BEARING CAPACITY CALCULATION FOR MIDDLE SLENDER AND SLENDER CFT CIRCULAR COLUMNS

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Summary: *In middle slender and slender CFT columns the loss of capacity is based on the problem of stability. In this case, we must take into account the effects of the second order. A nonlinear model was created in computer program Abaqus. In the model a nonlinear constitutive models for concrete core and steel hollow section, as well as the connection of these two elements was used. The critical buckling force of CFT column was obtained by modified Riks method. The influence of boundary conditions and the diameter and wall thickness of steel profiles on the critical buckling force was analyzed.*

Key words: *CFT columns, nonlinear analysis, stability*

1. INTRODUCTION

Calculation of bearing capacity of CFT columns is based on calculations according to limit stages theory. Determining bearing capacity of CFT columns is complex because of nonlinear characteristics of concrete and steel, imperfections of geometry, residual stress in steel profiles, load history, load eccentricity, second order effects etc. On the other hand determining critical load is geometrical, static and material nonlinear problem, because deformations are big, boundary conditions are set on a deformed configuration and stress-strain relationship for materials is nonlinear, therefore, superposition principle is not valid. With middle slender and slender CFT columns the loss in bearing capacity is based on the stability problems. In that case one must take in consideration the second order effects. Constitutive material models have significant impact on the precision of the results. In the literature many constitutive models are described with smaller or bigger accuracy of concrete and steel behaviour [1, 2, 3, 4, 5, 6]. Using computer program Abaqus [7] and Riks method which is available in this program critical buckling forces are determined. An analysis of the D/t ratio is conducted, where D is the diameter of CFT column and t is the thickness of the steel profile. Additionally, the effects of boundary conditions on the critical buckling force is analyzed. The obtained results are compared with the forces calculated according to EC4 regulations [8].

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2. NUMERICAL MODELING OF AXIAL BEARING CAPACITY OF MIDDLE SLENDER AND SLENDER CIRCULAR COLUMNS

Bearing capacity of middle slender and slender CFT columns with length of 4.0m is determined using finite element analysis in computer program Abaqus. For modeling of concrete core and hollow steel profile C3D8R and S4R finite elements were used, respectively. Connection between this two elements was modeled with surface to surface contact elements. In order to properly model a CFT column it was necessary to use appropriate constitutive models for concrete-and structural steel.

2.1 Constitutive model for concrete

Constitutive model for concrete has a significant impact on the results and the precision of calculations. For the purpose of this analyses a Concrete Damaged Plasticity model was used. The angle of dilatation was adopted with a value of 20º [1], and the Poisson's coefficient with a value of 0.2. Since the slenderness is in the middle range the effects of confinement is neglected. The stress-strain relationship for concrete is described with the curve presented on the Figure 1., which was proposed by Moon J. et all [1].

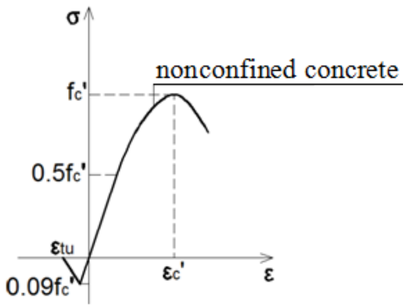

Figure 1. σ-ε diagram for non confined concrete

Dilatation $\varepsilon_c^{'}$ which corresponds to the compressive strength of nonconfined concrete $f_c^{'}$ was adopted with a value of 0.003. A linear behaviour of concrete is adopted until the value of $0.5 \cdot f_c^{'}$. Elasticity modulus was determined according to EC2 [9] with the following expression:

$$E_c = 22000 \cdot \left[\frac{f_c' + 8}{10}\right]^{0.3} [\text{MPa}] \tag{1}$$

