

Punching Shear in Reinforced Concrete Slabs

Editor

Maria Anna Polak



American Concrete Institute®
Advancing concrete knowledge

SP-232

First printing, October 2005

DISCUSSION of individual papers in this symposium may be submitted in accordance with general requirements of the ACI Publication Policy to ACI headquarters at the address given below. Closing date for submission of discussion is March 2006. All discussion approved by the Technical Activities Committee, along with closing remarks by the authors, will be published in the July/August 2006 issue of either the *ACI Structural Journal* or *ACI Materials Journal* depending on the subject emphasis of the individual paper.

The Institute is not responsible for the statements or opinions expressed in its publications. Institute publications are not able to, nor intended to, supplant individual training, responsibility, or judgment of the user, or the supplier, of the information presented.

The papers in this volume have been reviewed under Institute publication procedures by individuals expert in the subject areas of the papers.

Copyright © 2005
AMERICAN CONCRETE INSTITUTE
P.O. Box 9094
Farmington Hills, Michigan 48333-9094

All rights reserved, including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by any electronic or mechanical device, printed or written or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.

Printed in the United States of America

Editorial production: Lindsay K. Kennedy

Library of Congress catalog card number: 2005934399
ISBN: 0-87031-193-X

PREFACE

The development of high-performance materials and advanced computational tools has allowed the building of relatively thin concrete slabs supported on columns. The simple appearance of such structural systems and ease of their construction make them economically attractive and popular. However, these structures develop complex three-dimensional stresses in the slab, at the columns, which can eventually lead to a potentially catastrophic brittle punching shear failure. Although structural failures are rare, in part due to high safety factors, understanding punching shear phenomenon is crucial for safe and rational design of flat reinforced concrete slabs supported on columns.

As part of the activities of the ACI/ASCE Committee 445, Shear and Torsion, members of Subcommittee 445-C, Punching Shear, organized a symposium in conjunction with this special publication devoted to the state of the art on punching shear. The symposium was held during the ACI Fall 2005 Convention under the sponsorship of ACI/ASCE Committee 445. The papers in this volume were reviewed in accordance with ACI review procedures.

The last significant and comprehensive overviews on this topic were completed several years ago by the *fédération internationale du béton (fib)* in their state-of-the-art report on punching shear (2001)¹ and in the Proceedings of the International Workshop on Punching Shear Capacity of RC Slabs (2000)² published by the Royal Institute of Technology in Sweden. Subcommittee 445-C has compiled a series of papers that present updated developments in the state of the art and research regarding behavior, rational design, and evaluation of code provisions related to punching shear. The papers are grouped into two parts. The first part contains a report, coauthored by members of Subcommittee 445-C, on issues related to design philosophy, code provisions, contributions of flexural and shear reinforcements, and seismic and blast loads. The second part includes papers on new developments obtained from different research centers from around the world. The aim is to present comprehensive and objective information on the topic of punching shear.

It is hoped that this publication will be important for the engineering design community in its efforts to improve long-term strength and ductility of slab-column structural systems.

Maria Anna Polak
Editor and Chair of Subcommittee 445-C

¹ “Punching Shear of Structural Concrete Slabs,” *Bulletin* No. 12, *fédération internationale du béton (fib)* Lausanne, Switzerland, 2001, 307 pp.

² Silwerbrandt, J., and Hassanzadeh, G., eds., “International Workshop on Punching Shear Capacity of RC Slabs,” *Bulletin* No. 57, Royal Institute of Technology, Department of Structural Engineering, Stockholm, Sweden, 2000, 527 pp.

TABLE OF CONTENTS

Preface	iii
---------------	-----

Part I

SP-232—1: 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 1 by N.J. Gardner	
SP-232—2: ACI 318 Moment Transfer Strength and Stiffness Considerations..... 23 by N.M. Hawkins	
SP-232—3: Effects of Size, Geometry and Material Properties on Punching Shear Resistance 39 by D. Mitchell, W.D. Cook, and W. Dilger	
SP-232—4: Effect of Flexural Reinforcement on Punching Shear Resistance 57 by W. Dilger, G. Birkle, and D. Mitchell	
SP-232—5: Shear Reinforcement for Concrete Flat Slabs 75 by M.A. Polak, E. El-Salakawy, and N.L. Hammill	
SP-232—6: A Design Perspective on Punching Shear 97 by S.D.B. Alexander and N.M. Hawkins	
SP-232—7: Reinforced Concrete Slabs Subjected to Blast or Localized Impact 109 by T. Krauthammer	
SP-232—8: Slab-Column Connections Under Seismic Actions 127 by W.H. Dilger, D.C. Dechka, and S.J. Brown	

Part II

SP-232—9: Interior Slab-Rectangular Column Connections Under Biaxial Lateral Loadings 147 by Y. Tan and S. Teng	
SP-232—10: Effect of the Column Dimensions on the Punching Shear Strength of Edge Column-Slab Connections 175 by A.G. Sherif, M.B. Emara, A. Hassanein, and S.A. Magd	
SP-232—11: Punching Shear Strength of Post-Tensioned Concrete Flat Plates 193 by N.J. Gardner	

SP-232—12: Punching of Reinforced Concrete Flat Slabs—ACI and German Guidelines 209
by J. Hegger, A. Sherif, and R. Beutel

SP-232—13: Punching Research at the Royal Institute of Technology (KTH) in Stockholm 229
by H. Sundquist

SP-232—14: Eurocodes and North American Codes Predictions of Punching Shear Capacity in View of Experimental Evidence 257
by A. Pisanty

SP-232—15: Effect of HPC and FRHPC Precast Heads on Punching Shear Resistance of Flat Plates 277
by A.B. Ajdukiewicz and J.S. Hulimka

Part I

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 Zaghoor (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 $1.5d$, $1.5d$, and $2.0d$ 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

2 Gardner

ACI Fellow N. J. Gardner is a member of ACI Committees 209 Shrinkage and Creep, 231 Early-Age Properties of Concrete, 347 Formwork and 435 Deflection of Concrete Building Structures. His research interests are shrinkage and creep of concrete, early age loads due to the construction process, early age behaviour of reinforced concrete members, and formwork pressures.

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.

$\text{Specified loads} \times \text{load factor} < \text{resistance} \times \text{partial safety factor} \times \text{predicted resistance}$

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 $1.35 \times 1.5 = 2.025$ for dead load.

4 Gardner

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 $1.4 \times 1.25 = 1.75$. 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 $1.75 \times 1.15 = 2.013$.

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:

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

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

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

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.

$$v_f = \frac{V_f}{A_c} \quad (3)$$

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

Punching Shear in Reinforced Concrete Slabs 5

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:

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

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:

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

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

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

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

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

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 $1300/(1300 + d)$.

6 Gardner

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)

$\alpha_s = 4$ 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.

$$v_r = v_c = \beta \lambda \phi_c \sqrt{f'_c} \quad (7)$$

$\beta = 0.21$ 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;

$$\sum A_{sb} \geq 2V_{se} / f_y \quad (8)$$

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.

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

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:

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

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:

$$v_{max} = 1.25 V_f / u d \quad (11)$$

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:

$$v_f = \frac{V_f}{u_l d} + \frac{KM_{fu}}{W_l d} \quad (12)$$

Where

$$W_l = \int_0^{u_l} |e| dl \quad (13)$$

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

8 Gardner

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:

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

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

$$v_f = \beta \frac{V_f}{u_1 d} \quad (15)$$

$u = \pi(c + 3d)$ (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 $\beta = 1.05$ for an

Punching Shear in Reinforced Concrete Slabs 9

interior column connections, 1.4 for edge column connections and 1.5 for corner column connections.

The shear strength is given by;

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

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

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

$$< 0.02$$

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

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

Eurocode 2-2003

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

$$v_f = \beta \frac{V_f}{u_1 d} \quad (17)$$

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

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.

$$W_1 = \int_0^{u_1} |e| dl$$

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, $\beta = 1$, 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

10 Gardner

the columns, using equation (15) with $\beta = 1.15$ for an interior column connection, 1.4 for edge column connections and 1.5 for corner column connections.

The shear strength is calculated as;

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

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

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

concrete strength, depending on the code, not the mean concrete strength reported for experimental studies. The Punching Shear Working Group of *fib* Commission 4 developed a data bank of 205 well documented punching shear results for concentrically loaded specimens. The Working Group eliminated 14 results which were attributed to flexural failure and 13 results for specimens with shear spans less than 4. Results for slabs which had compression reinforcement were not used reducing the number of results used in this study to 132. The validity of the no moment transfer, punching shear provisions ACI 318-05, CSA A23.3-04, BS 8110-97, CEB-FIP MC 90 and DIN 1045-1 (2001) have been evaluated using the information from the data bank

The code punching shear predictions were calculated using the reported mean concrete cylinder strengths, f_{cm} , in place of the specified strength in the ACI and CSA equations and the characteristic strength, f_{ck} , in the CEB-FIP, BS8110, DIN 1045-1 and EuroCode 2-2003 equations. The ratios of the measured to predicted punching shear force are plotted against mean concrete strength in Figures 4, 5, and 6 for ACI, CSA and Figures 7, 8 and 9 for BS 8110, CEB-FIP MC90/EuroCode 2 and DIN 1045-1 respectively. There is no need for restrictions on the range of applicability of any of the prediction equations with concrete strength.

The means, standard deviations and coefficients of variation are given in Table 2. The coefficients of variation of the ACI and CSA equations are much larger than those of BS 8110, CEB-FIP 1990 Model Code/Eurocode 2 (2003) and DIN 1045-1 (2001-07). This is due to the lack of reinforcement ratio and size effect terms in the ACI and CSA equations.

To demonstrate the difference in the definition of the concrete strength between the various codes the punching capacity statistics were re-calculated assuming $f_c' = f_{cm} - 2.4$ MPa and $f_{ck} = f_{cm} - 4.0$ MPa. The statistical indicators of these calculations are given in Table 3. As the specified/characteristic strengths are less than the mean strengths, the mean test/predicted and 5% fractile test/predicted values are larger than those in Table 2. Surprisingly only ACI 318 with rounded corners shear perimeter satisfies the requirement of a 5% fractile value greater than one.

