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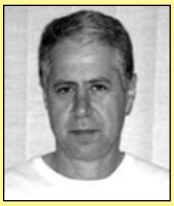
Proposal of design model for column-base connection by socket of precast concrete structures

Proposta de modelo de projeto para a ligação pilar-fundação por meio de cálice em estruturas de concreto pré-moldado



R. M. F. CANHA[a]
rejane_canha@yahoo.com.br

M. K. EL DEBS[b]
mkdebs@sc.usp.br

Abstract

This paper presents theoretical-experimental results of column-foundation connection by socket of precast concrete structures, with emphasis on pedestal walls. The experimental program included five models subjected to normal load with large eccentricity, changing the type and condition of interface: three models had smooth interface with embedded length 2 times the height of column cross section, and two models had rough interface with embedded length 1.6 times the height of column cross section. In two of smooth models, the bond was eliminated to represent the more appropriate situation of design. Two different shear key configurations were used in rough models. The experimental results indicated the need to revalue the principal design methods for this connection. That way, for the embedded length not less than that used in these tests, a design model was proposed for the smooth socket connection, considering friction between the interfaces, and the design of longitudinal walls was suggested as corbels. As the proximity of rough physical models behavior with a monolithic connection was verified, the design of their vertical reinforcements is recommended, admitting the total transfer of internal forces.

Keywords: connection, socket base, precast concrete, pedestal walls, design model.

Resumo

Este trabalho apresenta resultados teórico-experimentais da ligação pilar-fundação por meio de cálice em estruturas de concreto pré-moldado, com ênfase no colarinho. No programa experimental, foram ensaiados cinco protótipos sob força normal com grande excentricidade, variando-se o tipo e condição de interface: três com interface lisa, com comprimento de embutimento de 2 vezes a altura da seção transversal do pilar, onde em dois foi eliminada a adesão para representar a situação mais adequada de projeto; e dois com interface rugosa, com comprimento de embutimento de 1,6 vezes a altura da seção transversal do pilar, com duas configurações diferentes de chaves de cisalhamento. Os resultados experimentais indicaram a necessidade de se reavaliar os principais métodos de projeto para essa ligação. Desta forma, para comprimento de embutimento do pilar não inferior àqueles empregados nos ensaios, é proposto, para o cálice liso, um modelo de projeto considerando o atrito entre as interfaces e o cálculo das paredes longitudinais como consolos. Já para o cálice com interface rugosa, como foi verificada a proximidade do comportamento dos protótipos físicos rugosos com uma ligação monolítica, recomenda-se o dimensionamento de suas armaduras verticais, admitindo a transferência total dos esforços.

Palavras-chave: ligação, cálice de fundação, concreto pré-moldado, colarinho, modelo de projeto.

[a] *Department of Structural Engineering EESC, USP, rejane_canha@yahoo.com.br, Av. Trabalhador São-carlense – nº 400 – SET/EESC/USP – Centro – 13566-590, São Carlos/SP, Brazil;*

[b] *Department of Structural Engineering, EESC, USP, mkdebs@sc.usp.br, Av. Trabalhador São-carlense – nº 400 – SET/EESC/USP – Centro – 13566-590, São Carlos/SP, Brazil.*

1 Introduction

Despite the fact that socket base is quite used in the whole world and it is the column-foundation connection more employed in precast concrete structures of Brazil, a gap still exists to be filled between the design models and a theoretical model more consistent based on experimental researches. Besides, bibliography about this subject is little and experimental results specifically related to the behavior of pedestal walls do not exist.
Based on this, CANHA [1] made a theoretical-experimental research of this connection, in way to contribute for academic professionals, with the solution and more realistic explanation of the problem; and for technical professionals, with subsidies for the appropriate design meeting the criteria of safety and economy.
Cases of load with large eccentricity were evaluated, because they are the most common in precast concrete structures. Except investigation of OSANAI *et al.* [2] about the connection subjected to horizontal and sloped force with a large distance in relation to the top of pedestal walls, well-known theoretical-experimental researches about this connection just focus their behavior due to centered load or load with small eccentricity.
Another aspect included in this work refers to the percentage of conservatism contained in models of LEONHARDT & MÖNNIG [3] and NBR-9062/85 [4] for design of this connection. The friction portion that contributes to the strength of smooth socket base was verified, through the comparison among theoretical and experimental results. Since then, a design model that induces to a smaller amount of reinforcement is proposed.
Confirming the behavior of rough socket base as a monolithic connection was necessary. In this way, the adaptation of the bending theory for design of this socket was verified, from experimental results.

