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DOI
10.1590/S1983-41952023000300008

Publication date
2023

Document Version
Final published version

Published in
Revista IBRACON de Estruturas e Materiais

Rev. IBRACON Estrut. Mater., vol. 16, no. 3, e16308, 2023

IBRACON Structures and Materials Journal
Revista IBRACON de Estruturas e Materiais
ISSN 1983-4195 ismj.org

ORIGINAL ARTICLE

Recommendations to assess the shear and punching capacity of one-way slabs under concentrated loads according to the ABNT NBR 6118:2014

Recomendações para avaliação da resistência à força cortante e punção de lajes unidirecionais sob cargas concentradas de acordo com a ABNT NBR 6118:2014

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Received 14 September 2022
Accepted 27 February 2023

Abstract: This study describes a set of recommendations to improve the precision of shear and punching shear capacity predictions for one-way slabs under concentrated loads, regardless of the governing failure mechanism, using the ABNT NBR 6118:2014 code provisions. For this purpose, a database of 143 test results was developed, including one-way slabs with different support conditions and loading layouts and that failed by different shear failure mechanisms: one-way shear, punching shear or a mixed mode. The key parameters influencing the load capacity and failure mechanism of these slabs were considered for the proposed recommendations: load position and slab width. Adjustments to the effective shear width definition and shear resisting control perimeter were described. Arching action for loads close to the support was also considered in both one-way shear and punching shear predictions. Considering the whole database and without separation by the failure mechanism, the ratio between tested and predicted resistances with the one-way shear expressions shows an average value of 1.22 and a coefficient of variation of 18.3%. The respective ratio between tested and predicted resistances with the punching shear expressions reached an average ratio of 1.23 with a coefficient of variation of 21.3%. Therefore, the proposed recommendations allow for reaching enhanced levels of precision in assessing the shear and punching shear capacity of one-way slabs under concentrated loads, regardless of the governing failure mechanism of the slabs.

Keywords: one-way shear, punching shear capacity, one-way slabs, concentrated loads.

Resumo: Este estudo descreve um conjunto de recomendações para melhorar a precisão nas previsões de resistência ao cisalhamento e à punção de lajes unidirecionais sob cargas concentradas, independentemente do mecanismo governante na ruptura, utilizando as disposições da norma ABNT NBR 6118:2014. Para isso, foi organizado um banco de dados com 143 resultados de ensaios, incluindo lajes unidirecionais com diferentes condições de apoio e configurações de carregamento e que apresentaram diferentes mecanismos de ruptura por cisalhamento. Os principais parâmetros que influenciam a capacidade de carga e o mecanismo de ruptura dessas lajes foram considerados nas recomendações propostas. Ajustes para a definição da largura

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Financial support: The authors acknowledge the financial support provided by the São Paulo Research Foundation (FAPESP) (grant number #2018/21573-2, grant number #2019/20092-3 and grant number #2021/13916-0) and by Brazilian National Council for Scientific and Technological Development (CNPq) (grant number #303438/2016-9). This study was also partially financed by the Coordenação de Aperfeiçoamento de Pessoal de Nível Superior – Brasil (CAPES) – Finance Code 001.
Conflict of interest: Nothing to declare.
Data Availability: the main database used in the investigations is available through the open-access repository Zenodo at: https://zenodo.org/record/5911469 (accessed on 24/11/2022). Other supplementary data that support the findings of this study are available from the corresponding author, [AMDS], upon reasonable request.

efetiva de cisalhamento e perímetro de controle resistente ao cisalhamento foram descritos. A ação de arqueamento para cargas próximas ao apoio também foi considerada nas previsões de resistência ao cisalhamento unidirecional e ao puncionamento. Considerando todo o banco de dados, a razão entre as resistências testadas e previstas com as expressões de cisalhamento unidirecional apresenta um valor médio de 1.22 e um coeficiente de variação de 18.3%. A respectiva relação entre as resistências testadas e previstas com as expressões de puncionamento atingiu uma relação média de 1.23 com um coeficiente de variação de 21.3%. Portanto, as recomendações propostas permitem alcançar maiores níveis de precisão na previsão da resistência ao cisalhamento e ao puncionamento de lajes unidirecionais sob cargas concentradas, independentemente do mecanismo de ruptura governante das lajes.

Palavras-chave: cisalhamento unidirecional, resistência à punção, lajes unidirecionais, cargas concentradas.

How to cite: A. M. D. Sousa, E. O. L. Lantsoght, D. L. Araújo, L. P. Prado, and M. K. E. Debs, "Recommendations to assess the shear and punching capacity of one-way slabs under concentrated loads according to the ABNT NBR 6118:2014," *Rev. IBRACON Estrut. Mater.*, vol. 16, no. 3, e16308, 2023, https://doi.org/10.1590/S1983-41952023000300008

1 INTRODUCTION

In the last decades, many bridges built between 1960 and 1970 are reaching the end of their designed service life [1]. To extend the service life of these structures, it is necessary to attest that these bridges meet the requirements of the current design codes, considering that sometimes traffic loads and intensities have increased and the resistance models have become more conservative for certain applications. In this context, many bridge deck slabs (Figure 1a – Brazilian design truck) were rated as critical in shear assessments, despite these structures not showing any signs of distress upon inspection. Consequently, it was concluded that some widespread evaluation approaches could be overly conservative, which motivated the testing of one-way reduced-scale slabs under concentrated loads (Figure 1b) in laboratories from several countries, mainly in Europe [2]–[7].

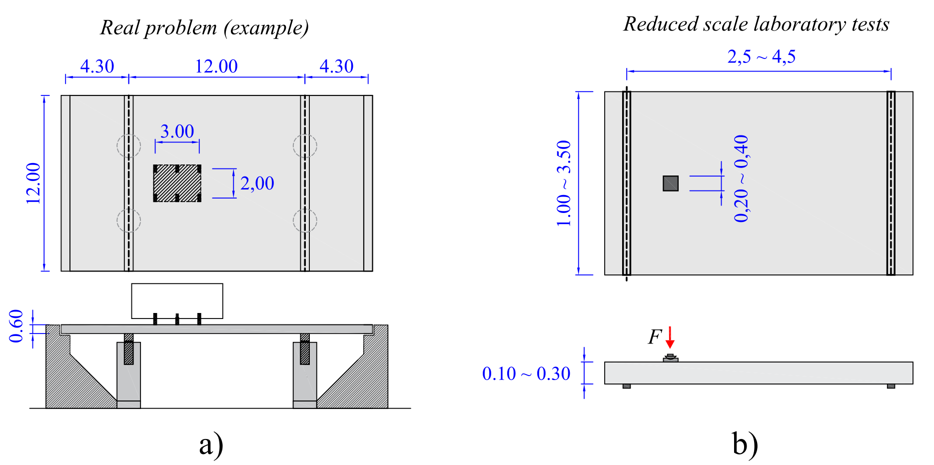

Figure 1 – a) Example of one-way slabs loaded by the design truck prescribed in Brazilian code ABN NBR 7188:2013 [8]; b) example of reduced-scale laboratory test of a one-way slab under a single concentrated load. Dimensions in m.

One-way slabs under concentrated loads may present a more complex failure mechanism compared to beams loaded over the entire width (Figure 2a) or with slab-column connections under concentric loads (Figure 2b). In fact, these members may fail either by one-way shear as wide beams or by punching shear around the load, depending on the load position, slab width b_{slab}, and other parameters. In addition, two-way flexure influences the outcomes. Nevertheless, until now, most publications addressed only how to check the one-way shear capacity of such slabs [4], [6], [7], [9], sometimes neglecting that the evaluated slabs failed by punching. Additionally, most analytical approaches to evaluate the one-way shear capacity or punching capacity of these slabs with design code expressions show a large scatter between tested and predicted resistances [3], [10]. In practice, most available approaches to evaluate the one-way shear capacity of these slabs overestimate the contributing slab strip to the shear capacity (frequently named the effective shear width b_{eff}) when the distance from the load to the support increases [3], [9]. Besides that, no design codes address how the slab width could influence the effective contribution of some sides of the control perimeter to the punching capacity. At this point, the reader shall realize that the free edges decrease the shear flow that goes through the sides of the control perimeter in the transverse direction, depending on the slab width [11] (see Figure 2c).

