EUROPEAN ACADEMIC RESEARCH
Vol. IX, Issue 3/ July 2021

Impact Factor: 3.4546 (UIF)
DRJI Value: 5.9 (B+)

ISSN 2286-4822
www.euacademic.org

Computational Code in MATLAB for Design of Columns Using Simplified Methods and Coupled to M, N, 1/r Diagrams

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Abstract

The design of reinforced concrete columns using manual calculation takes considerable time, requiring several steps for the validation of the results, in view of the involvement of several geometric and mechanical parameters. In this context, to make it easier to solve problems by manual calculation, it is proposed to develop a computational code that presents, in a sequential and clear way, the procedure for design the column with slenderness index $\lambda \leq$ 140 of reinforced concrete with rectangular section subjected to normal composite flexion following the Brazilian Code ABNT NBR 6118

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(ABNT, 2014) using the (i) standard-column method with approximate curvature, (ii) standard-column method with approximate κ stiffness by the Brazilian Code and by a direct process presented by Banki (2004) and (iii) standard-column method coupled to M, N, 1/r diagrams are developed. Based on user input data, for example, column section, concrete and steel strength, length, among others, the code will allow the calculation of eccentricities and total moments, in addition to calculating the steel rate by plotting abacuses and present a report with the possibilities of reinforcement configuration, based on the requirements of the user. At the end of the process, it is possible to view the eccentricities in the main axes of inertia, and also the final reinforcement arrangement from the one chosen by the user. The validation of the tool and the quality of the results were performed by comparing the results obtained with the tool with examples available in the literature. Finally, the tool shows satisfactory results for the maximum moment, reinforcement arrangement, among others, with an acceptable difference when compared with the results of the references.

Keywords: Columns design. Computational code in MATLAB. Standard-column method with approximate curvature. Standard-column method with approximate κ stiffness. Standard-column method coupled to M, N, 1/r diagrams.

INTRODUCTION

Columns are linear structural elements where the largest dimension is located in the vertical and their main function is to transmit the horizontal and vertical efforts from the wind, beams, and slabs to the foundations. Both the calculation method for the 2nd order effects and the classification can be performed according to the slenderness index λ as follows: for $\lambda \leq \lambda 1$, the considerations of the second-order effects are not necessary; for $\lambda \leq 90$, the simplified methods and the approximation by diagrams M, N, 1/r can be used; for $\lambda \leq 140$, only the approximate process by diagrams M, N, 1/r can be used and the fluency consideration must be performed; finally, for $\lambda \leq 200$, the use of the exact process and fluency consideration is required. The effects of physical and geometric nonlinearity become increasingly complex due to the increase in λ, that is, the increase in the level of analysis that must be implemented. As a result, it is necessary to automate the design process and several authors have presented processes to achieve this goal.

Venturini [1] presented the necessary equation to plot the abacuses to determine the reinforcement mechanical rate for calculating the steel area and designing the reinforced concrete section subjected to combined and axial bending.

Borges [2] studied the behaviour of reinforced concrete columns of any section, under oblique bending, observing both physical and geometric nonlinearities using the tool of Cadamuro [3] the analysis of real cases, to propose solutions and to widen the use of existing methods for slender reinforced concrete columns design.

Scadelai [4] presents a review of the design of reinforced concrete columns subjected to composite bending, according to Brazilian Code ABNT NBR 6118 [5], presenting the standard-column methods with approximate curvature, standard-column with approximate κ stiffness, and standard-column method coupled to the M, N, 1/r diagrams, in addition to reinforcement considerations, with practical design examples.

Pinto [6] presented guidelines for the design of columns subjected to oblique composite bending according to the requirements of the Brazilian Code [5], developing an algorithm that makes it possible to obtain the reinforcement rate without the need for normal compound bending diagrams.

Alves et al. [7] presented a formulation of the optimization problem, applying its use in columns with slenderness index between 90 to 140, jointly implementing the interaction of M, N, 1/r diagrams to consider the second-order effect, and also the fluency consideration. The optimization was performed using MATLAB.

Kochem [8] presented a computational code capable of designing slender columns in reinforced concrete with a rectangular section according to the Brazilian Code [5] using the simplified standard-column methods and the standard-column method coupled to the M, N, 1/r diagrams from the Pascal language together with the IDE (Integrated Development Environment) Lazzarus.

Souza [9] presented in his work a calculation spreadsheet in MICROSOFT EXCEL capable of carrying out the checks for the exhaustion of the resistant capacity and local stability of columns. The instability due to the second-order effects was analysed by the (i) simplified standard-column methods, (ii) by the standard-column method coupled to the M, N, 1/r diagrams, and (iii) by the general method according to [5].

In addition to traditional commercial software, such as TQS and Eberick, there are alternative tools for design columns and reinforcement considerations, such as P-Calc and MSCalc. Both allow the design of reinforced concrete columns according to [5] and present the user with the values of moment and the final reinforcement arrangement, in addition to other functionalities. On the one hand, both P-Calc [10] and MSCalc [11] do not have the computational codes used to determine the features presented, for example, determination of the active moments, in addition to the fact that MSCalc is free to a certain extent.

