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# Design of precast columns bases embedded in socket foundations with smooth interfaces

# *Projeto da base de pilares pré-moldados embutidos em cálices de fundação com interfaces lisas*



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## Abstract

This work compares the experimental and theoretical results of a new formulation for the design of precast column bases embedded in socket foundations with smooth interfaces. Through the proposed strut and tie model, the longitudinal and transverse reinforcements of column are calculated, and concrete crushing is verified. The proposed design model was rather rational in the reinforcements design, and its results were very close to experimental results. These latter results were generated from two specimens with smooth interfaces submitted to a normal load with large eccentricity, varying the embedded lengths. For loads with small eccentricities, a simplified model is proposed, which neglects the friction of interfaces and the eccentricity of normal reaction at the column base.

***Keywords***: precast column base, socket foundation, smooth interfaces, struts and ties

## Resumo

Esse trabalho apresenta a comparação de resultados experimentais e teóricos de uma nova formulação para o projeto da base de pilares pré-moldados embutidos em cálice de fundação com interfaces lisas. Através do modelo de bielas e tirantes proposto, são dimensionadas as armaduras longitudinal e transversal do pilar e verificado o esmagamento do concreto das bielas. O método de projeto proposto mostrou-se bastante racional no dimensionamento das armaduras, apresentando resultados bastante próximos dos resultados experimentais, sendo estes últimos referentes a ensaios de dois protótipos com interfaces lisas submetidos à força normal com grande excentricidade, nos quais foi variado o comprimento de embutimento. Para o caso de força normal com pequena excentricidade, é apresentada uma simplificação do modelo, em que o atrito das interfaces e a excentricidade da reação normal na base do pilar são desprezados.

***Palavras-chave:*** base de pilar pré-moldado, cálice de fundação, interfaces lisas, bielas e tirantes

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## 1. Introduction

Even though the column-foundation connection through socket is commonly used in precast concrete structures in Brazil, its behavior is still not well understood, specially that of the column base.

The behavior model was originally proposed by Leonhardt & Mönnig [1] and the subsequent methods are variations of it, adding the forces of friction and the eccentricity of normal reaction at the base. The most current design model of Canha [2] accounts for all of these additional variables by formulating accurate equations.

However, most of the existing calculation methods for that connection emphasize the socket design and detailing without including information and recommendations that describe the internal behavior and design of columns base; thus, the analysis of the column is restricted to calculating the loads at the base. In addition to the lack of studies related to the column base, the possibility of considering strength mechanisms such as the concrete confinement and the stress reduction at the column base due to friction, which were not included in the column theoretical models, represented an additional motivation for the study of Ebeling [3], who conducted an experimental investigation of the column-foundation connection by socket with smooth interfaces, focusing on the analysis of column base. After the tests and analysis of the results, a strut and tie model was proposed for the base design, presenting, however, incompatibilities with the socket model of Canha [2]. Thus, Campos [4] suggested a design model adapted to the column base that is compatible with the socket model of Canha [2] and included the contribution of concrete strength, which is presented in this study.

## 2. Existing models for the socket foundation connection with smooth interfaces

Figure 1 illustrates the forces transfer in the connection with smooth interfaces between the column and pedestal walls.

Through the cast-in-place concrete, the moment $M_d$ and the horizontal force $V_d$ acting on the column are transmitted to the transverse walls of the socket. On the interfaces among the column and these walls, friction forces are mobilized by the pressures from the force transfer in the connection. The friction force on the frontal wall has the same direction as the axial force $N_d$, whereas on the rear wall, the direction depends on the ratio between the internal forces and the geometry; so that this direction is upward for large eccentricities and, for small eccentricities, it can be inverted. The axial force $N_d$, which is reduced by the friction forces at the column interfaces with the transverse walls, is transmitted with an eccentricity to the column base, which also mobilizes the friction forces in the column interface with the foundation base.

The behavior of column-foundation connection through socket with smooth interfaces is similar in the several methods available in the literature; however, there are variations in the parameters of the friction forces and the normal reaction at the foundation base.

This behavior model was originally idealized by Leonhardt and Mönnig [1], though the friction forces and the eccentricity $e_{nb}$ of the normal reaction $F_{nb}$ are not taken into account in their design model.

In the methods of Willert and Kesser [6] and FIB PLANCHERS OSSATURES & CERIB [7], friction forces are mobilized on the interfaces with the transverse walls and with the foundation base; however, they neglect the eccentricity $e_{nb}$ of the normal reaction $F_{nb}$ in relation to the center of the column.

