Experimental study on precast beam-column connections with continuity reinforcement for negative bending moments

Análise experimental de ligações viga-pilar pré-moldadas com armadura de continuidade para momentos fletores negativos

Abstract

In this paper, an experimental investigation is presented on semi-rigid interior beam-to-column connections constituted by precast concrete beams supported on precast concrete column corbels and bending continuity reinforcement bars for bending negative moments. The main purpose of this paper was to analyze the influence of vertical interface grout filling between the corbel and the beam and the position of the bending continuity reinforcement bars (crossing only the column or crossing only the slab) on the behavior of this type of connection. Tests on eight specimens were performed. It was noticed that the vertical interface grout filling contributed to increase both rotational flexural stiffness and flexural strength capacity of the connections when compared to the connections without grout filling. It was also noticed that in the specimens in which the continuity bars crossed only the column, the rotational flexural stiffness was higher. For these last ones, coefficients \( k \) and \( b \) for predicting the secant stiffness by simplified expression present in Brazilian Code NBR 9062 were evaluate from experimental results. These evaluated coefficients may be regarded as indicative values for structural designers and helpful for future researches.

Keywords: precast concrete structures, precast beam-column connections, semirigid connections, structural analysis.

Resumo

Neste trabalho apresenta-se um estudo experimental de ligação viga-pilar semirrígida de pilar intermediário constituída de vigas de concreto pré-moldadas apoiadas em consolos de concreto pré-moldado, com presença de armaduras de continuidade resistentes ao momento fletor negativo. O presente trabalho teve como objetivo principal verificar a influência do preenchimento de gruarte na interface vertical entre o consolo e a viga e a posição da armadura de continuidade (passante somente no pilar e passante somente nas lajes). Foram ensaiados no total oito protótipos. Verificou-se que o preenchimento de gruarte na interface vertical consolo-viga contribuiu para aumentar tanto a rigidez e quanto a resistência da ligação à flexão quando à comparada a ligação sem o preenchimento desse gruarte. Verificou-se também que nos protótipos em que as armaduras de continuidade passaram somente no pilar a rigidez à flexão da ligação foi maior. Para estas tipologias de ligação, foram obtidos, a partir dos resultados experimentais, os coeficientes \( k \) e \( b \) presentes na expressão simplificada da NBR 9062 para o cálculo da rigidez secante. Os valores calculados para esses coeficientes podem servir de valores indicativos e orientativos para projetistas estruturais e para futuras pesquisas.

Palavras-chave: estruturas de concreto pré-moldado, ligações viga-pilar pré-moldadas, ligações semirrígidas, análise estrutural.
1. Introduction

The study of the connections between precast concrete structural members is of great importance for the development and execution of design, since the performance of the precast concrete structural system is directly related to the performance of their connections, affecting both the local behavior of the adjacent members and the global behavior of the structure.

In framed structural systems with precast members, the beam-column connections subjected to bending moments are classified as rigid, semi-rigid or pinned connections, depending on the bending moment mobilized and transferred among its members. The classification and quantification of the flexural stiffness in beam-column connections are important for structural analysis and design of the structure.

In codes overall, the classification of connections depends on the knowledge of rotational stiffness, which depends on the characteristics of the typology of the connection. In general, rotational stiffness should be obtained based on experimental results or through analytical models, which need to be calibrated or validated experimentally. In the last three decades, there have been international experimental studies on different types of beam-column connections [1-14], whose typologies analyzed were chosen mainly due to their concern with behavior under seismic loads.

In Brazil, research involving semi-rigid connections began just over two decades ago [15-25]. In Miotto [17], Souza [18], Baldissera [19], Kataoka [20] and Hadade [25], experimental results of beam-column connections with continuity reinforcement for negative bending moments can be found.