Thus, it is proposed to create a free computational code in MATLAB that uses the standard-column methods (i) with approximate curvature, (ii) with approximate κ stiffness by the interactive process by [5] and by direct process [12] and the standard-column method coupled to M, N, 1/r diagrams. In addition to the methods for considering second-order effects, it is proposed to determine the area of steel As, dimension, and reinforcement arrangement from the plot of the design abacuses proposed by Venturini [1], to present a report with the reinforcement arrangement possibilities and visualize the eccentricities in the main axes of inertia, the final reinforcement arrangement from the one chosen by the user and the corresponding design abacus.

DESIGN COLUMN METHODS ACCORDING TO BRAZILIAN CODE

To determine the eccentricities and maximum moments acting on the column, the Brazilian Code [5] presents approximate methods to determine the second-order local effects. For the scope of this research, the (i) Standard-column method with approximate curvature; the (ii) Standard-column method with approximate κ stiffness; and (iii) Standard-column method coupled to M, N, 1/r diagrams are used.

According to the column slenderness, this method is employed as show in Table 1 where the simplified methods in Table 1 refer to standard-column methods with approximate curvature and approximate κ stiffness.

Table 1. Brazilian Code Recommendations

Slenderness Index	Consideration of 2th Order Effects	Calculation Process			Fluency Consideration
		Exact	Coupled to Diagrams (M, N, 1/r)	Simplified	
$\lambda \leq \lambda 1$	Dispensable	-	-	-	-
$\lambda \leq 90$	Obligatory	Dispensable	Allowed	Allowed	Dispensable
$\lambda \leq 140$				Prohibited	Obligatory
$\lambda \leq 200$		Obligatory	Prohibited		

Font: adapted from Bendô [13].

As the code methodology for the simplified methods has already been presented in [14], only the standard-column method coupled to M, N, 1/r diagrams is discussed in more detail.

2.1 Standard-Column Method Coupled to M, N, 1/r Diagrams

According to Table 1, this method can be applied to columns with slenderness index $\lambda \leq 140$ to determine the second order local effects, in addition to the consideration of fluency effects for $\lambda > 90$. For this, the value from the specific moment-curvature diagram for the case should be used for the curvature of the critical section.

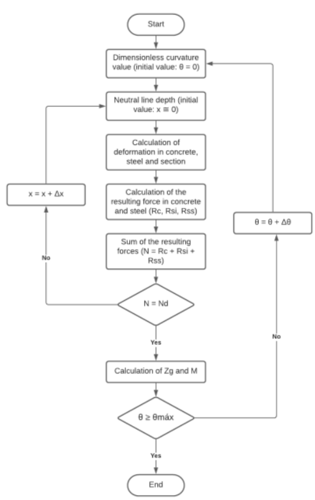

Figure 1. Moment-curvature abacus construction procedure (adapted from Ribeiro [15]).

Table 2. Expressions for strains as a function of curvature adapted from Pinto [6].

Strain	Expressions
In the concrete (top fibber)	$\varepsilon_c = \frac{\theta \cdot x}{h}$
In the concrete (bottom fibber)	$\varepsilon_c = \frac{\theta \cdot (x-h)}{h}$
In steel (any reinforcement)	$\varepsilon_c = \frac{\theta \cdot (x-d_n)}{h}$
On the fibber 3/7h away	$\varepsilon_{\left(\frac{3}{7}h\right)} = \frac{\theta \cdot \left[x - \left(\frac{3}{7} \cdot h\right)\right]}{h}$

Table 3. Resistance reinforcement expressions adapted from Pinto [6].

Resistance reinforcement		Expressions
Reinforcement	$\lvert\varepsilon_s\rvert > \lvert\varepsilon_{yd}\rvert$	$R_s = A_s \cdot f_{yd}$
	$\lvert\varepsilon_s\rvert \leq \lvert\varepsilon_{yd}\rvert$	$R_s = A_s \cdot f_{yd} \cdot \frac{\varepsilon_s}{\varepsilon_{yd}}$

Table 4. Resistance concrete and respective lever arms adapted from Ribeiro [15].

