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Bearing Capacity of Concrete Filled Steel Tube Columns

Anatoly Krishan

Abstract

Concrete filled steel tubes columns of circular cross section (CFST) have significant constructive, technological, economic advantages. Therefore, CFST are increasingly used in construction practice. Due to the complex nature of CFST load resistance, regulations of the Europe, Australia, Brazil, India, Canada, China, the USA, Japan, and of a number of other countries recommend using empirical formulas for calculating their bearing capacity. Despite the large number of the experiments, serving as a basis for these formulas, they do not always allow to obtain valid results. Besides, these methods, as a rule, do not allow the calculations of compressed CFST elements, which have any differences from a “classical” design, for example, the presence of a high-strength rod and (or) spiral reinforcement, various types of concrete, the effect of preliminary lateral compression of a concrete core, etc. The purpose of this monograph is to propose the method of deformation calculation of the bearing capacity of compressed CFST elements under short-term load action based on the phenomenological approach and the theoretical positions of reinforced concrete mechanics.

Keywords: concrete filled steel tube columns, spiral reinforcement, method of deformation calculation, strength, deformation, flexibility

1. Introduction

Concrete filled steel tubes columns (CFST) are composite structures. They feature a variety of advantages. CFST have significant constructive, technological, economic advantages and at the same time an architecturally expressive appearance [1–5]. Such obvious CFST advantages as decreased labor consumption of their production due to lack of forms and reinforcement cages and high speed of building erection are quite attractive for construction specialists. Besides, mechanical features of a steel shell and a concrete core combine quite rationally in these columns. The strong steel shell serves as a reliable frame for the concrete core ensuring good volumetric load conditions for it. Due to this, concrete strength of columns with circular cross-section increases 1.8÷2.5 times in average. Concrete, in its turn, protects the walls of the steel shell from loss of stability and corrosion from inside. As a result, concrete and steel mutually increase load-carrying ability of each other and that of the whole element.

In case of emergency (explosions, earthquakes, etc.), another important feature of such columns, high survivability, comes to the fore. It is ensured by high deformability of the concrete core, which, together with its high strength, ensures absorption of

large amounts of energy during strength resistance of the construction. Therefore, CFST of circular cross-section are increasingly used in construction practice.

The high strength and deformability of the concrete core ensure its main advantages, especially for short centrally loaded circular cross-section concrete-filled tubular elements. Due to the complicated nature of CFST load resistance, regulations of the Europe, Australia, Brazil, India, Canada, China, the USA, Japan, and a number of other countries recommend using empirical formulas to calculate their bearing capacity.

Despite the large number of the experiments serving as a base for these formulas they do not always allow to obtain valid results [6, 7]. They have significant limitations in the field of application. They were obtained either from the results of specific laboratory sample testing, or due to statistical processing of the relevant data. First, these formulas are valid only for normal concrete. They give unreliable results for the columns from other types of concrete (for example, fine-grained ones). Secondly, these methods, as a rule, do not allow the calculations of eccentrically compressed concrete filled steel tube elements, which have any differences from a “classical” design, for example, the presence of a high-strength rod [8, 9] and (or) spiral reinforcement [10–12], the application of various types of concrete [13], the effect of preliminary lateral reduction in a concrete core [14], etc.

According to the results of researches carried out by many scientists, the most reliable calculations of the strength of CFST columns can be performed based on the recommendations of the EN 1992-1-1 standard. Moreover, a simplified method is often used in the calculations. But it is based on empirical formulas and is very limited in scope. It is proposed to consider the general case of calculation as well. For its implementation, the following assumptions are made:

- internal forces are determined by elasto-plastic analysis;
- plane sections may be assumed to remain plane;
- contact strength between steel and concrete components must be maintained up to column failure;
- the tensile strength of concrete is neglected.

Design of column structural stability should take into account second-order effects including residual stresses, yielding of structural steel and of reinforcement, local instability, cracking of concrete, creep and shrinkage of concrete, geometrical imperfections.

However, there are no specific methods for practical implementation of such a calculation.

The purpose of this monograph is to propose the method of deformation calculation of the bearing capacity of compressed CFST under short-term load action based on the phenomenological approach.

2. Basic design provisions

2.1 Initial provisions

Initially, the diameter d and wall thickness δ of the tube should be assigned for CFST. Taking into account the research results [7] for columns of circular cross-section, it is recommended to use the following restrictions:

$$20\sqrt{\frac{235}{f_y}} \leq \frac{d}{\delta} \leq 150\frac{235}{f_y}, \quad (1)$$

where f_y is a yield stress of the steel shell, MPa.

For monolithic columns, the possibility of loss of stability of the tube wall at the stage of installation of the supporting structures of the frame should be taken into account. The steel tube can be used as a supporting structure for several overlying floors even before it is filled with concrete, which significantly speeds up the process of constructing a building. In this case, local buckling is impossible when

$$\frac{d}{\delta} \leq 85\sqrt{\frac{235}{f_y}}. \quad (2)$$

If condition (2) is not met, it is necessary to check the stability of the tube walls under the action of corresponding loads. For this purpose, for example, the recommendations of European norm procedure (EN 1993-1-1 Steel Design) can be used.

For a short centrally loaded CFST column, the cross-sectional strength is usually determined. Most researchers use a fairly simple formula for this

$$N = f_{cc}A + \sigma_{pz}A_p, \quad (3)$$

where f_{cc} is strength of volumetrically loaded concrete core;

σ_{pz} is axial direction compression in the steel shell in CFST limit state;

A and A_p are cross-section areas of the concrete core and the steel shell.

Thus, in order to calculate the CFST strength, it is necessary to know the values of the strength of the volumetrically loaded concrete core and the compression in the steel shell. Various approaches and relationships for determining f_{cc} and σ_{pz} are recommended. They are reviewed below.

2.2 Known approaches for determining the strength of a concrete core

Compression strength is a very important mechanical attribute of CFST concrete core. In the limiting state centrally loaded circular section column, concrete is in the conditions of three-axis compression by axial direction strain σ_{cz} and transverse strain σ_{cr} .

A quite simple relationship, being in fact the Mohr-Coulomb strength condition, is most often used in calculations for such conditions

$$f_{cc} = f_c + k\sigma_{cr}, \quad (4)$$

where f_c is concrete unconfined compression strength;

k is coefficient of lateral pressure.

Considering experiments, the value of the k coefficient is usually taken as constant in this formula: $k = 4,1$ or $k = 4,0$.

Though the Eq. (4) was recommended by American researches F. Richard, A. Brandtæg and R. Brown as far back as in 1929, it is currently used by many researches, including for designing columns with different types of confinement reinforcement. The relationships to determine the volumetrically loaded concrete recommended by regulations in many countries have been obtained based on this very formula. However, the gained new experimental materials evidence that the Eq. (4) does not always allow to get a valid result.

This is caused by many reasons. One of them is inaccuracies in determination of lateral strain σ_{cr} . The second reason is ignoring the scale factor. Since CFST frequently have significant cross-sectional dimensions (630 ... 1000 mm and more for high buildings), this factor shall be considered. The research [4] devoted to a review of a government program of concrete-filled tubes research carried out in the end of the 20th century in Japan introduces the following relationship

$$f_{cc} = \gamma_c f_c + k \sigma_{cr}, \quad (5)$$

in which γ_c scale is factor coefficient determined by the formula

$$\gamma_c = 1,67 \cdot d_c^{-0,112} \geq 0,85, \quad (6)$$

where d_c is concrete core diameter in mm.

A similar dependence was proposed in [15].

Regarding such approach as conceptually correct, it is worth mentioning a quite limited range of CFST cross section diameters, where usage of relationships (6) allows to obtain a result acceptable for practical purposes. According to this formula, first, $\gamma_c \approx 1$ when the concrete diameter is 100 mm, and a $\gamma_c \approx 0.95$ when $d_c = 150$ mm. In most countries, square-sided cube test pieces or cylinders with a cross-sectional diameter of 150 mm are considered reference concrete. In this case, the regulations provide that $\gamma_c = 1.05$ when $d_c = 100$ mm. Secondly, one has to take $\gamma_c = 0.85$ already when the cross-section diameter exceeds 300 mm, which does not correspond to experimental data of researches of large scaled samples with cross section diameters between 630 and 1020 mm.

Considering the results of the research [16], the coefficient γ_c is recommended to be determined by the formula

$$\gamma_c = 0,75 + 0,25 \left(\frac{d_0}{d_c} \right)^{0,5}, \quad (7)$$

where d_0 is reference cylinder diameter taken equal to 150 mm.