In the context of the present paper, it is appropriate to highlight the tests of Miotto’s connections [17], in which the precast beams (dapped-end beams) were supported by concrete corbel and fixed by steel dowels on it. In these connections there was no grout filling between the corbel interface and the lower vertical face of the beam. Half of the continuity reinforcement crossed the column and the other half was placed in the slab. On the other hand, Kataoka [20] also presents two types of beam-column connections with precast beams supported by concrete corbels. In one connection, the continuity reinforcement was fully placed crossing the column; at the other connection, half of the continuity reinforcement crossed the column and the remaining half was distributed in the slab.

This paper presents results of experimental studies conducted by Lacerda [26] about the behavior of four types of precast beam-column connections that can be used in the construction of structures with more than one story, such as factory constructions, industries, and residential buildings. The types of investigated beam-column connections are designed for negative bending moments and are composed of continuity reinforcement, precast beam supported by precast column corbel and fixed by steel dowels and grout filling at the beam and column interface.

The main goal of Lacerda [26] was to evaluate the behavior of the connections under the influence of the filling the lower interface between the beam and the corbel (with or without grout) and the influence of the position of the continuity reinforcement (crossing only the column or only the slab). In order to evaluate the flexural stiffness for negative moments of the connections, it was necessary to obtain experimentally the moment-rotation curves and the secant stiffness of the studied connections.

ABNT NBR 9062 [27] presents a simplified expression for obtaining the secant stiffness $R_{sec}$ (Figure 1) for six connection typologies. The prediction of the secant stiffness allows the consideration of the
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deformability (rotational) of the beam-column connections in simplified linear analysis, useful to obtain internal forces and displacements in the structural system. However, it should be noted that the expression indicated in NBR 9062 [27] for obtaining secant stiffness does not include the types of connections investigated in this paper. Thus, as an additional contribution of this paper, it were obtained (for two types of connection) the coefficients \( k \) and \( e \) present in the simplified expression of NBR 9062 [27] for the prediction of the secant stiffness, from the experimental results obtained by Lacerda [26].

2. Experimental tests

2.1 Characteristics of the beam-column connections specimens

The experimental program was performed by Lacerda [26] and consisted of the investigation of interior precast beam-column connections specimens under negative bending moment. The main goals of the experimental program were to verify the influence of two variables:
- Vertical grout filling at the interface between corbel and beam (Figure 2);
- Position of the continuity reinforcement (crossing only the column or only the slab (Figures 3 and 4).

Four variations of specimens were tested (two for each variation). Thus, eight specimens were tested. Table 1 presents the nomenclature used for the specimens.

Figure 3 shows the connections where continuity reinforcement only pass through the column (PC and PS) and Figure 4, the connections where continuity reinforcement only pass through the slab (LC and LS). For all specimens, the total area of continuity reinforcement was the same (4.02 cm²).

As shown in Figures 3 and 4, each specimen is composed of a precast column, two precast cantilever beams supported by column corbels and seated on cushion of grout and the presence of steel dowel. The specimens also present cast-in-situ concrete slab, grout filling at the interface between the column and beam, continuity reinforcement and whether or not the grout has been fulfilled at the vertical interface between the corbel and the beam. The geometric characteristics of all specimens and the details of the steel dowel positioning are shown, respectively, in Figures 5 and 6.

Details of the reinforcement columns, corbels, beams and slabs are shown in Figures 7 to 11.

2.2 Mechanical properties of the materials

For the construction of the precast beams and columns, a 40 MPa compressive strength for the concrete was specified. For the cast-in-situ concrete slab, a 25 MPa compressive strength was specified. Two CA50 steel bars of 16 mm diameter were used as continuity reinforcement, totaling an area of 4.02 cm² in each specimen. This reinforcement was cast in concrete slab, providing the flexural strength capacity for the connection (negative bending moment). The corbels and beams were left with holes for fixing the steel dowels (CA60 steel bars of 9.5 mm diameter). Later these holes were filled with grout. For the filling of grout at the vertical interface (between corbel and beam: PC and LC specimens) a self-compacting shrinkage compensated grout of 40 MPa compressive strength was specified.