Interior columns with moment transfer

The data used was taken from the University of Alberta Research Report No.223 by Afhami, Alexander and Simmonds (1998). Figure 10 shows the ratio of test/predicted results plotted against a non-dimensionalised eccentricity ($M_u/(V_u * 25h)$) for interior columns (this implies that a full panel would have a span thickness ratio of 25). ACI 318 is consistently conservative except for one result. BS 8110 and CEB MC 90 are consistent, unconservative, but no less conservative than the no-moment interior column predictions. The DIN 1045-1 provisions should only be compared for eccentricities between 5% and 7% of span. At eccentricities of circa 5% DIN 1045-1 is unconservative and becomes less conservative with increasing moment. DIN 1045-1 should add a provision, similar to equation (18), for combined shear force and bending moment for interior column slab connections.

12 Gardner

Edge connections under gravity loads

The data used was taken from the University of Alberta Research Report No.223 by Afhami, Alexander and Simmonds (1998). Figure 11 shows the ratio of test/predicted results plotted against a normalised eccentricity ($M_u/(V_u * 25h)$) for edge columns which have moment transfer. ACI 318 is consistently conservative for edge column slab connections. The non-dimensionalised eccentricity for gravity loads for edge columns would be about 6% to 8%. BS 8110 and DIN 1045-1, which are similar in concept using a shear force multiplier, are not conservative for eccentricities between 6 and 8% and inappropriate for other eccentricities. The comparisons for CEB-FIP MC 90, which uses a reduced perimeter, are similar to those for BS 8110 and DIN 1045-1.

Corner connections under gravity loads

The data used was taken from Zaghoor (1971) and Walker and Regan (1987). The range of eccentricities for which results are available is limited. Figure 12 shows the ratio of test/predicted results plotted against a non-dimensionalised diagonal eccentricity ($M_u/(V_u * 25h)$) for corner column slab connections. Again the ACI eccentric shear moment interaction is consistently conservative. CEB, which uses a reduced perimeter, is also conservative. The CSA A23.3-05 one-way shear predictions are not conservative but better than those of CSA A23.3-94.

DISCUSSION OF COMPARISONS

As the 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.

BS 8110, CEB-FIP MC 90 and DIN 1045-1, which have size effect and reinforcement ratio terms and use control perimeters $1.5d$, $1.5d$ and $2.0d$ from the column, have smaller coefficients of variation than ACI 318 and CSA A23.3 for interior column slab connections. A control perimeter close to the column requires a strength enhancement term in the strength equation or the equations become conservative. For combined shear and moment transfer for edge and corner column connections with the moment vectors parallel to an edge the provisions of BS 8110, CEB-FIP MC 90 and DIN 1045-1 are only applicable for eccentricities less than 6% towards the interior of the slab. Presumably the reasons are that edge and corner column slab connections should not be assumed to contribute any frame resistance under lateral loads and the difficulties in determining an appropriate reinforcement ratio.

ACI 318 and CSA A23.3 should be modified to include size effect terms and the cube root of the concrete strength. A minimum flexural reinforcement ratio of 0.7% should be specified in regions susceptible to punching shear. The consequences of permitting use of a rectangular control perimeter have to be acknowledged.

The shear resistance coefficients of BS 8110, CEB-FIP MC 90 and DIN 1045-1 should be reduced by 17.6%, 26.6% and 23.5% respectively, from Table 3, to meet a 5% fractile value of unity. This would decrease the lack of conservatism for punching under

Punching Shear in Reinforced Concrete Slabs 13

combined shear force and gravity load bending moments for interior, edge and corner column slab connections. DIN 1045-1 should adopt the CEB-FIP MC 90 provision for combined shear force and bending moment for interior column slab connections.

Figure 13 shows the effect of reducing the coefficients in equations (9), (13) and (15) by 17.6%, 26.6% and 23.5% respectively and using equation (17) with equation (15) as a suggestion to DIN 1045 for calculation of punching capacity for interior column connections with moment transfer.

Correlations of predictions for interior connections without moment transfer show that incorporating the flexural steel ratio reduces the coefficient of variation. However for all cases other than concentric shear, determining the appropriate steel ratio is tenuous. At an interior connection the moment in one plane may cause reversal of the shear stress with consequent problems in determining the appropriate reinforcement ratio and the effect of flexural steel in the other plane, torsional faces, is not resolved. Edge column slab connections have different near-column steel ratios parallel and perpendicular to the slab edge. For edge and corner connections, where the eccentricity is towards the exterior, the bottom steel ratio would be appropriate but this steel may not be adequately anchored. The effects of flexural steel ratio parallel and perpendicular to the moment vector have to be established and codified.

It must be noted that while increasing the flexural steel increases the punching shear capacity, the behaviour of the connection becomes more brittle. Practically reinforcement should never be less than 0.5% and will rarely exceed 2% in real slabs.

CONCLUSIONS

The conclusions must be read noting that the median thickness of all concentrically loaded slabs used in the comparisons was only 140 mm with a maximum thickness of 320 mm. These slab thicknesses are smaller than slabs used in practice.

The code prediction methods to calculate the punching shear capacity of concentrically loaded slabs were compared to the 132 results in a data bank developed by the Punching Shear Working Group of *fib* Commission 4. The provisions of BS 8110-97, CEB-FIP 1990, MC/EuroCode 2-2003 and DIN 1045-1 have significantly smaller coefficients of variation than ACI 318-05 and CSA A23.3-04. However the 5% lower fracture results calculated for BS 8110-97, CEB-FIP 1990 MC/EuroCode 2-2003, CSA A23.3-04 and DIN 1045-1 are less than unity and un-conservative – they can be made conservative by reducing the equation coefficients by 17.6%, 26.6%, 19.0% and 23.5% respectively. However for combined shear and moment transfer for edge and corner column connections with the moment vectors parallel to an edge, BS 8110-97, CEB-FIP 1990 MC/EuroCode 2-2003 and DIN 1045-1 are only applicable for eccentricities circa 6%.

Anchorage of flexural reinforcement should be confirmed before assuming edge and corner column slab connections contribute any frame resistance under lateral loads perpendicular to the slab edge.

14 Gardner

The ACI 318 - CSA A23.3 eccentric shear interaction method, combined with a shear perimeter close to the column, is the **only** consistent code method of determining the effect of combined shear force and moment for punching shear for edge and corner column slab connections. Suggestions are given to improve the BS 8110, CEB-FIP MC90 and DIN 1045 predictions of punching strength for interior connections with combined shear and moment transfer. The problem of drift capacity under seismic displacements remains to be resolved.

All codes should adopt integrity steel provisions similar to those given in CSA A23.3 which minimise the possibility of punching shear at one connection leading to progressive collapse.

REFERENCES

ACI 318-05. Building Code Requirements for Reinforced Concrete, American Concrete Institute, Farmington Hills, Mich., p.430.

Afhai, A., Alexander, Scott D.B. and Simmonds, Sidney H. (1998), Strip model for capacity of slab-column connection. Structural Engineering Report No. 223, University of Alberta. P.231.

Bazant Zdenek P. and Cao, Zhiping (1987), "Size Effect in Punching Shear Failure of Slabs," ACI Structural Journal, V.84, No.1, Jan-Feb., pp.44-51.

BS 8110 (1997): Structural use of concrete, Part 1: Code of practice for design and construction, British Standards Institute, London, p.117.

CEB-FIP MC 90 (1993): Design of concrete structures. CEB-FIP-Model-Code 1990. Thomas Telford, 1993 construction. British Standards Institution, London. 2nd ed. 1997.

CSA A23.3-94 (1994), Design of Concrete Structure, Canadian Standards Association.

CSA A23.3-04 (2004), Design of Concrete Structure, Canadian Standards Association

DIN 1045-1 (2001-07): Plain, reinforced and prestressed concrete structures - Part 1: Design and construction. 122 pp. Normenausschuss Bauwesen (NABau) im DIN Deutsches Institut für Normung e.V. Beuth Verl. Berlin.

EC 2-1-1 (2003): Eurocode 2: design of concrete structures - Part 1: General rules and rules for buildings. prEN 1992-1-1. Final draft December 2003

Feld, Jacob, (1968) Construction Failures, published by John Wiley.

fib Bulletin 12 (2001): Punching of structural concrete slabs - technical report by the CEB/fib Task Group "Utilisation of concrete tension in design". *fib*, Lausanne, April 2001

Regan, P.E.; Braestrup, M.W. (1985): Punching shear in reinforced concrete – a state of the art report. CEB-Bull. 168. Lausanne. Jan. 1985

Punching Shear in Reinforced Concrete Slabs 15

Silfwerbrand, J.; Hassanzadeh, G. (Edts.) (2000): International Workshop on punching shear capacity of rc slabs - Proceedings. Kungl Tekniska Högskolan Stockholm, TRITA-BKN. Bull. 57.

Teng, S., Cheong, H.K., Kuang, K.L. and Geng, J.Z. (2004) Punching shear strength of slabs with openings and supported on rectangular columns. ACI Structural Journal, V.101, No.5, Sept.-Oct. pp.678-687.

Zaghool, E.R.F., (1971) Strength and behaviour of corner and edge column slab connections in reinforced concrete flat plates. Ph.D. Thesis, Department of Civil Engineering, University of Calgary. p.366

Walker P.R. and Regan, P.E., (1987) Corner column-slab connections in concrete flat plates. ASCE Journal of Structural Engineering, Vol.113, No. 4, April, pp.704-720.

