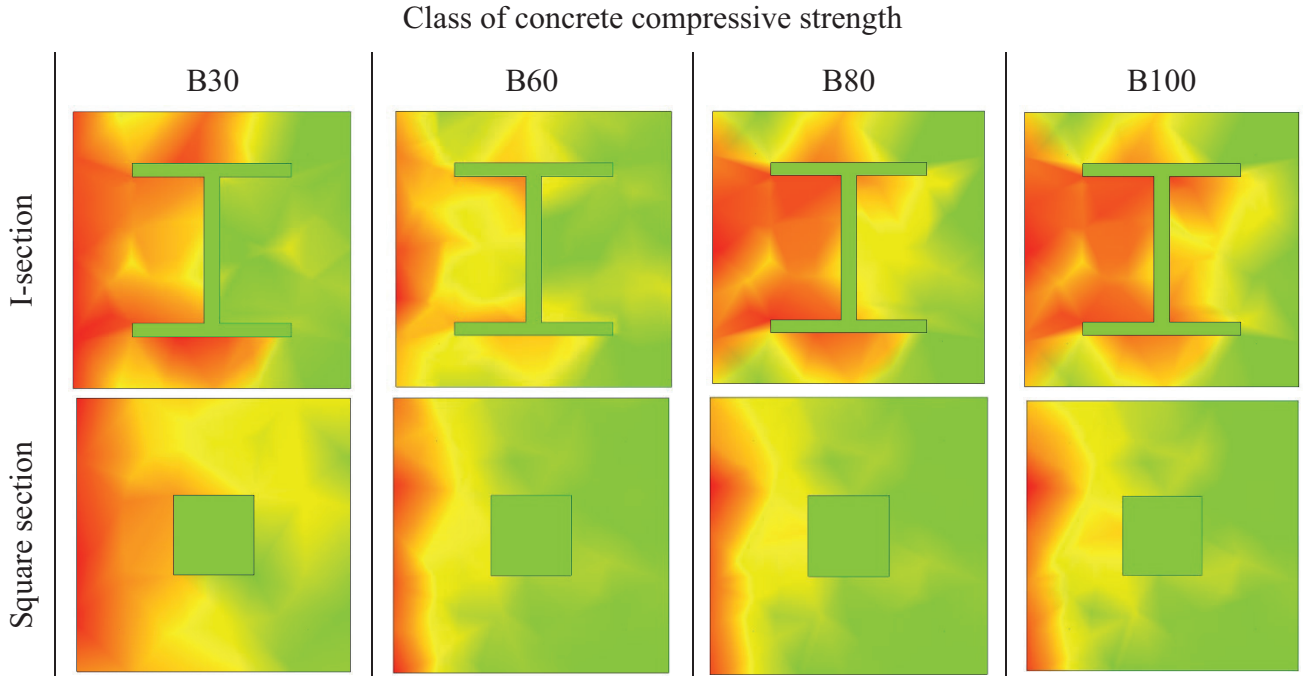


Figure 6. Comparison of calculation results for bars with I-beam and square core with different concretes: a - B30, b - B60, c - B80, d - B100

Additionally, let us analyze the pattern of cracking in the cross-section of struts with different types of rigid reinforcement. Table 2 shows comparative cracking patterns at the last calculation step before the ultimate state is reached. Crack patterns are given for a strut of slenderness 20 for an example with varying concrete performance diagrams from B30 to B100. It can be seen that the struts with B30 concrete exhibit more active material involvement throughout the cross-section. That is, cracks (and thus plastic strains in concrete under compression) propagate more actively from the periphery of the most compressed fiber to the axial areas of the cross-section. Thus, in structures made of concrete B30 the steel core is more actively involved in the work. In the case of high-strength concrete the steel core is used less effectively. In the actual performance of structures, this pattern will also be confirmed due to the fact that ordinary strength concrete (B20...B35) has a smoother deformation diagram and a descending branch after reaching the tensile strength [25]. Unlike high-strength concretes (B80...B100), which have brittle fracture behavior, the use of ordinary strength concretes increases the reliability of the structure, although it has the effect of increasing material consumption.