This formula does not need any limitations in a quite wide range of $d_c = 100$ to 3000 mm, which is convenient for practical calculations.

Another reason of the results obtained by the Eq. (4) not always corresponding to experimental data is the value of the coefficient of lateral pressure $k = 4.1$ taken as constant here. The research [17] shows theoretically that this value is variable. Our researches [16] found that the lateral pressure σ_{cr} reaches sometimes a value of 10÷15 MPa and more for CFST concrete cores before concrete destruction. Meanwhile, the values of the coefficient of lateral pressure can be within a range $k = 2,5 \div 7$. Therefore, it is obvious that even insignificant inaccuracies in determination of k frequently lead to significant errors in determination of concrete core strength f_{cc} and load-carrying ability of a designed element.

Some of researches recommend considering this point. For example, in the research [18] it was correctly mentioned that, other factors being equal, the value of the coefficient of lateral pressure decreases while this pressure increases. A formula is recommended for its determination

$$k = 6,7(\sigma_{cr})^{-0,17}. \quad (8)$$

However, recently a formula of J. Mander has been used more frequently than others [19].

$$\frac{f_{cc}}{f_c} = 2,254 \sqrt{1 + 7,94 \frac{\sigma_{cr}}{f_c}} - 2 \frac{\sigma_{cr}}{f_c} - 1,254. \quad (9)$$

This formula was received based on the results of statistical processing of a large amount of experimental data and is usable for not only medium- but also high-strength concrete with f_c of up to 120 MPa.

However, two main disadvantages of the Eq. (9) should be mentioned. First, lateral pressure σ_{cr} shall be known in CFST limit state to use it. As previously noted, this pressure is unknown when the load-carrying ability of such columns is calculated. Experiments with 180 samples of concrete-filled tubular elements [16] showed that σ_{cr} depends on geometry and design parameters of a designed column and may vary in wide limits. In addition, the relationship (9) is correct only for normal concrete. E.g. it is well known that fine grain concrete resists volumetric compression somewhat worse [17]. That is why other relationships shall be obtained for other concrete types, which causes certain inconveniences in calculations.

Processing of a number of experimental data evidences the existence of a stable relationship between σ_{cr} and a constructive coefficient of concrete-filled tubes ξ determined with use the formula

$$\xi = \frac{f_y A_p}{f_c A}. \quad (10)$$

The appropriate formulas are used in Chinese Technical Code for CFST structures (GB50936–2014).

2.3 State of stress in steel tube

Two methods to assess state of stress in a steel shell are known. The first one hypothesizes that a steel tube acts only transversely in limit state. In this case, the axial direction compression in the steel shell σ_{pz} is equal to zero. Then hoop stress determining the value of the lateral pressure in concrete reaches the yield stress of steel $\sigma_{p\tau} = f_y$. However, in general, it does not correspond to the real state of stress in a steel shell. Most researchers believe that the value of stress σ_{pz} depends on geometry and design parameters of CFST.

In the limiting state, the stress intensity in the steel shell reaches the yield point. During the central compression of a short CFST element, the steel shell experiences a compression-tension-compression stress state. Radial compressive stresses in the wall of steel tubes with $d/\delta \geq 40$ are small and they are usually neglected. Then the plane stress state “compression-tension” is considered for the tube. For this case, the Hencky-Mises yield criterion is written as follows:

$$\sigma_{pz}^2 + \sigma_{p\tau}^2 - \sigma_{pz}\sigma_{p\tau} = f_y^2, \quad (11)$$

where $\sigma_{p\tau}$ is the steel tube hoop stress in CFST limit state. Then the stress σ_{pz} can be calculated using the formula

$$\sigma_{pz} = \sqrt{f_y^2 - 0,75\sigma_{p\tau}^2} - 0,5|\sigma_{p\tau}|. \quad (12)$$

Let us mention that the Eq. (12) is correct for thin-shell tubes when $d/\delta \geq 40$. These very tubes are generally used as steel shells for CFST.

The hoop stresses averaged by thickness in the steel shell for thin-shell tubes can be expressed through the lateral pressure by the following relationship with accuracy sufficient for practical calculations

$$\sigma_{p\tau} = -2\sigma_{cr} \frac{A}{A_p}. \quad (13)$$

Consequently, the axial direction compression in the steel shell depend on its yield stress f_y , the value of the lateral pressure from the concrete core σ_{cr} , and ratio of the column reinforcement.

2.4 Central compression strength

The literature review shows that obtaining a reliable formula for determining the strength of volumetric compressed concrete of CFST elements is not an easy task. Most often, empirical formulas, which have significant limitations depending on the conditions of carried out experiment, are used. In case of structural changes or the use of new types of concrete and steel grades, other formulas will be needed. In this case, it is necessary to correctly determine the lateral pressure of a steel tube σ_{cr} on concrete, which directly affects both the strength of the concrete f_{cc} and the stress in the tube σ_{pz} .

In this regard, it is important to obtain theoretically based, universal formulas for determining f_{cc} , σ_{pz} and the strength of CFST. The solution to this problem is proposed on the basis of the known strength function of volumetric compressed concrete [17]. In the case of uniform lateral pressure, the result of solving this function is Eq. (5) with a variable value k depending on the level of lateral pressure $m = \sigma_{cr}/f_{cc}$ and the type of concrete. For its determination, a formula is recommended

$$k = \frac{1 + a - am}{b + (c - b)m}, \quad (14)$$

where a , b are material coefficients determined based on experiments; c is a parameter determining the nature of strength surface in the area of all-around compression (for a dense concrete core, the strength surface is open, and $c = 1$).

The average values of strength of normal concrete, calculated with a reliability of 50%, correspond to the coefficients $b = 0.096$ and $a = 0.5b$.

The analysis of relationship (14) shows that with high levels of sidework (with $m \rightarrow 1$), the value of the lateral pressure coefficient is $k \rightarrow 1$. In such cases, concrete destruction will be of shear nature, according to Coulomb's law. With the above-mentioned coefficients k for CFST, volumetrically loaded concrete destruction occurs due to combinations of break and shear, which corresponds to numerous experimental data.

Inserting the Eq. (14) into the Eq. (5) and performing some transformations, we will obtain:

$$f_{cc} = \alpha_c \gamma_c f_c; \quad (15)$$

$$\alpha_c = 0,5 + 0,75\bar{\sigma} + 0,25\sqrt{(\bar{\sigma} - 2)^2 + 16\bar{\sigma}/b}, \quad (16)$$

where $\bar{\sigma}$ is a relative value of the lateral pressure from the steel shell on the concrete core in limit state $\bar{\sigma} = \sigma_{cr}/(\gamma_c f_c)$.

Using the relationship (12) and performing some little manipulations, we can write the Eq. (12) as follows

$$\sigma_{pz} = \gamma_c f_c \left(\sqrt{\xi^2 - 3\bar{\sigma}^2} - \bar{\sigma} \right) \frac{A}{A_p}. \quad (17)$$

The formula for $\bar{\sigma}$ calculation is received from solving the task of determination of the maximum compression force received by a short centrally loaded column. Inserting (15), (16) and (17) into the Eq. (3), we obtain the following equation

$$N = \gamma_c f_c A \left[1 + \left(\frac{\bar{\sigma} - 2}{4} + \sqrt{\left(\frac{\bar{\sigma} - 2}{4} \right)^2 + \frac{\bar{\sigma}}{b} - \frac{\bar{\sigma}}{2} + \sqrt{\xi^2 - 3\bar{\sigma}^2}} \right) \right]. \quad (18)$$

It is obvious that the total axial force received by concrete and steel with standard cross-section depends only on relative lateral pressure $\bar{\sigma}$ with fixed values of geometry and design parameters of CFST (f_c, f_y, A, A_p). For illustrative purposes, **Figure 1** represents diagrams of changes of relative forces received by concrete \bar{N}_c and the steel shell \bar{N}_p and their sum \bar{N} depending on $\bar{\sigma}$ value. All forces are determined here in relation to the destructive load.

