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Bearing Capacity of Concrete-Filled Steel Tube Column Sections under Long-Term Loading

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***Abstract*: This article presents the design methods for concrete filled circular columns subjected to long-term axial compression and bending. . There are two approaches: stress-based and strain-based for formulations. Both approaches are specified in Russian Code, SP 266.1325800.2016, and in European Code, EN 1994-1-1:2004. A numerical example shows the procedures to calculate the strength of a given column according to two different Codes, the influence of parameters such as steel contribution ratio, relative slenderness to the results in two methods are consider.**

***Keywords*: Concrete-filled steel, nonlinear deformation, long-term loading, stress-based, strain-based**

I. INTRODUCTION

Experimental work on the behaviour of eccentric compressed concrete-filled steel tube columns (CFST) was carried out with the compression force N set their axial distance a segment e_0. Experiment with different eccentricities and during each experiment, the axial force of N is increased from zero to the value that will cause the column to fail.

Experimental results indicate, with eccentricity e_0 is small, center the entire section is compressed and the damage starts from the concrete in the more compressed fibers. With eccentricity e_0 is great, a part of section is subjected to compression, the other is tensile, the tensile concrete may be cracked, the destructive can start from compressed concrete or reinforcement, steel tube sections in tensile.

There are two approaches for formulations: stress-based and strain-based.

On the stress-based approach, structure will fail when the stress in materials reached and exceed the specific strengths of the materials. In this way, the strength of cross section is calculated based on the specific strength of compression concrete, reinforcement and steel tube, but not their strain values. This approach is specified in SP 266.1325800.2016[1], in terms of limited internal force method and in EN 1994-1-1:2004 [2], in terms of simplified method of design.

On the strain-based approach, structure will fail if the strain in outer most fibers of the cross section reach their ultimate values. Strength of cross section is calculated according to SP 266.1325800.2016 [1] in terms of nonlinear deformation model method, (and to EN 1994-1-1:2004) [2] in terms of general method.

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II. THEORY

This section will present the formulate for calculating the strength of cross section according to stress-based approach, under limit internal force method and simplified method of design, and strain-based approach, under nonlinear deformation model method or general method of design.

A. Resistance of cross section according to stress-based approach

1) According to Russian standard, in terms of limit internal force method

In the state of multi axial stress, the design strength of concrete in tube is increased, while that of steel is reduced. The design strengths of steel and concrete are calculated respectively in Eq. (1) and (2) as follow

$$R_{pc} = R_y - \frac{1}{4}R_y\left(1 - \frac{7{,}5e}{D_p - 2t_p}\right) \quad (1)$$

$$R_{bp} = R_b + \Delta R_b\left(1 - \frac{7{,}5e}{D_p - 2t_p}\right) \quad (2)$$

where $1 - \frac{7{,}5e}{D_p - 2t_p} \geq 0$, D_p is outer diameter of tube; t_p is wall thickness, R_y is design value yield strength of steel tube; e is the eccentricity of the vertical compression point with respect to the center of the cross-sectional area, including the eccentricity of eccentricity and the effect of vertical bending,

$$\Delta R_b = R_b\left(2 + 2{,}52e^{\frac{-1}{c}(R_pA_p + R_bA_b)}\right)\frac{t_p}{D_p - 2t_p}.\frac{R_p}{R_b}.$$

The value of the constant c should be taken as 25 MN when measured by MPa. Equation (1) and (2) are valid for the ratio t_p/D_p is between 0,0064 and 0,046. With different ratios, design value of the cylinder compressive strength of concrete in tube should be determined based on the test results.

Resistance of members in uniaxial bending, should be satisfied:

$$N.e \leq \frac{2}{3}r_b^3R_{bp}\sin^3\alpha + \frac{1}{\pi}A_sr_s\left(R_s + R_{sc}\right)\sin\alpha + \quad (3)$$
$$+\frac{1}{\pi}A_pr_p\left(R_y + R_{pc}\right)\sin\alpha$$

where N is axial force by external force; e is eeccentricity of axial load with the consideration of, additional eccentricity, e_a, and effect of buckling by the slenderness of the column, e = $e_0.\eta$. The moment magnification factor η is given in [1], and eccentricity e_0 with an addition is given in [1].

