Leviat
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Ancon

Shearfix Punching Shear Reinforcement

Design Manual to AS 3600:2018

Ancon Shearfix Design Manual: Australian Code AS 3600: 2018

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Introduction

The Ancon Shearfix software from Leviat supports engineers in the design of punching shear situations. It helps to calculate the punching strength without shear reinforcement and provides a solution with Shearfix studs if shear reinforcement is necessary. Once a design has been completed, users can directly create a parts list for the specification or a dxf file to import the solution into a CAD drawing. Additionally, the output report provides a detailed verification of the proposed solution so that the engineer has the opportunity to check and verify the calculation performed by the software.

Since punching can be a rather brittle failure and thus lead to severe damage, engineers should take sufficient care in the design of such situations. Users should not only operate the design software but also understand the calculation behind it. This document provides the software users the necessary background information to follow the design. Additionally, it highlights certain aspects of the general punching shear design and provides additional information.

This document is based on Leviat's longstanding knowledge of punching shear design and experience of manufacturing a wide range of punching shear reinforcement systems in numerous countries. Leviat has performed dozens of punching shear tests over many years meaning engineers all over the world can use this guide, and specify punching shear reinforcement, with confidence.

Punching strength without shear reinforcement

Generally, design codes define the punching strength without shear reinforcement by assuming a concrete shear strength along a control section defined by the effective depth multiplied by a control perimeter. If the punching strength is larger than the design forces, the design is verified. However, only a few design codes include the deformation capacity related to the punching of flat slabs. Punching shear failure without shear reinforcement can be rather brittle, thus failure occurs at rather small deformations without any warning signs and without any possibility of load distribution to other supports. Consequently, the robustness of the overall structure can be limited by these column-slab connections. An increase in the robustness can be achieved by using shear reinforcement since it does not only increase the punching strength but also the deformation capacity. In fact, by adding shear reinforcement, the increase in the deformation capacity is greater than the increase in the punching strength. Therefore, in certain situations or special buildings, it may be recommended to use shear reinforcement even when the punching strength without shear reinforcement is larger than the design load in order to increase the robustness of the overall structure.

Regarding the punching strength design according to AS 3600, the concrete shear strength is defined by:

$$f_{cv} = 0.17\left(1 + \frac{2}{\beta_h}\right)\sqrt{f_c'} \leq 0.34\sqrt{f_c'} \qquad \text{Eq. 1}$$

where β_h is the aspect ratio of the column in the case of rectangular columns. Thus, if the length and width differ by more than a factor of two, the shear strength is reduced accounting for the concentration of shear force at the shorter side length of the column. These two formulations are like the ones of ACI 318. However, it can be noted that ACI 318 also knows a limit in the case of large column dimensions to account for the concentration of the shear force at the column corners. Additionally, since the 2019 revision, ACI 318 includes a size effect factor to account for the less-than-proportional increase in the shear strength with increasing slab thickness which is also not accounted for in AS 3600.

The punching shear strength of a slab without moment transfer V_{u0} can be calculated by multiplying the control perimeter, the effective depth, the concrete shear strength, and -if any- the compression resulting from prestressing.

$$V_{u0} = ud_{0m}\left(f_{cv} + 0.3\sigma_{cp}\right) \quad \text{Eq. 2}$$

The definitions regarding the control perimeter and additional information regarding the prestressing is given in subsequent chapters.

Unless special connections are designed, the support generally creates a certain rotational fixation, which results in flexural reactions within the column leading to a non-uniform distribution of the shear force along the control perimeter. AS 3600 accounts for this by adding the torsional shear stresses resulting from the moments in the torsion strips to the shear stresses resulting from the shear forces. In AS 3600, the punching shear strength V_u, which is calculated in each direction, is defined as:

$$V_{u,xz} = V_{u0}/\left[1.0 + uM_{v,x}^*/\left(8V^* a_y d_{0m}\right)\right] \quad \text{Eq. 3a}$$

$$V_{u,yz} = V_{u0}/\left[1.0 + uM_{v,y}^*/(8V^* a_x d_{0m})\right] \quad \text{Eq. 3b}$$