Figure 1 shows that the graph of the total force change has a maximum point. The maximum compressive force can be found from the equation $\frac{d}{d\bar{\sigma}}(N(\bar{\sigma})) = 0$. After determining the derivative we have the equation

$$\left(\frac{b(\bar{\sigma} - 2) + 8}{\sqrt{b}\sqrt{b(\bar{\sigma} - 2)^2 + 16\bar{\sigma}}} - \frac{12\bar{\sigma}}{\sqrt{\xi^2 - 3\bar{\sigma}^2}} - 1 \right) = 0. \quad (19)$$

As a result of solving Eq. (19), the following formula was obtained

$$\bar{\sigma} = 0,48e^{-(a+b)\xi^{0.8}}. \quad (20)$$

Thus, the necessary formulas to calculate the strength of a short centrally loaded CFST have been received.

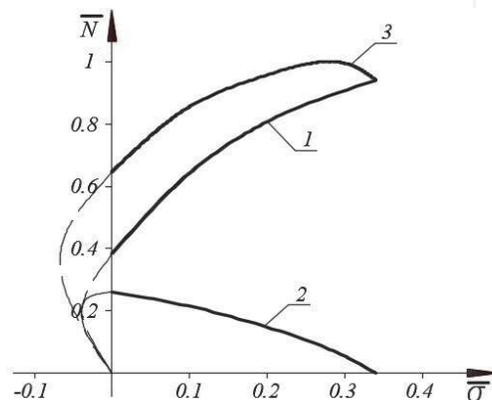


Figure 1. Diagrams of changes of relative compressive forces received by concrete (1) and the steel shell (2) and their sum (3) depending on $\bar{\sigma}$ value.

2.5 Strength calculation of elements with spiral reinforcement

The construction of CFST columns can be improved by placing spiral reinforcement in the concrete core (**Figure 2**). This will have a positive effect on the strength and survivability of columns. A spiral, installed at some distance from the inner surface of the steel tube, can also increase the fire resistance of columns. Experimental studies [10, 11, 20] confirm the high efficiency of such structures.

The widespread practical use of reinforced CFST columns is constrained by the lack of reliable methods for determining their strength. In work [12], a numerical finite element analysis of the load resistance of compressed CFST elements with spiral reinforcement was carried out. But empirical formulas were used here to determine the strength of concrete and lateral pressure on concrete in the limiting state.

The strength of short centrally compressed reinforced CFST column can be determined by formula:

$$N_{u0} = f_{cc}A_c + \sigma_{pz}A_p + \sigma_sA_s, \quad (21)$$

where σ_s is the compressive stress in longitudinal reinforcement in the limiting state of element;

A_s is cross-sectional area of the longitudinal reinforcement.

Under the action of axial compressive force N , lateral pressure on the concrete takes place due to the restraining effect of the outer steel tube and spiral reinforcement. It is impossible to determine this pressure by the superposition principle, since the current problem is physically nonlinear. Therefore, the following calculation method is proposed.

First, the load resistance of a spirally reinforced concrete element that does not have an external steel tube is considered. As a result, the strength of concrete with confinement reinforcement f_{cs} is calculated. At the second stage of the calculation, the interaction of this element and the outer steel shell is taken into account.

To determine the strength of the concrete core f_{cs} , Eq. (15) and (16) are used with the replacement of $\bar{\sigma}$ by $\bar{\sigma}_{sc}$.

The value of relative lateral pressure $\bar{\sigma}_{sc}$ is calculated by the formula:

$$\bar{\sigma}_{sc} = \rho_{sc} \frac{\sigma_{sc}}{\gamma_c f_c}, \quad (22)$$

where ρ_{sc} is coefficient of confinement reinforcement by spirals;

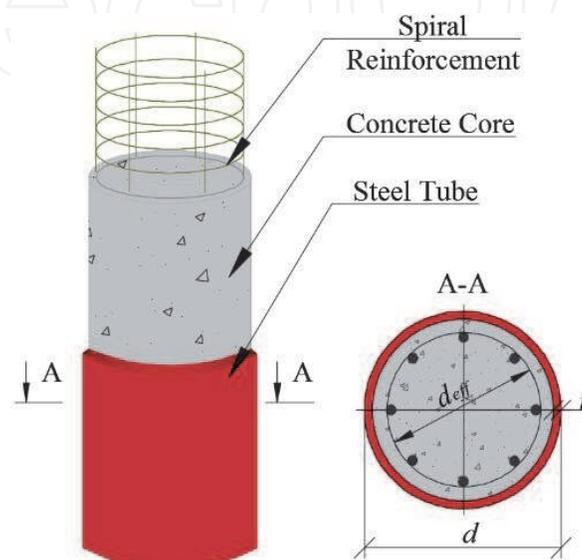


Figure 2.
Reinforce concrete filled steel tube column construction.

$\sigma_{s,c}$ is tensile stress in the spiral reinforcement, which can be determined from the formula:

$$\sigma_{s,c} = \varepsilon_{s,c} E_{s,c} \leq f_{y,c}, \quad (23)$$

where $\varepsilon_{s,c}$ is tensile strain of spiral reinforcement;
 $E_{s,c}$ is modulus of elasticity of steel of spiral reinforcement;
 $f_{y,c}$ is yield point of steel of spiral reinforcement.

The following formula for calculating the value $\varepsilon_{s,c}$ by consecutive approximations is derived in the work [11]:

$$\varepsilon_{s,c} = -\frac{\nu_{zr}}{q\nu_{cs}E_c} f_{cs}, \quad (24)$$

in which,

$$q = 1 - \frac{E_{s,c}}{E_c} \rho_{s,c} (1 - \nu_{rr}), \quad (25)$$

ν_{cs} is the coefficient of elasticity at the maximum stress of concrete with confinement reinforcement.

The value ν_{cs} is calculated using the formula:

$$\nu_{cs} = \frac{f_{cs}}{\varepsilon_{cs} E_c}, \quad (26)$$

where ε_{cs} is the strain of concrete with confinement reinforcement at the maximum stress.

The values of coefficients of transverse deformations ν_{zr} and ν_{rr} are calculated using the formulas obtained in work [16]. The strain ε_{cs} is calculated using the formula obtained below.

Then the strength of spirally reinforced concrete core f_{cc1} , which has an outer steel shell, is determined. For this purpose the Eq. (15) is used, in the right-hand side of which the value $\gamma_c f_c$ is substituted by f_{cs} . The relative lateral pressure $\bar{\sigma}_1$ depends on constructional coefficient ξ_1 , calculated by the formula:

$$\xi_1 = \frac{f_y A_p}{f_{cs} A}. \quad (27)$$

The lateral pressure on the concrete from the steel tube acts outside the diameter of the spiral d_{eff} . This pressure is calculated using the formula (20), but with the replacement of the coefficient ξ by ξ_2 . Constructive coefficient ξ_2 is determined from the formula (10) when the strength of concrete is $\gamma_c f_c$.

Depending on $\bar{\sigma}_2$, the strength of the concrete of the peripheral zone f_{cc2} is calculated.

In order to simplify the calculations it is offered to use the averaged design compressive strength of concrete core f_{cc} for the method of limiting forces. It is determined from the formula:

$$f_{cc} = f_{cc2} (1 - \beta_c^2) + f_{cc1} \beta_c^2, \quad (28)$$

where β_c is the coefficient determined using the formula $\beta_c = d_{eff}/d_c$.

The stress σ_{pz} in the steel tube is calculated by the following formula:

$$\sigma_{pz} = \gamma_c f_c \left[(\xi_2^2 - 3\bar{\sigma}_m^2)^{1/2} - \bar{\sigma}_m \right] \frac{A}{A_p}, \quad (29)$$

in which $\bar{\sigma}_m$ – averaged value of relative lateral pressure of concrete core, calculated by the formula:

$$\bar{\sigma}_m = \bar{\sigma}_1 \beta_c + \bar{\sigma}_2 (1 - \beta_c). \quad (30)$$

The compressive stress in the longitudinal reinforcement σ_s should be determined from the condition of its combined deformation with the concrete core $\varepsilon_s = \varepsilon_{cz}$.

3. Deformation calculation of strength

3.1 General provisions

In a number of earlier published works it is shown that the most reliable calculations of the bearing capacity of CFST columns, taking into account their design features, can be carried out on the basis of nonlinear deformation model. The calculation sequence of similar designs for deformation model is in detail stated in [16].

The calculations are based on the assumptions specified in the EN 1992-1-1 standard. They are listed in the introduction. While processing the experimental data the values of random eccentricity are taken three times less than the values recommended by standards for design purposes. Thus, the centering of the samples along the physical axis is taken into account.

