University of Southern Queensland,
Faculty of Health, Engineering and Sciences

A comparison of the approaches between ACI 318-2014, Eurocode 2 and AS 3600-2018 in the analysis of punching shear

A dissertation submitted by
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Abstract

Punching shear is a structural behaviour encountered in the analysis and design of reinforced concrete flat slabs. It is caused by the concentration of a load, referred to as a shear force, over a small area. There are differences in the code provisions between the American (ACI 318-2014), European (EN 1992-1-1-2004+A1-2014) and Australian (AS 3600-2018) concrete design codes for the assessment of punching shear.

The aims of this project are to investigate and compare these differences and to rate the cost effectiveness of the three codes for punching shear design. Although various experiments and papers have undertaken to compare the punching shear provisions between codes, there is a research gap for research that considers and rates the cost effectiveness between the codes. Furthermore, when the provisions of codes are compared in experiments, ACI 318 and EN 1992 are often considered, however AS 3600 is seldom included in these experiments. The project aims were achieved through a literature review and a hand calculation comparison between the provisions of the three codes on three typical column locations on a proposed typical residential building floor slab. Utilisation ratios (applied shear/shear capacity) were determined for each of the locations and used as the basis of the findings. A software analysis, using Tekla Structural Designer (TSD), was used to validate and correlate the results from the hand calculation analysis.

The findings of the research are that the punching shear code provisons of ACI 318 and AS 3600 are similar in their approches, however the provisions of EN 1992 differ significantly to the those of the other two codes. EN 1992 was determined to be the most conservative code for punching shear design, with AS 3600 and ACI 318 being second and third respectively. One reason identified for the conservatism noted in EN 1992 is the ultimate limit state (ULS) dead load factor of 1.35 that is considered in the code, while 1.2 is considered in both ACI 318 and AS 3600. These findings indicate that designers who are familiar with the provisions of ACI 318 and AS 3600 should be aware that higher punching shear utilisation ratios will be observed when designing building structures in jurisdictions covered by the Eurocode.

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Acknowledgements

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"If we knew what we were doing, it would not be called research, would it?"
- Albert Einstein

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List of Abbreviations

ACI	American Concrete Institute
AS	Australian Standard
ASCE	American Society of Civil Engineers
FE	Finite Element
fib	The International Federation for Structural Concrete
NBR	Norma Brasileira Regulamentadora (Brazilian National Standards)
NLFEA	Nonlinear finite element analysis
TSD	Tekla Structural Designer
ULS	Ultimate limit state

Chapter 1: Introduction

1.1 Introduction

Punching shear is a structural behaviour encountered in the design of reinforced concrete flat slabs. It is caused by the concentration of a load, or shear force, over a small area.

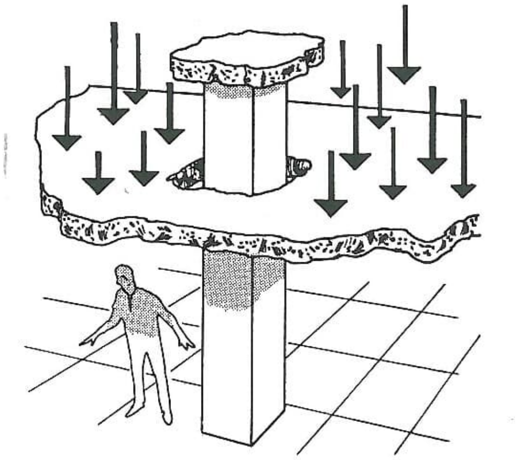


Figure 1: Image depicting punching shear failure (Mirzael and Muttoni, 2008)

Punching shear in flat slabs is assessed based on the provisions of the international design code being considered for the structural design. However, these code provisions differ between the various international design codes.

This project aims to identify and clarify the differences between three design codes, namely the American code (ACI 318-2014), the British National Annex of the European code (EN 1992-1-1-2004+A1-2014) and the Australian code (AS 3600-2018). The project also aims to rate the cost effectiveness of the three codes for punching shear based on the utilisation ratios (applied shear/shear capacity) derived from each of the codes.

This first chapter of the project report will introduce the research topic by providing the background, statement of the research problem, the rationale and scope of the project.

1.2 Background

Punching shear is a brittle failure mode encountered in the design of reinforced concrete flat slabs, where the slab fails in shear (or sliding) at a distance from the face of a column. Because punching shear is a brittle failure mode, it is critical that structural designers understand and design for punching shear safely.
McCormack and Brown (2014), state that punching shear is the critical factor in design for concrete slabs supported directly on columns.

Figure 2: Floor collapse of Wolverhampton Parking due to punching shear failure (Wood, 2003)

According to Lantsoght (2009), The methods currently used to describe punching shear cannot adequately explain the mechanics of punching shear and therefore semi-experimental formulas have been developed which lead to safe designs for commonly used structures.

Based on this approach, various building and design codes approach the analysis and design of punching shear differently. One underlying method which is used by the various international design codes is the ***shear strength method***, where the shear stress on a critical section at a certain distance from the face of the column is compared to a maximum shear stress (Lantsoght,2009). This method was developed by Moe (1961) and is the basis for the punching shear provisions of ACI 318, EN 1992 and AS 3600. The distance considered from the face of the column for each of the design codes, was determined through statical analyses (Lantsoght, 2009) and determined to be 2xd for EN 1992 and 0.5xd for ACI 318 and AS 3600, where d is the effective depth of the slab.

An alternative method developed for the assessment of punching shear is the ***critical shear crack theory*** method. This method describes the relationship between the punching shear strength of a slab and its rotation at failure (Muttoni, 2008). The critical shear crack theory method will form the basis of the punching shear sections of the new Eurocode design standard, which has a planned publication date of 2026 (Concrete-Centre, 2020).

1.3 Statement of the problem

The provisions of the codes for punching shear have been compared through various experiments and papers over the years. These comparisons include various papers considering the effect of openings, concrete strength and thickness of slabs on the punching shear strength of slabs. However, there appears to be a gap for research that considers which code is more cost effective for punching shear design based on utilisation ratios (applied shear/shear capacity) determined for each location. A reason for this gap, may be attributed to the fact that the punching shear equations in each design code cannot be directly compared to each other due to the different philosophies used in their derivations (Gardner, 2005). Furthermore, when the provisions of codes are considered, ACI 318 and EN 1992 are often compared to each other, however AS 3600 is often not included in these comparison studies.

Punching shear theory continues to be an evolving topic with the recent relevance of *critical shear crack theory* versus the traditional *shear strength method*. Furthermore, the engineering industry is becoming more inter-connected, where engineers can work on projects outside their local geographies. This project aims to provide a comparison of punching shear analysis provisions between three codes especially as ACI 318 and EN 1992 are used widely in regions outside the USA and Europe.

1.4 Rationale

The aim of this project will be to compare the differences in approach for the assessment of punching shear between three codes, namely ACI 318-2014, EN 1992-1-1 and AS 3600-2018. The project also aims to rate the cost effectiveness of the three codes for punching shear based on the utilisation ratios derived from each code.

Therefore, the research questions to be considered for the project are:

1. What are the differences in the approach to the assessment of punching shear between ACI 318-2014, EN 1992-1-1 and AS 3600-2018?
2. Which design code, between ACI 318-2014, EN 1992-1-1 and AS 3600-2018, is more cost effective in its approach to punching shear design?

The project aims will be achieved through the research objectives which are to:

1. To conduct a literature review on the punching shear provisions of ACI 318-2014, EN 1992-1-1 and AS 3600-2018
2. Undertake a hand calculation design comparison between the three codes on three typical column locations on a typical residential floor. A utilisation ratio (applied shear/shear capacity) will be determined for each typical location based on each code.
3. Model the typical residential floor slab in Tekla Structural Designer (TSD), a building analysis and design software program that incorporates a Finite Element (FE) engine with automated FE meshing tools (Tekla, 2023). The TSD model will be used to check the three typical slab locations based on the provisions of the three codes and a utilisation ratio determined from each one.
4. Compare the results of the hand calculation analysis to the TSD results.

1.5 Scope

The scope of the project is limited to lightly loaded reinforced concrete flat slabs with simple geometry, for example slabs that would be specified for residential multi-storey buildings. The project is limited to punching shear locations that are not provided with punching shear reinforcement. Therefore, the requirements for the provision of punching shear reinforcement as specified in the three codes will not be considered.

Furthermore, the basis of the study on EN 1992 will be limited to the British National Annex, NA+A2-14 to BS EN 1992-1-1-2004. The national annexes of the other countries that EN 1992 covers will not be considered.

In addition, the provisions of the latest ACI 318-2019 will not be considered. The project will be based on the widely used ACI 318-2014. ACI 318-2019 excludes the Direct Design method in the provisions for two-way slabs (Moehle, 2019), however the Direct Design method was used to determine the slab bending moments in the hand calculation analysis. It should be noted that the Direct Design method is still permitted by ACI 318-2019 although it has been excluded from the code.

1.6 Structure of the project

The remainder of the project report is included in Chapters 2 to 5 and structured as follows:

Chapter 2:

Provides a literature review highlighting the relevant background information related to the punching shear code provisions. The chapter focuses on the code provisions of the three codes being considered. The conclusion of the Chapter includes a summary table of the punching shear code provisions between the three codes.

Chapter 2 will address the first research aim.

Chapter 3:

The detailed methodology for the project will be discussed in Chapter 3. This chapter will include the hand calculation and the software analysis results.

Chapter 3 will address the second research aim, however the results thereof will be discussed and presented in Chapter 4.

Chapter 4:

Chapter 4 continues on from Chapter 3 and presents the results derived from the methodology chapter. Discussions on the results will be presented in this Chapter.

Chapter 5:

Chapter 5 will present the conclusions of the research project and the recommendations for potential further work. This chapter will conclude the project report.

Chapter 2: Literature Review

2.1 Introduction

Chapter 2 will focus on a review of a selection of the existing literature on the code provisions for the assessment of punching shear. In addition, this review will include a review of the studies that have been carried out to compare punching shear provisions between design codes.

The chapter will highlight the basis of the punching shear code provisions, the differences in the approaches between the three codes and the factors that affect the punching shear strength of slabs.

Through the literature review the research gap of a comparative study that includes AS 3600-2018 and highlights which code is more conservative will be established. The sources for the literature review are academic journals and industry websites, however the literature review is based primarily on the three design codes on which the research project is based.

The literature review chapter is divided into six parts, namely:

1. The basis of the punching shear code provisions
2. The factors affecting punching shear resistance.
3. The code provisions of ACI 318
4. The code provisions of EN 1992
5. The code provisions of AS 3600
6. Summary

2.2 Basis of the punching shear code provisions

This section aims to provide an overview of the basis of the punching shear code provisions currently encoded in the three design codes under consideration.

2.2.1 Shear Stress Theory

According to Alexander and Simmonds (1986), the shear stress theory, which compares the shear stress on a critical section with a maximum shear stress, is perhaps the simplest approach to the mechanics of punching shear. This theory is favoured by most design codes including the three codes being considered for this research project.

Moe (1961) conducted tests on numerous slabs and reported inclined cracking at 60% of the ultimate load. Based on these tests, he introduced three levels of shear force comprising the shear force at which inclined cracks form, the shear force at which failure in the compression zone (the soffit of the slab) occurs and the shear force at the ultimate flexural strength. A statistical analysis of the test data showed that that the best agreement between the shear force values resulted for a critical perimeter at a distance of 0.5xd away from the face of the column.

Moe (1961) developed a semi-empirical formula for the ultimate shear strength, where the nominal shear stress can be presented as:

$$v = V/bd$$

Where: V = Shear force

b = width of critical section in shear

d = effective depth of the slab

This empirical formula is used to determine the applied shear force at the punching shear locations and is included in all three of the codes being considered.

Although there are several studies on the weaknesses of the shear stress theory, Lantsoght (2009) states that it is still the most commonly used method and serves as a basis for the code provisions.

2.2.2 Critical Shear Crack Theory

An alternative method to the shear stress theory is the critical shear crack theory. According to Muttoni (2008), this theory describes the relationship between the punching shear strength of a slab and its rotation at failure. The theory is based on the assumption that the shear strength of a slab without transverse (or punching shear) reinforcement is governed by the width and roughness of an inclined shear crack that develops through the included compression strut carrying shear.

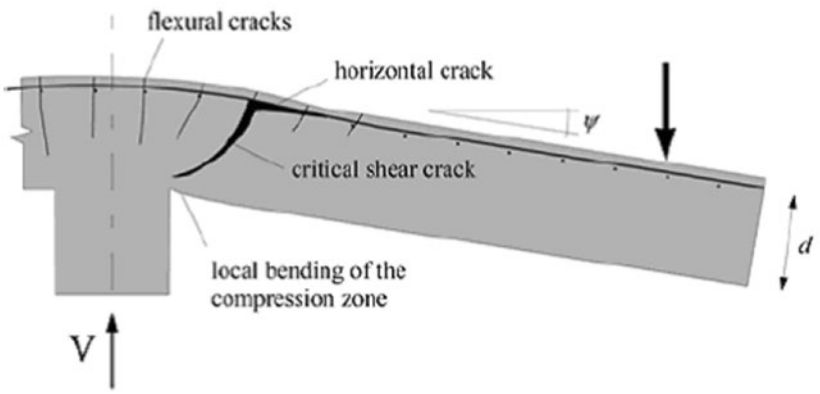


Figure 3: Interpretation of measurements according to critical shear crack theory (Guandalini et al, 2009)

The critical shear crack theory method will form the basis of the punching shear sections of the new Eurocode design standard, which has a planned publication date of 2026 (Concrete-Centre, 2020).

2.3 Factors affecting punching shear resistance

Several factors have been identified in the existing literature that have a significant impact on the punching shear resistance of a slab location. Three of the main factors are included in this section as part of the literature review.

2.3.1 Concrete Strength

Punching shear strength is directly related to the concrete strength of the flat slab, however it is not clear if this relationship is a square or cubic root dependence (Lantsoght, 2009).

Mitchell et al (2005), state that it is not clear if the punching shear strength is proportional to the square or cubic root and additional research is required to establish this relationship. The code provisions of ACI 318 and AS 3600 consider a square root relationship while the provisions of EN 1992 consider a cubic root relationship.

2.3.2 Provided Flexural Reinforcement

Mitchell et al (2005), studied the influence of the provided tension reinforcement ratio on ultimate shearing strength and found that an increase in the flexural reinforcement ratio increases the shear load carrying capacity of a location. It is to be noted that the flexural reinforcement ratio is included in the shear strength equations of EN 1992, however the provisions of ACI 318 and AS 3600 do not include this ratio.

2.3.3 Slab Openings

A comparative topic that is well represented in the literature considers the differences in code provisions for slab openings adjacent to column positions.

Lourenco et al (2021), state that openings located adjacent to loaded areas decrease the resistance of slabs as they result in the removal of concrete and reinforcement at the opening, reducing the critical shear perimeter. The authors conducted a study to investigate the effect of the location of openings in relation to column positions and compared these values to the values predicted by four codes, namely ACI 318-19, EN 1992, fib Model Code 2010 (MC2010) and NBR 6118. ACI was found to be more conservative with a 16% average value higher than EN 1992. It should be noted that the ultimate limit state (ULS) load factors were not incorporated in these analyses.

Al-Rousan and Alnemrawi (2023), conducted a study of twenty-one models using NLFEA to assess the effect of opening sizes and locations. It was found that the opening size in slabs affects the flat slab behaviour in all aspects including cracking, ultimate load, and ultimate deflection. The code provisions of ACI 318-2019, EN 1992 and fib Model Code 2010 were compared to each other where it was observed that the ACI 318 and MC2010 have a close prediction in most cases to the results derived from the NLFEA analyses. Furthermore, the ACI 318 provisions were found to be the most accurate from among the tested codes.

The punching shear code provisions for ACI 318-2014, EN 1992 and AS 3600-2018 are presented in the sections to follow.

2.4 ACI 318-2014

The ACI 318-2014 code provisions for punching shear are detailed in Clause 8.4.4 ad 22.6 of the code.

2.3.1 Notation and Terminology

A	=	loaded area = Total area – critical area
b_o	=	perimeter of critical section for two-way shear in slabs
d	=	the average of the effective depths in the two orthogonal directions
f'_c	=	compressive cylinder strength of concrete at 28-days
h	=	overall depth of a slab
M_{sc}	=	factored slab moment that is resisted by the column at a joint
V_c	=	nominal shear strength provided by concrete
V_u	=	maximum factored two-way shear force

W_u = factored load
α_s = constant used to calculate V_c in slabs
β = ratio of the long to short sides of the column
γ_f = factor used to determine the fraction of M_{sc} transferred by slab flexure at slab-column locations
γ_v = factor used to determine the fraction of M_{sc} transferred by eccentricity of shear at slab-column locations
λ = modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal-weight concrete of the same compressive strength = 1.0 for normal-weight concrete
λ_s = size effect factor used to modify shear strength based on the effects of member depth
φ = strength reduction factor = 0.75 for shear

2.3.2 Effective Applied Shear Force - V_u

ACI 318-2014 prescribes that one-way shear and two-way shear be checked for flat slab systems. One way shear, which is analogous to beam shear, is beyond the scope of the project and will not be considered.

Two-way shear, or punching shear, is checked in the vicinity of columns, concentrated loads and reaction areas, according to the equation below:

The applied shear force $V_u = w_u A$

2.3.2.1 Critical Perimeter – b_0

The critical shear perimeter is checked at a distance of 0.5xd from the face of the column. Straight lines are assumed to define the critical perimeter for rectangular and square columns.

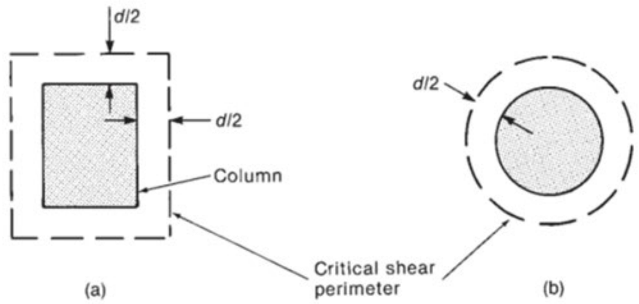


Figure 4: Basic control perimeters for different column geometries (Wight, 2016)

The code prescribes that the critical perimeter be reduced for slab openings located closer than 4h from the face of a column.

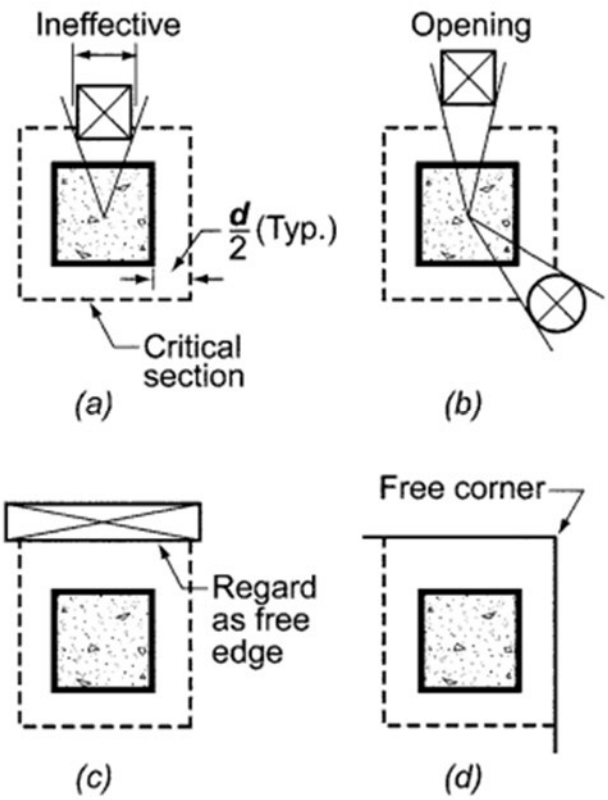


Figure 5: Openings close to loaded perimeters – Figure R22.6.4.3 in the code (American-Concrete-Institute, 2014)

2.3.3 Punching Shear Resistance

The procedure for checking the punching resistance of as slab location is dependent on the moment transfer at the location.

2.3.3.1 Uniform two-way shear (Without moment transfer)

Determine V_c:
The punching shear resistance of a slab location is determined as the lesser of:

v_c		
Least of (a), (b), and (c):	$0.33\lambda_s\lambda\sqrt{f'_c}$	(a)
	$\left(0.17+\frac{0.33}{\beta}\right)\lambda_s\lambda\sqrt{f'_c}$	(b)
	$\left(0.17+\frac{0.083\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f'_c}$	(c)

Figure 6: Equations for determining V_c - Table 22.6.5.2 in the code (American-Concrete-Institute 2014)

Where:

$\lambda_s = \sqrt{(2/(1+0.004d))} \leq 1$
α_s = 40 for internal columns, 30 for edge columns and 20 for corner columns
$\sqrt{f'c} \leq 8.3$ MPa

Determine $V_{u,max}$:

$V_{u,max}$ is the maximum permissible shear force in two-way shear at the location and therefore if $V_u > V_{u,max}$ a redesign of the structural system is required.

$V_{u,max} = \varphi/2 \sqrt{f'c}\ b_o\ d$

2.3.3.2 Two-way shear with an unbalanced moment transfer

The transfer on an unbalanced moment is critical for edge and corner columns, where the unbalanced moment is transferred to the column by flexure and eccentric shear.

$\gamma_f M_{sc}$ is the fraction of the moment transferred by flexure and $\gamma_v M_{sc}$ is the fraction of the moment transferred by eccentric shear.

$\gamma_f = 1/(1+(2/3)\sqrt{(b_1/b_2)})$ and $\gamma_v = 1- \gamma_f$

b_1 and b_2 are the widths of the critical cross section in the longitudinal and transverse directions respectively.

