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Load-Bearing Capacity of Short Concrete-Filled Steel Tube Columns of Circular Cross Section.

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ABSTRACT

Estimation of load strength of confined concrete tube elements is characterized with peculiar features. Their concrete core and steel shell are in the state of spatial stress condition which in the course of loading varies both quantitatively and qualitatively. Thus, analytical description of determining correlations and process of redistribution of loads between the components of this system is somewhat difficult. The proposed procedure of calculation of load-bearing capacity of eccentrically confined composite structure, concrete filled tube column, is based on non-linear strain model. It takes into account spatial stress-strain state and collaboration of concrete core and steel shell. Practical implementation of this model is carried out by iterative calculation involving non-elastic strain of material and variation of coefficients of transversal strains of concrete and steel with increase in stresses. It is proposed to perform this calculation in two stages. At the first stage the diagrams of strain of concrete core and steel shell are plotted for axial direction of columns on the basis of calculations. At first the coordinates of parametric points of the required diagrams are determined, then the relative strain--stress data array is established. At the second stage, on the basis of known dependences of non-linear strain model, the load-bearing capacity of eccentrically confined column is calculated. Practical implementation of the proposed calculation procedure is aided by specially developed software. Comparisons of theoretical and experimental values of breaking loads for 110 centrally confined and 72 eccentrically confined column specimens evidence that the proposed procedure enables reliable estimation of their stress-strain state at any level of loading. The procedure can be applied for confined concrete tube columns manufactured by various technologies on the basis of concretes of various strength and types.

Keywords: confined concrete tube columns, load-bearing capacity, spatial stress state, non-linear strain model.

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INTRODUCTION

Confined concrete tube column consists of steel shell filled with concrete. Determination of load-bearing capacity of confined concrete tube elements can be considered as complicated problem of estimation of load strength of composite structure. The main difficulty is the necessity to consider for spatial stress-strain state and collaboration of concrete core and steel shell [1–5].

Taking into account peculiar features of stress-strain state of concrete core--steel shell system, which in the course of loading of confined concrete tube element, depending on the ratio of coefficients of transversal strains of concrete u_b and steel u_p , varies both qualitatively and quantitatively (Fig. 1), the analytical description of major correlations and redistribution of loads between the components of this system encounters certain difficulties. Herewith, the issue criterion corresponding to achievement of extreme limit state of confined concrete tube columns is important up till now [6].

Our studies [7] evidence that achievement of maximum compressing force in the course of gradually applied load should be considered as a criterion of loss of load-bearing capacity of confined concrete tube columns. Taking into account the pattern of reinforcing of such columns, it is recommended to perform this calculation by means of non-linear strain model [8].

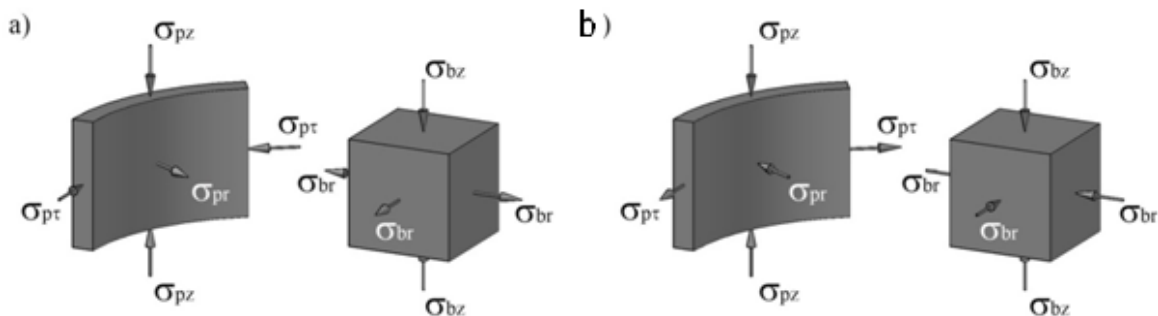


Fig. 1. Stress state if steel shell and concrete core of confined concrete tube column of circular transversal cross section at various loading stages of: a) at $u_p > u_b$; b) at $u_p < u_b$.