The design safety is fulfilled if the punching shear strength multiplied by the capacity reduction factor ϕ (ϕ = 0.7) is larger than the design load V*.

$$V^* \leq \phi V_{u,xz} \quad \text{Eq. 4a}$$

$$V^* \leq \phi V_{u,yz} \quad \text{Eq. 4b}$$

Punching strength with shear reinforcement

If shear reinforcement is present, one can generally distinguish between three punching failure modes for the design of flat slabs:

- Failure of the compression strut next to the column
- Failure within the shear-reinforced area
- Failure outside the shear-reinforced area

The latter two failures can be avoided by installing more shear reinforcement in terms of cross-sectional area within the shear critical area and by increasing the shear-reinforced area, respectively. The failure of the compression strut however is not influenced by the amount of shear reinforcement but rather by its location -accounted for by the detailing rules in design codes- and the anchorage performance of the shear reinforcement as well as the geometrical boundary conditions such as the column size and the slab thickness. Therefore, the compression failure is often called the maximum punching strength since it is an upper limit for a certain design situation.

The punching strength against failure of the compression strut in x-direction is given by

$$V_{u\ max,x} = 3 \cdot V_{u\ min,x} \cdot \sqrt{x/y} \quad \text{Eq. 5}$$

where

$$V_{u,min,x} = 1.2V_{u0}/\left[1.0 + uM^*_{v,x}/\left(2V^* a_y^2\right)\right] \quad \text{Eq. 6}$$

and

$$x = min\left(a_y; h\right) \quad \text{Eq. 7a}$$

$$y = max\left(a_y; h\right) \quad \text{Eq. 7b}$$

The punching strength against failure of the compression strut in y-direction is given by

$$V_{u\ max,y} = 3 \cdot V_{u\ min,y} \cdot \sqrt{x/y} \quad \text{Eq. 8}$$

where

$$V_{u,min,y} = 1.2V_{u0}/\left[1.0 + uM^*_{v,y}/(2V^* a_x^2)\right] \quad \text{Eq. 9}$$

and

$$x = min(a_x; h) \quad \text{Eq. 10a}$$

$$y = max(a_x; h) \quad \text{Eq. 10b}$$

The design safety is fulfilled if the punching shear strength multiplied by the capacity reduction factor ϕ (ϕ = 0.7) is larger than the design load V*.

$$V^* \leq \phi V_{u,max,x} \quad \text{Eq. 11a}$$

$$V^* \leq \phi V_{u,max,y} \quad \text{Eq. 11b}$$

It can be noted that for the calculation of the maximum punching strength, the method to multiply the punching strength related to the situation without punching shear reinforcement by a certain design factor is used in several design codes. However, the increasing factor used in AS 3600 can lead in certain cases to rather large maximum punching strength values compared to other standards such as Eurocode or ACI 318. Thus, the user should use caution and may consider applying a self-imposed limit.

Regarding the failure within the shear-reinforced area, AS 3600 only provides formulations for closed stirrups - not for shear studs. Compared to stirrups, shear studs can only resist shear forces and thus do not provide additional torsional strength. Thus, in case of stud reinforcement, the effects of shear stresses resulting from the shear force and shear stresses resulting from the torsion need to be treated separately. Thus, the influence of the shear studs is only considered by the factor k within the first (shear) term in the denominator and does not influence the second (torsional) term. This approach was proposed by Lim and Rangan. However, compared to the formulation presented in the work from Lim and Rangan, one value has been changed from 1.0 to 0.75 ("0.75+k" instead of "1+k"; refer to subsequent discussion) leading to the following expressions:

$$V_{u,xz} = \frac{V_{u0}}{\left[\frac{1}{0.75 + k_x} + uM^*_{v,x}/\left(8V^* a_y d_{0m}\right)\right]} \qquad \text{Eq. 12a}$$

$$V_{u,yz} = \frac{V_{u0}}{\left[\frac{1}{0.75 + k_y} + uM^*_{v,y}/(8V^* a_x d_{0m})\right]} \qquad \text{Eq. 12b}$$

where

$$k_x = \frac{1}{V_{u0}} A_{v,x} \cdot f_{vy} \cdot \frac{d}{s} \cdot \frac{u}{a_y} \geq 0.25 \qquad \text{Eq. 13a}$$

$$k_y = \frac{1}{V_{u0}} A_{v,y} \cdot f_{vy} \cdot \frac{d}{s} \cdot \frac{u}{a_x} \geq 0.25 \qquad \text{Eq. 13b}$$

$A_{v,x}$ and $A_{v,y}$ are equal to the cross-sectional area of a row of shear reinforcement in the torsion strip under consideration.