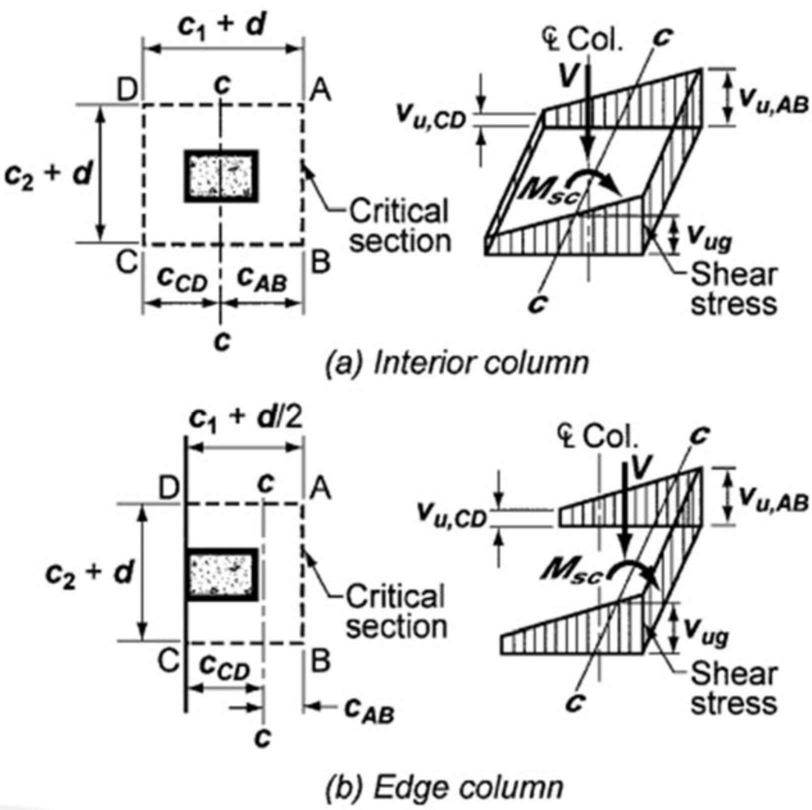


Figure 7: Assumed distribution of shear stress - Figure R8.4.4.2.3 in the code (American-Concrete-Institute 2014)

2.5 **EN 1992-1-1:2004**

The provisions for punching shear according to EN 1992 are set out in Clause 6.4 of the code.

2.5.1 Notation and Terminology

d = mean effective depth of the slab taken as $(d_y+d_x)/2$, where d_x and d_y are the slab effective depths in the x and y directions respectively

f_{cd} = design value of concrete compressive strength = f_{ck}/γ_c

f_{ck} = characteristic compressive cylinder strength of concrete at 28 days

u_1 = length of the control perimeter under consideration

V_{Ed} = design value of the applied shear force

$V_{Rd,c}$ = the design shear resistance of the member without shear reinforcement

$V_{Rd,max}$ = design value of the maximum shear force which can be sustained by the member

γ_c = partial factor for concrete = 1.5

2.5.2 Effective Applied Shear Stress - V_{Ed}

The code presents the method for calculating the effective applied shear stress as per the equation below:

$V_{Ed} = \beta \, V_{Ed}/u_1 \, d$

2.5.2.1 Beta Factor – β

The beta factor is a magnification factor introduced in the calculation of the effective shear stress to account for shear force eccentricities at the punching shear location. These eccentricities may be due to unbalanced moments for unequal slab spans.

A rigorous method of calculating the Beta factors is presented in Clause 6.4.3.(3) to 6.4.3.(5) of the code. However, a simplified method is presented in Clause 6.4.3.(6). The simplified method is limited to building structures where:

- The lateral stability of the structure does not depend on column moment frames; and
- The adjacent spans of the slab do not differ by more than 25%.

The simplified β values are presented in Figure 8 below.

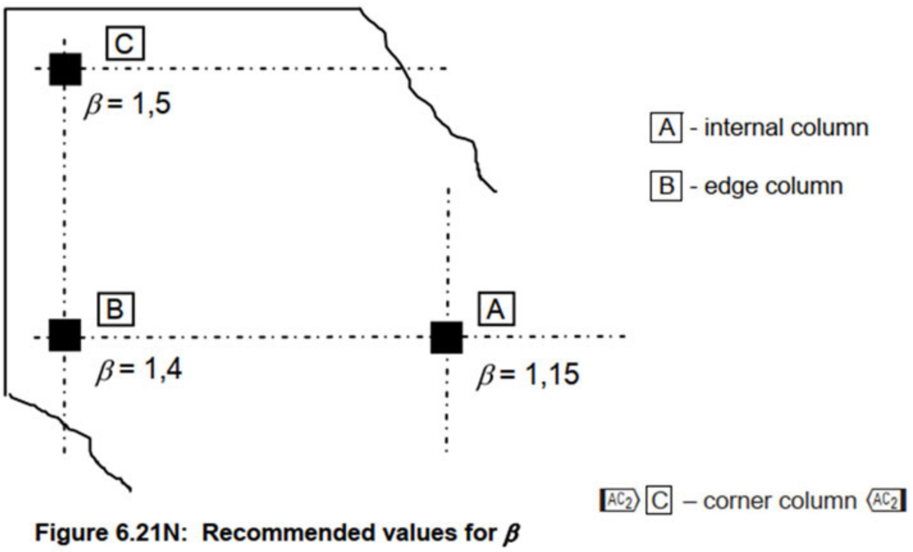


Figure 8: Simplified recommended values for β - Figure 6.21N in the code (British-Standard 2004)

This research project will be incorporate the simplified Beta factors.

2.5.2.2 Basic Control Perimeter – u_1

The code prescribes the punching shear perimeter be located at a distance of 2xd from the column face, where d is the mean slab effective depth. This perimeter is referred to as the basic control perimeter u_1.

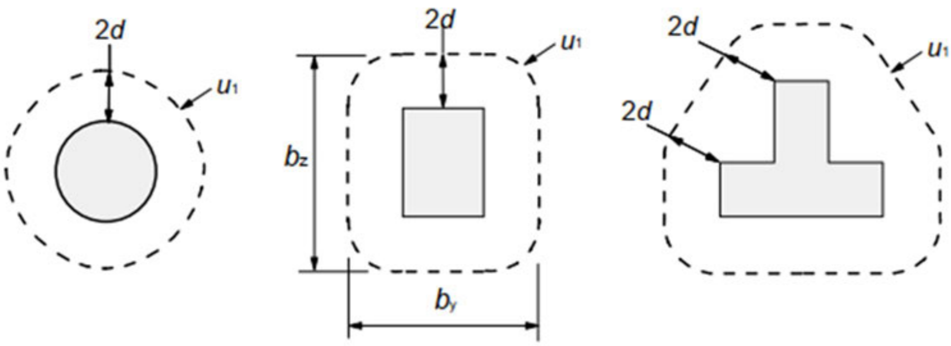


Figure 9: Basic control perimeters for internal columns – Figure 6.13 in the code (British-Standard, 2004)

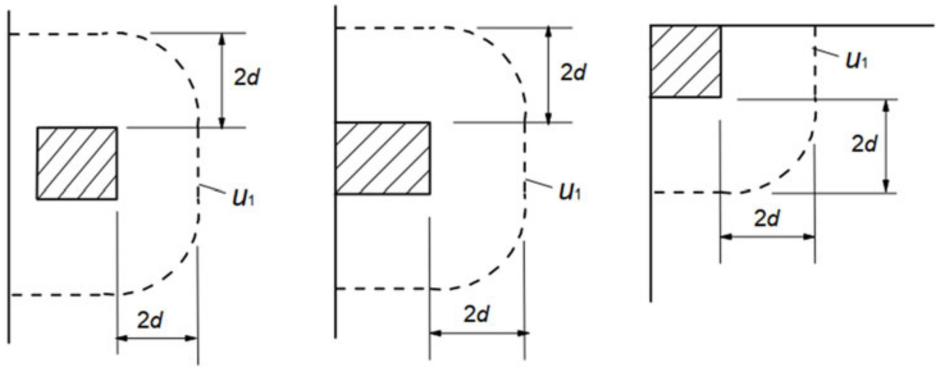


Figure 10: Basic control perimeters for edge/corner columns – Figure 6.15 in the code (British-Standard, 2004)

The basic control perimeter is reduced when openings in the slab are located closer than 6d from the face of the column, as indicated in Figure 11 below:

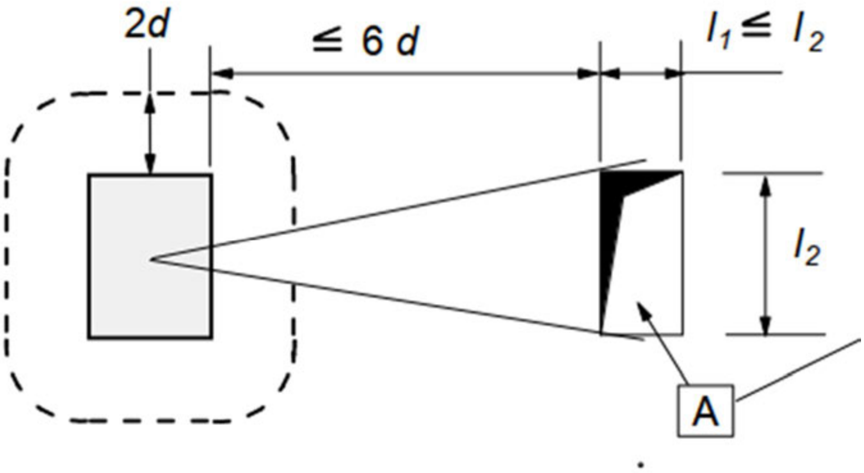


Figure 11: Openings close to loaded perimeters – Figure 6.14 in the code (British-Standard, 2004)

2.5.3 Punching Shear Resistance

EN 1992 details the procedure for checking a location's punching shear resistance as follows:

2.5.3.1 Determine $V_{Rd,c}$:

The code defines $V_{Rd,c}$ as the punching shear resistance of the slab location without punching shear reinforcement being considered.

$$V_{Rd,c} = C_{Rd,c}\ k\,(100\rho_1 f_{ck})^{1/3} + k_1\sigma_{cp} \geq (v_{min} + k_1\sigma_{cp})$$

σ_{cp} in the equation represents the normal concrete stresses in the concrete that result from longitudinal forces caused by a load or more commonly a prestressing action. Prestressing will not be considered for this research project and therefore the equation for $V_{Rd,c}$ simplifies to:

$$V_{Rd,c} = C_{Rd,c}\ k\,(100\rho_1 f_{ck})^{1/3} \geq v_{min}$$

Where:

$C_{Rd,c} = 0.18/\gamma_c$,
$v_{min} = 0.035\ k^{3/2}\ f_{ck}^{1/2}$,
$k_1 = 0.1$,
$k = 1 + \sqrt{(200/2)} \leq 2.0d$ in mm; and
$\rho_1 = \sqrt{(\rho_{ly}\ .\ \rho_{lz})} \leq 0.02$ ρ_{ly} and ρ_{lz} are the bonded tension steel mean areas in the y and z directions over a slab width plus 3 x d on each side of the column.

2.5.3.2 Determine $V_{Rd,max}$:

The code defines $V_{Rd,max}$ as the maximum allowable design value of the shear force which can be sustained by the slab location.

$$V_{Rd,max} = 0.5\ v\ f_{cd}$$

Where:

$v = 0.6\ (1-f_{ck}/250)$ and
$f_{cd} = f_{ck}/\gamma_c$

If $V_{Ed} > V_{Rd,max}$ a redesign is required, this can be achieved by reducing the applied loads, increasing the slab depth or introducing a column head. However if $V_{rd,c} \leq V_{Ed} < V_{Rd,max}$, punching shear reinforcement is required and should be designed and detailed to the requirements of the code. This case falls outside the scope of this project and will not be considered.

2.6 **AS 3600-2018**

The punching shear code provisions of AS 3600-2018 are presented in Clause 9.3 of the code.

2.6.1 Notation and Terminology

a = dimension of the critical shear perimeter measured parallel to the direction of M_v^*
d_o = distance from the extreme compressive fibre of the concrete to the centroid of the outermost layer of tensile reinforcement
d_{om} = mean value of d_o averaged around the critical perimeter
f_{cv} = concrete shear strength
f'_c = characteristic compressive cylinder strength of concrete at 28 days
f_{sy} = characteristic yield strength of reinforcement
M_v^* = design bending moment to be transferred from a slab to a support
u = length of the critical shear perimeter
V^* = design shear force at a cross-section
V_u = ultimate shear strength
V_{uo} = ultimate shear strength of a slab with no moment transfer
φ = capacity reduction factor for design using linear elastic analysis, $\varphi = 0.75$ for shear in members with class N fitments

2.6.2 Effective Applied Shear Stress – v

The effective applied shear stress at any column location is calculated from:

$\nu = V^{*}/u\, d_{om}$

2.6.2.1 Critical Perimeter

The critical shear perimeter is checked at a distance of $0.5 x d_{om}$ from the face of the column. Typical column geometries are indicated in Figure 12 below:

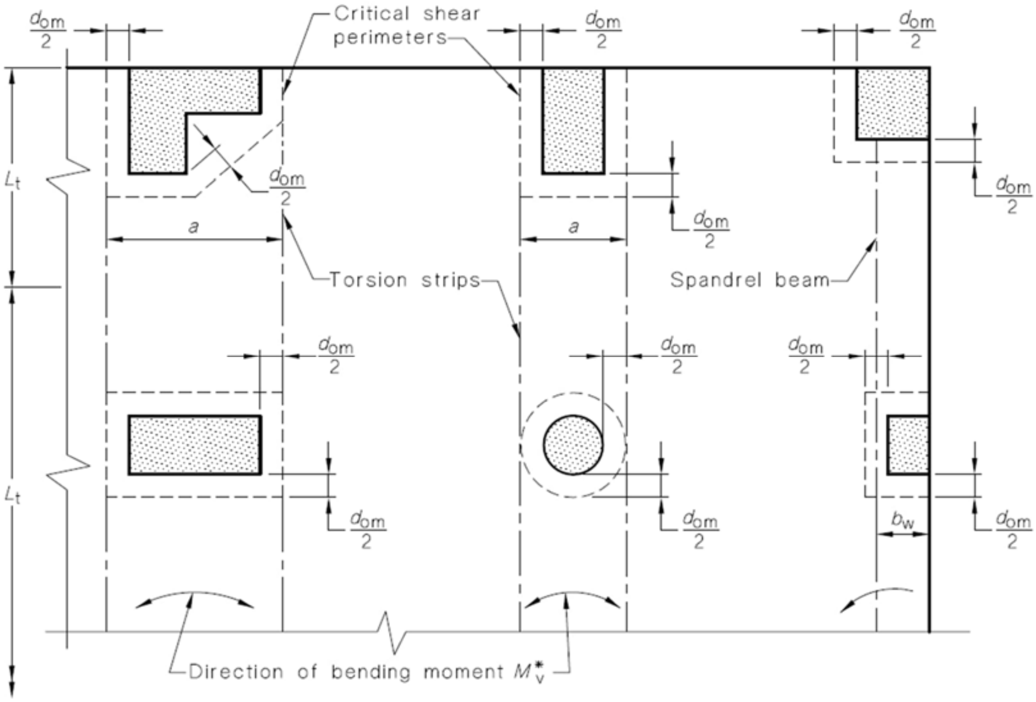


Figure 12: Basic control perimeters for various column geometries - Figure 9.3(B) in the code (Australian-Standard, 2018)

The code defines critical openings as openings that are located closer than $2.5 x b_o$ from the edge of the critical perimeter, where b_o is the dimension of the opening.

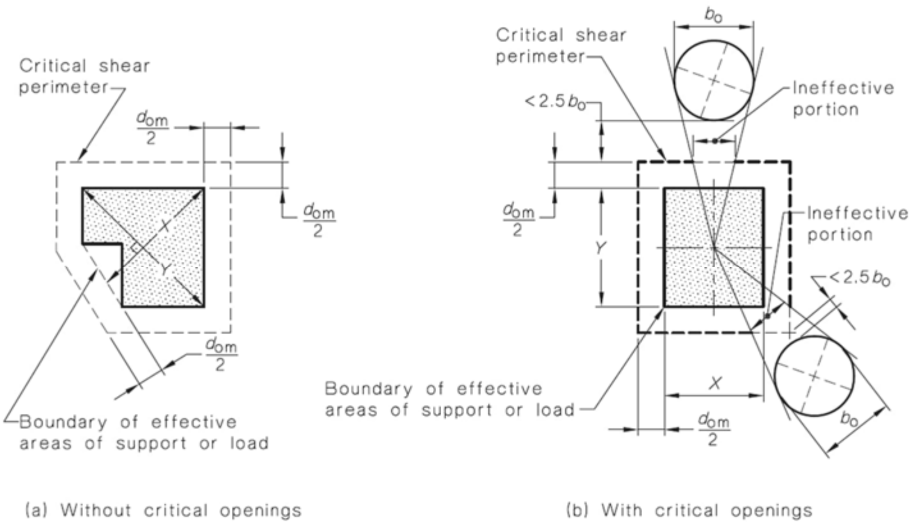


Figure 13: Basic control perimeters for different column geometries - Figure 9.3(A) in the code (Australian-Standard, 2018)

2.6.3 Punching Shear Resistance

The code details the procedure for determining punching shear resistance of a slab location based on the transmitted moment, M_v^*, from the slab into the column.

2.6.3.1 Where $M_v^* = 0$:

$\varphi V_u = V_{uo} = u\ d_{om}.(f_{cv} + 0.3\sigma_{cp})$,

σ_{cp} is the average intensity of the effective prestress in the concrete. Therefore, when prestress stresses are not considered the formula simplifies to:

$\varphi V_u = V_{uo} = u\ d_{om}.(f_{cv})$

Where: $f_{cv} = 0.17(1 + 2/\beta_h)\sqrt{f'_c} \leq 0.34\sqrt{f'_c}$

$\beta_h = Y/X$ (Y = Longest dimension of the effective loaded area, X = Overall dimension measured perpendicular to Y)

2.6.3.2 Where $M_v^* \neq 0$:

Where $M_v^* \neq 0$, the moment is transmitted by flexure and torsion on the critical perimeter.

$\varphi V_u = V_{uo}/(1.0 + u\ M_v^*/(8\ V^*\ a\ d_{om}))$

Where:

a is the dimension of the critical shear perimeter measured parallel to the direction of M_v^*

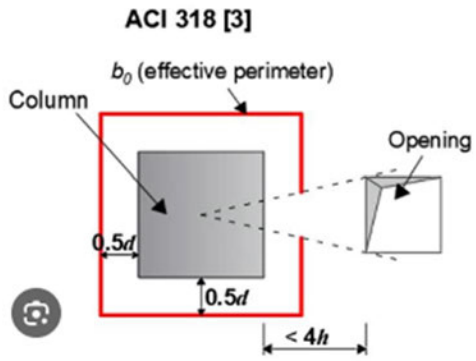
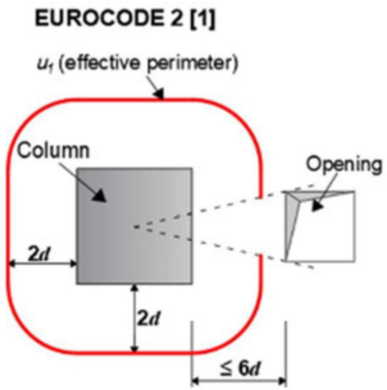
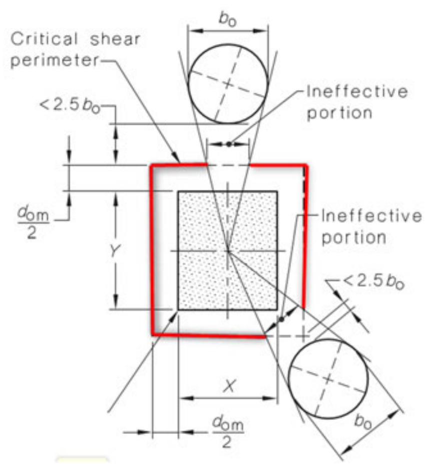
2.7 Summary

The literature review has highlighted that although the punching shear code provisions between ACI 318, EN 1992 and AS3600 are based on the shear stress theory, their execution differs between the three codes. It is evident from the literature review that the provisions of AS 3600 and ACI 318 are similar than those presented in EN 1992.

This section of Chapter 2 summarises the punching shear code provisions for the three codes under consideration and addresses the first aim of the research project, which was to identify and clarify the punching shear code provisions of the three codes.

Table 1: Punching shear code provisions for ACI 318, EN 1992 and AS 3600

Criteria for punching shear	**ACI 318-14**	**BS EN 1992-1-1**	**AS 3600-2018**
ULS factors	1.2DL + 1.6LL	1.35DL + 1.5LL	1.2DL + 1.5LL
Critical perimeter	b_o at 0.5xd	u_1 at 2xd	At 0.5xd_{om}
Length of critical perimeter	2*(d+c_1)+2*(d+c_2) (c_1 and c_2 are the column dimensions)	2*(2*2d+ c_1)+2*(2*2d+c_2)	2*(d_{om}+c_1)+2*(d_{om}+c_2)
Slab openings to consider	Closer than 4h from the face of the column	Closer than 6d from the face of the column	Closer than 2.5x the opening size from the edge of the critical perimeter

			
Applied shear	$v_u = W_u A$ (kN)	$V_{Ed} = \beta V_{Ed}/u_1 d$ (MPa)	$v = V^*/u\, d_{om}$ (MPa)
Accounting for unbalanced moments	Unbalanced moments are transferred to the columns by flexure (γ_f) and eccentric shear (γ_v) $\gamma_f = 1/(1+(2/3)\sqrt{(b_1/b_2)})$ $\gamma_v = 1- \gamma_f$	By the β factor	Unbalanced moments are transmitted by flexure and torsion on the critical perimeter. $\varphi V_u = V_{uo}/(1.0 + u\, M_v^*/(8\, V^*\, a\, d_{om}))$
Punching shear resistance	V_c (kN) is the lesser of: $0.33\lambda_s\lambda\sqrt{f_c'}$ $\left(0.17+\frac{0.33}{\beta}\right)\lambda_s\lambda\sqrt{f_c'}$ $\left(0.17+\frac{0.083\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f_c'}$	$V_{Rd,c} = C_{Rd,c}\, k\, (100\rho_1 f_{ck})^{1/3}$ (MPa)	$\varphi V_u = V_{uo} = u\, d_{om}.(f_{cv})$ (kN)

Maximum allowable punching shear stress	$V_{u,max} = \varphi/2 \sqrt{f'c}\ b_o\ d$	$V_{Rd,max} = 0.5\ v\ f_{cd}$	

In addition to the items noted in Table 1 above, it should also be noted that:

- The flexural reinforcement ratio (ρ) is taken into account when the punching shear resistance is determined according to EN 1992. However, the ratio is not considered for ACI 318 and AS 3600.
- ACI 318 and AS 3600 incorporate the square root of the concrete strength to determine the punching shear resistance. EN 1992 incorporates the cube root of the concrete strength.
- EN 1992 incorporates a partial material safety factor for concrete ($\gamma_c = 1.5$) to account for possible unfavourable deviations from the characteristic design values. ACI 318 and AS 3600 incorporate a shear capacity or strength reduction factor ($\phi = 0.75$) for this purpose.