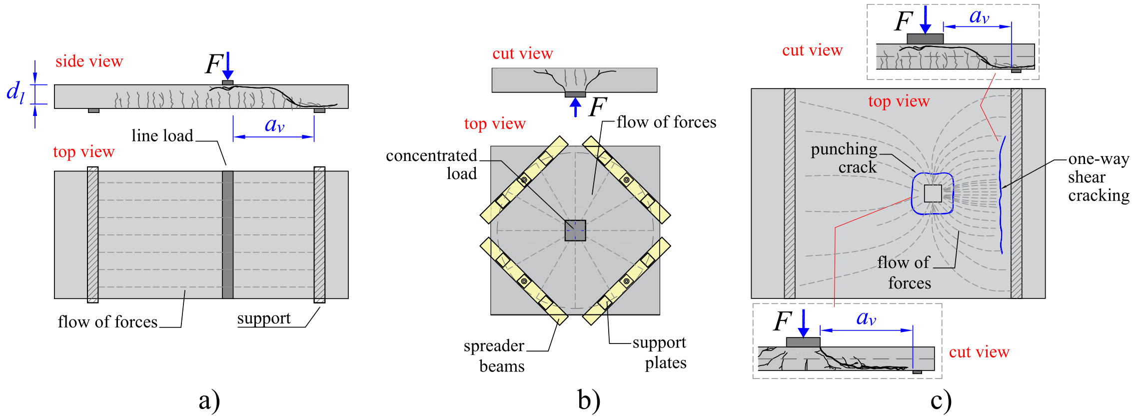

Figure 2 - One-way slabs loaded over the entire width failing in one-way shear; b) slab-column connections failing in punching; and c) one-way slabs under concentrated loads subjected to both one-way shear and punching shear failures (adapted from [12]). Note: a_v is the clear distance between the faces of support and load and d_l is effective depth to longitudinal reinforcement.

In 2019, the preliminary investigation of one-way slabs under concentrated loads with the Brazilian code expressions for shear and punching shear was performed using the most traditional rules to define the effective shear width b_{eff} and the shear resisting control perimeter for punching [3]. At that time, some shortcomings were highlighted: (i) it was concluded that the one-way shear predictions tend to become unsafe when the slabs fail by punching, (ii) additionally, the predictions of punching capacity also became unsafe when the tests failed in one-way shear as wide beams. Since the most critical failure mechanism of the slabs is not known in most cases and depends on a large number of parameters, it would be important to assure conservative predictions regardless of the most critical failure mechanism of the slabs being the one-way shear or punching shear. Until now, no specific recommendation has been published addressing this issue regarding the ABNT NBR 6118:2014 code expressions.

In this study, we propose to describe a set of enhanced recommendations to assess the shear and punching capacity of one-way slabs under concentrated loads using the current ABNT NBR 6118:2014 [13] expressions for one-way shear and two-way shear (punching). Since these slabs may fail either by one-way shear or two-way shear, the idea of this work is to improve the level of accuracy of both one-way shear and punching resistance approaches in such a way as to provide the most accurate predictions of failure load, regardless of the most critical failure mechanism.

Section 2 discusses the traditional approaches to define the effective shear width and shear resisting control perimeter for punching capacity evaluations. In Section 3, the one-way shear and punching shear expressions of the ABNT NBR 6118:2014 [13] are presented. At this point, the Section 2 and Section 3 combination represents the approach to be used in the evaluations, which can be compared with the proposed approach. Next, Section 4 brings the recommendations developed to improve the predictions of shear and punching capacity with the Brazilian code expressions, regardless of the governing failure mechanism of the slabs being one-way shear or punching. In Section 5, the database of one-way slabs under concentrated loads used to validate the proposed recommendations is discussed (143 test results). In the end, a comparison between tested and predicted resistances using one-way shear and punching shear expressions is described (Section 6), comparing the predictions with and without the proposed recommendations.

2 BACKGROUND

2.1 One-way shear

The traditional approach to evaluate the one-way shear capacity of one-way slabs under concentrated loads is based on the definition of a slab strip that it is supposed to contribute effectively to the one-way shear capacity, called effective shear width b_{eff}. Theoretically, this effective shear width can be defined based on the distribution of shear demand v_E (shear force per unit length) from linear elastic finite element analyses, for instance (Figure 3a). In this way, the effective shear width would be defined as the length that multiplied by the peak shear demand equals the total shear force V_E. However, analytically, the most traditional approaches to defining the effective shear width are based on the assumption of a horizontal load spreading from the loading plate to the supports under a fixed angle, typically 45 degrees (Figure 3b-3c).

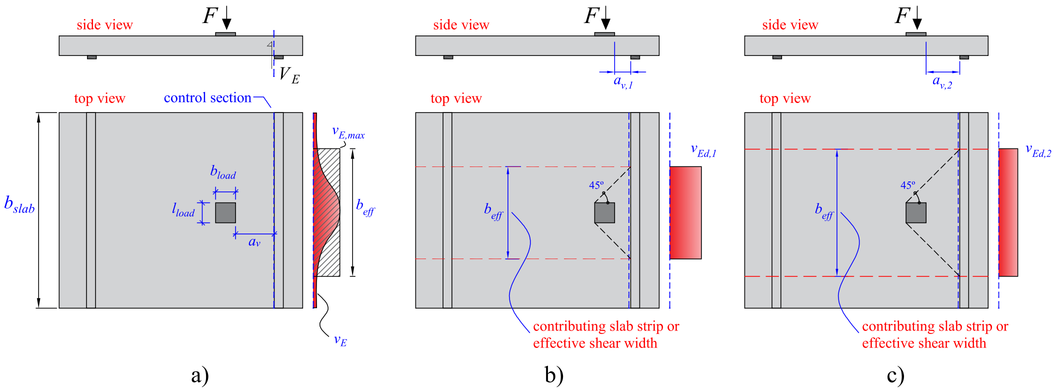

Figure 3 – a) Definition of the effective shear width based on the distribution of the unitary shear demand v_E; b) and c) effective shear width according to the French guidelines varying the clear shear span a_v.

The effective shear width calculated as in Figure 3b is commonly named the French effective shear width, as it was first identified on the French guidelines of design [14], [15]. This approach has already been demonstrated to provide good predictions of shear capacity, mainly for loads close to the support $a_v < 2d_l$ [2], [3]. However, Figure 3c shows that this approach assumes that the effective shear width increases by increasing the clear shear span a_v (herein, a_v is the clear distance between load and support). Because of this, some studies have already identified that this approach leads to an overestimation of the one-way shear capacity for tests that failed in punching [16].

2.2 Punching shear

In evaluating the punching capacity of one-way slabs under concentrated loads with the Brazilian code expressions, a well-consolidated approach compares different layouts of the control perimeter to define the most critical mechanism (Figure 4). The shear resisting control perimeter (blue lines) is considered at the distance of $2d_{avg}$ of the loaded area ($k = 2$ in Figure 4).

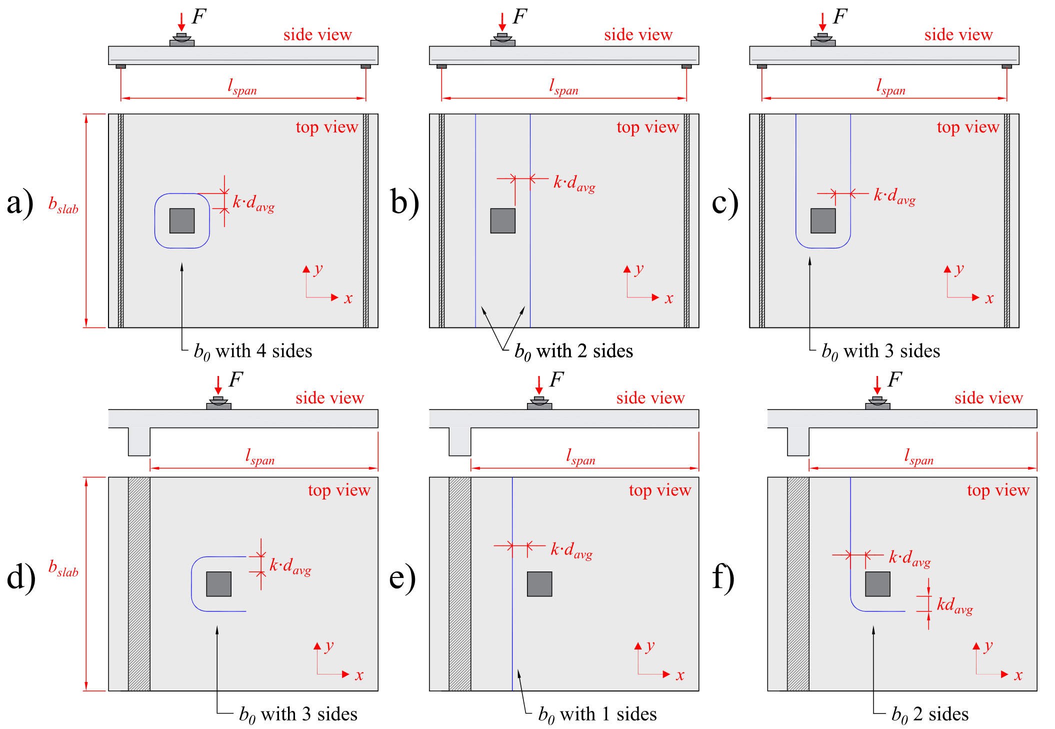

Figure 4 – Possible definition of the shear resisting control perimeter for simply supported (a, b and c) and cantilever slabs (d, e and f) (adapted from [12]). Note: in the Brazilian code [13], $k = 2$; d_{avg} is the average effective depth of the tensiled reinforcement; b_{slab} is the slab width.