EXPERIMENTAL

One of the major advantages of non-linear strain model is unified systematic approach to determination of load-bearing capacity and stress-strain state of confined short and flexible confined concrete tube elements at all stages of their operation under centrally and eccentrically applied short- and long-term loads. Practical implementation of this model is carried out by iterative calculation accounting for non-elastic strains of materials and variation of coefficients of transversal strains of concrete and steel upon increase in stress level.

The calculated dependences in the applied model are based on determining correlations between stresses and strains for concrete and steel. For calculations of conventional reinforced concrete structures, where concrete and steel reinforcement collaborate under uniaxial compression or tension, the strain diagrams of these materials are usually approved according to valid regulations of designing. While processing experimental data it is possible to apply diagrams obtained by experimental results of regular concrete prisms tested for axial compression and steel specimens tested for axial tension.

For confined concrete tube columns the diagrams of strain of concrete core and steel shell collaborating under spatial stress state are unknown prior to calculations. This circumstance hinders application of the strain model. In this relation it is proposed to perform calculations of load-bearing capacity of confined concrete tube elements in two stages.

At the first stage the strain diagrams of concrete core and steel shell are plotted for axial direction of columns by calculations. This is aided with calculation of strength of normal cross section of short (with flexibility of effective cross section $\lambda_{eff} \leq 12$) centrally confined concrete tube column. For instance, in order to exclude the influence of flexibility of column of circular cross section with the diameter d their calculated

length is set equal to $l_0 = (3 \div 4)d$. The remaining geometrical and design parameters of the considered column are set similar to initial data of the current task.

The most important problem upon plotting of strain diagrams of concrete core and steel shell is the determination of coordinates of their parametric points. For confined concrete core it is recommended to apply curvilinear strain diagram (Fig. 2). In this case the major parametric point is the diagram vertex. Maximum stress of axial direction σ_{bz} corresponds to its strength at three-axial compression R_{b3} , and the respective relative strain is denoted as ϵ_{b00} .

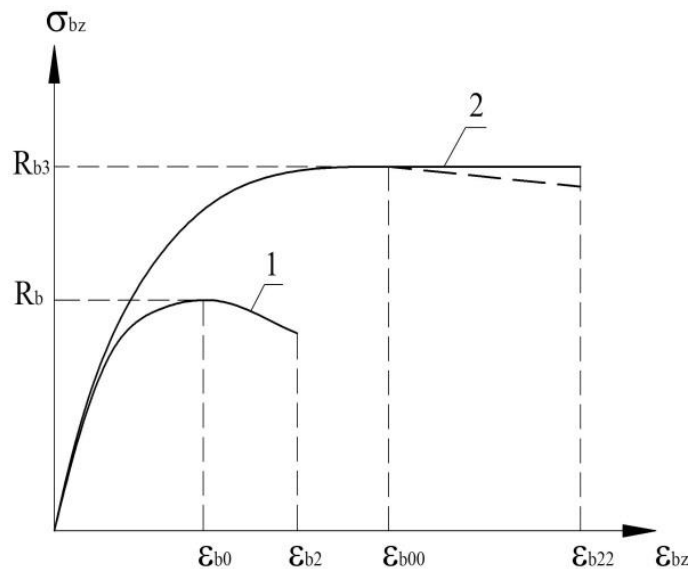


Fig. 2. Diagrams of concrete strain: 1 - concrete work at uniaxial compression; 2 – the same at three-axial compression.

For spatially stressed steel shell the Il'yushin hypothesis of single curve is applied [9]. Herewith, the relation between intensity of current stresses and intensity of relative strain is considered as the Prandtl diagram.

In the course of calculation of strength of normal cross section of the column a portion of steel shell in axial direction can be extended. Aiming at simplification of calculations this portion (with negligible error of strength reserve) can be considered as performing axial tension. The strain diagram for extended area of steel is also considered as two-linear with maximum stresses equal to yield point σ_y .

At the second stage, using known dependences of non-linear strain model, given, for instance, in Regulations SP 63.13330.2012, the load-bearing capacity of eccentrically compressed column is calculated.