Chapter 3: Research methodology

3.1 Introduction

The research methodology for the project is based on the literature review conducted in Chapter 2. The punching shear code provisions that were detailed in Sections 2.4 to 2.6 of Chapter 2 will be used as the basis of Chapter 3.

Chapter 3 details the methodology that was followed for the project. The aim of the chapter is to determine the punching shear utilisation ratios ((Shear stress/shear capacity or Shear force/Shear resistance) at three typical slab-column locations on a proposed typical residential floor slab, based on the code provisions of ACI 318, EN 1992 and AS 3600. The values of the utilisation ratios will be used to rate the efficiency of each of the codes, i.e. a lower utilisation ratio indicates a higher punching shear code efficiency while a higher utilisation ratio indicates a more conservative, and therefore less efficient punching shear code efficiency.

The second project aim will be achieved by undertaking a hand calculation analysis at each of the three typical locations based on the provisions of the three codes being considered. A software analysis will also be undertaken for the proposed typical slab at the same three typical locations. The purpose of the software analysis is to correlate and validate the results of the hand calculation analysis. Tekla Structural Designer (TSD) will be used for the software analysis. TSD is a building analysis and design software program that incorporates a Finite Element (FE) engine with automated FE meshing tools (Tekla, 2023).

Chapter 3 is divided into three parts, the structural system and inputs considered, the hand calculation analysis, the software analysis and a summary of the chapter.

3.2 Structural System and Inputs

The structural system considered for the research project comprises a 250mm thick residential floor slab supported by 450mmx450mm reinforced concrete columns on a 6mx6m column grid.

This structural system is included in Figure 14 below, where the three typical locations which will be considered are indicated by the blue circles.

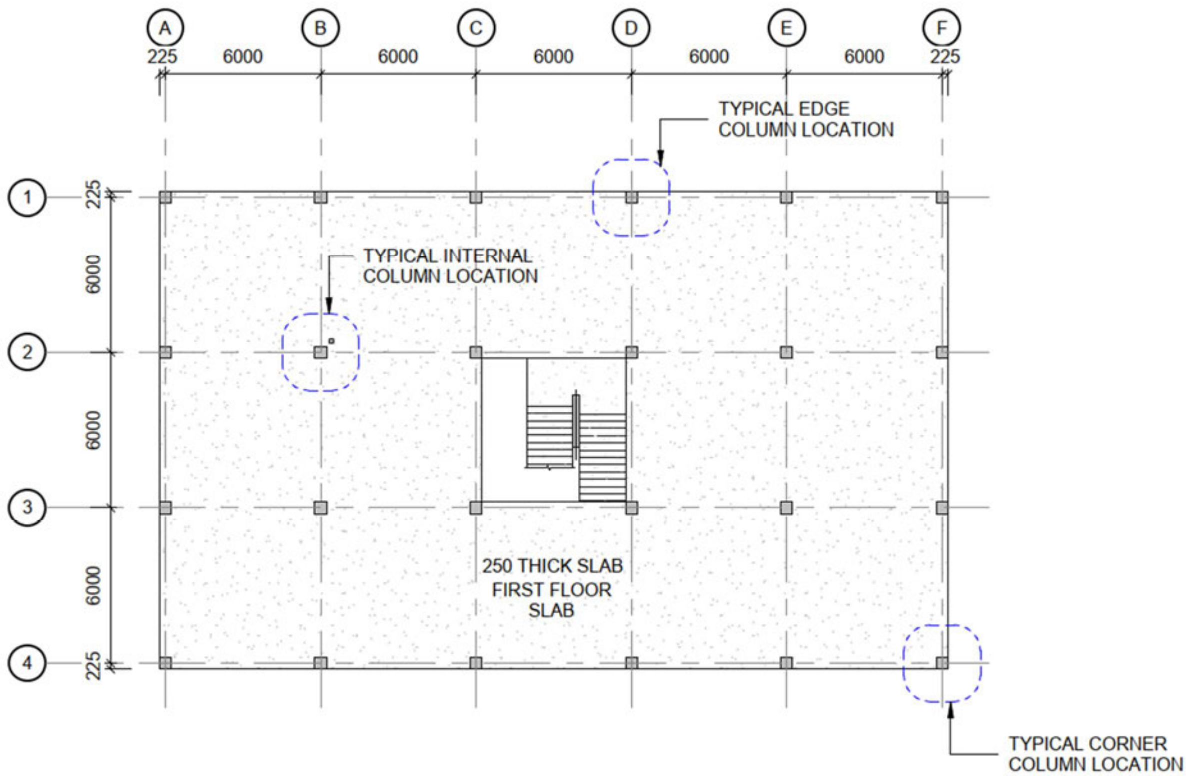


Figure 14: Floor layout indicating the typical punching shear locations to be checked.

The applicable material and loading inputs considered for the project are presented in the tables below:

Table 2: Material Input Table

Material Property	Material Class
Concrete Grade, f_{ck} / f'_c	32 MPa
Reinforcement grade, f_y / f_{sy}	500 MPa

Table 3: Loading Input Table

Load Type	Load
Reinforced concrete density (ρ)	2500 kg/m^3
Superimposed Dead Load (SDL)	2.00 kPa
Imposed Load (LL)	2.00 kPa

It should be noted that the lateral stability of the structural system has not been considered for the structural system and only vertical/gravity loads are included. Cladding and wall loads are not included in the calculations, therefore the checks only consider the area loads as indicated in Table 3.

3.3 Hand Calculation Analysis

The hand calculations analyses were carried out in Microsoft Excel and snapshots of the calculations are included in the Section that follows.

3.3.1 ACI 318-2014

Typical Internal Column Location

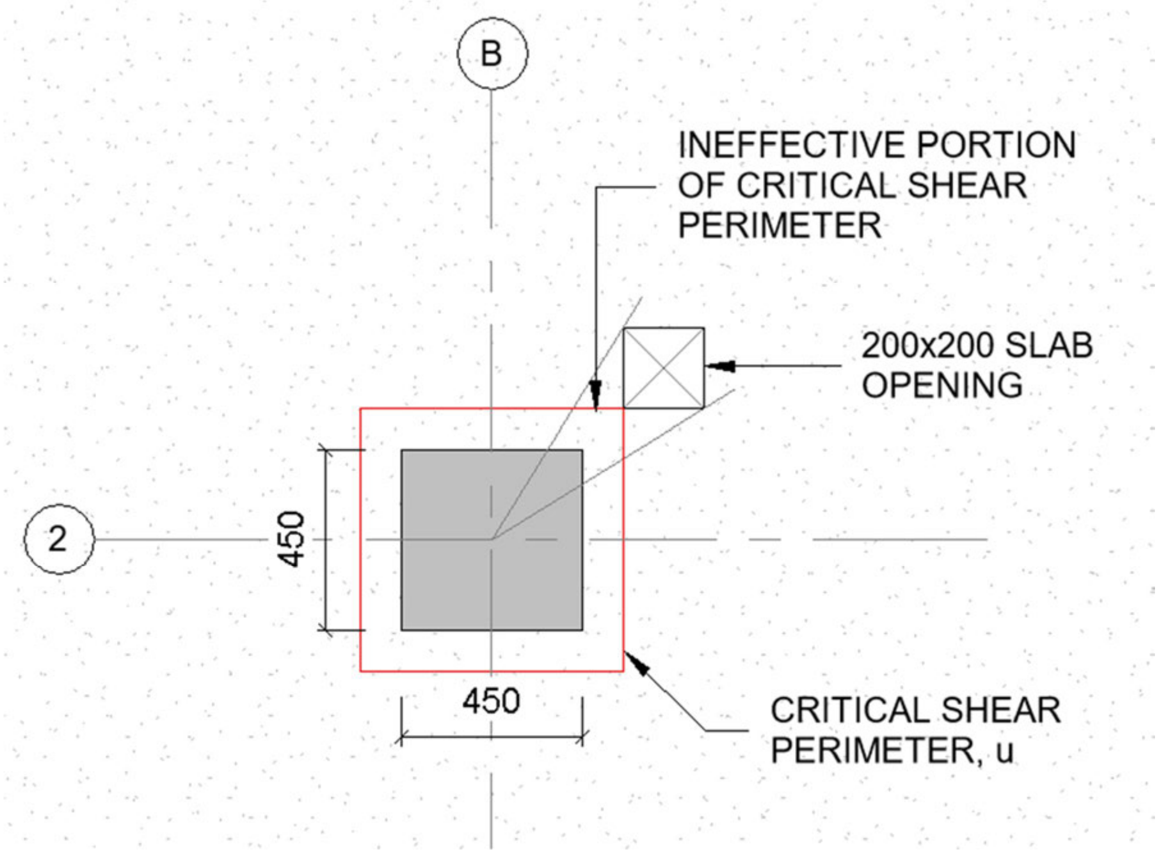


Figure 15: ACI 318 - Typical internal column location

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600			
Design Title:	ACI 318-2014 - Analysis			
Design Title:	Typical Internal Column		Designed By:	G. Mpai
Sheet Number:	1 of 3		Design Date:	2023/08/19

INPUT DATA

Materials:

Reference: ACI 318-2014

f'_c		=	32	MPa
f_y		=	500	MPa
$\rho_{concrete}$		=	2500	kg/m^3
		=	24.5	kN/m^3
Slab depth, h		=	250	mm
Concrete cover		=	30	mm
Column dimensions,	A	=	450	mm
	B	=	450	mm
Opening dimensions,	D	=	200	mm
	E	=	200	mm
Tributary Area		=	6m· 6m	
		=	36	m^2

<u>Provided slab tension reinforcement</u>

(Assumed reinforcement)

x-direction:	dia	=	16	mm
	spacing	=	200	mm
		=	1005	mm^2/m
y-direction:	dia	=	16	mm
	spacing	=	200	mm
		=	1005	mm^2/m
Effective depth, d		=	204	mm

<u>Loading:</u>

Slab DL $= \rho_{concrete} \cdot h$ = 6.13 kPa

Super-imposed DL, SDL = 2.00 kPa

Live Load, LL = 2.00 kPa

q_u = 1.2·(Slab DL + SDL) + 1.6·LL

= 12.96 kPa

V_u = q_u · Tributary Area

= 466.5 kN

RESULTS

0.5·d perimeter at = 102 mm

Determine M_{ol}

M_{ol} = $q_u \times l_2 \times l_n^2/8$ (Equation 8.10.3.2)

= 299 kN.m

Where :

l_2 = 6.00 m

l_n = 6.00 - A/2 -B/2

= 5.55 m

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	ACI 318-2014 - Analysis		
Design Title:	Typical Internal Column	Designed By:	G. Mpai
Sheet Number:	2 of 3	Design Date:	2023/08/19

Reference

Determine the unbalanced moment:

$M_u = 0.70 \cdot M_{ol\ end\ span} - 0.65 \cdot M_{ol\ internal\ span}$ — ACI 318-2014 Table 8.10.4.2 and 8.10.4.1

$= 14.97$ kN.m

(Assume that 100% of the load is to be resisted by the column strip)

Moment transfer design:

Total moment to be transferred

$0.3 \cdot M_{ol} = 89.8$ kN.m — 8.10.4.6

Fraction of unbalanced moment carried by eccentricity of shear

$\gamma_f = 1/(1+(2/3)\sqrt{(b1/b2)})$ — Equation 8.4.2.3.2

$= 0.60$

$b_1 = A + d = 654$ mm

$b_2 = B + d = 654$ mm

$\gamma_v = 1 - \gamma_f$ — Equation 8.4.4.2.2

$= 0.40$

Properties of critical section for shear:

$A_c = (2a + 2b - \text{ineffective perimeter}) \cdot d$

$= 5E+05$ mm^2

$C_{AB} = C_{CD} = 327$ mm

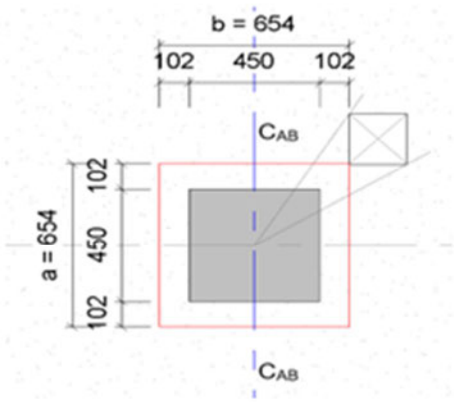
$J_c = d \cdot (a^3/6 + b a^2/2) + a \cdot d^3/6$

$= 4E+10$ m^4

Where :

a = 654 mm

b = 654 mm

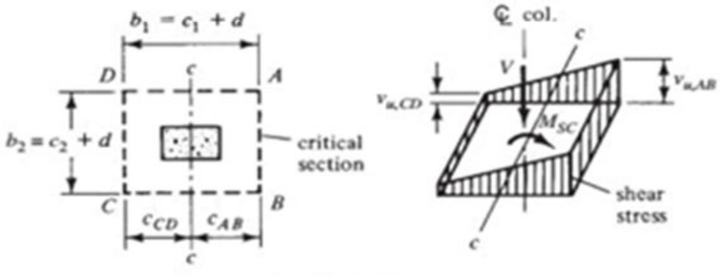


Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	ACI 318-2014 - Analysis		
Design Title:	Typical Internal Column	Designed By:	G. Mpai
Sheet Number:	3 of 3	Design Date:	2023/08/19

Reference

ACI 318-2014

Figure R8.4.4.2.3



(a) Interior column

Gravity load shear to be transferred:

V_u = 466.5 kN

Combined stresses:

V_u = $V_u/A_c + \gamma v \cdot 0.3 \cdot M_{ol} \cdot C_{AB}/J_c$ — R8.4.4.2.3

= 0.966 + 0.301

= 1.268 MPa

V_u should not be greater ϕv_c

ϕv_c = $0.75 \cdot v_c$ — Table 21.2.1

= 1.40 MPa

Where :

v_c is the lesser of: — 22.6.5.2

Table 22.6.5.2

$0.33 \cdot \lambda \cdot \sqrt{f'c}$	=	1.87	MPa
$0.17 \cdot (1 + 2/\beta) \cdot \lambda \cdot \sqrt{f'c}$	=	2.88	MPa
$0.083 \cdot (2 + \alpha_s \cdot d / b_o) \cdot \lambda \cdot \sqrt{f'c}$	=	9.50	MPa

$\therefore V_u$ < $\phi \cdot v_c$

UR = 0.91

Slab has adequate punching capacity

Typical Edge Location

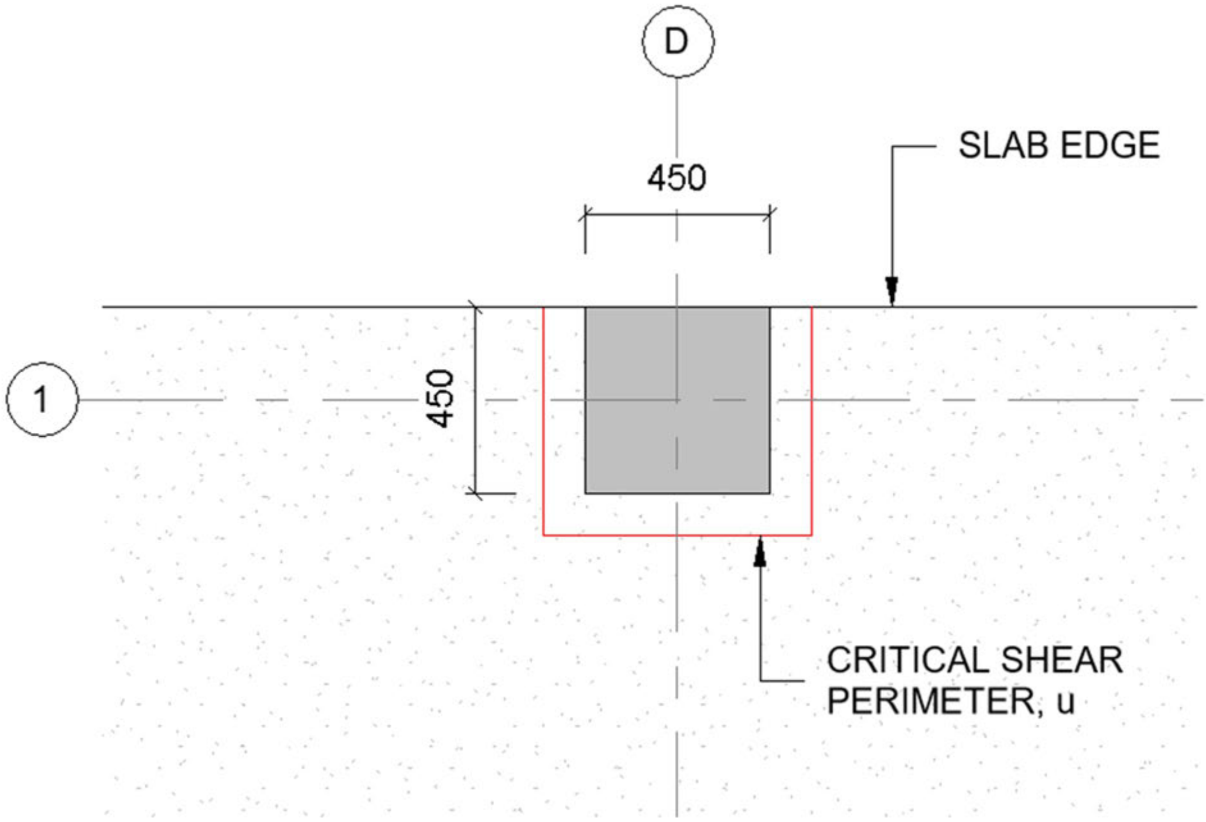


Figure 16: ACI 318 - Typical edge column location

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	ACI 318-2014 - Analysis		
Design Title:	Typical Edge Column	Designed By:	G. Mpai
Sheet Number:	1 of 3	Design Date:	2023/09/02

UniSQ

INPUT DATA

Materials:

Reference: ACI 318-2014

f'_c = 32 MPa

f_y = 500 MPa

$\rho_{concrete}$ = 2500 kg/m^3

= 24.5 kN/m^3

Slab depth, h = 250 mm

Concrete cover = 30 mm

Column dimensions, A = 450 mm

B = 450 mm

Opening dimensions, D = 200 mm

E = 200 mm

Tributary Area = 6m·6m/2

= 18 m^2

Provided slab tension reinforcement

x-direction: dia = 16 mm (Assumed reinforcement)

spacing = 200 mm

= 1005 mm^2/m

y-direction: dia = 16 mm

spacing = 200 mm

= 1005 mm^2/m

Effective depth, d = 204 mm

Loading:

Slab DL = $\rho_{concrete} \cdot h$ = 6.13 kPa

Super-imposed DL, SDL = 2.00 kPa

Live Load, LL = 2.00 kPa

q_u = 1.2·(Slab DL + SDL) + 1.6·LL

= 12.96 kPa

V_u = q_u · Tributary Area

= 233.2 kN

RESULTS

0.5·d perimeter at = 102 mm

Determine M_{ol}

M_{ol} = $q_u \times l_2 \times l_n^2/8$ (Equation 8.10.3.2)

= 299 kN.m

Where :

l_2 = 6.00 m

l_n = 6.00 - A/2 -B/2

= 5.55 m

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	ACI 318-2014 - Analysis		
Design Title:	Typical Edge Column	Designed By:	G. Mpai
Sheet Number:	2 of 3	Design Date:	2023/09/02

UniSQ

Reference

Determine the unbalanced moment: — ACI 318-2014 Table 8.10.4.2

M_u = $0.26 \cdot M_{ol\ endspan}$

= 77.8 kN.m

100% of the load is to be resisted by the column strip

Moment transfer design:

Total moment to be transferred

$0.3 \cdot M_{ol}$ = 89.8 kN.m — 8.10.4.6

Fraction of unbalanced moment carried by eccentricity of shear

γ_f = $1/(1+(2/3)\sqrt{(b1/b2)})$ — Equation 8.4.2.3.2

= 0.62

b_1 = $A + 0.5 \cdot d$ = 552 mm

b_2 = $B + d$ = 654 mm

γ_v = $1 - \gamma_f$ — Equation 8.4.4.2.2

= 0.38

Properties of critical section for shear:

A_c = $(2a+b) \cdot d$

= 358632 mm^2

C_{AB} = $(2a \cdot d \cdot a/2)/A_c$

= 86.66 mm

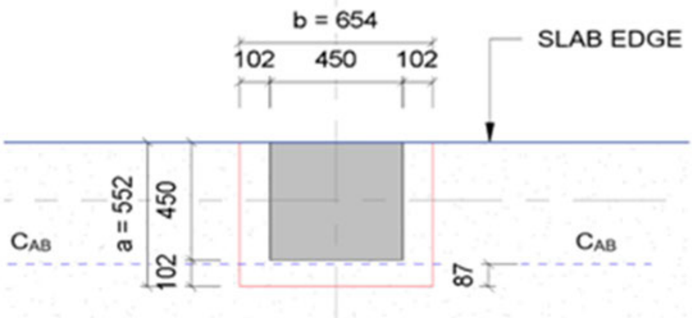
J_c = $d \cdot (2 \cdot a^3/3 - (2 \cdot a + b) \cdot (C_{AB})^2) + a \cdot d^3/6$