Curved part of the σ-ε diagram which ranges between the value of proportionality border of $0.5 \cdot f_c'$ until the value of f_c' is defined with the following expression [4]:

$$\sigma = \frac{E_c \cdot \varepsilon}{1 + (R + R_E - 2) \cdot \left(\frac{\varepsilon}{\varepsilon_c}\right) - (2 \cdot R - 1) \cdot \left(\frac{\varepsilon}{\varepsilon_c}\right)^2 + R \cdot \left(\frac{\varepsilon}{\varepsilon_c}\right)^3} \tag{2}$$

where R_E and R are obtained by the following expressions:

$$R_E = \frac{E_c \cdot \varepsilon}{f_c'} \tag{3}$$

$$R = \frac{R_E \cdot (R_\sigma - 1)}{(R_\varepsilon - 1)^2} - \frac{1}{R_\varepsilon} \tag{4}$$

and R_σ and R_ε are equal to number 4 [10]. Tension strength of concrete f_t is adopted as approximately 9% of concrete compressive strength f_c'. After reaching the tension strength of concrete a softening of material occurs. Corresponding strain for the tension strength was adopted with a value 0.001 [7].

2.2 Constitutive model for structural steel

For the purpose of modelling structural steel a Von-Misses model with isotropic hardening was used. Figure 2. presents the mentioned constitutive model [1]. Where f_y represents the yield strength of steel and ε_y corresponding strain. The breaking limit is reached at the point labelled with f_u and ε_u is the corresponding strain. Young's modulus of elasticity was adopted with a value of 210GPa, and Poisson's coefficient with a value of 0.3.

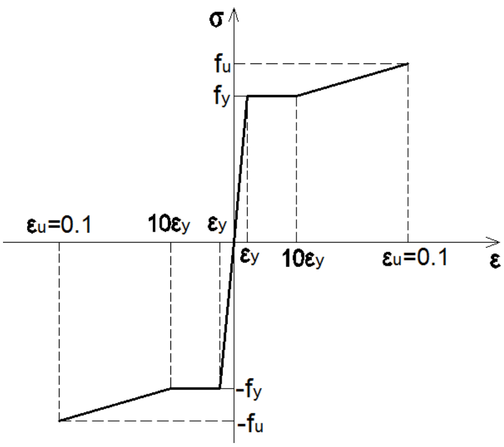

Figure 2. σ-ε diagram for structural steel

2.3 The connection of the hollow steel profile and the concrete core

The connection of the hollow steel profile and the concrete steel profile was modelled by using surface to surface contact elements [7]. The adopted contact elements define the behaviour in tangential and perpendicular direction. For perpendicular direction Hard Contact elements were used with allowed dividing of contact surfaces but without penetration due to pressure. On the other hand, for the tangential direction a Coulomb friction elements were used. According to many authors the value of friction coefficient ranges between 0.2 to 0.6 [2, 11, 12]. For these models the value of 0.47 was used [13].

3. NUMERICAL SIMULATION

Since the analyzed CFT columns are middle slender and slender it was to be expected that the loss in the bearing capacity will occur due to buckling rather than in axial cross section capacity. The boundary conditions and the D/t ratio effects on the critical force P_{cr} was analyzed. Loading forces were applied with modified Riks approach [7] which is based on Newton-Raphson method. Modified Riks approach is widely used for stability of geometrical nonlinear problems as well as material nonlinear problems. This approach is usually used with eigenvalue analyses which gives the complete picture of the stability loss and buckling of the analyzed construction.

3.1 The effects of boundary conditions on the critical buckling force P_{cr}

The boundary conditions were analyzed with two different symmetrically conditions. First is fixed support on both ends and second hinged support on both ends. Steel class is S355 and concrete class C25/30. Ratio of D/t=101.6mm/2.7mm was entered. Figures 3,4 and 5 are presenting the results of the performed calculations in Abaqus. Values of first eigenvalue, horizontal movement and critical force are shown, respectively. The minimal eigenvalue of $\lambda=1.95670 \cdot 10^5$ and the deformation pattern was calculated in the first model. These results were used in the second model where buckling analysis according to Riks approach and the following imperfection formula was used:

$$\Delta x_i = \sum_{i=1}^{M} w_i \cdot \phi_i \tag{5}$$

where: w_i-scale factor, ϕ_i- i-th buckling ton.
Maximal value of horizontal displacement of 24.67mm was reached in 29th increment of buckling analysis with the corresponding buckling force of 162.70kN.