In practice, the reader may realize that the most critical shear resisting control perimeter will be a function of parameters such as the slab width and load position. For instance, the control perimeter b_0 with two sides tends to govern over the perimeter with four sides for slabs with a reduced slab width (Comparing Figure 4a and Figure 4b). In the same way, the control perimeter with three sides governs when the concentrated loads are placed close to the free edges of simply supported slabs (layout of Figure 4c).

In the Brazilian code ABNT NBR 6118:2014 [13], the shear stress concentration in the case of loads close to the free edge of simply supported slabs (Figure 5a-5b) and cantilevers slabs (Figure 5c-5d) is considered through the definition of a reduced control perimeter (dashed red lines) that is a function of the slab effective depth d_{avg} and size of the concentrated load). In Figure 5b-5d, note that the lengths b_{load} and l_{load} refer to the size of the concentrated loads in the spanning and transverse directions, respectively. B_1 and B_2 refer to the size of the control perimeter considered in the transverse and spanning directions (x and y) to consider the shear stress concentration at the load corners depending on the loaded area geometry.

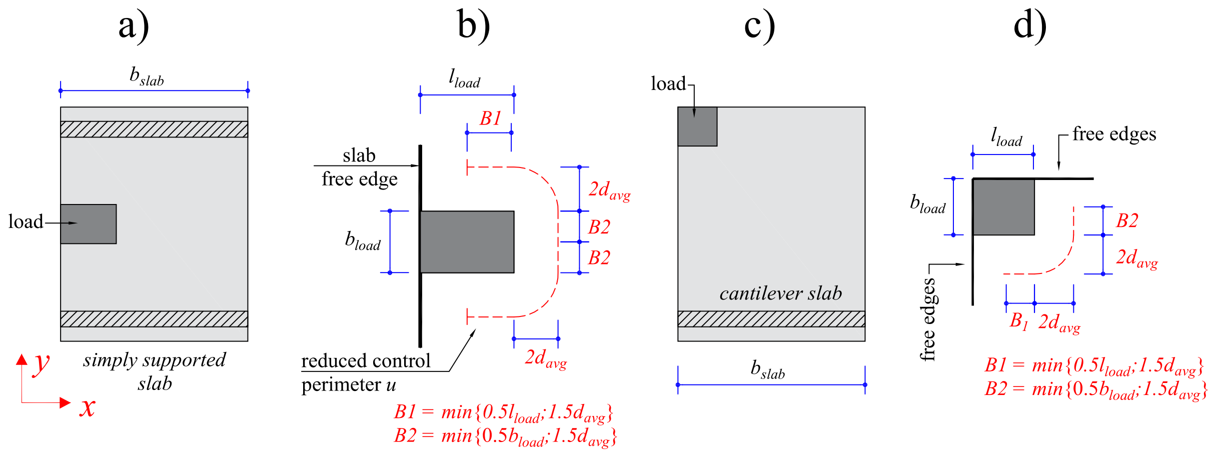

Figure 5 - Definition of the reduced control perimeter for a) and b) concentrated loads close to free edges of simply supported slabs; and for c) and d) corner of cantilever slabs according to the ABNT NBR 6118:2014. Note: l_{load} and b_{load} refer to the size of the concentrated loads; *B1* and *B2* refer to the lengths of the control perimeter.

No clear mention about the influence of the rectangular index of the load α in the definition of the control perimeter is provided in the current Brazilian code, with α given by Equation 1:

$$\alpha = \max\{l_{load};b_{load}\} / \min\{l_{load};b_{load}\} \tag{1}$$

In practice, when the rectangular index α increases and the load becomes more elongated in the span direction, the shear stresses concentrate in the corners (Figure 6a.1). However, when the elongated side is running parallel to the line support (Figure 6a.2), the shear flows in the elongated side is predominant, and hence, a lower reduction in the resisting control perimeter occurs [17]. Since the effect of the rectangular index is similar to that of edge and corner columns [17] for flat slabs, it is assumed that the sketch of Figure 6 should be used in the case of rectangular loads evaluated in this study. In the same context, cantilever slabs under concentrated loads should be evaluated as slab-edge column connections in the definition of the reduced control perimeter (Figure 6b).

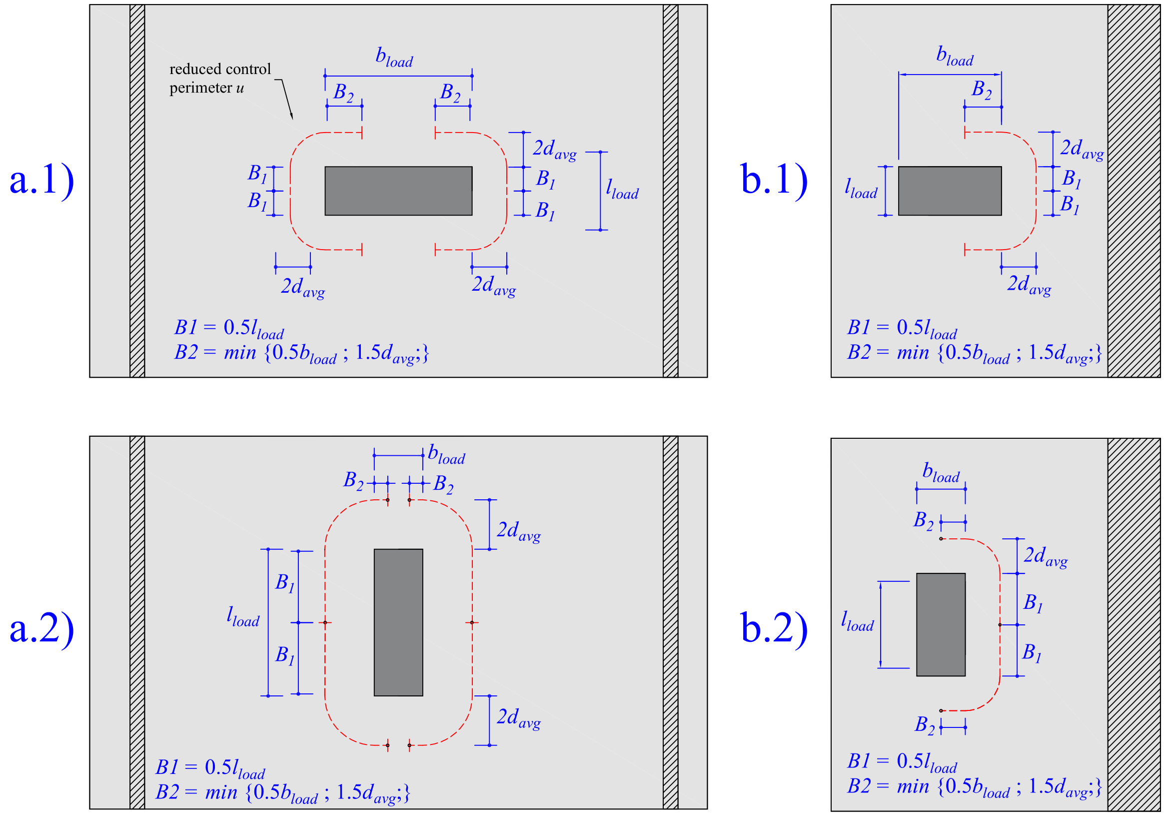

Figure 6 - Definition of the reduced control perimeter for simply supported and cantilever slabs under concentrated loads according to the Brazilian code ABNT NBR 6118:2014 [13].