= 2.1E+10 mm^4

Where :

a = 552 mm

b = 654 mm



Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	ACI 318-2014 - Analysis		
Design Title:	Typical Edge Column	Designed By:	G. Mpai
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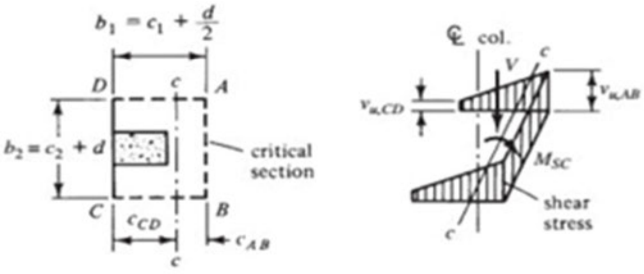
Reference

ACI 318-2014

Figure R8.4.4.2.3



(b) Edge column

Gravity load shear to be transferred:

V_u = 233.2 kN

Combined stresses:

V_u = $V_u/A_c + \gamma v \cdot 0.3 \cdot M_{ol} \cdot C_{AB}/J_C$ — R8.4.4.2.3

= 0.65 + 0.14

= 0.79 MPa

V_u should not be greater ϕv_c

ϕv_c = $0.75 \cdot v_c$ — Table 21.2.1

= 1.40 MPa

Where :

v_c is the smaller of: — 22.6.5.2

$0.33 \cdot \lambda \cdot \sqrt{f'c}$ = 1.87 MPa — Table 22.6.5.2

$0.17 \cdot (1 + 2/\beta) \cdot \lambda \cdot \sqrt{f'c}$ = 2.88 MPa

$0.083 \cdot (2 + \alpha_s \cdot d / b_o) \cdot \lambda \cdot \sqrt{f'c}$ = 9.50 MPa

$\therefore V_u$ < ϕv_c

UR = 0.57

Slab has adequate punching capacity

Typical Corner Location

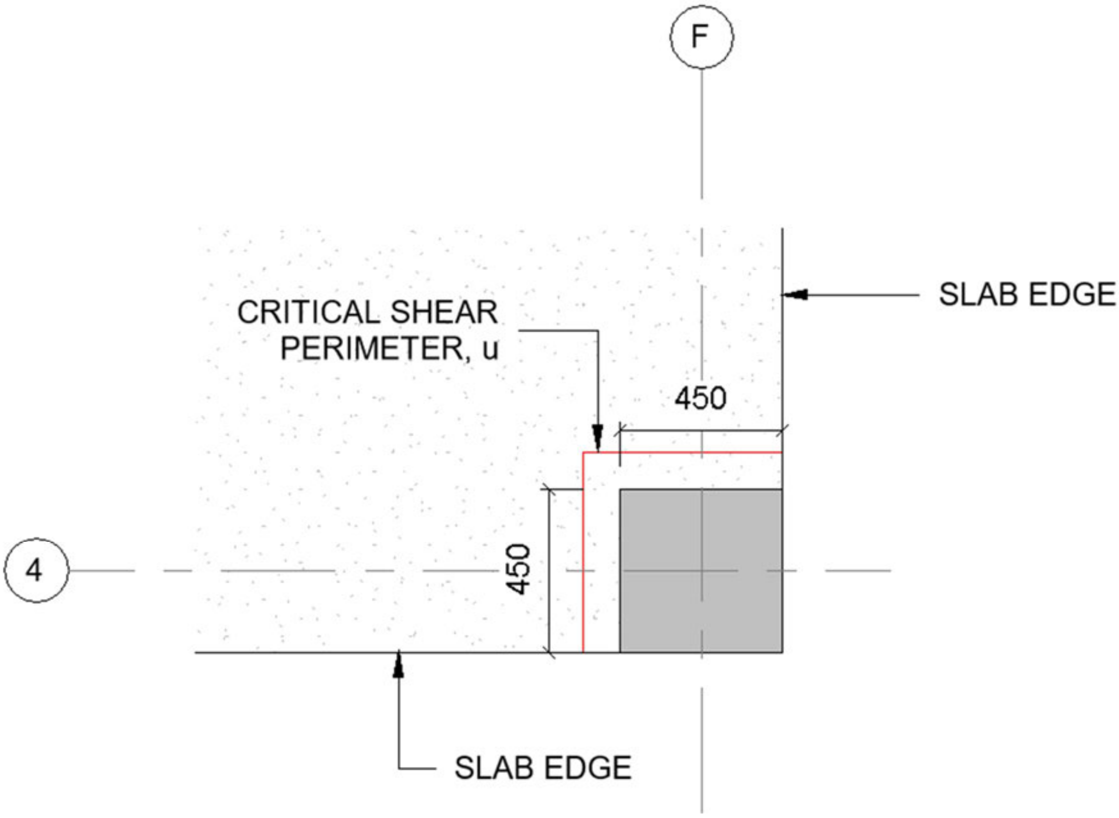


Figure 17: ACI 318 - Typical corner column location

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600			UniSQ
Design Title:	ACI 318-2014 - Analysis			
Design Title:	Typical Corner Column	Designed By:	G. Mpai	
Sheet Number:	1 of 3	Design Date:	2023/09/02	

INPUT DATA

Materials:

Reference
ACI 318-2014

f'_c = 32 MPa

f_y = 500 MPa

$\rho_{concrete}$ = 2500 kg/m^3

= 24.5 kN/m^3

Slab depth, h = 250 mm

Concrete cover = 30 mm

Column dimensions, A = 450 mm

B = 450 mm

Opening dimensions, D = 200 mm

E = 200 mm

Tributary Area = 6m/2 · 6m/2

= 9.0 m^2

Provided slab tension reinforcement

x-direction: dia = 16 mm (Assumed reinforcement)

spacing = 200 mm

= 1005 mm^2/m

y-direction: dia = 16 mm

spacing = 200 mm

= 1005 mm^2/m

Effective depth, d = 204 mm

Loading:

Slab DL = $\rho_{concrete} \cdot h$ = 6.13 kPa

SDL = 2.00 kPa

Live Load = 2.00 kPa

q_u = 1.2·(Slab DL + SDL) + 1.6·LL

= 12.96 kPa

V_u = q_u · Tributary Area

= 116.6 kN

RESULTS

0.5·d perimeter at = 102 mm

Determine M_{ol}

M_{ol} = $q_u \times l_2 \times l_n^2/8$ (Equation 8.10.3.2)

= 299 kN.m

Where :

l_2 = 6.00 m

l_n = 6.00 - A/2 -B/2

= 5.55 m

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	ACI 318-2014 - Analysis		
Design Title:	Typical Corner Column	Designed By:	G. Mpai
Sheet Number:	2 of 3	Design Date:	2023/09/02

Reference

Determine the unbalanced moment in each direction: — ACI 318-2014 Table 8.10.4.2

$M_u = 0.26 \cdot M_{ol\ endspan}$

$= 77.8$ kN.m

100% of the load is to be resisted by the column strip

Moment transfer design:

Total moment to be transferred

$0.3 \cdot M_{ol} = 89.8$ kN.m — 8.10.4.6

Fraction of unbalanced moment carried by eccentricity of shear

$\gamma_f = 1/(1+(2/3)\sqrt{(b1/b2)})$ — Equation 8.4.2.3.2

$= 0.60$

$b_1 = A + 0.5 \cdot d = 552$ mm

$b_2 = B + 0.5 \cdot d = 552$ mm

$\gamma_v = 1 - \gamma_f$ — Equation 8.4.4.2.2

$= 0.40$

Properties of critical section for shear:

$A_c = (a + b) \cdot d$

$= 2E{+}05$ mm^2

$C_{AB} = (a \cdot d \cdot a/2)/A_c$

$= 69.0$ mm

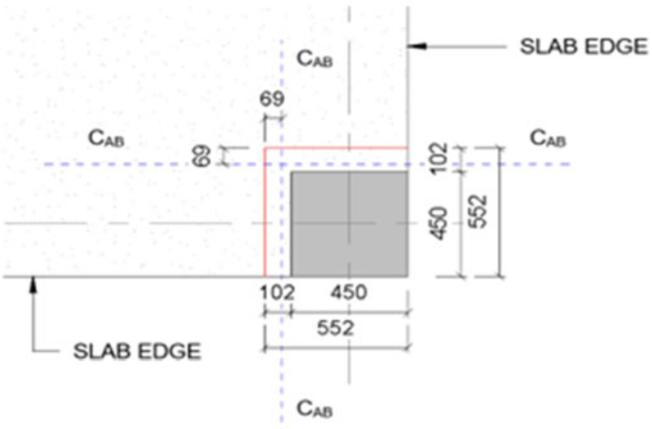
$J_c = d \cdot (2 \cdot a^3/3 - (2 \cdot a + b) \times (C_{AB})^2) + a \cdot d^3/6$

$= 2E{+}10$ mm^4

Where :

a = 552 mm

b = 552 mm



Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	ACI 318-2014 - Analysis		
Design Title:	Typical Corner Column	Designed By:	G. Mpai
Sheet Number:	3 of 3	Design Date:	2023/09/02

Reference

ACI 318-2014

Gravity load shear to be transferred:

V_u = 116.6 kN

Combined stresses:

v_u = $V_u/A_c + \gamma v \times 0.3\ M_{di} \times C_{AB}/J_c$ — R8.4.4.2.3

= 0.518 + 0.11

= 0.630 MPa

v_u should not be greater ϕv_c

ϕv_c = $0.75 \times v_c$ — Table 21.2.1

= 1.400 MPa

Where:

v_c is the smaller of: — 22.6.5.2

$0.33 \times \lambda \times \sqrt{f'c}$ = 1.87 — Table 22.6.5.2

$0.17\,(1 + 2/\beta)\,\lambda\,\sqrt{f'c}$ = 2.885

$0.083\,(2 + \alpha_s\,d / b_o)\,\lambda\,\sqrt{f'c}$ = 9.501

$\therefore V_u$ < ϕv_c

UR = 0.45

Slab has adequate punching capacity

3.3.2 EN 1992-1-1:2004

Typical Internal Column Location

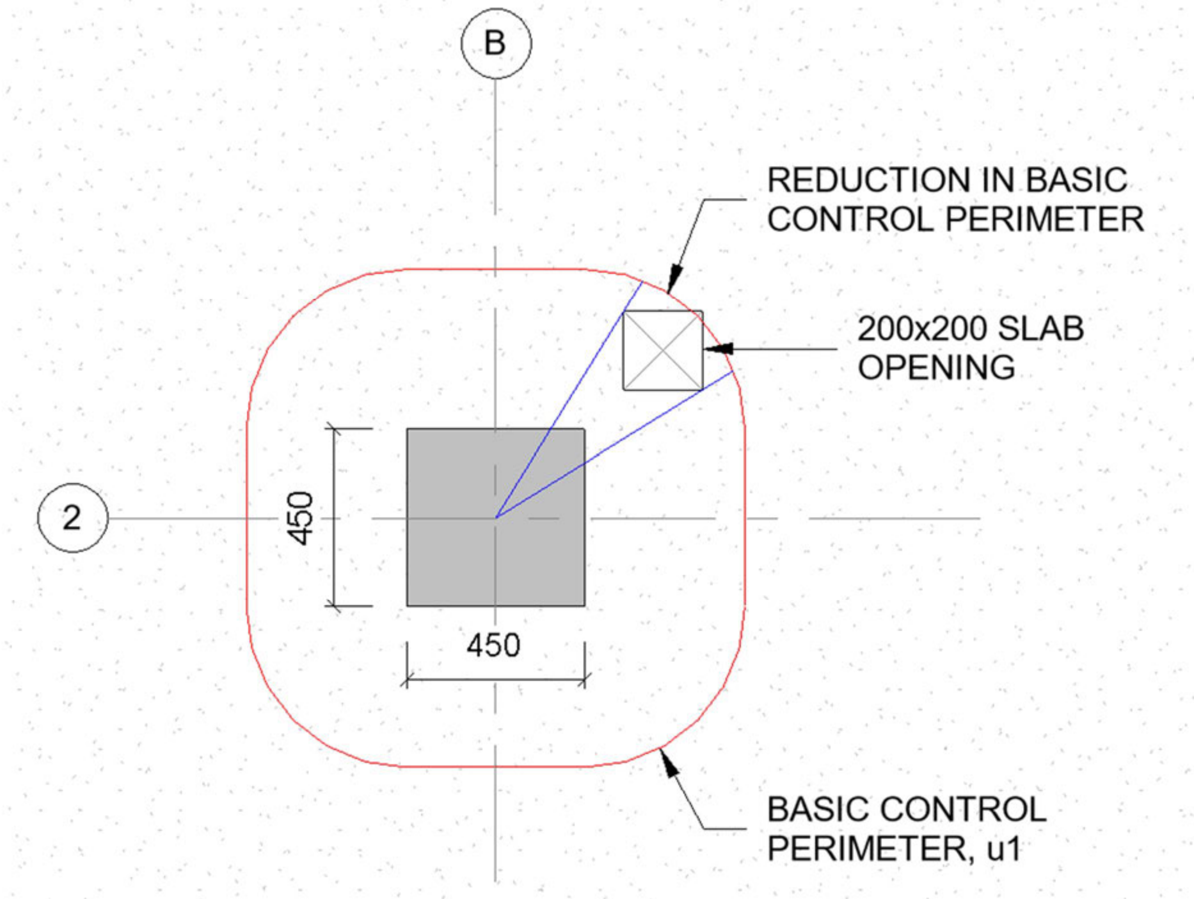


Figure 18: EN 1992- Typical internal column location

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600			
Design Title:	Eurocode 2 - Analysis			
Design Title:	Typical Internal Column		Designed By:	G. Mpai
Sheet Number:	1 of 2		Design Date:	2023/06/30

UniSQ

INPUT DATA					**Reference**
Materials:					BS EN 1992-1-1-2004
f_{ck}		=	32	MPa	
f_{yk}		=	500	MPa	
γ_c		=	1.5		
$\rho_{concrete}$		=	2500	kg/m^3	
		=	24.5	kN/m^3	
Slab depth, h		=	250	mm	
Concrete cover		=	30	mm	
Column dimensions,	A	=	450	mm	
	B	=	450	mm	
Opening dimensions,	D	=	200	mm	
	E	=	200	mm	
Tributary Area		=	6m·6m		
		=	36	m^2	
Beta factor, β		=	1.15		6.4.3.(6)
Provided slab tension reinforcement					
x-direction:	dia	=	16	mm	(Assumed reinforcement)
	spacing	=	200	mm	
		=	1005	mm^2/m	
y-direction:	dia	=	16	mm	
	spacing	=	200	mm	
		=	1005	mm^2/m	
Effective depth, d		=	204	mm	6.4.2
Loading:					
Slab DL		=	$\rho_{concrete} \cdot h$		
		=	6.13	kPa	
Super-imposed DL, SDL		=	2.00	kPa	
Live Load, LL		=	2.00	kPa	
Ultimate UDL		=	1.35·(Slab DL + SDL) + 1.5·LL		
		=	13.98	kPa	
V_{Ed}		=	Tributary Area · UDL		
		=	503.2	kPa	
$\beta \cdot V_{Ed}$		=	578.7	kPa	

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	Eurocode 2 - Analysis		
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UniSQ

				Reference
RESULTS				BS EN 1992-1-1-2004
2d perimeter located at	=	408	mm	6.4.2
u_0	=	$2 \cdot A + 2 \cdot B$		
	=	1800	mm	
u_1	=	$2 \cdot A + 2 \cdot B + 2 \cdot \pi \cdot r$		6.4.2.(1)
	=	4364	mm	
Hole reduction	=	335	mm	6.4.2.(3)
$u_{1,\,eff}$	=	4029	mm	
At column face:				
V_{Ed}	=	$\beta \cdot V_{Ed}/u_0 \cdot d$		
	=	1.58	MPa	
$V_{Rd,max}$	=	$0.5 \cdot v \cdot f_{cd}$		6.4.5.(3)
	=	5.58	MPa	
Where:				
v	=	$0.6 \cdot (1 - f_{ck}/25)$		Equation 6.6N
	=	0.52		
f_{cd}	=	f_{ck}/γ_c		
	=	21.33	MPa	
$V_{ed}/V_{rd,c\ max}$	=	0.28		
Slab has adequate face shear punching capacity				
At basic control perimeter:				
V_{Ed}	=	$\beta \cdot V_{Ed}/u_1 \cdot d$		Equation 6.38
	=	0.70	MPa	
Vrd,c				
$V_{Rd,c}$	=	$C_{Rd,c} \cdot k \cdot (F_1 \cdot f_{ck})^{1/3} \geq v_{min}$		Equation 6.47
	=	0.60	MPa	
Where:				
$C_{Rd,c,sc}$	=	0.12		
k	=	$1 + \sqrt{200/d}$		
	=	1.99		
F_1	=	$100 \cdot A_s$		
	=	$b_w \cdot d$		
	=	0.493		
v_{min}	=	$0.035 \cdot k^{3/2} \cdot fck^{1/2}$		
	=	0.56	MPa	
$V_{ed}/V_{Rd,c}$	=	1.18		
Slab has inadequate punching capacity				
A redesign of the slab is required				

Typical Edge Location

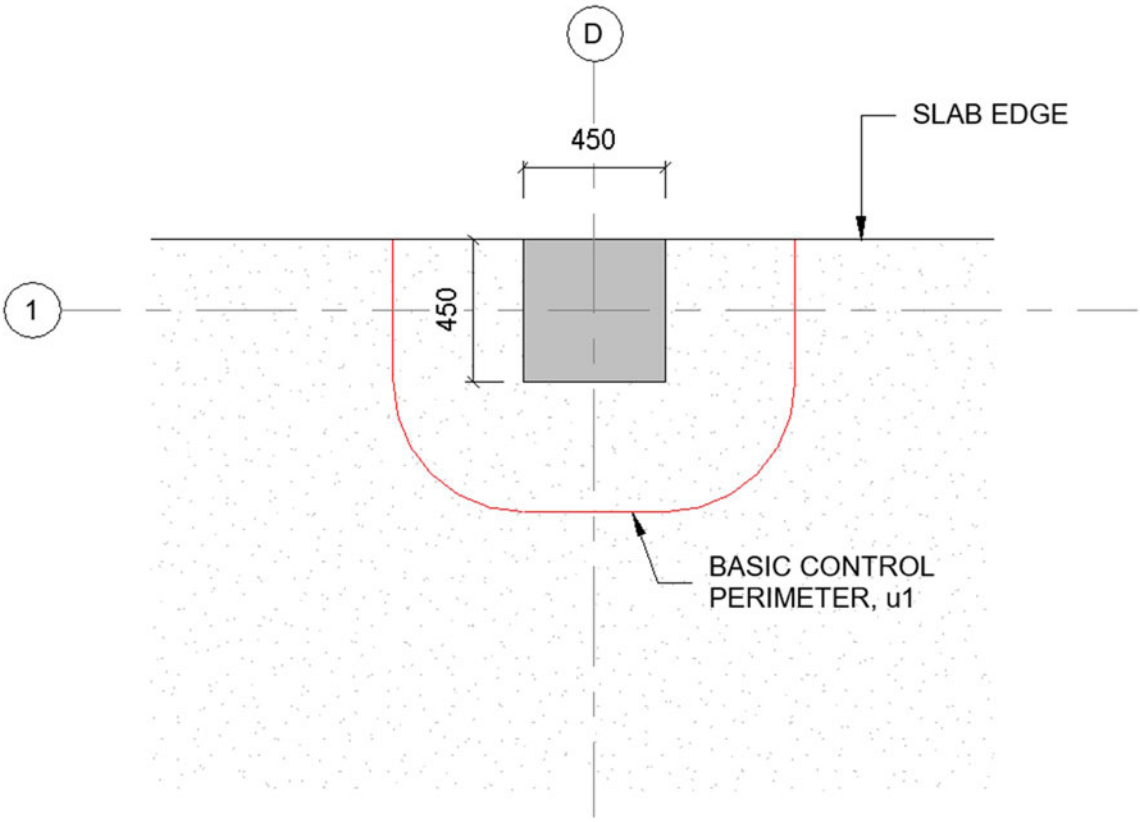

Figure 19: EN 1992 - Typical edge column location

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	Eurocode 2 - Analysis		
Design Title:	Typical Edge Column	Designed By:	G. Mpai
Sheet Number:	1 of 2	Design Date:	2023/06/30

INPUT DATA

Materials:

					Reference
f_{ck}		=	32	MPa	BS EN 1992-1-1-2004
f_{yk}		=	500	MPa	
γ_c		=	1.5		
$\rho_{concrete}$		=	2500	kg/m^3	
		=	24.5	kN/m^3	
Slab depth, h		=	250	mm	
Concrete cover		=	30	mm	
Column dimensions,	A	=	450	mm	
	B	=	450	mm	
Opening dimensions,	D	=	200	mm	
	E	=	200	mm	
Tributary Area		=	6m·6m/2		
		=	18	m^2	
Beta factor, β		=	1.4		6.4.3.(6)

Provided slab tension reinforcement

x-direction:	dia	=	16	mm	(Assumed reinforcement)
	spacing	=	200	mm	
		=	1005	mm^2/m	
y-direction:	dia	=	16	mm	
	spacing	=	200	mm	
		=	1005	mm^2/m	
Effective depth, d		=	204	mm	6.4.2

Loading:

Slab DL	=	$\rho_{concrete} \cdot h$		
	=	6.13	kPa	
Super-imposed DL, SDL	=	2.00	kPa	
Live Load, LL	=	2.00	kPa	
Ultimate UDL	=	$1.35 \cdot (\text{Slab DL} + \text{SDL}) + 1.5 \cdot \text{LL}$		
	=	13.98	kPa	
V_{Ed}	=	Tributary Area · UDL		
	=	251.6	kPa	
$\beta \cdot V_{Ed}$	=	352.2	kPa	

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	Eurocode 2 - Analysis		
Design Title:	Typical Edge Column	Designed By:	G. Mpai
Sheet Number:	2 of 2	Design Date:	2023/06/30

				Reference
RESULTS				BS EN 1992-1-1-2004
2d perimeter located at	=	408	mm	6.4.2
u_0	=	$2 \cdot A + B$		
	=	1350	mm	
u_1	=	$2 \cdot A + B + 2 \cdot (2 \cdot \pi \cdot r/4)$		6.4.2.(1)
	=	2632	mm	
Hole reduction	=	0	mm	6.4.2.(3)
$u_{1, eff}$	=	2632	mm	
At column face:				
V_{Ed}	=	$\beta \cdot V_{Ed}/u_0 \cdot d$		
	=	1.28	MPa	
$V_{Rd,max}$	=	$0.5 \cdot v \cdot f_{cd}$		6.4.5.(3)
	=	5.58	MPa	
Where:				
v	=	$0.6 \cdot (1 - f_{ck}/25)$		Equation 6.6N
	=	0.52		
f_{cd}	=	f_{ck}/γ_c		
	=	21.33	MPa	
$V_{ed}/V_{rd,c\ max}$	=	0.23		
Slab has adequate face shear punching capacity				
At basic control perimeter:				
V_{Ed}	=	$\beta \cdot V_{Ed}/u_1 \cdot d$		Equation 6.38
	=	0.66	MPa	
Vrd,c				
$V_{Rd,c}$	=	$C_{Rd,c} \cdot k \cdot (F_1 \cdot f_{ck})^{1/3} \geq v_{min}$		Equation 6.47
	=	0.60	MPa	
Where:				
$C_{Rd,c,sc}$	=	0.12		
k	=	$1 + \sqrt{200/d}$		
	=	1.99		
F_1	=	$100 \cdot A_s$		
	=	$b_w \cdot d$		
	=	0.493		
v_{min}	=	$0.035 \cdot k^{3/2} \cdot fck^{1/2}$		
	=	0.56	MPa	
$V_{ed}/V_{Rd,c}$	=	1.10		
Slab has inadequate punching capacity				
A redesign of the slab is required				

Typical Corner Location

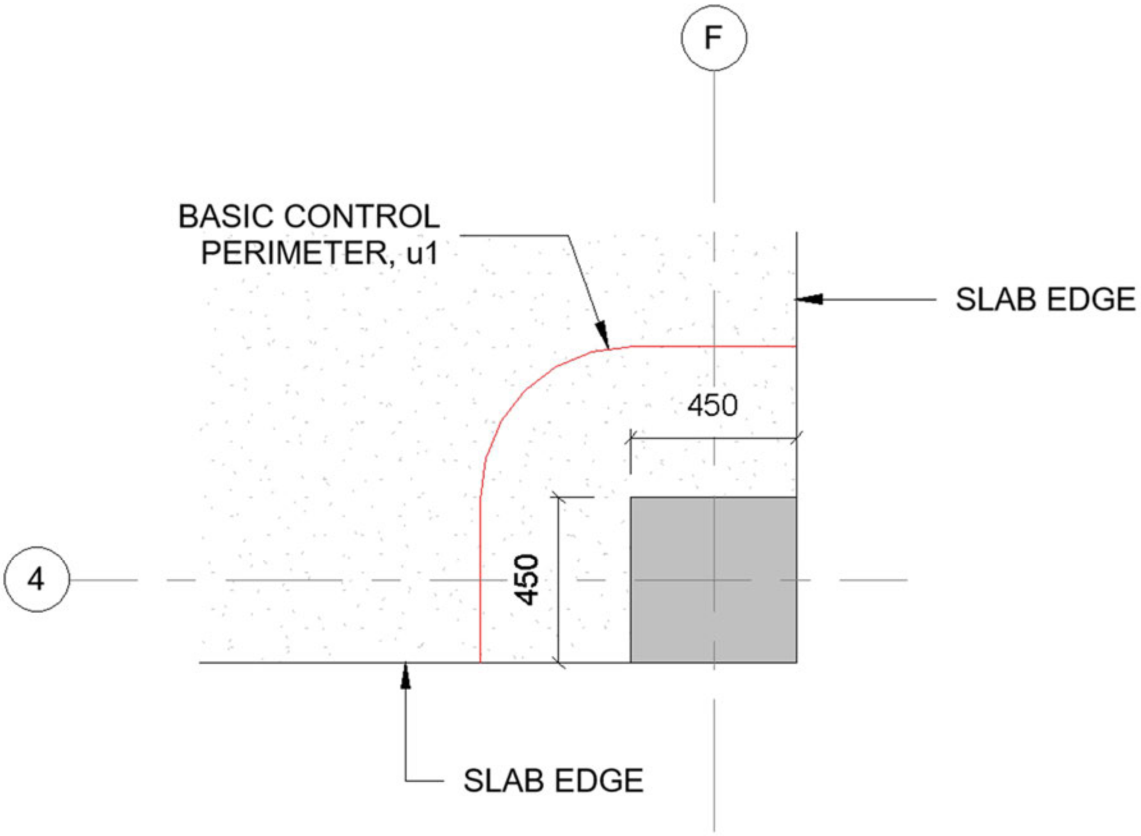

Figure 20: EN 1992 - Typical corner column location

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600			
Design Title:	Eurocode 2 - Analysis			
Design Title:	Typical Corner Column	Designed By:	G. Mpai	
Sheet Number:	1 of 2	Design Date:	2023/06/30	

INPUT DATA

Materials:

					Reference
f_{ck}		=	32	MPa	BS EN 1992-1-1-2004
f_{yk}		=	500	MPa	
γ_c		=	1.5		
$\rho_{concrete}$		=	2500	kg/m^3	
		=	24.5	kN/m^3	
Slab depth, h		=	250	mm	
Concrete cover		=	30	mm	
Column dimensions,	A	=	450	mm	
	B	=	450	mm	
Opening dimensions,	D	=	200	mm	
	E	=	200	mm	
Tributary Area		=	$6m/2 \cdot 6m/2$		
		=	9	m^2	
Beta factor, β		=	1.5		6.4.3.(6)

Provided slab tension reinforcement

x-direction:	dia	=	16	mm	(Assumed reinforcement)
	spacing	=	200	mm	
		=	1005	mm^2/m	
y-direction:	dia	=	16	mm	
	spacing	=	200	mm	
		=	1005	mm^2/m	
Effective depth, d		=	204	mm	6.4.2

Loading:

Slab DL	=	$\rho_{concrete} \cdot h$		
	=	6.13	kPa	
Super-imposed DL, SDL	=	2.00	kPa	
Live Load, LL	=	2.00	kPa	
Ultimate UDL	=	$1.35 \cdot (\text{Slab DL} + \text{SDL}) + 1.5 \cdot \text{LL}$		
	=	13.98	kPa	
V_{Ed}	=	Tributary Area $\cdot$ UDL		
	=	125.8	kPa	
$\beta \cdot V_{Ed}$	=	188.7	kPa	

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		UniSQ
Design Title:	Eurocode 2 - Analysis		
Design Title:	Typical Corner Column	Designed By:	G. Mpai
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			Reference
RESULTS			BS EN 1992-1-1-2004
2d perimeter located at	= 408	mm	6.4.2
u_0	= A+B		
	= 900	mm	
u_1	= $A+B+2\cdot\pi\cdot r/4$		6.4.2.(1)
	= 1541	mm	
Hole reduction	= 0	mm	6.4.2.(3)
$u_{1,\ eff}$	= 1541	mm	
At column face:			
V_{Ed}	= $\beta\cdot V_{Ed}/u_0\cdot d$		
	= 1.03	MPa	
$V_{Rd,max}$	= $0.5\cdot v\cdot f_{cd}$		6.4.5.(3)
	= 5.58	MPa	
Where:			
v	= $0.6\cdot(1-f_{ck}/25)$		Equation 6.6N
	= 0.52		
f_{cd}	= f_{ck}/γ_c		
	= 21.33	MPa	
$V_{ed}/V_{rd,c\ max}$	= 0.18		
Slab has adequate face shear punching capacity			
At basic control perimeter:			
V_{Ed}	= $\beta\cdot V_{Ed}/u_1\cdot d$		Equation 6.38
	= 0.60	MPa	
Vrd,c			
$V_{Rd,c}$	= $C_{Rd,c}\cdot k\cdot(F_1\cdot f_{ck})^{1/3} \geq v_{min}$		Equation 6.47
	= 0.60	MPa	
Where:			
$C_{Rd,c,sc}$	= 0.12		
k	= $1+\sqrt{200/d}$		
	= 1.99		
F_1	= $100\cdot A_s$		
	= $b_w\cdot d$		
	= 0.493		
v_{min}	= $0.035\cdot k^{3/2}\cdot fck^{1/2}$		
	= 0.56	MPa	
$V_{ed}/V_{Rd,c}$	= 1.00		
Slab has inadequate punching capacity			
A redesign of the slab is required			

3.3.3 AS 3600-2018

Typical Internal Location

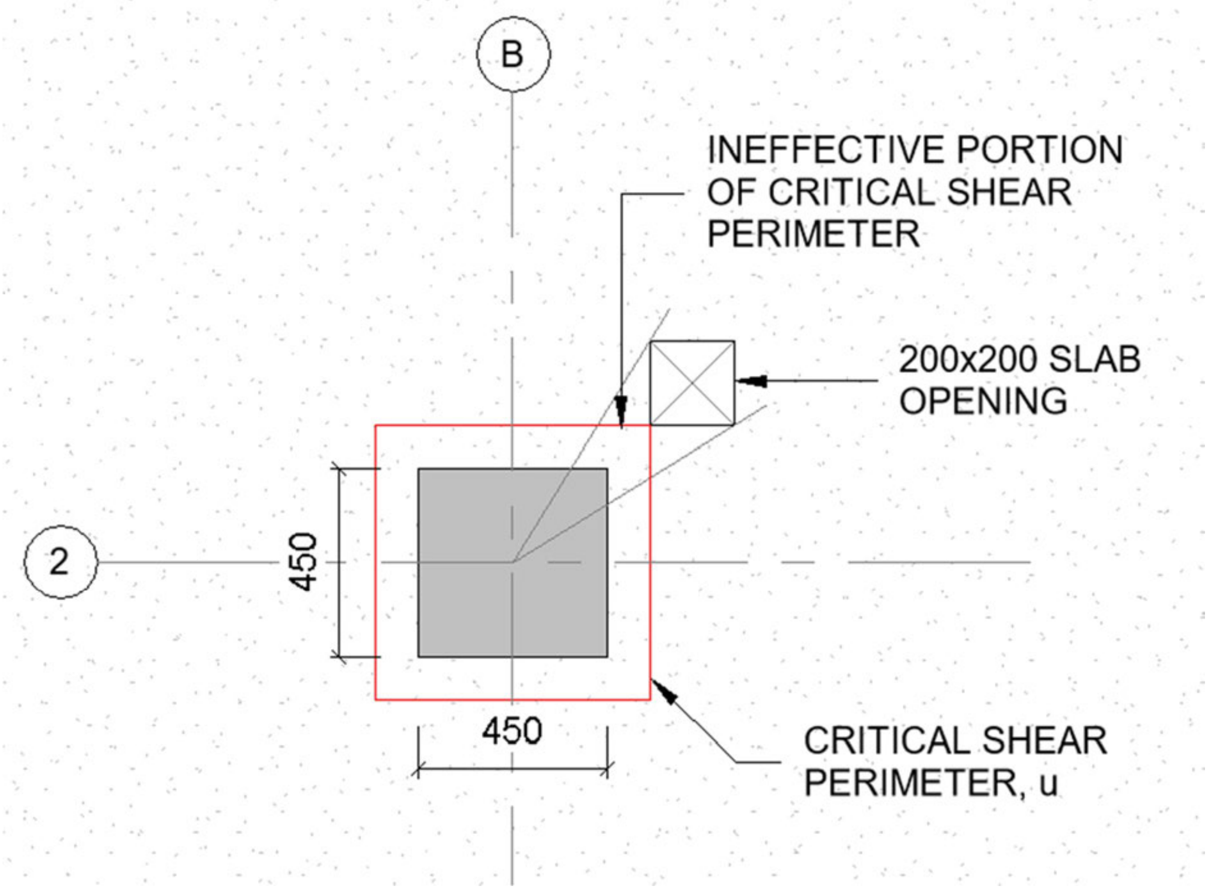

Figure 21: AS 3600 - Typical internal column location

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	AS 3600-2018 - Analysis		
Design Title:	Typical Internal Column	Designed By:	G. Mpai
Sheet Number:	1 of 2	Design Date:	2023/07/30

INPUT DATA

Materials:

					Reference
					AS 3600-2018
f'_c		=	32	MPa	
f_{sy}		=	500	MPa	
$\rho_{concrete}$		=	2500	kg/m^3	
		=	24.5	kN/m^3	
Slab depth, h		=	250	mm	
Concrete cover		=	30	mm	
Column dimensions,	A	=	450	mm	
	B	=	450	mm	
Opening dimensions,	D	=	200	mm	
	E	=	200	mm	
Tributary Area		=	6m· 6m		
		=	36	m^2	

Provided slab tension reinforcement

x-direction:	dia	=	16	mm	(Assumed reinforcement)
	spacing	=	200	mm	
		=	1005	mm^2/m	
y-direction:	dia	=	16	mm	
	spacing	=	200	mm	
		=	1005	mm^2/m	
Effective depth, d_{om}		=	204	mm	

Loading:

Slab DL	=	$\rho_{concrete} \cdot h$	
	=	6.13	kPa
Super-imposed DL, SDL	=	2.00	kPa
Live Load, LL	=	2.00	kPa
F_d	=	1.2·(Slab DL + SDL) + 1.5·LL	
	=	12.76	kPa
V*	=	Tributary Area · UDL	
	=	459.3	kPa

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		UniSQ
Design Title:	AS 3600-2018 - Analysis		
Design Title:	Typical Internal Column	Designed By:	G. Mpai
Sheet Number:	2 of 2	Design Date:	2023/07/30

		Reference
RESULTS		AS 3600-2018
$0.5\cdot d_{om}$ perimeter at	= 102 mm	9.3.1.3
u	$= 2\cdot(A+2\cdot 0.5\cdot d_{om})+2\cdot(B+2\cdot 0.5\cdot d_{om})$	
	= 2616 mm	9.3.1.2
Hole reduction	= 249 mm	
$u_{,eff}$	= 2367 mm	
At critical shear perimeter:		
Concrete shear strength:		
$f_{cv} = 0.17\cdot(1+2/\beta_h)\sqrt{f'c} \leq 0.34\sqrt{f'c}$		9.3.3.(a)
	= 1.92 MPa	
Where:		
β_h	= 450/450	
	= 1.00	
Ultimate shear strength of slab with no moment transfer:		
$V_{uo} = u\cdot d_{om}\cdot f_{cv}$	= 928.7 kN	9.3.3(1)
Ultimate shear strength of slab modified for moment transfer in major/minor direction:		
$V_u = V_{uo}/(1.0 + u\cdot M_v^*/(8\cdot V^*\cdot a\cdot d_{om}))$		9.3.4(1)
	= 808 kN	
ϕV_u	$= 0.7\cdot V_u$	Table 2.2.2
	= 565.6 kN	
Where:		
a	$= A + d_{om}$ = 654 mm	
M^*_v	= Out of balance moment between the two adjacent faces of the interior column	
	$= 0.75\cdot M_{o\ end\ span} - 0.65\cdot M_{o\ internal\ span}$	Based on the **simplified method**
	= 30.92 kN.m	0.75 coefficient - Table 6.10.4.3 (A)
(Assume that 100% of the load is to be resisted by the column strip)		0.65 coefficient - Table 6.10.4.3 (B)
$M_{o\ end\ span}$	$= F_d\cdot L_t\cdot L_o^2/8$	6.10.4.2
	= 309.2 kN.m	
L_o	$= 6m - 0.7\cdot 0.45/2 - 0.7\cdot 0.45/2$	
	= 5.69 m	
L_t	= 6.00 m	
$M_{o\ internal\ span}$	$= F_d\cdot L_t\cdot L_o^2/8$	6.10.4.2
	= 309.2 kN.m	
$\therefore \phi V_u$	$> V^*$	
UR	= 0.81	
Slab has adequate punching capacity		

Typical Edge Location

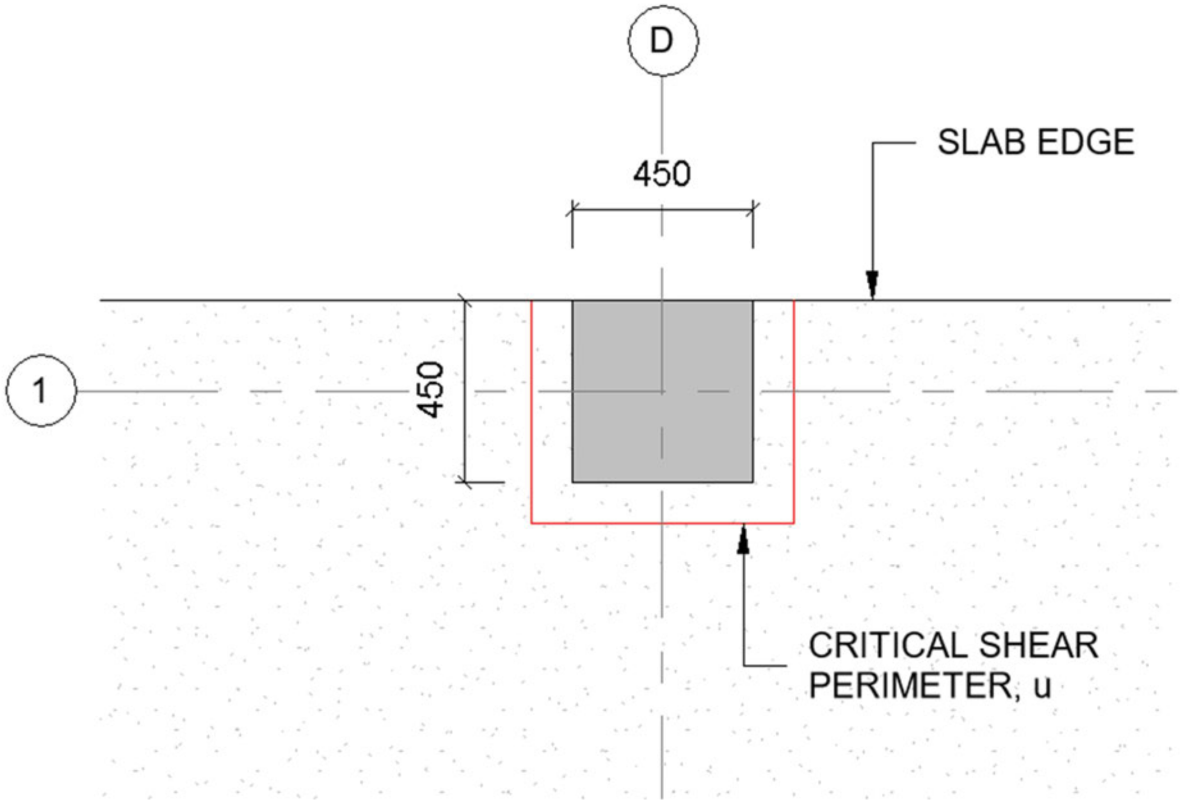

Figure 22: AS 3600 - Typical edge column location

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	AS 3600-2018 - Analysis		
Design Title:	Typical Edge Column	Designed By:	G. Mpai
Sheet Number:	1 of 2	Design Date:	2023/07/30

UniSQ

INPUT DATA

Materials:

Reference

AS 3600-2018

f'_c		=	32	MPa
f_{sy}		=	500	MPa
$\rho_{concrete}$		=	2500	kg/m^3
		=	24.5	kN/m^3
Slab depth, h		=	250	mm
Concrete cover		=	30	mm
Column dimensions,	A	=	450	mm
	B	=	450	mm
Tributary Area		=	6m·6m/2	
		=	18	m^2

Provided slab tension reinforcement (Assumed reinforcement)

x-direction:	dia	=	16	mm
	spacing	=	200	mm
		=	1005	mm^2/m
y-direction:	dia	=	16	mm
	spacing	=	200	mm
		=	1005	mm^2/m
Effective depth, d_{om}		=	204	mm

Loading:

Slab DL	=	$\rho_{concrete} \cdot h$	
	=	6.13	kPa
Super-imposed DL, SDL	=	2.00	kPa
Live Load, LL	=	2.00	kPa
F_d	=	1.2·(Slab DL + SDL) + 1.5·LL	
	=	12.76	kPa
V*	=	Tributary Area · UDL	
	=	229.6	kPa

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	AS 3600-2018 - Analysis		
Design Title:	Typical Edge Column	Designed By:	G. Mpai
Sheet Number:	2 of 2	Design Date:	2023/07/30