Table 2. Comparison of cracking patterns for different concretes and cross sections for slender struts $l_0/h = 20$



There is also a pronounced cracking in the space between the flanges of the I-beam in the most compressed part of the cross-section. The analysis of the principal stresses, as well as stresses in separate directions shows that the concrete actively works not only in compression along the main axis of the core, but also in compression in the horizontal plane due to the constraint of lateral strains by the I-beam flanges. A "cage" effect is observed. This causes the I-beam core columns to work more efficiently (see Figure 6) and have a slightly higher value of the φ coefficient and ultimate bearing capacity than the square core columns. The comparison of ultimate bearing capacity of cores with different cores is summarized in Table 3. The maximum bearing capacity difference of 9.8% in favor of I-beam core is recorded in columns with maximum slenderness made of B60...B100 concrete. The fact that the

difference will be maximum for high-strength concrete is obvious since as the strength class of concrete increases, the actual value of Poisson's ratio also increases from 0.2 for B30 to 0.245 for B100 according to [26].

Table 3. Comparison of ultimate bearing capacity of struts with different reinforcement

Concrete class	l_0/h	I-section			Square section			$1-\frac{N_{FEM(3)}}{N_{FEM(6)}}$
		N_{FEM}, kN	N_{theor}, kN	φ	N_{FEM}, kN	N_{theor}, kN	φ	
1	2	3	4	5	6	7	8	9
B100	20	1628	2507	0,649	1502	2507	0,599	-8,4%
	15	1925		0,768	1874		0,748	-2,7%
	10	2130		0,849	2113		0,843	-0,8%
	6	2219		0,885	2285		0,911	2,9%
B80	20	1434	2182	0,657	1315	2182	0,603	-9,1%
	15	1693		0,776	1635		0,749	-3,6%
	10	1876		0,860	1855		0,850	-1,1%
	6	1979		0,907	2017		0,924	1,9%
B60	20	1213	1857	0,653	1105	1857	0,595	-9,8%
	15	1449		0,780	1392		0,749	-4,2%
	10	1626		0,875	1594		0,859	-1,9%
	6	1713		0,922	1732		0,932	1,1%
B30	20	872	1370	0,636	848	1370	0,619	-2,8%
	15	1061		0,775	1048		0,765	-1,2%
	10	1221		0,891	1166		0,851	-4,7%
	6	1294		0,945	1251		0,914	-3,4%

Summarizing the research data presented above, and also considering the peculiarities of steel-reinforced concrete structures in the form of reinforced concrete struts with rigid reinforcement, it is proposed to carry out the bearing capacity calculation for compressed elements with eccentricities not exceeding the random eccentricities by the following formula:

$$\frac{N}{\varphi(R_b A+R_{sc}A_{s,tot}+R_y A_{st})} \leq 1, \qquad (5)$$

where R_b, R_y, R_{sc} are the design strengths of concrete in compression, yield strength for rigid reinforcement and rebars respectively, A_{st}, A, $A_{s,tot}$ are the cross-section areas for rigid reinforcement, concrete and steel rebars respectively. The values of the coefficient φ for continuous action of loads are accepted according to Table 7.1 of SP 266.1325800, and for short-term action of loads are accepted according to Table 4 of this paper. Comparison of the proposed coefficients (blue and green lines) with the requirements of SP 266.1325800 (red dashed line) and Eurocode 4 (blue and green dashed lines) is shown in Figure 1.

The proposed values of the coefficient φ take into account the nature of the performance of concrete of different compressive strengths, as well as the peculiarities of the performance of steel-reinforced concrete compressed elements under load with different types of cores, the operation of the structure under deformed scheme.

Table 4. Coefficient φ for short-term loads

Concrete class	Slenderness ratio of the element			
	$l_0/h =$ 6	10	15	20
	$\lambda =$ 20.8	34.6	52.0	69.3
1	2	3	4	5
B30	0.945	0.891	0.78	0.61
B60	0.920	0.875		
B80	0.890	0.860		
B100	0.860	0.850		

4. CONCLUSIONS

1) Experimental and theoretical studies of the performance of compressed steel-reinforced concrete struts of different slenderness ratio have been performed. Russian and foreign studies of the stability of compressed reinforced concrete and steel reinforced concrete bars have been analyzed.