RESULTS AND DISCUSSION

Determination of coordinates of diagram vertexes of material strain is a sufficiently complicated problem. Preliminary analysis revealed that the decisive influence on these coordinates is exerted by constantly increasing with loading lateral pressure of steel shell on concrete core. The value of this pressure is unknown. Thus, at first approximate values of the required coordinates are calculated, which are subsequently specified by means of successive approximations.

Let us consider the solution of this problem form centrally confined concrete tube columns of circular transversal cross section.

Comprehensive analysis of published data in [10] evidences that numerous researchers, aiming at determination of strength of spatially stressed concrete core R_{b3} upon uniform lateral confinement by stresses $|\sigma_{br}| < |\sigma_{bz}|$, apply widely known equation:

$$R_{b3} = R_b + k\sigma_{br}, \tag{1}$$

where R_b is the concrete strength upon axial confinement; σ_{br} is the lateral pressure on concrete by steel shell; k is the coefficient of lateral pressure.

The value of coefficient k in this equation is considered to be constant (while determining the dependences in EN 1994-1-1:2004 Eurocod 4 this coefficient was set equal to 4.1; A. A. Gvozdev considered $k = 4.0$). At present it has been proven [11] that the value of k varies and mainly depends on the level of lateral confinement $m = \sigma_{br}/R_{b3}$ and type of concrete.

We discovered [12] that for concrete core of confined concrete tube columns the coefficient k varies in sufficiently wide range (from 2.5 to 7). Since prior to concrete failure the lateral pressure σ_{br} can achieve 15÷20 MPa and higher, even minor inaccuracies in determination of k can lead to significant errors in determination of concrete strength R_{b3} and load-bearing capacity of a designed element.

Frequently applied Mander equation [13] in this case cannot be applied, since it is based on known σ_{br} . However, for confined concrete tube elements the lateral pressure on concrete core in ultimate state depends of geometrical and engineering variables of designed structure.

In order to solve this problem let us assume theoretically proven statement [14] for determination of the coefficient of lateral pressure k for neatly all stone materials by relative level of their lateral confinement $m = \sigma_{br}/R_{b3}$

$$k = \frac{1 - a - am}{b + (f - b)m}, \tag{2}$$

where a, b are the coefficients of material determined experimentally, herewith, $a = 0.5b$; f is the parameter determining the strength surface pattern in the vicinity of all-round compression (for dense concrete the strength surface is discontinued and set equal to $f = 1$).

The use of Eq. (2) makes it possible to consider for the dependence of strength of spatially compressed concrete not only on lateral pressure but also on structural peculiarities of concrete itself, which is important for obtaining of more reliable solutions. Analysis of this equation demonstrates that at high levels of lateral confinement (at $m \rightarrow 1$) the coefficient of lateral pressure $k \rightarrow 1$. In such cases the concrete failure according to the Coulomb law will be of shear pattern. With the aforementioned coefficients $k = 2.5\div 7$ the failure of spatially confined concrete results from combinations of breakage and shear, which agrees with experimental results [15].

Substituting Eq. (2) into Eq. (1) and after some transformations we obtain as follows:

$$R_{b3} = R_b \left[1 + \left(0,25\bar{\sigma} + \frac{\bar{\sigma} - 2}{4} + \sqrt{\left(\frac{\bar{\sigma} - 2}{4} \right)^2 + \frac{\bar{\sigma}}{b}} \right) \right], \tag{3}$$

where $\bar{\sigma}$ is the relative value of lateral pressure of steel shell on concrete core in ultimate state $\bar{\sigma} = \sigma_{br}/R_b$.

In loaded confined concrete tube column the lateral pressure influences significantly not only on the strength of concrete core but also on the stress of axial direction in steel shell σ_{pz} . In ultimate stage of centrally confined column we express this stress also by $\bar{\sigma}$. We apply the Hencky--Mises plasticity condition for this aim.

Since in practice the columns are usually fabricated of thin-wall tubes (with the ratio of wall thickness δ to diameter of cross section $\delta/d \leq 0,025$), the steel shell of centrally confined concrete tube structure can be approximately considered as operating under the conditions of compression--tension plane stress state. For such stressed state the following equation is obtained from the yield condition:

$$\sigma_{pz} = \sqrt{\sigma_y^2 - 0,75\sigma_{p\tau}^2} - 0,5\sigma_{p\tau}, \quad (4)$$

where σ_y is the yield point of steel of external shell; $\sigma_{p\tau}$ is the stress of tangential direction of steel shell in ultimate state (see Fig. 1,b).