		Reference
RESULTS		AS 3600-2018
$0.5 \cdot d_{om}$ perimeter at	= 102 mm	9.3.1.3
u	$= 2 \cdot (A+0.5 \cdot d_{om})+(B+2 \cdot 0.5 \cdot d_{om})$	
	= 1758 mm	
Hole reduction	= 0 mm	9.3.1.2
$u_{,eff}$	= 1758 mm	
At critical shear perimeter:		
Concrete shear strength:		
$f_{cv} = 0.17 \cdot (1+2/\beta_h)\sqrt{f'c} \leq 0.34\sqrt{f'c}$		9.3.3.(a)
	= 1.92 MPa	
Where:		
β_h	= 450/450	
	= 1.00	
Ultimate shear strength of slab with no moment transfer:		
$V_{uo} = u \cdot d_{om} \cdot f_{cv}$	= 689.8 kN	9.3.3(1)
Ultimate shear strength of slab modified for moment transfer in major direction:		
$V_u = V_{uo}/(1.0 + u \cdot M_v^*/(8 \cdot V^* \cdot a \cdot d_{om}))$		9.3.4(1)
	= 444 kN	
ϕV_u	$= 0.7 \cdot V_u$	Table 2.2.2
	= 310.6 kN	
Where:		
a	= = 654 mm	
$M^*_{v,major}$	= Out of balance moment	
(Only the direction of the critical out of balance moment is considered)		
	$= 0.25 \times M_{o\ end\ span}$	Based on the **simplified method**
	= 77.31 kN.m	Table 6.10.4.3 (A)
(Assume that 100% of the load is to be resisted by the column strip)		
$M_{o\ end\ span}$	$= F_d \cdot L_t \cdot L_o^2/8$	6.10.4.2
	= 309.2 kN.m	
L_o	$= 6m - 0.7 \cdot 0.45/2 - 0.7 \cdot 0.45/2$	
	= 5.69	
L_t	= 6.00 m	
$\therefore \phi V_u$	$> V^*$	
UR	= 0.739	
Slab has adequate punching capacity		

Typical Corner Location

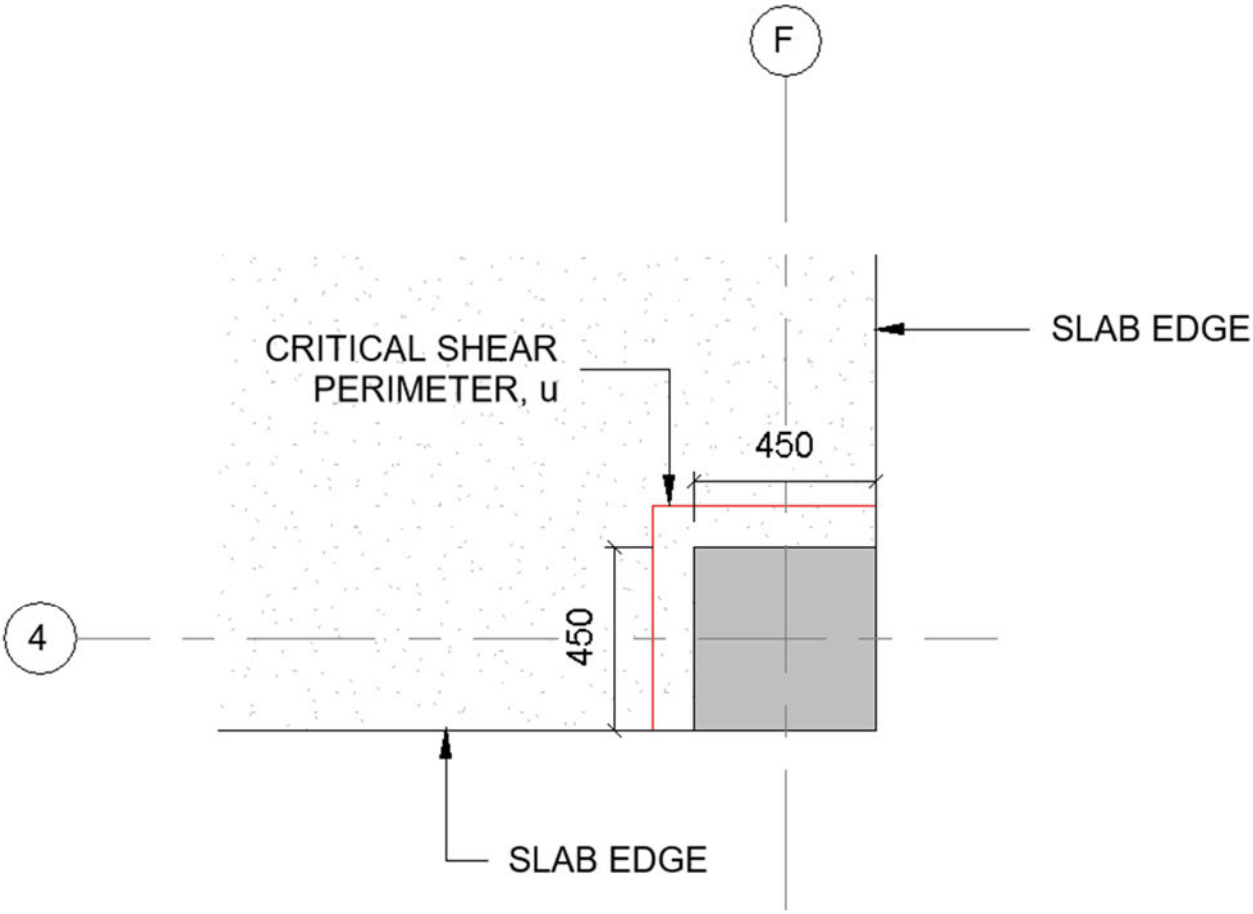

Figure 23: AS 3600 - Typical corner column location

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		
Design Title:	AS 3600-2018 - Analysis		
Design Title:	Typical Corner Column	Designed By:	G. Mpai
Sheet Number:	1 of 2	Design Date:	2023/07/30

INPUT DATA

Materials:

Reference: AS 3600-2018

f'_c = 32 MPa

f_{sy} = 500 MPa

$\rho_{concrete}$ = 2500 kg/m^3

= 24.5 kN/m^3

Slab depth, h = 250 mm

Concrete cover = 30 mm

Column dimensions, A = 450 mm

B = 450 mm

Tributary Area = 6m/2·6m/2

= 9 m^2

Provided slab tension reinforcement

x-direction: dia = 16 mm (Assumed reinforcement)

spacing = 200 mm

= 1005 mm^2/m

y-direction: dia = 16 mm

spacing = 200 mm

= 1005 mm^2/m

Effective depth, d_{om} = 204 mm

Loading:

Slab DL = 1.2·(Slab DL + SDL) + 1.5·LL

= 6.13 kPa

Super-imposed DL, SDL = 2.00 kPa

Live Load, LL = 2.00 kPa

F_d = 1.2·(Slab DL + SDL) + 1.5·LL

= 12.76 kPa

V* = Tributary Area · UDL

= 114.8 kPa

Project Name:	Punching Shear comparison between ACI 318, EC2 and AS 3600		UniSQ
Design Title:	AS 3600-2018 - Analysis		
Design Title:	Typical Corner Column	Designed By:	G. Mpai
Sheet Number:	2 of 2	Design Date:	2023/07/30

Reference
AS 3600-2018

RESULTS

$0.5 \cdot d_{om}$ perimeter at	= 102 mm		9.3.1.3
u	= $(A+0.5 \cdot d_{om})+(B+0.5 \cdot d_{om})$		
	= 1104 mm		
Hole reduction	= 0 mm		9.3.1.2
$u_{,eff}$	= 1104 mm		

<u>At critical shear perimeter:</u>

Concrete shear strength:

$f_{cv} = 0.17 \cdot (1+2/\beta_h)\sqrt{f'c} \leq 0.34\sqrt{f'c}$ — 9.3.3.(a)

= 1.92 MPa

Where:

β_h = 450/450

= 1.00

Ultimate shear strength of slab with no moment transfer:

$V_{uo} = u \cdot d_{om} \cdot f_{cv}$ = 433.2 kN — 9.3.3(1)

Ultimate shear strength of slab modified for moment transfer in major/minor direction:

$V_u = V_{uo}/(1.0 + u \cdot M_v^*/(8 \cdot V^* \cdot a \cdot d_{om}))$ — 9.3.4(1)

= 255 kN

ϕV_u = $0.7 \cdot V_u$ — Table 2.2.2

= 178.7 kN

Where :

a = $A + d_{om}$ = 654 mm

M^*_v = Out of balance moment in either direction

= $0.25 \times M_{o\ end\ span}$ — Based on the **simplified method**

= 77.31 kN.m — Table 6.10.4.3 (A)

$M_{o\ end\ span}$ = $F_d \cdot L_t \cdot L_o^2/8$ — 6.10.4.2

= 309.2 kN.m

L_o = $6m - 0.7 \cdot 0.45/2 - 0.7 \cdot 0.45/2$

= 5.69

L_t = 6.00 m

$M_{o\ end\ span}$ = $F_d \cdot L_t \cdot L_o^2/8$ — 6.10.4.2

= 309.2 kN.m

$\therefore \phi V_u$ > V^*

UR = 0.64

Slab has adequate punching capacity

3.4 Software Analysis

Tekla Structural Designer (TSD) was used for the software analysis of the slab. TSD is a building analysis and design software program that incorporates a Finite Element (FE) engine with automated FE meshing tools (Tekla, 2023). It is uniquely useful for this research project as its design functionality allows for checking punching shear directly in the program based on the requirements of many of the major design codes used around the world.

The software analysis is intended to validate and correlate the results from the hand calculation analysis, i.e., to ensure that the results obtained from the hand calculation analysis are reasonable and relatively accurate.

Three sets of models were created to incorporate the requirements of each of the codes under consideration. Each of the models incorporated the same material properties and the loading as applied to the hand calculation analysis.

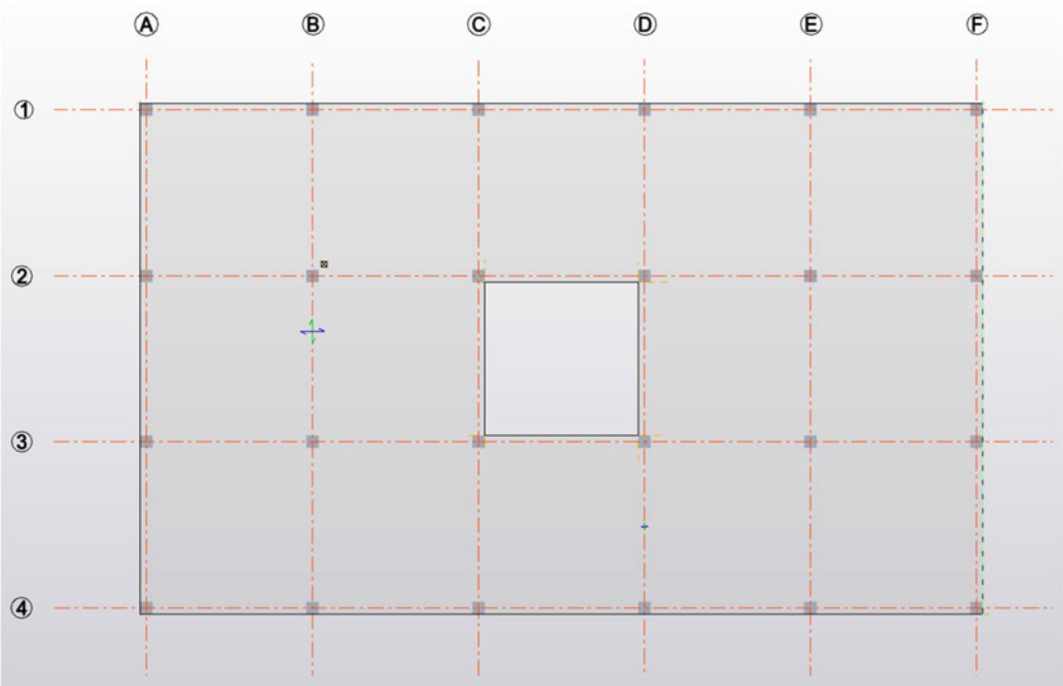

Figure 24: TSD Floor model - Layout

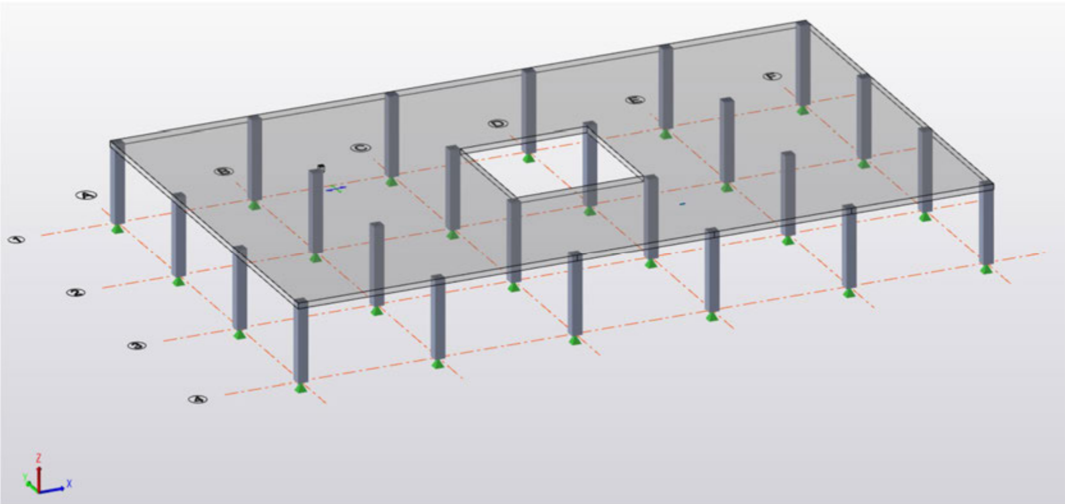

Figure 25: TSD Floor model – 3D view

The results obtained from each of the typical punching checks in each of the three models are presented in the sections to follow, these results are included as snapshots taken from TSD.

3.4.1 ACI 318-2014

Typical Internal Location

1-C8-PC1 results (ACI 318, 2014)

- Summary
- 1 $LRFD_2$-1.2D+1.6L
  - FE Chase Down
    - Critical Perimeter

1 $LRFD_2$-1.2D+1.6L - FE Chase Down - Critical Perimeter

Shear force	V_u = **541.4** kN	
Total adjustment	$\Delta V_u = L + Swt + R$ = **4.9** kN	
Adjusted perimeter shear force	$V_{u,red} = V_u - \Delta V_u$ = **536.5** kN	
Moment about major axis	M_{major} = **16.6** kNm	
Moment about minor axis	M_{minor} = **-10.4** kNm	
Closed perimeter breadth	B_{perim} = **654.0** mm	
Closed perimeter depth	D_{perim} = **654.0** mm	
Effective critical perimeter length	b_o = **2367.1** mm	
Effective depth to tension reinforcement in slab	d = **204.0** mm	
Concrete shear resistance	$v_c = MIN[v_{ca}, v_{cb}, v_{cc}]$ = **1.838** N/mm^2	
Shear stress at critical perimeter	$v_u = v_{u,max}$ = **1.291** N/mm^2	ACI 318-14 Section 8.4.4.2.3
Strength reduction factor	ϕ = **0.750**	
Unreinforced shear resistance	$\phi \times v_n = \phi \times v_c$ = **1.379** N/mm^2	
Shear stress utilization ratio		

$v_u \leq \phi \times v_n$: shear reinforcement not required

✔ Pass

Typical Edge Location

1-C4-PC3 results (ACI 318, 2014)

Summary
1 $LRFD_2$-1.2D+1.6L
FE Chase Down
Critical Perimeter

1 $LRFD_2$-1.2D+1.6L - FE Chase Down - Critical Perimeter

Shear force	V_u = **225.6** kN
Total adjustment	ΔV_u = L + Swt + R = **4.6** kN
Adjusted perimeter shear force	$V_{u,red} = V_u - \Delta V_u$ = **221.0** kN
Moment about major axis	M_{major} = **-96.8** kNm
Moment about minor axis	M_{minor} = **-2.3** kNm
Closed perimeter breadth	B_{perim} = **654.0** mm
Closed perimeter depth	D_{perim} = **552.0** mm
Effective critical perimeter length	b_o = **1758.0** mm
Effective depth to tension reinforcement in slab	d = **204.0** mm
Concrete shear resistance	$v_c = MIN[v_{ca}, v_{cb}, v_{cc}]$ = **1.838** N/mm^2
Shear stress at critical perimeter	$v_u = v_{u,max}$ = **0.627** N/mm^2 ACI 318-14 Section 8.4.4.2.3
Strength reduction factor	ϕ = **0.750**
Unreinforced shear resistance	$\phi \times v_n = \phi \times v_c$ = **1.379** N/mm^2
Shear stress utilization ratio	

$v_u \leq \phi \times v_n$: shear reinforcement not required

✔ Pass

Typical Corner Location

1-C24-PC2 results (ACI 318, 2014)

- Summary
- 1 $LRFD_2$-1.2D+1.6L
  - FE Chase Down
    - Critical Perimeter

1 $LRFD_2$-1.2D+1.6L - FE Chase Down - Critical Perimeter

Shear force	V_u = **108.0** kN	
Total adjustment	ΔV_u = L + Swt + R = **3.9** kN	
Adjusted perimeter shear force	$V_{u,red} = V_u - \Delta V_u$ = **104.2** kN	
Moment about major axis	M_{major} = **57.3** kNm	
Moment about minor axis	M_{minor} = **-56.6** kNm	
Closed perimeter breadth	B_{perim} = **552.0** mm	
Closed perimeter depth	D_{perim} = **552.0** mm	
Effective critical perimeter length	b_o = **1104.0** mm	
Effective depth to tension reinforcement in slab	d = **204.0** mm	
Concrete shear resistance	$v_c = MIN[v_{ca}, v_{cb}, v_{cc}]$ = **1.838** N/mm^2	
Shear stress at critical perimeter	$v_u = v_{u,max}$ = **0.463** N/mm^2	ACI 318-14 Section 8.4.4.2.3
Strength reduction factor	ϕ = **0.750**	
Unreinforced shear resistance	$\phi \times v_n = \phi \times v_c$ = **1.379** N/mm^2	
Shear stress utilization ratio		

$v_u \leq \phi \times v_n$: shear reinforcement not required

✔ Pass

3.4.2 EN 1992-1-1:2004

Typical Internal Location

1-C8-PC1 results (BS EN 1992-1-1 + UK NA, 2004)

- Summary
- 1 LC1
- 2 STR$_1$-1.35G+1.5Q+1.5RQ
  - FE Chase Down

2 STR$_1$-1.35G+1.5Q+1.5RQ - FE Chase Down

Shear force	$V_{Ed} = \mathbf{588.7}$ kN	
Total adjustment	$\Delta V_{Ed} = L + Swt + R = \mathbf{18.2}$ kN	
Adjusted perimeter shear force	$V_{Ed,red} = V_{Ed} - \Delta V_{Ed} = \mathbf{570.5}$ kN	
Moment about major axis	$M_{major} = \mathbf{18.3}$ kNm	
Moment about minor axis	$M_{minor} = \mathbf{-11.5}$ kNm	
Bounding rectangle breadth	$B_{bound} = \mathbf{450.0}$ mm	
Bounding rectangle depth	$D_{bound} = \mathbf{450.0}$ mm	
Equivalent rectangle breadth	$B_{equiv} = \mathbf{450.0}$ mm	
Equivalent rectangle depth	$D_{equiv} = \mathbf{450.0}$ mm	
Effective depth to tension reinforcement in slab	$d = \mathbf{204.0}$ mm	
Tension reinforcement ratio in slab resisting bending about major axis	$\rho_{l,major} = \mathbf{0.51}$ %	
Tension reinforcement ratio in slab resisting bending about minor axis	$\rho_{l,minor} = \mathbf{0.47}$ %	
Tension reinforcement ratio in slab	$\rho_l = MIN[\sqrt{\rho_{l,major} \times \rho_{l,minor}}, 2\%] = \mathbf{0.49}$ %	EN 1992-1-1:2004 Section 6.2.2(1)
Loaded perimeter	$u_0 = \mathbf{1628.9}$ mm	
First basic control perimeter	$u_1 = \mathbf{4026.2}$ mm	
Unreinforced punching shear resistance	$v_{Rd,c} = MAX[C_{Rd,c} \times k \times (100 \times \rho_l \times f_{ck})^{1/3}, v_{min}] = \mathbf{0.599}$ N/mm^2	EN 1992-1-1:2004 Section 6.4.4(1)
Magnification factor	$\beta = 1 + (k_{major} \times (\lvert M_{Ed,major}\rvert / \lvert V_{Ed,red}\rvert) \times (u_1 / W_{1,major})) + (k_{minor} \times (\lvert M_{Ed,minor}\rvert / \lvert V_{Ed,red}\rvert) \times (u_1 / W_{1,minor})) = \mathbf{1.073}$	EN 1992-1-1:2004 Section 6.4.3(3)
Shear stress at face of loaded perimeter	$v_{Ed,0} = \beta \times V_{Ed} / (u_0 \times d) = \mathbf{1.901}$ N/mm^2	
Maximum punching shear resistance	$v_{Rd,max} = 0.5 \times \nu \times f_{cd} = \mathbf{5.581}$ N/mm^2	EN 1992-1-1:2004 Section 6.4.5(3)
Shear stress utilization ratio	$v_{Ed,0} / v_{Rd,max} = \mathbf{0.341}$	
✔ Pass		
Shear stress at basic control perimeter	$v_{Ed,1} = \beta \times V_{Ed,red} / (u_1 \times d) = \mathbf{0.745}$ N/mm^2	
Unreinforced shear strength ratio	$v_{Ed,1} / v_{Rd,c} = \mathbf{1.244}$	
Steel design strength	$f_{ywd} = \mathbf{434.8}$ N/mm^2	
Effective shear reinforcement yield strength	$f_{ywd,ef} = MIN[250 + (0.25 \times d), f_{ywd}] = \mathbf{301.0}$ N/mm^2	EN 1992-1-1:2004 Section 6.4.5(1)
Shear reinforcement ratio required	$A_{sw} / s_r = (u_1 / (1.5 \times f_{ywd,ef})) \times (v_{Ed,1} - (0.75 \times v_{Rd,c})) = \mathbf{2638}$ mm^2/m	
Reinforcement area provided	$A_{v,provided} / s = \mathbf{0}$ mm^2/m	
UR not applicable because the area of reinforcement provided is null		
✖ Fail		

Typical Edge Location

1-C4-PC3 results (BS EN 1992-1-1 + UK NA, 2004)