Conditions	Expressions
$\varepsilon_c \leq 2‰$ $x \leq \mathrm{h}$	$R_{cc} = \sigma_{cd} \cdot b \cdot x \cdot \left(\frac{\varepsilon_c}{2} - \frac{\varepsilon_c^2}{12}\right)$ $a = \frac{x}{4} \cdot \frac{(8-\varepsilon_c)}{(6-\varepsilon_c)}$
$\varepsilon_c \leq 2‰$ $x > \mathrm{h}$	$R_{cc} = \sigma_{cd} \cdot b \cdot h \cdot \left(\varepsilon_c + \frac{\varepsilon_c\theta}{4} - \frac{\varepsilon_c^2}{4} - \frac{\theta}{2} - \frac{\theta^2}{12}\right)$ $a = \frac{\varepsilon_c \cdot h}{\theta} - \frac{h \cdot \left[\frac{\varepsilon_c^3}{3} - \frac{\varepsilon_c^4}{16} - \frac{(\varepsilon_c-\theta)^3}{3} + \frac{(\varepsilon_c-\theta)^4}{16}\right]}{\theta^2\left(\varepsilon_c + \frac{\varepsilon_c\theta}{4} - \frac{\varepsilon_c^2}{4} - \frac{\theta}{2} - \frac{\theta^2}{12}\right)}$
$\varepsilon_c < 2‰$ $x \leq \mathrm{h}$	$R_{cc} = \sigma_{cd} \cdot b \cdot x \cdot \left(1 - \frac{2}{3\varepsilon_c}\right)$ $a = x \cdot \frac{(3\varepsilon_c^2 - 4\varepsilon_c + 2)}{2\varepsilon_c(3\varepsilon_c - 2)}$
$\varepsilon_c < 2‰$ $x > \mathrm{h}$	$R_{cc} = \sigma_{cd} \cdot b \cdot h \cdot \left(\frac{12\varepsilon_c - 8 - (\varepsilon_c-\theta)^2(6-\varepsilon_c+\theta)}{12\theta}\right)$ $a = \frac{h}{\theta}\left\{\varepsilon_c - \frac{\left[24\varepsilon_c^2 - 16 - (\varepsilon_c-\theta)^3(16-3\varepsilon_c+3\theta)\right]}{4\left[12\varepsilon_c - 8 - (\varepsilon_c-\theta)^2(6-\varepsilon_c+\theta)\right]}\right\}$

The design method is the one proposed by Ribeiro [15], also used and validated by Pinto [6]. The determination of the moment-curvature correlation is developed from the iterative process, between successive attempts and approximations. Because of this, manual calculation is unviable, and it is necessary to align with computational codes, as proposed in this paper. The steps proposed are found in the flowchart of Figure 1.

For the first interaction, an initial dimensionless curvature value θ and neutral line x are adopted as equal and approximately zero, respectively. From there, the variables related to steel and concrete are defined, according to the expressions presented in Table 2 to Table 4.

In Table 2, d_n is the distance from the analysed reinforcement layer to the top of the section [15].

In Table 4, the design normal force and the design bending moment is given by Equations 10 and 11, respectively.

$$N = R_{cc} + R_{s1} + R_{s2} + \cdots + R_{sn} \quad (1)$$

$$M = R_{s1} \cdot \left(\frac{h}{2} - d_1\right) + R_{s2} \cdot \left(\frac{h}{2} - d_2\right) + \cdots + R_{sn} \cdot \left(\frac{h}{2} - d_n\right) + R_{s2} \cdot \left(\frac{h}{2} - a\right) \quad (2)$$

ABACUS PROCEDURE FOR REINFORCEMENT RATE DETERMINATION

Venturini [1] presents a methodology for plotting the abacuses to determine the reinforcement mechanical rate ω and the steel area A_s. The main expressions of the strains along any section are summarized in Table 5. Figures 2-4 show the configurations of strain regions I, II, and III.

In Table 5, δ, β_y, and ξ are the dimensionless of the steel position, dimensionless integration limits and neutral line position.

Venturini [1] presents Equations 12 and 13 to evaluate the axial resistance N_R and the bending resistance M_R in a section of reinforced concrete with geometry and material characteristics defined in terms corresponding to the parabolic parts, between values of y_1 and y_2, and rectangular, valid between the values of y_2 and y_3.

$$N_R = 850 f_{cd} b_w \int_{y_{c2}}^{y_{c1}} (1 + 250\varepsilon_c)\varepsilon_c dy - 0{,}85 f_{cd} b_w (y_2 - y_3) + \sum_{i=1}^{N} \sigma_{si} A_{si} \quad (3)$$

$$M_R = 850 f_{cd} b_w \int_{y_2}^{y_1} y(1 + 250\varepsilon_c)\varepsilon_c dy - 0{,}425 f_{cd} b_w (y_2^2 - y_3^2) + \sum_{i=1}^{N} y_{si} \sigma_{si} A_{si} \quad (4)$$

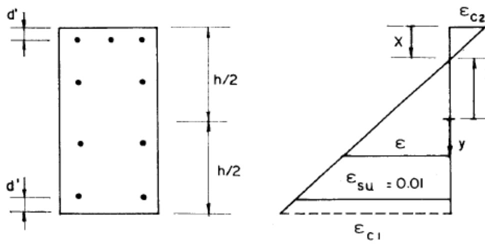

Figure 2. Strains of region I [1].

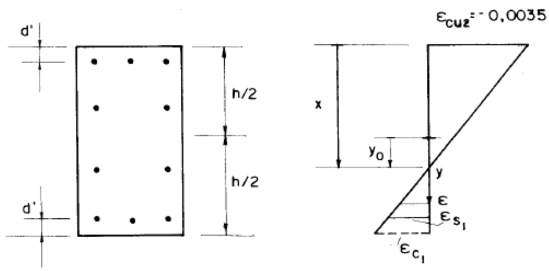

Figure 3. Strain of region II [1].

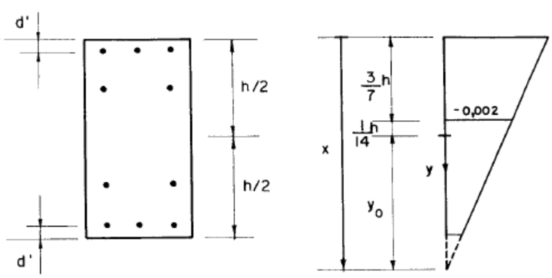

Figure 4. Strain of region III [1].