Thickness-averaged tangential stresses in steel shell for thin-wall tubes can be sufficiently accurately expressed by lateral pressure using the following equation:

$$\sigma_{p\tau} = -2\sigma_{br} \frac{A}{A_p}, \quad (5)$$

where A and A_p are the surface areas of cross sections of concrete core and steel shell, respectively.

Then, let us apply the structural coefficient of tube confined concrete ρ , calculated as follows:

$$\rho = \frac{\sigma_y A_p}{R_b A}. \quad (6)$$

Taking into account Eqs. (4)-(6) the stress in steel shell σ_{pz} can be determined as follows:

$$\sigma_{pz} = \sqrt{\rho^2 R_b^2 \frac{A^2}{A_p^2} - 3\sigma_{br}^2 \frac{A^2}{A_p^2}} - \sigma_{br} \frac{A}{A_p}. \quad (7)$$

Using lateral pressure $\bar{\sigma}$ the equation for determination of σ_{pz} is written as follows:

$$\sigma_{pz} = R_b \left(\sqrt{\rho^2 - 3\bar{\sigma}^2} - \bar{\sigma} \right) \frac{A}{A_p}. \quad (8)$$

The strength of short centrally confined concrete tube column can be determined as the sum of forces acting on concrete core N_b and steel shell N_p as follows:

$$N = N_b + N_p. \quad (9)$$

Longitudinal force acting on concrete with accounting for Eq. (3) will be as follows:

$$N_b = R_b A \left[1 + \left(0,25\bar{\sigma} + \frac{\bar{\sigma} - 2}{4} + \sqrt{\left(\frac{\bar{\sigma} - 2}{4} \right)^2 + \frac{\bar{\sigma}}{b}} \right) \right]. \quad (10)$$

Longitudinal compressing force acting on steel shell with accounting for Eq. (8) is as follows:

$$N_p = R_b A \left(\sqrt{\rho^2 - 3\bar{\sigma}^2} - \bar{\sigma} \right). \quad (11)$$

Then the ultimate load is determined by the following equation:

$$N = 0,25R_b A \left(\sqrt{(\bar{\sigma} - 2)^2 + 16\bar{\sigma}/b} - (\bar{\sigma} - 2) + 4\sqrt{\rho^2 - 3\bar{\sigma}^2} \right). \quad (12)$$

It should be noted that at fixed values of geometrical and engineering parameters of columns (R_b, σ_y, A, A_p, b) the cumulative longitudinal force acting on concrete and steel in normal cross section depends only on relative lateral pressure $\bar{\sigma}$. Maximum longitudinal force corresponds to the condition:

$$\frac{dN}{d\bar{\sigma}} = 0. \quad (13)$$

After determination of derivative of Eq. (12) we obtain the following equation:

$$\left(\frac{b(\bar{\sigma} - 2) + 8}{\sqrt{b} \sqrt{b(\bar{\sigma} - 2)^2 + 16\bar{\sigma}}} - \frac{12\bar{\sigma}}{\sqrt{\rho^2 - 3\bar{\sigma}^2}} - 1 \right) = 0. \quad (14)$$

Solving numerically Eq. (14) we obtain the expression for determination of relative lateral pressure of steel shell on concrete core in ultimate state of the column:

$$\bar{\sigma} = 0,48e^{-(a+b)} \rho^{0,8}. \quad (15)$$

With known $\bar{\sigma}$ using Eqs. (3) and (8) we determine the required stresses in concrete core and steel shell.

The initial equation for calculation of relative strain of compression of concrete core ε_{b00} in the vertex of strain diagram is as follows:

$$\varepsilon_{b00} = \frac{R_{b3}}{V_{b3} E_b}, \quad (16)$$

where V_{b3} is the coefficient of concrete elasticity in the vertex of strain diagram; E_b is the initial elasticity modulus.