2) The curves of buckling coefficient contained in SP 266.1325800 and Eurocode 4 do not accurately reflect the peculiarities of steel-reinforced concrete compressed bars with small eccentricities. The coefficient of SP 266.1325800, which is based on the standards for the design of reinforced concrete structures, does not correspond to the theoretical assumptions in its current form, as it does not smoothly decrease with increasing slenderness ratio. It is also not confirmed by the analyzed experimental data. The coefficient in Eurocode 4, which gives a reference to the curves of a similar coefficient for steel structures from Eurocode 3, also cannot be considered sufficiently justified. It actually ignores the performance of reinforced concrete in the cross-section. The concrete at the periphery of the cross-section is included in the work at the very first stages of loading of the structure and significantly changes the performance and hence the coefficient curve of the compressed steel column surrounded by concrete. The concrete significantly improves the stability performance of the compressed strut, which should be taken into account in the calculations.

3) It is found that the compressive strength class of concrete affects the stability of bars with slenderness ratios from 6 to 10, with little or no influence of the shape of the cross-section of the core type of rigid reinforcement. The curves of the φ coefficient for the calculation of compressed elements at small eccentricities and short-term action of loads have been developed. A formula for calculating the load-bearing capacity of compressed struts has been proposed.

4) Based on the analysis performed, the buckling coefficients should be checked and updated when calculating the long-term action of loads, taking into account the shrinkage and creep of materials. Due to the presence of cracking concentrators in the form of steel-concrete joints, studs, etc., the creep measure used to calculate steel-reinforced concrete structures may differ from standard reinforced concrete structures.

REFERENCES

1. **Sahnovskij K.V.** (1961) *Zhelezobetonnye konstrukcii* [Reinforced concrete structures]. Moscow: Gosudarstvennoe Izdatel'stvo literatury po stroitel'stvu, arhitekture i stroitel'nym materialam, 840 p. (in Russian)
2. **Vasiliev, A.P.** (1941) *Zhelezobeton s zhestkoj armaturoj* [Reinforced concrete with rigid reinforcement]. Moscow: State Publishing House of Construction Literature, 123 p. (in Russian)
3. **Zhitnitskaya, E.L.** (1957) *Issledovanie vnecentrenno szhatyh zhelezobetonnyh kolonn s nesushchej armaturoj pri malyh*

ekscentrisitetah [Investigation of out-of-center compressed reinforced concrete columns with load-bearing reinforcement at small eccentricities] (PhD Thesis). Moscow. (In Russian)