Table 1
Nomenclature of the specimens

| Specimen | Continuity reinforcement | Variable Filling with grout | Quantity |
|----------|--------------------------|-----------------------------|----------|
| PS       | Only crossing the column | Without                     | 2        |
| PC       | Only crossing the column | With                        | 2        |
| LS       | Only crossing the slab   | Without                     | 2        |
| LC       | Only crossing the slab   | With                        | 2        |

PS – Specimen without grout filling; PC – Specimen with grout filling; LS – Specimen without grout filling; LC – Specimen with grout filling
Tests on cylinder specimens (100 mm x 200 mm) were performed to control concrete/grout compressive strength, according to the Brazilian standards. Concrete compressive strength was approximately 48 MPa for precast beams and columns and 35 MPa for concrete slabs. Grout compressive strength was 50 MPa. The yielding and ultimate strength of the continuity reinforcement are approximately 600 MPa and 730 MPa, respectively.

2.3 Construction of the specimens

A steel shoring system was used for positioning and leveling the precast elements and for supporting the slab formwork. The sequence of steps for the construction of each specimen was:

1. Positioning of the column;
2. Insertion of the steel dowels and filling corbel holes with grout;
3. Setting the cushion of grout on the corbels;
4. Setting the beams on the corbels;
5. Filling vertical space between corbel and beam with grout for specimens PC e LC;
6. Setting up slab formwork;
7. Filling spaces between beam and column with grout;
8. Insertion of the continuity reinforcement bars and slab reinforcement bars;
9. Filling the column holes for the continuity reinforcement with grout;
10. Casting of the concrete slabs.

Figure 12 illustrates the lifting procedure of the columns and beams and how the steel dowels and the cushion of grout of the beams are used.
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Figure 7
Details of the columns (length dimensions in cm)
installation were performed. Figure 13 illustrates how the beam-to-column union was performed, the vertical grouting of the bottom face of the beam with column corbel (in PC and LC specimens), as well as it illustrates the detail of the continuity reinforcement crossing the column (in PC specimens).

### 2.4 Test setup and procedures

A hydraulic jacket (actuator with 500 kN load capacity) was used to induce negative bending moments in the specimens. The actuator applied to the base of the column increasing forces (F) upwards (according to Figure 14). Through the reaction structure (Figure 15), forces (F/2) were applied to the ends of each beam. The column was supported on a steel plate above the actuator for the linear distribution of the load on the slab ends loading steel bars were centralized over the elastomeric bearings at the points of the reaction forces - 1.07 meters from the column faces. Above each steel bar there were a plate, steel devices (to increase the available distance between the top of the column and reaction structure) and the load cell to capture the reaction force at the ends of the beams. Figure 15 illustrates the test setup for all specimens.

Displacement transducers (LVDTs) were used to measure: i) beam deflections at point of load; ii) to determine relative rotation between beam and column (both sides of the column); iii) to monitor the connection zone. Figure 16 indicates the position of the displacement transducers in the specimens. Figure 17 gives a general view of the equipment used in the tests.
3. Experimental results analysis

3.1 Cracking

The cracking was concentrated on the cast-in-situ slabs for all specimens. Figures 18 and 19 show the cracking pattern in the slabs of the tested connections. It can be noticed that the distribution of these cracks is approximately symmetrical and parallel to the smallest column face. For all specimens, cracks appeared on the column slab interfaces mainly toward the least direction of the column. In specimens with crossing slab continuity reinforcement (LC and LS), the cracking was more uniform and extended to the intermediate slab interfaces. The cracking pattern is shown in Figures 18 and 19.