Summary
1 LC1
2 STR_1-1.35G+1.5Q+1.5RQ
FE Chase Down

2 STR_1-1.35G+1.5Q+1.5RQ - FE Chase Down

Shear force	$V_{Ed} =$ **246.2** kN	
Total adjustment	$\Delta V_{Ed} = L + Swt + R =$ **13.9** kN	
Adjusted perimeter shear force	$V_{Ed,red} = V_{Ed} - \Delta V_{Ed} =$ **232.4** kN	
Moment about major axis	$M_{major} =$ **-108.2** kNm	
Moment about minor axis	$M_{minor} =$ **-2.5** kNm	
Bounding rectangle breadth	$B_{bound} =$ **450.0** mm	
Bounding rectangle depth	$D_{bound} =$ **450.0** mm	
Equivalent rectangle breadth	$B_{equiv} =$ **450.0** mm	
Equivalent rectangle depth	$D_{equiv} =$ **450.0** mm	
Effective depth to tension reinforcement in slab	$d =$ **204.0** mm	
Tension reinforcement ratio in slab resisting bending about major axis	$\rho_{l,major} =$ **0.51** %	
Tension reinforcement ratio in slab resisting bending about minor axis	$\rho_{l,minor} =$ **0.47** %	
Tension reinforcement ratio in slab	$\rho_l = MIN[\sqrt{\rho_{l,major} \times \rho_{l,minor}}, 2\%] =$ **0.49** %	EN 1992-1-1:2004 Section 6.2.2(1)
Loaded perimeter	$u_0 =$ **1350.0** mm	
Maximum loaded perimeter	$u_{0,max} = (3 \times d) + D_{equiv} =$ **1062.0** mm	EN 1992-1-1:2004 Section 6.4.5(3)
Loaded perimeter	$u_0 = MIN[u_0, u_{0,max}] =$ **1062.0** mm	
First basic control perimeter	$u_1 =$ **2631.8** mm	
Unreinforced punching shear resistance	$v_{Rd,c} = MAX[C_{Rd,c} \times k \times (100 \times \rho_l \times f_{ck})^{1/3}, v_{min}] =$ **0.599** N/mm^2	EN 1992-1-1:2004 Section 6.4.4(1)
Magnification factor	$\beta = MIN[\beta_A, \beta_B] =$ **1.410**	
Shear stress at face of loaded perimeter	$v_{Ed,0} = \beta \times V_{Ed} / (u_0 \times d) =$ **1.603** N/mm^2	
Maximum punching shear resistance	$v_{Rd,max} = 0.5 \times \nu \times f_{cd} =$ **5.581** N/mm^2	EN 1992-1-1:2004 Section 6.4.5(3)
Shear stress utilization ratio	$v_{Ed,0} / v_{Rd,max} =$ **0.287**	
✔ Pass		
Shear stress at basic control perimeter	$v_{Ed,1} = \beta \times V_{Ed,red} / (u_1 \times d) =$ **0.610** N/mm^2	
Unreinforced shear strength ratio	$v_{Ed,1} / v_{Rd,c} =$ **1.019**	
Steel design strength	$f_{ywd} =$ **434.8** N/mm^2	
Effective shear reinforcement yield strength	$f_{ywd,ef} = MIN[250 + (0.25 \times d), f_{ywd}] =$ **301.0** N/mm^2	EN 1992-1-1:2004 Section 6.4.5(1)
Shear reinforcement ratio required	$A_{sw} / s_r = (u_1 / (1.5 \times f_{ywd,ef})) \times (v_{Ed,1} - (0.75 \times v_{Rd,c})) =$ **939** mm^2/m	
Reinforcement area provided	$A_{v,provided} / s =$ **0** mm^2/m	
UR not applicable because the area of reinforcement provided is null		
✖ Fail		

Typical Corner Location

1-C24-PC2 results (BS EN 1992-1-1 + UK NA, 2004)

Summary
1 LC1
2 STR_1-1.35G+1.5Q+1.5RQ
FE Chase Down

2 STR_1-1.35G+1.5Q+1.5RQ - FE Chase Down

Shear force	V_{Ed} = **118.2** kN	
Total adjustment	$\Delta V_{Ed} = L + Swt + R$ = **9.6** kN	
Adjusted perimeter shear force	$V_{Ed,red} = V_{Ed} - \Delta V_{Ed}$ = **108.6** kN	
Moment about major axis	M_{major} = **63.6** kNm	
Moment about minor axis	M_{minor} = **-63.0** kNm	
Bounding rectangle breadth	B_{bound} = **450.0** mm	
Bounding rectangle depth	D_{bound} = **450.0** mm	
Equivalent rectangle breadth	B_{equiv} = **450.0** mm	
Equivalent rectangle depth	D_{equiv} = **450.0** mm	
Effective depth to tension reinforcement in slab	d = **204.0** mm	
Tension reinforcement ratio in slab resisting bending about major axis	$\rho_{l,major}$ = **0.51** %	
Tension reinforcement ratio in slab resisting bending about minor axis	$\rho_{l,minor}$ = **0.47** %	
Tension reinforcement ratio in slab	$\rho_l = MIN[\sqrt{\rho_{l,major} \times \rho_{l,minor}}, 2\%]$ = **0.49** %	EN 1992-1-1:2004 Section 6.2.2(1)
Loaded perimeter	u_0 = **900.0** mm	
Maximum loaded perimeter	$u_{0,max} = 3 \times d$ = **612.0** mm	EN 1992-1-1:2004 Section 6.4.5(3)
Loaded perimeter	$u_0 = MIN[u_0, u_{0,max}]$ = **612.0** mm	
First basic control perimeter	u_1 = **1540.9** mm	
Unreinforced punching shear resistance	$v_{Rd,c} = MAX[C_{Rd,c} \times k \times (100 \times \rho_l \times f_{ck})^{1/3}, v_{min}]$ = **0.599** N/mm²	EN 1992-1-1:2004 Section 6.4.4(1)
Magnification factor	$\beta = MIN[\beta_A, \beta_B]$ = **1.500**	
Shear stress at face of loaded perimeter	$v_{Ed,0} = \beta \times V_{Ed} / (u_0 \times d)$ = **1.420** N/mm²	
Maximum punching shear resistance	$v_{Rd,max} = 0.5 \times \nu \times f_{cd}$ = **5.581** N/mm²	EN 1992-1-1:2004 Section 6.4.5(3)
Shear stress utilization ratio	$v_{Ed,0} / v_{Rd,max}$ = **0.254**	
✔ Pass		
Shear stress at basic control perimeter	$v_{Ed,1} = \beta \times V_{Ed,red} / (u_1 \times d)$ = **0.518** N/mm²	
Unreinforced shear strength ratio	$v_{Ed,1} / v_{Rd,c}$ = **0.865**	
$v_{Ed,1} \leq v_{Rd,c}$		
✔ Pass		

3.4.3 AS 3600-2018

Typical Internal Location

1-C8-PC1 results (AS:3600, 2018)

- Summary
- FE Chase Down
  - 1 Cb_2-1.2G+1.5Q+1.5Qr
    - Critical Shear Perimeter 1
  - 2 Cb_6-G+ψ_cQ+ψ_cQr
    - Critical Shear Perimeter 1

FE Chase Down - 1 Cb_2-1.2G+1.5Q+1.5Qr - Critical Shear Perimeter 1

Design shear force	$V^* = \mathbf{550.0}$ kN	
Loads applied within perimeter	$L = \mathbf{2.2}$ kN	
Slab self weight	$S_{swt} = \mathbf{3.0}$ kN	
Support reaction	$R = \mathbf{0.0}$ kN	
Total adjustment	$\Delta V^* = L + S_{swt} + R = \mathbf{5.1}$ kN	
Adjusted perimeter shear force	$V^*_{red} = V^* - \Delta V^* = \mathbf{544.9}$ kN	
Moment about major axis	$M^*_{v.major} = \mathbf{17.1}$ kNm	
Moment about minor axis	$M^*_{v.minor} = \mathbf{-10.8}$ kNm	
Critical perimeter at, 0.5 × effective depth		
Effective depth to tension reinforcement	$d_{om} = \mathbf{214.0}$ mm	
Equivalent critical shear perimeter breadth	$B_{equiv.perim} = \mathbf{664.0}$ mm	
Equivalent critical shear perimeter depth	$D_{equiv.perim} = \mathbf{664.0}$ mm	
Critical shear perimeter	$u = \mathbf{2403.2}$ mm	
Ratio of longer side to shorter side of column	$\beta_h = \mathbf{1.000}$	
Concrete compressive strength	$f'_c = \mathbf{32.000}$ N/mm^2	
Concrete shear strength	$f_{cv} = MIN[0.17 \times (1 + 2 / \beta_h), 0.34] \times \sqrt{f'_c} = \mathbf{1.9}$ N/mm^2	
Ultimate shear strength of slab with no moment transfer	$V_{uo} = u \times d_{om} \times f_{cv} = \mathbf{989.2}$ kN	AS 3600-2018 Clause 9.3.3(a)
Dimension of critical shear perimeter measured parallel to $M^*_{v.major}$	$a_{major} = \mathbf{664.0}$ mm	
Dimension of critical shear perimeter measured parallel to $M^*_{v.minor}$	$a_{minor} = \mathbf{664.0}$ mm	
Ultimate shear strength of slab modified for moment transfer in major direction	$V_{u.major} = V_{uo} / (1 + u \times \lvert M^*_{v.major}\rvert / (8 \times \lvert V^*_{red}\rvert \times a_{major} \times d_{om})) = \mathbf{927.7}$ kN	AS 3600-2018 Clause 9.3.4(a)
Ultimate shear strength of slab modified for moment transfer in minor direction	$V_{u.minor} = V_{uo} / (1 + u \times \lvert M^*_{v.minor}\rvert / (8 \times \lvert V^*_{red}\rvert \times a_{minor} \times d_{om})) = \mathbf{949.5}$ kN	AS 3600-2018 Clause 9.3.4(a)
Effective ultimate shear strength of slab modified for moment transfer	$V_u = MIN[V_{u.major}, V_{u.minor}] = \mathbf{927.7}$ kN	
Capacity reduction factor for shear	$\phi = \mathbf{0.700}$	AS 3600-2018 Table 2.2.2(e)
Design shear strength of slab	$\phi \times V_u = \mathbf{649.4}$ kN	AS 3600-2018 Clause 9.3.2

$V^*_{red} \leq \phi \times V_u$

✔ Pass

Typical Edge Location

1-C4-PC3 results (AS:3600, 2018)

- Summary
- FE Chase Down
  - 1 Cb_2-1.2G+1.5Q+1.5Qr
    - Critical Shear Perimeter 1
  - 2 Cb_8-G+ψ_cQ+ψ_cQr

FE Chase Down - 1 Cb_2-1.2G+1.5Q+1.5Qr - Critical Shear Perimeter 1

Design shear force	$V^* =$ **230.1** kN	
Loads applied within perimeter	$L =$ **2.0** kN	
Slab self weight	$S_{swt} =$ **2.8** kN	
Support reaction	$R =$ **0.0** kN	
△ Total adjustment	$\Delta V^* = L + S_{swt} + R =$ **4.8** kN	
Adjusted perimeter shear force	$V^*_{red} = V^* - \Delta V^* =$ **225.3** kN	
Moment about major axis	$M^*_{v.major} =$ **-101.1** kNm	
Moment about minor axis	$M^*_{v.minor} =$ **-2.3** kNm	
Critical perimeter at, 0.5 × effective depth		
Effective depth to tension reinforcement	$d_{om} =$ **214.0** mm	
Equivalent critical shear perimeter breadth	$B_{equiv.perim} =$ **664.0** mm	
Equivalent critical shear perimeter depth	$D_{equiv.perim} =$ **557.0** mm	
Critical shear perimeter	$u =$ **1778.0** mm	
Ratio of longer side to shorter side of column	$\beta_h =$ **1.000**	
Concrete compressive strength	$f'_c =$ **32.000** N/mm²	
Concrete shear strength	$f_{cv} = MIN[0.17 \times (1 + 2 / \beta_h), 0.34] \times \sqrt{f'_c} =$ **1.9** N/mm²	
Ultimate shear strength of slab with no moment transfer	$V_{uo} = u \times d_{om} \times f_{cv} =$ **731.8** kN	AS 3600-2018 Clause 9.3.3(a)
Dimension of critical shear perimeter measured parallel to $M^*_{v.major}$	$a_{major} =$ **557.0** mm	
Dimension of critical shear perimeter measured parallel to $M^*_{v.minor}$	$a_{minor} =$ **664.0** mm	
Ultimate shear strength of slab modified for moment transfer in major direction	$V_{u.major} = V_{uo} / (1 + u \times \lvert M^*_{v.major}\rvert / (8 \times \lvert V^*_{red}\rvert \times a_{major} \times d_{om})) =$ **398.5** kN	AS 3600-2018 Clause 9.3.4(a)
Ultimate shear strength of slab modified for moment transfer in minor direction	$V_{u.minor} = V_{uo} / (1 + u \times \lvert M^*_{v.minor}\rvert / (8 \times \lvert V^*_{red}\rvert \times a_{minor} \times d_{om})) =$ **720.3** kN	AS 3600-2018 Clause 9.3.4(a)
Effective ultimate shear strength of slab modified for moment transfer	$V_u = MIN[V_{u.major}, V_{u.minor}] =$ **398.5** kN	
Capacity reduction factor for shear	$\phi =$ **0.700**	AS 3600-2018 Table 2.2.2(e)
△ Design shear strength of slab	$\phi \times V_u =$ **278.9** kN	AS 3600-2018 Clause 9.3.2

$V^*_{red} \leq \phi \times V_u$

✔ Pass

Typical Corner Location

1-C24-PC2 results (AS:3600, 2018)

- Summary
- FE Chase Down
  - 1 Cb_2-1.2G+1.5Q+1.5Qr
    - Critical Shear Perimeter 1
  - 2 Cb_6-G+ψ_cQ+ψ_cQr

FE Chase Down - 1 Cb_2-1.2G+1.5Q+1.5Qr - Critical Shear Perimeter 1

Design shear force	$V^* =$ **110.4** kN	
Loads applied within perimeter	$L =$ **1.7** kN	
Slab self weight	$S_{swt} =$ **2.3** kN	
Support reaction	$R =$ **0.0** kN	
△ Total adjustment	$\Delta V^* = L + S_{swt} + R =$ **4.0** kN	
Adjusted perimeter shear force	$V^*_{red} = V^* - \Delta V^* =$ **106.4** kN	
Moment about major axis	$M^*_{v.major} =$ **59.5** kNm	
Moment about minor axis	$M^*_{v.minor} =$ **-58.8** kNm	
Critical perimeter at, 0.5 × effective depth		
Effective depth to tension reinforcement	$d_{om} =$ **214.0** mm	
Equivalent critical shear perimeter breadth	$B_{equiv.perim} =$ **557.0** mm	
Equivalent critical shear perimeter depth	$D_{equiv.perim} =$ **557.0** mm	
Critical shear perimeter	$u =$ **1114.0** mm	
Ratio of longer side to shorter side of column	$\beta_h =$ **1.000**	
Concrete compressive strength	$f'_c =$ **32.000** N/mm^2	
Concrete shear strength	$f_{cv} = MIN[\,0.17 \times (1 + 2 / \beta_h),\, 0.34\,] \times \sqrt{f'_c} =$ **1.9** N/mm^2	
Ultimate shear strength of slab with no moment transfer	$V_{uo} = u \times d_{om} \times f_{cv} =$ **458.5** kN	AS 3600-2018 Clause 9.3.3(a)
Dimension of critical shear perimeter measured parallel to $M^*_{v.major}$	$a_{major} =$ **557.0** mm	
Dimension of critical shear perimeter measured parallel to $M^*_{v.minor}$	$a_{minor} =$ **557.0** mm	
Ultimate shear strength of slab modified for moment transfer in major direction	$V_{u.major} = V_{uo} / (1 + u \times \lvert M^*_{v.major}\rvert / (8 \times \lvert V^*_{red}\rvert \times a_{major} \times d_{om})) =$ **277.4** kN	AS 3600-2018 Clause 9.3.4(a)
Ultimate shear strength of slab modified for moment transfer in minor direction	$V_{u.minor} = V_{uo} / (1 + u \times \lvert M^*_{v.minor}\rvert / (8 \times \lvert V^*_{red}\rvert \times a_{minor} \times d_{om})) =$ **278.6** kN	AS 3600-2018 Clause 9.3.4(a)
Effective ultimate shear strength of slab modified for moment transfer	$V_u = MIN[\,V_{u.major}, V_{u.minor}\,] =$ **277.4** kN	
Capacity reduction factor for shear	$\phi =$ **0.700**	AS 3600-2018 Table 2.2.2(e)
△ Design shear strength of slab	$\phi \times V_u =$ **194.2** kN	AS 3600-2018 Clause 9.3.2

$V^*_{red} \le \phi \times V_u$

✔ Pass

3.5 Summary

Chapter 3 presented the methodology for the second project aim as well as the results obtained from the detailed analyses. The results of this chapter will be presented and discussed in Chapter 4 which continues on from Chapter 3.

Chapter 4: Research Results & Discussions

4.1 Introduction

The aim of Chapter 4 is to be present and discuss the results that were obtained from the hand calculation and software analyses in Chapter 3. The results from Chapter 3 are collated, summarised and discussed in this chapter.

The chapter is divided into three sections: the results, the discussions and the summary of the chapter.

4.2 Results

As discussed in Section 3.4 of Chapter 3, the software analysis was undertaken primarily to validate and correlate the results of the hand calculation analysis. The point being that by analysing each of the three typical locations by the two different methods the margin of error is minimised if both sets of analyses give similar results.

Therefore, **Error! Reference source not found.** is included on the following page to present both sets of utilisation ratio results for each location according to each code. The percentage difference between each pair of utilisation ratios is also included in the table.

Table 4: Hand Calculation vs Software Analysis Results

Column Location	ACI 318			EC2			AS 3600		
	(a) Hand Calculation U/R	(b) TSD U/R	% difference between hand calculation and TSD U/Rs	(a) Hand Calculation U/R	(b) TSD U/R	% difference between Hand calculation and TSD U/Rs	(a) Hand Calculation U/R	(b) TSD U/R	% difference between hand calculation and TSD U/Rs
Typical Internal location	0.906	0.936	3%	1.176	1.244	6%	0.812	0.839	3%
Typical Edge location	0.565	0.455	-22%	1.095	1.019	-7%	0.739	0.808	9%
Typical Corner location	0.450	0.335	-29%	1.002	0.865	-15%	0.642	0.548	-16%

** The percentage difference was calculated as 100%* (a-b)/((a+b)/2)*

The results presented in Table 4 are presented in the graphs below to highlight the differences observed between the utilisation ratios.

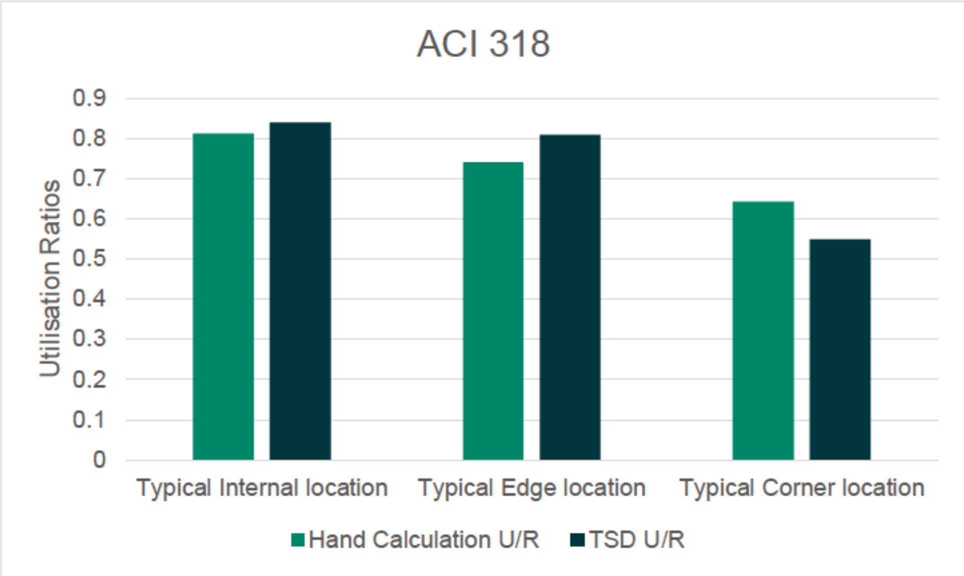

Figure 26: Variances between the hand calculation and TSD URs - ACI 318

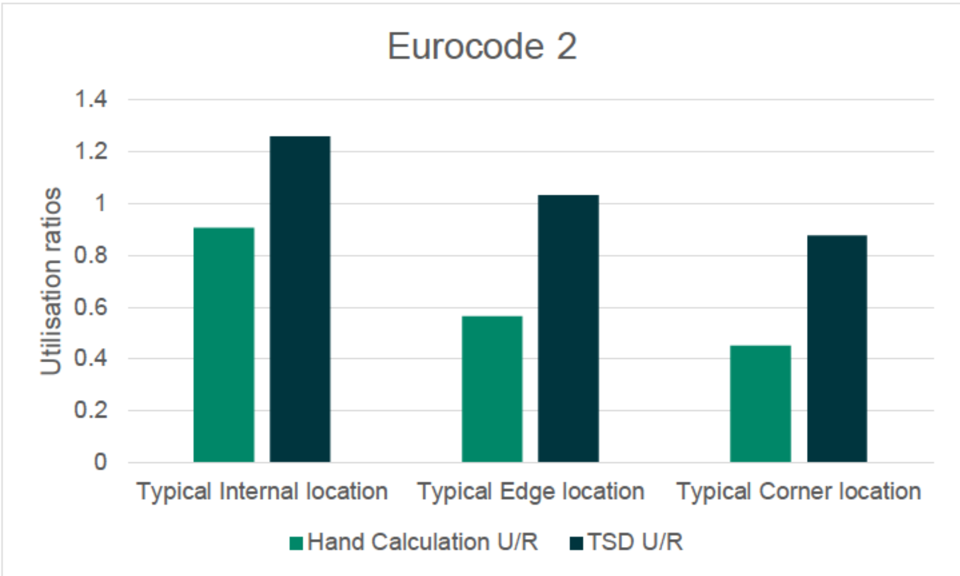

Figure 27: Variances between the hand calculation and TSD URs - EC2

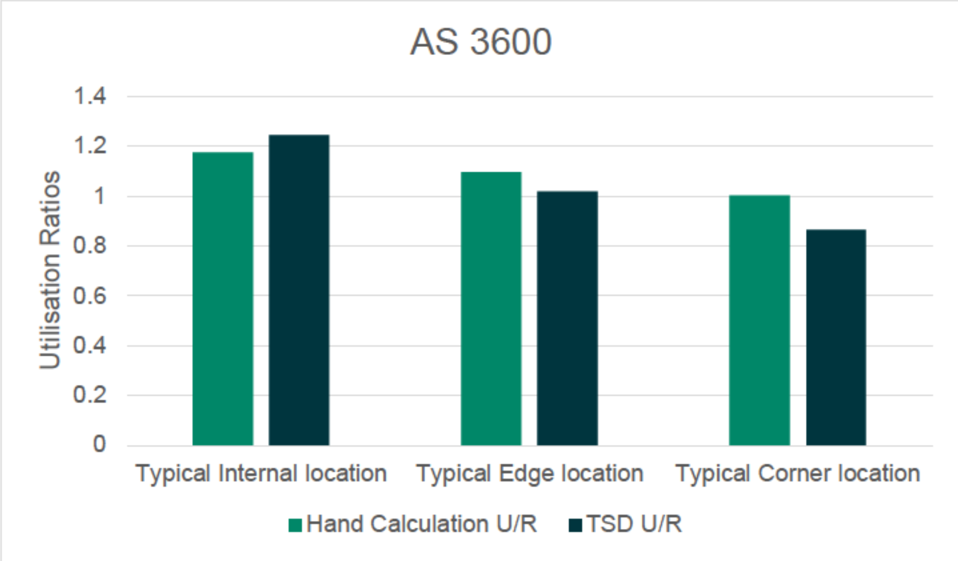

Figure 28: Variances between the hand calculation and TSD URs – AS 3600

From these results, it can be observed that the utilisation ratios between the two sets of analyses are relatively similar with the largest difference being observed for the typical corner column location according to ACI 318. These differences, among other reasons, can be attributed to the fact that the unbalanced moments determined in the hand calculations are based on the simplified methods presented in the codes rather than an exact determination of the moments as the software analysis would carry it out.