NOTATION

b = side dimension of square column or loaded area, (mm)
 c = diameter of a circular column or loaded area (mm)
 d = average effective depth of tension reinforcement, (mm)
 d_x, d_y = effective depths of tension reinforcement in the x and y directions, (mm)
 f_{ck} = characteristic concrete cylinder strength, (MPa)
 f'_c = specified cylinder strength of concrete, (MPa)
 f'_{cm} = mean cylinder strength of concrete, (MPa)
 f_y = yield stress of reinforcement, (MPa)
 h = overall thickness of slab (mm)
 M_u = measured moment (N.mm)
 u = critical section for punching shear, (mm)
 v_r = factored shear stress resistance(MPa)
 v_f = factored applied shear stressforce (MPa)
 v_u = measured shear force (MPa)
 V_r = factored shear force resistance(N)
 V_f = factored applied shear force (N)
 V_u = measured shear force (N)
 β_c = ratio of longer to shorter dimension of the loaded area.
 λ = 1 for normal density concrete, 0.85 for lightweight concrete
 ρ = ratio of flexural tensile reinforcement calculated over a width $c + 6d$
 ρ_x, ρ_y = ratios of reinforcement in the x and y directions
 ϕ_c = CSA strength reduction factor for concrete = 0.65 (CSA A23.3-04)

Table 1– Fraction of unbalanced moment resisted by shear stress

c_1/c_2	0.5	1.0	2.0	3.0
K	0.45	0.6	0.7	0.8

K = fraction of the unbalanced moment M_f resisted by shear stresses

c_1 = column dimension parallel to the eccentricity of load

c_2 = column dimension perpendicular to the eccentricity of load

Table 2 – Statistical Indicators for Code Provisions for Concentric Punching Shear
(calculated using mean concrete strength)

Code	ACI 318-05	*ACI 318-05	*CSA A23.3-04	BS 8110-97	CEB-FIP MC90	DIN 1045-1
Mean	1.50	1.35	1.25	1.06	0.99	1.02
Standard Deviation	0.27	0.25	0.26	0.15	0.14	0.14
Coefficient Variation	0.18	0.19	0.21	0.14	0.14	0.14
5% fractile	1.06	0.93	0.82	0.82	0.76	0.78

*Square perimeter used

Table 3 – Statistical Indicators for Code Provisions for Concentric Punching Shear
(calculated using specified or characteristic concrete strength as appropriate)

Code	ACI 318-05	*CSA A23.3-04	BS 8110-97	CEB-FIP MC90	DIN 1045-1
Mean	1.63	1.31	1.12	1.04	1.07
Standard Deviation	0.33	0.29	0.17	0.15	0.16
Coefficient Variation	0.20	0.22	0.15	0.14	0.15
5% fractile	1.09	0.84	0.85	0.79	0.81

*Square perimeter used

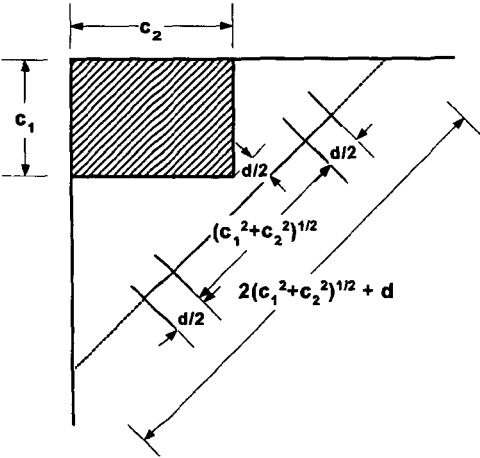


Figure 1— CSA A23.3-94 definition of one-way shear perimeter for corner column connections.

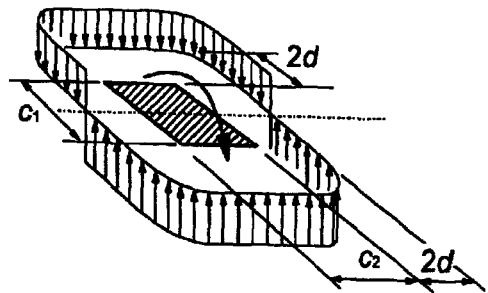


Figure 2 — Distribution of shear stress assumed by CEB-FIP MC 1990.

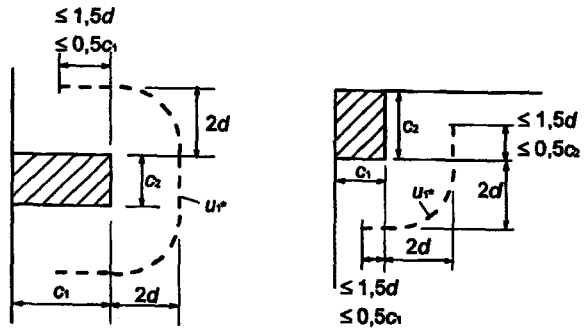


Figure 3 — Reduced shear perimeter permitted by CEB-FIP MC 1990.

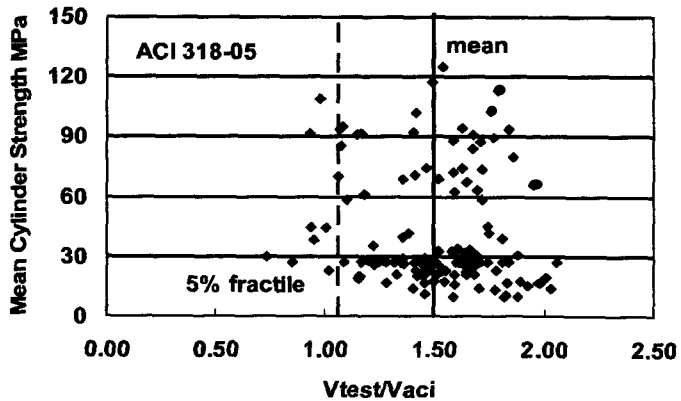


Figure 4 —Comparison of test/predicted using ACI 318-05.

18 Gardner

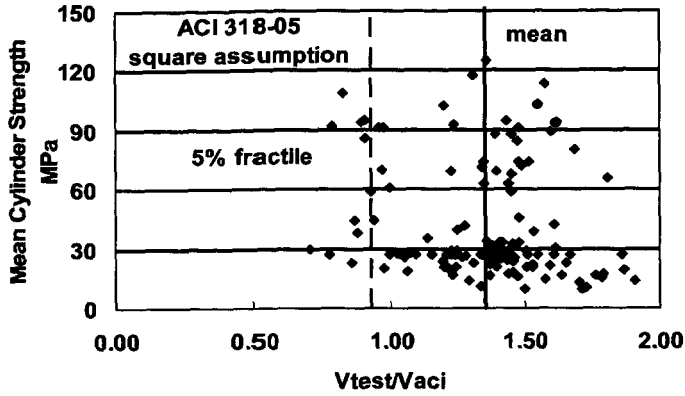


Figure 5 — Comparison of test/predicted using ACI 318-05 with assumption of square shear perimeter.

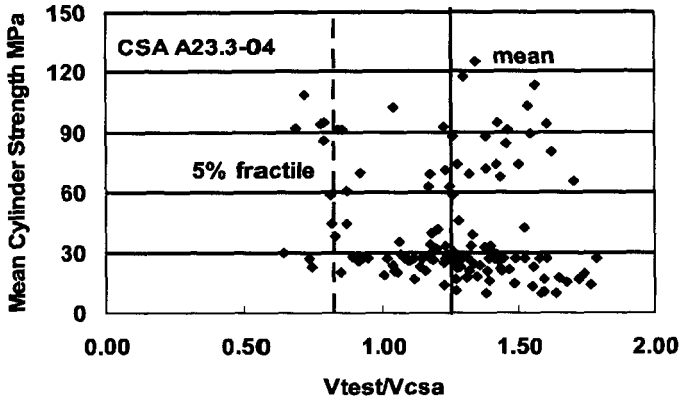


Figure 6 — Comparison of test/predicted using CSA A23.3-04.

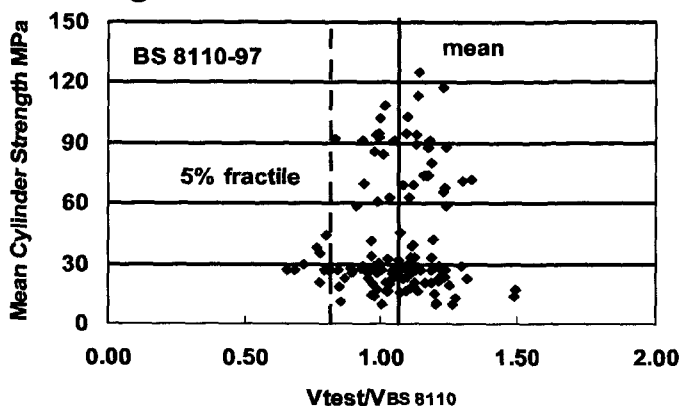


Figure 7 — Comparison of test/predicted using BS 8110-97.

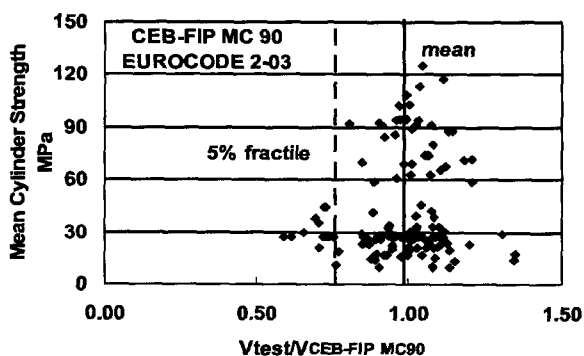


Figure 8 — Comparison of test/predicted using CEB-FIP MC90 and Eurocode 2-03.

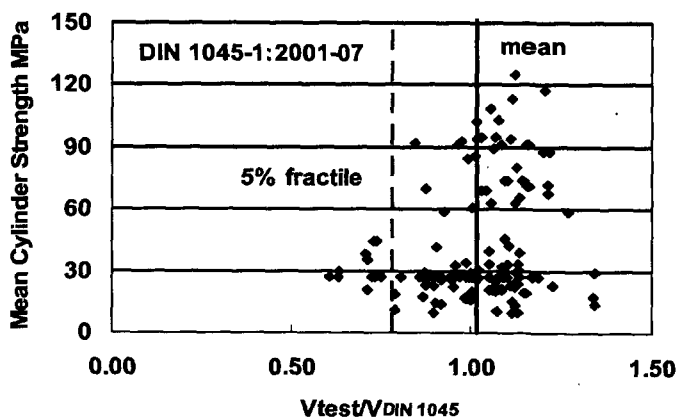


Figure 9 — Comparison of test/predicted using DIN 1045-1.