To make calculations easier, the equations are not dependent on the dimensions of the cross-section, h and b_w, and on the calculation tension f_{cd}. For this, the dimensionless force value ν and the dimensionless bending moment μ, shown in Equations 14 and 15, respectively, are defined.

$$\nu = \frac{N_R}{A_c f_{cd}} \quad (5)$$

$$\mu = \frac{M_R}{A_c h f_{cd}} \tag{6}$$

Table 5. Main expressions of determinations along any section configuration adapted from Venturini [1].

Variable	Region I	Region II	Region III
Strain	$\varepsilon = \varepsilon_{su} \dfrac{\xi - (\beta_y + 0{,}5)}{\xi - 1 + \delta}$	$\varepsilon = \varepsilon_{cu} \dfrac{\xi - (\beta_y + 0{,}5)}{\xi}$	$\varepsilon = \varepsilon_u \dfrac{\xi - (\beta_y + 0{,}5)}{\xi - \xi_0}$
Array	$-\infty \leq \xi \leq 0{,}259(1-\delta)$	$0{,}259(1-\delta) \leq \xi \leq 1$	$\xi \geq 1$
Ultimate strain	$\varepsilon_u = \varepsilon_{su} = 0{,}01$	$\varepsilon_u = \varepsilon_{cu} = -0{,}0035$	$\varepsilon_u = \varepsilon_{cu} = -0{,}002$
Initial strain	$\xi_0 = 0$	$\xi_0 = 0$	$\xi_0 = 3/7$

Making the necessary substitutions, integrating and also considering the adequacy for the intervals between parabola and rectangle, we have Equations 16 and 17.

$$\begin{aligned} \nu = {} & \frac{850\varepsilon_u}{\xi - \xi_0}\left\{\frac{250\varepsilon_u}{\xi - \xi_0}\left[\frac{(\beta_1^3 - \beta_2^3)}{3} - (\xi - 0{,}5)(\beta_1^2 - \beta_2^2) + \right.\right. \\ & \left. + (\xi - 0{,}5)^2(\beta_1 - \beta_2)\right] - \frac{(\beta_1^2 - \beta_2^2)}{2} + \\ & \left. + (\xi - 0{,}5)(\beta_1 - \beta_2)\right\} - 0{,}85(\beta_2 - \beta_3) + \frac{\omega}{\mathrm{f_{yd}}}\sum_{i=1}^{N} \sigma_{si}\eta_{si} \end{aligned} \tag{7}$$

$$\begin{aligned} \mu = {} & \frac{850\varepsilon_u}{\xi - \xi_0}\left\{-\frac{(\beta_1^3 - \beta_2^3)}{3} + \frac{(\xi - 0{,}5)(\beta_1^2 - \beta_2^2)}{2} + \frac{250\varepsilon_u}{\xi - \xi_0}\right. \\ & \left[\frac{-2(\xi - 0{,}5)(\beta_1^3 - \beta_2^3)}{3} + \frac{(\xi - 0{,}5)^2(\beta_1^2 - \beta_2^2)}{2} + \right. \\ & \left.\left. + \frac{(\beta_1^4 - \beta_2^4)}{4}\right]\right\} - 0{,}425(\beta_2^2 - \beta_3^2) + \frac{\omega}{f_{yd}}\sum_{i=1}^{N} \sigma_{si}\beta_{si}\eta_{si} \end{aligned} \tag{8}$$

where ω is the reinforcement mechanical ratio of the cross-section and η_{si} is the ratio of the area of a bar and the total area, given respectively by Equations 18 and 19.

$$\omega = \frac{f_{yd} A_s}{f_{cd} A_c} \tag{9}$$

$$\eta_{si} = \frac{A_{si}}{A_s} \tag{10}$$

3.1 Reinforcement determination by the Direct Formulation

Particularly for rectangular sections and, considering the design reinforcement for a given requirement, the balance is imposed by equating design resistance with resistance for axial force and bending, that is, as presented by Equations 20 and 21 [1].

$$N_d = -0{,}68 f_{cd} b_w x + A_{s1}\sigma_{s1} + A_{s2}\sigma_{s2} \tag{11}$$

$$M_d = -0{,}34 f_{cd} b_w x (h - 0{,}8x) + (0{,}5h - d)(A_{s1}\sigma_{s1} + A_{s2}\sigma_{s2}) \tag{12}$$

where *x* is the position of the neutral line, bw is the base of the column, A_{s1}, A_{s2}, σ_{s1} and σ_{s2} refer to the steel area and the tension of the upper and lower reinforcement layer.

To obtain the final steel area A_s, a value for *x* must be arbitrated until the M_d value converges.

COMPUTATIONAL CODE IMPLEMENTATION PROGRAM

For the development of the tool, the MATLAB (student version) was used, using scripts together with functions, to make programming simpler and to facilitate the identification of errors. Figure 5 shows the implemented code flowchart.