3 EXPRESSIONS TO EVALUATE THE ONE-WAY SHEAR AND PUNCHING CAPACITY ACCORDING TO THE ABNT NBR 6118:2014

The one-way shear capacity V_R of slabs is predicted by multiplying the nominal shear capacity (shear force per unit length $v_{R,shear}$) of these slabs by an effective shear width b_{eff} (Equation 2):

$$V_R = v_{R,shear} \cdot b_{eff} \tag{2}$$

The punching capacity P_R, in the same way, is calculated by multiplying the nominal punching capacity (shear capacity per unit length $v_{R,punch}$) by the shear resisting control perimeter b_0 (Equation 3):

$$P_R = v_{R,punch} \cdot b_0 \tag{3}$$

3.1 One-way shear capacity according to the ABNT NBR 6118:2014

According to the ABNT NBR 6118:2014 [13], the nominal one-way shear capacity of reinforced concrete members without stirrups is calculated as follows (Section 19.4, SI units, Equation 4):

$$v_{R,shear} = \left[\tau_{Rd} \cdot k_{shear} \cdot \left(1.2 + 40 \cdot \rho_1\right) + 0.15 \cdot \sigma_{cp}\right] \cdot d_l$$
$$\text{with } \tau_{Rd} \text{ and } \sigma_{cp} \text{ in [MPa]} \tag{4}$$

$$\tau_{Rd} = 0.25 \cdot f_{ctd} \tag{5}$$

$$f_{ctd} = f_{ctk,\inf} / \gamma_c \tag{6}$$

$$f_{ctk,\inf} = 0.7 f_{ctm} \tag{7}$$

$$f_{ctm} = \begin{cases} 0.3 f_{ck}^{\ 2/3} & \text{for } f_{ck} \le 50 \text{ MPa} \\ 2.12 \ln\left(1 + 0.11 f_{ck}\right) & \text{for } 50 \text{ MPa} < f_{ck} \le 90\text{MPa} \end{cases} \tag{8}$$

$$\rho_l = \frac{A_s}{b_w \cdot d_l} \le 0.02 \tag{9}$$

$$k_{shear} = \begin{cases} 1 \text{, if at least } 0.50 A_s \text{ does not reach the support} \\ \left|1.6 - d_l\right| \text{, with } d_l \text{ in [m], for the other cases} \end{cases} \tag{10}$$

τ_{Rd} is the design shear capacity of the concrete (in MPa, Equation 5); k_{shear} is the size effect factor for one-way shear in the Brazilian code (see Equation 10); ρ_l is the reinforcement ratio in the longitudinal direction (Equation 9); σ_{cp} is the external axial stress in the section (tensile stress are considered with a negative signal); d_l is the effective depth towards the longitudinal reinforcement; f_{ctd} is the design value of the tensile strength of concrete (Equation 6); $f_{ctk,inf}$ is the lower bound value of the characteristic tensile strength of concrete (Equation 7); γ_c is the concrete safety factor (assumed equal to 1 in the comparisons between tested and predicted resistances from this paper and 1.4 in design calculations from professional practice); f_{ctm} is the mean value of tensile strength of concrete (Equation 8); f_{ck} is the characteristic value of compressive strength of concrete (in the comparisons between tested and predicted resistances, f_{ck} was replaced by f_{cm}); b_w is the considered length in the evaluation of A_s and; A_s is the are of longitudinal reinforcement distributed along b_w.

The ABNT NBR 6118:2014, as well as most design codes, does not guide how to define the effective shear width of slabs under concentrated loads. In the ABNT NBR 6118:1980 [18] (replaced version), the following expression was provided (Equations 11 and 12):

$$b_{NBR} = l_{load} + h_{slab} \tag{11}$$

$$\begin{aligned} &\text{if } b_{NBR} > l_{span}: \ b_{eff,NBR} = b_{NBR} \\ &\text{if } b_{NBR} \le l_{span}, \ b_{eff,NBR} = \begin{cases} b_{NBR} + 0.5 \cdot a \cdot \left(1 - \dfrac{b_{NBR}}{l_{span}}\right), & \text{if cantilever slab} \\ b_{NBR} + a \cdot \left(1 - \dfrac{b_{NBR}}{l_{span}}\right), & \text{for simply sup. and continuous slabs} \end{cases} \end{aligned} \tag{12}$$

Herein, h_{slab} is the slab thickness; l_{load} is the side of the load in the width direction of one-way slabs; b_{load} is the side of the load in the spanning direction of one-way slabs; l_{span} is the span length, a is the shear span between axes of support and load; and a_v is the clear distance between the support and the load.

In most publications [3], [10], however, the commonly named French effective shear width model is employed (Equation 13, see Figure 3b-3c):

$$b_{eff,french} = l_{load} + 2 \cdot (a_v + b_{load}) \leq b_{slab} \tag{13}$$

This occurs since it leads to reasonable levels of accuracy for the tests that fail as wide beams in one-way (WB) shear [16] or for loads close to the support [2] compared to other approaches. Figure 7 compares the effective shear widths $b_{eff,NBR}$ and $b_{eff,french}$ for a simply supported slab ($b_{load} = l_{load} = 0.40$ m; $b_{slab} = 3.0$ m and $h_{slab} = 0.30$ m). As can be seen, the predicted effective shear width with the replaced Brazilian code is significantly lower than that predicted with the French effective shear width.

In this study, it was verified that the predictions with the French effective shear width model would provide the best results between the two approaches, mainly for the tests with $a_v/d_l \leq 2$. Based on that, the one-way shear capacity with the reference approach was assumed as Equation 14:

$$V_{R,reference} = v_{R,shear} \cdot b_{eff,french} \tag{14}$$

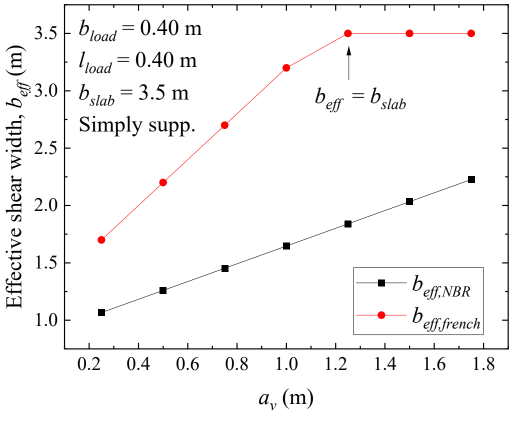

Figure 7 - Comparison between the predicted effective shear with according to the replaced Brazilian code $b_{eff,NBR}$ (ABNT NBR 6118:1980) and the French effective shear width model $b_{eff,french}$.

3.2 Nominal punching capacity according to the ABNT NBR 6118:2014

According to ABNT NBR 6118:2014, Section 19.5.3.2, the nominal punching capacity $v_{R,punch}$ (shear force per unit length), can be calculated as in Equation 15:

$$v_{R,punch} = \frac{0.18}{\gamma_c} k_{punch} (100 \rho_{avg} f_{ck})^{1/3} \cdot d_{avg} \tag{15}$$

$$\rho_{avg} = (\rho_l \cdot \rho_t)^{1/2} \tag{16}$$

$$d_{avg} = (d_l + d_t)/2 \tag{17}$$

$$k_{punch} = 1 + \sqrt{\frac{200}{d_{avg}}} \leq 2, \text{ with } d_{avg} \text{ in [mm] and } f_{ck} \text{ in [MPa]} \quad (18)$$

k_{punch} is the size effect factor for punching (Equation 18). In the current code [13], k_{punch} is not explicitly limited to 2. In practice, this recommendation appears only in the book of recommendations and examples of the code application [19], which was followed in the study; ρ_{avg} is the average reinforcement ratio considered for punching (Equation 16); ρ_l and ρ_t are the reinforcement ratios in the longitudinal and transverse directions, respectively, d_{avg} is the average effective depth of the reinforcement for punching (Equation 17).

In the case of slabs with large thicknesses, the self-weight may significantly increase the shear demand around the control perimeter (Figure 8). In this case, it shall be noted that the control perimeter shall resist the stresses caused by the concentrated load and the those caused by the self-weight. In the literature, it is usually considered that the self-weight acts only in the longitudinal direction of one-way slabs (based on the shear flow considering only the self-weight) [20]. Using this assumption, a net shear resistance can be calculated on the sides of the control perimeter influenced by the self-weight $v_{R,net}$. In this way, the shear demand caused by the self-weight (v_{sw}) shall be subtracted from the unitary shear resistance calculated by the code expression $v_{R,punch}$ (see Equation 19):

