ACI 318-05, CSA A23.3-04, Eurocode 2 (2003), DIN 1045-1 (2001), BS 8110-97 and CEB-FIP MC 90 Provisions for Punching Shear of Reinforced Concrete Flat Slabs

by N.J. Gardner

Synopsis: This paper describes and evaluates the punching shear provisions of ACI 318-05, CSA A23.3-04, Eurocode 2 (2003), DIN 1045-1 (2001), BS 8110-97 and CEB-FIP MC 90 for interior columns without moment transfer and interior, edge and corner connections with moment transfer. All code column slab punching shear predictions with moment transfer are extensions of the no-moment transfer provisions - any lack of conservatism in the no-moment transfer provisions are reflected in the moment transfer comparisons.

The validity of the interior connection, no moment transfer, punching shear provisions were evaluated using the information from the data bank developed by the Punching Shear Working Group of *fib* Commission 4. The code provisions for interior columns with moment transfer and edge columns connections under gravity loads were compared with data taken from the University of Alberta Research Report No.223 (Afhami, Alexander and Simmonds 1998). The code provisions for corner column slab connections were compared with data from Zaghool (1971) and Walker and Regan (1987).

BS 8110, DIN 1045-1 and CEB-FIP MC 90, which have size effect and reinforcement ratio terms and use control perimeters 1.5*d*, 1.5*d*, and 2.0*d* from the column, have smaller coefficients of variation than ACI 318 and CSA A23.3 for interior column slab connections. However, these codes do not have general provisions for punching under combined shear and moment transfer for edge and corner connections. Their simple shear stress multipliers are limited to gravity load moments. Calculated statistical indicators show that only ACI 318 satisfies the requirement of a 5% fractile value greater than one for concentric punching shear. The shear resistance coefficients of BS 8110, CEB-FIP MC 90, CSA A23.3-04, and DIN 1045-1 should be reduced to meet a 5% fractile value of unity.

ACI 318 and CSA A23.3 should be modified to include size effect terms and use the cube root of the concrete strength. A minimum flexural reinforcement ratio of 0.75% should be specified in regions susceptible to punching shear. The consequences of permitting use of a rectangular control perimeter have to be acknowledged.

Keywords: code provisions; flat slabs; punching shear; structural design

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INTRODUCTION

Flat plates and slabs are an economical structural system for medium height residential and office buildings. However flat plates and flat slabs have to be treated with caution as they can be susceptible to failure by punching shear. Punching shear usually occurs around the support columns of flat slabs which are regions of large moment and where flexural cracks are observed around the periphery of the support. Punching shear is an undesirable mode of failure that occurs without warning and can lead to progressive collapse of large areas of slab or even complete structures. Punching shear can also occur due to large concentrated loads anywhere on the slab area. While most punching shear failures occur during construction when there is insufficient early-age punching shear capacity under the relatively high construction loads Feld (1968), ACI-ASCE 423 (1989), failure can also occur with mature structures. The 1995 Sampoong Department Store (Seoul) collapse by punching shear that killed 500 people is a severe example of a failure in service.

Most design codes calculate a nominal shear stress on a control perimeter some fraction/multiple of the slab depth away from the column. Reviews of codes and tests are given in the summarizing reports in *fib* Bulletin 12 (2001) and earlier by Regan and Braestrup (1985) and in several papers at the Punching Shear Colloquium in Stockholm, Silfwerbrand and Hassanzadeh (2000). Most experimental tests have been done on relatively thin slabs and as unit strength decreases with increasing size, code equations should incorporate a size effect term. Bazant and Cao (1987) established a theoretical basis for the necessity of size effect terms in the punching shear prediction equations for concrete members. The punching shear provisions of ACI 318-05, first adopted in ACI 318-63, are unusual in that there is no consideration of the effect of flexural steel or size effect. The British Code BS 8110-97, the CEB-FIP 1990 Model Code, DIN 1045-1 and EuroCode 2 (2003) have reinforcement ratio and size effect terms in their punching shear equations.

This paper summarises and evaluates the punching shear provisions of several major codes, ACI 318-05, CSA A23.3-04, BS 8110-97, CEB-FIP MC 90, DIN 1045-1 (2001), and EuroCode 2 (2003) for interior columns without moment transfer and interior column connections, edge column connections and corner column connections with moment transfer.

All codes require the reduced strength, in terms of load or stress, to be greater than the factored applied/specified loads or stresses respectively.

Specified loads * load factor < resistance partial safety factor * predicted resistance

The resistance partial safety factor should be less than unity in the numerator, ACI and CSA, or greater than one in the denominator, BS, CEB, DIN or EuroCode 2.