For a further discussion, if one looks only at the effect of the shear stresses due to the shear force and neglects the torsional term, the equation would result in the following expression, which is basically the summation of the punching shear strength of the concrete and the shear reinforcement.

$$V_u = V_{u0}(0.75 + k) = 0.75 V_{u0} + A_{v,i} \cdot f_{vy} \cdot \frac{d}{s} \cdot \frac{u}{a_j} \qquad \text{Eq. 14}$$

The approach of the summation of a concrete contribution and a shear reinforcement contribution is used in several design codes and therefore a well-accepted approach. Lim and Rangan proposed to use the full punching shear strength of the concrete (V_{u0}) and the full shear reinforcement contribution by using the yield strength. However, if shear reinforcement is used, higher load levels are achieved leading to larger slab deformations and thus to larger openings of shear cracks within the slab, which results in a decline of the concrete shear strength (refer to commentary in ACI 318). Thus, the ACI 318 uses only three quarters of the concrete shear strength if double headed studs are used as shear reinforcement. Based on this, the original formulation of Lim and Rangan has been changed and the contribution of the concrete was set to 0.75 V_{u0}. This change leads to the change of the expression from (1+k) to (0.75+k).

The design safety is fulfilled if the punching shear strength multiplied by the capacity reduction factor ϕ (ϕ = 0.7) is larger than the design load V*.

$$V^* \leq \phi V_{u,xz} \quad \text{Eq. 15a}$$

$$V^* \leq \phi V_{u,yz} \quad \text{Eq. 15b}$$

Regarding the punching strength outside the shear-reinforced area, AS 3600 does not provide a specific verification method. It provides a detailing rule stating that the fitments should extend for a distance not less than $L_t/4$ from the face of the support or concentrated load, where L_t is the width of the design strip. While the width of the design strip is easily defined where slabs are regularly supported -i.e. orthogonal arrangement of columns with equal spacing-, it can be rather difficult to define where slabs are unregularly supported. Consequently, this detailing rule is not considered within the Shearfix software. Instead, the Shearfix software applies the design rules from ACI 318. In this way the shear force along an outer perimeter is verified. (Refer to chapter “Control perimeter outside of the shear reinforcement”).

Compared to the verification of the punching shear strength without shear reinforcement, a reduced concrete shear strength is taken for the verification of the punching shear strength outside the shear-reinforced area leading to the following expression:

$$V_{u0,out} = u_{out} \cdot d_{0m} \cdot \left(0.17 \cdot \sqrt{f_c} + 0.3\sigma_{cp}\right) \quad \text{Eq. 16}$$

Although, from a theoretical point of view, the influence of the torsional moments at the outer control perimeter is less than at the column face, it is treated in the same way leading to the following expressions for the outer punching shear strength:

$$V_{u,out,xz} = V_{u0,out}/\left[1.0 + uM_{v,x}^*/\left(8V^* a_y d_{0m}\right)\right] \quad \text{Eq. 17a}$$

$$V_{u,out,,yz} = V_{u0,out}/\left[1.0 + uM_{v,y}^*/(8V^* a_x d_{0m})\right] \quad \text{Eq. 17b}$$

The design safety is fulfilled if the punching shear strength multiplied by the capacity reduction factor ϕ (ϕ = 0.7) is larger than the design load V*.

$$V^* \leq \phi V_{u,out,xz} \quad \text{Eq. 18a}$$

$$V^* \leq \phi V_{u,out,yz} \quad \text{Eq. 18b}$$

Control perimeter at the column

According to AS 3600, the perimeter is defined "by a line geometrically similar to the boundary of the effective area". Figure 1 shows this control perimeter. The control perimeter shown is valid for interior columns without nearby openings. If openings or slab edges are present, the control perimeter must be adjusted accordingly.