4. **Egorov, N.A.** (1975) *Issledovanie vnecentrenno szhatyh zhelezobetonnyh kolonn s nesushchej armaturoj pri malyh ekscentrisitetah* [Investigation of the strength and deformability of reinforced concrete columns with rigid reinforcement made of low-alloy steel] (PhD Thesis). Moscow, 142 p. (In Russian)
5. **Kajumov R.H.** (1972) *Ustojchivost' gibkih zhelezobetonnyh sterzhnej iz vysokoprochnogo betona* [Stability of flexible reinforced concrete rods made of high-strength concrete] (PhD Thesis). Kiev, 201 p. (In Russian)
6. **Bambura A.N.** (1978) *Issledovanie kratkovremennoj ustojchivosti szhatyh zhelezobetonnyh jelementov pri nesharnirnom zakreplenii na koncah* [Research of short-term stability of compressed reinforced concrete elements with non-hinged fastening at the ends] (PhD Thesis). Kiev, 199 p. (In Russian)
7. **Gichko V.V.** (2015) *Rozrahunok stijkosti gnuchkih zalizobetonnih stijok za deformacijnim metodom* [Calculation of the stability of flexible reinforced concrete struts using the deformation method] (PhD Thesis). Kiev, 137 p. (in Ukrainian)
8. **Handzhi V.V.** (1977) *Raschet mnogojetazhnyh zdanij so svjazevym karkasom* [Calculation of multi-storey buildings with a connection frame]. Moscow: Strojizdat, 189 s.
9. **Beglov, A.D., Sanzharovsky, R.S.** (2006) *Teoriya rascheta zhelezobetonnyh konstrukcij na prochnost i ustojchivost. Sovremennye normy i Evrostandarty* [Theory of calculation of reinforced concrete structures for strength and stability. Modern norms and European standards]. St. Petersburg – Moscow. ACB, 221 p. (In Russian)
10. **Sanzharovsky, R.S.** (2015) Zhelezobeton: aktualizirovannyj normativ [Reinforced concrete: updated standard]. *Construction newspaper*. Moscow, no. 21.
11. **Chistjakov E.A.** (1988) Osnovy teorii, metody rascheta i jeksperimental'nye issledovanija nesushhej sposobnosti szhatyh zhelezobetonnyh jelementov pri staticheskom nagruzhenii [Fundamentals of theory, calculation methods and experimental studies of the bearing capacity of compressed reinforced concrete elements under static loading] (doctoral Thesis). Moscow, 498 p. (In Russian)
12. **Travush V.I., Konin, D.V., Rozhkova, L.S., Krylov, A.S.** (2016) Eksperimentalnyye issledovaniya stalezhelezobetonnykh konstruktsiy, rabotayushchikh na vnetsentrennoye szhatiye [Experimental study of composite structures, working for eccentric compression]. *Academia. Architecture and Construction*, no. 3, pp. 127-135.
13. **Arleninov P.D**. (2016) Deformirovanie i ustojchivost' szhatyh i vnecentrenno szhatyh sterzhnevyh zhelezobetonnyh jelementov s uchetom polzuchesti i treshhinoobrazovanija [Deformation and stability of compressed and non-centrally compressed core reinforced concrete elements, taking into account creep and cracking] (PhD Thesis) Moscow, AO «NIC «Stroitel'stvo» (NIIZhB im. A.A. Gvozdeva), Moscow, 143 p. (In Russian)
14. **Krylov S.B., Arleninov P.D.** (2012) Kriticheskaja sila dlja zhelezobetonnyh sterzhnevyh jelementov [Critical force for reinforced concrete core elements] Academia. Architecture and Construction. *Foresight-Russia*, no 2, pp. 136-138.
15. **S.P. Chiew, Y.Q. Cai.** (2018) *Design of high strength steel reinforced concrete columns: a Eurocode 4 approach*, Boca Raton: CRC Press, 105 p.
16. **Terjanik V.V**. (2003) O probleme ustojchivosti vnecentrenno szhatyh betonnyh i zhelezobetonnyh jelementov

srednej i bol'shoj gibkosti [On the problem of stability of non-centrally compressed concrete and reinforced concrete elements of medium and high flexibility] *Vestnik Samarskogo Gosudarstvennogo Tehnicheskogo Universiteta. Fiziko-Matematicheskie Nauki*, Samara, no 19, pp. 167-169.

