Punching Shear Design to AS5100.5:2017 Amendment 2

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Abstract: The amended clause 9.2 in AS5100.5 Bridge Design – Concrete is a significant departure from the previous clause and presents a different approach to the shear design and reinforcement detailing of two-way concrete slabs. The revised method draws on the content of current international standards. The revised method is considered an interim method and will be replaced when a strain-based design method for punching shear is agreed by the international concrete structures design community. This paper explains background to the Draft clause 9.2 in AS5100.5 Amendment 2 and provides additional information to guide users in the adoption of this revised method for punching shear design.

Keywords: AS5100.5, Punching Shear, Slabs, Critical Shear Perimeter, Shear Reinforcement

1. Introduction

The reasons for the proposed change in approach to punching shear design and appreciation of ongoing research into punching shear is documented in a separate paper [Ref 4] presented at CIA 2022 conference and at a CIA webinar in 2020. Some previous code clauses could be mis-interpreted and in some circumstances lead to non-conservative outcomes. Previous code rules for punching shear referred designers to both AS5100.5 and AS3600, which was confusing. The previous rules were not suited to large thick two-way slabs encountered in the transport sector. Amended shear reinforcement detailing rules were needed to better respond to punching shear stress distributions and improve constructability.

The authors found that design teams designing large thick two-way slabs in the transport sector often reverted to either ACI 318 or BS EN 1992 for guidance on shear design. Unlike building slabs, these thick slabs often include shear reinforcement. The scope of AS5100 is wider than many international bridge design codes and is adopted for the design of below ground structures in the transport sector.

This paper provides background and presents thoughts on the application of the Amendment 2 clauses for slab shear that may assist designers in their decisions in the application of the clauses. It recognizes Public Comment received in June 2023. Thoughts presented are the authors suggestions and have not been peer reviewed or more widely assessed.

2. General Overview of Amended AS5100.5 Clause 9.2 Punching Shear

The Draft for Public Comment Amendment 2 approach is based on the ACI 318M-19 [Ref 5] method modified by consideration of AS3600-2018 [Ref 2], supplement to AS3600 [Ref 3], CSA-S6;19 [Ref 7], BS EN 1992-1-1 incl 2014 amendment [Ref 8], Sept 2021 draft of BS EN 1992-1-1 2022 [Ref 9], AASHTO LRFD, BS EN 1992-2 2005 and fib Bulletin 81 [Ref 6]. The drafting and approval of the Amendment 2 clause follows an approved process. Reference standards are updated and revised. Consequently, Amendment 2 text is not entirely based on consideration of the latest version of each reference standard.

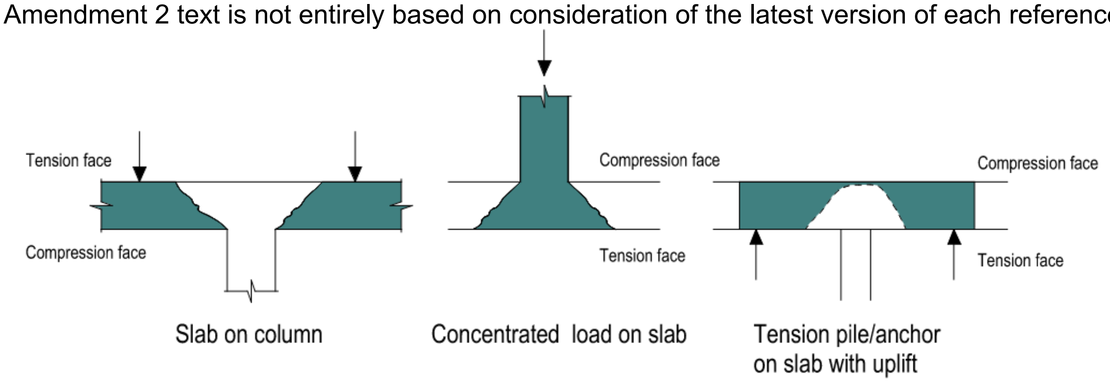

Figure 1. Selected forms of punching shear failure in slabs

The Amendment 2 punching shear method is based on the truss analogy as used in beam shear before the introduction of the current strain based or MCFT approach.