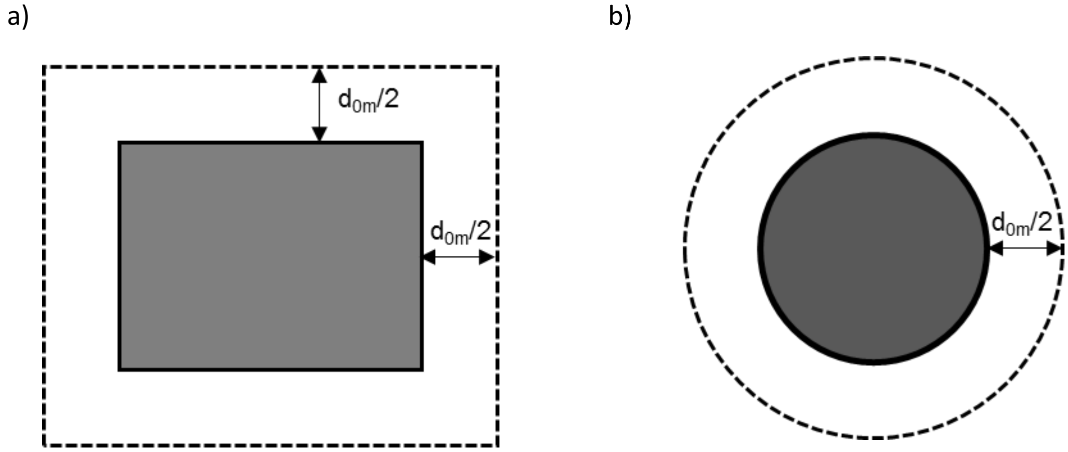

Figure 1: Control perimeter at the column for interior columns without openings

Openings must be considered in the design if they are closer than a distance of $2.5b_0$ from the control perimeter, where b_0 is the projected dimension of an opening. In such cases, the control perimeter is reduced by an ineffective section defined by two tangential lines starting at the center of the column (refer to figure 2).