**Figure 1 – Transfer of internal forces in the socket foundation with smooth interfaces – adapted from El Debs (5)**

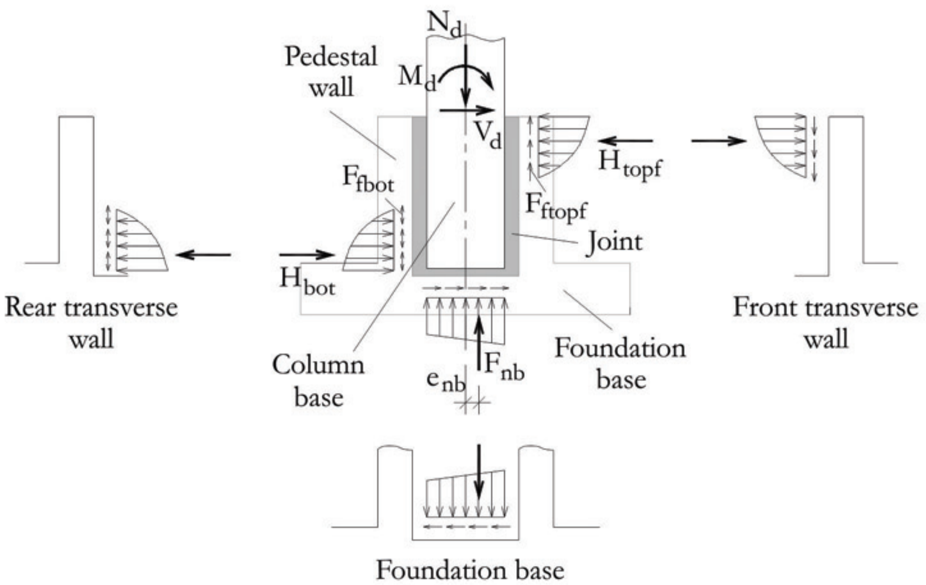
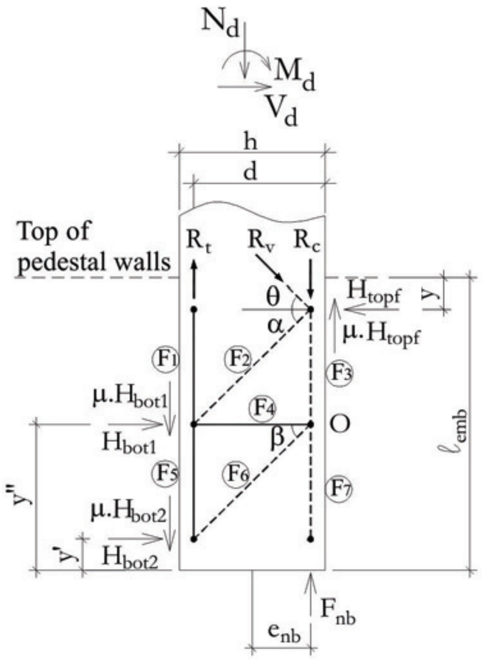


**Figure 2 – Design model for the smooth column base proposed by Canha et al. (10)**




Internal forces:

$$F_1 = \frac{M_d - N_d . e_{nb} + V_d . y}{d - 0.5h + e_{nb}}$$

$$F_2 = -\left(\frac{H_{topf} - V_d}{\cos \alpha}\right)$$

$$F_3 = -\left(N_d + R_t - H_{topf} . (\mu + \tan \alpha) + V_d . \tan \alpha\right)$$