The code provisions are grouped by the similarity of their provisions; namely ACI and CSA and the European provisions. The CEB Model Codes of 1970, 1978 and 1990 have been influential in the formulation of many codes.

A further complication is the definition of concrete compressive strength. The ACI and CSA codes use specified concrete strength, f'_c , where one in eleven of the cylinder strength results can fall below the mean cylinder strength, i.e. 9 % fractile, and the European codes use characteristic strength, f_{ck} , where 5% of the measured results can fall below the mean strength. Reineck (1999) suggested that specified strength can be related to the characteristic strength by the following relationship.

$$f_{ck} = f'_c - 1.60 \text{ Mpa} \quad (1)$$

An increasing concern is the drift capacity of flat plate and flat slab structures under seismic displacements. Typically the connections will be subjected to cyclic displacements and a shear load which is a fraction, 40%-60%, of the no moment capacity of the connections. Flat plate column connections should not be the primary lateral load resisting component of the structure; ductile flexural walls, commonly called shear walls, should be provided.

Information on the influence of perforations and openings through the slab on punching shear strength can be found in Teng et al (2004).

Nominal Safety Factors

It must be observed that the nominal safety factors implied in the various codes, which are the combined effect of load factors and material resistance factors, are not identical. Occupancy (live) loads specified in various national building codes are not considered in the nominal safety factor calculations given below. It can be noted that the nominal safety factors of ACI 318-05 and CSA A23.3-04 are some 6% less than those of BS 8110-97, CEB-FIP MC 90 and DIN 1045-1.

ACI 318-05

The load factors on dead load and live loads are 1.2 and 1.6 respectively but the factored load shall not be less than 1.4 times dead load and the behaviour factor for shear is 0.75 giving a combined effect for dead load of 1.4/0.75 = 1.87.

CSA A23.3-2004

The load factors on dead load and live loads are 1.25 and 1.5 respectively and the material partial safety factor for concrete is 0.65 giving a combined effect of 1.25/0.65 = 1.92 for dead load.

CEB-FIP MC 90, DIN 1045-1 (2001-07), EuroCode 2-2003

The load factor on dead load and live loads are 1.35 and 1.5 respectively and the partial safety factor for concrete is 1.5 giving a combined effect of 1.35 x 1.5 = 2.025 for dead load.

BS 8110-97

The load factors on dead load and adverse imposed loads are 1.4 and 1.6 respectively and the partial safety factor for shear is 1.25 giving a combined effect for dead load of 1.4 x 1.25 = 1.75. An additional factor of 1.15 is applied for punching shear at an interior column, as opposed to punching shear due to an applied load, to take account of possible unbalanced moments increasing the nominal safety factor to 1.75 x 1.15 = 2.013.

CODE PROVISIONS

ACI 318-05

The provisions of ACI 318-2005 have evolved from the provisions of ACI 318-63 and are unchanged from those of ACI 318-02, ACI 318-99 and ACI 318-95. ACI 318-05 specifies that the shear capacity be calculated on the minimum perimeter located at a distance *d/2* from the periphery of the column or the concentrated load. However for square or rectangular columns the critical section can be taken with four straight sides. The nominal shear strength for nonprestressed slabs and footings v_r (v_c in ACI 318 nomenclature) shall be the smallest of:

$$v_r = v_c = 0.083\left(2 + \frac{4}{\beta_c}\right)\lambda\phi\sqrt{f'_c} \tag{2a}$$

$$v_r = v_c = 0.083\left(\alpha_s \frac{d}{u} + 2\right)\lambda\phi\sqrt{f'_c} \tag{2b}$$

$$v_r = v_c = 0.33\lambda\phi\sqrt{f'_c} \tag{2c}$$

f'_c = specified concrete cylinder strength, MPa.
v_r = nominal shear strength (v_c ACI nomenclature), MPa.
α_s = 40 for interior columns, 30 for edge columns, 20 for corner columns.
β_c = ratio of longer to shorter dimension of the loaded area.
λ = factor to account for concrete density (1.0 for normal density concrete)
Φ = 0.75 partial safety factor for shear

For shear loads without moment transfer the factored applied shear stress is given by; where V_f is the factored shear force and v_f is the nominal factored shear stress.

$$v_f = \frac{V_f}{A_c} \tag{3}$$

A_c = area of critical section = ud
b = side dimension of rectangular column
c = diameter of circular column
d = average effective slab depth
u = shear perimeter = $\pi(c + d)$ (interior circular columns)
$u = \Sigma b + 4d$ (interior rectangular columns)

When gravity load, wind, earthquake, or other lateral forces cause a transfer of moment M_f

between a slab and column, a fraction $\gamma_v M_f$ of the unbalanced moment is considered to be transferred by eccentricity of shear, which is assumed to vary linearly about the centroid of the critical section.