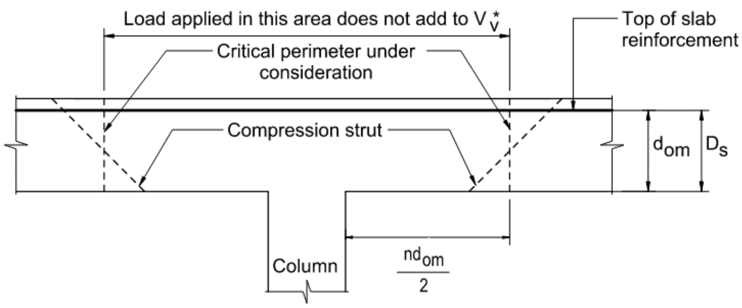

Figure 2. Example punching shear at internal column

The punching shear failure plane follows the compression strut with the principle tensile stress square to the compression strut.

The Draft Amendment 2 clauses assume that applied slab loading is reasonably uniform; shear reinforcement comprises vertical ligatures; the angle of the compression strut θ_v = 45 degrees; and concrete f'_c is less than 65 MPa.

The shear clauses rely on the hog flexural reinforcement meeting minimum flexure reinforcement and integrity reinforcement requirements – all as required by flexure design. Control flexural cracking at the punching shear area. Flexure cracking may be detrimental to the two-way shear capacity. The two effects can be interdependent. Punching shear and flexural reinforcement design and detailing can be important at the ends of supporting walls particularly if the wall or barrette terminates internally below a slab.

The slab shear design clauses rely on the adoption of appropriate measures to manage slab cracking from early thermal, shrinkage, and long-term thermal effects or other sources of applied in plane strain. The design approach includes provision for the influence of in-plane loads or slab prestress. Commonly in plane compression will assist the slab shear performance while in plane tensions will be detrimental.

3. Discussion and Guidance on Amended Clauses

3.1 AS5100.5 Clause 9.2.1 General Requirements for Slab Shear

This clause is largely unchanged from the existing clause in AS5100.5-2017 incl Amendment 1. Punching shear clauses generally use capacity and load effects per unit length of shear perimeters. Shear is considered in each orthogonal direction separately.

The more critical of one-way beam shear and punching shear shall be considered. Judgement may be necessary when considering the non-dominant shear form. It may not be necessary to apply minimum beam shear rules as well as minimum punching shear rules to the entire slab, especially if the non-dominant load effects are well below the slab capacity.

The designer is guided to the use of strut and tie models when appropriate. Strut and tie models may result in more efficient outcomes for deep elements such as pile caps.

*3.2 AS5100.5 Clause 9.2.2 Design Punching Shear V**

3.2.1 Design Shear on Critical Perimeter (Clause 9.2.2.1)

V*/u is calculated on two orthogonal axes on each critical perimeter. The largest value then commonly adopted for the entire length of that critical perimeter. Consistent with the approach for square critical perimeters, adopt the average load per meter over an arc of 90 degrees for circular critical perimeters or a square critical perimeter of area equal to that of the circular critical perimeter. Successive critical perimeters are assessed.

The combination of the shear force transferred by the column and the moment transferred from slab to column is based on the ACI 318 method. ACI uses shear stresses on a perimeter rather than force/m as adopted in Amendment 2. BS EN 1992-1-1uses the β_e term to accommodate moment transfer into the column and for the influence of the column dimensions. See figures 3 and 4 for (V*/u) at rectangular columns to AS5100.5. Refer Figure 3 for the equation for J for a rectangular and circular critical perimeter symmetrical about the column and moment about axis C-C. Designers may calculate the force distribution on irregular shaped critical perimeters by modelling the shape as a thin-walled section in suitable structural software for member section analysis. Apply a moment about the relevant axis. The stress distribution output from the software is multiplied by the modelled thin wall thickness to provide the force per unit length of perimeter.