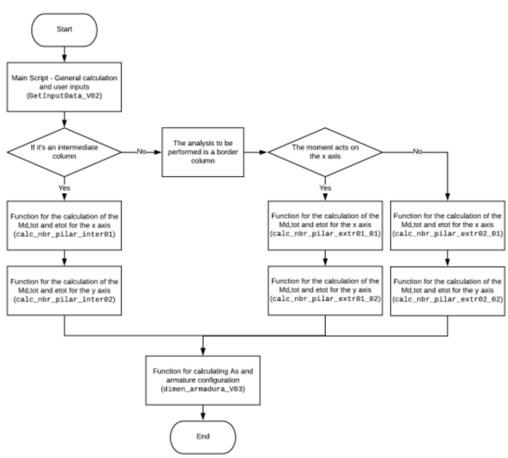

Figure 5. Complete code flowchart.

From the compilation of the main script, it is possible to obtain (i) the total eccentricity of the columns studied in this work, (ii) the reinforcement mechanical rate or the steel area, depending on the user's choice, based on the one proposed by Venturini [1] for normal composite flexion, (iii) a text file report with the reinforcement arrangement possibilities, and (iv) the graphical presentation of the column cross-section with the reinforcement arrangement chosen by the user.

In addition to the integration of the approximate methods described, the script allows the abacus to be plotted to determine the mechanical reinforcement rate from the chosen column configuration. The user can also choose not to view the abacus and directly define the value of the steel area A_s.

Equations 20 and 21 were generalized for any reinforcement arrangement, not necessarily for only two layers, as shown in Equations 22 and 23.

$$N_d = -0{,}68 f_{cd} b_w x + \left(\sum_{i=1}^{N} \frac{\sigma_{si} n_{si}}{n} \right) A_s \tag{13}$$

$$M_d = -0{,}34 f_{cd} b_w x (h - 0{,}8x) + \sum_{i=1}^{N} \frac{\sigma_{si} n_{si} \beta_{si} h}{n} \tag{14}$$

where n is the number of bars at the end and A_s is the desired steel area.

From the resolution of Equation 23, arbitrating the value of x and performing the necessary calculations in relation to concrete and steel, only A_s is unknown. Figure 6 shows the flowchart of the computational code for plotting the design abacuses and the subroutine created for plotting the abacus can be found in Appendix A.

To generate the report of possibilities of reinforcement configuration, firstly, from the input data, the shear reinforcement diameter Φt and the aggressiveness class are chosen to determine the nominal coverage with cnom. The code then creates a matrix to store all possibilities of a number of bars, both along the base and along with the height of the cross-section. After filling in the matrix, it goes through screens that apply the requirements presented in the Brazilian Code [5].

Such requirements are maximum steel area *As,max*, minimum steel area *As,min*, calculated steel area *As,calc*, the minimum amount of steel, and spacing between longitudinal reinforcements. For this, the spacing between the longitudinal reinforcement is formulated according to Equations 24 and 25.

$$h_{si} = h_{pilar} - 2\phi_t - 2c_{nom} \tag{15}$$

$$b_{si} = b_{pilar} - 2\phi_t - 2c_{nom} \tag{16}$$

Once the storage matrix has been defined with the possible number of bars, the other requirements laid down by the standard apply. In the end, the storage matrix presents only the possibilities for the column frame, creating from the matrix a report in a text file of possibilities with commercial steel bars. The flowchart of the step for determining the reinforcement arrangement is shown in Figure 7.

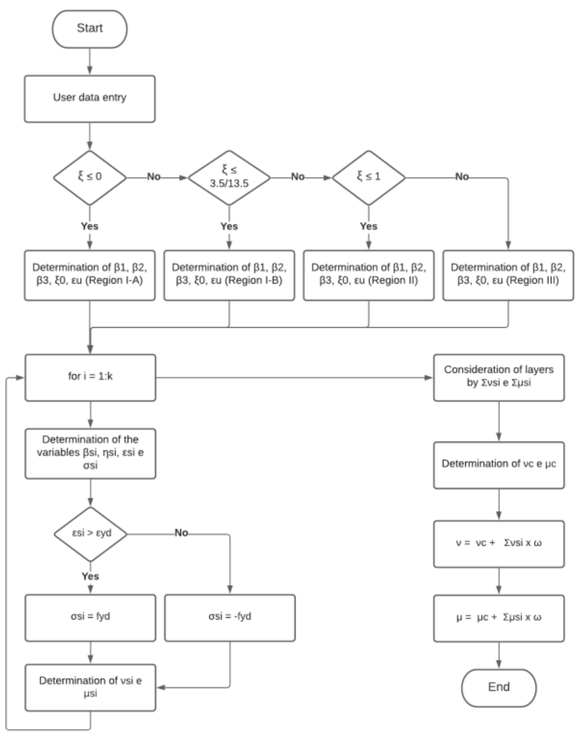

Figure 6. Venturini's abacus procedure.

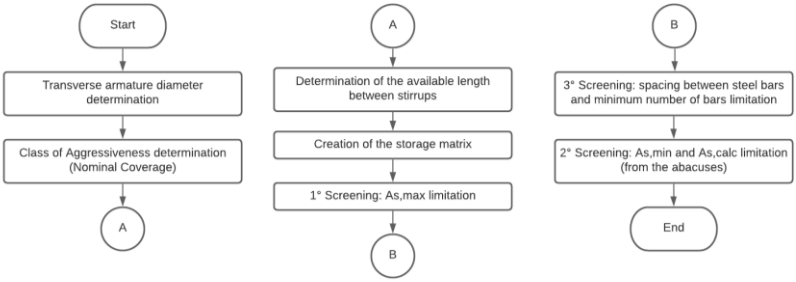

Figure 7. Reinforcement arrangement determination procedure.