The fraction of the unbalanced moment transferred by shear is given by:

$$\gamma_{vx} = 1 - \frac{1}{1 + (2/3)\sqrt{b_1 / b_2}} \tag{4}$$

where b_1 and b_2 are the sides of the control perimeter of a rectangular column, with b_1 the width of the shear section in the direction of the span in which the moment is determined (perpendicular to the moment vector) and b_2 perpendicular to b_1.

The nominal factored shear stress v_f can be calculated by:

$$v_f = \frac{V_f}{A_c}\left[1 + \frac{A_c \gamma_{vx} M_{fx}}{J_{cx} V_f} y + \frac{A_c \gamma_{vy} M_{fy}}{J_{cy} V_f} x\right] \tag{5}$$

V_f, M_{fx} and M_{fy} are the factored shear force and factored unbalanced moments determined at the centroidal axis of the critical section; A_c is the concrete area of the assumed critical section and x and y are the coordinates of any point on the critical section from the centroidal axis. The shear force V_f and the moments M_{fx} and M_{fy} are not easily determined for continuous flat slab systems. The quantities J_{cx} and J_{cy} used in Equation (5) are calculated properties of the assumed critical section analogous to the polar moment of inertia.

CSA A23.3-2004

Prior to 1984 the punching shear provisions of CSA A23.3 were similar in format to those of ACI 318. CSA A23.3-84M replaced the ACI behaviour factor, φ, with material partial safety factors, φ_c and φ_s, and different load factors. To maintain an appropriate level of safety the coefficients in the punching shear expressions were increased. CSA A23.3-04 calculates the shear capacity on the minimum perimeter located at a distance *d/2* from the periphery of the column or the concentrated load.

For nonprestressed slabs and footings the shear strength v_r shall be the smallest of

$$v_r = v_c = \left(1 + \frac{2}{\beta_c}\right) 0.19 \lambda \phi_c \sqrt{f'_c} \tag{6a}$$

$$v_r = v_c = \left(\alpha_s \frac{d}{u} + 0.19\right) \lambda \phi_c \sqrt{f'_c} \tag{6b}$$

$$v_r = v_c = 0.38 \lambda \phi_c \sqrt{f'_c} \tag{6c}$$

If the effective depth used in the two way shear calculations exceeds 300 mm, then v_r obtained from equations 6a, 6b and 6c shall be multiplied by 1300/(1300 + d).

b = side dimension of rectangular column
c = diameter of circular column
d = slab average effective depth
u = shear perimeter = $\pi(c + d)$ (interior circular columns)
$u = \Sigma b + 4d$ (interior rectangular columns)
f'_c = specified concrete cylinder strength, not to be taken greater than 64 MPa
v_r = nominal shear strength, MPa.
β_c = is the ratio of longer to shorter dimension of the loaded area.
λ = factor to account for concrete density (1.0 for normal density concrete)
φ_c = strength reduction factor for concrete (0.65 CSA A23.3-2004, 0.60 CSA A23.3-1994)
α_s = 4 for interior connections, 3 for edge connections and 2 for corner connections.

CSA A23.3 uses the Equations (4) and (5) for moment transfer of unbalanced moments at interior, edge and corner column connections.

CSA A23.3-94 required that corner column slab connections be checked for one-way shear on the minimum length straight line located at $d/2$ across the corner, Figure 1. However CSA A23.3-04 requires that one way shear be checked along a critical section $d/2$ from the edge of the column or column capital.

$$v_r = v_c = \beta\lambda\phi_c\sqrt{f'_c} \tag{7}$$

$\beta = 0.21$ if the slab thickness is less than 350 mm. If the slab thickness exceeds 350 mm $\beta = 230/(1000 + d_v)$ where d_v is the greater of $0.9d$ and $0.72\,h$.

CSA A23.3 requires integrity steel on all faces of the connection such that;

$$\sum A_{sb} \geq 2V_{se} / f_y \tag{8}$$

Where ΣA_{sb} is the total of the anchored bottom steel passing through the column and V_{se} shear transmitted to the column due to specified unfactored loads but not less than the shear corresponding to twice the self weight of the slab.