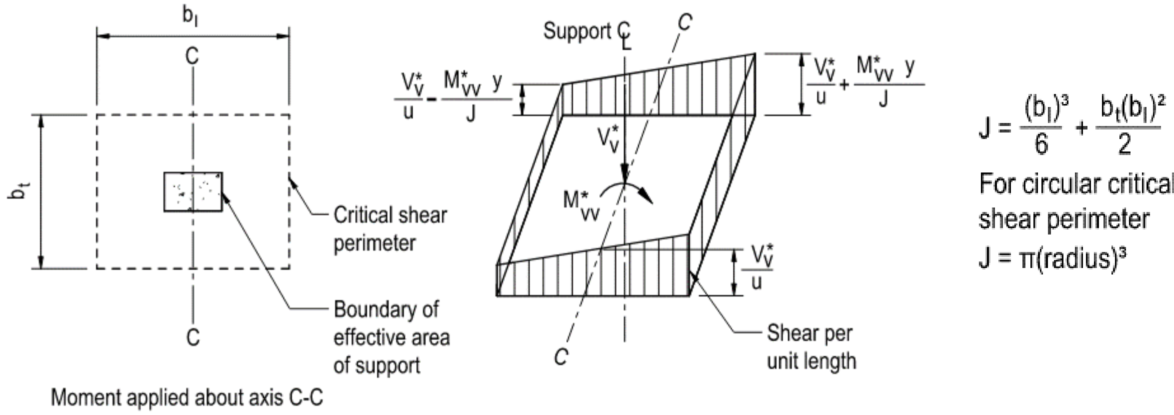

Figure 3. Example shear per meter at rectangular internal column

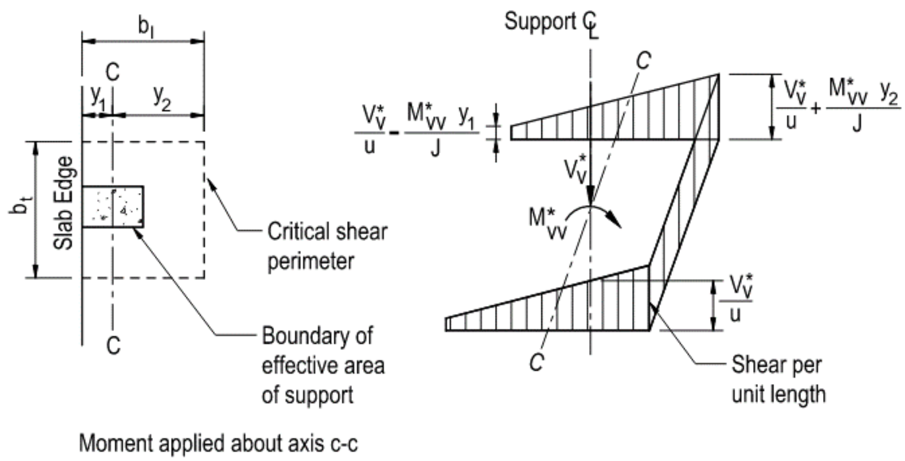

Figure 4. Example shear per meter at rectangular edge column

The note in clause 9.2.3.3 explains that it is acceptable to use different values of V*/u for each side of the column. This added complexity may be justified when considering non-symmetrical columns or non-symmetrical loading. V*/u is a uniform force per metre on one side of the critical perimeter. It is not the intention that a peak or corner load/m be used for the design. A sensible approach is necessary for complex column and critical perimeter shapes.

For symmetrical rectangular critical perimeters with sides b_l and b_t where b_l is in the direction of moment being considered, M^*_{vv} may be taken as $M^*_v(1 - 1/(1+0.67\sqrt{(b_l/b_t)}))$. Designers may consider reducing M^*_{vv} to a minimum of zero at corner and edge columns for moments perpendicular to the slab edge provided: $V^*_v/(ud_{om}) \leq \phi(0.5f_{cv})$ and M^*_f is increased accordingly. ACI 318 clause 8.4.2.2 provides guidance on the assessment of M^*_{vv} for edge columns or unusual column shapes. Consider potential increased shear stresses near the ends of elongated supports if the ratio of support sides is greater than six.