$$F_4 = H_{bot2}$$

$$F_5 = R_t - \mu . H_{bot1} - (H_{topf} - V_d) . \tan \alpha$$

$$F_6 = -\left(\frac{H_{bot2}}{\cos \beta}\right)$$

$$F_7 = -(N_d - \mu . V_d)$$

The Elliott [8] model presents two distinct situations, depending on the action or not of the shear force $V_d$ together with the eccentric axial force $N_d$. In both cases, the friction forces are considered only on the interfaces with the transverse walls, and the axial force is transmitted with null eccentricity to the foundation base. The difference between the two situations is that when there is shear force $V_d$, the friction force is not considered on the interface where that force acts. Although the Osanai et al. [9] formulation accounts for the friction forces on the interfaces of the column with the transverse walls and with the foundation base as well as the eccentricity of the normal reaction at the column base, that model can only be applied to the cases of centered normal force and moment caused by horizontal shear force applied at the top of the column.
The most complete model is the one suggested by Canha [2], in which all the internal forces $N_d$, $M_d$ and $V_d$ are considered in addition to the friction forces on the interfaces of the column with the transverse walls and with the foundation base and the eccentricity of the normal reaction $F_{nb}$; presenting an accuracy in the balance equations to determine the connection main forces.
This model is applicable to the cases with large eccentricity of the axial force ($M_d/N_d \geq 2h$) and with embedded lengths recommended by the NBR 9062:2006 [10]. Because the friction force direction in the rear transverse wall is not well known for cases with small eccentricities ($M_d/N_d \leq 0.15h$), Campos [4] recommends neglecting all the friction forces and the eccentricity $e_{nb}$ of the normal reaction $F_{nb}$, such as the Leonhardt & Mönnig [1] model.
With respect to the column, the models mentioned above are restricted to calculate the connection main forces ($H_{top}$, $H_{bot}$ and $F_{nb}$) and do not present information about the internal behavior and the column base design.

## 3. Canha et al. [11] design model

Considering the distribution of forces and stresses path, Ebeling [3] suggests a strut and tie model in his dissertation to represent the behavior of the precast column base embedded in socket foundation. This method is presented again by Canha et al. [11], with some modifications (Figure 2).
Two compression struts are identified in the column base: one of them causes pressures at the top of the frontal wall and in the middle of the rear wall, and the other causes pressure at the bottom of the rear wall. Canha et al. [11] suggests that these inclined struts are similar to the stress paths of loads close to the beam supports. Therefore, from these stress paths, it is possible to determine the pressure resultants on the walls and the eccentricity of the reaction at the foundation base.
The tg $\alpha$ and tg $\beta$ values are calculated from equations 1 and 2, respectively:

$$\mathrm{tg}\alpha = \frac{\ell_{emb} - y - y''}{d - 0.5h + e_{nb}} \quad (1)$$

$$\mathrm{tg}\beta = \frac{y'' - y'}{d - 0.5h + e_{nb}} \quad (2)$$

where

$$y = \frac{\ell_{emb}}{6} \qquad y' = \frac{\ell_{emb}}{10} \qquad y'' = 0.5\ell_{emb} \qquad e_{nb} = 0.5h - \frac{0.8x}{2}$$

These values should be adopted so that they meet the condition that the angle formed between the diagonal struts axes and the cords is between 18.4° and 45°, according to the dimensioning criteria of the code model CEB-FIP MC-90 [12].

The $R_t$, $R_v$ and $R_c$ values are obtained by the bending-compression theory used to calculate the columns and are presented in equations 3, 4 and 5, respectively:

$$R_t = \frac{M_d - N_d \cdot e_{nb} + V_d \cdot y}{d - 0.5h + e_{nb}} \tag{3}$$

$$R_v = \frac{V_d}{\cos\theta} \tag{4}$$

$$R_c = N_d + R_t - V_d \cdot \tan\theta \tag{5}$$

The problem has three equilibrium equations: two of forces in the x and y directions and one of moment in the z direction related to point O of Figure 2, which are presented, respectively, in equations 6, 7 and 8:

$$H_{bot1} + H_{bot2} = H_{top} - V_d \tag{6}$$

$$F_{nb} = N_d - \mu . V_d \tag{7}$$

$$tg\alpha \cdot H_{top} + \mu(H_{bot1} + H_{bot2}) + tg\beta \cdot H_{bot2} - \frac{M_d - N_d \cdot e_{nb} + V_d \cdot y}{d - 0.5h + e_{nb}} - tg\alpha \cdot V_d = 0 \tag{8}$$

The problem is statically indeterminate because there are three equations of equilibrium and four unknowns to be determined. In this way, static compatibility was used to solve the problem by defining the pressures $H_{bot1}$ and $H_{bot2}$ as percentages of pressure $H_{top}$ reduced from the shear force $V_d$ and they are given by the equations below:

$$H_{bot1} = (1 - \eta) \cdot (H_{top} - V_d) \tag{9}$$

$$H_{bot2} = \eta \cdot (H_{top} - V_d) \tag{10}$$

The necessary combinations resulted in equation 11 to calculate $H_{top}$:

$$H_{top} = \frac{\frac{M_d - N_d \cdot e_{nb}}{d - 0.5h + e_{nb}} + V_d \cdot \left(\mu + tg\alpha + \eta \cdot tg\beta + \frac{y}{d - 0.5h + e_{nb}}\right)}{tg\alpha + \mu + \eta \cdot tg\beta} \tag{11}$$