Using Eqs. (1), (3) and interrelation between ultimate relative strains and strength of uniaxial confined concrete we transform Eq. (16) as follows:

$$\varepsilon_{b00} = \varepsilon_{b0} \frac{V_{bu} R_{b3}}{V_{b3} R_b}, \quad (17)$$

where ε_{b0} is the ultimate value of relative strain of concrete at axial compression; V_{bu} is the coefficient of elasticity in the vertex of diagram of uniaxial confined concrete.

In the mechanics of reinforced concrete the ratio of coefficients of elasticity in the vertices of diagrams of uniaxial and spatial confined concrete is defined in reverse proportion with respect to the ratio of respective stresses:

$$\frac{V_{bu}}{V_{b3}} = \left(\frac{R_{b3}}{R_b} \right)^n. \quad (18)$$

In [16] the first approximation is as follows $n \approx 1$. With such assumption Eq. (17) acquires the form proposed in EN 1992-1-1. Eurocode 2: Design of Concrete Structures. However, statistic analysis of numerous experimental data obtained in our studies of confined concrete tube columns and those of Japanese

researchers [15] demonstrated that the ratio v_{bu}/v_{b3} is mainly determined by the structural coefficient of tube confined concrete and can be determined by the following expression:

$$\frac{v_{bu}}{v_{b3}} = 1 + 0,5\rho^{0,5}. \quad (19)$$

Taking this into account, the relative compressive strain of concrete core ε_{b00} in the vertex of diagram $\sigma_{bz} - \varepsilon_{bz}$ for confined concrete tube column of circular cross section can be calculated as follows:

$$\varepsilon_{b00} = \varepsilon_{b0} \left(1 + 0,5\rho^{0,5} \right) \frac{R_{b3}}{R_b}. \quad (20)$$

According to Regulations SP 63.13330.2012 the value of ε_{b0} under short term axial compression can be set equal to 2‰ irrespectively of the concrete grade. For more accurate calculations using the strain model of reinforced concrete and applying curvilinear diagram $\sigma_b - \varepsilon_b$ the regulations propose to determine ε_{b0} by empirical equation depending on the grade upon compression B and initial elasticity modulus of concrete. As applied to heavy concrete, this equation is as follows:

$$\varepsilon_{b0} = \frac{B}{E_b} \frac{1 + 0.75B/60 + 0./B}{0.12 + B/60 + 0.2/B}. \quad (21)$$

However, comparison of calculation results according to the proposed equation with the data given in EN 1992-1-1. Eurocode 2 evidences the existence of significant discrepancies between them. Statistic processing revealed that the variation coefficient of error vector is 9.2%, and the maximum deviation reaches 21%.

Aiming at better agreement with the European norms, the following equation is proposed:

$$\varepsilon_{b0} = (1.2 + 0.16\sqrt{B})/1000. \quad (22)$$

Comparison of the calculation results according to Eq. (22) with the data of European norms demonstrates [17] that the variation coefficient of error vector in this case was 1.2%, and the maximum deviation - 2%.

The ultimate relative strain of steel shell contraction in confined area with accounting for compatibility of strain with concrete is set to $\varepsilon_{p2} = \varepsilon_{b00}$. The ultimate relative strain of steel shell in extended area is set to $\varepsilon'_{p2} = 0.025$.

Therefore, in order to determine the coordinates of required parametric points of strain diagrams of concrete core $\sigma_{bz} - \varepsilon_{bz}$ and steel shell $\sigma_{pz} - \varepsilon_{pz}$ the theoretical relations have been obtained.

It is not required to plot descending segment of the strain diagram of concrete core (see Fig. 2). At relative strains $\varepsilon_{pz} = 0.0012 \div 0.0016$ (which is significantly lower than ε_{b00}) the steel shell transfers into yield condition. In this condition any further increase in load leads to increase in the stresses σ_{pz} and decrease in the stresses σ_{pz} , thus, in the load N_p . That is, upon axial compression the ultimate deformability of specimen cannot exceed ε_{b00} . Upon eccentric compression, even with reinforcing of the core with high-tensile reinforcement, the strains of extreme compressed fiber of normal cross section of confined concrete tube column are not usually higher than ε_{b00} . As a consequence of increased deformability of concrete (the practical range of relative strains of concrete is $\varepsilon_{b00} = 0.004 \div 0.008$). In this regard the descending branch of strain can be conventionally replaced with horizontal segment.