The calculation is based on the relationships between stresses and strains for the concrete core $\sigma_{cz} - \varepsilon_{cz}$, steel tube $\sigma_{pz} - \varepsilon_{pz}$ and reinforcement (if any) $\sigma_s - \varepsilon_s$. The concrete core and steel tube operate under conditions of volumetric stress state, which can change quantitatively and qualitatively with increasing load (**Figure 3**). The accuracy of calculations largely depends on the reliability of the adopted diagrams. At that, the diagrams contained in the regulatory documents are not suitable to evaluate the strength resistance of a concrete core and a steel shell. Therefore, reliable diagrams $\sigma_{cz} - \varepsilon_{cz}$ and $\sigma_{pz} - \varepsilon_{pz}$ need to be constructed initially. The form of a diagram set is recommended to be multipoint one. It is shown in [17] that such a method is the most universal one. At the second stage the strength of the compressed CFST is calculated.

3.2 The first stage of calculation

At the first stage, the deformation diagrams of the concrete core and the steel tube are constructed for the axial direction of the element. For this purpose, the load resistance of a short centrally compressed CFST element is considered. Load is imposed quickly. The concrete core is considered as a transversely isotropic body. The steel tube is considered to be an isotropic body. In the tube the stresses arise in the axial, circumferential and radial directions – σ_{pz} , $\sigma_{p\tau}$, σ_{pr} . The stress signs of the concrete core and steel shell depend on the load level. At low load levels, the value of the coefficient of transverse strains of steel exceeds the value of the coefficient of transverse strains of concrete. For these levels, there is no triaxial compression of concrete (**Figure 3b**). When the value of the coefficient of transverse strains of concrete exceeds the value of the coefficient of transverse strains of steel, the volumetric compression of concrete takes place (**Figure 3c**).

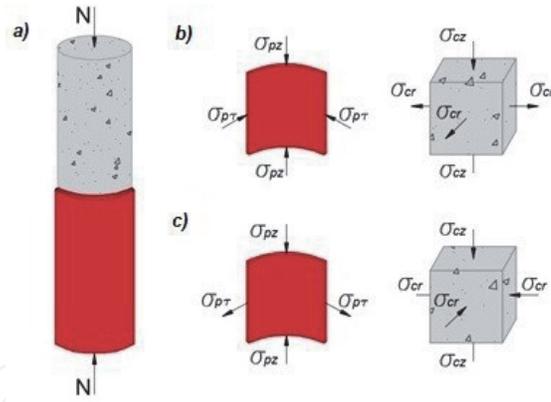


Figure 3. Tension of steel tube and concrete core of the central compressed CFST column: a – scheme of loading; b – at low loading levels; c – at high loading levels.

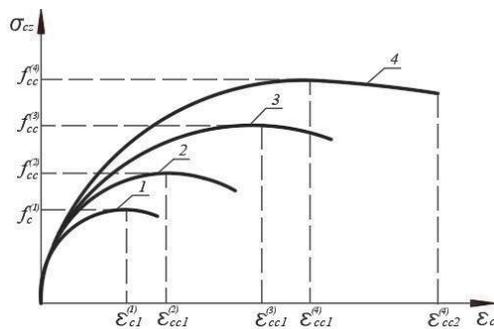


Figure 4. Branch of concrete deformation charts at step-by-step strengthening of axial deformations: 1 - uniaxial compression, 2,3 - volume compression at the intermediate stages of deformation; 4 - volume compression in a limit state.

Curvilinear deformation diagrams are accepted for the concrete core. The coordinates of vertex of each diagram depend on the lateral pressure on the concrete from the steel tube. It is assumed that with an increase of the compressive force N , the lateral pressure on the concrete σ_{cr} goes up from zero to a certain limiting value. Therefore, the calculation requires the use of many such diagrams (**Figure 4**).

The coordinates of vertex of each diagram determine the strength of the concrete core (uniaxially compressed f_c or volumetrically compressed f_{cc}) and its strain (ε_{c1} or ε_{cc1} respectively).

There are many proposals for determining the strain ε_{cc1} in the literature. The main disadvantage of the above formulas is that they are all obtained from the results of the corresponding experiments. It greatly limits the scope of their application.

Let's show how one can get the corresponding formula based on the phenomenological approach.

Figure 5 shows the stress–strain diagram of compressed concrete, corresponding to the maximum reached stress and compare it with the uniaxial compressed concrete diagram. It follows from the above that the initial modulus of elasticity E_c for both diagrams is the same.

The strain ε_{cc1} at the vertex of the diagram $\sigma_{cz} - \varepsilon_{cz}$ is made up of elastic ε_{el} and plastic ε_{pl} components

$$\varepsilon_{cc1} = \varepsilon_{el} + \varepsilon_{pl}. \quad (31)$$

Elastic strain ε_{el} is associated with the elastic part of the strain of uniaxially compressed concrete ε'_{el} by the following relationship:

$$\varepsilon_{el} = \varepsilon'_{el} \frac{f_{cc}}{\gamma_c f_c}. \quad (32)$$

Plastic strain ε_{pl} is associated with the plastic part of the strain of uniaxially compressed concrete ε'_{pl} by a similar relationship:

$$\varepsilon_{pl} = \varepsilon'_{pl} \left(\frac{f_{cc}}{\gamma_c f_c} \right)^m, \quad (33)$$

where m is the exponent, $m > 1$.

The parameter m takes into account the fact that the increase in strains of volumetrically compressed concrete is more intense than the increase in its strength.

Thus, the total deformation of the volume-compressed concrete at the maximum stress is determined by the formula

$$\varepsilon_{cc1} = \varepsilon_{c1} \alpha_c^m \left[1 - \frac{\gamma_c f_c}{\varepsilon_{c1} E_c} \left(1 - \alpha_c^{(1-m)} \right) \right]. \quad (34)$$

The performed statistical analysis showed that the best match with the results of the experiments corresponds to a value of m , calculated by the formula

$$m = 1.7 + \frac{3.5}{\sqrt{\gamma_c f_c}}, \quad (35)$$

where f_c is in MPa.

According to the recommendations of [21] the ultimate strain of a volume-compressed concrete is determined by the formula

$$\varepsilon_{cc2} = \varepsilon_{c2} \frac{\varepsilon_{cc1}}{\varepsilon_{c1}}, \quad (36)$$

where ε_{c2} is the ultimate strain for uniaxial compressed concrete.

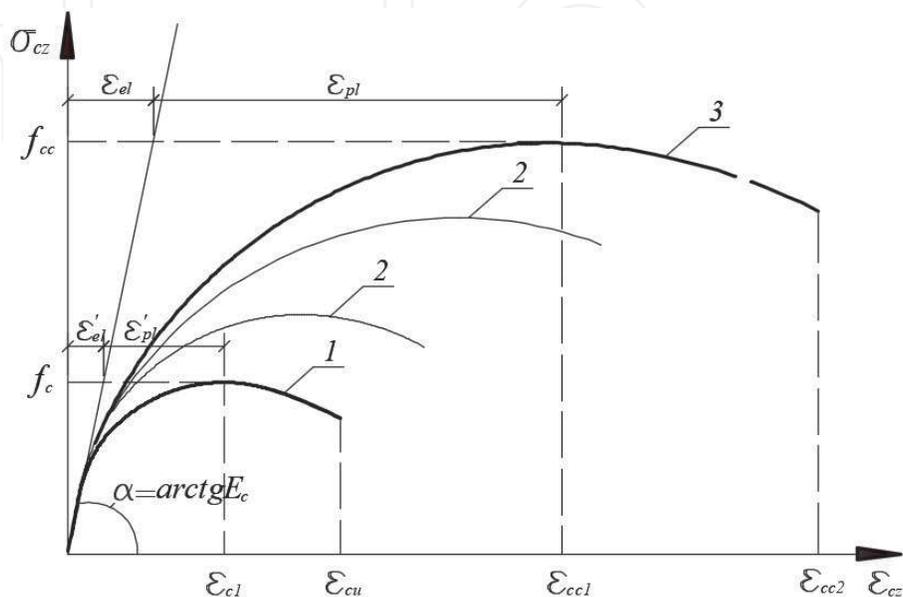


Figure 5.
The graphs of deformation for uniaxial compressed (1) and volume-compressed (2,3) concrete.

When coordinates of parametric points of the deformation charts of volumetrically compressed concrete are known, it is possible to calculate the bearing capacity of CFST columns based on the deformation model analysis.