3.2.2 Minimum Flexure and Torsion at Supports (Clause 9.2.2.2)

The minimum design bending moment within the central strip of width ($3D_s$ + column width) ensures the design is consistent with clause 9.2.2.1 assumptions. This is a minimum moment and is not added to the moment from flexural design. If the flexural design moment in this central strip in the orthogonal direction under consideration is larger than M^*_f, then design the flexural reinforcement in the central strip for the flexural moment. This requirement is not intended to manage flexural cracking. Flexural cracking relates to the full moment in the slab and not a portion of the moment transferred to the column.

Torsion effects near the free edges of slabs, especially slabs without spandrel beams, can lead to poor slab performance. Provide torsion ligatures at these slab edges as appropriate.

3.2.3 Critical Shear Perimeter (Clause 9.2.2.3)

Critical shear perimeters are generally assessed at spacing of $d_{om}/2$ and at discontinuities such as drop panel edges. (See also figures 5, 8, 9, and 10). This spacing is consistent with ACI 318, AS3600 and AS5100. BS EN 1992-1-1 has the first perimeter at 2D from support however the draft BS EN 1992-1-1 reverts to D/2 critical perimeter spacing.

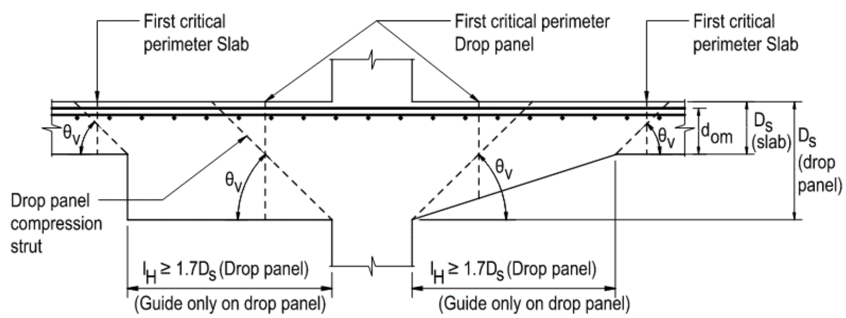

Figure 5. First critical perimeters at and adjacent to drop panels

3.3 AS5100.5 Clause 9.2.3 Punching Shear Strength

3.3.1 General Ultimate Shear Strength (Clause 9.2.3.1)

Equation 9.2.3(1) allows for the reduced concrete shear strength in slabs with shear reinforcement when the concrete is cracked. Both ACI 318 and BS EN 1992-1-1allow for this effect but in different ways. Amendment 2 follows the draft BS EN 1992-1-1 method (refer equations 8.104 and 8.105). The slab shear strength decreases in a steady rather than step function related to the ratio of V* to ϕV_{uc}. The approach does not create an iterative design process because ϕV_{uc} will be known when designing shear reinforcement. The approach in ACI 318 table 22.6.6.1 does not integrate into the proposed Amendment 2 method easily and would result in steps in capacity as reinforcement is added. The limit to k_{vuc} of 0.5 is taken from ACI. The reduction factor k_{vuc} only applies to slabs with shear reinforcement because the term is always 1.0 or larger for an unreinforced slab as ϕV_{uc} will be greater than V*. The implication of concrete cracking in slabs without shear reinforcement are accounted for in equation 9.2.3 (4) and clause 9.2.3.4.

The ϕ is included in punching shear strength design meaning that $V^*/u \leq \phi V_u /u$. Refer clause 9.2.1.

Equation 9.2.3 (2) defines an upper limit on V_u that may often control design. It derives from ACI 318 Table 22.6.6.3. BS EN 1992-1-1 has a factor k_{max} in Equation 6.52 that has a similar effect in that $(V_{uc}+V_{us}) \leq 1.5V_{uc}$. ACI 318 increases the 0.5 factor to 0.66 for headed bar shear reinforcement. The draft BS EN 1992-1-1 Equation 8.89 suggests this factor can be increased for most column geometries and standard fitments. This stress limit relates to concrete cracking at fitments. Consequently, headed bars have a higher limit in ACI 318. Equation 9.2.3 (2) includes $\sqrt{f'_c}$, being related to flexural strength and not compressive strength. The constant k_{vuc} is introduced into the equation to allow designers to alter this constant with approval in response to increased understanding of this constant, such as implied in the draft BS EN 1992-1-1 standard, or in response to recognition of headed shear fitments in AS5100.5. This stress limit applies to slabs with shear reinforcement.