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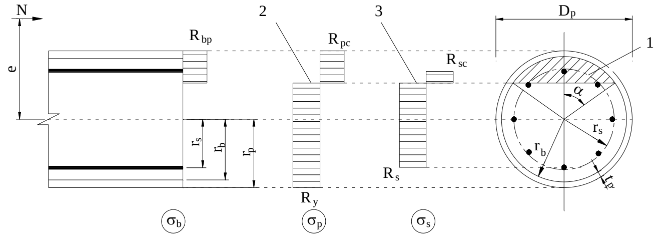

Figure 1. Schematics of internal force and stress diagram in section perpendicular to the vertical axis
of concrete-filled steel tube section in uniaxial compression (1 - compression zone of concrete; 2 - stress in steel tube; 3 - stress in the relative reinforcement)

Angle α (radian), in all cases $\alpha \le \pi$, found from the equation:

$$r_b^2\left(\alpha-\frac{1}{2}\sin 2\alpha\right)R_{bp}+\frac{\alpha}{\pi}A_sR_{sc}-\left(1-\frac{\alpha}{\pi}\right)A_sR_s+ \\ +\frac{\alpha}{\pi}A_pR_{pc}-\left(1-\frac{\alpha}{\pi}\right)A_pR_y=N \quad (4)$$

The value of the coefficient η based on first-order linear elastic analysis, is given by:

$$\eta=\frac{1}{1-\dfrac{N}{N_{cr}}} \quad (5)$$

where N_{cr} is the elastic critical normal force are given by,

$$N_{cr}=\frac{\pi^2 D}{L_0^2} \quad (6)$$

where D is flexural stiffness of concrete-filled steel tube section element in limited state by bearing capacity, determine suitable for deformation calculation, can be determined by the formula [1]:

$$D=\min\left\{\begin{array}{l} k_bE_{b1}I+k_s\left(E_sI_s+E_pI_p\right); \\ k_bE_{b1}I+k_sE_sI_s+\dfrac{L_0^2}{\pi^2}R_{pc}A_p \end{array}\right\} \quad (7)$$

for E_s and E_p are design value of modulus of elasticity of reinforcing steel and tube; E_{b1} is design value of modulus of elasticity mention to long-term loading, given in [1]; I, I_s and I_p are second moment of area of the un-cracked concrete section, steel reinforcement (equivalent scarf section) and tube for the bending plane being considered; L_0 is effective length of elements, given in [3].

2) (2) According to European standard, in terms of simplified method of design [2]

Concrete-filled steel tube columns can be applied simplified method of design when the relative slenderness $\bar{\lambda}$ should fulfil the following conditions:

$$\bar{\lambda}\le 2,0 \quad (8)$$

Relative slenderness $\bar{\lambda}$ for the plane of bending being considered is given by:

$$\bar{\lambda}=\sqrt{\frac{N_{pl,Rk}}{N_{cr,ef1}}} \quad (9)$$

where $N_{pl,Rk}$ is the characteristic value of the plastic resistance to compression given by [2], $N_{pl,Rk} = A_pf_y + A_bf_{ck} + A_sf_{sk}$. Thus f_y, f_{ck}, f_{sk} is nominal value of the yield strength of tube, concrete and structural steel section.

For the determination of the relative slenderness $\bar{\lambda}$ and the elastic critical force $N_{cr,ef1}$, the characteristic value of the effective flexural stiffness $(EI)_{eff,1}$ of a cross section of a composite column should be calculated from:

$$(EI)_{eff,1}=E_pI_p+E_sI_s+K_eE_{cm}I \quad (10)$$

where K_e is a correction factor that should be taken as 0,6; E_{cm} is Secant modulus of elasticity of concrete. Account should be taken to the influence of long-term effects on the effective elastic flexural stiffness. The modulus of elasticity of concrete E_{cm} should be reduced to the value $E_{c,eff}$ accordance with the following expression:

$$E_{c,eff}=E_{cm}\frac{1}{1+\left(\dfrac{N_{g,Ed}}{N}\right)\varphi_t} \quad (11)$$

where φ_t is the creep coefficient [2]; N is the total design normal force; $N_{g,Ed}$ is the part of this nominal force that is permanent.

For concrete confined tubes of circular cross-section, account may be taken of increase in strength of concrete caused by confinement provided that the relative slenderness $\bar{\lambda}$ does not exceed 0,5 and $e/D_p< 0,1$, where e is the eccentricity of loading given by M_{ed}/N_{Ed} and D_p is the external diameter of the column. Design value of the cylinder compressive strength of concrete in tube f_{cc} and design value of the yield strength of structural tube concrete filled f_{yc} are given by:

$$f_{cc}=f_{cd}\left(1+\eta_c\frac{t_p}{D_p}\frac{f_y}{f_{ck}}\right),\ f_{yc}=\eta_af_{yd} \quad (12)$$

where f_{cd} and f_{yd} are design value of the cylinder compressive strength of concrete and design value of the yield strength of structural tube; η_a and η_c are given by:

$$\eta_a=\eta_{ao}+(1-\eta_{ao})(10e/D_p);\quad \eta_c=\eta_{co}\left(1-10e/D_p\right), \quad (13)$$

where η_{ao} and η_{co} are given by:

$$\eta_{ao}=0,25\left(3+2\bar{\lambda}\right)\le 1,0;\quad \eta_{co}=4,9-18,5\bar{\lambda}+17\bar{\lambda}^2\ge 0 \quad (14)$$

For $e/D_p> 0,1$, $\eta_a = 1$ and $\eta_c = 0$.