To construct the diagrams $\sigma_{cz} - \varepsilon_{cz}$ and $\sigma_{pz} - \varepsilon_{pz}$, a step-by-step increase in the strains of the concrete core and steel tube is carried out while ensuring the condition $\varepsilon_{cz} = \varepsilon_{pz}$. All components of the stress-strain state of concrete and steel are calculated at each j -th step.

The analytical relationship between strains and stresses for any point of the concrete core is written in the form of a system of equations:

$$\begin{Bmatrix} \varepsilon_{cz} \\ \varepsilon_{cr} \end{Bmatrix} = \frac{1}{E_c} \times \begin{bmatrix} \nu_{cz}^{-1} & -2\nu_{zr}\nu_{ci}^{-1} \\ -\nu_{zr}\nu_{ci}^{-1} & (\nu_{cr}^{-1} - \nu_{rr}\nu_{ci}^{-1}) \end{bmatrix} \times \begin{Bmatrix} \sigma_{cz} \\ \sigma_{cr} \end{Bmatrix}. \quad (37)$$

The elastic-plastic properties of concrete are taken into account by the coefficients of elasticity ν_{ck} ($k = z, r, i$) and variable coefficients of transverse strains ν_{zr}, ν_{rr} . The subscripts z and r are used for axial and transverse directions, and the subscript i is used for the coefficient of elasticity depending on the intensity of stress and intensity of strain.

The values of the intensity of stresses and strains are calculated using the well-known formulas of solid mechanics. Using the coefficients of elasticity ν_{ci} and transverse strains ν_{zr}, ν_{rr} , the strains along one direction (axial or transverse) depending on the stresses of the other direction are calculated in a matrix of system pliability (37).

The stress state of a steel tube obeys the hypothesis of a uniform curve [22]. In accordance with this hypothesis, the dependence $\sigma_{pi} - \varepsilon_{pi}$, obtained under uniaxial tension, is accepted for complex stress states. Here σ_{pi} is the intensity of stresses, and ε_{pi} is the intensity of strains.

The initial diagram " $\bar{\sigma}_p - \bar{\varepsilon}_p$ " is recommended to be tri-linear (rules of Russia - CR 266.1325800.2016). However, when modelling steel sections under a complex stress state, it is advisable to use a deformation diagram calculated using the generalized parameters $\bar{\sigma}_{pi} = \sigma_{pi}/f_y$ and $\bar{\varepsilon}_{pi} = \varepsilon_{pi}E_p/f_y$ (Figure 6). The coordinate values of the characteristic points of the generalized diagram can be taken from Table 1.

Communication between strains and stresses for any point of an external steel shell in elastic and elasto-plastic stages can be presented the following equations system:

$$\begin{Bmatrix} \varepsilon_{pz} \\ \varepsilon_{p\tau} \\ \varepsilon_{pr} \end{Bmatrix} = \frac{1}{\nu_p E_p} \times \begin{bmatrix} 1 & -\nu_p & -\nu_p \\ -\nu_p & 1 & -\nu_p \\ -\nu_p & -\nu_p & 1 \end{bmatrix} \times \begin{Bmatrix} \sigma_{pz} \\ \sigma_{p\tau} \\ \sigma_{pr} \end{Bmatrix}. \quad (38)$$

Here $\sigma_{pz}, \sigma_{p\tau}, \sigma_{pr}$ are normal (main) stresses in a tube in the axial, circumferential and radial directions; $\varepsilon_{pz}, \varepsilon_{p\tau}, \varepsilon_{pr}$ are strains of a steel tube in the corresponding directions; E_p is the initial module of tubes elasticity; ν_p is coefficient of steel elasticity; ν_p is coefficient of tubes cross strain.

The stresses and strains acting on the principal planes are used in Eqs. (37) and (38). Experiments show [16] that in the stage of yield Chernov-Luders lines appear on the surface of the steel tube. These lines are angled 45° to the longitudinal axis of the CFST. Therefore, shear stresses and shear strains are equal to zero here.

The stress-strain states of the concrete core and steel tube largely depend on the values of the coefficients of transverse strain and the coefficients of elasticity of the materials. Therefore, their reliable determination is very important when

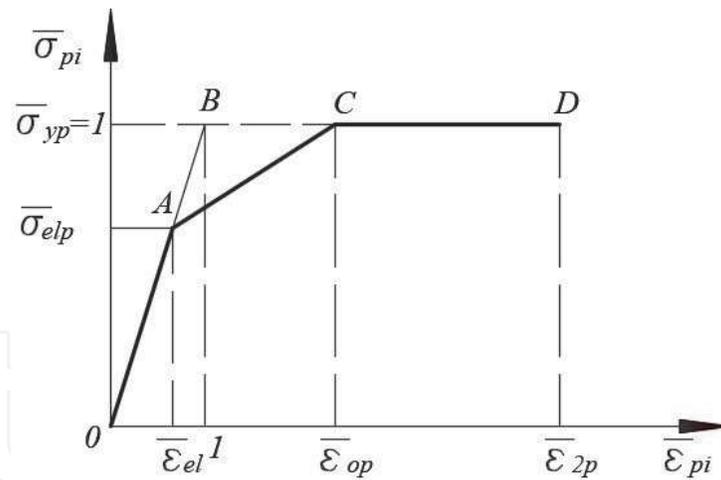


Figure 6. Generalized calculation diagram of steel, operating under conditions of complex stress state.

Parameter of diagram	Steel classes according to the set of rules Russia - SP 16.13330.2018					
	S245, S255	S285	S345, S345K, S375	S390	S440	S590, S590K
$\bar{\epsilon}_{el}$	0,80	0,80	0,80	0,90	0,90	0,90
$\bar{\sigma}_{elp}$	0,92	0,92	0,92	1,00	1,00	1,00
$\bar{\epsilon}_{op}$	1,70	1,70	1,70	1,70	1,70	1,70
$\bar{\sigma}_{yp}$	1,00	1,00	1,00	1,00	1,00	1,00
$\bar{\epsilon}_{2p}$	14,0	15,0	16,0	17,0	17,0	18,0

Table 1. Coordinates of characteristic points of the generalized steel deformation diagram, constructed in the axes $\bar{\sigma}_{pi} - \bar{\epsilon}_{pi}$.

calculating the strength of CFST columns. Formulas for calculating these coefficients are given in work [16].

The solution of the Eqs. (37) and (38), taking into account the joint deformation of concrete and steel tube, allows obtaining the formula for calculating the lateral pressure

$$\sigma_{cr} = \frac{\left(v_p - v_{zr} \frac{d_c}{d_c + \delta} \frac{\nu_{cz}}{\nu_{ci}} \right) \epsilon_{cz}}{K_p + K_c}, \quad (39)$$

in which K_p and K_c are the parameters defining condition of steel shell and concrete core.

$$K_p = \frac{0,5v_p}{\nu_p E_p} \left[v_p \left(\frac{d}{\delta} - 1 \right) - \left(\frac{d}{\delta} + 1 \right) \right]; \quad (40)$$

$$K_c = \frac{\beta_r}{\nu_{ci} E_c} \left(\frac{2v_{zr}^2 \nu_{cz}}{\nu_{ci}} + v_{rr} - \frac{\nu_{ci}}{\nu_{cr}} \right). \quad (41)$$

When the strain ϵ_{cz} and lateral pressure σ_{cr} are known, all other components of the stress–strain state of CFST column can be calculated. The strains are incrementally increased until the stress σ_{cz} reaches the strength of volumetrically compressed

concrete $f_{cc}^{(n)}$ (Figure 4), previously calculated using formulas (15) and (16). The calculation is performed on a computer.

After that we compare the last value of strain ε_{cz} with the strain calculated on a formula (34) $\varepsilon_{cc1}^{(n)}$ in top of the deformation chart of concrete. In the existing incoherence $|\varepsilon_{cz} - \varepsilon_{cc1}^{(n)}| > \Delta_\varepsilon$ (Δ_ε – the accuracy of calculations set by the estimator) we specify value of an exponent m in a Eq. (34) and repeat all calculations.

Upon termination of calculations we receive arrays of numerical data for deformation charting of concrete core $\{\varepsilon_{cz}\} - \{\sigma_{cz}\}$ and steel shell $\{\varepsilon_{pz}\} - \{\sigma_{pz}\}$.

3.3 The second stage of calculation

At the second stage, the bearing capacity of the eccentrically loaded CFST element is calculated. The design scheme of the normal section of element is shown in Figure 7.