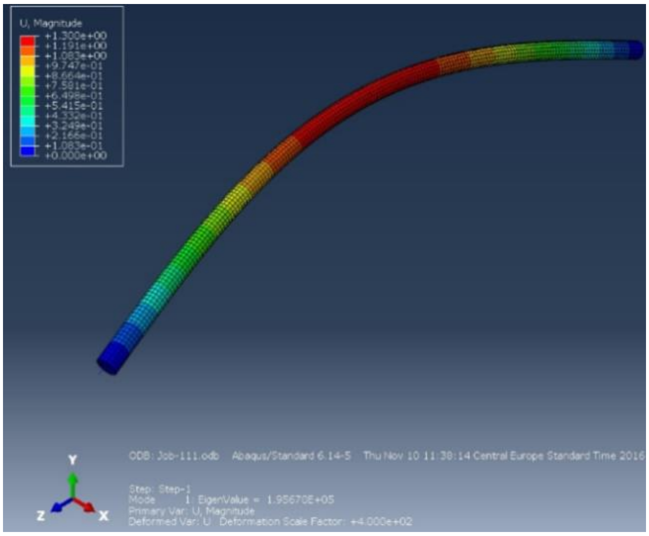

Figure 3. Calculation results- first eigenvalue

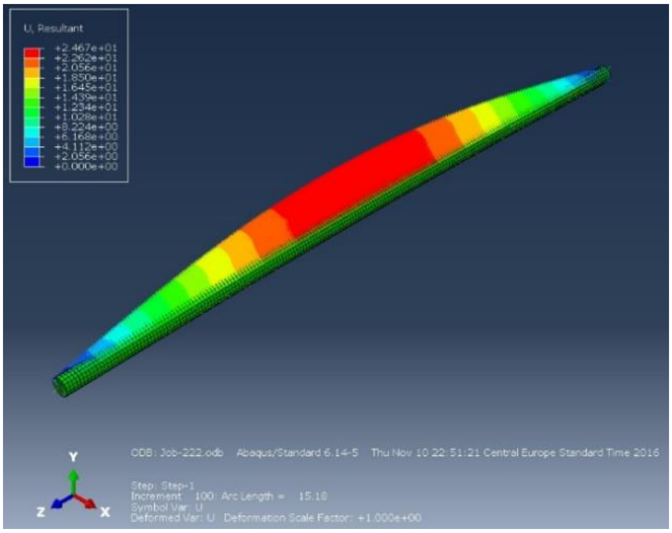

Figure 4. Calculation results- horizontal displacement

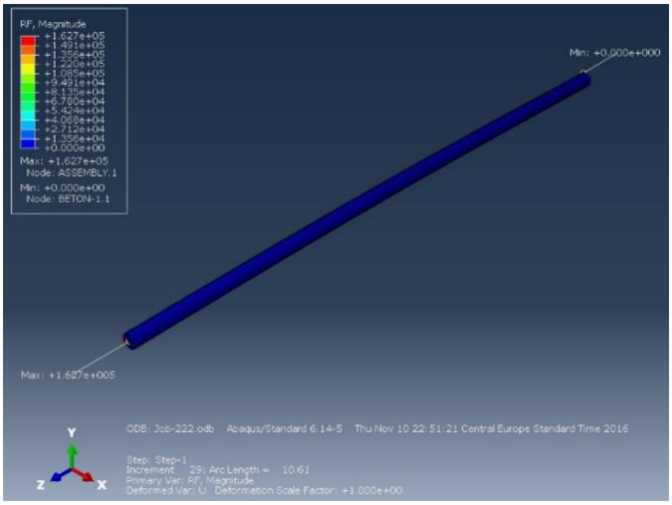

Figure 5. Calculation results- critical buckling force

In all models the first eigenvalue was adopted as dominant ton of buckling with smallest buckling force. Additionally, it should be mentioned that in the case of fixed supports the loss of axial capacity occurred before the loss of stability and buckling. Therefore, these results are not presented in this paper.