For member verification, analysis should be based on second-order linear elastic analysis. Within the column length, second-order effects may be allowed for by multiplying the greatest first-order design bending moment M_{Ed} by a factor k given by:

$$k=\frac{\beta}{1-N/N_{cr,ef2}}\ge 1,0, \quad (15)$$

where $N_{cr,ef2}$ is the critical nominal force for the relevant axis and corresponding to the effective flexural stiffness for effective flexural stiffness $(EI)_{eff,2}$, with the effective length taken as the column length; β is an equivalent moment factor given in Table 6.4 [2], should be taken as β = 1. Design value of effective flexural stiffness $(EI)_{eff,2}$ should be determined from the following expression:

$$(EI)_{eff,2}=K_0\left(E_pI_p+E_sI_s+K_{e,2}E_{cm}I\right) \quad (16)$$

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where $K_{e,2}$ is a correction factor which should be taken as 0,5; K_0 is a calibration factor which should be taken as 0,9. Long-term effects should be taken into account in accordance with 6.7.3.3 (4) [2].

The following expression based on the interaction curve determined according to Eurocode 4, should be satisfied:

$$\frac{M_{Ed}}{M_{pl,N,Rd}} \le \alpha_M, \tag{17}$$

where M_{Ed} is the greatest of the end moments and the maximum bending moment within the column length, including imperfections and second order effects if necessary: M_{Ed} = k.N.e_0; $M_{pl,N,Rd}$ is the plastic bending resistance taking into account the nominal force N, given by interaction curve in Eurocode 4; α_M should be taken as 0,9 and for steel grades S420 and S460 as 0,8.

The construction of interactive curves is quite complex, below the proposed direct calculation of $M_{pl,N,Rd}$ by the expression:

$$M_{pl,N,Rd} = \frac{2}{3} r_b^3 f_{cc} \sin^3\alpha + \frac{1}{\pi} A_s r_s 2 f_{sd} \sin\alpha + \frac{1}{\pi} A_p r_p \left(f_{yc} + f_{yd}\right) \sin\alpha \tag{18}$$

where α is root of the equation:

$$r_b^2\left(\alpha - \frac{1}{2}\sin 2\alpha\right) f_{cc} - \left(1 - \frac{2\alpha}{\pi}\right) A_s f_{sd} + \frac{\alpha}{\pi} A_p f_{yc} - \left(1 - \frac{\alpha}{\pi}\right) A_p f_{yd} = N. \tag{19}$$

B. Resistance of cross section according to strain-based approach

1) According to Russian standard, in terms of nonlinear deformation model [1]

The transition from stress diagramming concrete and steel to general internal force determined by numerous differential stress function on the cross section. Stress in the counterclaims considered ddistributed evenly (get averages).

When calculating cross section of concrete-filled steel tube columns under eccentric compression, have the following relations:

- Equilibrium equations between external and internal forces in cross section:

$$M_x = N.e = \sum_i \sigma_{bi}.A_{bi}.Y_{bi} + \sum_j \sigma_{sj}.A_{sj}.Y_{sj} + \sum_k \sigma_{pk}.A_{pk}.Y_{pk}, \quad N = \sum_i \sigma_{bi}.A_{bi} + \sum_j \sigma_{sj}.A_{sj} + \sum_k \sigma_{pk}.A_{pk}, \tag{20}$$

- Equations determined distribution strain according to cross section element:

$$\varepsilon_{bi} = \varepsilon_0 + \psi_y.Y_{bi};\ \varepsilon_{sj} = \varepsilon_0 + \psi_y.Y_{sj};\ \varepsilon_{pk} = \varepsilon_0 + \psi_y. \tag{21}$$

Relationship between relative stress and strain of concrete, reinforcement and tube steel take it bi-linear, in the reinforced concrete and steel structures standard respectively [3, 4] (Fig 2).