In the calculation process, the deformation of the most compressed fiber of the concrete core $\varepsilon_{c\max}$ is increased step-by-step. At each step, using the Bernoulli hypothesis, the diagram of strain fiber of the steel tube $\varepsilon_{p\min}$. The search for this value is carried out with a gradual shortening strain decrease (starting from $\varepsilon_{c\max}$) or the build-up of the elongation strain $\varepsilon_{p\min}$ (starting from zero).

The normal section of the calculated element is conditionally divided into small sections with areas of concrete A_{ci} and steel shell A_{pk} . In the presence of longitudinal reinforcement, the cross-sectional area of each bar is designated as A_{sj} .

The origin of coordinates is aligned with the geometric center of the element's cross section. If the Bernoulli hypothesis is observed, there is a strain in the center of each section of concrete and steel tube. With known strains, the corresponding stresses are determined according to the results of the first stage of the calculation. The stresses are assumed to be evenly distributed within each section of concrete

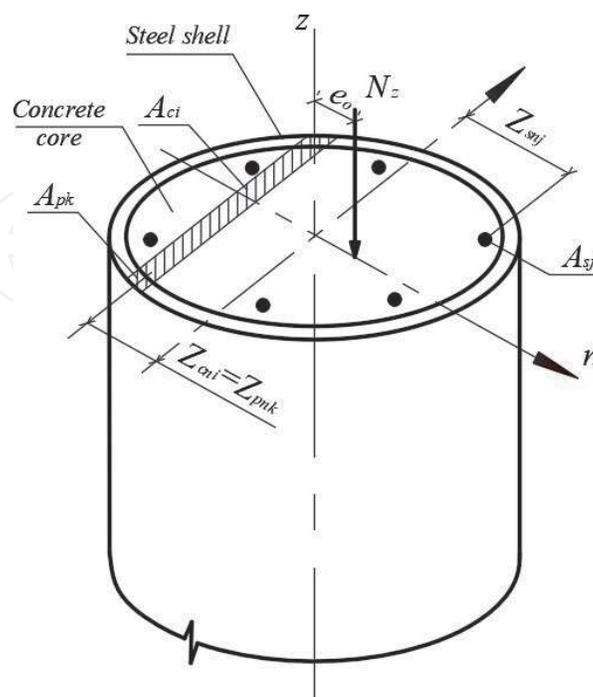


Figure 7. Design model of the normal section of the CFST element deformations of the normal cross section is designed, corresponding to the equilibrium condition of the calculated element. In order to develop such a diagram it is required to find the corresponding value of the strain of the least compressed (stretched).

and steel tube. After each step of strain $\varepsilon_{c \max}$ increasing, it is necessary to ensure that the equilibrium conditions are met:

$$N_z = \sum_i \sigma_{czi} A_{ci} + \sum_k \sigma_{pzk} A_{pk} + \sum_j \sigma_{sj} A_{sj}; \quad (42)$$

$$N_z e_0 = \sum_i \sigma_{czi} A_{ci} Z_{cni} + \sum_k \sigma_{pzk} A_{pk} Z_{pnk} + \sum_j \sigma_{sj} A_{sj} Z_{snj}, \quad (43)$$

in which Z_{cni} , σ_{czi} – the coordinate of the gravity center of the i -th section of concrete and the stress of the axial direction at the level of its gravity center; Z_{pnk} , σ_{pzk} – the coordinate of the gravity center of the steel shell k -th section and the stress of the axial direction at the level of its gravity center; Z_{snj} , σ_{sj} – the coordinate of the gravity center of the of the longitudinal reinforcement j -th bar and the stress in it.

When both equilibrium conditions are met, the value of the compressive force N_z corresponding to the given strain $\varepsilon_{c \max}$ is fixed. Next, the strain of the most compressed fiber of the concrete core increases and all calculations are repeated. The limiting values of this strain ε_{ccu} can be accepted according to the recommendations [21].

The problem of determining the strength reduces to finding the value of the strain of the most compressed fiber $\varepsilon_{c \max} \leq \varepsilon_{cc1}$, corresponding to the maximum value of the compressive longitudinal force N_u . The calculation results show that under certain design parameters of CFST columns, the strain $\varepsilon_{c \max}$ does not reach ε_{cc1} . Then the stress in the concrete σ_{cz} cannot achieve its strength at triaxial compression. This design situation occurs when using low strength concrete and a strong steel shell with a small ratio d/δ . Therefore, the criterion for the loss of the strength of the column is the achievement of the maximum value of compressive force in the process of increasing the strain of the most compressed fiber.

The proposed method makes it possible to limit the axial strains of the columns. It is known from experiments that the strain of compressed CFST elements can reach 5 ÷ 10% [16]. With such strains, the operation of the columns of the buildings becomes impossible. Thus, excessive strain can determine the ultimate limit state of the CFST column. The maximum permissible values of these strains can be set by a structural engineer, depending on a specific design situation for a designed building or a structure.

3.4 Calculation of flexible elements

Due to the complex nature of load resistance of CFST columns, in design practice, as a rule, the simplified methods of calculation of their bearing capacity are used. At that, flexibility is usually taken into account by the coefficient of longitudinal bending, determined according to empirical relationships. In the monograph we consider the deformation calculation of CFST column bearing capacity.

A rod of a circular cross-section with a constant length, loaded by a compressive force N applied to the ends with the same initial eccentricity e_0 (no less accidental than e_a) and hinged at its ends is regarded as a basic case. The deformation scheme of such a rod is shown on **Figure 8**.

According to the known positions of structural mechanics, if we apply force N along the axis that coincides with the physical gravity center of an elastic rod cross-section, the rod will remain a rectilinear one until the force reaches the value of the critical load N_u corresponding to the moment of stability loss. Only after that the middle part of the rod will receive the corresponding deflection f_{cr} .

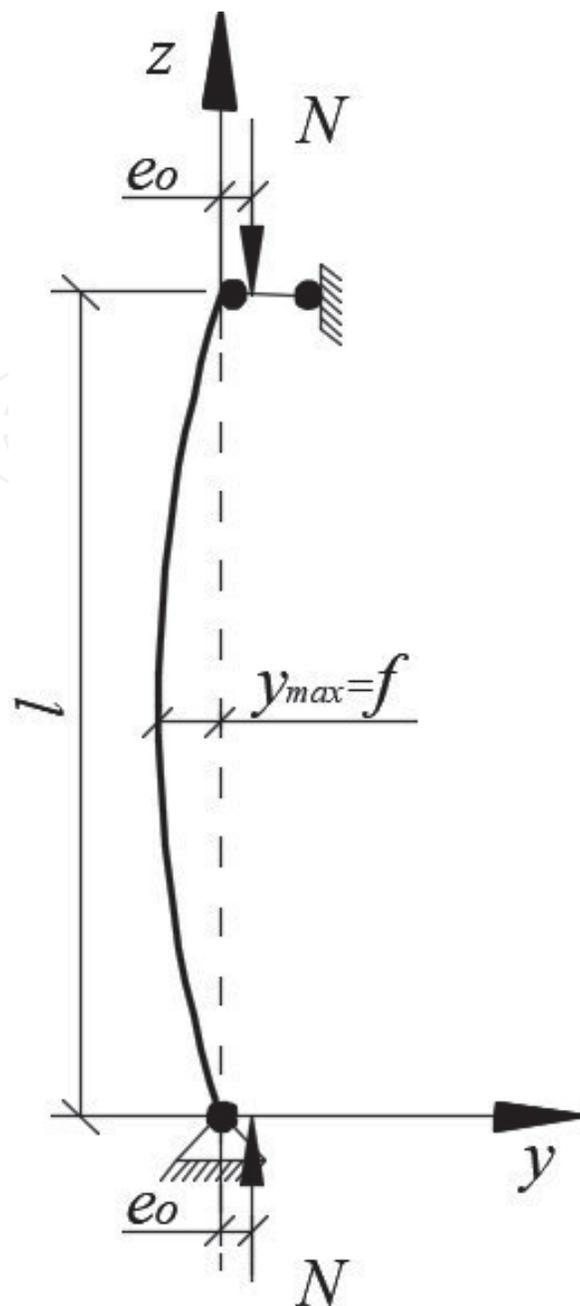


Figure 8.
 The scheme of a compressed rod deformation.

A bending moment M will appear in any section along the length of the bar from the compressive force N . The moment M is calculated by the formula

$$M = N(e_0 + y), \quad (44)$$

where y is the horizontal displacement value of the cross-section in question.