The first stage of calculation of load-bearing capacity

Practical implementation of the first stage is in incremental increase $\Delta\varepsilon_{pz} = \Delta\varepsilon_{bz}$ of relative axial strains of contraction of steel shell and concrete core of the column. At each step the required parameters of the strain diagrams are calculated by combined solution of the set of equations which present physical interrelations between stresses and strains in the form of the generalized Hooke's law. Physical non-linearity is accounted for by the use of variable coefficients of elasticity of concrete and steel. The essential aspect of the proposed procedures is also accounting for the variable coefficients of transversal strain of materials. Herewith, concrete is considered as transversally isotropic material, and steel shell as isotropic one.

The interrelation between strains and stresses for any point of external steel shell is expressed by the set of equations:

$$\begin{Bmatrix} \varepsilon_{pz} \\ \varepsilon_{p\tau} \\ \varepsilon_{pr} \end{Bmatrix} = \frac{1}{\nu_p E_{s,p}} \times \begin{bmatrix} 1 & -\nu_p & -\nu_p \\ -\nu_p & 1 & -\nu_p \\ -\nu_p & -\nu_p & 1 \end{bmatrix} \times \begin{Bmatrix} \sigma_{pz} \\ \sigma_{p\tau} \\ \sigma_{pr} \end{Bmatrix}. \tag{23}$$

In Eq. (23) $\sigma_{pz}, \sigma_{p\tau}, \sigma_{pr}$ are the normal (major) stresses in axial, tangential and radial directions; $\varepsilon_{pz}, \varepsilon_{p\tau}, \varepsilon_{pr}$ are the relative strains of steel shell along the respective directions; $E_{s,p}$ is the initial elasticity modulus of steel; ν_p is the elasticity coefficient of steel; ν_p is the coefficient of transversal strain of tube steel.

On the basis of experimental data (direction of the Chernov-Lüders lines on the surface of steel tube) the stresses and strains in this case are considered as acting along the main sites, that is, tangential stresses and shear strains are zero.

With bilinear strain diagram in the elastic stage the elasticity coefficient $\nu_p = 1$, and in the plastic stage it is calculated as follows:

$$\nu_p = \frac{\sigma_y}{\varepsilon_{pi}}. \tag{24}$$

In the proposed procedure of non-linear calculation, aiming at elimination of simultaneous use of two iterations, it is recommended to account for non-elastic properties of steel by means of one variable parameter: the elasticity coefficient ν_p , on the basis of which the coefficient of transversal strain ν_p is determined.

It is assumed that the coefficient of transversal strain ν_p varies proportionally to the elasticity coefficient: from initial ν_o (in elastic stage) up to ultimate value ν_{pu} at $\nu_p = \nu_{pu}$ (at the end of yield plateau). Preliminary calculations according to [18] demonstrated that ν_{pu} can be set equal to 0.48 with the accuracy, sufficient for practical calculations. Then, the equation for determination of ν_p can be written as follows:

$$\nu_p = 0.48 - (0.48 - \nu_o) \left(\frac{\nu_p - \nu_{pu}}{\nu_{po} - \nu_{pu}} \right). \tag{25}$$

The values of ν_p determined by Eq. (25) are in good agreement with the coefficients of transversal strains calculated by Regulations SNiP 2.05.06-85*, as well as with the data in [19].

The set of equations describing the interrelation between stresses and strains for any point of transversally isotropic concrete core in elastic and elastic-plastic stages is as follows:

$$\begin{Bmatrix} \varepsilon_{bz} \\ \varepsilon_{br} \end{Bmatrix} = \frac{1}{E_b} \times \begin{bmatrix} \nu_{bz}^{-1} & -2\nu_{zr}\nu_{bi}^{-1} \\ -\nu_{zr}\nu_{bi}^{-1} & (\nu_{br}^{-1} - \nu_{rr}\nu_{bi}^{-1}) \end{bmatrix} \times \begin{Bmatrix} \sigma_{bz} \\ \sigma_{br} \end{Bmatrix}, \quad (26)$$

where E_b is the initial elasticity modulus of concrete.