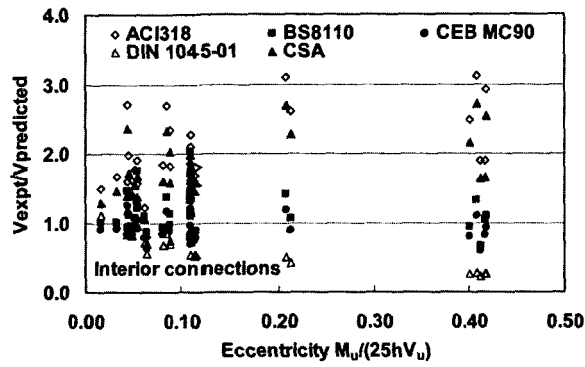


Figure 10 — Comparison of test/predicted for interior connections with moment transfer.

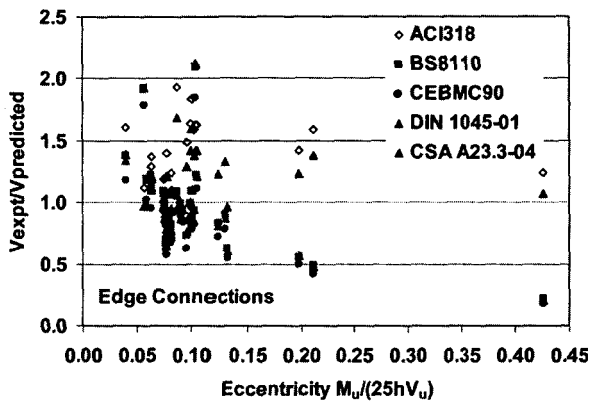


Figure 11 — Comparison of test/predicted for edge column slab connections.

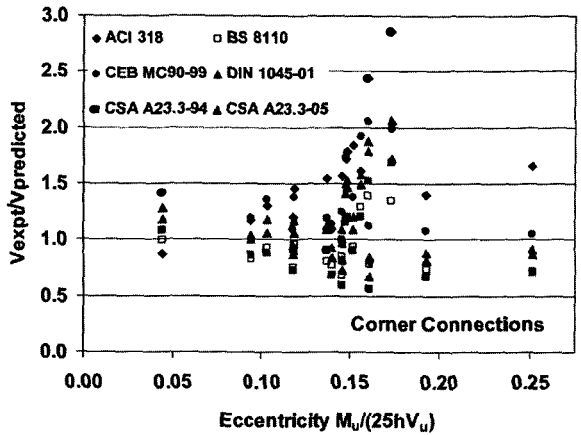


Figure 12 — Comparison of test/predicted for corner column slab connections.

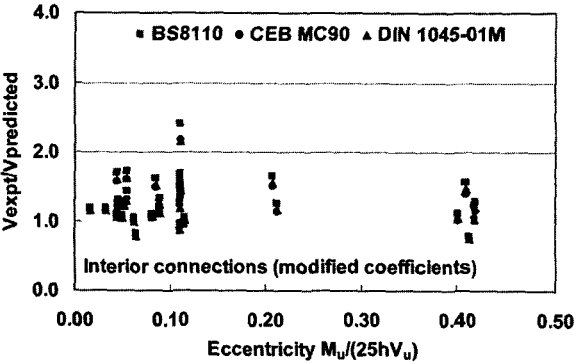


Figure 13 — Comparison of test/predicted for interior connections with moment transfer with prediction equations modified as proposed.

ACI 318 Moment Transfer Strength and Stiffness Considerations

by **N.M. Hawkins**

Synopsis: The 2005 Edition of ACI 318 contains a new provision in Sec. 21.11.5 that is aimed at reducing the potential for punching shear failures at columns in two-way slabs located in regions of high seismic risk or in structures assigned to high seismic performance or design categories. The provision requires that slab-column connections in two-way slabs without beams, where the slabs are assumed not to contribute to the lateral resistance of the structure, shall be provided with slab shear reinforcement unless: (1) it is shown that punching will not occur under the design shear and the moment induced between slab and column at the design displacement; or (2) the design story drift does not exceed a limiting value that is a function of the ratio of the design shear to the nominal shear strength of the connection under gravity loading.

This paper presents an overview of the ACI 318-05 provisions for moment transfer to columns, summarizes background information on the existing provisions and their relation to the new provision and summarizes existing information on how to calculate the lateral load stiffness of two-way slab structures in accordance with ACI Code 318-05 requirements. The paper describes how to utilize the provisions of Sec. 11.12.6 (shear resulting from moment transfer to columns) and Sec. 13.5.3 (reinforcement requirements for moment transfer to columns and moment redistribution between columns) to determine if the first alternative for application of the new provision can be satisfied. The paper summarizes the background information for the new provision and also discusses the relation between these provisions and those of Sec. 21.12.6 for intermediate moment frames consisting of two-way slabs without beams.

Keywords: concrete design codes; lateral loading; moment transfer; punching shear; seismic effects; slab-column connections

24 Hawkins

Neil M. Hawkins, FACI, is Professor Emeritus, Department of Civil Engineering, University of Illinois, Urbana, Illinois, USA. He is on ACI 318 and chairs its Task Group on Seismic Design of Piles. He is a member of Committees 215, Fatigue of Concrete; 408, Bond and Development of Reinforcement; 445, Shear and Torsion; 446, Fracture Mechanics; 550, Precast Concrete Structures; and ITG-5, Precast Shear Walls for High Seismic Applications. He also chairs the concrete subcommittees of the Seismic Task Group of ASCE/SEI 7 on Minimum Design Loads for Buildings and of the BSSC that develops the NEHRP Recommended Seismic Provisions for New Buildings.

Requirements for Transfer of Moments to Columns in Chapter 11 of ACI 318

Procedures for calculating the shear strength of slab-column connections transferring moment were first introduced into ACI 318 in the 1971 Edition in Sec. 11.12.2 as a result of tests by Hanson and Hanson (1971). They investigated experimentally a theoretical approach developed by DiStasio and Van Buren (1960) and described in the Commentary to the slab provisions of the 1963 Code. Most of the data available when ACI 318-71 was being prepared were for square columns where about 60 percent of the moment was transferred by flexure across the critical section used for punching shear evaluations and 40 percent by eccentricity of the shear about the same critical section. To allow for the use of rectangular columns it was assumed that the portion of the moment transferred by shear would decrease as the width of the face of the critical section resisting the moment increased. Accordingly the fraction of the moment transferred by shear, γ_v , was specified in the shear provisions as:

$$\gamma_v = 1 - [1 / (1 + 2/3 \sqrt{\{c_1 + d\} / \{c_2 + d\}})] \quad (1)$$

The resulting assumed linear distribution of shear stress is as shown in Fig.1 where $c_1 + d$ is the width of the critical section in the direction of moment transfer and $c_2 + d$ is the width transverse to that direction. The maximum factored shear stress is calculated from:

$$v_{u(AB)} = V_u / A_c + \gamma_v M_u c_{AB} / J_c \quad (2)$$

A_c is the concrete area of the assumed critical section and J_c is a property of the critical section analogous to the polar moment of inertia. For an interior column connection

$$A_c = 2d(c_1 + c_2 + 2d) \text{ and } J_c = [d(c_1 + d)^3 + (c_1 + d)d^3 + 3d(c_2 + d)(c_1 + d)^2] / 6 \quad (3)$$

ACI 318R notes that similar expressions can be developed for A_c and J_c for columns located at the edge or corner of the slab. However, those expressions are not provided.

The maximum shear stress $v_{u(AB)}$ is not to exceed $\phi V_c / b_o d$ where V_c is the punching shear strength when there is no moment transfer. For reinforced concrete slabs ACI 318-05 contains three expressions for V_c and the designer is to use the smaller value. However, the moment transfer provisions were derived using the V_c value of $4\sqrt{f'_c} b_o d$ and it is conservative to use the V_c value of the other two expressions. For prestressed concrete slabs there are again effectively three expressions for V_c and again the moment transfer

Punching Shear in Reinforced Concrete Slabs 25

expressions were derived using the V_c value of $(3.5\sqrt{f'_c} + 0.3f_{pc})b_o d + V_p$. It is conservative to use the V_c value from the other two expressions.

The fraction of the moment $(1 - \gamma_v)M_u$ not transferred by eccentricity of shear was to be transferred by flexure across the same critical section. In the 1977 Edition of ACI 318 tests reported subsequently by Hawkins et al. (1989) were used to liberalize the limitation in Chapter 13 of the width within which reinforcement could be placed for moment transfer. All slab reinforcement within lines one and one-half times the slab or drop panel thickness, $1.5h$, on each side of the column were permitted to contribute to that moment. That resulting width can be conveniently termed the column-head width. The provisions of Chapters 11 concerning calculations of the moment transfer strength have subsequently remained unchanged through ACI 318-05. However, provisions governing determination of the fraction of the moment not transferred by eccentricity of shear, and specified in Chapter 13 and subsequently also in Chapter 21, have continued to evolve.

It has long been realized that this linear variation in shear stress method does not provide an accurate model of the manner in which shears and moments are transferred to columns (ACI-ASCE Committee 426, 1974, 1977). For the test results available through 1977 the procedure was found to underestimate shear strengths for square interior columns with column-head slab reinforcement ratios greater than 0.8 percent, and rectangular columns with a c_2+d dimension significantly greater than the c_1+d dimension. For edge and corner columns the underestimations in shear strength with increasing reinforcement ratios were even greater. The reason is that the ACI procedure underestimates the moments and shears that can be transferred through torsional actions at the c_1+d faces. However, prior to failure, slabs with column-head reinforcement ratios greater than 0.8 percent showed little ductility while slabs with less than 0.8 percent reinforcement showed increasing amounts of ductility with decreasing reinforcement ratios.