Figure 10
Details of the slabs: specimens LC and LS (length dimensions in cm)
Figure 11
Details of the slabs: specimens PC and PS (length dimensions in cm)
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Figure 12
Lifting of the precast elements, steel dowel and cushion of grout of the beams

Figure 13
Setting beams on corbels, vertical grouting (PC and LC) and continuity reinforcement crossing the column (PC)

Figure 14
Loading applied to the specimens

LS), it was observed that there was a greater crack distribution (lower cracking spacing) during loading in relation to specimens with crossing column reinforcement (PC and PS), as shown in Figures 18 and 19. This fact is related to the different diameters of the continuity reinforcement bars used in the specimens, since the diameter of a steel bar in tension influences the spacing between flexural cracks.

3.2 Flexural strength

Table 2 shows the maximum values of the applied loads to the beam ends (F/2) by the reaction structure. From these forces and knowing that the distance between the point of application of the loads and the face of the column is 1.07 m, the ultimate moment (flexural strength) of the specimens was obtained.

Based on Table 2, it can be seen that the grout filling at the vertical interface between the corbel and the beam led to an increase of about 38% in the flexural strength between the LC and LS specimens and about 36% between PC and PS specimens. The main contribution attributed to this increase in strength was higher lever arm of internal tensile and compressive forces in negative bending section of the LC and PC specimens in relation to the LS and PS specimens.

Still based on Table 2, it can be noticed that the position of the continuity reinforcement (passing only in the slab or only in the
column) did not lead to relevant differences. The maximum bending moments achieved in LC specimens were about 6.6% higher than those achieved in PC specimens. Similarly, the LS specimens achieved 5.3% higher ultimate moments than those achieved by the PS specimens.

### 3.3 Obtaining the moment-rotation curve

The bending moments at the connections were determined by Equation 1. The relative rotations were obtained by Equation 2 using the LVDTs readings shown in Figure 20.

\[ M = \frac{F}{2} \times L \]  
\[ \theta = \text{mean}(2E/2D) + \text{mean}(4E/4D) \times \frac{1}{390} \]

where

- \( F \) is the load applied by the actuator;
- \( L \) is the distance between the application point of the load and the column face (1.07 m);
- \( M \) is the bending moment at the connection;
- \( \theta \) is the relative rotation between beam and column;

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**Table 2**

| Specimen | \( F/2 \) (kN) | \( M_u \) (kN.m) |
|----------|----------------|--------------------|
| LC1      | 115.77         | 123.87             |
| LC2      | 115.68         | 123.78             |
| LS1      | 75.96          | 81.28              |
| LS2      | 92.07          | 98.51              |
| PC1      | 105.13         | 112.49             |
| PC2      | 111.93         | 119.77             |
| PS1      | 81.96          | 87.70              |
| PS2      | 77.56          | 82.99              |

**Table 3**

| Specimen | \( M_y \) (kN.m) | \( \theta_y \) (rad) | Secant stiffness (kN.m/rad) | Mean secant stiffness (kN.m/rad) |
|----------|------------------|----------------------|----------------------------|---------------------------------|
| LC1      | 106.65           | 0.0042969            | 24820                      | 25525                           |
| LC2      | 106.62           | 0.0040647            | 26231                      |                                 |
| LS1      | 55.517           | 0.017937             | 3095                       | 3956                            |
| LS2      | 73.102           | 0.015178             | 4816                       |                                 |
| PC1      | 105.51           | 0.0032868            | 32101                      | 32256                           |
| PC2      | 110.99           | 0.0034244            | 32412                      |                                 |
| PS1      | 72.957           | 0.013449             | 5425                       | 6823                            |
| PS2      | 74.269           | 0.0090347            | 8220                       |                                 |

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**Figure 15**

Test setup for the specimens and reaction structure (dimensions in cm)
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2E, 2D, 4E and 4D are readings from the LVDTs 2E, 2D, 4E and 4D, respectively (in mm).