RESULTS AND DISCUSSIONS

The code validation for the simplified methods was performed in Silva and Sampaio [14]. To validate the standard-column method coupled to M, N, 1/r diagrams, one example by Ribeiro [15] and two examples by Scadelai [4] were used.

1.1 Example 1

The example is shown in Figure 8. The problem data are concrete C25, Steel CA-50, nominal coverage $c_{nom} = 2.5\ mm$, the diameter of the stirrups $\Phi t = 6.3\ mm$, and diameter of the bars $\Phi l\ = 16\ mm$. With normal design force $Nd = 730\ kN$, length $l = 320\ cm$ and rectangular cross-section 20 cm x 40 cm.

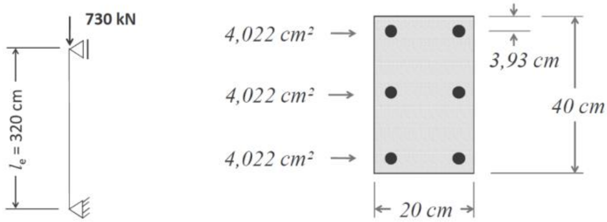

Figure 8. Column configuration of the Example 1 [15].

The results for the dimensionless curvature value θ and the total bending moment M_{tot} are shown in Table 6, comparing the interactions of the reference work and those obtained by the computational code developed. Figure 9 shows the generated moment-curvature diagram.

From Table 6, it is observed that there was a small relative deviation of 2.5% in relation to the reference solution. This small deviation is due to the accuracy and difference of the input data and treatment of the variables within the code.

Table 6. Comparison of results for the Example 1.

θ (Ribeiro, 2011)	Mtot (Ribeiro, 2011)	Mtot (code)	Relative difference (%)
0,00	0,00	0,00	0,00
0,50	25,78	25,49	1,14
1,00	50,94	52,30	2,60
1,50	68,81	69,01	0,29
2,00	81,49	81,95	0,56
2,50	91,78	93,14	1,46
3,00	100,62	102,30	1,64
3,44	107,40	110,17	2,05

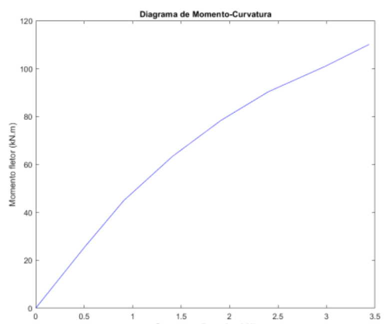

Figure 9. Example 1 moment-curvature diagram.

1.2 Example 2

The second example is a column whose cross-section has width b = 26 cm and height h = 40 cm. Concrete is class C25 and steel CA-50. The nominal coverage is $c_{nom} = 2.5\ cm$ and $d' = 4\ cm$. The applied characteristic force is $N_k = 930\ kN$ and the column length is given by $l = 675\ cm$ (Figure 10).

Two analyses are performed. The first considers that the column is supported with no transversal forces for $\alpha_b = 0,4$, while the second considers that the column fits the characteristic of bi-supported or in balance columns with moments less than the minimum, that is, with $\alpha_b = 1,0$.

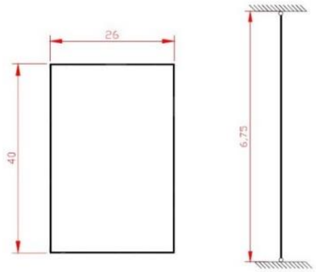

Figure 10. Column configuration of the example 2 [4].

Table 7. Comparison of results for the Example 2.

	Variables	Scadelai (2004)	Code	Relative difference (%)
$\alpha_b = 1{,}00$[1]	A_s	26,05	25,01	4,0
$\alpha_b = 0{,}40$[2]	A_s	17,94	17,36	3,2

[1] Bi-supported or in balance columns with moments less than the minimum.
[2] Bi-supported columns without shear forces.