Equation 9.2.3 (3) defines the limit on V_u from compression strut failure. This limit is unlikely to be critical due to the influence of Equation 9.2.3 (2). Design checks are required at the face of support or load and at significant changes in section such as at drop panel perimeters. Note the use of d_v and d_{om} in different clauses. The equation is based on AS3600 Equation 8.2.3.3 (1) with θ_v = 45 degree for the strut inclination and vertical reinforcement. $D_v = 0.9d_{om}$. Equation 9.2.3 (3) includes f'_c to reflect the potential compression failure mode, not its square root as would be applicable to tension failure.

3.3.2 Concrete Contribution to Shear Strength (Clause 9.2.3.2)

The 0.17 factor in Equation 9.2.3 (5) for f_{cv} is taken from AS3600. The 8MPa limit relates to upper bound f'_c of 65MPa. Constant k_{vuc} is the second factor for shear strength reduction in deep/thick sections. Contribution for average prestress σ_{cp} in equation 9.2.3(4) is evaluated for effective values relevant to each critical shear perimeter under consideration and assessed separately for each orthogonal direction.

Equations 9.2.3 (4) and 9.2.3 (5) are consistent with ACI 318 but with adjustments to allow for slab depth effects consistent with AS3600 and CSA S6. Constant k_{vuc} is taken from the second term in Equation 8.2.4.2 (2) in AS3600 which is a size effect factor. There is no easy method to calculate the strain for two-way slabs, so this component is based on a strain value of 0.00106 (refer AS3600 commentary), N500 reinforcement and f'_c less than 65MPa. Constant k_{vucs} applies to members without shear reinforcement (Figure 6) and k_{vucs} is set at 1.0 when at least minimum shear reinforcement is provided.

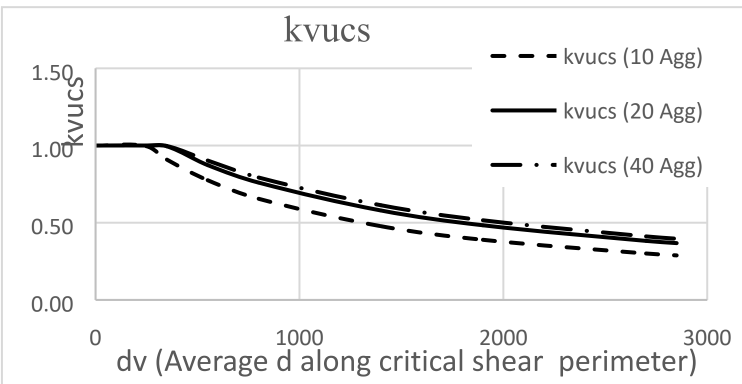

Figure 6. Constant k_{vucs} with no shear reinforcement.

Constant k_{vucs} is multiplied with k_{vs} in clause 9.2.3.4 so effectively there are two factors for size effect multiplied together. ACI 318 (Equation 22.5.5.1.3) and BS EN 1992-1-1only have a single size factor. The AS3600 commentary explains the reason for the two factors with reference to CSA S6 however the CSA S6 commentary suggests the two size factors should be compared not combined. The Canadian Building Code CSA A23.3:19 does not include two size effect factors.

3.3.3 Shear Reinforcement Contribution to Shear Strength (Clauses 9.2.3.3)

Equation 9.2.3(6) is based on the traditional truss theory for shear reinforcement. Fitment legs are vertical because it would be uncommon to use inclined fitments on a perimeter within a slab as the geometry may get messy. Inclined fitment legs could be adopted with appropriate adjustment to the equation.

3.3.4 Minimum shear reinforcement (clause 9.2.3.4)

The term $k_{vs}\phi V_{uc}$ is taken from AS3600 and would be calculated on a per meter basis in two-way slab shear calculations. The value of k_{vs} varies between 1.0 for $D_s \leq 300$ mm to 0.5 for $D_s \geq 650$ mm.