Figures 21 to 24 contain the experimental moment-rotation curves obtained with Equations 1 and 2. From these curves the parameters necessary to calculate the flexural secant stiffness ($R_{sec}$) were extracted, whose values are presented in Table 3.

In Table 3:
- $M_y$ is the bending moment corresponding to the beginning of yielding of the continuity reinforcement;
- $\theta_y$ is the relative rotation corresponding to the beginning of yielding of the continuity reinforcement.

The results of Table 3 indicate that the grout filling at the vertical interface between the corbel and the beam contributed to the flexural stiffness increase of the connections. The average secant stiffness of LC specimens was about 6.5 times higher than the average secant stiffness of LS specimens. In turn, the average secant stiffness of PC specimens was about 4.7 times higher than the average secant stiffness of PS specimens.

Regarding the position of the continuity reinforcement, the specimens in which this reinforcement was crossing only in the column presented higher flexural stiffness when compared to the specimens where the continuity reinforcement was crossing only in the slabs. The average secant stiffness of PC specimens was about 26.3% higher than the average secant stiffness of LC specimens. In turn, the average secant stiffness of PS specimens was about 72.5% higher than the average secant stiffness of LS specimens.

### 3.4 Experimental secant stiffness compared with simplified equation presented in Brazilian code NBR 9062 [27]

The updated Brazilian code for design of precast concrete structures - NBR 9062 [27] – provides a simplified equation for predicting the flexural secant stiffness of beam-column connections with continuity reinforcement in the negative bending moment region, according to Equation 3:

$$R_{sec} = k \frac{A_s E_s d^2}{l_{ud}} = k \frac{A_s E_s d^2}{\beta \phi + l_u}$$

where
- $A_s$ is the area of continuity reinforcement;
$E_s$ is the modulus of elasticity of steel;
$d$ is the effective depth of the beam;
$L_{ed}$ is the effective deformation length related to elongation of the continuity reinforcement;
$\phi$ is the diameter of the continuity reinforcement bars;
$L_c$ is the distance from the column face to the center of rotation (concrete corbel);

$k$ is an adjustment coefficient for secant stiffness and $\beta$ is a non-dimensional coefficient. Both coefficients depend on the type of connection.
NBR 9062 [27] presents the values $k$ and $\beta$ for two types of interior beam-column connections ($k=0.75$ and $k=1.0$; $\beta=25$ and $\beta=20$). However, the typologies investigated in this paper are not considered by the referred code. Thus, the secant stiffness and the coefficients $k$ and $\beta$ should be evaluated experimentally.

A procedure to evaluate the values of $k$ and $\beta$ is presented as follows, using the experimental results obtained for the PC and PS specimens. According to Ferreira et al. [28] and Ferreira [29], the relative rotation between beam and column is due to two deformation mechanisms. The first is related to the slippage of the continuity reinforcement inside the column (Mechanism A). The second mechanism is a consequence of the sum of the continuity reinforcement slips caused by cracks opening in the beam next to the column (Mechanism B). According to the FIB Guide [30], the slip of the continuity reinforcement inside the column ($s_a$) along the transmission length ($l_t$) in the elastic range can be calculated by:

$$s_a = \varepsilon_s \left( \frac{\sigma_r}{E_s} \frac{\phi}{4} + 2\phi \right)$$  \hspace{1cm} (4)

$$l_t = \frac{\sigma_r}{E_s} \frac{\phi}{4} + 2\phi$$  \hspace{1cm} (5)

Figure 19
Cracking pattern of the specimens PS and PC

Figure 20
LVDTs used to determine relative rotations for all specimens
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where $\varepsilon_s$ is the strain of the reinforcement bar;  
$\sigma_s$ is the stress of the reinforcement bar;  
$\tau_b$ is the average bond stress along the transmission length;  
$\phi$ is the steel bar diameter.

The average bond stress can be estimated by:

$$\tau_b = \sqrt{f_c}$$  \hspace{1cm} (6)

where $f_c$ is the concrete compressive strength of column (in MPa).