BS 8110-97

BS 8110-97 is the most recent evolution of the Code of Practice CP 110-72 and BS 8110-85. The British Standard BS 8110-97 was uses a rectangular control perimeter 1.5 d from the loaded area for both circular and rectangular loaded areas.

$$\frac{V_f}{ud} < v_r = 0.79(100\rho)^{1/3}(400/d)^{1/4} < 0.8\sqrt{f_{cu}} \tag{9}$$

f_{cu} = characteristic concrete cube strength, MPa.
$u = 4\,(c + 3d)$ for circular loaded areas, mm
$u = 4\,(b + 3d)$ for square loaded areas, mm
$\rho = (\rho_x + \rho_y)/2 < 0.03$

ρ = flexural steel ratio calculated for a width equal to $(c + 3d)$ or $(b + 3d)$

For characteristic concrete cube strengths greater than 25 MPa, v_r may be multiplied by $(f_{cu}/25)^{1/3}$. The value of f_{cu} should not be taken as greater than 40 MPa.

The British code provides two methods to calculate the effect of combined shear and unbalanced moments of interior columns; a variation of the eccentric shear expression or simple shear force multipliers. The nominal factored shear stress v_f at an interior column can be calculated by:

$$v_f = \frac{V_f}{A_c}\left[1 + \frac{1.5\, A_c M_{fx}}{V_f\, x}\right] \tag{10}$$

V_f and M_f are the factored shear force and unbalanced moments determined at the centroidal axis of the critical section; A_c is the concrete area of the assumed critical section and x is the length of the side of the control perimeter parallel to the axis of bending.

Alternatively the nominal shear force is increased by 15% to accommodate unbalanced moments at an interior column. Edge column connections and corner column connections subjected to moments perpendicular, moment vector parallel, to the slab edge acting towards the interior of the panel are treated by a single expression independent of the eccentricity of the load:

$$v_{max} = 1.25\, V_f / ud \tag{11}$$

CEB-FIP Model Code 1990

CEB-FIP MC 90 assumes a plastic distribution of shear stress on a critical section at $2d$ from the periphery of the loaded areas as shown in Figure 2.

The applied shear stress at the critical section due to a factored concentrated force, V_f, at an internal connection transferring an unbalanced moment, M_{fu}, is calculated as:

$$v_f = \frac{V_f}{u_l d} + \frac{K M_{fu}}{W_1 d} \tag{12}$$

Where

$$W_1 = \int_0^{u_l} |\, e \,|\, dl \tag{13}$$

d = effective thickness of the slab
dl = an elementary length of the perimeter
e = the distance of dl from the moment M_f axis
l = the subscript refers to perimeter u_l
M_{fu} = applied unbalanced moment at the critical section due to factored loads

V_f = applied shear force due to factored loads
v_f = applied shear stress due to factored loads
K = fraction of the unbalanced moment M_f resisted by shear stresses which is a function of c_1/c_2. The value of K may be obtained from Table 1.
c_1 = column dimension parallel to the eccentricity of load
c_2 = column dimension perpendicular to the eccentricity of load

The shear stress resistance of the concrete, v_r, is given by:

$$v_r = (0.18/\gamma_c)\xi(100\rho f_{ck})^{\frac{1}{3}} \tag{14}$$

f_{ck} is the characteristic concrete cylinder strength, MPa.
ξ = size effect = $(1+(200/d)^{0.5}$ with d in mm
ρ = flexural reinforcement ratio = $(\rho_x\rho_y)^{1/2}$
γ_c = partial safety factor for concrete (presumably = 1.5) incorporated in equation (14)

For edge and corner connections, provided the eccentricity of the loading is towards the interior of the slab, the shear stress shall be calculated on the assumption of uniform shear on the reduced perimeters shown in Figure 3.

DIN 1045-1:2001

The critical section for DIN 1045-1 is the minimum perimeter, rounded corners, 1.5d from the loaded region. Illustrations are used to define the treatment of openings near the loaded area. Except for the definition of the critical section the DIN 1045-1 (2001) punching shear provisions are logically similar to the provisions of CEB-FIP MC 90.

The calculated factored shear stress must be less than the available shear strength.

The applied shear stress at the critical section due to a factored concentrated force, V_f, is

$$v_f = \beta\frac{V_f}{u_1 d} \tag{15}$$

$u = \pi(c + 3d)$ (interior circular columns)
$u = 2a + 2b + 3\pi d$ (interior rectangular columns where $a/b < 2.0$)

when $a/b > 2.0$
$u = 2a_1 + 2b_1 + 3\pi d$

$$a_1 < \begin{cases} a \\ 2b \\ 5.6d - b_1 \end{cases}$$

$$b_1 < \begin{cases} b \\ 2.8d \end{cases}$$

For edge and corner connections, provided the eccentricity of the loading is towards the interior of the slab, the load factored shear stress can be calculated with β= 1.05 for an interior column connections, 1.4 for edge column connections and 1.5 for corner column connections.