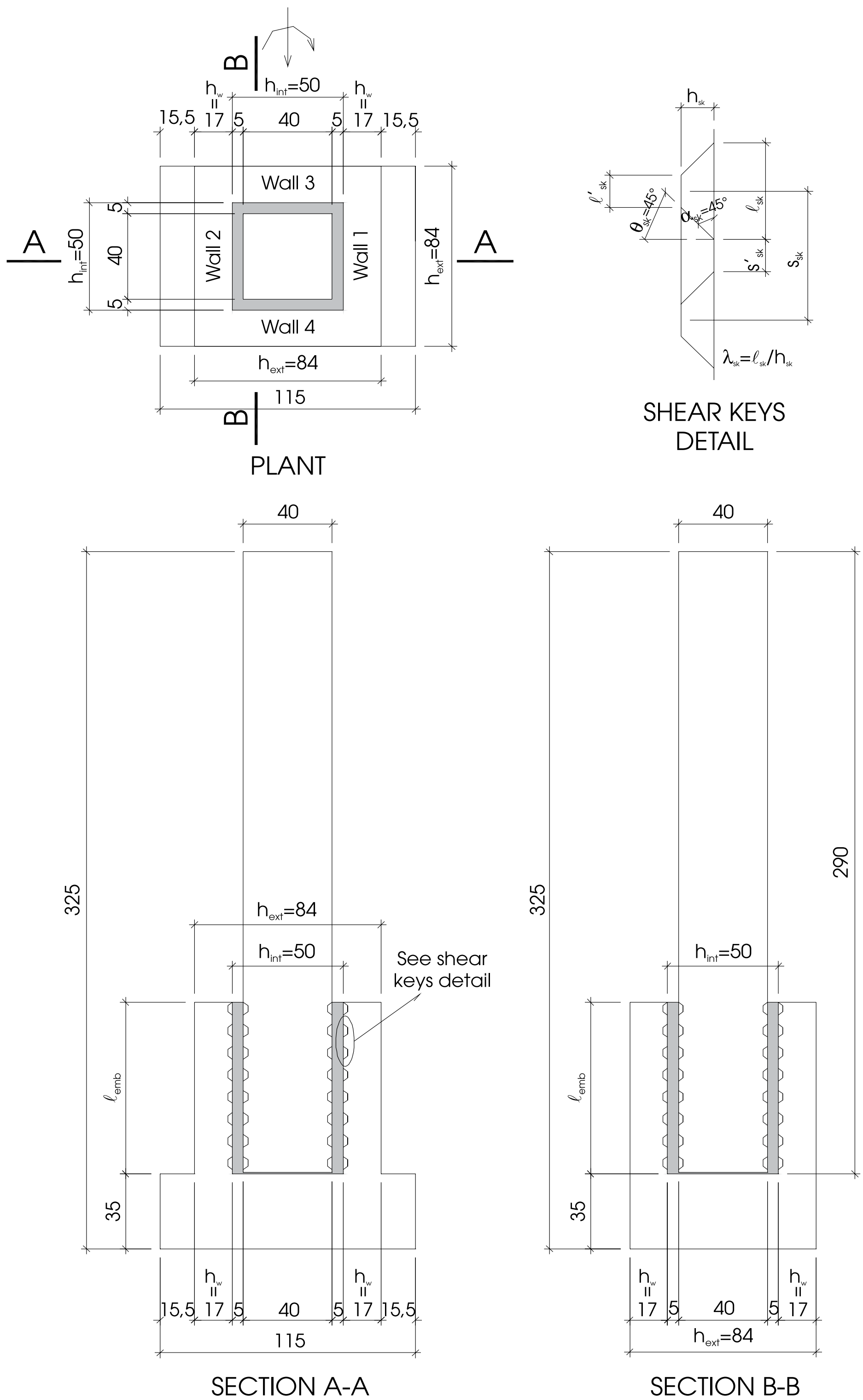
2 Experimental program

The experimental program included 5 specimens subject to load with large eccentricity. Three specimens had smooth interface in contact with cast-in-place concrete and two specimens had rough interface.
The specimens were built with scale $1:1$ and their geometry was chosen from the column cross section of $40\ cm \times 40\ cm$, which is practically the minimum dimension used in precast concrete structures. The minimum value of thickness of the pedestal wall (h_w) suggested by LEONHARDT & MÖNNIG [3] model was used. This value is equal to a third of inner distance among pedestal walls ($h_{int} / 3$). The embedded length values of $2h$ for smooth interface and $1.6h$ for rough interface indicated by the Brazilian code, NBR-9062/85 [9], were adopted. These embedded lengths seem to be enough for the transfer of efforts in connection with smooth and rough interfaces. The design of pedestal walls reinforcements was done according to the LEONHARDT & MÖNNIG [3] model, recommendations of NBR-9062/85 [4] and those presented in EL DEBS [5] for corbels.
Figure 1 and table 1 present, respectively, the nomenclature and the summary of dimensions of tested specimens, which compose series SI (Smooth Interface) and RI (Rough Interface). The reinforcements of socket base of these specimens are illustrated in figures 2 and 3. Some stages of the specimens casting and tests assembly of series SI are shown on figure 4. Figure 5 presents one of the specimens with the elements that make a self-balanced test system. The designed test set-up is indicated in figure 6.
Specimen SI1 was built with normal casting of joint, in way to verify the total transfer of moment and normal force from column to socket base. In specimens SI2 and SI3, oil was applied at interfaces in order to avoid the bond. This situation would be more suitable for design, because the perfect contact among interfaces is not guaranteed and due to the cast-in-place concrete shrinkage and alternate load from the wind, the bond among elements can be lost. Due to the limitation of load capacity of hydraulic actuator, the first two specimens (SI1 and SI2) were tested with a larger eccentricity e ($1.85\ m$) in order to guarantee the socket base rupture. After the experimental strength of connection SI2 was known, the eccentricity of specimen SI3 was reduced to $1.20\ m$. The same treatment to avoid the bond at the interface was used in specimens RI1 and RI2, in order to compare their behavior with specimen SI3. For rough interface specimens, an eccentricity e of $1.20\ m$ was initially used, but due to limitations of the actuator load capacity, this eccentricity e was increased to $1.85\ m$, in order to better characterize the rupture of these specimens. All columns of

Table 1 – Summary of geometric characteristics of tested specimens

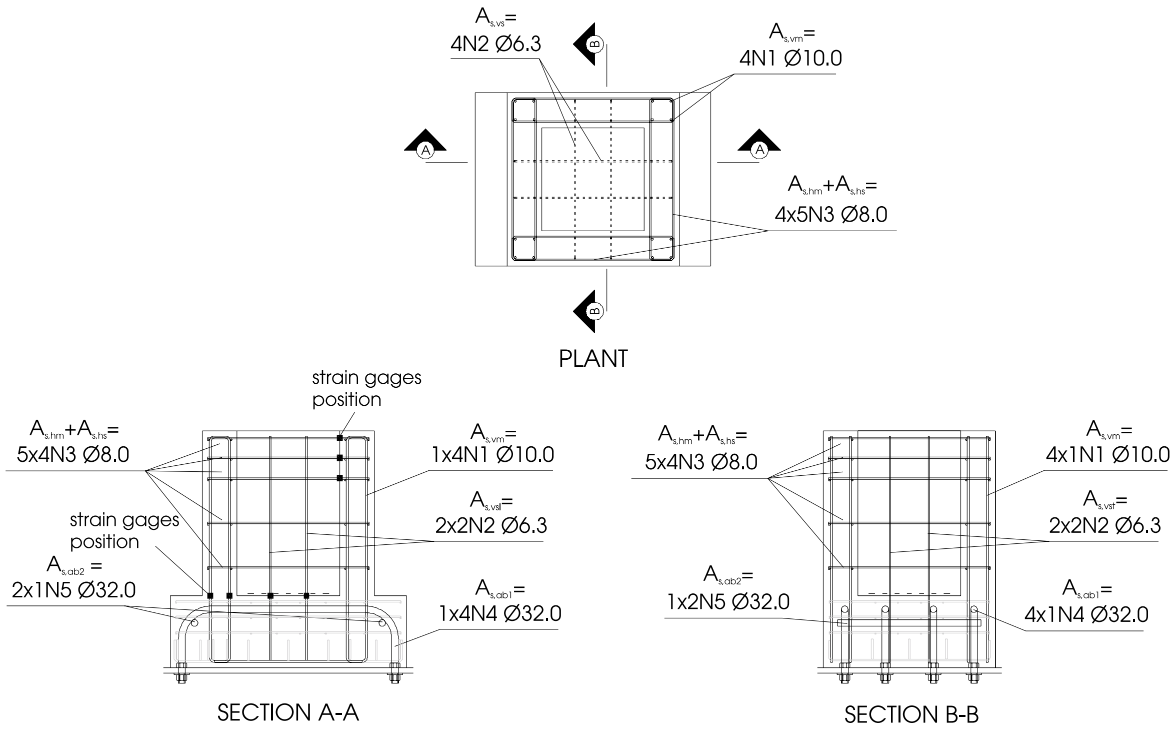
Series	Specimen	Interface	Interface condition	e (cm)	ℓ_{emb} (cm)	h_w (cm)	α_{sk}	h_{sk} (cm)	ℓ_{sk} (cm)	s'_{sk} (cm)
SI	SI1	Smooth	Adherent	185	80	17	-	-	-	-
	SI2	Smooth	Without bond and with friction							
	SI3	Smooth	Without bond and with friction	120						
RI	RI1	Rough	Without bond and with friction	120/	64	17	45°	1	6	4
	RI2	Rough	Without bond and with friction	185					3	1

Figure 1 – Nomenclature of dimensions adopted for tested specimens



Obs.: dimensions in cm

Figure 2 – Socket base reinforcement of specimens of series SI



Obs.: Reinforcements diameters in mm.