With the increase of the bending moment, the strength of a compressed rod normal section decreases, which must be taken into account during the calculation. On the other hand, the axial load increase to a critical value in the columns of great flexibility can lead to a very significant increase of transverse deformations - the loss of stability of the second kind. With a certain transverse deflection, the compressive load reaches a maximum value, after which its decrease is observed with a further deflection increase (**Figure 9**). At the same time, the strength properties of materials from which the column is made will not be implemented fully.

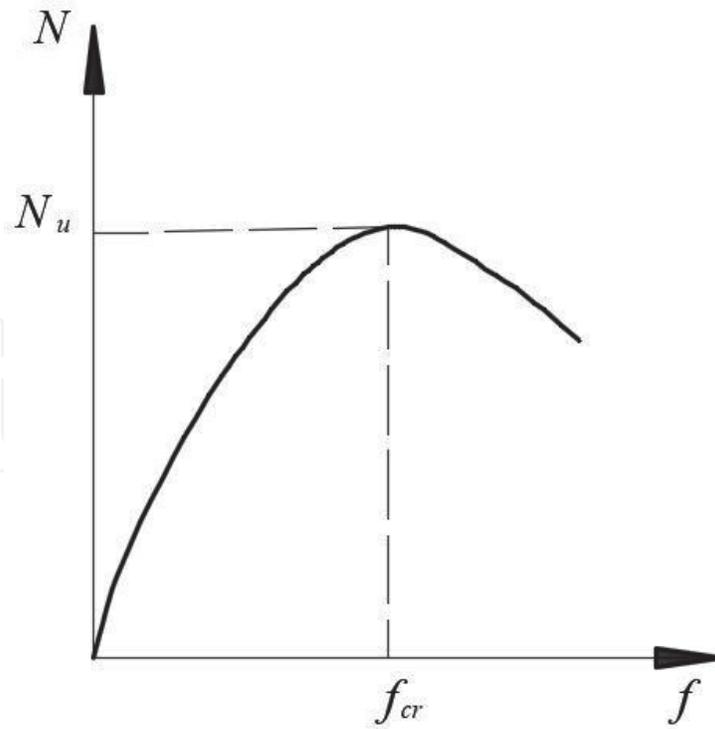


Figure 9.
The dependence of compressive force on deflection

The main assumptions that are directly relevant to this study are the following ones:

- the calculation is based on the theory of small displacements;
- the shear deformations are neglected in comparison with the bending deformations of the rod axis;
- the distribution of deformations along a cross section corresponds to the hypothesis of plane cross sections.

The flexibility of the column is determined for the reduced cross-section. For the base case under consideration, this flexibility can be approximated by the following formula: in which l is the estimated length of the rod; $(EI)_{eff}$, $(EA)_{eff}$ are effective stiffness of the most loaded reduced section for bending and compression.

$$\lambda_{eff} = l \cdot \sqrt{\frac{(EA)_{eff}}{(EI)_{eff}}}. \quad (45)$$

It is recommended to calculate the stiffness $(EI)_{eff}$ and $(EA)_{eff}$ in the first approximation by the following formula:

$$(EI)_{eff} = 0,5E_cI_c + 0,5E_pI_p + E_sI_s; \quad (46)$$

$$(EA)_{eff} = 0,5E_cA_c + 0,5E_pA_p + E_sA_s, \quad (47)$$

where I_c, I_p, I_s are the moments of inertia of a concrete core, a steel tube and a longitudinal reinforcement; E_c, E_p, E_s are the moduli of elasticity of concrete, steel case and longitudinal reinforcement.

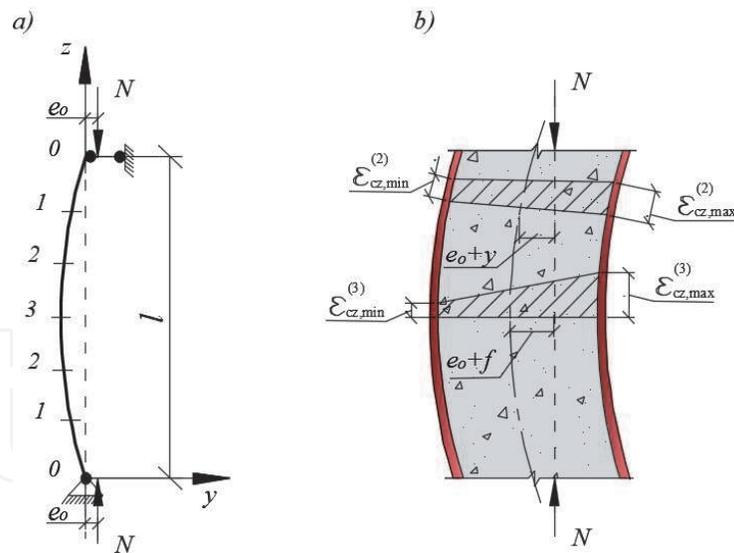


Figure 10. The design scheme of a flexible pipe-concrete column: a - the decomposition of the compressed rod along the length; b - distribution diagrams of concrete relative deformations in Section 2 and 3.

Flexibility can have a significant effect on the load capacity of compressed elements when the condition $\lambda_{eff} > \lambda_0$ is performed, in which the threshold value of flexibility is calculated by the following formula

$$\lambda_0 = \pi \sqrt{0,01(EA)_{eff}/N_{u0}}, \quad (48)$$

where N_{u0} is the strength of a short, centrally compressed CFST element.

The compressive stress in the longitudinal reinforcement σ_s is determined from the condition of its joint deformation with the concrete core. This takes into account the limitation $\sigma_s \leq f_{y,s}$.

The calculation is based on the step-iteration method. During the second stage, an eccentrically loaded compressed element is divided along its length into n equal segments, at that $n \geq 6$ (Figure 10). Normal sections at the end of each segment are divided into small sections conventionally with the areas of concrete A_{ci} and steel A_{pk} tube.

The area of one rod of longitudinal reinforcement is A_{sj} . Then the calculation process is performed in the following sequence. First, only one normal cross-section of a rod is considered, in which the maximum bending moment arises. This cross section is located in the middle of the column height for the articulated column loaded by a compressive force N with the initial eccentricity e_0 . The strain of the most compressed fiber of the concrete core $\epsilon_{cz,max}$ is increased stepwise in this section.

At each step, the relative deformation of the least compressed (stretched) fiber $\epsilon_{cz,min}$ is determined, corresponding to the conditions of equilibrium cross section. The equilibrium conditions are written in the form of the following equation system:

$$N = (EA)_{eff} \epsilon_0; \quad (49)$$

$$N(e_0 + f) = (EI)_{eff} \frac{1}{r}, \quad (50)$$

where N is the longitudinal compressive force corresponding to the accepted deformation diagram; ϵ_0 is a fiber relative deformation located at the gravity center

of calculated section; f is the deflection at the point of maximum bending moment; $\frac{1}{r}$ is the curvature of the longitudinal axis in the considered cross-section, determined by the following formula

$$\frac{1}{r} = \frac{\varepsilon_{cz \max} - \varepsilon_{cz \min}}{d - 2\delta}. \quad (51)$$

Cross-section stiffnesses $(EA)_{eff}$ and $(EI)_{eff}$ are found taking into account the corresponding elastic coefficients of concrete and steel [7].

The effect of longitudinal bending is taken into account via the eccentricity of the longitudinal force increase by the amount of rod deflection f in the calculated section. In the first approximation, the deflection value is determined depending on the curvature of the calculated normal section. Taking into account the dependence (51), we can write the following formula

$$f = \frac{l^2}{\pi^2} \frac{\varepsilon_{cz \max} - \varepsilon_{cz \min}}{d - 2\delta}, \quad (52)$$

where l is the estimated length of the considered rod.

An improved deflection value f should be found at each calculation step for a more reliable calculation of a compressed rod longitudinal bending. This can only be done by adjusting the stiffness along a rod length.

The numerical solution of the problem of calculating the deflection [16] with the number of partitions $n = 6$ allows us to obtain the following formula

$$f = \frac{l_0^2}{266} \left(\frac{1}{r_0} + 6 \frac{1}{r_1} + 12 \frac{1}{r_2} + 8 \frac{1}{r_{\max}} \right), \quad (53)$$

where $\left(\frac{1}{r_0}\right)$ is the curvature of the element on the upper (lower) supports; $\left(\frac{1}{r_i}\right)$ is the curvature of the element in the i -th section; $\left(\frac{1}{r_{\max}}\right)$ is the curvature in the middle of the height.