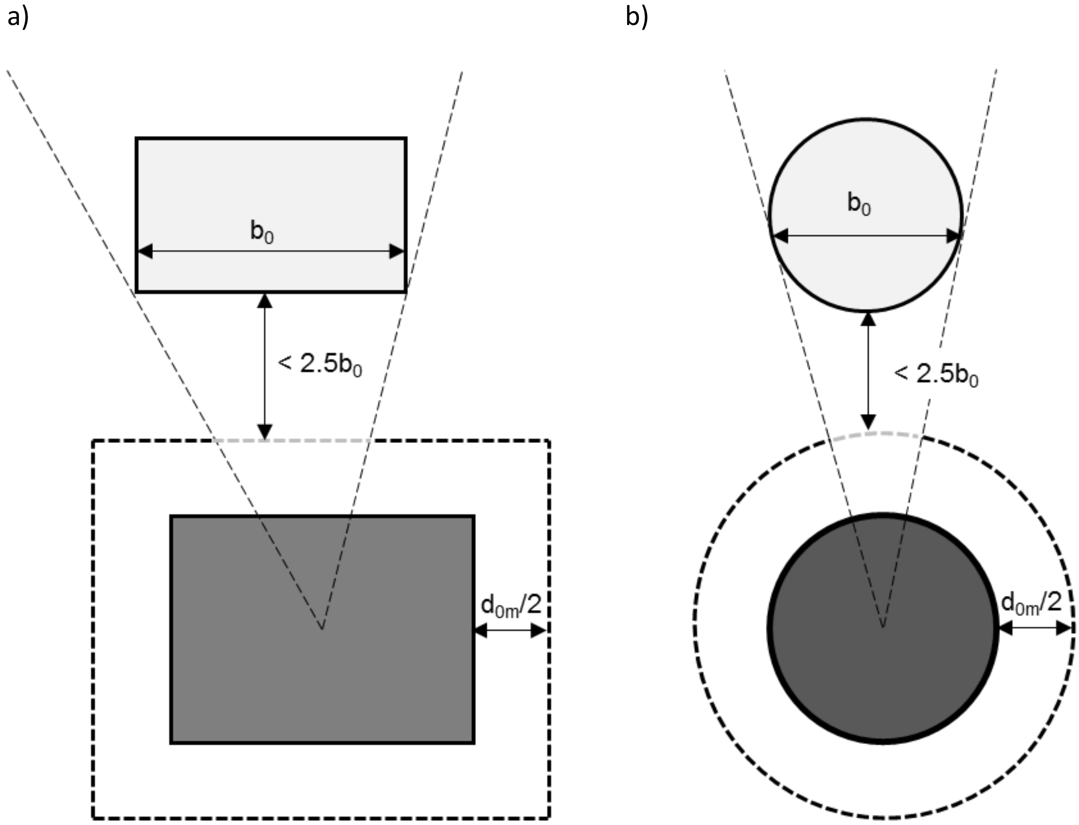

Figure 2: Control perimeter at the column for interior columns with openings

It has to be noted that the Shearfix software considers all the openings if they are defined by the user even if they are further away than $2.5b_0$ from the control perimeter. The reason for it is that they may not have to be considered for the verification at the column face, but they may affect the verification of the shear reinforcement or the outer control perimeter. The software will however display an information message if an opening is further than $2.5b_0$ from the control perimeter so that users can decide for themselves if the opening should be kept or deleted.

In the case of a column close to a slab edge, i.e. edge column, corner column, or re-entrant corner, the control perimeter is taken as shown in Figure 1, 3 or 4 – whichever gives the smallest perimeter.

a) b)

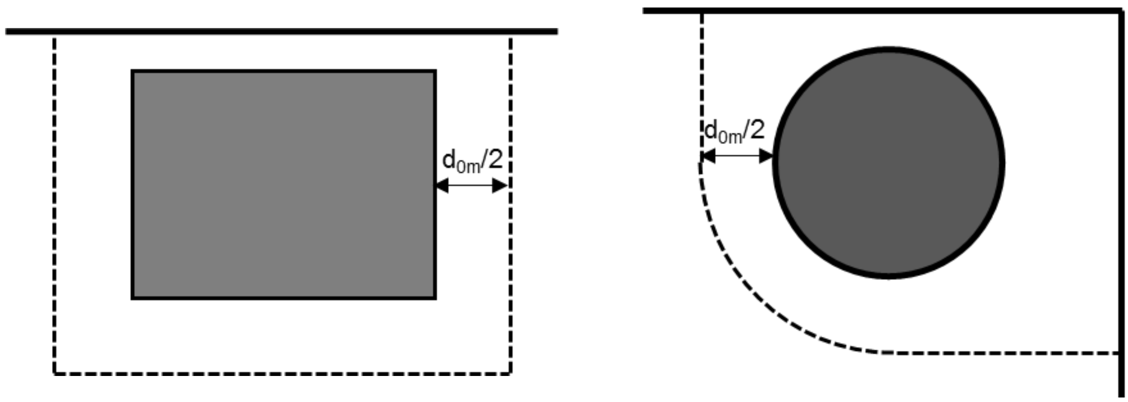

Figure 3: Control perimeter at the column for (a) an edge or (b) a corner column

In the case of a re-entrant corner, several possible control perimeters can be assumed. The Shearfix software distinguishes three possibilities: either the control perimeter runs perpendicular to the slab edge (figure 4a), it connects to the corner of the two slab edges (figure 4b), or it uses the continuous control perimeter (figure 1). Thus, depending on the location, shape, and orientation of the column, multiple different control perimeters can be defined. The Shearfix software always uses the smallest one that can be found.

a) b)