17. **Krishan A.L., Krishan M.A.** (2014) Vlijanie gibkosti na nesushhuju sposobnost' szhatyh zhelezobetonnyh jelementov [The effect of flexibility on the bearing capacity of compressed reinforced concrete elements] // *Novye idei novogo veka,* Habarovsk, no 3, pp. 308-314.
18. **Lu T., Jin H., Guan K.** (2023) Experimental Research on Axial Compression Performance of High-Performance-Fiber-Reinforced-Cement-Composite-Prefabricated Monolithic Composite Columns. *Buildings*, no.13, pp. 1748-1763.
19. **Liu X., Pan M., Li W., Jing C., Chang W., Zhang H.** (2024) Experimental Study and Bearing Capacity Calculation of Compression-Reinforced Concrete Columns Strengthened with Ultra-High-Performance Concrete. *Applied Sciences*, no 14, pp. 1911-1928.
20. **Ince E.G., Özkal F.M.** (2024) Optimization of Structural Steel Used in Concrete-Encased Steel Composite Columns via Topology Optimization. *Applied Sciences*, no. 14, pp. 1170-1182.
21. **Polskoy P.P, Mailyan D.** etc. (2023) Strength of Compressed Reinforced Concrete Elements Reinforced with CFRP at Different Load Application Eccentricity. *Polymers*, no. 15, p. 26.
22. **Deng F., Chen T., Xiao C.** (2014) *Performance and capacity of isolated steel reinforced concrete columns and design approaches*. Beijing: China Academy of Building Research (CABR) Technology Co., Ltd.
23. **Bogdan T., Gerardy J.-C., Davies D.W., Popa N.** (2017) *Performance and capacity of composite "mega columns" with encased hot rolled steel sections.* Copenhagen: Eurosteel, p. 11.
24. **Muhamediev T.A., Starichkova O.I.** (2006) Raschet prochnosti stalezhelezobetonnyh kolonn s ispol'zovaniem deformacionnoj modeli [Calculation of the strength of steel-reinforced concrete columns using a deformation model] *Beton i zhelezobeton*, no 4 (541), pp. 18-22.
25. **Bezgodov I., Kaprielov S., Sheynfeld A**. (2022) Relationship between strength and deformation characteristics of high-strength self-compacting concrete. *International Journal for Computational Civil and Structural Engineering*, no 18(2), pp. 175–183.
26. **Kaprielov S., Sheinfeld, A., Selyutin N.** (2022) Control of heavy concrete characteristics affecting structural stiffness. *International Journal for Computational Civil and Structural Engineering*, no 18(1), pp. 24–39.

СПИСОК ЛИТЕРАТУРЫ

1. **Сахновский К.В.** Железобетонные конструкции – М.: Государственное Издательство литературы по строительству, архитектуре и строительным материалам, 1961. – 840 с.
2. **Васильев А.П.** Железобетон с жесткой арматурой – М.: Государственное издательство строительной литературы, 1941. – 123 с.
3. **Житницкая Э.Л.** Исследование внецентренно сжатых железобетонных колонн с несущей арматурой при малых эксцентриситетах: диссертация … кандидата технических наук – М., 1957.
4. **Егоров Н.А.** Исследование прочности и деформативности железобетонных колонн с жесткой арматурой из низколегированной стали: диссертация … кандидата технических наук: 05.23.01 – М., 1975. – 142 с.