The value of E_b for practical calculations can be readily calculated according to one of the proposed below equations [16]:

- at known concrete grade B

$$E_b = 55.25 - \frac{122}{\sqrt{B}}; \quad (27)$$

- at known actual (prism) strength of concrete R_b , MPa

$$E_b = 56 - \frac{122}{\sqrt{R_b}}. \quad (28)$$

Equations (27), (28) make it possible to determine the value of initial elasticity modulus of concrete in GPa for concrete grades from B12.5 to B100.

Non-elastic properties of spatially confined concrete core are considered by the use of variable elasticity coefficients ν_{bj} ($j = z, r, i$) and transversal strain ν_{zr} , ν_{rr} of concrete in Eq. (26). Herewith, the elasticity coefficient with the index $j = i$ is determined as a function of stress intensity and strain intensity in concrete.

Since the increase in the stresses σ_{bz} and σ_{br} upon loading occurs not proportionally (complex mode of loading), the elasticity coefficients for different directions are different. In order to calculate them it is possible to apply any known equations providing for sufficient accuracy of estimation of stress-strain state of structure, for instance, proposed in [20].

In order to determine current values of the coefficients of transversal strains ν_{jr} ($j = z, r$) the following equation is proposed:

$$\nu_{jr} = \nu_{jru} - (\nu_{jru} - \nu_b) \left(\frac{\nu_{bi} - \nu_{biu}}{\nu_{oj} - \nu_{biu}} \right)^{0,5}, \quad (29)$$

where $\nu_b = 0.18 \div 0.25$ is the Poisson coefficient for concrete (if accurate data are unavailable it is recommended to assume $\nu_b = 0.2$); ν_{jru} is the ultimate value of coefficient of transversal deformation ν_{jr} , determined according to the recommendations in [21].

The convergence of ν_{jr} , calculated by Eq. (29), with the experimental data in [22], [23] is satisfactory.

On the basis of solution of the sets of equations (23) and (26), equations of strain compatibility of concrete and steel in axial and transversal directions, with accounting for conditions of element equilibrium we obtain the equation for determination of lateral pressure at each step of increments of relative axial strains:

$$\sigma_{br} = \frac{\left(\nu_p - \beta_r \nu_{zr} \frac{\nu_{bz}}{\nu_{bi}} \right) \varepsilon_{bz}}{K_p + K_b}, \quad (30)$$

where K_p and K_b are calculated as follows:

$$K_p = \frac{0,5v_p}{v_p E_{s,p}} \left[v_p \left(\frac{d}{\delta} - 1 \right) - \left(\frac{d}{\delta} + 1 \right) \right]; \tag{31}$$

$$K_b = \frac{\beta_r}{v_{bi} E_b} \left(\frac{2v_{zr}^2 v_{bz}}{v_{bi}} + v_{rr} - \frac{v_{bi}}{v_{br}} \right), \tag{32}$$

where the coefficient β_r is the ratio of external diameter of steel tube to the internal one.

With known σ_{br} on the basis of solution of Eqs. (23) and (26) the stresses σ_{bz}, σ_{pz} are calculated and respective strain diagrams are plotted. The calculation is carried out until the stress σ_{bz} achieves preliminary determined strength of concrete R_{b3} at relative lateral pressure calculated by Eq. (15).

Then, the correspondence of the last value of relative strain ϵ_{bz} to preliminary calculated strain in the vertex of concrete deformation diagram ϵ_{b00} is verified. If misalignment is higher than the pre-set value $|\epsilon_{bz} - \epsilon_{b00}| > \Delta_\epsilon$, the value of ϵ_{b00} is adjusted and all calculations are repeated.

The second stage of calculation

At the second stage the column load-bearing capacity is determined. The calculation flowchart of normal cross section of eccentrically confined concrete tube element is illustrated in Fig. 3.

Transfer from the strain diagram in concrete, steel shell and rod reinforcement (if any) to generalized internal forces is performed using the procedure of numerical integration of stresses over normal cross section. This is aided by conventional subdivision of normal cross section into minor segments with the surface areas of concrete A_{bi} , steel shell A_{pk} , and rod reinforcement A_{sj} .