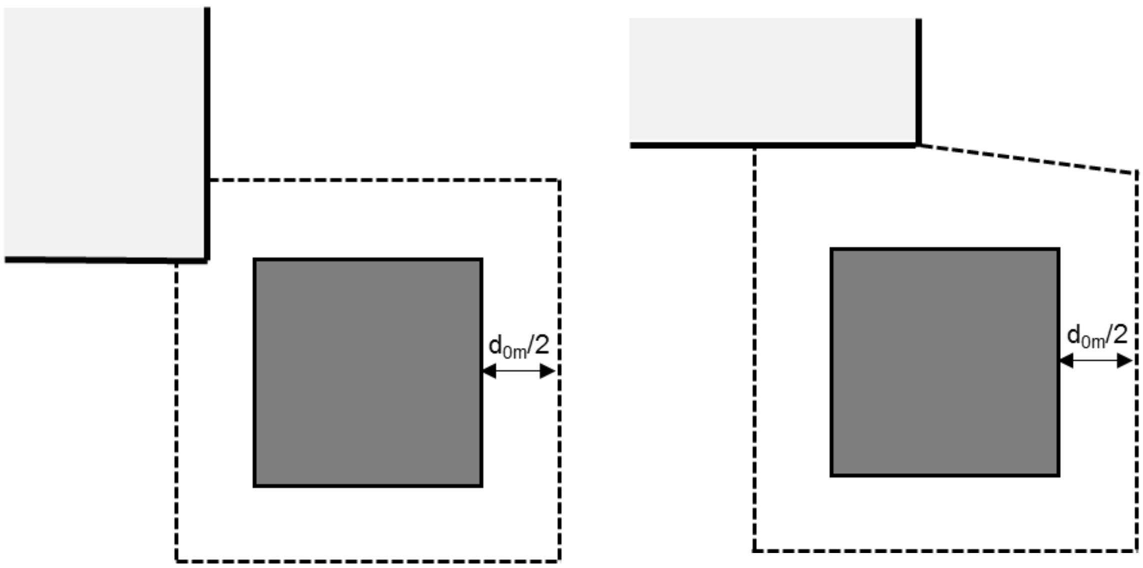

Figure 4: Control perimeter in the case of a re-entrant corner

Regarding the control perimeter in the case of openings and columns close to slab edges, one has to note that an inconsistency exists in the definition of the control perimeter. For example, if a large opening is defined, the control perimeter might be smaller than if the situation is designed as an edge column. For these cases, ACI 318 – for example – recommends the use of the edge column control perimeter. However, even this recommendation does not rule out all the inconsistencies. Thus, it is up to the user, to carefully choose the correct and safe design situation.

Control perimeter outside of the shear reinforcement

The control perimeter for the verification outside the shear-reinforced area is located at a distance $d_{0m}/2$ beyond the outermost shear stud of each rail (refer to figure 5). Openings and slab edges are similarly considered for the control perimeter outside of the shear-reinforced area as in the case of the control perimeter at the column (refer to figure 2, 3, and 4).