The linear variation in shear stress model was developed prior to modern computer programs that now often predict biaxial moment transfer conditions for some connections, even for gravity loadings only. Such biaxial moment transfer conditions can arise due to pattern loads, unequal adjacent spans, and similar factors, in addition to lateral loadings. ACI 318-05 is silent as to how to account for biaxial moment transfer conditions. Many researchers and several ACI Committees have suggested the addition to Eq. (1) of a second term for the orthogonal moment effect. However, that approach means that the maximum v_u value occurs at only one location on the perimeter of the critical section whereas for the uniaxial moment transfer situation that maximum v_u occurs along the length of one face of the critical section. Thus, the latter is much more likely to lead to failure than the former and ratios of measured to calculated strengths for biaxial moment transfer tests and capacities computed using two moment terms in Eq. (1) confirm that fact (Akiyama and Hawkins, 1984). When the moment in one direction is small compared to that in the other direction, the effect of the smaller moment is almost negligible. Based on the test results existing in 1977, Committee 426 recommended that if the moment to be transferred to the column in either direction is less than $0.2 V_c d$, then the effect of that moment transfer can be neglected in evaluations of shear capacity. More recently Hueste and Wight (1999) have made a similar recommendation by

26 Hawkins

concluding that the effect of moment transfer on the shear capacity can be neglected if the rotation caused by that moment is less than 0.005 radians.

Adjustment of Magnitude of Moments

In ACI 318-95 Sec.13.5.3.3 was introduced allowing the designer to adjust downward the level of moments transferred by shear without revising member sizes. The adjustment is permitted for reinforced but not for prestressed slab systems. The adjustment recognizes that the moment transfer model of Fig.1 is empirical. For specimens where the slab reinforcement is yielding in the vicinity of the column prior to punching failure, the moment transferred-connection rotation relationship is markedly non-linear, and the validity of the Fig. 1 model is questionable. The column-head width over which the reinforcement is effective in transferring moments to the column is greater than $c_1 + 3h$.

At exterior supports, for unbalanced moments about an axis parallel to the edge, (the situation shown in Fig. 1(b)), Sec. 13.5.3.3 allows the portion of the moment transferred by eccentricity of shear $\gamma_v M_u$ to be reduced, (and the moment to be transferred by flexure correspondingly increased), provided the factored shear V_u at the support does not exceed $0.75\phi V_c$ for edge, or $0.5\phi V_c$ for corner, columns. V_c is the punching shear strength when there is no moment transfer. Analysis of test data (ACI-ASCE Committee 352, 1988; Moehle, 1988) has shown that there is no significant interaction between shear and unbalanced moment at exterior supports for moment transfer normal to the edge. Either the connection fails at its pure shear strength or at its pure flexural strength, whichever occurs first. From frame tests the model of Fig. 1(b) was found to be very conservative.

For unbalanced moments about an axis normal to the edge at exterior supports, and for interior columns, Sec. 13.5.3.3 also allows some redistribution of moments but with more severe limitations. The value of $(1-\gamma_v)$ may be increased by up to 25 percent provided V_u does not exceed $0.4\phi V_c$ and the column-head reinforcement ratio does not exceed $0.375\rho_b$. When moments are reversed for opposite faces of an interior support the top reinforcement for one face, and the bottom reinforcement for the opposite face, are both effective. Thus, this provision can be utilized only if both the shear force transferred to the column is low and the reinforcement ratio within the column-head width is low so that yielding of that steel will occur before the capacity of the connection is reached. Further, moment adjustments are permitted for reinforced concrete slabs only. No adjustments are permitted for moments in prestressed concrete slabs.

The limitations of this part of the provision were based on an analysis of data derived from tests by Hawkins et al. (1989), Islam and Park (1976), Pan and Moehle (1989,1992), and Robertson and Durrani (1992).

Stiffness and Moment Determinations

For gravity loadings, the total static loading acting on the slab must be transferred to the column supports by shears. The moments transferred along with the shears, and the effect of moments on the distribution of shears between supports, depends on the stiffness

characteristics of the slab to column connections. Accurate modeling of those stiffness characteristics is essential to the realistic determination of the moments and shears acting on each connection, and therefore the factor of safety against failure at each connection. For many years Chapter 13 of ACI 318 has provided two methods of analysis for flat plate and flat slab systems; the direct design method and the equivalent frame method. The specific provisions of both design methods are limited to orthogonal frames subject to gravity loads and the resulting distribution of moments to the critical sections of the slab reflects the effects of reduced stiffness values due to concrete cracking and support geometry. However, it must also be recognized that for gravity loadings, and the slab span to depth ratios for which those procedures were derived, the stiffness values inherently include concrete tension stiffening effects.

When the lateral load due to wind or seismic effects acts on a reinforced concrete flat plate or flat slab system, the slab is already cracked due to gravity loadings and volume changes. Few tension-stiffening effects remain for increased stress levels. To assure that drifts are not underestimated, reinforced concrete slabs should be taken as fully cracked. However, prestressed concrete slabs are not as fully cracked as reinforced ones and stiffness values greater than the fully cracked stiffness are appropriate. For reinforced concrete slabs of unbraced frames an equivalent width of between 25 and 50 percent of the full panel width is often used to reflect the stiffness reduction due to cracking. From their tests on interior column-slab sub-assemblages, Pan and Moehle (1992) found that biaxial lateral loading reduced the lateral stiffness, strength, and ductility of connections and that based on a 0.2% drift criterion, an effective beam width equal to one-third of the elastic value gave a close estimate of the stiffness of their specimens. Alternatively, for the equivalent frame model a stiffness reduction factor of one-third gave a good estimate of the lateral stiffness. Where there are no edge beams the use of a constant effective beam width for the length of a frame overestimates the frame stiffness and the moment transferred at exterior column supports because for that location the effective width can be no greater than the column-head width. Further, for exterior column supports of prestressed concrete slabs only the tendons anchored through the column are effective in applying prestress to the slab. For prestressed concrete slabs the stiffness at exterior column supports is unlikely to be significantly greater than that for a reinforced concrete slab. Additional guidance on selecting stiffness values for lateral loadings are provided in Carpenter et al. (1973), Vanderbilt and Corley (1983), Akiyama and Hawkins (1984), Pan and Moehle (1992), and Hueste and Wight (1999).

During the 1970s an extensive series of column-slab sub-assemblage and frame tests were conducted at the University of Washington (Akiyama and Hawkins, 1984). Test specimens represented each of the hatched slab areas shown in Fig. 2. The scope of the tests and the principal variables are summarized in Table 1. Typical specimen dimensions and results for an exterior column connection with moment transfer normal to the edge, and for an interior column connection, are shown in Figs. 3 through 6, respectively (Hawkins, 1980). Major changes in stiffness were observed with torsional cracking at the c_1 faces in Fig. 1 and with top bar yielding at the c_2 face in Fig. 1. Once torsional cracking occurred specimens with shear reinforcement showed greater stiffness than those without such reinforcement. Further a substantial portion of the rotation of the

28 Hawkins

connection was associated with bond slip of the slab reinforcing bars anchored within the column and transferring a portion of the moment not transferred by shear. However, as apparent from Figs. 4 and 6, the largest fraction of the moment transfer was through torsion at the c_1 faces. The results of a stiffness model for the various elements of a connection, (torsion, bending, and bond slip), lead to the unbroken line responses for loadings to failure as shown in Figs. 4 and 6. A cyclic loading version of that model and an element that could be added to the computer program DRAIN-2D (Powell, 1973) were developed and used to predict the capacities and responses up to failure of the sub-assemblages and frames shown in Table 1 (Akiyama and Hawkins, 1984).

During the 1971 San Fernando earthquake, the Holiday Inn, Orion Avenue, Los Angeles, suffered considerable non-structural damage due to excessive displacements. However, there were no punching failures at the slab-column connections. The building was a seven story flat plate structure 3 bays wide in the transverse direction with 18-inch square columns at 20ft 6in. centers on average. The reinforced concrete slab was 10 inches thick at the second floor and 8.5 inches thick for the third through seventh floors. There were 16 inch wide by 22 inch deep spandrel beams on the perimeter of the building. Strong motion accelerographs recorded earthquake motions at the ground, fourth floor and roof levels. For the first 6 seconds of motion the fundamental period of the building in each direction was about 0.7 seconds. However, after 9 seconds the fundamental period changed to 1.5 seconds and displacements increased markedly. Akiyama and Hawkins (1984) modeled the transverse direction of the building with a beam analogy grid for the column-head region and examined the effects on the structure of gravity loading and the subsequent earthquake motions using their slab-column connection model. The measured ground motion for the first floor was used as input to that model and predicted responses compared with measured responses for the fourth floor and roof. Good agreement was obtained between measured and calculated responses. For all floors below the fifth floor, calculated relative deformations exceeded 1 inch and were as large as 1.6 inches (1.5% drift). That pattern was consistent with the damage to the building. At between 6 and 9 seconds it was predicted that the negative moments at the interior column faces exceeded the yield moments, causing the period to lengthen and displacements to increase. However, none of the same connections were predicted to reach positive moment yielding allowing the structure to maintain its capacity.

During the 1994 Northridge earthquake, the Four Seasons Building in Sherman Oaks suffered considerable damage at the column-slab connections of the interior gravity load carrying floor system. There were nine such connections per floor. Seven on the first floor, nine on the second floor and two on the third floor showed punching shear damage (Hueste and Wight (1999), and Bondy (2004)). The lateral load resisting system was continuous perimeter special moment frames that performed satisfactorily in the earthquake. In Hueste and Wight a stiffness and punching failure model is reported that can be used with the DRAIN-2DM computer program to predict the seismic response of slab-column frames. The program DRAIN-2DM (Tang and Goel, 1988) is a modification of the DRAIN-2D program. Hueste and Wight used that model to successfully predict the response and punching failures of the Four Seasons building and the loss in rotational stiffness of the connections of that building caused by the punching failures.