The above equation can be expressed as function of $R_t$, that results in the following expression for pressure $H_{top}$:

$$H_{top} = \frac{R_t + V_d \cdot (\mu + tg\alpha + \eta \cdot tg\beta)}{tg\alpha + \mu + \eta \cdot tg\beta} \tag{12}$$

The coefficient $\eta$ is the coefficient that ponders the values of the bottom pressures and represents a percentage of $H_{top}$ reduced from the shear force $V_d$. This coefficient depends on the friction coefficient m and is calculated from equation 13:

$$\eta = 0{,}42 \cdot e^{-1{,}64.\mu} \tag{13}$$

Equation 13 was defined based on the representation of the proposed strut and tie model by a hyperstatic truss, applying the forces $R_t$, $R_v$ and $R_c$, obtained from the ultimate forces of the tests, and with the reactions $H_{bot1}$, $H_{bot2}$, $H_{top}$ and $F_{nb}$ acting at the supports of that truss, as shown in Figure 3(a). The friction coefficient $\mu$ was varied for the two tested specimens CB-1 and CB-2. From the curves $\mu$ x $\eta$ of the two specimens, the mean curve presented in Figure 3(b) was obtained, and equation 13 was determined.

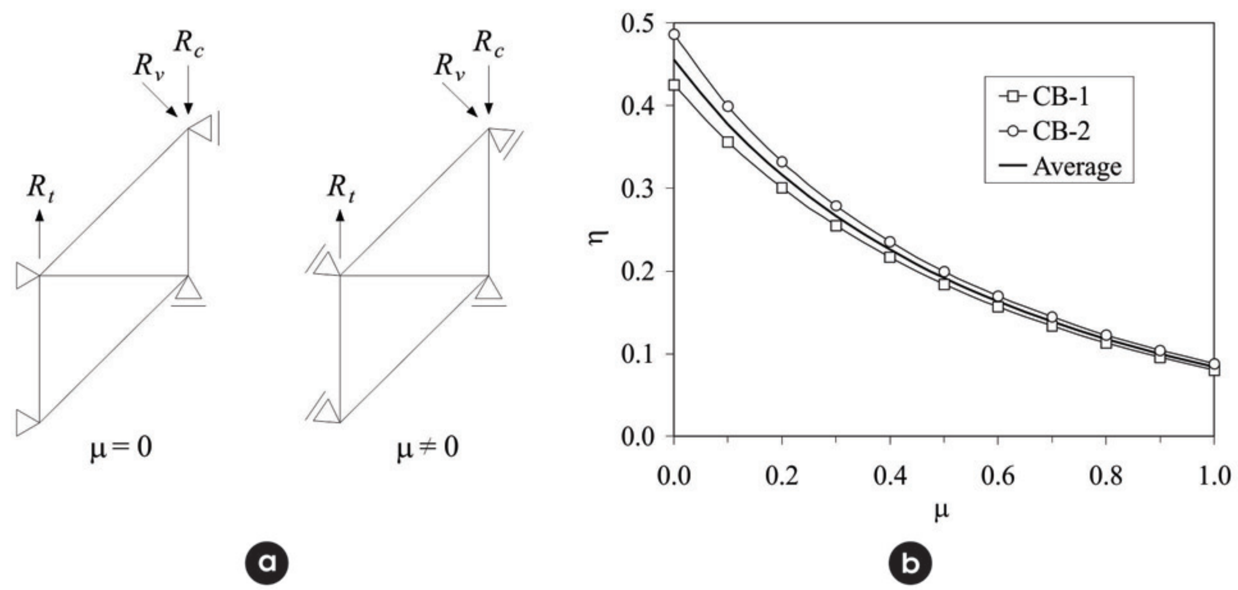
The proposed model is valid for cases with large eccentricity and for embedded lengths between 1.6 h to 2.0 h. The coefficient $\eta$ is function of the friction coefficient m, which is equal to 0.3 for smooth interfaces. Therefore, applying equation 13, the coefficient $\eta$ is 0.26.

The internal forces of the model are determined from the analysis of the truss equilibrium of Figure 2 and are presented in this same figure. After determining the force $F_4$, the transverse reinforcement can be calculated. This reinforcement has to be distributed in the column base area equivalent to the distance of y”- y’. For this method, the friction force at the base was not considered because the strains of the stirrup in that region were small.