the specimens were designed for the load equivalent to the maximum hydraulic actuator capacity ($500\ kN$) with an eccentricity e of $1.85\ m$, in order to assure the socket base rupture.
The dimensions and spacing among shear keys of rough interface specimens were changed: the first specimen with the minimum roughness recommended by NBR-9062/85 [4] ($1\ cm$ to each $10\ cm$ of joint), and the other specimen with a larger roughness ($1\ cm$ to each $4\ cm$). The main purpose of this alteration is to analyze if there is a total transfer of efforts among the connection elements and if the strength of the connections would change due to the difference between the two shear keys geometry.
The objective of this study of interfaces was to propose recommendations and a more reasonable design model of the connection, taking into account the friction for smooth socket base and a monolithic behavior for rough socket base.
The main experimental results are presented in table 2. The strength of the specimens was determined by the ultimate load resisted by socket base, since the column was designed for the maximum capacity of actuator. The rupture of all specimens occurred with the yielding of main vertical reinforcements and the yielding of secondary vertical reinforcements, according to the specimen. The forces transmitted by reinforcements were obtained based on their stress, and this stress was calculated with the average values of strain gage of each reinforcement. In this way, when the strain was greater than the yield one, the average yield strength f_{ym} was used to determinate the reinforcement force.

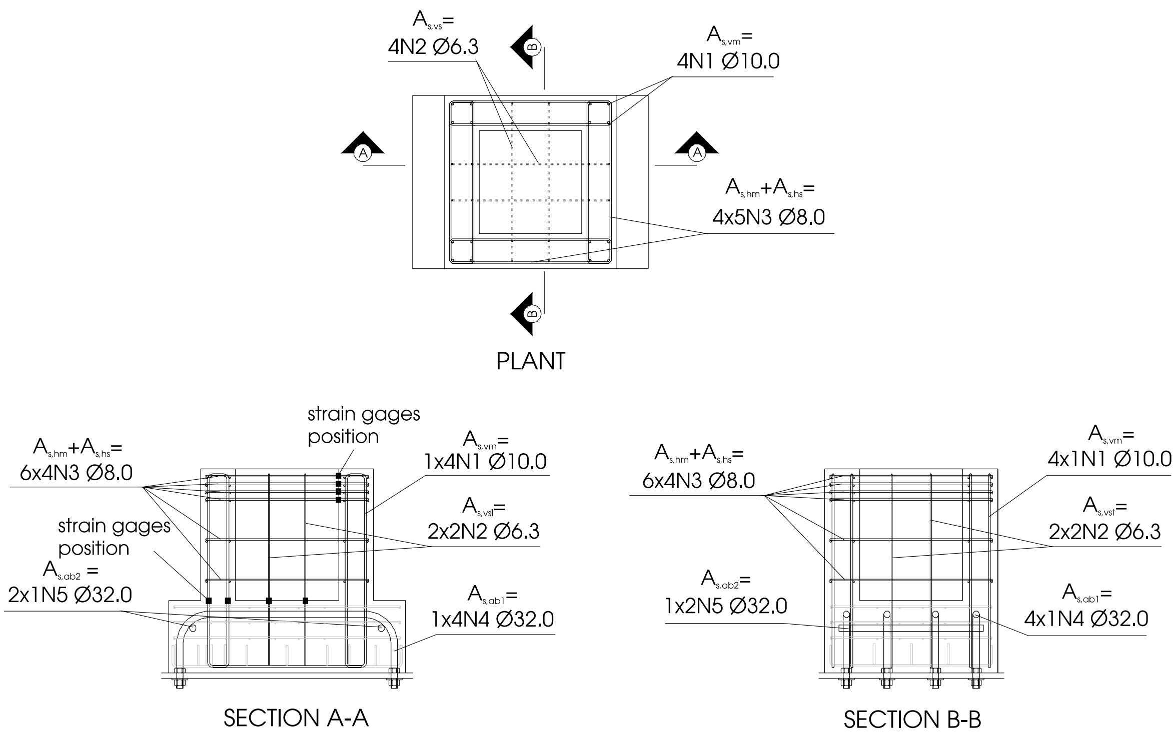
3 Proposal of design model

3.1 Socket base with smooth interface

From the experimental results, a design model is proposed for socket base with smooth interface, which takes into account the contribution of the friction forces $F_{fri,top,d}$, $F_{fri,bot,d}$ and $F_{fri,b,d}$ and the eccentricity e_{nb} of reaction $F_{nb,d}$ on the column base. The design of connection forces is presented in figure 7.
The top, bottom and foundation base friction forces are defined by the multiplication of the friction coefficient for corresponding normal force, according to the following equations:

$$F_{fri,top,d} = \mu . H_{top,d} \quad (1)$$

Figure 3 – Socket base reinforcement of specimens of series RI



Obs.: Reinforcements diameters in mm.

$$F_{fri,bot,d} = \mu . H_{bot,d} \tag{2}$$

$$F_{fri,b,d} = \mu . F_{nb,d} \tag{3}$$

From equilibrium conditions, the following equations can be obtained:

• Equilibrium of vertical forces:

$$N_d - F_{nb,d} - F_{fri,top,d} + F_{fri,bot,d} = 0 \tag{4}$$

• Equilibrium of horizontal forces:

$$V_d + F_{fri,b,d} - H_{top,d} + H_{bot,d} = 0 \tag{5}$$

• Equilibrium of moments at point O:

$$M_d - N_d . e_{nb} + V_d . \ell_{emb} - H_{top,d} . (\ell_{emb} - y) + H_{bot,d} . y' - F_{fri,top,d} . (0.5h - e_{nb}) + - F_{fri,bot,d} . (0.5h + e_{nb}) = 0 \tag{6}$$

Figure 4 – Casting and assembly stages of specimens of series SI

1st) Assembly of socket base reinforcement

2nd) Casting of socket base

3rd) Assembly of column reinforcement

4th) Casting of column

5th) Raising of socket base for transition piece

6th) Application of oil on socket base and column of specimens SI2 and SI3

7th) Raising and temporary fixation of column on socket base

8th) Raising of specimen for reaction steel base

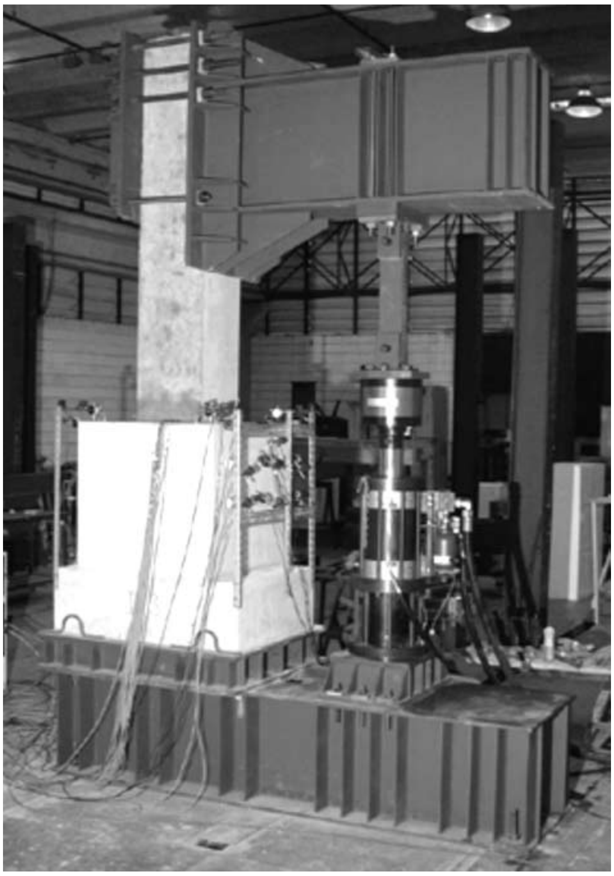
9th) Joining of column

10th) Casting of connection joint of elements

11th) Assembly of other steel pieces

12th) Ready specimen to be tested

Figure 5 – Test of one specimen



Combining equations (4) and (5) and replacing the values of equations (1) to (3), the force $F_{fri,b,d}$ and $H_{bot,d}$ results in:

$$F_{fri,b,d} = \frac{\mu.N_d - \mu^2.V_d}{1+\mu^2} \tag{7}$$

$$H_{bot,d} = H_{top,d} - \frac{\mu.N_d + V_d}{1+\mu^2} \tag{8}$$

Then, the values of $H_{bot,d}$, $F_{fri,top,d}$ and $F_{fri,bot,d}$ are substituted in equation (6), that results in expression (9) for calculation of $H_{top,d}$:

$$H_{top,d} = \frac{M_d - N_d\left(e_{nb} + \frac{\mu.y' - \mu^2(0.5h+e_{nb})}{1+\mu^2}\right) + V_d\left(\ell_{emb} - \frac{y' - \mu.(0.5h+e_{nb})}{1+\mu^2}\right)}{\ell_{emb} - y - y' + \mu.h} \tag{9}$$

The main difference of this proposed model related to the OSANAI *et al.* [2] modified model, presented in CANHA [1], is that the first model takes into account the friction force $F_{fri,b,d}$ acting on the column base for assembly of moment in expression (6), and the last model is simplified, that is, the bottom horizontal force $H_{bot,d}$ and the friction force $F_{fri,b,d}$ act on height y'' equivalent to half of y'.

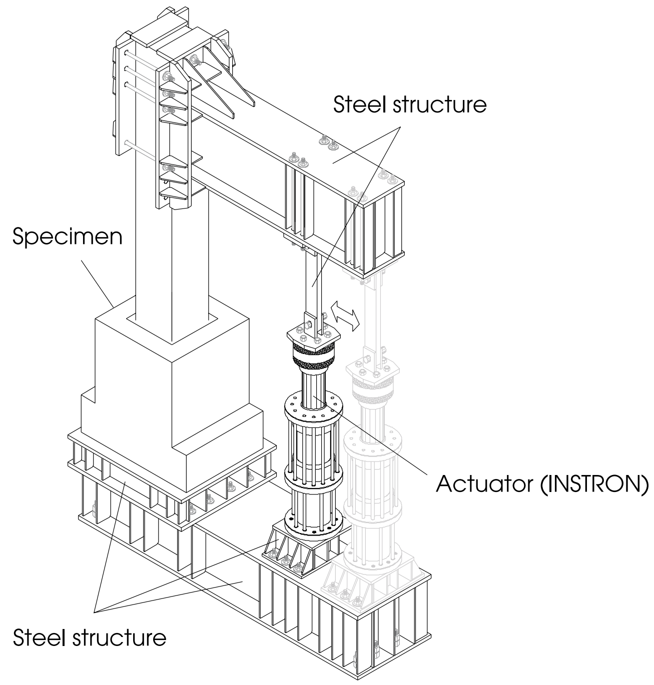
In case of the tested connections in which the horizontal shear force V_d is null and $M_d = N_d.e$, the following equation for $H_{top,d}$ is given:

$$H_{top,d} = \frac{N_d\left(e - e_{nb} - \frac{\mu.y' - \mu^2(0.5h+e_{nb})}{1+\mu^2}\right)}{\ell_{emb} - y - y' + \mu.h} \tag{10}$$

3.2 Socket base with rough interface

As the two models with rough interface presented a behavior very close to a monolithic connection, in other words, the total transfer of moment and normal force from

Figure 6 – The test set-up



column to socket base was verified, the design of vertical reinforcements by bending theory is suggested, according to figure 8. The comparison of the theoretical results of this procedure and of the design model for smooth socket base using $\mu = 1$ for rough interface with the experimental results is presented in the following section.

4 Comparison among theoretical and experimental results

With the purpose of observing which model approaches more of the experimental rupture force, the theoretical models of technical review were applied in the tested specimens: LEONHARDT & MÖNNIG [3], WILLERT & KESSER [6], OLIN *et al.* [7], ELLIOTT [8] and OSANAI *et al.* [2] modified, which were presented in CANHA [1]. The materials properties and some calculation considerations are based on table 2, which shows the main experimental results of the physical specimens.
Table 3 presents the values of experimental strengths and of this theoretical application in the tested specimens.
For specimens SI2 and SI3, the longitudinal walls 3 and 4 behaved as corbels. The main reinforcements $A_{s,hm}$ and $A_{s,vm}$ reached the yielding and the compression strut can be inferred, as shown in figure 9. The force H_{top} was determined with the contribution of whole main horizontal reinforcement $A_{s,hm}$. Although the external branches of those stirrups have not yielded, the design applications can be simplified using average strains of those reinforcements, which practically reached the yield. This force H_{top}, calculated by yield of the reinforcement $A_{s,hm}$, was smaller than that originated by yield of the reinforcement $A_{s,vm}$ and the strut crushing of the design of longitudinal walls 3 and 4 as corbels. The friction coefficient μ of 0.6 for smooth interface was considered. The value for eccentricity e_{nb} of normal reaction recommended by OLIN *et al.* [7], equivalent to $h/6$, was adopted. For the distance y' of $H_{bot,d}$ to base, the value suggested by OSANAI *et al.* [2], that is $(\ell_{emb} - 2y)/3$, was used.
With the purpose of verifying the reserve of safety between the ultimate load of each design model and the experimental value, the percentile differences were considered as the quotient of the difference of experimental value with the theoretical ultimate load by the theoretical ultimate load.
The connection strength given by the LEONHARDT & MÖNNIG [3] design model was underestimated by 99% for specimen SI2 and by 114% for specimen SI3. This fact shows the importance of taking into account the friction in the design of socket base connection. As presented in CANHA [1], OSANAI *et al.* [2] modified model was the most economical model among all models, with difference between the theoretical load and the experimental ultimate value of 31% for specimen SI2 and of 41% for specimen SI3. Among the models that consider friction, the OLIN *et al.* [7] and ELLIOTT [8] models were more conservative, the first because it does not take into account the bottom friction force $F_{fri,bot}$ of traverse wall 2 and the second because it does