Punching Shear in Reinforced Concrete Slabs 29

For the seismic design of a given structure the information in the three foregoing sections provides the background needed for a designer to determine if the column-slab connections of the gravity load-carrying frame of that structure are likely to punch under the shear and moment induced in that frame when the lateral-force resisting system for the structure reaches its design displacement. However, two provisos need to be added to that conclusion. The first is that the laboratory test data suggests that the limiting shear capacity V_c when there is no moment transfer decreases when the slab at the critical section location becomes cracked through its depth due to moment reversals. That decrease in capacity is limited to about 15%. Second, little information is available from laboratory tests on the seismic performance of connections with shear drop panels. The Four Seasons building had such drop panels and the information reported by Bondy (2004) suggests that the critical section for shear was not located at $d/2$ outside the perimeter of the drop as would normally be assumed when the stress distribution of Fig. 1 is used. Rather the critical section was located in the slab at a distance $d/2$ beyond the location where the 45-degree line drawn from the column-drop panel soffit intersection met the soffit of the slab. Thus, the shear drop panel was less effective than would normally be assumed. That observation is consistent with degradation having occurred in the drop panel due to cracking caused by loading reversals.

Two-Way Slabs Without Beams and Not Designated as Part of the Lateral-Force-Resisting System

For structures in regions of high seismic risk or for structures assigned to high seismic performance or design categories, ACI 318-05 contains a new provision affecting the detailing of connections of flat-slab gravity load systems paralleling special structural walls or special moment frames that resist seismic forces. The provision is intended to replace a similar, but different provision, that is part of IBC 2003. The IBC provision was based on a concept developed initially by Robertson and Durrani (1992), expanded by Pan and Moehle (1992), and formalized by Megally and Ghali (2000). The ACI provision combines that IBC concept with the other ACI 318 requirements described earlier. The ACI provision appears as a new Sec. 21.11.5 that is added to Sec. 21.11.

Shown in Fig. 7 is the plot justifying the new provision. Test data from numerous sources, shown by various symbols in Fig. 7, indicate clearly that the lateral drift capacity of interior reinforced concrete slab-column connections decreases continuously as the intensity of the gravity load shear acting simultaneously on the connection increases. Three curves of best fit to the test data are shown on Fig. 7. Curve 1 is for connections without shear reinforcement. Curve 2 is for connections with shear reinforcement in the form of closed stirrups, and the uppermost curve, Curve 3, is for connections with stud shear reinforcement. While the data of Fig. 7 show that studs are more effective as shear reinforcement than closed stirrups, Robertson et al. (2002) have shown that may not necessarily be true. They found open stirrups with clip closure, and single leg stirrups, to be as effective as studs when anchored around slab bars for connections that, without shear reinforcement, had shear capacities close to their flexural capacity.

30 Hawkins

Also shown on Fig. 7 are lines corresponding to 1.5%, 2% and 2.5% drift. The 2.5% value is a common design displacement for structures with fundamental periods less than 0.7 sec. The 2% value is a common design displacement for structures with fundamental periods of 0.7 sec or more. ACI-ASCE Committee 352 (1988), Pan and Moehle (1989), and Hueste and Wight (1999) have all advocated the 1.5% as the minimum drift ratio that slab-column connections should be able to undergo without failure.

The concepts of Fig. 7 allow for a simple procedure that does not require calculation of induced moments but does ensure that punching shear failures do not occur as a result of seismic motions. For seismic design, the designer selects a seismic-force-resisting system, applies the seismic forces to that system and calculates the design displacement. With that displacement the designer can use a provision that is the equivalent of Fig. 7 to determine if punching shear is likely and therefore if shear reinforcement should be used at the slab-column connections. The actual relationship specified in ACI 318-05 is shown in Fig. 8. The relationship is a conservative lower bound to the test data for interior column-slab connections without shear reinforcement.

The ACI 318-05 Sec. 21.11.5 requires that for slab-column connections of two-way slabs without beams, slab shear reinforcement satisfying the detailing requirements of Sec. 11.12.3 and providing a shear strength contribution V_s not less than $3.5\sqrt{f'_c}b_o d$ shall extend at least four times the slab thickness from the support unless either (a) or (b) is satisfied:

- (a) the maximum shear stress calculated according to the concepts of Fig.1, using the design shear V_u and the induced moment M_u transferred between the slab and column under the design displacement, is less than the nominal shear strength of the connection;
- (b) The design story drift ratio does not exceed the larger of 0.005 and $[0.035 - 0.05(V_u/\phi V_c)]$.

The design story drift is to be taken as the larger of the design story drift ratios of the adjacent stories above and below the slab-column connection, V_c is the nominal shear strength provided by the concrete when there is no moment transfer and V_u is the factored shear force for two-way action. V_u is calculated for the load combination of $1.2D + 1.0L + 0.2S$. Where column capitals, drop panels, or other changes in slab thickness occur, the requirements need to be evaluated for all potential critical sections.

The provision of Sec. 21.11.5 allows the designer two options. The shear V_u and the moment M_u can be calculated and a determination made as to whether shear reinforcement is necessary. Option (b) is to use Fig. 8 for that determination. For option (a), selection of an appropriate stiffness value for the slab-column connection is necessary for calculation of M_u . Factors to be considered in that selection have been discussed previously. Use of option (a) allows the designer to not require shear reinforcement at all connections but only at those connections where needed to satisfy nominal shear strength requirements.

Option (b) offsets the need to calculate M_u and therefore the difficulties of selecting appropriate stiffness values for slab-column connections. Once the design shear, V_u , the nominal shear strength provided by the concrete, V_c , and the design displacement are known, the designer uses Fig. 8 to determine if shear reinforcement is required. If shear reinforcement is required, the amount must satisfy that specified. Justification for that amount is provided in Megally and Ghali (2000). Alternatively the designer could increase the slab thickness or reduce the design story drift to where shear reinforcement is not required.

There are several limitations of this provision that should be appreciated. First, option (b) requirements are based solely on information for reinforced concrete slabs. Little information is available on the reversed cyclic loading behavior of prestressed slab-column connections. However, data that are available (Hawkins, 1981) indicate that requirements of option (b) are likely to be conservative. Use of option (a) is likely to be a better approach for prestressed slabs although punching shear failures still occurred in the lightweight concrete prestressed slabs of the Four Seasons building, (Bondy, 2004). Second, option (b) requirements are based on information for interior column connections. Shown in Fig. 9 is the plot from Megally and Ghali (2000) similar to that of Fig. 7. Figure 9 shows capacities for exterior column connections where moments are transferred about an axis parallel to the slab edge. The broken line on that figure is the relationship specified in option (b). Clearly the use of option (b) is very conservative for exterior column connections. In many such cases it will be possible to waive shear reinforcement requirements for such connections.

For the typical interior column connection of a typical floor of the Holiday Inn, Orion Ave, the load combination specified in Sec. 21.11.5 results in a V_u of 57.1 kips. For the specified concrete strength for those floors of 3,000 psi the V_c value of the interior connection is 167.6 kips. The V_u/V_c value is 0.34 and therefore from Fig. 7 failure was not to be expected when the maximum inter-story drift was 1.6% as calculated by Akiyama and Hawkins (1984). However, the relationship of Fig. 7 predicts that shear reinforcement should have been provided if the inter-story drift reached 1.8%. In the analysis by Akiyama and Hawkins it was found that the moments and forces calculated as acting on a typical connection had reached their limiting values for all faces of the connection except that where positive moments were created by the lateral loading. For that face the calculations indicate that the imposition of 20% larger drifts would cause the limiting moment and shear capacities to be reached and the connection to fail. Again, this result is consistent with the requirements of Sec. 21.11.5 and with the findings by Hueste and Wight for the Four Seasons building.

Use of Two-Way Slabs Without Beams as Intermediate Moment Frames

Requirements concerning the use of two-way slabs without beams for frames in regions of moderate seismic risk were first incorporated into ACI 318 in its 1983 Edition. The terminology “frames in regions of moderate seismic risk” was changed to “intermediate moment frames” in the 1995 Edition. The relevant provisions now appear in Sec. 21.12.6. All reinforcement to resist seismic effects is to be concentrated in the column

32 Hawkins

strip and arranged as shown in Fig. 10. That requirement is consistent with the Chapter 13 distribution requirements for reinforcement for moment transfer.

For the 2002 Edition, based on the work of Pan and Moehle (1989), a new provision 21.12.6.8 was added limiting the two-way shear caused by factored gravity loads and acting simultaneously with seismic effects. Consistent with the value of V_u above which γ_v values cannot be adjusted according to Sec. 13.5.3.3, V_u is limited to $0.4\phi V_c$ unless the maximum earthquake-induced factored shear stress transferred by eccentricity of shear, calculated in accordance with Fig. 1, is less than one-half of the allowable ϕv_n value. This provision was added because connections, both reinforced and prestressed, exhibit reduced lateral displacement capacities when the recommended limits are exceeded.

In addition to the foregoing changes, where the column protrudes beyond the edge of the slab, the effective slab width c_t for shear transfer calculations is limited to the distance from the interior face of the column to the slab edge measured parallel to c_1 in Fig. 1, but not to exceed c_1 . This same recommendation was made by Committee 426 (1977) and should also apply in Chapter 11.

The foregoing requirements apply only to two-way slabs that are part of the primary seismic force-resisting system. Conceivably, a structure could have frames that are part of that primary seismic-force-resisting system that are paralleled by other two-way slab frames that are not proportioned to resist earthquake motions.

Conclusions

- (1) With the addition of Sec. 21.11.5 there are now provisions in ACI 318-05 for the safe design of slab-column connections from gravity loadings through to extreme events.
- (2) For lateral loadings many of the provisions include simplifications that do not require calculation of the moment transferred through the connection.
- (3) With careful selection of the stiffness model the moments transferred through the connections and the lateral drifts of flat-plate structures can be accurately calculated. Models that use an “equivalent” constant beam width for the length of the flat-plate frame are not likely to provide accurate predictions. Different stiffness models are usually needed for interior and exterior connections and account taken of cracking, its type and extent, and of the level of stress in the slab reinforcement, in the column-head region.
- (4) Additional research is needed on moment transfer in slabs with shear drop capitals.