$$v_{R,net} = v_{R,punch} - v_{SW} \quad (19)$$

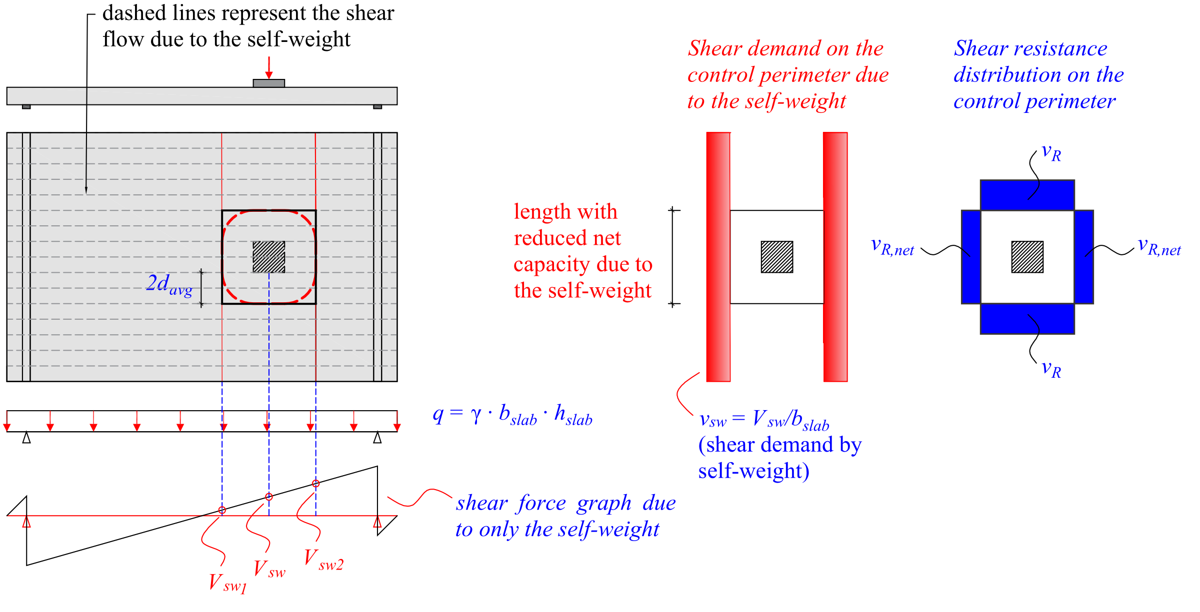

Figure 8 - Effect of the self-weight on the shear demand around the control perimeter and definition of the net shear resistance (the control perimeter was represented with a square shape for simplicity).

In this study, the reference punching capacity (without taking into account the arching action and other parameters influencing the problem) can be expressed as in Equation 20:

$$P_{R,reference} = v_{R,net} \cdot \left(b_{0x,1} + b_{0,x2}\right) + v_{R,punch} \cdot \left(b_{0y,1} + b_{0,y2}\right), \text{ for SS and CS slabs}$$
$$P_{R,reference} = v_{R,net} \cdot b_{0,x1} + v_{R,punch} \cdot \left(b_{0y,1} + b_{0,y2}\right), \text{ for CT slabs} \quad (20)$$

4 PROPOSED APPROACHES

4.1 Proposed approach for the one-way shear predictions

Loads close to the support ($a_v/d_l \leq 2$) benefit from arching action to transmit the load to the supports [21]. In the current Brazilian code [13], this effect is mentioned only for beams. To avoid overly conservative predictions of the shear capacity for loads close to the support, the arching action is considered for slabs in the proposed approach through a factor β, as suggested in the *fib* Model Code 2010 [22] and current European codes [23] (Equation 21):

$$\beta = \frac{a_v}{2d_l}, \text{ with } 0.25 \leq \beta \leq 1 \tag{21}$$

Along the effective shear width, it is assumed that the unitary shear capacity can be enhanced using the following expressions (Equation 22):

$$\begin{aligned} V_{R,proposed} &= \left(v_{R,shear} \cdot \mu_{shear,1}\right) \cdot b_{eff,proposed} \\ \mu_{shear,1} &= 1/\beta \end{aligned} \tag{22}$$

Another key aspect of the predictions of one-way shear capacity is the definition of the effective shear width [2], [3]. Some publications identified that the French effective shear width tends to overestimate the contributing slabs strip for one-way shear predictions when the loads are placed far away from the support ($a_v/d_l > 2$, for instance) or when the slabs are critical in punching instead of one-way shear [3], [16]. Consequently, the predicted one-way shear capacity commonly exceeds the tested one-way shear resistance V_{test} (sectional shear reached in the test). Since for the design or assessment of existing structures, we do not know a priori which is the most critical failure mechanism, conservative predictions for shear and punching capacities should be obtained.

Since the French effective shear width model $b_{eff,french}$ provides good levels of predictions combined with the ABNT code expressions for loads close to the support ($a_v/d_l < 2$) [2], [3], [16], this approach is used as a starting point. Based on regression analyses of $V_{test}/V_{R,predicted}$ using $\mu_{shear,1}$ and $b_{eff,french}$, a factor $\mu_{shear,2}$ was derived to correct the predicted effective shear width according to the shear slenderness a_v/d_l (Equation 24). In this way, the predicted effective shear width $b_{eff,proposed}$ (Equation 23), decreases as the load distance from the support increases. Consequently, the predictions of the one-way shear capacity improve for the tests that present a local failure close to the load by asymmetrical punching:

$$b_{eff,proposed} = b_{eff,french} \cdot \mu_{shear,2} \begin{cases} \leq b_{slab} \\ \geq l_{load} + 4d_l \end{cases} \tag{23}$$

$$\begin{aligned} \mu_{shear,2} &= -0.184 \cdot a_v / d_l + 1.376, \text{for CT} \\ \mu_{shear,2} &= -0.128 \cdot a_v / d_l + 1.280, \text{for CS and SS} \end{aligned} \tag{24}$$

On which CT = load applied on cantilever slab, CS = loads close to continuous support and SS = load close to simple support (hinged support).

4.2 Proposed approach for the punching shear predictions

Inspired by the work from Regan [24], this paper suggests considering the enhanced shear capacity for the side of the control perimeter facing to the support when the load is placed at distances $a_v \leq 2d_l$. This is accomplished by multiplying the unitary shear capacity $v_{R,net}$ in $b_{0,x1}$ by the factor μ_{punch1} (see Figure 9), which has the same expressions as $\mu_{shear,1}$. In this way, the arching action for loads close to the support is considered only for the relevant part of the control perimeter.

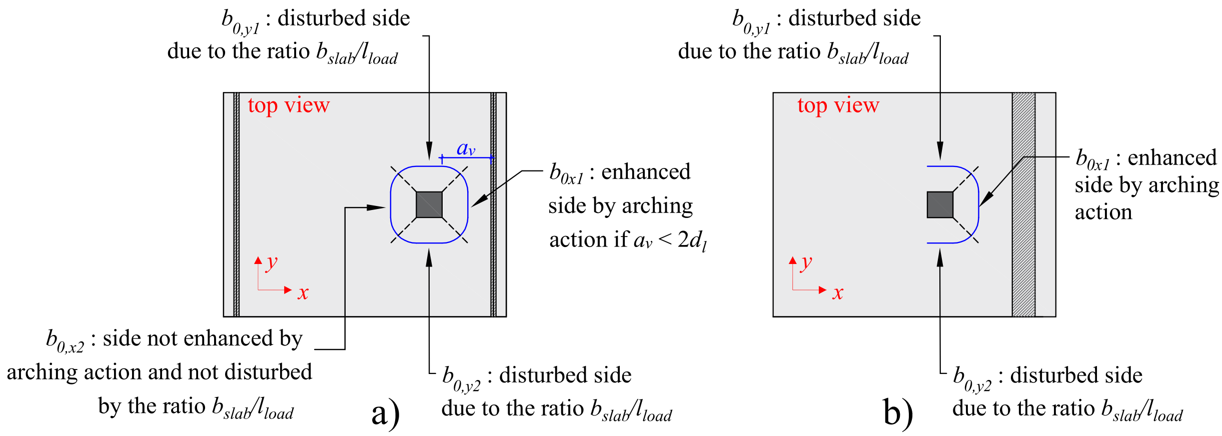

Figure 9 - Sketch of the assumed control perimeter sides enhanced by arching action according to the ratio a_v/d_l, and disturbed according to the ratio b_{slab}/l_{load} (adapted from [12]).

When the loads are placed close to the support, the intersection of the control perimeter with the support should be considered in the definition of the sides of the control perimeter (Figure 10a). In practice, different assumptions could be used to define the length $b_{0,x1}$, $b_{0,x2}$, $b_{0,y1}$ and $b_{0,y2}$. For instance, someone could consider $b_{0,x1}$ as only the straight length that touches the support between the dashed blue lines (Figure 10c). However, this tends to underestimate the length $b_{0,x1}$ when the control perimeter when the load is placed too close of the support ($a_v \approx 0$). Another definition can be based on the variable angle of the reference dashed lines (blue lines in Figure 10). For instance, Figure 10d assumes that the reference line touches the intersection of the support with the control perimeter, which also underestimates the length of the side $b_{0,x1}$ when the load is place at $a_v = 2d_{avg}$. In this study, it is assumed that the reference dashed line always touches the middle of the rounded side to define the length $b_{0,x1}$ (Figure 10e). In summary, we start calculating the point on which the control perimeter intercepts the support and, after, we calculate the length of the rounded corner. In the end, we add the straight length with half of the rounded corners to define $b_{0,x1}$.