For all specimens investigated in this paper:

$$\sigma_s = f_y = 600\text{MPa} \quad f_c = 48\text{MPa} \quad \phi = 16\text{mm}$$

Thus, Equation 5 gives the following result for the transmission length at beginning of the yielding of the continuity reinforcement:

$$l_b = \frac{600}{48} \times \frac{16}{4} + 2 \times 16 = 378\text{mm}$$

The slip of continuity reinforcement bars cannot be calculated by Equation 4. However, assuming that the the steel stress and steel stain vary linearly along the transmission length $l_t = 0.5h_p$, it can be deduced the expression for the calculation of the slip by integrating the strain of steel bar:

$$s_A = \varepsilon_s \left[ 0.5h_p - 2 \frac{\tau_b}{\sigma_s} \times \left( \frac{0.5h_p - \phi}{\phi} \right)^2 \right]$$  \hspace{1cm} (7)

Considering $E_s = 210\text{GPa}$ for the modulus of elasticity of steel, the slip associated with the first deformation mechanism (Mechanism A) corresponding to the beginning of the yielding of the continuity reinforcement is calculated by:

$$s_{pA} = \frac{600}{210000} \left[ 0.5 \times 300 - 2 \frac{\sqrt{48}}{600} \times \left( \frac{0.5 \times 300 - 2 \times 16}{16} \right) \right] = 0.3711\text{mm}$$

The slip ($s_B$) related to the formation of flexural cracks at the beam end next to the joint region (Mechanism B) can be evaluated by Alva and El Debs [31]:

$$s_B = \frac{\varepsilon_s \left[ l_p + s_p \right]}{2}$$  \hspace{1cm} (8)

Figure 21  
Moment-rotation curve: specimens LC1 and LC2

Figure 22  
Moment-rotation curve: specimens LS1 and LS2
where
\( L_p \) is the plastic hinge length (where is higher concentration of cracks in the connection region);
\( s_R \) is the crack spacing.

According to Paulay and Priestley [32], a good estimate of the plastic hinge length for a cantilever beam may be obtained by:

\[
L_p = 0.08L + 0.022 \times \phi \times f_y
\]  

(9)

where
\( L \) is the span of the beam (in meters). For the specimens investigated: \( L = 1.07m \);
\( \phi \) is the steel bar diameter (in meters). For the specimens investigated: \( \phi = 0.016m \);
\( f_y \) is the yield stress of reinforcement steel (in MPa). For the specimens investigated: \( f_y = 600\text{MPa} \).

Substituting values in Equation 9:
\( L_p = 296.80\text{mm} \)  (for all specimens)

The crack spacing (\( s_R \)) may be evaluated from code expressions, such as Equation 10 presented in Eurocode 2 [33]:

\[
s_R = k_1c + k_2k_3 \frac{\phi}{\rho_{s,\text{eff}}} \]  

(10)

where
\( c \) is the concrete cover;
\( \rho_{s,\text{eff}} = \frac{A_s}{A_{c,\text{eff}}} \)
\( A_s \) is the area of reinforcing steel in the tensile zone;
\( A_{c,\text{eff}} \) is the effective area of concrete in tension surrounding the reinforcement;
\( k_1 = 0.8 \)  \( k_2 = 0.5 \)  \( k_3 = 3.4 \)  \( k_4 = 0.425 \).

For all specimens: \( c = 22\text{mm} \) and \( A_s = 4.02\text{cm}^2 \).