Table 7 compares the results for the steel area obtained by the present formulation with that ones obtained by Scadelai [4]. It is believed that the observed variation is due to the calculation for determining the steel area *As* to have been obtained directly from the printed abacuses of Venturini (1987), while the present tool makes the calculation automatically during the processing of the problem. Analysing the relative difference between the results obtained, it can be said that the quality of results generated by the code is satisfactory.

```
Opção 1. bitola = 10.0 | n° de barras nas extremidades = 3 | quantidade de camadas = 12 | n° de barras totais = 26 | As,real = 20.42
Opção 2. bitola = 10.0 | n° de barras nas extremidades = 4 | quantidade de camadas = 11 | n° de barras totais = 26 | As,real = 20.42
Opção 3. bitola = 10.0 | n° de barras nas extremidades = 5 | quantidade de camadas = 10 | n° de barras totais = 26 | As,real = 20.42
Opção 4. bitola = 10.0 | n° de barras nas extremidades = 6 | quantidade de camadas = 9 | n° de barras totais = 26 | As,real = 20.42
Opção 5. bitola = 10.0 | n° de barras nas extremidades = 7 | quantidade de camadas = 8 | n° de barras totais = 26 | As,real = 20.42
Opção 6. bitola = 12.5 | n° de barras nas extremidades = 2 | quantidade de camadas = 9 | n° de barras totais = 18 | As,real = 22.09
Opção 7. bitola = 12.5 | n° de barras nas extremidades = 3 | quantidade de camadas = 8 | n° de barras totais = 18 | As,real = 22.09
Opção 8. bitola = 12.5 | n° de barras nas extremidades = 4 | quantidade de camadas = 7 | n° de barras totais = 18 | As,real = 22.09
Opção 9. bitola = 12.5 | n° de barras nas extremidades = 5 | quantidade de camadas = 6 | n° de barras totais = 18 | As,real = 22.09
Opção 10. bitola = 12.5 | n° de barras nas extremidades = 6 | quantidade de camadas = 6 | n° de barras totais = 20 | As,real = 24.54
Opção 11. bitola = 16.0 | n° de barras nas extremidades = 2 | quantidade de camadas = 5 | n° de barras totais = 10 | As,real = 20.11
```

Figure 11. Reinforcement arrangement possibilities Report.

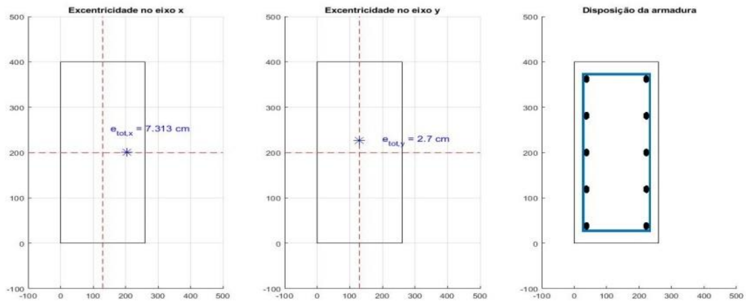

Figure 12. Eccentricities and reinforcement arrangement.

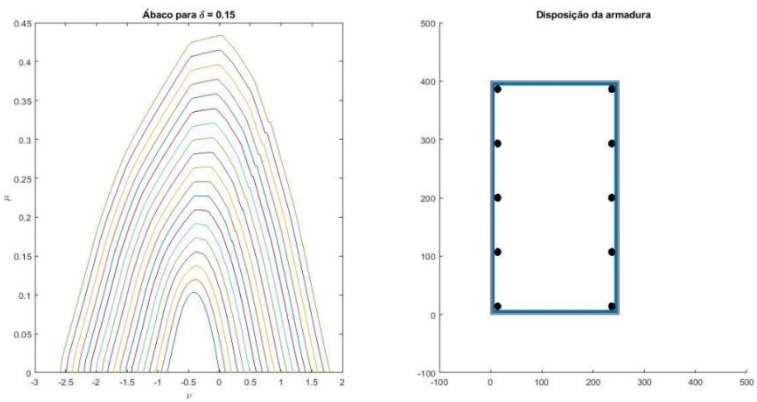

Figure 13. Venturini's abacus plot and reinforcement arrangement.

In addition to the total moment, eccentricity, and steel area, the developed tool allows to detail the studied section. At the end of the processing, the code presents a report of possibilities of reinforcement arrangement according to Figure 11. From this report, the user can choose the desired arrangement. Once the choice is made, the tool presents the cross-section according to Figure 12 and the corresponding Venturini's abacus as shown in Figure 13.

CONCLUSIONS

A computer code in MATLAB for the design of rectangular columns subjected to normal composite flexion was developed, where it can determine the total eccentricity of rectangular columns with slenderness index $\lambda \leq 140$, determine the steel area A_s from of the method proposed by Venturini (1987) for plotting the abacus for

design and exporting a report of the configuration of the longitudinal reinforcement in the cross-section of the column according to the one chosen by the user from the possible configurations. The standard-column methods with approximate curvature, approximate κ stiffness through the interactive process by the Brazilian Code [5] and by the direct process presented by Banki [12] and the standard-column method coupled to M, N, 1/r diagrams were used. Performed in open and free code in the MATLAB educational version, it allows the design to be learned simply by the user, thanks to its sequential form of development, without having to spend time on manual calculation. From the analysis of results among the examples, the code demonstrates good precision.