Table 2 – Main experimental results

Specimen	Concrete σ_{cm} (MPa)	Concrete f_{ctm} (MPa)	Concrete E_{cm} (GPa)	Steel f_{ym} (MPa) $A_{s,hm}$	Steel f_{ym} (MPa) $A_{s,vm}$	Steel f_{ym} (MPa) $A_{s,vs}$	e (m)	N_r (kN)	Yield[c] $A_{s,hm}$	Yield[c] $A_{s,vm}$	Yield[c] $A_{s,vst}$	Yield[c] $A_{s,vsl1}$	Yield[c] $A_{s,vsl2}$
SI1	27.63	2.24	28.6	584	639	593	1.85	241	no	yes	yes	yes	no
SI2							1.85	203	yes	yes	yes	no	no
SI3	33.67	1.95	29.1				1.20	336	yes	yes	yes	no	no
RI1	24.64	1.84	24.9				1.20[a]	448	no	yes	yes	yes	no
							1.85[b]	302	no	yes	yes	yes	yes
RI2							1.20[a]	469	no	yes	no	yes	no
							1.85[b]	304	no	yes	yes	yes	yes

[a] 1[st] test
[b] 2[nd] test
[c] Defined with average strains of reinforcements

$A_{s,hm}$: Main horizontal reinforcement to resist H_{top}
$A_{s,vm}$: Main vertical reinforcement concentrated in corners
$A_{s,vs}$: Secondary vertical reinforcement (wall 2 or walls 3 and 4)
$A_{s,vst}$: Secondary vertical reinforcement of wall 2
$A_{s,vsl1}$: More stressed secondary vertical reinforcement of walls 3 and 4, near from $A_{s,vm}$
$A_{s,vsl2}$: Less stressed secondary vertical reinforcement of walls 3 and 4, far from $A_{s,vm}$

not take into account the eccentricity of normal reaction on foundation base F_{nb} and due to the small distance z between the forces H_{top} and H_{bot}. On the other hand, in spite of the WILLERT & KESSER [6] model considers the normal reaction F_{nb} centered on column base, the theoretical value of this model was quite close to the value of the OLIN *et al.* [7] model.

Despite the fact that part of the main horizontal reinforcement $A_{s,hm}$ of specimen SI1 almost yielded due to maintained loading, its contribution was taken into account in the design models of technical review, because the force H_{top} for this condition was smaller than the corresponding force determined by yield of the main vertical reinforcement and the crush of strut of the longitudinal walls that behaved as corbels. In case of specimens RI1 and RI2, the force H_{top}, resulting from the application of the strut and tie model on longitudinal walls 3 and 4, was used. For this model, the yield of main vertical reinforcement with area $A_{s,vm}$ was considered. In these three specimens, the friction coefficient equal to unit was used, that is indicated for rough interface.

Although the interfaces bond is not recommended for de-

Table 3 – Experimental and design models values of normal rupture force N_r

Design model/Experimental	SI1	SI2	SI3	RI1[a]	RI2[a]
LEONHARDT & MÖNNIG (1977)	127	102	157	134	134
WILLERT & KESSER (1983)	177	147	228	206	206
OLIN *et al.* (1985)	153	142	224	168	168
ELLIOTT (1996)	145	114	176	173	173
OSANAI *et al.* (1996) modified	181	155	239	211	211
Experimental	241	203	336	302	304

[a] second test
N_r values in kN

Table 4 – Theoretical-experimental results of ultimate load N_r changing e_{nb}, y', y and μ - Specimens SI2 and SI3

μ constant	0,6			
y constant	ℓ_{emb} /6 = 13.3 cm			
y' constant	(ℓ_{emb} – 2y)/ 3 = 17.8 cm			
e_{nb} variable	excent. of R_{cd} = 10.9 cm	h / 2 = 20 cm	h / 3 = 13.3 cm	h / 6 = 6.7 cm
Specimen	SI2 SI3	SI2 SI3	SI2 SI3	SI2 SI3
Proposed model	147 235	153 250	149 239	145 228
Experimental	203 336	203 336	203 336	203 336
μ constant	0,6			
y constant	ℓ_{emb} /6 = 13.3 cm			
e_{nb} constant	h / 4 = 10 cm			
y' variable	(ℓ_{emb} – 2y)/ 3 = 17.8 cm	ℓ_{emb} /10 = 8 cm		0
Specimen	SI2 SI3	SI2 SI3		SI2 SI3
Proposed model	147 233	162 255		175 271
Experimental	203 336	203 336		203 336
μ constant	0,6			
y' constant	ℓ_{emb} / 10 = 8 cm			
e_{nb} constant	h / 4 = 10 cm			
y variable		ℓ_{emb} /6 = 13.3 cm	3 ℓ_{emb} /16 = 15 cm	
Specimen		SI2 SI3	SI2 SI3	
Proposed model		162 255	159 249	
Experimental		203 336	203 336	
y constant	ℓ_{emb} /6 = 13.3 cm			
y' constant	ℓ_{emb} / 10 = 8 cm			
e_{nb} constant	h / 4 = 10 cm			
μ variable		0,6	0,3	
Specimen		SI2 SI3	SI2 SI3	
Proposed model		162 255	142 226	
Experimental		203 336	203 336	

N_r values in kN

sign of socket bases, the theoretical calculation of specimen SI1 was executed in order to illustrate its safety reserve related to specimen SI2 without bond. As the rupture of specimens RI1 and RI2 was characterized in second test, the calculation of theoretical strength was made with an eccentricity equivalent to $1.85\ m$ of the second test.

The correlation among the experimental and theoretical values of specimen SI1 for all analyzed design models was very close of the correlation regarding specimen SI2, which was tested with the same eccentricity. The safety reserve of specimen SI1 related to SI2 for the OSANAI *et al.* [2] modified model was 17%, while this correlation in experimental values was about 19%. Consequently, if the characteristics of specimen SI1 were used for design, the OSANAI *et al.* [2] modified model, which supplied better results, could be applied with friction coefficient μ equal to the unit.

The LEONHARDT & MÖNNIG [3] model underestimated strength of specimens RI1 and RI2 in up to 127%, and the OSANAI *et al.* [2] modified model in up to 44%. Therefore, they are conservative for design of rough socket base.

For the adjustment of the proposed model for smooth socket base with the experimental values, the eccentricity e_{nb} of normal reaction on column base $F_{nb,d}$, the height y' of bottom pressure resultant $H_{bot,d}$ and the position y of top pressure resultant $H_{top,d}$ were changed. The usual friction coefficient μ of 0.6 for smooth interface and the yielding of all stirrups $A_{s,bm}$ were used initially for determination of the theoretical strength, as in the previous section. Reminding that EUROCODE 2 [9] limits the friction coefficient in 0.3 for smooth interface, a comparison of the strengths with the two friction coefficients ($\mu = 0.6$ and $\mu = 0.3$) is presented.