4.2.1 Code Comparisons

In this section the utilisation ratios from the three codes based on the two sets of analyses will be interrogated in order to rate the efficiencies of the codes.

Comparisons of the hand calculation utilisation ratio results are presented in Table 5 and

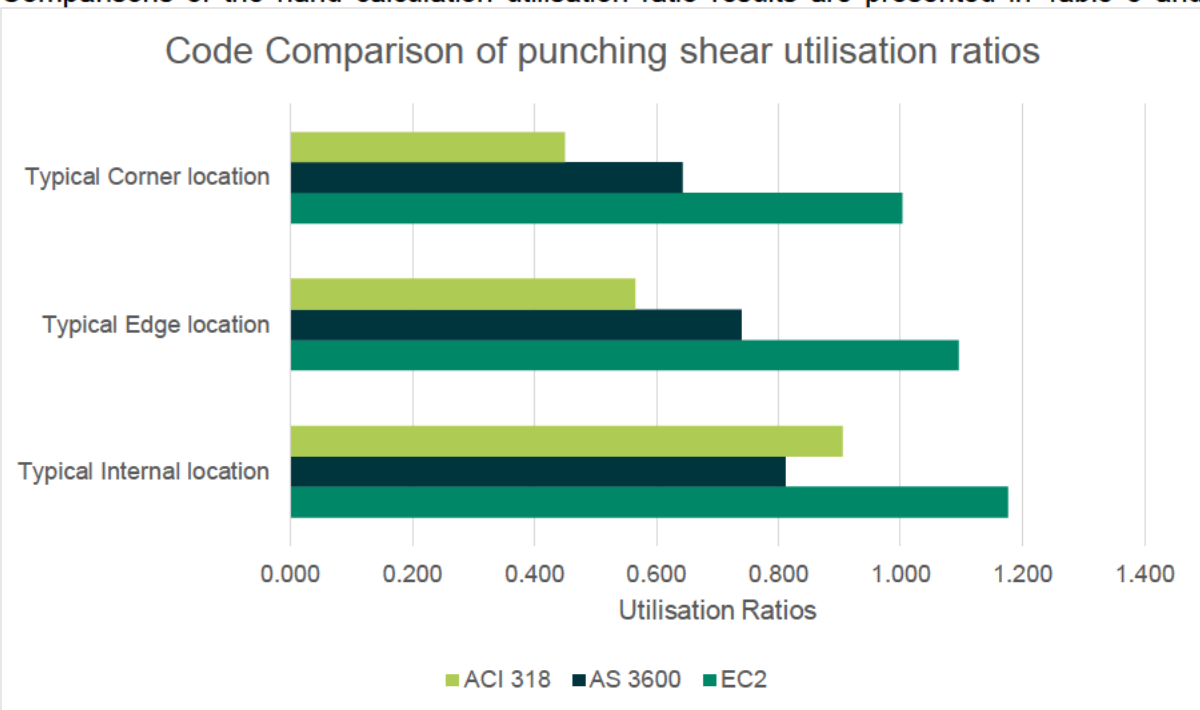

Figure 29 below:

Table 5: Hand Calculation Utilisation Ratios

Code	Utilisation Ratios		
	Typical Internal location	Typical Edge location	Typical Corner location
ACI 318	0.906	0.565	0.450
EC2	1.176	1.095	1.002
AS 3600	0.812	0.739	0.642

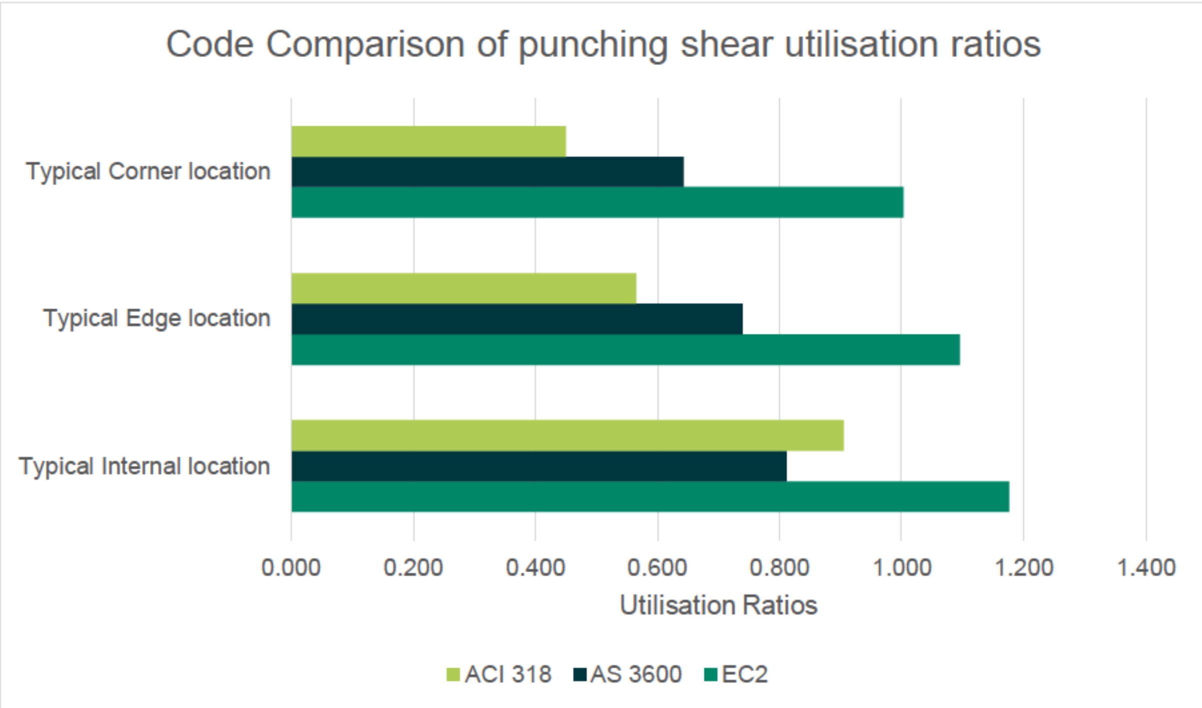

Figure 29: Code Comparison of punching shear utilisation ratios – Hand calculation analysis

Comparisons of the software analysis utilisation ratio results are included in Table 6 and Figure 30 below:

Table 6: TSD Utilisation Ratios

Code	Utilisation Ratios		
	Typical Internal location	Typical Edge location	Typical Corner location
ACI 318	0.936	0.455	0.335
EC2	1.244	1.019	0.865
AS 3600	0.839	0.808	0.548

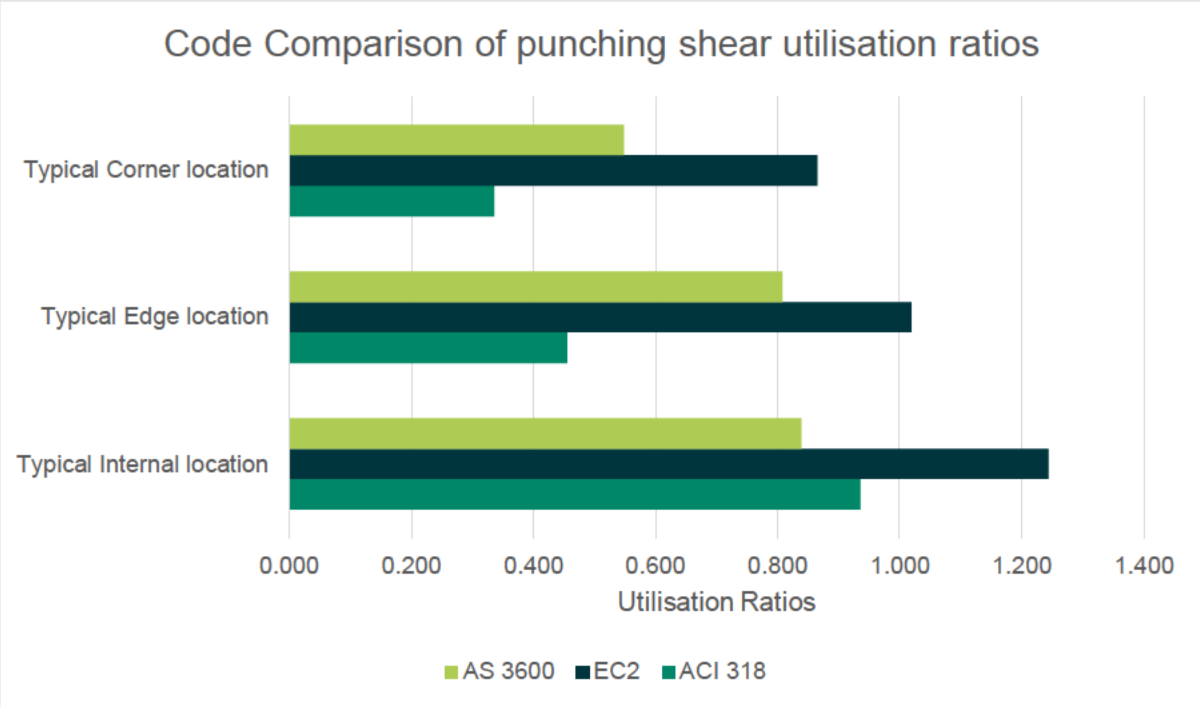

Figure 30: Code Comparison of punching shear utilisation ratios – TSD analysis

4.3 Discussion

According to the provisions on EN 1992, **insufficient** punching shear resistance is observed at each of the three slab locations considered. A redesign of these locations would be required in order to comply with the requirements of the code. These locations can be redesigned by incorporating a higher class of concrete, by providing drop head panel at each column location to increase the effective slab depth or by including punching shear reinforcement.

One reason for the conservatism observed in EN 1992 can be attributed to the the ultimate limit state (ULS) load factors prescribed by the code. The ULS factors for EN 1992 are 1.35DL+1.5LL, while the factors for AS 3600 and ACI 318 are 1.2DL+1.5LL and 1.2DL+1.6LL respectively. These factors are based on statistical analyses and research to minimise the probability of structural failure and instability.

An additional check was carried out to determine the differences in the utilisation ratios when the ULS factors are standardised across the codes and set to 1.0DL +1.0LL. The results for the standardised hand calculation analysis are presented below:

Table 7: Hand Calculation Utilisation Ratios without ULS factors

Code	Utilisation Ratios		
	Typical Internal location	Typical Edge location	Typical Corner location
EC2	0.852	0.794	0.727
AS 3600	0.645	0.587	0.510
ACI 318	0.708	0.455	0.335

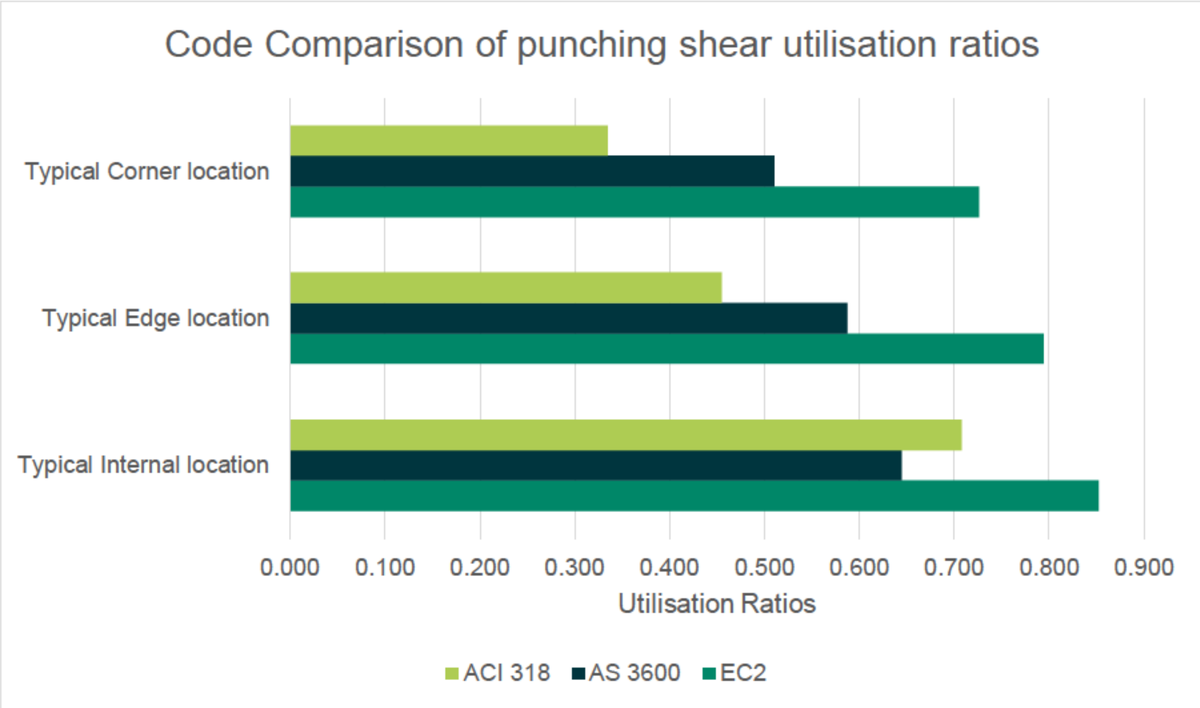

Figure 31: Code Comparison of punching shear utilisation ratios – Hand calculation analysis without ULS factors

With the inclusion of standardised ULS factors, EN 1992 remains the conservative code however it can be observed that the margin of difference between the utilisation ratios between the codes decreases. Therefore, the conservatism observed for EN 1992 can be partially attributed to the higher dead load ULS factor. However even when this variable is removed, the code provisions for EN 1992 remain more conservative than those of ACI 318 and AS 3600.

It was observed that, in general, ACI 318 provided the lowest utilisation ratios for the locations and is therefore rated to be the most efficient code for punching shear design. The results observed for AS 3600 are similar to those observed for ACI 318.

4.4 Summary

Chapter 4 presented the findings from Chapter 3 and continued on from Chapter 3 in addressing the second project aim. The conclusions from this chapter and the prior ones will be presented in Chapter 5.

Chapter 5: Conclusions and recommendations

5.1 Introduction

Chapter 5 concludes the research project and summarises the findings and discussions from Chapters 2 to 4.

The chapter is divided into two sections, the project conclusions and the recommendations for further work.

5.2 Conclusions

The aims of the project were to identify and clarify the differences between the three design codes and to rate their cost effectiveness.

The first aim was achieved through the literature review in Chapter 2, where a review of the existing literature was carried out. The focus of the review was on the three codes being considered. From the literature review it was found that the punching shear code provisions of all three codes are based on the shear stress method. The punching shear code provisions of ACI 318 and AS 3600 were found to be similar in their approach and execution, while the code provisions of EN 1992 differed from those of the other two codes.

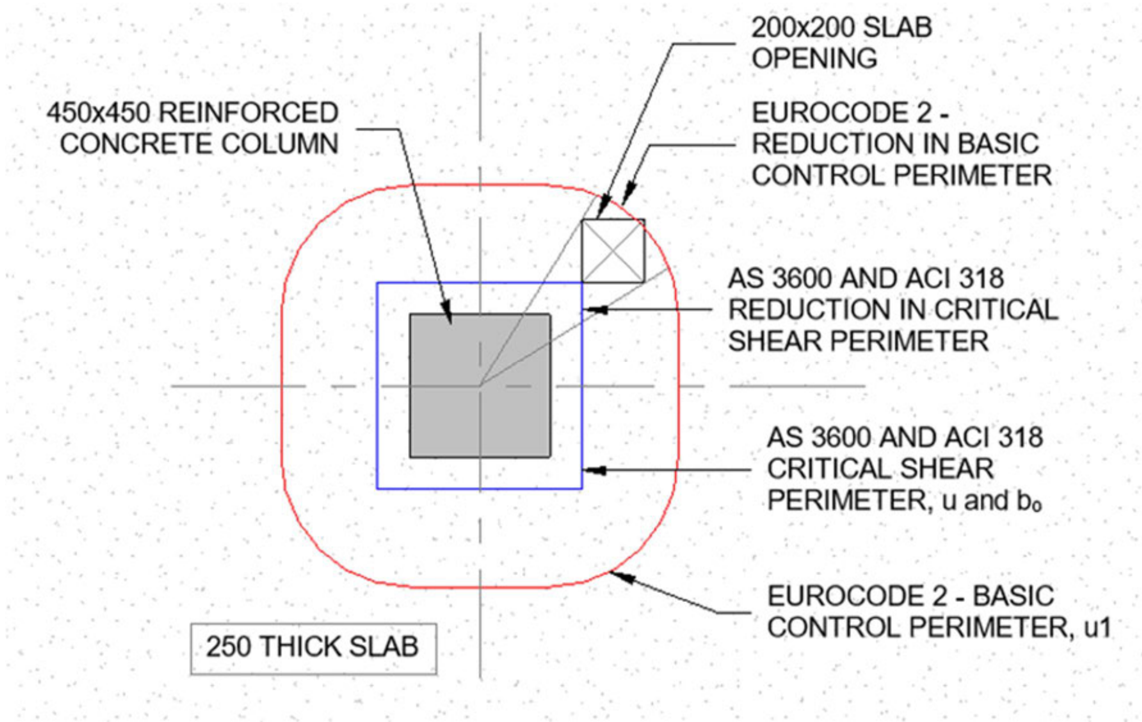

Figure 32: Critical shear perimeters for EC2 and AS 3600/ACI 318

The second project aim was researched through the analyses detailed in Chapter 3 and the corresponding findings presented in Chapter 4. EN 1992 was found to be the most conservative code where the typical locations considered required a redesign in order to meet the requirements of the code. ACI 318 was determined to be the most cost-effective code, with lower utilisation ratios observed from this code for both the typical corner and edge locations. The efficiency of AS 3600 was similar to that of ACI 318.

Comparing the results of this project to the existing literature is challenging because the existing comparative studies do not focus on utilisation ratios. However as discussed in Chapter 2, Lourenco et al (2021) conducted a study to investigate the effect of the location of openings in relation to column positions and compared these values to the values predicted by four codes, namely ACI 318-19, EN 1992, fib Model Code 2010 (MC2010) and NBR 6118. ACI was found to be more conservative with a 16% average value higher than EN 1992. However, it should be noted that the ultimate limit state (ULS) load factors were not incorporated in this study and the study was focused on the effect of opening sizes rather than comparing utilisation ratios.

A study by Al-Rousan and Alnemrawi (2023), considered twenty-one models using NLFEA to assess the effect of opening sizes and locations. The code provisions of ACI 318-2019, EN 1992 and fib Model Code 2010 were compared to each other where it was observed that the ACI 318 and MC2010 have a close prediction in most cases to the results derived from the NLFEA analyses. Furthermore, the ACI 318 provisions were found to be the most accurate from among the tested codes.

It should be noted that the conclusions from the research project are based on the punching shear code provisions and calculations only and limited to only the specific scenario studied for this project.

5.3 Further Work

There is room for further work and expansion of this research topic. Further research may address some of the limitations addressed for this project which primarily includes researching the effect that the inclusion of punching shear reinforcement will have on the results.

In addition, further work can be done to understand the embedded conservatisms of the punching shear code provisions of EN 1992 which were not addressed by this project.

Once the new EN 1992 code is published in 2026, further work can be done to compare its efficiency to ACI 318 and AS 3600 as it will incorporate critical shear crack theory for its punching shear code provisions.

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