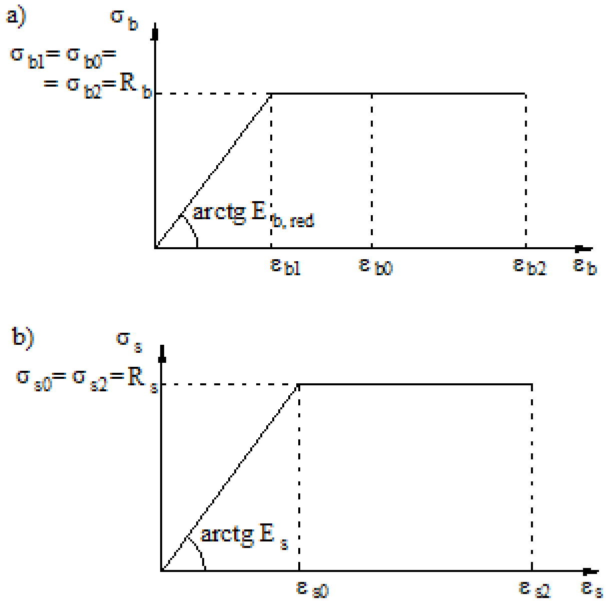

Figure 2.Stress-strain diagram of the concrete (a) reinforcement or tube (b)

By integral method, identify internal forces in cross section:

For concrete, by ignoring stress in the tension zone of the concrete, so internal force in concrete is:

$$M_b = \sum_i \sigma_{bi}.A_{bi}.Y_{bi} = \int_0^{\alpha_1} R_{bp}\, y dA_b + \int_{\alpha_1}^{\pi} E_{bred}\varepsilon_b\, y dA_b = R_{bp}\int_0^{\alpha_1} y dA_b + E_{bred}\int_{\alpha_1}^{\alpha_2}\left(\varepsilon_0 + \psi_y.y\right) y dA_b \tag{22}$$

where α_1 is angle corresponding extreme concrete fiber in crack;

$$\varepsilon_1 = \varepsilon_0 + \psi_y.Y_{b1} = \varepsilon_0 + \psi_y.r_b.\cos\alpha_1 = \varepsilon_{b1,red}, \tag{23}$$

α_2 is angle corresponding extreme concrete fiber un-deformation (at the neutral axis):

$$\varepsilon_2 = \varepsilon_0 + \psi_y.Y_{b2} = \varepsilon_0 + \psi_y.r_b.\cos\alpha_2 = 0, \tag{24}$$

After some manipulations, expression for moment is:

$$M_b = R_{bp} r_b^3\left(\frac{\sin\alpha_1}{2} - \frac{\sin 3\alpha_1}{6}\right) + E_{bred} r_b^3\left(\frac{\sin\alpha_2 - \sin\alpha_1}{2} - \frac{\sin 3\alpha_2 - \sin 3\alpha_1}{6}\right)\varepsilon_0 + E_{bred}\frac{r_b^4}{4}\left(\alpha_2 - \alpha_1 - \frac{\sin 4\alpha_2 - \sin 4\alpha_1}{4}\right)\psi_y \tag{25}$$

The same, expression for vertical force is:

$$N_b = R_{bp} r_b^2\left(\alpha_1 - \frac{\sin 2\alpha_1}{2}\right) + \tag{26}$$

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$$+E_{bred}r_b^2\left(\alpha_2-\alpha_1-\frac{\sin 2\alpha_2-\sin 2\alpha_1}{2}\right)\varepsilon_0+$$
$$+E_{bred}r_b^3\left(\frac{\sin\alpha_2-\sin\alpha_1}{2}-\frac{\sin 3\alpha_2-\sin 3\alpha_1}{6}\right)\psi_y.$$

Setting the following symbols:

$$A_1=\alpha_2-\alpha_1-(\sin 2\alpha_2-\sin 2\alpha_1)/2;$$
$$A_2=(\sin\alpha_2-\sin\alpha_1)/2-(\sin 3\alpha_2-\sin 3\alpha_1)/6$$
$$A_3=\alpha_1-(\sin 2\alpha_1)/2;$$
$$A_4=\alpha_2-\alpha_1-(\sin 4\alpha_2-\sin 4\alpha_1)/4;$$
$$A_5=(\sin\alpha_1)/2-(\sin 3\alpha_1)/6;$$
$$N_1=R_{bp}r_b^2A_3;$$
$$B_1=E_{bred}r_b^2A_1;$$
$$B_2=E_{bred}r_b^3A_2;$$
$$B_3=E_{bred}A_4(r_b^4)/4;$$
$$M_1=R_{bp}r_b^3A_5.$$

With the above symbols, the internal forces in the concrete are:

$$N_b=N_1+B_1\varepsilon_0+B_2\psi_y;\quad M_b=M_1+B_2\varepsilon_0+B_3\psi_y. \tag{27}$$