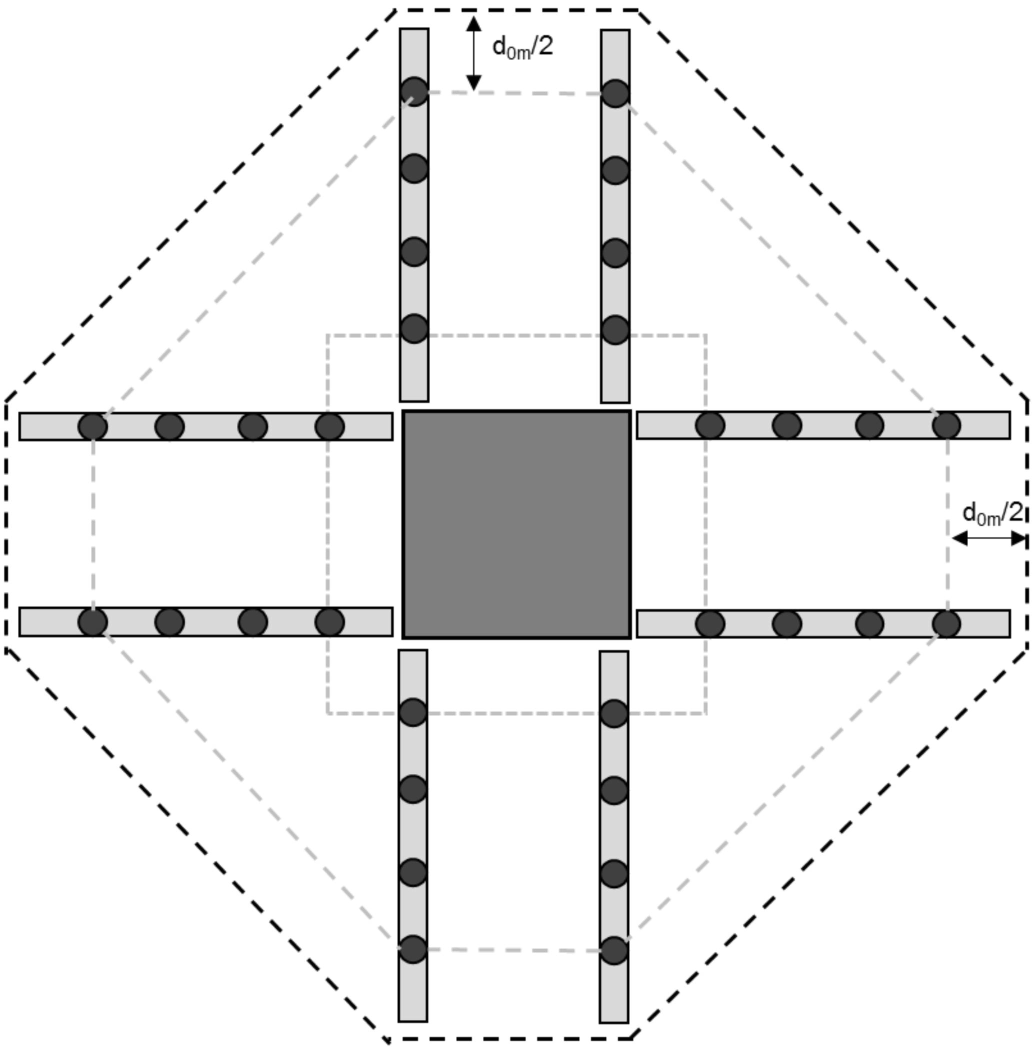

Figure 5: Control perimeter outside of the shear reinforcement

Moments in the torsion strips

The design model for punching in AS 3600 considers that the rotational stiffness of the column leads to moments in the torsion strips. The shear stresses resulting from these torsional moments are added to the shear stresses due to the shear forces within the torsional strip. It has to be noted that the Moment M* considered in the design formulation references to the central axis of the torsion strip. For the design however, it is generally easier to work with the reaction moments of the column since they are easily obtained by any finite-element software. In cases, by which the center line of the torsion strip passes through the center of gravity of the cross section of the column, the moment in the torsion strip corresponds to the reaction moment. However, in case of edge and corner columns, the center line of the torsion strip does not pass through the center of gravity of the cross section of the column (refer to figure 6). Thus, the offset needs to be considered in the design. The Shearfix software automatically considers this so that the user can also in these cases input simply the reaction moments of the columns without further calculations.

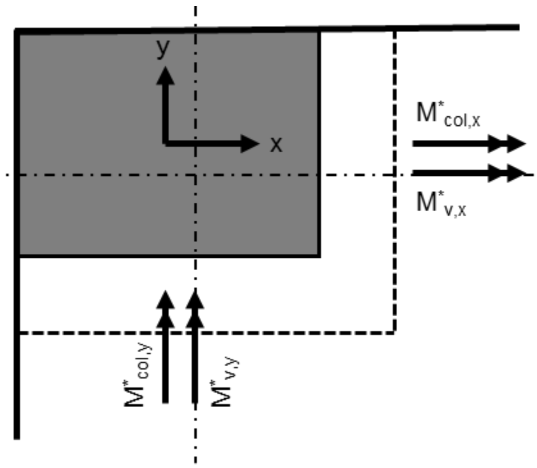

Figure 6: Moments in torsion strips at a corner column

Prestressing

Prestressing has a beneficial effect on the punching strength. The compressive stresses due to prestressing reduce the cracking of the concrete and thus increase the concrete shear strength. However, it is important to note that one should be rather careful when considering any compression from prestressing in the punching shear design. Besides the obvious factors such as creep, shrinkage, and relaxation which reduce the compression stresses, one should be sure that the compression forces are acting in the vicinity of the column. The compression forces are applied at each end of the anchored prestressing tendons. If rigid structures such as shear walls are present, the compression forces may diminish within the slab and there will be no effect in the vicinity of the column. Thus, no beneficial effect will occur in terms of the punching strength. A certain precaution of the consideration of this beneficial effect is thus recommended. Additionally, ACI 318 limits the maximum compression stress that can be considered in the design. AS 3600 uses a similar approach to consider the compression stresses but does not set any limit. Since this limit seems to be reasonable because of the lack of experimental data above this limit, the Shearfix software will give an information message if the average compression stress due to prestressing exceeds the limit set by ACI 318 (3.45 MPa).