Table 4 presents the comparison among theoretical and

Figure 7 – Scheme of forces that act on proposed design model for socket base with smooth interface

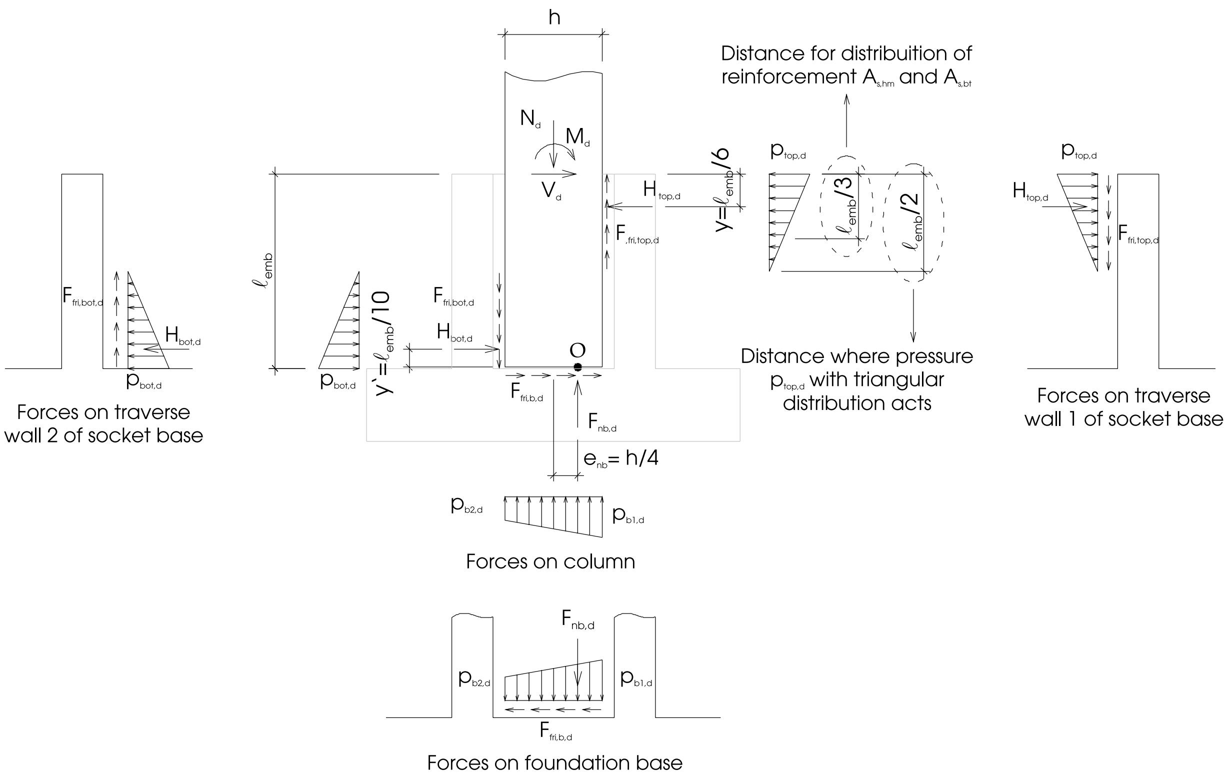


Figure 8 – Design of vertical reinforcements for socket base with rough interface

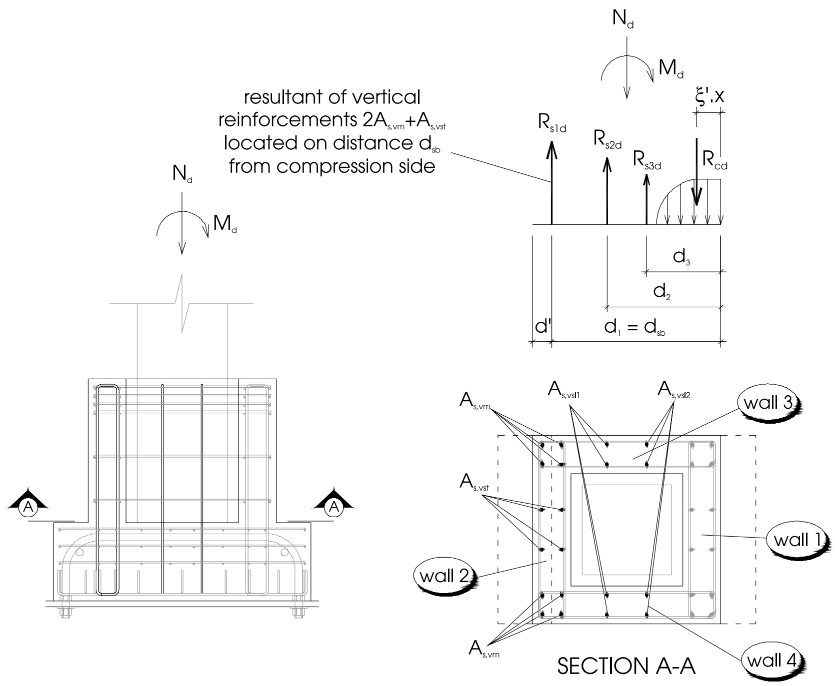


Figure 9 – Cracking of wall 4 of specimen SI2

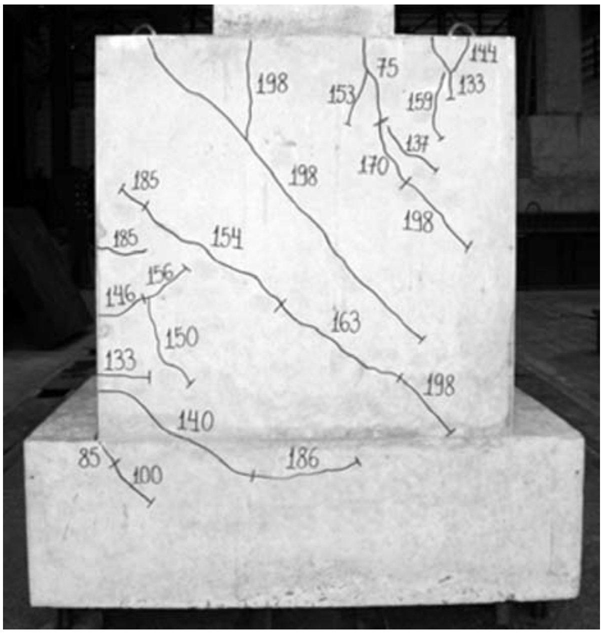
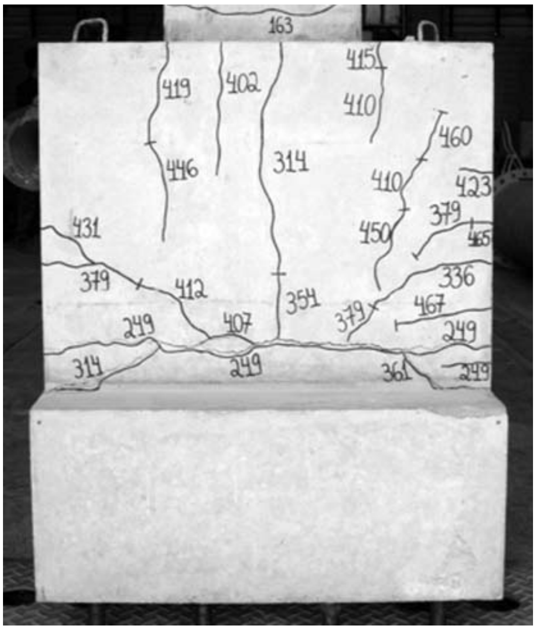


Figure 10 – Cracking of wall 2 of specimen RI2



experimental results of specimens SI2 and SI3 with the variation of parameters e_{nb}, y, y' and μ.
According to the OSANAI *et al.* [2] model, e_{nb} is defined as the eccentricity of the compression force R_{cd} on column base, which is the resulting value of column design. In agreement with the strain domain considered, the

position of neutral axis changes. In other words, for usual and more economical design, for instance, in limit between domains 3 and 4, the eccentricity is smaller than that regarding the domain 3 or 2. In specimens where bond was removed, the position of the compression resultant might have changed in function of the slip among column, joint and socket base. However, in safety side, the e_{nb} value resulting from the column design between domains 3 and 4 was adopted initially. Other e_{nb} values in function of the height h of column section were also attributed for adjustment of the proposed model. Naturally, the possible maximum eccentricity e_{nb} for cases of simple bending and bending-compression, considering as the position of R_{cd} in column design, is equivalent to $h / 2$.