The calculation of V_{uc} in Equation 9.2.3 (4) and (5) incorporates the factor k_{vucs} which accounts for slab thickness. Clause 9.2.3.4 introduces k_{vs} which also accounts for the slab thickness. These two factors are multiplied together to determine the requirement for minimum shear reinforcement, or conversely the shear capacity of a slab without shear reinforcement. Refer also discussion in section 3.3.2.

The constant in the calculation of the area of minimum shear reinforcement is increased from 0.06 to 0.08 as per BS EN 1992-1-1 and AS3600. CSA S6 and ACI 318 retain the 0.06 constant. The BS EN 1992-1-1 code may introduce a change that allows a reduction in this constant if N ductility grade ligatures are used. The research was not fully verified, consequently this could not be included into Amendment 2.

Amendment 1 of AS5100.5 Clause 8.2.1.6 (c) requires that minimum shear reinforcement be provided in all beams with a depth greater than or equal to 750mm. That requirement then flowed into design of one-way slabs and via clause 9.2.1 can be interpreted to apply to two-way slabs as well. This clause was introduced in response to laboratory testing that suggested that the shear strength of unreinforced slabs was not proportional to element depth. Testing of slabs greater than 400mm was limited and the maximum slab depth tested was close to 750mm.

Recent international testing has shown that the model for strength reduction with increasing beam depth can be extended to 3m deep beams. This is now reflected by the change to clause 8.2.1.6 (c) in Amendment 2. There is now an agreed model for the cause of the strength reduction. Inclined shear cracks initially form in the beam. These inclined cracks are effectively restrained in the compression zone and again at the tension reinforcement zone. However, they are poorly restrained between these two zones. As a beam depth increases so the height of this poorly restrained zone increases resulting in increased crack widths and consequently reduced shear transfer across the crack at these mid beam depth zones. Papers in the fib Bulletin 81 describe this model and the factors that influence it.

The Amendment 2 slab shear provision for punching shear has conservatively adopted the same twin factor approach used for beam shear in AS3600 and AS5100.5 Amendment 2 clauses 8.2.1.6 (a) and equation 8.2.4.2 (1) second term. The fib bulletin 81 states that the size effect should be considered in the design of one- and two-ways slabs. The bulletin does states that the effect is much diminished from that of beams. The one-way slab strength reduction curve is between the beam and two-way slab curves.

Minimum shear reinforcement is one way to control the width of the inclined shear cracking in the mid height zone of the beam. Consequently, k_{vucs} is taken as 1.0 when at least minimum shear ligatures are included into the slab. Horizontal and inclined bars may also manage shear crack widths in thick slabs.

3.3.5 Shear Reinforcement Detailing for Punching Shear (Clause 9.2.3.6)

The maximum spacing of shear ligatures is an extension of that proposed for beams with additional consideration of the geometry of slabs and the stress levels in two-way slabs as dictated by equations such as equation 9.2.3 (2). The spacing rules are also tested against the rules in the reference international standards. The spacing as a proportion of D_s is reasonably common between international standards. Few international standards place fixed limits on the max reinforcement spacing, contrary to AS3600 and AS5100 provisions.

The authors are aware of deep/thick slabs detailed to the beam shear rules in AS5100.5 that have been very congested and complex to place and inspect. There were potentially SiD issues regarding fixing reinforcement and cleaning/preparing formwork. New spacing rules aim to assist with these challenges. The design and then detailing of shear ligatures in slabs is based on average stresses along a length of critical perimeter. Consequently, the detailing and pragmatic arrangement of ligatures should result in a regular rectangular grid in most slabs. The plan area containing shear fitments for a circular critical perimeter could be placed within a square that has the same area as the circular perimeter, and then extended by the minimum distance D_s (Figure 10).

The position of the first fitment adjacent to the column face is important as this first critical perimeter is often the most highly stressed. If the first leg is too close to the column face, then the shear crack can pass below the ligature in the cover concrete.