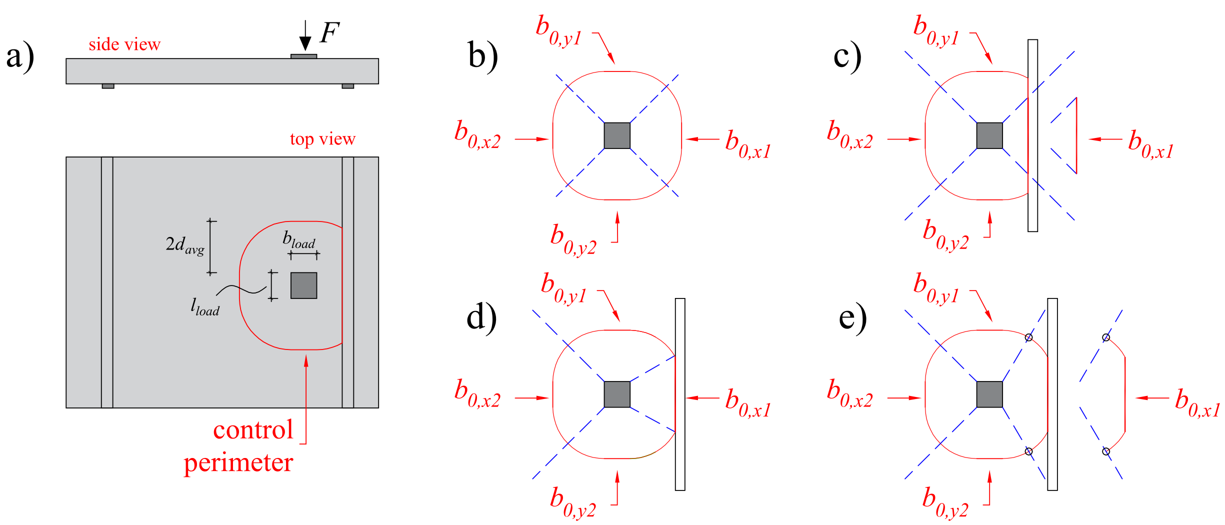

Figure 10 - Definition of the lengths $b_{0,x1}$, $b_{0,x2}$, $b_{0,y1}$ and $b_{0,y2}$ when the control perimeter intercepts the support: a) sketch of the control perimeter when the load is placed at $a_v < 2d_{avg}$; b) original control perimeter without intersection with the support; c) approach 1 with a fixed angle of the reference dashed line; d) approach 2 with a variable angle of the reference dashed line and e) approach 3 with a variable angle of the reference dashed line (used in this study).

Additionally, the second aspect to be considered in the punching capacity predictions is the effect of the slab width-to-load size (b_{slab}/l_{load}) in the effective contribution of the lateral sides of the control perimeter ($b_{0y,1}$ and $b_{0,y2}$). In practice, by decreasing the slab width and fixing other parameters, a lower shear flow is transferred by the lateral sides of the control perimeter ($b_{0y,1}$ and $b_{0,y2}$) [12]. Therefore, these sides provide a lower contribution to the punching capacity compared to the sides $b_{0,x1}$ and $b_{0,x2}$.

In this study, it is proposed to multiply the unitary shear resistance of the sides $b_{0,y1}$ and $b_{0y,2}$ by the factors $\mu_{punch,2}$ (Equation 25):

$$\mu_{punch,2} = \begin{cases} 0.14 \cdot \lambda - 0.14, \text{ if } \lambda \leq 8 \\ 1, \text{ for } \lambda > 8 \end{cases}$$
$$\lambda = \left(b_{slab} - l_{load}\right) / \left(2 \cdot d_{avg}\right) \tag{25}$$

The effect of the non-proportional shear demand between the frontal and back sides of the load also influences the ultimate capacity of the slabs. In practice, this can be explained by the asymmetrical punching failure around the load for such slabs when the loads are placed closer to the support [6]. Since one side of the control perimeter is more heavily loaded than the other, the less demanded side contributes less to the punching capacity. Comparisons between tested and predicted resistances in this study, however, indicate that this effect would have a considerable influence only for cantilever slabs, which behave as edge columns. However, for concentrated loads close to the free edge of simply supported or continuous slabs, a similar effect can be expected (in this study, we evaluated only slabs under concentrated loads placed at mid-width). Therefore, a third factor that considers the non-proportional shear demand in the shear span direction is employed only for cantilever slabs (Equation 26):

$$\mu_{punch,3} = \begin{cases} 1, \text{ for simply supported and continuous slabs} \\ 0.728 \cdot \left(a / l_{span}\right)^{0.71}, \text{ for cantilever slabs } \left(a / l_{span} \geq 0.20\right) \end{cases} \tag{26}$$

Therefore, the total punching capacity can be calculated for simply supported and continuous slabs as in Equation 27:

$$P_{R,proposed} = \left(v_{R,net} \cdot \mu_{punch,1}\right) \cdot b_{0,x1} + v_{R,net} \cdot b_{0,x2} + v_{R,punch} \cdot \left(b_{0,y1} + b_{0,y2}\right) \cdot \mu_{punch,2} \tag{27}$$

and can be calculated for cantilever slabs as in Equation 28:

$$P_{R,proposed} = \left[\left(v_{R,net} \cdot \mu_{punch,1}\right) \cdot b_{0,x1} + v_{R,punch} \cdot \left(b_{0,y1} + b_{0,y2}\right) \cdot \mu_{punch,2}\right] \cdot \mu_{punch,3} \tag{28}$$

Since the effect of the self-weight in the shear demand calculated at the control perimeter sides v_{sw} was limited when compared to $v_{R,punch}$ in the database, (due to the large reinforcement ratios and thickness employed in most tests), one can replace the term $v_{R,net}$ by $v_{R,punch}$ for simplicity. However, such simplifications shall be avoided in the evaluation of bridge decks slabs, on which the self-weight is considerably larger than for laboratory tests [20], [25].

5 DATABASE OF TEST RESULTS

A database of test results was organized for the evaluation of the proposed approach and it was published in the public domain [26]. This dataset contains 143 test results of slabs under concentrated loads failing by shear as wide beams (WB: 91 tests), punching (P: 40 tests) or a mixed mode between shear and punching (WB+P: 12 tests).

The dataset includes tests from the following references: Bui et al. [5], Carvalho [27], Coin and Thonier [14], Damasceno [28], Ferreira [29], Lantsoght [30], Natário et al. [7], [31], Reiβen [32], Regan [24], Regan and Rezai-Jarobi [33], Rombach and Latte [34], [35], Rombach and Henze [36], Vaz Rodrigues [37] and Vida and Halvonik [38].

Table 1 - Ranges of parameters in the database.

Parameter	min	max
h [m]	0.10	0.30
b_{slab} [m]	0.60	4.50
l_{span} [m]	0.90	4.00
b_{slab}/l_{load} [-]	1.67	23.08
b_{slab}/d_l [-]	5.66	29.41
a_v/d_l [-]	0.24	7.66
f_c [MPa]	19.20	77.74
ρ_l [%]	0.602	2.150
ρ_t [%]	0.132	1.526

Table 1 shows the ranges of parameters in the database. The reinforcement ratio of the slabs ρ_l and ρ_t reported in the database and used in the calculations was recalculated based on the spacing of the flexural rebars. The value of the compressive strength measured on cube specimens was corrected by a factor of 0.82 to estimate the compressive strength on cylinder specimens [2]. Only tests with a ratio $b_{slab}/d_l \geq 5$ were included in the dataset to fit the requirement of the ABNT NBR 6118:2014 regarding the definition of the slab members. Besides, only tests with (b_{slab}-l_{load}) > $4d_l$ were evaluated in this dataset to increase the proportion of tests that could be critical to both shear and punching failures. At this point, the reader shall realize that including members almost loaded over the entire width $(b_{slab}-l_{load})/2d_l \leq 4$ would significantly increase the proportion of tests failing as wide beams over the tests that failed by punching.

6 RESULTS AND DISCUSSIONS

6.1 Comparison between tests and studied methods

Figure 11 compares the ratio between tested and predicted resistances for one-way shear and punching shear analyses. In the comparisons between tested and predicted resistances, partial safety factors were assumed equal to 1 and measured material properties were used instead of characteristic values. The results "No μs" are the ones following the traditional approaches of evaluation according to the Brazilian code (Section 2 and 3 of this paper) and do not include the proposed factors μ for shear and punching capacity predictions. In this way, $V_{R,predicted}$ and $P_{R,predicted}$ are equal, respectively, to $V_{R,reference}$ and $P_{R,reference}$ for "No μs". The results "with μs" are the ones following the recommendations of Section 4 of this paper. In this way, $V_{R,predicted}$ and $P_{R,predicted}$ are equal to $V_{R,proposed}$ and $P_{R,proposed}$, respectively. In Figure 9, the following notations are applied: WB: test failed as a wide beam in one-way shear; P: test failed by punching; and WB+P: the test failed by a mixed mode between one-way shear and punching.