In the same way, the internal forces in the tube are:

$$M_p=\sum_k\sigma_{pk}.A_{pk}.Y_{pk}=2R_{pc}r_p^2t_p\sin\alpha_3-2E_pr_p^2t_p\sin\alpha_3.\varepsilon_0+$$
$$+E_pr_p^3t_p\left(\pi-\alpha_3-\frac{\sin 2\alpha_3}{2}\right)\psi_y; \tag{28}$$

$$N_p=\sum_k\sigma_{pk}.A_{pk}=\int_0^{\pi}\sigma_p dA_p=2R_{pc}r_pt_p\alpha_3+$$
$$+2E_pr_pt_p(\pi-\alpha_3).\varepsilon_0-2E_pr_p^2t_p\sin\alpha_3.\psi_y. \tag{29}$$

where α_3 is angle corresponding extreme steel tube fiber in first plastic:

$$\varepsilon_{p3}=\varepsilon_0+\psi_y.Y_{p3}=\varepsilon_0+\psi_y.r_p.\cos\alpha_3=\varepsilon_{p0}=R_{pc}/E_p. \tag{30}$$

setting

$$N_2=2R_{pc}r_pt_p\alpha_3,$$
$$B_4=2E_pr_pt_p(\pi-\alpha_3),$$
$$B_5=-2E_pr_p^2t_p\sin\alpha_3,$$
$$M_2=2R_{pc}r_p^2t_p\sin\alpha_3,$$
$$B_6=E_pr_p^3t_p.A_6,$$
$$A_6=\pi-\alpha_3-(\sin 2\alpha_3)/2.$$

then:

$$N_p=N_2+B_4\varepsilon_0+B_5\psi_y;\quad M_p=M_2+B_5\varepsilon_0+B_6\psi_y. \tag{31}$$

To set the formula calculated internal forces in reinforcement, whole reinforcement is converted to a thin tube which radius is r_s equal to the radius of the circle reinforcement layout, and the thickness of the conversion $t_s=A_s/(2\pi r_s)$. Similar to steel tube, internal force in reinforcement will be

$$N_s=N_3+B_7\varepsilon_0+B_8\psi_y;\quad M_s=M_3+B_8\varepsilon_0+B_9\psi_y, \tag{32}$$

where

$$N_3=2R_{sc}r_st_s\alpha_4,$$
$$B_7=2E_sr_st_s(\pi-\alpha_4),$$
$$B_8=-2E_sr_s^2t_s\sin\alpha_4,$$
$$M_3=2R_{sc}r_s^2t_s\sin\alpha_4,$$
$$B_9=E_sr_s^3t_s.A_7,$$
$$A_7=\pi-\alpha_4-\frac{\sin 2\alpha_4}{2};$$

for α_4 is angle corresponding extreme reinforcement fiber in first plastic:

$$\varepsilon_{s4}=\varepsilon_0+\psi_y.r_s.\cos\alpha_4=\varepsilon_{s0}=R_{sc}/E_s. \tag{33}$$

Summary, equilibrium equations between external and internal forces are

$$N=N_b+N_p+N_s=(N_1+N_2+N_3)+$$
$$+(B_1+B_4+B_7)\varepsilon_0+(B_2+B_5+B_8)\psi_y$$
$$M_x=M_b+M_p+M_s=(M_1+M_2+M_3)+$$
$$+(B_2+B_5+B_8)\varepsilon_0+(B_3+B_6+B_9)\psi_y. \tag{34}$$

put:

$$C_{10}=N-(N_1+N_2+N_3)$$
$$C_{11}=B_1+B_4+B_7,\ C_{12}=B_2+B_5+B_8$$
$$C_{22}=B_3+B_6+B_9$$
$$C_{20}=M_x-(M_1+M_2+M_3).$$

So, equation system of the form

$$\begin{cases}C_{11}\varepsilon_0+C_{12}\psi_y=C_{10};\\ C_{12}\varepsilon_0+C_{22}\psi_y=C_{20};\end{cases} \tag{35}$$

Solve equation system (35), receive:

$$\varepsilon_0=\frac{C_{10}C_{22}-C_{12}C_{20}}{C_{11}C_{22}-C_{12}C_{12}};$$
$$\psi_y=\frac{C_{11}C_{20}-C_{12}C_{10}}{C_{11}C_{22}-C_{12}C_{12}}.$$

From there, calculate the characteristic angles

$$\alpha_1=\arccos\left(\frac{\varepsilon_{b1,red}-\varepsilon_0}{\psi_yr_b}\right);$$
$$\alpha_2=\arccos\left(\frac{-\varepsilon_0}{\psi_yr_b}\right);$$
$$\alpha_3=\arccos\left(\frac{\varepsilon_{p0}-\varepsilon_0}{\psi_yr_b}\right);$$

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$$\alpha_4 = \arccos\left(\frac{\varepsilon_{s0} - \varepsilon_0}{\psi_y r_b}\right)$$

and the extreme deformation $\varepsilon_{1b} = \varepsilon_0 - \psi_y r_b$; $\varepsilon_{2b} = \varepsilon_0 + \psi_y r_b$ for concrete core, $\varepsilon_{p,max} = \varepsilon_0 - \psi_y r_p$ for steel tube.

The equation system is solved by the iteration method, initial hypothesis is concrete core in compression fully plastic ($\alpha_1=\alpha_2=\pi$), reinforcement and steel tube are elastic ($\alpha_3=\alpha_4=0$).