References

ACI-ASCE Committee 352, 1988, “Recommendations for the Design of Slab-Column Connections in Monolithic Reinforced Concrete Structures,” *ACI Structural Journal*, V. 85, No. 6, pp. 675-696.

ACI-ASCE Committee 426, 1974, “The Shear Strength of Reinforced Concrete Members-Slabs,” *Journal of the Structural Division, ASCE*, V. 100, No. ST8, pp. 1543-1591

Punching Shear in Reinforced Concrete Slabs 33

ACI-ASCE Committee 426, 1977, *Suggested Revisions to Shear Provisions for Building Codes (ACI 426.1R-77)*, American Concrete Institute, Farmington Hills, MI, 82pp.

Akiyama, H., and Hawkins, N.M., 1984, "Response of Flat Plate Concrete Structures to Seismic and Wind Forces," *Structures and Mechanics Report SM 84-1*, Department of Civil Engineering, University of Washington, Seattle, WA , 267p.

Bondy, K., (2004) "Post-Tensioned Slab Analysis- Four Seasons Building" SEAOC Research Committee Seminar, *Four Seasons Forced Vibration Testing*, Structural Engineers Association of Southern California, Los Angeles.

Carpenter, J.E., Kaar, P. H., and Corley, W.G., 1973 "Design of Ductile Flat-Plate Structures to Resist Earthquakes," *Proceedings, Fifth World Conference on Earthquake Engineering, Rome*, International Association for Earthquake Engineering, V. 2, pp. 2016-2019.

DiStasio, J. and Van Buren, M.P., 1960, "Transfer of Moment Between Flat Plate Floor and Column," *ACI Journal*, V. 57, No. 3, pp. 299-314.

Hanson, N.W., and Hanson, J.M., 1968, "Shear and Moment Transfer Between Concrete Slabs and Columns," *Journal of the Portland Cement Association Research and Development Laboratories*, V. 10, No. 1, pp. 2-16.

Hawkins, N.M., 1980, "Lateral Load Design Considerations for Flat Plate Structures," *Solid Mechanics Division, Study No.14, Nonlinear Design of Concrete Structures*, University of Waterloo Press, Canada, pp. 581-614.

Hawkins, N.M., 1981, "Lateral Load Resistance of Unbonded Post-Tensioned Flat Plate Connections," *PCI Journal*, V. 26, No.1, pp.94-118.

Hawkins, N.M., Bao, A., and Yamazaki, J., 1989 "Moment Transfer from Slabs to Columns," *ACI Structural Journal*, V. 86, No.6, pp.705-716.

Hueste, M.B.D., and Wight, J.K., 1999, "Nonlinear Punching Shear Failure Model for Interior Slab-Column Connections," *Journal of the Structural Division, ASCE*, V. 125, No. 9, pp. 997-1008

Islam, S., and Park, R., 1976, "Tests of Slab-Column Connections with Shear and Unbalanced Flexure," *Journal of the Structural Division, ASCE*, V. 102, No. ST3, pp. 549-568.

Megally, S., and Ghali, A., 2000, "Punching Shear Design of Earthquake-Resistant Slab-Column Connections," *ACI Structural Journal*, V. 97, No. 5, pp. 720-730.

Moehle, J.P., 1988, "Strength of Slab-Column Edge Connections," *ACI Structural Journal*, V. 85, No. 1, pp. 89-98.

34 Hawkins

Pan, A.P., and Moehle, J.P., 1989, "Lateral Displacement Ductility of Reinforced Concrete Flat Plates," *ACI Structural Journal*, V.86, No. 3, pp. 250-258.

Pan, A.P., and Moehle, J.P., 1992, "An Experimental Study of Slab-Column Connections," *ACI Structural Journal*, V. 89, No. 6, pp. 626-638.

Powell, G.H., (1973), "DRAIN-2D, user's guide." *Report EERC 73-22*, Earthquake Engineering Research Center, University of California, Berkeley, CA.

Robertson, I.N., and Durrani, A.J., 1992, "Gravity Load Effect on Seismic Behavior of Interior Slab-Column Connections," *ACI Structural Journal*, V. 89, No. 1, pp. 37-45.

Robertson, I.N., Kawai, T., Lee, J., and Enomoto, B., 2002, "Cyclic Testing of Slab-Column Connections with Shear Reinforcement," *ACI Structural Journal*, V. 99, No. 5, pp.605-613.

Tang, X., and Goel, S.C., (1988), " DRAIN-2DM technical notes and user's guide," *Research Report UMCE 88-1*, Department of Civil Engineering, University of Michigan, Ann Arbor, MI.

Vanderbilt, M.D., and Corley, W.G., 1983, "Frame Analysis of Concrete Buildings," *Concrete International*, V. 11, No. 9, pp.33-43.

Specimen Type	No. of Spec.	No. with Shear Reinf.	Column Size in. x in.	Slab Thick. in.	Col. Head Reinf. Ratio %	$\frac{V_g}{V_c}$
Interior Connection	20	13	12x12 16x16 19.5x8	6.0	0.6-2.0	0.4-0.75
Exterior – Moment Normal to Edge	8	5	As above	6.5-7.0	0.8-1.6	0.3
Exterior – Moment Parallel to Edge	5	3	As above	6.5-7.0	0.8-2.00	0.3
Corner	5	3	As above	6.5-7.0	0.8-1.50	0.3
Frame with Two Interior Columns	1	1 Conn.	13x6	4.5	1.23	Added DL
Frame with one Edge & one Interior Column	1	Both Conn.	12x12	7.0	0.4	Full Scale

Table 1 Properties of Akivama and Hawkins (1984) Test Specimens

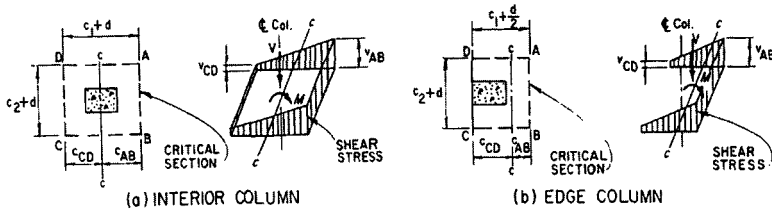


Figure 1 - Linear Variation in Shear Stress Assumed in ACI 318 Chapter 11.

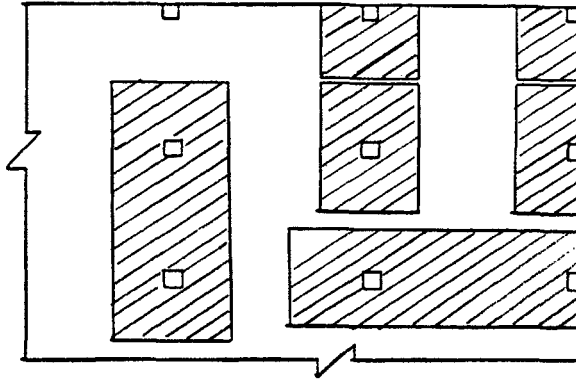


Figure 2 - Location of Akiyama and Hawkins Specimens within Floor System.

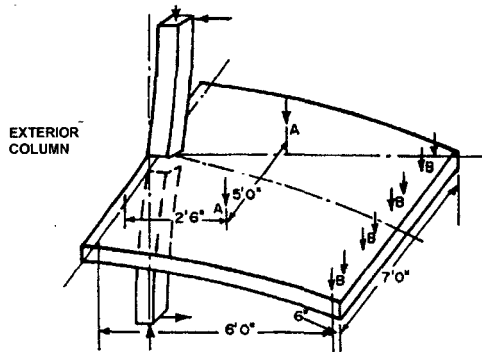


Figure 3 - Exterior Column-Slab Sub-Assemblage (Akiyama and Hawkins).

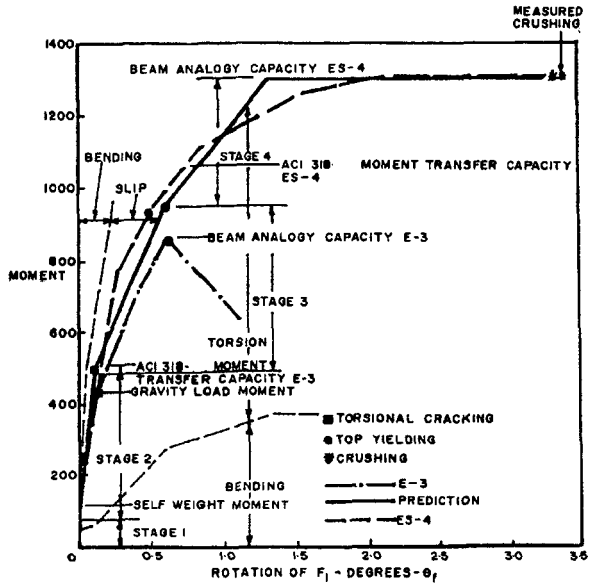


Figure 4 - Moment-Rotation Relationships for Exterior Column-Slab Connections.

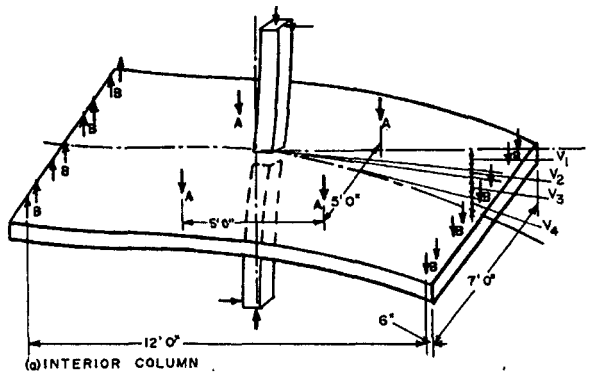


Figure 5 - Interior Column-Slab Sub-Assemblage (Akiyama and Hawkins).