The shear strength is given by;

$$v_r = 0.14\gamma_c\eta_1(1+(200/d)^{0.5})(\rho_1 f_{ck})^{1/3} - 0.12\sigma_{cd} \tag{16}$$

f_{ck} = the characteristic concrete cylinder strength, MPa
size effect = $1+(200/d)^{0.5}$ < 2.0 with d in mm
η_1= factor to account for concrete density; 1.0 for normal density concrete

$\rho_1 = (\rho_x + \rho_y)/2 < 0.40 f_{cd}/f_y$
< 0.02

$f_{cd} = 0.85 f_{ck}/\gamma_c$. with $\gamma_c = 1.5$

$\sigma_{cd} = \dfrac{N_x + N_y}{2A_c}$ = average in-plane stress (NB tension +ve)

Eurocode 2-2003

The Eurocode 2-2003 provisions are effectively identical to those of CEB-FIP MC 90 with a control perimeter 2*d* from the loaded area. The applied shear stress at the critical section due to a factored concentrated force, V_f, is calculated as:

$$v_f = \beta \frac{V_f}{u_1 d} \tag{17}$$

$$\beta = 1 + K\frac{M_{fu} u_1}{V_f W_1} \tag{18}$$

where
u = shear perimeter
$u = \pi(c + 4d)$ (interior circular columns)
$u = 2a + 2b + 4\pi d$ (interior rectangular columns where $a/b < 2.0$)
W_1 = a property of the critical section shown in Figure 2.

$$W_1 = \int_0^{u1} |e|\, dl$$

K is a coefficient determining the fraction of M_{fu} resisted by shear stresses and a function of c_1/c_2 and its value is a function of the proportions of the unbalanced moment transmitted by uneven shear on the one hand and by bending and torsion. The value of K can be obtained from Table 1.

For edge and corner connections, provided the eccentricity of the loading is towards the interior of the slab, the load factored shear stress can be calculated as being uniformly distributed, β= 1, along the reduced control perimeters shown in Figure 3 or, for structures where lateral stability does not depend upon frame action between the slab and the columns, using equation (15) with β= 1.15 for an interior column connection, 1.4 for edge column connections and 1.5 for corner column connections.

The shear strength is calculated as;

$$v_r = \frac{0.18}{\gamma_c}(1+(200/d)^{0.5})(\rho_1 f_{ck})^{1/3} + 0.10\sigma_{cp} \geq (v_{\min} + 0.10\sigma_{cp}) \qquad (19a)$$

Size effect = $1+(200/d)^{0.5} < 2.0$ with d in mm (19b)

f_{ck} = the characteristic concrete cylinder strength, MPa
v_r = shear resistance of concrete, MPa
ρ = flexural reinforcement ratio = $(\rho_x\rho_y)^{1/2} < 0.02$
γ_c = concrete partial safety factor (= 1.5)
σ_{cp} = average precompression $(\sigma_{cx} + \sigma_{cy})/2$
σ_{cx}, σ_{cy} = normal concrete stresses at critical section – compression positive.

COMPARISON OF PREDICTION WITH EXPERIMENTAL RESULTS

Punching shear tests can be done on a multi-panel structure or using isolated slab column connections. Multi-panel tests are time consuming, expensive and it is difficult to determine experimentally the shears and moments applied to the individual connections. Isolated slab column connection tests have the problem that the boundary conditions may not represent connections in a continuous structure. The slab boundary, line of contraflexure, can be either uniform displacement and the column loaded, or the column is supported and a series of loads are applied along the line of presumed contraflexure. Moment redistribution cannot occur in an isolated connection test. All comparisons in this paper are with the results from tests on isolated specimens except for Walker and Regans' (1987) results for corner column slab connections. The descriptor "gravity loads" for edge and corner connections is used to limit discussions to connections where the eccentricity is towards the interior of the slab panel.

BS 8110, CEB-FIP MC 90 and DIN 1045-1 do not have general provisions for punching under combined shear and moment transfer for moment vectors parallel to the edge for edge and corner connections. Their simple shear stress multipliers are limited to gravity load moments. DIN 1045-1 does not have a shear-moment transfer equation for an interior column connection.

All limitations on the magnitude of the concrete compressive strength, flexural reinforcement ratio and size effect are ignored in the comparisons given below. Ignoring the limitations on size effect and reinforcement ratio reduces the EuroCode 2 (2003) provisions to those of CEB-FIP MC 90 and they will be evaluated as a single model.

Interior connections without moment transfer

Comparison of the code provisions with experimental results is not straight forward because the code expressions were developed to be conservative and use specified or characteristic