Instead will be

$$\varepsilon_0^0 = \frac{N - A_b R_{bp}}{E_p A_p},$$

$$\psi_y^0 = \frac{M_x}{\left(E_p r_p^3 t_p + E_s r_s^3 t_s\right)\pi}.$$

In case $\varepsilon_0^0 < 0$, it mean, concrete core in compression not fully plastic, recalculate ε_0^0 according to the expression $\varepsilon_0^0 = N/(E_p A_p + E_{bred} A_b)$. Iteration will be stopped when the error of ε_{2b} is small enough.

2) According to European standard, in terms of general method of design [2]

General method of design presented in Eurocode 4, is a set of principles such as: should be based on second-order linear elastic analysis, design internal force should be determined base on plastic analysis structure, hypothesis cross section always flat et. So, can use method of design according to nonlinear deformation model method with replace the parameters and coefficient respectively, example:

(1) strength characteristics: $R_{bp} \rightarrow f_{cc}$, $R_{sc} \rightarrow f_{sd}$, $R_s \rightarrow f_{sd}$, $R_{pc} \rightarrow f_{yc}$, $R_y \rightarrow f_{yc}$;

(2) strain characteristics: $\varepsilon_{b1,red} \rightarrow \varepsilon_{c3}$, $\varepsilon_{b2} \rightarrow \varepsilon_{cu3}$, $\varepsilon_{p0} \rightarrow \varepsilon_{a0} = f_{yc}/E_p$.

III. RESULT AND DISCUSION

Illustrated for the calculation, following will examined the bearing capacity of concrete-filled steel tube columns in uniaxial compression, to clarify the effect eccentricity of the vertical force point and the time of impact of the load to bearing capacity of the column.

Initial data of column:

Concrete-filled steel tube columns with outer diameter D_p = 0,63 m, wall thickness t_p = 0,004 m, effective length of columns L_0 = 8 m. The tube is made from hot rolled steel grade C235 (S235 with Eurocode 3) and concrete-filled grade B25 (equivalent grade C20/25 with Eurocode 2). The geometrical characteristics of tube as area and second moment of cross section are A_p = 0,007867 m^2, I_p = 0,000385 m^4; and concrete core are inertia radius, area and second moment of cross section are r_p = 0,311 m, A_p = 0,304 m^2, I_b = 0,00735 m^4.

Initial data of materials:

- According to Russian approach [3, 4], for the concrete grade B25, compressive strength of concrete R_b = 14,5 MPa, initial elastic module E_b = 30 GPa; for the tube grade C235, design value of the yield strength R_y = 225 MPa, elastic module E_p = 206 GPa.

- According to Eurocode approach [5, 6], for the concrete grade C20/25, compressive strength of concrete f_{ck} = 20 MPa, elastic module E_{cm} = 30 GPa; for the tube grade S235, design value of the yield strength f_{yk} = 235 MPa, elastic module E_a = 210 GPa.

A. Bearing capacity of column with short-term loading

When the column under short-term loading, will be calculated with material characteristics the following approach respectively:

1) According to Russian approach

Characteristics of materials when subjected to short-term loading, limited value of deformation of concrete is $\varepsilon_{b1,red}$ = 0,0015, ε_{b2} = 0,0035, modular deformation of concrete when subjected to compression is $E_{b1} = 0{,}85E_b$, coefficient is k_b = $0{,}15/(0{,}3 + \delta_e)$ for $\delta_e = e_0/(2r_b)$ is relative value eccentricity of vertical force, should exceed 0,15 and not exceed 1,5. Value of relative deformation modular of concrete should be taken as $E_{b,red} = R_{bp}/\varepsilon_{b1,red}$.

Surveying bearing capacity of column when the initial eccentricity e_0 from 1,0 cm to 10 cm:

a) Limit internal force method

By this method, using eq. (3), one has:

$$N_{max}.e \le \frac{2}{3} r_b^3 R_{bp} \sin^3\alpha + \frac{1}{\pi} A_s r_s (R_s + R_{sc}) \sin\alpha + \frac{1}{\pi} A_p r_p (R_y + R_{pc}) \sin\alpha = M_{max} \quad (38)$$

Based on excel spreadsheet, get the Table 1.

Table 1. Results For The Column Under Short-Term Loading, Limited Internal Force Method.

e_0(m)	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	0,10
ε_{max}	0,0079	0,0050	0,0901	0,1089	0,0310	0,0186	0,0132	0,0091	0,0082	0,0069
N_{max}(MN)	6,696	6,068	5,805	5,450	5,081	4,750	4,445	4,143	3,950	3,757

In the table above, present maximum compression force that the column cross section can resistance N_{max} with eccentricity e, with initial eccentricity e_0 have to mention reduction factor for flexural buckling respectively. For comparison, in the table present maximum deformation of the most compressed concrete fibers, receive by method nonlinear deformation model ε_{max}. Found that, deformation values are beyond the limited deformation $\square_{b2}$ = 0,0035. So, should be design column according to the nonlinear deformation model.

b) Nonlinear deformation model method

Base on equations already made in section 2.2, excel spreadsheets get the Table 2.