For specimens PC: \( A_{c,\text{eff}} = 105.00\text{cm}^2 \)  \( \rho_{s,\text{eff}} = 0.0383 \)  and  \( s_R = 145.85\text{mm} \)

For specimens PS: \( A_{c,\text{eff}} = 72.44\text{cm}^2 \)  \( \rho_{s,\text{eff}} = 0.0555 \)  and  \( s_R = 123.81\text{mm} \)

Thus, at the beginning of yielding of the continuity reinforcement,
Equation 8 gives:
Specimens PC:
\[ s_{y,B} = \frac{600 \left( 296.80 + 145.85 \right)}{210000} = 0.6324 \text{mm} \]
Specimens PS:
\[ s_{y,B} = \frac{600 \left( 296.80 + 123.81 \right)}{210000} = 0.6009 \text{mm} \]

Therefore, the total slip induced by the two mentioned deformation mechanisms is:
Specimens PC:
\[ s_y = s_{y,A} + s_{y,B} = 0.3711 + 0.6324 = 1.0035 \text{mm} \]
Specimens PS:
\[ s_y = s_{y,A} + s_{y,B} = 0.3711 + 0.6009 = 0.9720 \text{mm} \]

Since the effective deformation length \( L_{ed} \) is related to elongation of the continuity reinforcement, the following relationship at the beginning of yielding of the continuity reinforcement can be established:
\[ \varepsilon_y = \frac{s_y}{L_{ed}} \]  
(11)

For specimens PC:
\[ L_{ed} = \frac{1.0035}{600} = 351.23 \text{mm} = 35.123 \text{cm} \]
For specimens PS:
\[ L_{ed} = \frac{0.9720}{600} = 340.20 \text{mm} = 34.020 \text{cm} \]

The experimental values \( k \) and \( \beta \) in Equation 3 are obtained from experimental values of flexural stiffness (see Table 3).

For specimens PC:
\[ 32256 \times 10^2 = \frac{k \times 4.02 \times 21000 \times 36^2}{35123} \quad \rightarrow \quad k = 1.036 \]
\[ 35.123 = \beta \times 1.6 + \frac{17}{2} \quad \rightarrow \quad \beta = 16.64 \]

Likewise, the experimental values of the coefficients \( k \) and \( \beta \) are evaluated for specimens PS:
\[ 6823 \times 10^2 = \frac{k \times 4.02 \times 21000 \times 19^2}{34020} \quad \rightarrow \quad k = 0.762 \]
\[ 34.020 = \beta \times 1.6 + \frac{17}{2} \quad \rightarrow \quad \beta = 15.95 \]

4. Conclusions

Based on the experimental results presented in this paper, the following conclusions can be highlighted:

- Grout filling at the vertical interface between the corbel and beam contributed significantly to increase the bending stiffness of the connections. In specimens where the entire continuity reinforcement crossed the column, the secant flexural stiffness was about 5 times higher than the prototype stiffness without filling. In specimens that the entire continuity reinforcement was distributed only on the slab, the secant flexural stiffness was about 6.5 times higher than the prototype stiffness without filling. This is because the existing free space between the corbel and beam in specimens PS and LS allows greater rotational freedom in these connections;
- In terms of flexural strength, the results indicated that the connections with the continuity reinforcement crossing only the slab and crossing only the column are similar, that is, no significant differences were found between the strength of the connections due to the positioning of the reinforcement continuity. This finding refers to both specimens with vertical grout filling and specimens without such grout;
- For the same amount of continuity reinforcement, the specimens in which this reinforcement only crossed the columns showed higher flexural stiffness than the specimens in which this reinforcement only passed in the slabs: PC about 26% higher than the LC and PS about 72% higher compared to LS;
- The benefit/cost ratio resulting from the increased difficulty to perform vertical grout filling in the analyzed typologies is high, considering the gains obtained in terms of flexural strength and flexural stiffness of the connections;
- The non-dimension coefficients \( k \) and \( \beta \) used to calculate the flexural secant stiffness of the connection in the expression proposed by NBR 9062 (2017) were evaluated from the experimental results for the two connection types investigated (PS and PC), which are not yet covered by that code. In turn, the values obtained for these coefficients in this study can be useful as indicative and guiding values for structural designers or useful for future research on the subject.

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