The origin of coordinates should be preferentially positioned into the geometrical center of transversal cross section of steel shell. Upon bending of the column in two planes it is recommended to split the cross section in polar coordinates.

The strain diagram is assumed corresponding to the Bernoulli hypothesis. Depending on the values of respective strains, using the calculation results of the previous stage, the stresses are determined in each segment of concrete core and steel shell. Then, in the limits of each segment the stresses are assumed to be uniformly distributed (averaged).

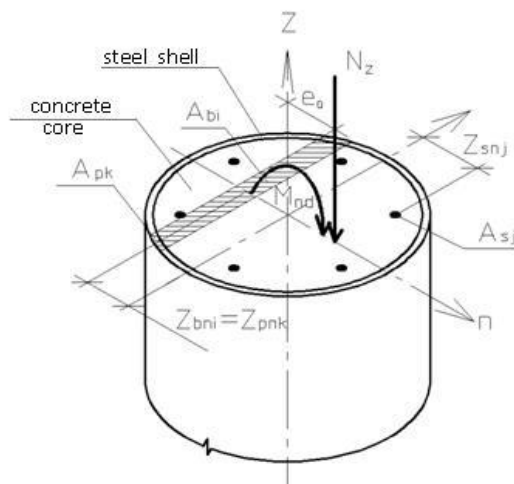


Fig. 3. Flowchart of calculation of normal cross section of confined concrete tube structure.

The essence of calculations is reduced to plotting of strain diagram of normal cross section corresponding to the stage of ultimate equilibrium of the calculated element. Such diagram is plotted as follows.

Sufficiently low value of relative strain of uniform structure compression is set. Then, the relative strain diagram is step-by-step transformed from the rectangular shape to the required trapezium shape conforming the equilibrium condition of eccentrically confined element. Corresponding longitudinal force is determined. Then, the strain is incrementally increased from the side of the most confined fiber and the overall procedure is repeated. Breaking load corresponds to the maximum value of thus determined longitudinal forces.

CONCLUSIONS

The developed procedure of calculation of load-bearing capacity of eccentrically confined composite structure, confined concrete tube column, is based on non-linear strain model, it takes into account spatial stress-strain state and collaboration of concrete core and steel shell. It enables reliable estimation of stress-strain state of columns at any level of loading, which is advantageous for practical designing. The procedure can be applied for confined concrete tube columns, manufactured by various technologies on the basis of concretes of various strength and types.

In order to implement the proposed procedure a dedicated software was developed, on the basis of which theoretical breaking loads N_u were calculated for confined concrete tube elements previously tested for central and eccentric compression. The initial data for calculations were taken from published materials of the most well-known USSR and Russian scientific schools in the field of study of confined concrete tube columns guided by L. I. Storozhenko [22], I. G. Lyudkovskii, [20], V. I. Rimshin, and A. L. Krishan [21-29], experiments by Japanese researchers [28], as well as own tests by A. L. Krishan [29].

With the illustrative purposes and reliability of comparison of theory and practice the confined concrete tube elements were selected with various geometric and engineering parameters:

- outer diameter of external steel shell from 93 mm to 1020 mm;
- wall thickness of external steel shell from 0.8 mm to 13.3 mm;
- yield point of shell steel from 240 MPa to 440 MPa;
- concrete prism strength from 11.7 MPa to 104 MPa;
- relative eccentricities of compressing load from 0 to 1.

Theoretical and experimental values of braking loads were compared for 110 centrally confined and 72 eccentrically confined column specimens. The results evidence their satisfactory agreement: for centrally confined specimens the highest discrepancies were +17...-9% at the coefficient of variation of error vector $V_\delta = 0.04$; for eccentrically confined specimens the highest discrepancies were +18...-14% at $V_\delta = 0.08$. The calculations according to the most accurate procedure of European norm (EN 1994-1-1) give maximum discrepancies +42...-19% at axial compression $V_\delta = 0.13$ and at eccentric compression the maximum discrepancies are significantly higher.

The advantages of the proposed procedure are obvious.

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