## 4. Proposed design model

Analyzing the design model proposed by Canha et al. [11] for the precast column base, which is an adaptation from the one

Figure 3 – Determination of $\eta$: (a) variation of $\mu$; (b) curve $\mu$ x $\eta$ (Canha et al. (10))




proposed by Ebeling [3], it is verified that there is an incompatibility with the proposed model of Canha [2] for the dimensioning of the socket foundation with smooth interfaces, as presented in Figure 4. In the strut and tie method of the column, presented in Figure 2, there are two resultants of bottom pressure $H_{bot}$ at the tension side of the column, whereas in the socket design model, there is only one resultant $H_{bot}$ at the rear wall. In addition, the model shown in Figure 2 does not consider the friction forces at the column base, different from the design model of the design model of the socket foundation.

Thus, this article presents a new design model adapted to the column base analysis. In addition to the removal of the intermediate support, the inclusion of the friction forces at the column base is also analyzed so that this method becomes similar to the behavior model of the socket foundation.

Another observation is that the proposed strut and tie model suggested by Ebeling [3] does not consider the portion supported by the concrete when determining the forces and later the dimensioning of the transverse reinforcement. Therefore, the contribution of the concrete in the connection strength must be considered so that the model of the column base represents the behavior in that region well. Furthermore, the concrete in the embedded region is confined, such that the portion of strength of the concrete to the shear force is even higher.

The adapted model with the proposed alterations is represented in Figure 5.

The signs were included in the equation to determine the internal forces; the positive sign is for tension, and the negative sign is for compression.

The values of y, y' and $e_{nb}$ are calculated from equations 14, 15 and 16, respectively:

$$y = \frac{\ell_{emb}}{10} \tag{14}$$

$$y' = \frac{\ell_{emb}}{10} \tag{15}$$

$$e_{nb} = 0.5h - \frac{0.8x}{2} \tag{16}$$

For practical issues, the eccentricity $e_{nb}$ at the column base can be defined by equation 17, as in the foundation socket model.

$$e_{nb} = \frac{h}{4} \tag{17}$$

The resultants $R_t$, $R_v$ and $R_c$ are defined by equations 18, 19 and 20, respectively:

$$R_t = \frac{M_d - N_d \cdot e_{nb} + V_d \cdot y}{d - 0.5h + e_{nb}} \quad (18)$$

$$R_v = \frac{V_d}{\cos\theta} \quad (19)$$

$$R_c = N_d - V_d \cdot tg\theta + R_t \quad (20)$$

The inclination angle $\alpha$ of the struts in relation to the reinforcements is determined by equation 21:

$$tg\alpha = \frac{\ell_{emb} - y - y'}{2(d - 0{,}5.h + e_{nb})} \quad (21)$$

The problem, now statically determined and with three unknowns ($H_{top}$, $H_{bot}$ e $F_{nb}$), can be solved with the three equations of equilibrium, two of forces in the x and y directions, and one of moment in the z direction relative to point O of Figure 5, which are presented, respectively, in equations 22, 23 and 24:

$$H_{top} - H_{bot} - \mu \cdot F_{nb} - V_d = 0 \quad (22)$$

$$N_d - \mu.V_d - (1 + \mu^2) F_{nb} = 0 \quad (23)$$

**Figure 4 – Design model for smooth socket proposed by Canha (2)**

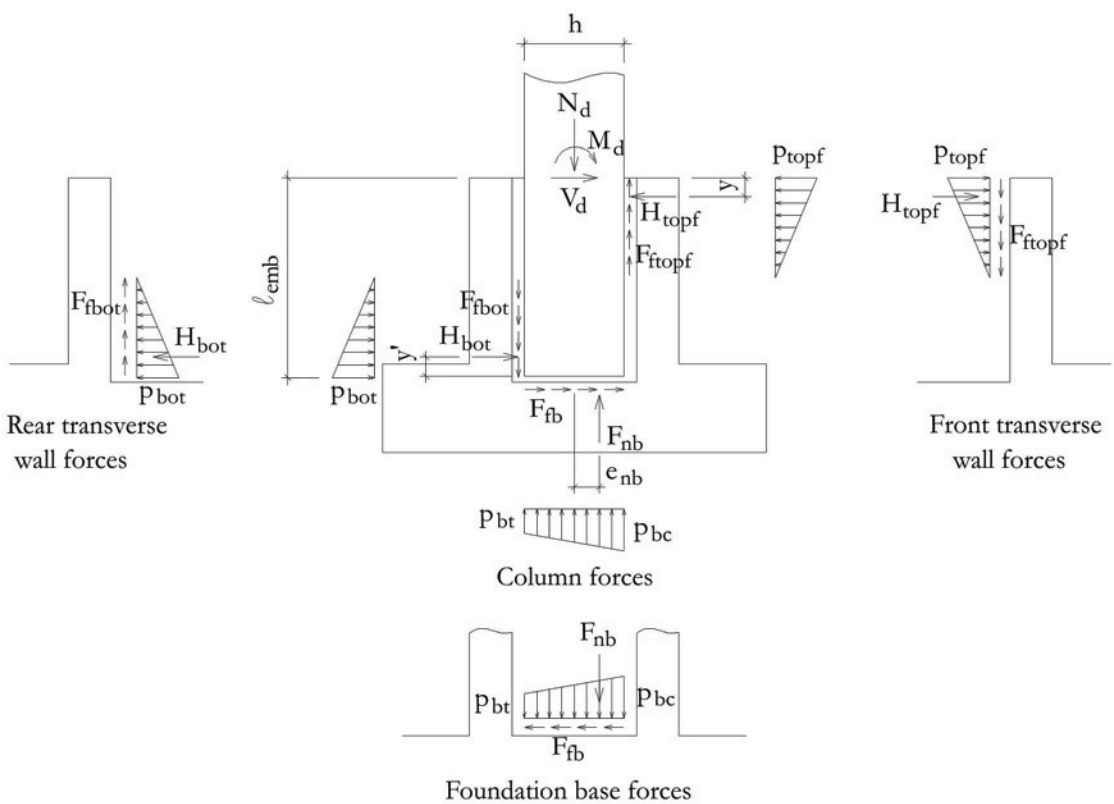
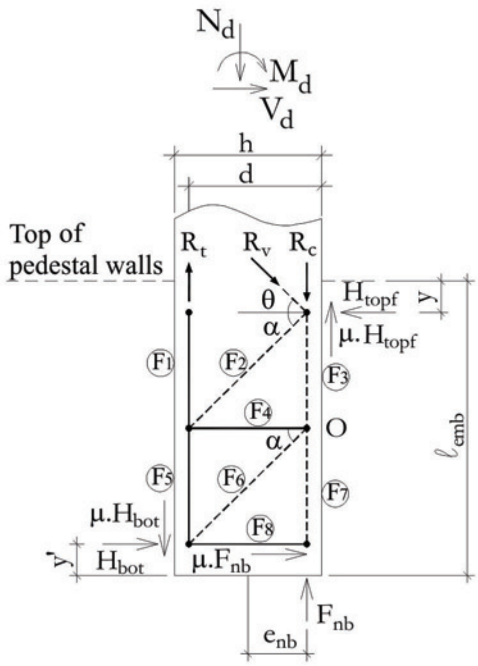


**Figure 5 – Adapted design model proposed for the analysis of the precast column base (Campos (4))**




Internal forces:

$$F_1 = \frac{M_d - N_d . e_{nb} + V_d . y}{d - 0.5h + e_{nb}}$$

$$F_2 = -\left(\frac{H_{topf} - V_d}{\cos\alpha}\right)$$

$$F_3 = -\left(N_d + R_t - H_{topf} . (\mu + \tan\alpha) + V_d . \tan\alpha\right)$$

$$F_4 = H_{bot}$$

$$F_5 = (\mu + \tan\alpha) . H_{bot}$$

$$F_6 = -\left(\frac{H_{bot}}{\cos\alpha}\right)$$

$$F_7 = -\left(\frac{N_d - \mu . V_d}{1 + \mu^2}\right)$$

$$F_8 = \frac{\mu . N_d - \mu^2 . V_d}{1 + \mu^2}$$

$$tg\alpha \cdot H_{top} + (\mu + tg\alpha) \cdot H_{bot} + \mu \cdot tg\alpha \cdot F_{nb} - tg\alpha \cdot V_d - \frac{M_d - N_d \cdot e_{nb} + V_d \cdot y}{d - 0.5h + e_{nb}} = 0 \quad (24)$$

After combining these equations of equilibrium, the frontal transverse wall pressure $H_{top}$, the rear transverse wall pressure $H_{bot}$ and the normal reaction at the foundation base $F_{nb}$ are given, respectively, by equations 25, 26 and 27.

$$H_{top} = \frac{\dfrac{M_d}{d - 0.5h + e_{nb}} + N_d \cdot \left(\dfrac{\mu^2}{1+\mu^2} - \dfrac{e_{nb}}{d - 0.5h + e_{nb}}\right) + V_d \cdot \left(\dfrac{\mu}{1+\mu^2} + \dfrac{y}{d - 0.5h + e_{nb}} + 2tg\alpha\right)}{\mu + 2tg\alpha} \quad (25)$$

$$H_{bot} = H_{top} - \frac{\mu \cdot N_d + V_d}{1 + \mu^2} \quad (26)$$

$$F_{nb} = \frac{N_d - \mu \cdot V_d}{1 + \mu^2} \quad (27)$$

This calculation model is recommended for column bases embedded in socket foundations with smooth interfaces that are submitted to a normal load of large eccentricity and with embedded lengths determined in compliance with NBR 9062:2006 [10], because the Ebeling [3] study showed that the physical specimen with embedded length smaller than that recommended presented larger displacements and stresses in the reinforcements.