According to table 4, for e_{nb} calculated as the position of the compression force R_{cd} of column design, a difference of 38% and 43% is observed for specimens SI2 and SI3, respectively, with the experimental values. As the e_{nb} value decreases, the proposed theoretical model becomes more conservative. In case of $e_{nb} = h / 2$, a larger proximity among theoretical and experimental results was verified, with an excess in the proposed model of 33% for specimen SI2 and 34% for specimen SI3. The value $e_{nb} = h / 6$ suggested by OLIN *et al.* [7] seems be quite conservative for cases of large eccentricity ($e \geq 2h$) and this is more suitable for average eccentricity ($0.15h \leq e \leq 2h$). As the position e_{nb} of he compression force R_{cd} resulting from column design for the limit between strain domains 3 and 4 is about $h / 4$, this value for e_{nb} is recommended.

Adopting $e_{nb} = h / 4$, then, the height y' of the resultant of top pressures $H_{top,d}$ was changed to the values recommended by OSANAI *et al.* [2], OLIN *et al.* [7] and zero. As expected, as y' decreased, the theoretical force approached more of the experimental value. For the more conservative y' value, the differences between the experimental ultimate load and the value calculated by the proposed model were 38% and 44% for specimens SI2 and SI3, respectively. Despite the fact that the bottom compression force $H_{bot,d}$ is absorbed directly by the foundation base due to its small height in relation to the base, a value different from zero is more careful for design of socket base, in order to embrace the whole interval of large eccentricity ($e \geq 2h$). The value recommended by OLIN *et al.* [7] $y' = \ell_{emb} / 10$ is suggested in this work. In this case, the differences between the proposed model and the experimental value were 25% and 32%, respectively, for specimens SI2 and SI3.

If the compression stresses on traverse wall 1 with parabolic distribution are considered instead of the stresses with triangular distribution, the position of resultant $H_{top,d}$ acting on the height of geometric center of these stresses changes from $y = \ell_{emb} / 6$ to $y = 3\ell_{emb} / 16$, still distancing more the theoretical strength of the experimental result, as table 4 indicates. In case of $y = 3\ell_{emb} / 16$ for the proposed model, the differences in relation to the experimental ultimate load were 28% and 35%, respectively, for specimens SI2 and SI3, against 25% and 32% for $y = \ell_{emb} / 6$. This value of $y = \ell_{emb} / 6$ is then suggested; however, the reinforcement $A_{s,hm}$ continues being distributed evenly along the top region of height equal to $\ell_{emb} / 3$, where the largest concentration of tensions occurs.

With the analysis of variation of the friction coefficient, the value of $\mu = 0.3$ supplied strengths with differences of 43% and 49%, respectively, for the experimental results of specimens SI2 and SI3, against the percentile values of 25% and 32% with $\mu = 0.6$. The friction coefficient of 0.3 indicates more conservative values. In compensation, the usual value of 0.6 for friction coefficient results in a smaller amount of reinforcement. In case of metallic forms, in which the friction on interfaces is more reduced, $\mu = 0.3$ can be used; while for wooden forms or similar forms, $\mu = 0.6$ seems to be more reasonable.

Based on those results, with the known efforts M_d, N_d and V_d on column, expressions (9) and (10) are recommended for the design of socket base with smooth interface, using the parameters $e_{nb} = h / 4$, $y = \ell_{emb} / 6$, $y' = \ell_{emb} / 10$ and μ according to the form material of the connection elements. These values are appropriate for cases in which the embedded length is not smaller than the value suggested by NBR-9062/85 [4], which was $2h$ for this case.

This model should be applied for cases of large eccentricity, in which the predominant action of moment about axial force tends to generate the friction force $F_{fri,b,d}$ on the foundation base with the same direction of $H_{bot,d}$, and the friction force $F_{fri,bot,d}$ at transverse wall 2 with upward direction and at the column with downward direction, as showed in figure 7. For small eccentricity e, the proposed equation could be used after an experimental investigation, and the correct directions of the friction forces $F_{fri,bot,d}$ and $F_{fri,b,d}$ should be analyzed, which can be influenced by the relation among the efforts M_d, V_d and N_d and by the geometry.

For socket base with smooth interface, the design of the main vertical reinforcement $A_{s,vm}$ and the verification of the compression strength should be made considering the longitudinal walls 3 and 4 as corbels according to the LEONHARDT & MÖNNIG [3] model. Although the secondary horizontal reinforcement $A_{s,hs}$ and the secondary vertical reinforcement $A_{s,vst}$ and $A_{s,vsl}$ are little requested, they are indispensable for cracking control and combat of secondary efforts. They should be calculated according to the corbel recommendations.

In the case of specimens of RI series, the monolithic strength of the connection was calculated by bending theory, in other words, with the total transfer of moment and of axial force. The value of σ_{cm} of the socket base concrete and a parabolic-rectangular distribution of stresses were considered in the calculation of the compression resultant of concrete. As the neutral axis was located in the domain $2a$, with the maximum shortened concrete smaller than $2^o/_{oo}$, only the space equivalent to the stresses diagram with parabolic distribution was considered and the contribution of the compressed reinforcement was neglectful. The proposed model with the parameters $\mu = 1$, $e_{nb} = h / 4$, $y = 3\ell_{emb} / 20$ ($= 0.15\ \ell_{emb}$), and $y = \ell_{emb} / 10$ was also applied in these specimens, just to show the conservatism of this model for rough socket base when compared with the connection strength calculated with the yielding of the vertical reinforcements.

Table 5 – Theoretical and experimental values of ultimate axial force N_r - Specimens SI1, RI1 and RI2

Design model/Experimental		SI1	RI1	RI2
Proposed model for smooth socket base with $\mu = 1$		189	219	
Total transfer of M and N	a	239	238	
	b	263	262	
	c	-	276	
Experimental		241	302	304

a $2A_{s,vm} + A_{s,vst}$
b $2A_{s,vm} + A_{s,vst} + A_{s,vsl1}{}^*$
c $2A_{s,vm} + A_{s,vst} + A_{s,vsl1}{}^* + A_{s,vsl2}{}^*$ (more accurate calculation)
* With itself distance from compression side
N_r values in kN

Table 5 presents the values of theoretical and experimental strengths of the second test of these specimens, in which the rupture occurs. As the strains of the secondary vertical reinforcements of wall 2 ($A_{s,vst}$) and walls 3 and 4 ($A_{s,vsl}$) were larger than those at the yield beginning, part of the efforts was absorbed by these reinforcements and they can be considered in the calculation. Therefore, the ultimate load of the monolithic connection was calculated with different options of contribution of the secondary vertical reinforcement. These design models were also applied in specimen SI1.