The problem under consideration is solved as follows. The deviations y of the longitudinal axis of the compressed rod from the vertical are calculated in the sections at the boundaries of each segment into which an element is divided with the deflection found in the first approximation according to the formula

$$y = f \sin(\pi z/l). \quad (54)$$

Then the distribution of the relative deformations is established for these cross-sections, using the Eqs. (49) and (50) and by the replacement of f into y . Moreover, during the determination of $\varepsilon_{cz \max}$ and $\varepsilon_{cz \min}$ for each section, it is necessary to satisfy two conditions:

- the equilibrium of the normal section, i.e. the observance of equalities by the Eqs. (49) and (50);
- the constancy of the longitudinal force value, which is assumed to be the same as for the mean most stressed section.

Let's note that the stiffness characteristics $(EA)_{eff}$ and $(EI)_{eff}$ depend on the parameters of the strain diagram. Therefore, they will be different for each section.

After the determination of $\varepsilon_{cz \max}$ and $\varepsilon_{cz \min}$ according to the Eq. (51), the curvatures in the support and intermediate sections of the rod are found, and by the Eq. (53) the deflection f is specified. The process of deflection refinement can be repeated until a predetermined calculation accuracy is achieved.

They record the value of the compressive longitudinal force N for the assumed value of the relative strain of the most compressed fiber of the concrete core $\varepsilon_{cz \max}$ of the average cross-sectional rod and the refined deflection f . Then the strain of the most compressed fiber of the concrete core $\varepsilon_{cz \max}$ is increased and the whole procedure of calculations is repeated. Thus, the dependence “ $N - f$ ” is developed (see **Figure 9**). The maximum value of the longitudinal force N_u , perceived by the rod, is taken as the bearing capacity.

4. Comparison of calculated bearing capacity with experimental data

According to the proposed method, the algorithm for estimate the stress–strain state and calculate the load-bearing capacity of compressed concrete filled steel tube elements was developed and this algorithm was implemented in the computer program. The results of the calculations are compared with the experiment data of CFST samples made of normal concrete. These data were obtained by many researchers for 569 experiments with short centrally compressed columns, 512 flexible centrally compressed columns and 292 eccentrically compressed elements.

Experimental data was taken from research works [16, 23, 24].

In order to obtain more objective information, the experimental data of samples were analyzed with a large range of geometric and structural parameter variation:

- an outer diameter of an outer steel shell – $d = 89 \div 1020$ mm;
- the thickness of an outer steel shell wall – $\delta = 0.8 \div 13.3$ mm;
- the yield point of a shell steel – $R_s = 165.8 \div 853$ MPa;
- the prismatic strength of the initial concrete – $R_b = 11.7 \div 127$ MPa;
- various concretes (normal, ultrahigh-strength, pre-stressing);
- length to diameter ratio $l/d = 2 \div 49$;
- the relative eccentricity of the longitudinal force $e_0/d = 0 \div 0.94$.

The results of the comparison show a completely satisfactory coincidence of experimental destructive loads with theoretical values (**Table 2**).

The data in **Table 2** show a good agreement between theory and practice.

According to the results of the data of work [23], the calculations according to Eurocode 4 (EN 1994-1-1: 2004) have a slightly worse accuracy. However, the main advantage of the proposed calculation method is its versatility. In particular, when using this method, one can take into account the presence of a high-strength rod and (or) spiral reinforcement, the effect of preliminary lateral compression of the concrete core [16]. The research work [13] verified the acceptability of the EN 1994-1-1: 2004 method for calculating the strength of compressed CFST made of various types of concrete: normal, ultrahigh-strength, self-compacting, light-weight concretes and engineered cementitious composite. It is concluded that the

Type of tested elements	No of tests	Average Test/Calculate	Stand. Deviation Test/Calculate
Short No Moment	569	1.04	0.068
Long No Moment	512	1.08	0.077
Long and Short with Moment	292	1.06	0.072
The overall	1373	1.07	0.073

Table 2.

Summary of Comparison of Calculated Bearing Capacity with Experimental Data.

calculation accuracy is satisfactory only for normal concrete. The proposed method makes it possible, with an appropriate selection of the material coefficients a and b in Eqs. (14), (16) and (20), to provide the required accuracy of calculations.

Based on the results of the carried out analysis, the following values of the coefficients of materials for various types of concrete can be recommended:

- for fine grained and for ultrahigh-strength concrete – $b=0.13$ and $a=0.5b$;
- for self-compacting concrete – $b = 0.098$, $a = 0.5 b$;
- for lightweight concrete and for engineered cementitious composite – $b = 0.3$, $a = 0.5 b$.

Given recommendations are preliminary and need to be clarified, since they have been obtained on the basis of processing a very limited amount of experiments.

5. Discussion

The analysis of the results of the carried out researches shows that there are very significant advantages of the nonlinear deformation model in comparison with the currently used methods for calculating the bearing capacity of CFST columns. The proposed calculation method takes into account the complex stress state of the concrete core and steel tube, which is constantly changing with increasing load, and the physical and geometric nonlinearity of the structure. In the course of the calculation, it is possible to obtain a clear picture of the stress–strain state of the structure at various stages of loading.

The main dependences for finding the strength and strain characteristics of a concrete core and a steel tube are obtained phenomenologically. They correspond to the basic principles of solids mechanics. The resulting formulas are more universal than empirical dependencies. For example, they are true for different types of concrete. In principle, the developed method is applicable for calculating the bearing capacity of composite columns with various cross-sectional shapes and various variants of reinforcement of a concrete core. Differences in designs are easily taken into account when developing calculation algorithms for specific tasks.

The use of a multi-point method for constructing the diagrams of concrete deformation allows improving the accuracy of calculations. Previously, these diagrams were accepted either for uniaxially compressed concrete, or for volumetrically compressed concrete at the stage of ultimate equilibrium of the structure. In

the first case the value of the bearing capacity turned out to be underestimated, and in the second case - overestimated.

The proposed criterion for achieving the bearing capacity of CFST columns is important for practical calculations. The use of this criterion makes it possible to identify the cases when the strength properties of a concrete core cannot be fully used. Calculation by the method of limiting efforts does not always reflect the physical essence of the process and can lead to significant errors.

From the point of view of modern concepts of solid mechanics, steel-reinforced concrete structures refer to nonlinear and non-equilibrium deformable systems. The feature of such system calculation is the need to refine the values of the existing forces and displacements consistently, since the internal forces and the rigidity of the structures are interdependent.

The proposed method of CFST load capacity calculation allows to take into account these features. Considering flexibility the higher stiffness of the compressed rod is taken into account at the sites located closer to its supports. In this regard, it is obvious that the correct implementation of this method in practice will allow to obtain more reliable calculation results in comparison with the currently used semi-empirical approach.

Besides, this method makes it possible to perform the calculations of normal cross section and stability strength from a unified point of view. During the calculation, it is possible to track (in terms of longitudinal deformation value) the completeness of concrete and steel strength property use. If the material deformations reach the maximum permissible values, it can be concluded that the strength of the structures is lost. If this is not observed in the loss of the load-bearing capacity of the structure, a conclusion can be made about the loss of stability of the second kind.

It is especially important, that the proposed method with an appropriate refinement can be used for calculating the compressed structures made of various constructional materials.

One more important circumstance should be noted. It is known that in CFST columns, even before the onset of complete loss of bearing capacity, axial deformations can reach excessively large values at which the operation of real structures becomes impossible. In these cases, the limiting deformation can become dominant, determining ULS. In this regard, during the calculation of bearing capacity the axial deformations of the compressed CFST elements should be limited. This approach can be implemented only when calculating with the use of a nonlinear deformation model of reinforced concrete.

The proposed method can be effectively used to calculate long-term load columns [25].

6. Conclusions

A new technique to determine the strength of compressed CFST was proposed. Based on the known principles of deformation calculation, it takes into account the specific features of CFST adequately. The methodology uses new dependencies to determine the strength and the ultimate deformation of a concrete core, as well as the way of concrete deformation diagram development. It allows to perform the combined calculation of CFST strength, taking into account their flexibility and the calculation of possible stability loss. There is no need for an empirical formula to determine the critical force proposed by modern design standards for composite structural steel structures in the practical application of the method.

The versatility of this method should be emphasized separately. The method is acceptable for CFST columns made of various types of concrete using various technologies.

The practical use of the proposed method gives a reliable estimate of the stress–strain state and the strength of concrete filled steel tube columns.

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