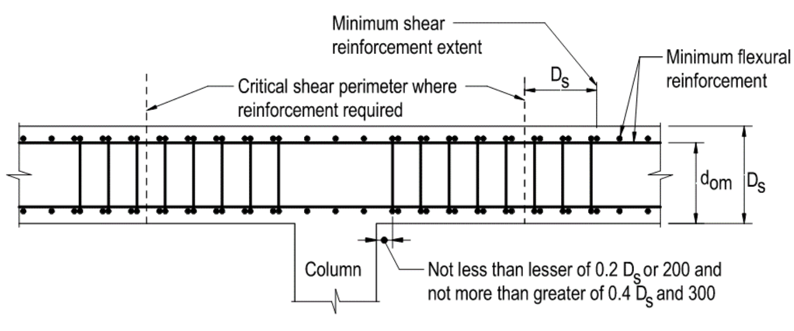

Figure 7. Shear reinforcement D_s beyond perimeter required

With irregular column and critical perimeter shapes it is not pragmatic to have every shear leg within the range of distances specified. The proportion of D_s should take precedence over the fixed distance value. It is important that the mix and average positions of these legs on each side of the column be assessed against the expected shear crack shapes.

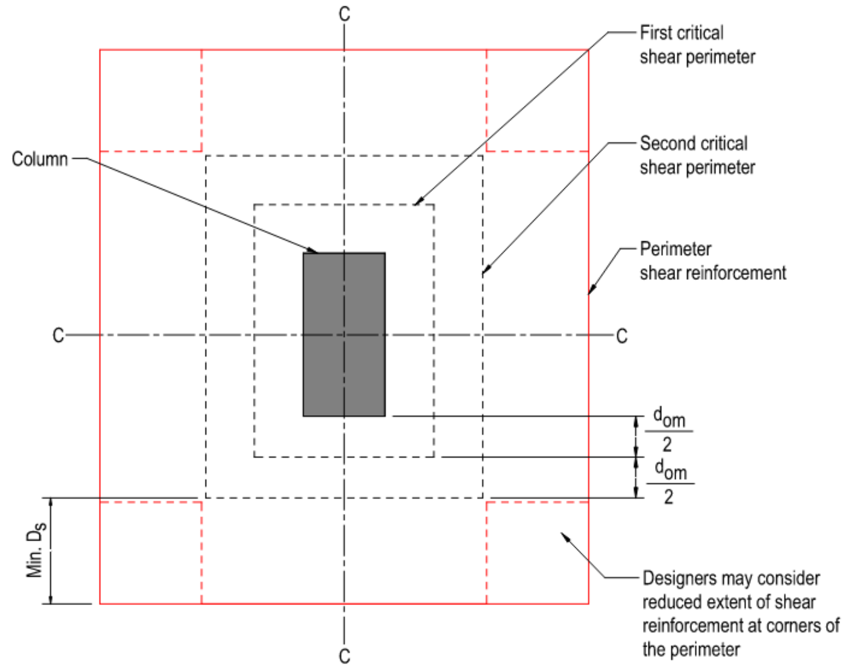

Figure 8. Flat slab - rectangular column
(Shear Reinforcement required to 2nd critical shear perimeter)