Spacing and detailing of the shear reinforcement

The Shearfix software considers the following detailing rules:

- The distance to the first stud is predefined as $0.5d_{0m}$ but a value between $0.35 – 0.5\ d_{0m}$ can be chosen by the user. However, a reduction of the distance does not have any beneficial effect in the design according to AS 3600.
- The distance between the studs within one rail is the minimum of 75% of the slab thickness, 70% of the stud height and 500 mm.
- The distance between individual rails is limited to the minimum of the slab thickness and 600 mm.
- On each side of the column, a minimum of two rails are placed.

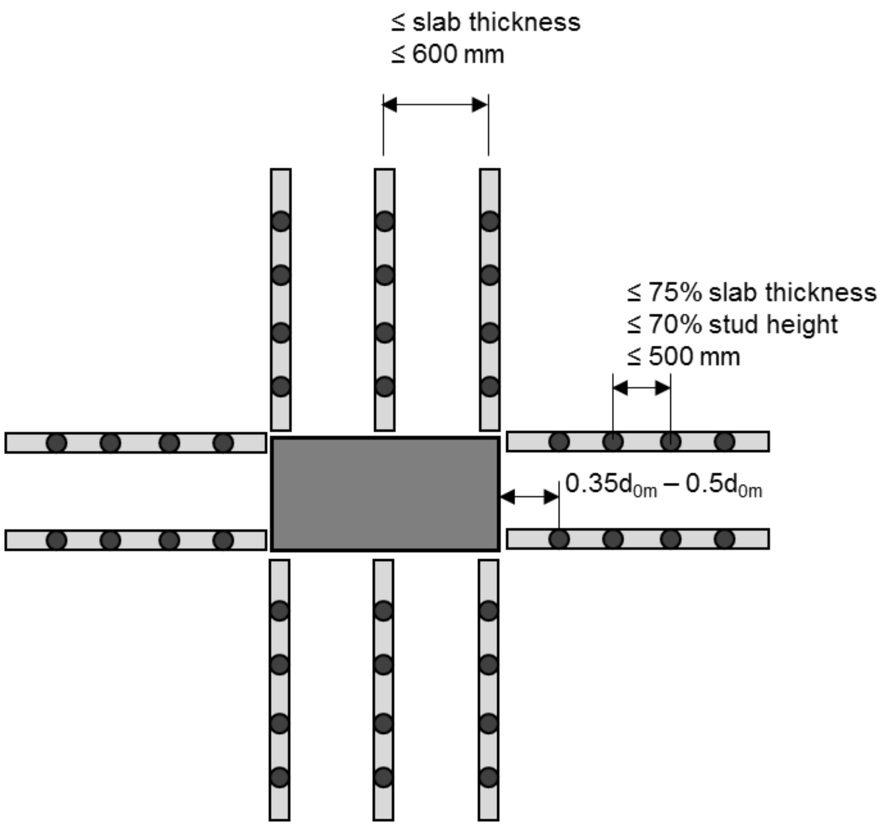

Figure 7: Detailing rules of Shearfix studs

It can be noted that these detailing rules vary from the ones of AS 3600, which basically cover stirrup shear reinforcement, and the ones from previous practice. These newly implemented, more stringent, detailing rules for stud shear reinforcement consider the newest developments in punching shear research and the developments in other design codes leading to an up-to-date design.

References

ACI 318-19, Building Code Requirements for Structural Concrete, American Concrete Institute ACI, 628 pp. 2019

AS 3600, Australian Standard Concrete Structures, Standards Australia, 269 pp., 2018

Lim, F. K. and Rangan, B. V., Strength of Concrete Slabs with Stud Shear Reinforcement in the vicinity of edge columns, Australian Civil Engineering Transactions, Vol CE35, No. 2, pp. 95-105, 1993