5. **Каюмов Р.Х.** Устойчивость гибких железобетонных стержней из высокопрочного бетона: диссертация … кандидата технических наук: 05.23.01 – Киев, 1972. – 201 с.
6. **Бамбура А.Н.** Исследование кратковременной устойчивости сжатых железобетонных элементов при нешарнирном закреплении на концах: диссертация … кандидата технических наук: 05.23.01 – Киев, 1978. – 199 с.
7. **Гичко В.В.** Расчет устойчивости гибких железобетонных стоек по деформационному методу (Розрахунок стійкості гнучких залізобетонних стійок за деформаційним методом): диссертация … кандидата технических наук: 05.23.01 – Киев, 2015. – 137 с.
8. **Ханджи В.В.** Расчет многоэтажных зданий со связевым каркасом – М.: Стройиздат, 1977. – 189 с.
9. **Беглов А.Д., Р.С. Санжаровский.** Теория расчета железобетонных конструкций на прочность и устойчивость. Современные нормы и Евростандарты – Санкт-Петербург – Москва: АСВ, 2006. – 221 с.
10. **Санжаровский Р.С.** Железобетон: актуализированный норматив // Строительная газета. – М., 2015, №21.
11. **Чистяков Е.А.** Основы теории, методы расчета и экспериментальные исследования несущей способности сжатых железобетонных элементов при статическом нагружении: диссертация … доктора технических наук: 05.23.01. – М., 1988. – 498 с.
12. **Травуш В.И., Конин Д.В., Рожкова Л.С., Крылов А.С. и др.** Экспериментальные исследования сталежелезобетонных конструкций, работающих на внецентренное сжатие // ACADEMIA. Архитектура и строительство. – М.: 2016, №3. С. 127-135.
13. **Арленинов П.Д.** Деформирование и устойчивость сжатых и внецентренно сжатых стержневых железобетонных элементов с учетом ползучести и трещинообразования – Кандидатская диссертация, АО «НИЦ «Строительство» (НИИЖБ им. А.А. Гвоздева) – М., 2016. – 143 с.
14. **Крылов С.Б., Арленинов П.Д.** Критическая сила для железобетонных стержневых элементов // Academia. Архитектура и строительство, 2012. №2. С. 136-138.
15. **S.P. Chiew, Y.Q. Cai.** Design of high strength steel reinforced concrete columns: a Eurocode 4 approach // Boca Raton: CRC Press, 2018, 105 p.
16. **Теряник В.В.** О проблеме устойчивости внецентренно сжатых бетонных и железобетонных элементов средней и большой гибкости // Вестник Самарского Государственного Технического Университета. Серия: Физико-Математические Науки. – Самара.: 2003, №19. С. 167-169.
17. **Кришан А.Л.**, **Кришан М.А**. Влияние гибкости на несущую способность сжатых железобетонных элементов // Новые идеи нового века. – Хабаровск: 2014. Том 3. С. 308-314.
18. **Lu T., Jin H., Guan K.** Experimental Research on Axial Compression Performance of High-Performance-Fiber-Reinforced-Cement-Composite-Prefabricated Monolithic Composite Columns // Buildings, 2023, No.13, pp. 1748-1763.
19. **Liu X., Pan M., Li W., Jing C., Chang W., Zhang H.** Experimental Study and Bearing Capacity Calculation of Compression-Reinforced Concrete Columns Strengthened with Ultra-High-Performance Concrete // Applied Sciences, 2024, No. 14, pp. 1911-1928.
20. **Ince E.G., Özkal F.M.** Optimization of Structural Steel Used in Concrete-Encased Steel Composite Columns via Topology Optimization. // Applied Sciences, 2024. No. 14, pp. 1170-1182.

21. **Polskoy P.P, Mailyan D. etc** Strength of Compressed Reinforced Concrete Elements Reinforced with CFRP at Different Load Application Eccentricity. // Polymers, 2023. No. 15, pp. 26.
22. **Deng F., Chen T., Xiao C.** Performance and capacity of isolated steel reinforced concrete columns and design approaches // Beijing: China Academy of Building Research (CABR) Technology Co., Ltd., 2014.
23. **Bogdan T., Gerardy J.-C., Davies D.W., Popa N.** Performance and capacity of composite "mega columns" with encased hot rolled steel sections // Eurosteel 2017, Copenhagen, 11 p.
24. **Мухамедиев Т.А., Старичкова О.И.** Расчет прочности сталежелезобетонных колонн с использованием деформационной модели // Бетон и железобетон, 2006, №4 (541). С. 18-22.
25. **Bezgodov I., Kaprielov S., Sheynfeld A.** Relationship between strength and deformation characteristics of high-strength self-compacting concrete // International Journal for Computational Civil and Structural Engineering, 2022, №18(2), pp. 175–183.
26. **Kaprielov S., Sheinfeld, A., Selyutin N.** Control of heavy concrete characteristics affecting structural stiffness // International Journal for Computational Civil and Structural Engineering, 2022, №18(1), pp. 24–39.

Denis V. Konin – Ph.D (Engineering), deputy director for scientific work Research Institute of Building Constructions (TSNIISK) named after V. A. Koucherenko JSC Research Center of Construction: 6, 2nd Institutskaya street, Moscow, 109428, Russia; E-mail: konden@inbox.ru, +7-926-853-99-73.

Конин Денис Владимирович - кандидат технических наук, заместитель директора по научной работе Центрального научно-исследовательского института строительных конструкций (ЦНИИСК) им. В.А. Кучеренко АО «Научно-исследовательский центр (НИЦ) «Строительство»): д. 6, 2-я Институтская ул., Москва, 109428, Россия,; E-mail: konden@inbox.ru, +7-926-853-99-73.