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Table 2. Results For The Column Under Short-Term Loading, Nonlinear Deformation Model Method.

e_0(m)	0,010	0,020	0,030	0,040	0,050	0,060	0,070	0,080	0,090	0,100
ε_{max}	0,0033	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035
N_{max}(MN)	6,596	5,990	5,500	5,090	4,735	4,422	4,141	3,888	3,700	3,540

Comparison of longitudinal force values of the two tables found that, to achieve the deformation limit allowed, should reduce the force value received according to limited internal method force from1,5 to 7,4%.

2) According to Eurocode approach

Characteristics of materials when subjected to short-term loading, limited value of deformation of concrete is $\varepsilon_{b1,red}$ = 0,0015 → ε_{c3} = 0,00175, ε_{b2} = 0,0035 →ε_{cu3} = 0,0035, modular deformation of concrete when subjected to compression is $E_{cm1} = K_e.E_{cm}$, correction factor K_e = 0,6. Value of relative deformation modular of concrete should be taken as $E_{c,red} = f_{cc}/\varepsilon_{c3}$.

Surveying bearing capacity of column when the initial eccentricity e_0 from 1,0 cm to 10 cm:

a) According to simplified method of design

By this method, formula to check is formulas (18), can be rewritten as:

$$M_{Ed} = N_{max}.e \le M_{max} = \alpha_M.M_{pl,N,Rd} = \alpha_M.\left(\frac{2}{3} r_b^3 f_{cc} \sin^3\alpha + \frac{1}{\pi} A_s r_s 2 f_{sd} \sin\alpha + \frac{1}{\pi} A_p r_p \left(f_{yc} + f_{yd}\right) \sin\alpha\right) \quad (39)$$

Based on excel spreadsheet, get the Table 3:

Table 3. Results for the column under short-term loading, simplified method of design.

e_0(m)	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	0,10
ε_{max}	0,0581	0,1110	0,1593	0,2062	0,2520	0,2966	0,3393	0,5244	0,5107	0,4974
N_{max}(MN)	5,400	5,170	4,960	4,756	4,556	4,361	4,174	3,995	3,823	3,656

In the table above, present maximum compression force that the column cross section can resistance N_{max} with eccentricity e, with initial eccentricity e_0 have to mention second-order linear elastic analysis. For comparison, in the table present maximum deformation of the most compressed concrete fibers, receive by method nonlinear deformation model ε_{max}. Found that, deformation values are beyond the limited deformation ε_{cu3} = 0,0035. So, should be design column according to general method of design.

b) According to general method of design

Base on equations already made in section 2.2, excel spreadsheets get the Table 4.

Comparison of longitudinal force values of the two tables found that, to achieve the deformation limit allowed, should reduce the force value received according to simplified method of design from 0,7 to 3,6%.

Table 4. Results for the Column under Short-Term Loading, General Method of Design.

e_0(m)	0,010	0,020	0,030	0,040	0,050	0,060	0,070	0,080	0,090	0,100
ε_{max}	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035
N_{max}(MN)	5,360	5,091	4,845	4,620	4,412	4,210	4,034	3,870	3,712	3,560

B. Bearing capacity of column with long-term loading

When the column under long-term loading, will be calculated with material characteristics the following approach respectively:

1) According to Russian approach

Characteristics of materials when subjected to long-term loading depend on the humidity of the environment, example when humidity is greater than 75% ultimate strain of concrete is $\varepsilon_{b1,red}$ = 0,0024, ε_{b2} = 0,0042, modular deformation of concrete when subjected to compression is $E_{b1} = E_b/(1 + \varphi_{b,cr})$, with $\varphi_{b,cr}$ is creep coefficient of concrete, which should be taken as 0,8 for concrete grade B25; factor $k_b = 0{,}15/[(\varphi_1(0{,}3 + \delta_e)]$,with $\varphi_l = 1 + M_{L1}/M_L$ – factor, mention to the influence of long-term loading, should not exceed 2. For safety, should be taken as φ_1 = 2.

Surveying bearing capacity of column when the initial eccentricity e_0 from 1,0 cm to 10 cm by limit internal force method combined with calculate the maximum deformation value according to the nonlinear deformation model, get the results as Table 5.