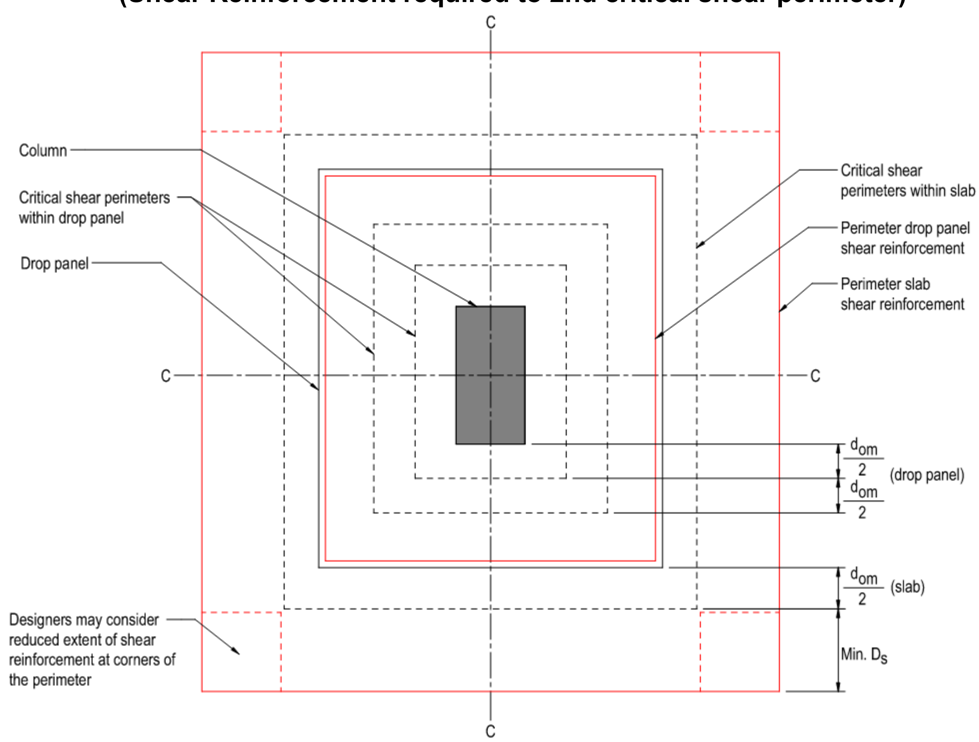

Figure 9. Slab with drop panel – rectangular column
(Shear reinforcement required in drop panel and to 1st critical perimeter in slab)

Lapping of shear legs is permitted because the legs are within the slab body. The few shear legs at the slab edge or down stand face are a minor proportion of the total area of shear legs on any critical perimeter. The clause 8.3.2.4 (b) rule regarding a 0.8 factor for ligatures anchored in the tension face of the beam is not included because control of cracking at ligatures in two-way slabs is covered by equation 9.2.3 (2). Bends in fitments shall enclose a longitudinal bar, usually a bar in flexural bar layer 2.

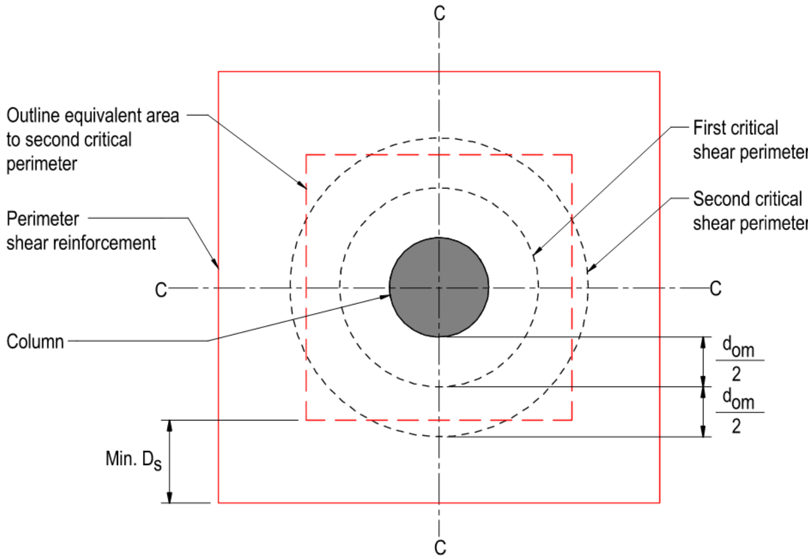

Figure 10: Flat Slab – Circular column: (Shear reinforcement required to 2nd critical perimeter)

4 Conclusions

The new approach to the design of two-way slabs for shear, particularly thick slabs, offers an approach that has clarity; has a single process irrespective of the size of moment transfer to the column; has bespoke ligature detailing rules appropriate to punching shear in two-way slabs and is largely independent of AS3600. The method for punching shear design is based on the traditional truss analogy that is currently used in international standards for punching shear design and incorporates the latest thinking regarding the decrease in shear strength due to cracking of either slabs with or without shear reinforcement. The method presented in Amendment 2 is expected to be replaced when a suitable strain-based design method for punching shear has been developed, agreed, and verified.

5 Acknowledgement

The authors thank the CIA for providing a platform for discussion on punching shear at the national webinar of 28 October 2020 and acceptance of the authors paper[4] at Concrete 2021. The authors thank the BD 090-05 Committee at Standards Australia for the development of the draft clauses into a concise and effective Amendment for slab punching shear to AS 5100.5 Bridge Design Code – concrete.

6 References

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