Figure 11a shows two main aspects of the results without the proposed recommendations (No μs): (i) neglecting the arching action for slabs under concentrated loads close to the support can significantly underestimate the ultimate capacity of the slabs in one-way shear (see Detail 1 in Figure 11a); and (ii) the predictions of one-way shear capacity become critically unsafe for large shear slenderness, for instance, when $a_v/d_l > 4$. In the last case, this occurs because the French effective shear width increases by increasing the shear slenderness a_v/d_l and the ultimate load that causes the failure (P_{test}) does not increase by increasing a_v/d_l [6]. In fact, most tests with $a_v/d_l > 4$ failed by punching or a mixed mode between one-way shear and punching (see Detail 2 in Figure 11a). For such tests, increasing the ratio a_v/d_l increases the slab rotations around the load and, consequently, the crack opening for the same load level, which results in lower punching capacities according to the Critical Shear Crack Theory [39]. Consequently, the predicted one-way shear capacity increased excessively by increasing the shear slenderness.

Using the proposed recommendations through the factors $\mu_{shear,1}$ and $\mu_{shear,2}$ (Section 4), the average ratio $V_{test}/V_{R,predicted}$ changes from 1.37 to 1.22 and the coefficient of variation decreases from 63.7% to 18.3% (see Figure 11c). Therefore, using the proposed recommendations allows for reaching enhanced predictions of shear capacity, even when the tests failed by punching for large values of a_v/d_l.

Figure 11b shows that without the proposed recommendations for punching (No μs), the predictions of punching capacity can be critically unsafe for the slabs that failed as wide beams in one-way shear (WB) due to the small ratio b_{slab}/l_{load} (see Detail 1 in Figure 11b). Besides, the predictions of punching capacity can, in the same way, be overly conservative if arching action is not considered in the calculations (tests of Detail 2 in Figure 11b), regardless if the test failed as a wide beam or by punching. Using the proposed recommendations for punching capacity predictions (Figure 11d), the average ratio $P_{test}/P_{R,predicted}$ changes from 1.44 to 1.23, and the coefficient of variation decreases from 40.1% to 21.3%.

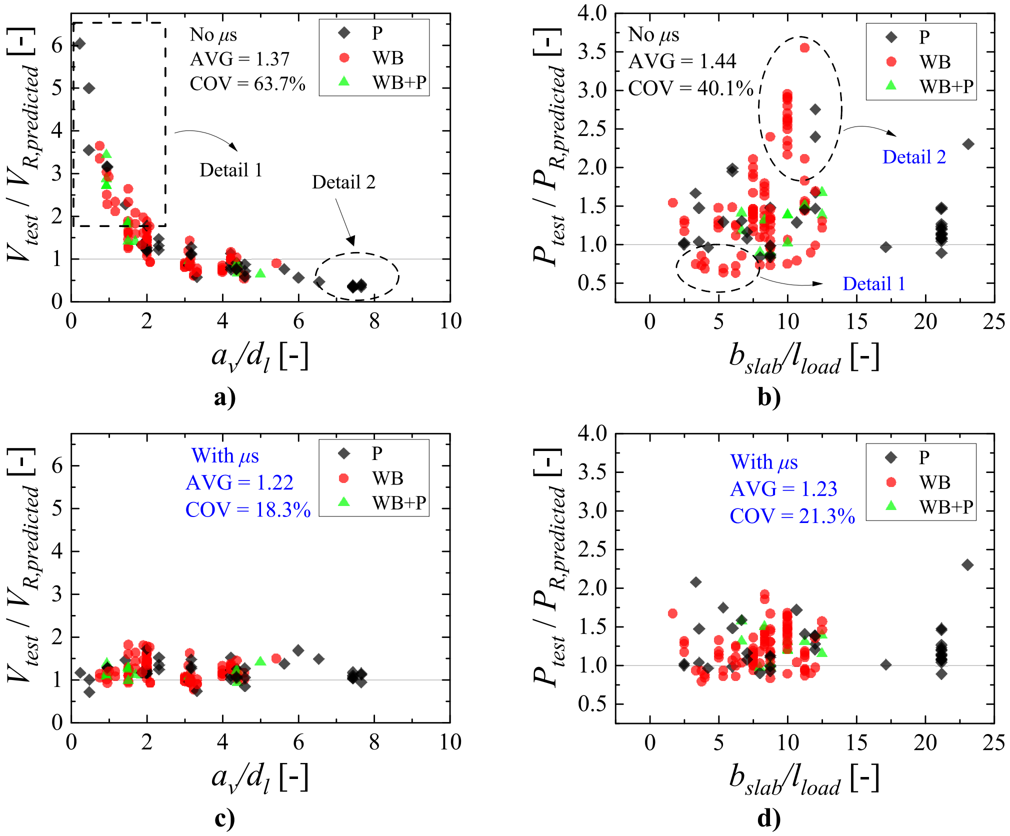

Figure 11 - Comparison between tested and predicted resistances with the reference and proposed approaches for: a) and c) one-way shear; and b) and d) punching shear expressions. The results "No μs" are the ones following the reference approach (Section 3) and the results "With μs" are reached with the proposed recommendations (Section 4). Note: P = punching failure; WB = wide beam shear failure in one-way shear; WB+P = mixed mode between one-way shear and punching.

Comparing the predictions of shear and punching capacity using the proposed recommendations, it can be observed that the predictions are quite similar. The average ratio between tested and predicted resistances (1.22 and 1.23) differs by less than 1%, and the coefficients of variation (18.3% and 21.3%) differ by less than 15%.

In this study, however, it can be observed that the predictions of one-way shear capacity performed better that those for punching (with a lower coefficient of variation) and used a fewer number of correction factors (2 for shear and 3 for punching). In practice, this occurs because of the more significant number of tests that failed as wide beams in the database but also because the one-way shear failure mode seems to represent better the local failure between the load and the support. In this study, it is assumed that the one-way shear approach represents the problem closely because it directly considers the more significant influence of the resistance and shear demand on the front side of the load close to the support. In this context, it is important to note that in the punching capacity predictions, a uniform shear resistance and shear demand around the load is assumed with the Brazilian code expressions. Consequently, it would be necessary to consider the different contributions of each side of the control perimeter (unbalanced shear resistance and unbalanced shear demand) around the load to reach better predictions with the punching expressions.

6.2 Resistance model uncertainty

The analysis of the resistance model's uncertainty partial safety factor (γ_{Rd}) concerning the shear and punching capacity using the proposed recommendations is carried out. When considering structural reliability, model uncertainties can be related to models for action effects and for resistance models, which are based on simplified relationships or complex numerical models. Therefore, the model uncertainty can be defined as a basic variable related to the accuracy of the physical model. It is commonplace to consider model uncertainty as a random variable.

In this paper, the principles presented in the *fib* Model Code 2010 [22] and other references are used [40]–[44] to estimate the resistance model uncertainty. For this analysis, the model error (ME) is defined as in Equation 29:

$$\text{ME} = \frac{\text{Experimental Capacity}}{\text{Capacity calculated from proposed model}} \tag{29}$$

Goodness of fit tests provide a statistical tool for selecting an appropriate type of probability distribution. A normal distribution is usually used to represent the lower tail of model uncertainty for resistance functions. However, the normality test, using the Shapiro-Wilk test at the 0.05 significance level, concluded that data for both analyses were not significantly drawn from a normality distributed population. Figure 12 presents the frequency histogram of shear and punching capacity using the proposed recommendations. Both the Chi-square and the Kolmogorov-Smirnov tests have confirmed at the 0.05 significance level the possibility to adopt log-normal probabilistic distributions for both analyses.

Figure 12 also presents the sample versus theoretical probability plot for the natural logarithm of ME. The horizontal axis (x) represents the expected value of the standard normal distribution, and the vertical axis (y) denotes the natural logarithm of ME. The good linear fit confirms that the log-normal probabilistic distribution is suitable for the model error. The mean and standard deviation of the model error can be obtained using the fit linear on log-normal probability paper and are presented in Table 2.