In the table below, present maximum compression force that the column cross section can resistance N_{max} with eccentricity e, with initial eccentricity e_0 have to mention reduction factor for flexural buckling respectively. Found that, deformation values are beyond the limited deformation ε_{b2} = 0,0042. So, shouldn't be surveying bearing capacity column according to nonlinear deformation model method.

Table 5. Results for the column under long-term loading, limited internal force method.

e_0(m)	0,010	0,020	0,030	0,040	0,050	0,060	0,070	0,080	0,090	0,100
ε_{max}	0,0012	0,0012	0,0013	0,0013	0,0013	0,0014	0,0015	0,0014	0,0014	0,0015
N_{max}(MN)	2,940	2,712	2,547	2,417	2,277	2,224	2,142	2,025	1,925	1,847

In the table above, present maximum compression force that the column cross section can resistance N_{max} with eccentricity e, with initial eccentricity e_0 have to mention reduction factor for flexural buckling respectively.

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Found that, deformation values are beyond the limited deformation ε_{b2} = 0,0042. So, should'nt be surveying bearing capacity column according to nonlinear deformation model method.

2) According to Eurocode approach

Characteristics of materials when subjected to long-term loading depend on the humidity of the environment and the apparent size $h_0 = 2A_c/u = r_b$ = 311 mm, example when the outdoor humidity is 80% so last creep coefficient is $\varphi_t = 2.0$, for concrete grade C20/25 using cement grade S for the age of concrete at 28 days, modulus deformation of compressive concrete $E_{c1} = E_{cm}/(1+\psi_L\varphi_t)$, where ψ_L is coefficient due to creep, depends on the type of load should be taken as 1.1 for the long-term loading.

Surveying bearing capacity of column when the initial eccentricity e_0 from 1,0 to 10 cm by simplified method of design combined with calculate the maximum deformation value according to general method of design, get the results as follows:

Table 6. Results for the column under long-term loading, simplified method of design.

e_0(m)	0,010	0,020	0,030	0,040	0,050	0,060	0,070	0,080	0,090	0,100
ε_{max}	0,0662	0,1265	0,1827	0,2357	0,2856	0,3327	0,5254	0,5107	0,4968	0,4837
N_{max}(MN)	5,379	5,111	4,862	4,628	4,409	4,203	4,007	3,823	3,649	3,485

In the table above, present maximum compression force that the column cross section can resistance N_{max} with eccentricity e, with initial eccentricity e_0 have to mention second-order linear elastic analysis. For comparison, in the table present maximum deformation of the most compressed concrete fibers, receive by method nonlinear deformation model ε_{max}. Found that, deformation values are beyond the limited deformation ε_{cu3}= 0,0035. So, should not be surveying bearing capacity column according to general method of design.

a) According to the general method of design

Base on equations already made in section 2.2, excel spreadsheets get the Table 7.

Table 7. Results for the column under long-term loading, general method of design.

e_0(m)	0,010	0,020	0,030	0,040	0,050	0,060	0,070	0,080	0,090	0,100
ε_{max}	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035	0,0035
N_{max}(MN)	5,323	5,010	4,740	4,486	4,271	4,065	3,879	3,720	3,560	3,420

Comparison of longitudinal force values of the two tables found that, to achieve the deformation limit allowed, should reduce the force value received according to simplified method of design from 1,1 to 3,4%.

So, design concrete-filled steel tube columns according to conception deformation give results more realistic, but it takes a lot of effort. In practical design need to check the columns with same structure under different load combination, so if calculated directly according to conception deformation it takes a lot of time. That's why, authors proposed set interaction curve for first point correspondence with initial eccentricitye_{01} = 1cm (random eccentricity) have $N_{max}(e_{01})$ and end point correspondence withe_{0max} have $N_{max}(e_{0max})$. Values $N_{max}(e_{01})$, $N_{max}(e_{0max})$ identified according to conception deformation suitable with the rules of the standard. Intermediate points with eccentricity e_0(cm) be calculated from the fol1owing expression:

$$N_{max}(e_0) = N_{max}(e_{01}).\left(\frac{N_{max}(e_{0max})}{N_{max}(e_{01})}\right)^{\frac{e_0 - e_{01}}{e_{0max} - e_{01}}} \quad (40)$$

Noticed that, if using the formula just proposed for the last case (general method of design) get results pretty close compare to direct calculation (error less than 1%).

IV. CONCLUSION

The article provided the survey results bearing capacity of concrete-filled steel tube columns under short-term and long-term loading according to views on stress and deformation. Proposed practical formula to build a load-bearing curve for the eccentric compression column. However, when calculating with long-term loading, used the coefficient creep for one-axis compressed concrete (exposed to air), it is not suitable for the behaviors of concrete in steel tube (completely isolated from the environment). Therefore, there should be experimental studies on the creep of concrete filled steel tube, as a basis for calculate the bearing capacity of similar structure.

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