The differences between the experimental result of rough specimens and the design model for smooth socket base were close to *40%*. Therefore, this model is not indicated for analysis of socket base with shear keys. The strengths of these rough socket bases calculated by bending theory were in favor of safety. Option *a*, in which only the reinforcement of traverse wall 2 contributes in the connection strength, is more conservative, with a difference of up to *28%* in relation to the experimental result. But this option already brings a larger economy in the design, when it is compared with the design models for smooth socket base that consider the friction. In the more refined calculation, which the vertical reinforcements are considered with their distances to the compression side, the differences were of up to *10%* in relation to the experimental value. Therefore, for the vertical reinforcement design of rough socket base with embedded length equal or superior to *1.6b*, the calculation of the monolithic connection is recommended by bending theory, with the simplified contribution of the reinforcement of traverse wall 2 ($A_{s,vm}$ and $A_{s,vst}$) or the most accurate calculation considering all vertical reinforcements with their distances to the compression side.

The predominant crack of rupture of rough socket bases had horizontal direction on traverse wall 2 and the larger opening was on the center of this wall, as figure 10 presents. The behavior of these specimens was close of a monolithic connection. Therefore, a reinforcement evenly distributed in wall 2 can be used. Though, for moments acting in two directions, the efficiency of the vertical reinforcements located in the central region of the walls should be analyzed.

An interesting aspect for specimen SI1 can be verified with the ultimate axial force calculated by bending theory regarding the option *a* of table 5. This theoretical value (*239 kN*) presents a relation practically accurate with the experimental rupture force of specimen SI1 (*241 kN*), with a difference smaller than *1%*. When half of the secondary vertical reinforcement of walls 3 and 4 ($A_{s,vsl1}$) contributes, because it yielded, the theoretical force is larger than the experimental value in *8%*. Using the proposed design model for smooth socket base with friction coefficient equal to unit, the difference among the theoretical and experimental rupture forces was in safety side and equal to *28%*. Although specimen SI1 has presented a mixed behavior among smooth and rough socket bases without bond, this specimen presented more characteristics of a specimen with rough interface. However, for the reasons exposed previously, specimen SI1 should not be considered for design.

5 Final remarks and conclusions

The specimens of the experimental program were subdivided into two series (SI and RI), changing the condition and the type of interface and the eccentricity of the load. Due to a possible wind effect at the first ages of the connection cast-in-place concrete and shrinkage of this concrete, the perfect contact between the cast-in-place concrete and other concretes is not guaranteed and the bond can be lost. Therefore, the appropriate situation for the design is not to take into account the bond of these interfaces and the bond was removed in four of five tested specimens. The casting of joint of specimen SI1 was made with normal procedure. In specimens SI2 and SI3, the bond was removed. In specimens SI2 and SI3, different configurations of shear keys were used and the bond was removed too.

Based on the experimental results, some recommendations and more consistent models were suggested for design of the reinforcements and verification of the concrete strength of pedestal walls:

a) According to the experimental results of specimens SI2 and SI3, without bond on interfaces, the transfer of the efforts of traverse wall 1 for longitudinal walls 3 and 4 occurred together with the action of the friction forces and the behavior of these longitudinal walls as corbels. This corbel behavior was according to the LEONHARDT & MÖNNIG [3] model, with the indirect transmission of the force H_{top}, that resulted in the resistant mechanism of the compression strut of concrete and of the tension in tie. However, the secondary vertical reinforcement of wall 3 and 4 $A_{s,vsl}$ was little requested.

b) The model proposed in this paper for design of the reinforcement $A_{s,hm}$ provided good results, with differences of 25% and 32%, at the safe side, for specimens SI2 and SI3, respectively, when $\mu = 0.6$ was used. This model should be used for smooth socket base with the embedded length equal or longer than $2h$. After the calculation of $A_{s,hm}$, the design of longitudinal walls 3 and 4 as corbels is indicated, that results in the design of the reinforcement $A_{s,vm}$ and the verification of crushing of the concrete compression strut.

c) Besides the indication of parameters $e_{nb} = h / 4$, $y = \ell_{emb} / 6$, $y' = \ell_{emb} / 10$ for the proposed design model, the friction coefficient is recommended according to the type of form used in the connection building. For forms in which the interfaces friction is reduced, as the steel moulds, $\mu = 0.3$ is indicated; for wooden or similar forms, $\mu = 0.6$ is suggested.

d) Although specimens RI1 and RI2 had different configurations of shear keys, the experimental strength of these specimens was practically the same. The rupture of these specimens was characterized in the second test with the yielding of the vertical reinforcements and was accompanied of a predominant crack of bending on traverse wall 2. As there was the total transfer of efforts from column to pedestal walls, in other words, all stressed vertical reinforcements ($A_{s,vm}$, $A_{s,vst}$ and $A_{s,vsl}$) contributed to the connection strength, specimens RI1 and RI2 with $\ell_{emb} = 1.6h$ presented behavior similar to a monolithic connection. Therefore, the bending theory is more appropriate for design of these vertical reinforcements. However, this is applied for rough socket bases with embedded length not shorter than $1.6h$ and the roughness must be in the range of this research.

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7 References

[01] CANHA, R.M.F. Theoretical-experimental analysis of column-foundation connection through socket of precast concrete structures. São Carlos. 279p. PhD Thesis. Escola de Engenharia de São Carlos, Universidade de São Paulo. 2004. (in Portuguese Estudo teórico-experimental da ligação pilar-fundação por meio de cálice em estruturas de concreto pré-moldado)

[02] OSANAI, Y.; WATANABE, F.; OKAMOTO, S. Stress transfer mechanism of socket base connections with precast concrete columns. ACI Structural Journal, v.93, n.3, p.266-276, May/June. 1996.

[03] LEONHARDT, F.; MÖNNIG, E. Vorlersungen uber massivabu. Berlin. Springer-Verlag. 1973. (Portuguese version Construções de concreto: Princípios básicos sobre armação de estruturas de concreto armado. v.3, 1.ed. Rio de Janeiro, Interciência. 1977.)

[04] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. NBR 9062 - Design and fabrication of precast concrete structures. Rio de Janeiro, ABNT. 1985. (in Portuguese Projeto e execução de estruturas de concreto pré-moldado)

[05] EL DEBS, M.K. Precast concrete: bases and applications. 1.ed. São Carlos, SP, Publicação EESC-USP. 2000. (in Portuguese Concreto pré-moldado: fundamentos e aplicações)

[06] WILLERT, O.; KESSER, E. Foundations for bottom-end fixed precast concrete columns. Betonwerk+Fertigteil-Technik, v.49, n.3, p.137-142. 1983.

[07] OLIN, J.; HAKKARAINEN, T.; RÄMÄ, M. Connections and Joints between precast concrete units. Espoo, Julkaisija-Utgivare. 1985.

[08] ELLIOTT, K.S. Multi-storey precast concrete framed structures. Oxford, Blackwell Science. 1996.

[09] COMITÉ EUROPEO DE NORMALIZACIÓN. Eurocódigo 2 - Proyecto de estructuras de homigón - Parte 1-3: Reglas generales. Elementos y estructuras. Prefabricados de hormigón. Madrid, AENOR. 1995. (Eurocode 2)