Canha [2] proved the importance of considering friction forces when determining the pressures because this conception approximates the theoretical and experimental values.

For the cases in which the normal load results in small eccentricity, the design model that considers neither the friction forces nor the eccentricity of the normal reaction at the column base is recommended. The forces $H_{top}$, $H_{bot}$ and $F_{nb}$ are, for this situation, calculated from equations 28, 29 and 30, respectively:

$$H_{top} = \frac{\dfrac{M_d}{d - 0.5h} + V_d \cdot \left(\dfrac{y}{d - 0.5h} + 2tg\alpha\right)}{2tg\alpha} \quad (28)$$

$$F_{nb} = N_d \quad (30)$$

Relative to the anchorage of the longitudinal reinforcement of the column, the analysis of the strains in the embedded region of column showed that, at distances of $0.5l_{emb}$ and $0.6l_{emb}$ from the base to the top of pedestal walls, the strains were close to the yield ones for the two specimens tested by Ebeling [3]. These values show that the transfer of stresses from the reinforcement to the concrete occurs from these points in the downward direction.

Therefore, the recommendation presented in the Leonhardt &

Mönnig [1] model shows that the anchorage length given by equation 31 is valid:

$$\ell_{anc} = \frac{\ell_{emb}}{2} \tag{31}$$

## 5. Results analysis of the proposed design model

An adapted strut and tie method was proposed in this study in order to make the compatibility of the behavior model proposed by Canha [2] for the socket foundation with the model of the precast column base presented in Canha et al. [11]. The alterations were the removal of one of the supports in the tension side of the column was removed, and the inclusion of the friction forces at the precast column base was verified. To quantify the difference between the two cases, considering these forces or not, a calculation example was performed and its results are shown in Table 1.

To calculate this example, the loading used was the one observed in the tests of Ebeling [3], which was 242 kN of normal force and 290 kN·m of bending moment. There was no shear force applied in the tested specimens. By choosing this load, it was possible to compare the theoretical results of the model with the experimental results. This particular load generates a situation with large eccentricity of the normal load.

The cross section of 40 cm x 40 cm for the precast column and the following parameters were adopted:

a) Joint with 5 cm and embedded length for large eccentricity in compliance with NBR 9062:2006 [10] (smooth interfaces: $l_{emb}$ = 2.0 h);
b) Friction coefficient μ=0.3;
c) Eccentricity of the normal reaction at the base $e_{nb}$ = h/4;
d) Distance of application of $H_{top}$ and $H_{bot}$ determined by y = y' = $l_{emb}$ / 10;
e) Average compression strength of concrete $f_{cm}$ = 54 MPa (strength of precast column tested by Ebeling [3]);
f) Yield strength for longitudinal reinforcement of 580 MPa and for transverse reinforcement of 613 MPa (values characterized in Ebeling [3]);
g) The inclination angle $\alpha$ of the struts relative to the reinforcements, resulting of this situation, is 49.8°.

The internal forces shown in Table 1 were determined by the equations described in Figure 5.

**Table 1 – Analysis of the precast column base**

| | | Situations | |
|---|---|---|---|
| | Variables | Considering friction forces at the base | Without friction forces at the base |
| Pressures (kN) | $H_{top}$ | 376 | 369 |
| | $H_{bot}$ | 310 | 369 |
| | $F_{nb}$ | 222 | 242 |
| Tension and compression resultants (kN) | $R_v$ | 0 | 0 |
| | $R_t$ | 984 | 984 |
| | $R_c$ | 1226 | 1226 |
| Internal forces (kN) | $F_1$ | + 984 | + 984 |
| | $F_2$ | - 583 | - 572 |
| | $F_3$ | - 668 | - 679 |
| | $F_4$ | + 310 | + 369 |
| | $F_5$ | + 460 | + 548 |
| | $F_6$ | - 480 | - 572 |
| | $F_7$ | - 222 | - 242 |
| | $F_8$ | + 67 | - |