In order to compare the obtained results of calculations performed in Abaqus, critical forces were calculated according to EC4 [8]. Obtained values and their ratios are presented in the Table 1.

Table 1. Comparison of the results obtained in Abaqus and according to EC4

D/t	$101.6/2.7$	$101.6/4.0$	$114.3/2.7$	$114.3/4.0$
$N_{cr,\ ABAQUS}/N_{EC4}$ Fixed lower end and hinged support on the top end	311.60 / 319.18 = 0.976	422.29 / 415.23 = 1.017	462.14 / 473.94 = 0.975	605.96 / 613.94 = 0.987
$N_{cr,\ ABAQUS}/N_{EC4}$ Hinged support on both ends	162.70 / 156.40 = 1.040	212.21 / 203.46 = 1.043	230.14 / 232.23 = 0.991	307.15 / 300.83 = 1.021

3.2 The effects of D/t ration on the critical buckling force P_{cr}

In the further analyses the ratio D/t: 101.6mm/2.7mm, 101.6mm/4.0mm, 114.3mm/2.7mm, 114.3mm/4.0mm was analyzed. Concrete class was again C25/30, and steel class S355. The CFT column is fixed on the lower end and has a hinged support on the top .

With increase of D/t ratio from 101.6/2.7mm to 114.3mm/4.0mm critical buckling force is increased 1.928 times. Figure 6. presents the dependence of critical buckling force with increase of D/t ratio.

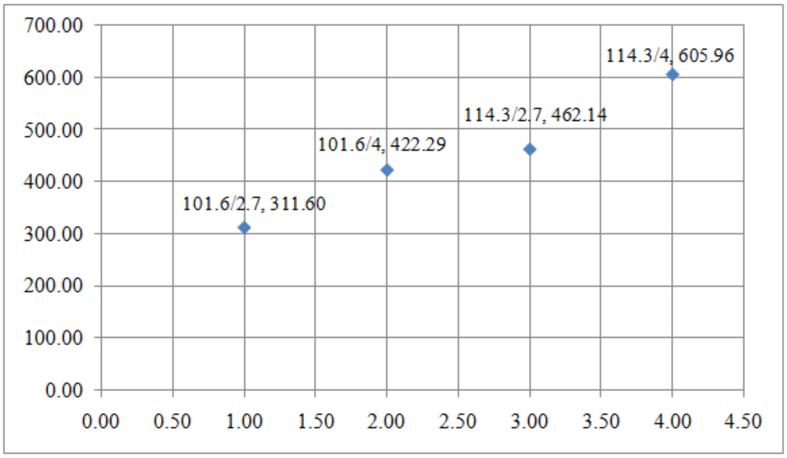

Figure 6. Effects of D/t ratio on critical buckling force

4. CONCLUSION

Successful modeling of bearing capacity of middle slender and slender CFT columns can be achieved by using Riks approach which is implemented in computer program Abaqus. The precision of the results is very dependent on the constitutive models which are used for modeling concrete core, steel profile and their connection. Additionally, the effects of boundary conditions and D/t ratio on the critical buckling force was analyzed. Obtained results were compared with the results obtained according to EC4. It can be observed that here is a good agreement.

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ПРОРАЧУН НОСИВОСТИ УМЕРЕНО ВИТКИХ И ВИТКИХ ЦФТ КРУЖНИХ СТУБОВА

Резиме: *Код умерено витких и витких ЦФТ стубова губитак носивости се заснива на проблему стабилности. У том случају морају се узети у обзир утицаји другог реда. Применом рачунског програма Abaqus у моделу су задати нелинеарни конститутивни модели за бетон језгра и конструкциони челик шупљег профила, као и веза ова два елемента. Применом модификоване Riks-ове методе срачуната је критична сила извијања ЦФТ стуба. Анализиран је утицај односа пречника и дебљине зида челичног профила, као и утицај граничних услова ослањања на вредност критичне силе извијања.*

Кључне речи: *ЦФТ стубови, нелинеарна анализа, стабилност*