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APPENDIX A – VENTURINI'S ABACUS SUBROUTINE

```
clc;clear all

% Entradas

delta = 0.05;
b1 = 2; %quantidade de barras na primeira camada
bn = b1; %quantidade de barras na segunda camada
n = 3; %número de camadas de barras de aço
nb = b1 + bn + (n-2)*2; %quantidade de barras totais
fyd = 50/1.15; %kN/cm²
E = 21000; %kN/cm²
epsilon_yd = fyd/E; %none

% Valores de linha neutra

mat_csi(1,1) = -1000;
mat_csi(2,1) = -5;

cont1 = 3;
for k1 = -2:0.01:2
   mat_csi(cont1,1) = k1;
   cont1 = cont1+1;
end

mat_csi(cont1,1) = 5;
mat_csi(cont1+1,1) = +1000;
[value_1,~] = size(mat_csi);

% Valores para a armadura
```

```
armadura(1,1) = b1;
if n > 2
  for i_2 = 2:1:(n-1)
    armadura(i_2,1) = 2;
  end
end
armadura(n,1) = bn;

for k1 = 1:1:value_1

  csi = mat_csi(k1,1);

  % Considerações iniciais:

  if csi <= 0 % Região I-A

    beta_1 = -0.5;
    beta_2 = -0.5;
    beta_3 = -0.5;
    csi_0 = 1 - delta;
    epsilon_u = 0.01;

  elseif csi <= 3.5/13.5 % Região I-B

    beta_1 = csi-0.5;
    if beta_1 < -0.5
      beta_1 = -0.5;
    elseif beta_1 > 0.5
      beta_1 = 0.5;
    end
    beta_2 = 1.2*csi+0.2*delta-0.7;
    if beta_2 < -0.5
      beta_2 = -0.5;
    elseif beta_2 > -1/14
      beta_2 = -1/14;
    end
    beta_3 = -0.5;
    csi_0 = 1 - delta;
    epsilon_u = 0.01;

  elseif csi <= 1 % Região II

    beta_1 = csi-0.5;
    if beta_1 < -0.5
      beta_1 = -0.5;
    elseif beta_1 > 0.5
      beta_1 = 0.5;
    end
    beta_2 = (3/7)*csi - 0.5;
    if beta_2 < -0.5
      beta_2 = -0.5;
    elseif beta_2 > -1/14
      beta_2 = -1/14;
    end
    beta_3 = -0.5;
    csi_0 = 0;
    epsilon_u = -0.0035;

  else % Região III

    beta_1 = csi-0.5;
    if beta_1 < -0.5
      beta_1 = -0.5;
```

```
    elseif beta_1 > 0.5
      beta_1 = 0.5;
    end
    beta_2 = -1/14;
    beta_3 = -0.5;
    csi_0 = 3/7;
    epsilon_u = -0.002;
  end

  % Considerações do aço:

  for k2 = 1:1:n

    beta_si = 0.5 - delta - (k2-1)*(1-(2*delta))/(n-1);
    eta_si = armadura(k2)/nb
    epsilon_si = epsilon_u*(csi-(0.5+beta_si))/(csi-csi_0);
    sigma_si = epsilon_si*E;

    if epsilon_si > epsilon_yd
      sigma_si = fyd;
    elseif epsilon_si < -epsilon_yd
      sigma_si = -fyd;
    end

    vet_ni_s_aux(k2,1) = sigma_si*eta_si/fyd;
    vet_mi_s_aux(k2,1) = sigma_si*beta_si*eta_si/fyd;

  end

  vet_ni_s(k1,1) =  sum(vet_ni_s_aux);
  vet_mi_s(k1,1) =  sum(vet_mi_s_aux);

  % Consideração do concreto

  aux1_ni_d = (((beta_1^3)-(beta_2^3))/3)-(csi-0.5)*((beta_1^2)-(beta_2^2))+((csi-0.5)^2)*(beta_1-beta_2);
  aux2_ni_d = -(((beta_1^2)-(beta_2^2))/2)+((csi-0.5)*(beta_1-beta_2));
  aux3_ni_d = ((250*epsilon_u/(csi-csi_0))*aux1_ni_d)+aux2_ni_d;
  vet_ni_c(k1,1) = (850*epsilon_u*aux3_ni_d/(csi-csi_0))-(0.85*(beta_2-beta_3));

  aux1_mi_d      =      (-2*(csi-0.5)*((beta_1^3)-(beta_2^3))/3)+(((csi-0.5)^2)*((beta_1^2)-(beta_2^2))/2)+(((beta_1^4)-
(beta_2^4))/4);
  aux2_mi_d = (-((beta_1^3)-(beta_2^3))/3)+((csi-0.5)*((beta_1^2)-(beta_2^2))/2);
  aux3_mi_d = (850*epsilon_u/(csi-csi_0))*(aux2_mi_d+((250*epsilon_u*aux1_mi_d/(csi-csi_0))));
  vet_mi_c(k1,1) = aux3_mi_d-0.425*((beta_2^2)-(beta_3^2));

end

[value_2,value_3] = size(vet_ni_c);

k3 = 1;
for omega = 0.0:0.1:1.8

  k4 = 1;
  while k4 <= value_2
    ni = vet_ni_c(k4,1) + (omega*vet_ni_s(k4,1));
    mi = vet_mi_c(k4,1) + (omega*vet_mi_s(k4,1));

    mat_ni(k4, k3) = ni;
    mat_mi(k4, k3) = mi;

    k4 = k4+1;
  end
  k3 = k3+1;
```

```
end


[~,value_5] = size(mat_mi);

k5 = 1;
while k5 <= value_5
   plot(mat_ni(:,k5), mat_mi(:,k5));
   hold on
   k5 = k5 + 1;
end

hold on
title('Relação entre esforços adimensionais últimos em uma seção retangular e aço CA-50A')
xlabel('\nu');
ylabel('\mu');
hold off
```