Convention: + for tension forces and – for compression forces

The analysis of the results shows that the resultant $H_{top}$ is close in the two situations, and it is only 2% higher when considering the friction forces at the base. When the shear force $V_d$ and the friction forces at the base are absent, the resultants $H_{top}$ and $H_{bot}$ are equal, as well as the reactions $F_{nb}$ and $N_d$.
The tension and compression resultants have the same value in the two situations, and the resultant $R_v$ is null, because there is no horizontal force.
The values of the internal forces were slightly different, and the main difference was the existence of the force $F_8$ when considering the friction force at the base. The force $F_4$, which is the reinforcement force at half of the embedded length, was approximately 20 % higher when there was no friction force at the base because it is the only area with transverse reinforcement.
In general, it is possible to affirm that the inclusion of the friction force at the base does not significantly influence the dimensioning of the precast column base. Therefore, in order to make the compatibility between the socket and precast column models and present a reinforcement distribution in the whole embedded length of column, the model, for cases with large eccentricity, should account for the friction force at the base. However, when the normal load results in small eccentricity, the model that does not consider any friction force should be adopted for the column design, as is recommended for the socket design.
To calculate the column transverse reinforcement in the embedded length, the portion supported by concrete must still be determined to reduce the force in the ties. In this example, since the average compression strength of concrete is $f_{cm}$ = 54 MPa, the portion supported by concrete is $V_c$ = 266 kN.
Given the values of the force in the tie $F_4$, presented in Table 1, and the portion $V_c$, it is possible to establish the force that should be supported by the reinforcement at half of the embedded length. After the calculations, the force was 43 kN, which is the force the reinforcement has to resist. This result is equivalent to a reinforcement of only 3.10 cm²/m.
Analyzing the results presented in Canha et al. [11], it is observed that the experimental force in the tie $F_4$, measured by the strain gage, was 40 kN, whereas the resulting force calculated by that model was of 147 kN, considering the friction coefficient μ=0.3.
Comparing the theoretical forces resulting from the application of both models with the obtained experimental force, the proposed model in this study provides a result more compatible with the experimental value and is therefore more appropriate for the precast column base analysis.
As the precast column in this area has to be reinforced with a transverse reinforcement either equal or superior to the minimal reinforcement, it has to be calculated and compared to the reinforcement resulting from the dimensioning. For this case, the minimum reinforcement is 6.90 cm²/m, and that obtained from the adapted model is 3.10 cm²/m. Therefore, in this case, the minimum reinforcement has to be adopted in the embedded region of the precast column in the socket foundation.

## 6. Final remarks and conclusions

After analyzing the precast column base method presented in Canha et al. [11] and the socket model proposed in Canha [2] for smooth interfaces, the two are incompatible due to the bottom pressures resultants on the wall of the tension side of the connection (socket rear wall) and the friction force on the foundation base.
Moreover, the model presented in Canha et al. [11] for the smooth precast column base does not take into account the contribution of the concrete in the connection strength. Thus, the formulation proposed in Campos [4] and presented in this study for the column base design was compatible with the socket model of Canha [2], with the addition of the concrete contribution to the connection strength, which was not previously included in the model of Canha et al. [11].
To confirm the new model proposed for the column base, the theoretical results were compared with the experimental results of the specimens of Ebeling [3], and two situations were analyzed: one that considers the friction forces at the base and another that neglects them.
After analyzing these results, some conclusions can be drawn:

a) The pressure resultant of the frontal wall $H_{top}$ was very close in both situations, and is only 2% higher when the friction forces are considered at the base;
b) When the shear force $V_d$ is null and the friction forces at the base are not considered, the pressure resultants of the frontal wall $H_{top}$ and of the rear wall $H_{bot}$ are the same, as well as the reactions at the base of the foundation $F_{nb}$ and the normal force $N_d$ are also the same;
c) The values of the internal forces of the strut and tie model were relatively close, where the main difference is the existence of a tie at the base when the friction forces at the base are considered;
d) The reinforcement force in the middle of the embedded length was around 20% higher when there was no friction force at the base;
e) Although the existence of friction forces at the base does not significantly affect the internal forces values at the column base, the new model recommended for the cases with large eccentricity accounts for these forces so that it is compatible with the socket method and also presents a reinforcement distribution along the whole embedded length of the column;
f) For the case of small eccentricity, it is recommended that no friction force be considered for both the socket and the column design;
g) The model proposed in this study produced results that are much closer to the experimental values than the method of Canha et al. [11]. For the tie $F_4$, the force experimentally obtained was 40 kN, while the theoretical forces were 147 kN for the method presented in Canha et al. [11] and 43 kN for the model proposed in this study. Therefore, the proposed model is the most appropriate one for the design of the smooth precast column base.

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