Equation 29 also includes the variability of the test procedures and the specimen geometry, so it represents more than just the accuracy of the model. The variability of the model error COV_{ME} can be estimated as in Equation 30 [45]:

$$COV_{ME} = \sqrt{COV_m^2 - COV_{test}^2 - COV_{spec}^2} \tag{30}$$

Where COV_m is the coefficient of variation of the measured and predicted strengths by the proposed recommendations obtained from statistical analysis of Figure 12, COV_{test} is the coefficient of variation of the measured test loads, and COV_{spec} is the uncertainty of specimen dimensions in the tests. The values of $COV_{test} = 0.02$ and $COV_{spec} = 0.04$, as proposed in reference [45], are used herein. The resulting model statistical parameters are presented in Table 2.

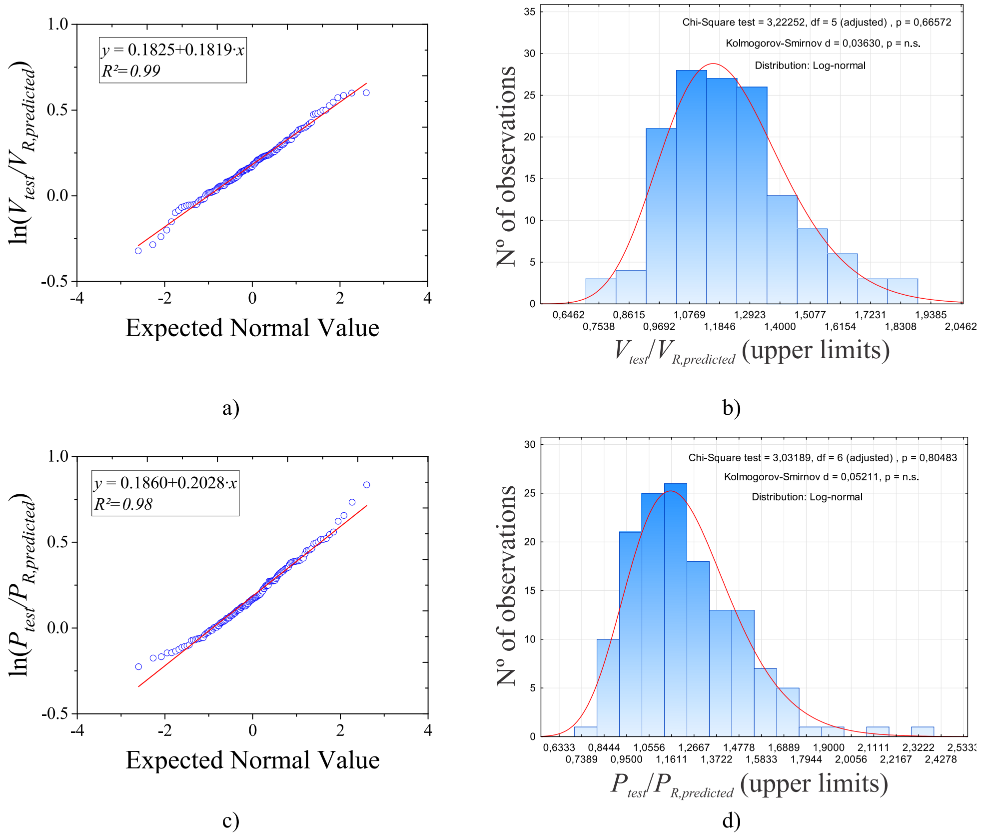

Figure 12 - Normal probability plot and frequency histogram for model error (ME).

Table 2 - Statistical parameters for model error and model uncertainty factors.

Proposed approach	Full data set		Statistical parameters				γ_{Rd}	
	Mean	COV	μ_R	σ_R	COV_m	COV_{ME}	$\alpha_R = 0.32$	$\alpha_R = 0.8$
One-way shear resistance	1.220	0.183	1.220	0.224	0.183	0.178	1.02	1.41
Punching shear resistance	1.230	0.213	1.229	0.252	0.205	0.200	1.04	1.49

Finally, under the lognormal distribution hypothesis, the resistance model uncertainties γ_{Rd} can be determined, according to references [22], [43], as follows (Equation 31):

$$\gamma_{Rd} = \frac{1}{\mu_R} \exp(\alpha_R \; \beta \; COV_{ME}) \tag{31}$$

where μ_R is the mean value of the ratio between the resistance obtained through the tests and the resistance achieved through the proposed recommendations and COV_{ME} is the coefficient of variation of the resistance model uncertainties. The first-order-reliability method (FORM) sensitivity factor for the variables (α_R) can be assumed to be 0.32 or 0.8 [41], [43], accounting for the hypothesis of non-dominant and dominant variables, respectively, and the term β denotes the reliability index. For new structural systems with moderate consequences due to a structural failure and a 50-year lifetime, *fib* Model Code 2010 [22] recommends $\beta = 3.8$ (i.e., a failure probability of 7×10^{-5}).

The value of the model uncertainty factor γ_{Rd} is dependent on the quality of the resistance model formulation. Fib Model Code 2010 [22] has recommended values of 1.06 for models with low uncertainties and 1.1 for models with high uncertainties. Table 2 shows the values of γ_{Rd} for the proposed recommendations, with resistance uncertainty as a non-dominant and dominant variable. The values for the non-dominant hypotheses are below the values recommended by the fib Model Code 2010 [22]. However, the non-dominant hypothesis of the model uncertainty factor γ_{Rd} can be adopted if the coefficient of variation (COV_{ME}) reported in Table 2 is less than 0.15, which is the coefficient of variation associated with the compressive strength of concrete according to references [22], [43]. Therefore, the dominant hypothesis should be adopted for the shear and punching recommendations proposed herein. Consequently, and as a simplification, it would be recommended to use a $\gamma_{Rd} = 1.5$ for both one-way shear and punching shear resistance predictions with the proposed approaches.

7 CONCLUSIONS

In this study, the expressions and most traditional approaches to predict the shear and punching capacity of one-way slabs under concentrated loads are evaluated, with emphasis to the ABNT NBR 6118:2014 code provisions. Based on the described analyses and proposed recommendations, the following conclusions can be drawn:

- The one-way shear and punching shear capacity enhance significantly for loads close to the support, here assumed at distances $a_v \leq 2d_l$. Using the factors $\mu_{shear,1}$ and $\mu_{punching,1}$ to consider the enhanced unitary shear and punching capacity of the slabs allows for improving the predictions of ultimate capacity when the load is placed relatively close to the support.
- For loads far away from the support, typically when $a_v/d_l > 4$, the predictions of one-way shear capacity with the Brazilian code combined with the French effective shear width model can be critically unsafe. This occurs because the French effective shear width model overestimates the contributing slabs strip to the one-way shear capacity. Using the factor $\mu_{shear,2}$ allows correcting the predicted effective shear width in a simple and effective way. Besides, this approach improves considerably the relation between tested and predicted resistances using the expressions of one-way shear resistance, even when the tests fail by punching.
- The predicted punching capacity with the Brazilian code expressions can be critically unsafe if the influence of the slab width is not considered for tests that are critical in one-way shear. This occurs because by decreasing the slab width, the shear flow concentrates on the sides of the control perimeter in the spanning direction and the sides parallel to the free edges present a smaller contribution to the punching capacity. In this study, it is proposed to apply a factor $\mu_{punching,2}$ as a function of the slab width on the sides of the control perimeter parallel to the free edges. Using this factor, the predicted punching capacity for tests critical in one-way shear are enhanced significantly (see Figure 11b-11d).
- One-way shear and punching capacity expressions can provide similar and enhanced predictions for one-way slabs under concentrated loads if parameters that influence the transition from shear to punching failure mechanisms (and vice-versa) are considered. In this study, the average ratio between tested and predicted resistances $V_{test}/V_{R,predicted}$

was 1.22 with a coefficient of variation of 18.3% using the proposed recommendations. The respective average ratio $P_{test}/P_{R,predicted}$ was 1.23 with a coefficient of variation of 21.3%.

- The uncertainty of the resistance model for shear and punching capacity were calculated using the recommended approaches. The coefficients of variation of the model error were greater than 0.15, so that the dominant hypothesis was adopted. Therefore, the resistance model uncertainties γ_{Rd} for the one-way shear and punching capacities were 1.41 and 1.49, respectively. As a general recommendation, the value of $\gamma_{Rd} = 1.5$ could be adopted for both one-way shear and punching shear.

ACKNOWLEDGEMENTS

The authors acknowledge the financial support provided by the São Paulo Research Foundation (FAPESP) (grant number #2018/21573-2, grant number #2019/20092-3 and grant number #2021/13916-0) and by Brazilian National Council for Scientific and Technological Development (CNPq) (grant number #303438/2016-9). This study was also partially financed by the Coordenação de Aperfeiçoamento de Pessoal de Nível Superior – Brasil (CAPES) – Finance Code 001.

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Editors: Leandro Trautwein, Mauro Real, Mario Pimentel.