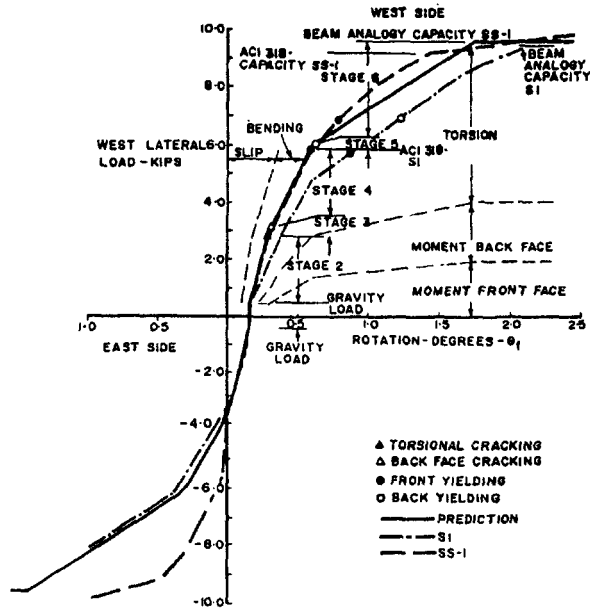


Figure 6 - Moment-Rotation Relationships for Interior Column-Slab Connections.

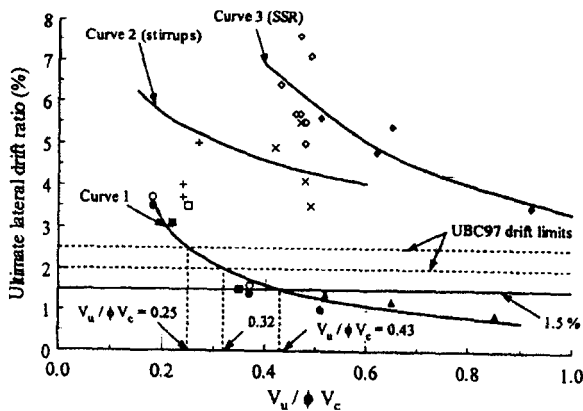


Figure 7 - Effect of Gravity Loads on Lateral Drift Capacity of Interior Slab-Column Connections (Megally and Ghali 2000).

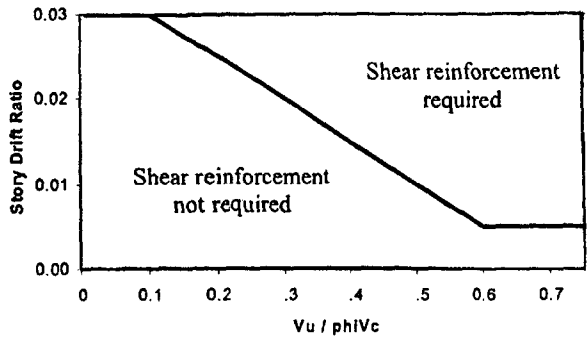


Figure 8 - Criterion for Slab Shear Reinforcement – Sec. 21.11.5 of ACI 318-05.

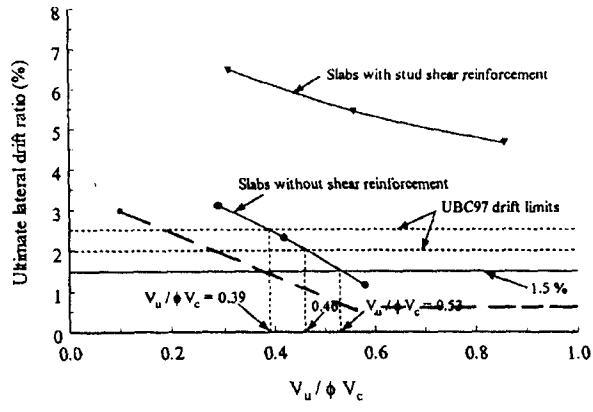


Figure 9 - Effect of Gravity Loads on Lateral Drift Capacity of Exterior Slab-Column Connections (Megally and Ghali 2000).

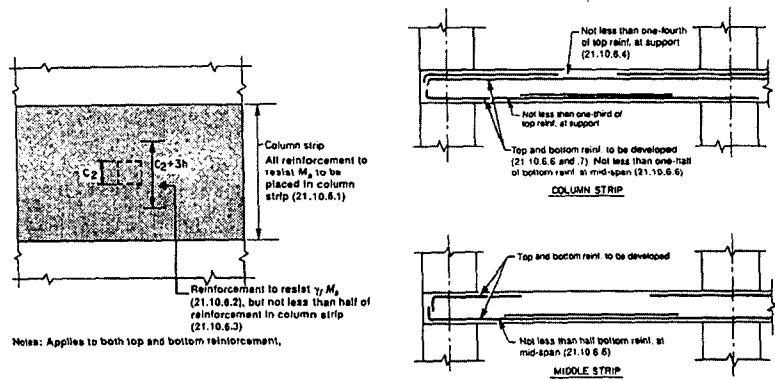


Figure 10 - Location and Arrangement of Reinforcement for Seismic Resistance (ACI 318-05).

Effects of Size, Geometry and Material Properties on Punching Shear Resistance

by D. Mitchell, W.D. Cook, and W. Dilger

Synopsis: This paper presents some code expressions for the punching shear strength of slab-column connections. The influence of slab thickness (size effect), column aspect ratio and concrete compressive strength are investigated by examining experimental results.

Keywords: column aspect ratio; concrete design codes; concrete strength; punching shear; size effects

40 Mitchell et al.

Denis Mitchell, FACI, is a James McGill Professor and Chair of Civil Engineering at McGill University. He is a member of ACI Committees 318-B, Structural Concrete Building Code, Reinforcement and Development; 408, Bond and Development of Reinforcement; and Joint ACI-ASCE Committee 445, Shear and Torsion. He is Chair of ACI's Fellows Nomination Committee. He also chairs the Canadian Standards Association Committee A23.3, Design of Concrete Structures.

William D. Cook, MACI, is a research engineer in the Department of Civil Engineering at McGill University. His interests include the non-linear analysis of reinforced concrete structures, the influence of high-strength concrete and the use of new construction materials.

Walter H. Dilger, FACI, FRSC, Professor Emeritus of Civil Engineering at the University of Calgary, Canada, is a member of ACI/ASCE Committee 445 and has been engaged in shear research since the 1960s. He is co-inventor of the shear stud reinforcement for flat slabs. Other research interests include effects of creep, shrinkage and temperature on structural concrete. He is also active in ACI Committee 209.

INTRODUCTION

This chapter provides a summary of the design expressions for punching shear strength in a number of international codes and standards such as ACI 318 (ACI 2005), CSA A23.3 (CSA 2004), CEB-FIP (1990), Eurocode 2 (EC2 2003) and the BS 8110 (1997). In particular, the influences of size effect, column aspect ratio and concrete strength are discussed.

The fib report (fib 2001) on punching of structural concrete slabs provides a comparison of codes and behavioral models as well as providing a data bank of test results on punching shear failures.

DISCUSSION OF CODE PROVISIONS

This chapter discusses the punching shear resistance of slab-column connections without shear reinforcement. A consistent set of symbols has been adopted for all of the code expressions and the equations are given in SI units (MPa and mm units). It is noted that the equations given below are for the nominal shear resistance (all capacity reduction factors and material resistance factors are taken as 1.0). The code equations for the nominal concrete shear capacity, V_c , are summarized below:

American Concrete Institute Code, ACI 318-05 (ACI 2005)

According to ACI 318-05 the nominal shear resistance, V_c , for normal density concrete is taken as the smallest of:

$$V_c = \left(1 + \frac{2}{\beta}\right) \frac{\sqrt{f'_c} b_o d_{av}}{6} \quad (3-1)$$

$$V_c = \left(\frac{\alpha_s d_{av}}{b_o} + 2\right) \frac{\sqrt{f'_c} b_o d_{av}}{12} \quad (3-2)$$

$$V_c = \frac{1}{3} \sqrt{f'_c} b_o d_{av} \quad (3-3)$$

where d_{av} is the average effective depth, b_o is the perimeter of the critical section located at a distance $0.5d_{av}$ from the face of the column, β is the ratio of the long side to short side of the concentrated load or reaction area and α_s is a factor accounting for the location of the slab-column connection (40 for interior columns, 30 for edge columns and 20 for corner columns). The ACI Code places an upper limit on $\sqrt{f'_c}$ of 8.33 MPa (100 psi). The special features of the ACI code expressions for punching shear include a factor for the rectangularity of the column, β , and a size effect when the critical shear periphery, b_o , becomes large compared to the effective depth, d_{av} . The shear strength is a function of the square root of the concrete compressive strength and is limited to a shear stress corresponding to a concrete with a compressive strength of 69 MPa (10,000 psi).

Canadian Standard CSA A 23.3-04 (CSA 2004)

The punching shear equations in the 2004 Canadian Standard CSA A23.3 are based on the same expressions as ACI for the nominal resistance. The shear stress coefficients in the design expressions for the factored punching shear resistance have been increased by 14% from the nominal values given in Equations (3-1) to (3-3) (e.g., from 1/3 to 0.38 in Equation (3-3)) due to the use of a relatively low material resistance factor for concrete ($\phi_c = 0.65$). The nominal resistance differs from ACI only in that it includes a size factor for slabs having an effective depth greater than 300 mm (12 in.). For such cases the punching shear strength is multiplied by a factor of:

$$\frac{1300}{1000 + d_{av}} \quad (3-4)$$

where d_{av} is the average effective depth for the slab.

Eurocode 2 (EC2 2003) and CEB-FIP Model Code (CEB-FIP 1990)

The expression for the nominal punching strength is:

42 Mitchell et al.

$$V_c = 0.18 \left[1 + \sqrt{\frac{200}{d_{av}}} \right] (100 \rho_{av} f_{ck})^{1/3} b_o d_{av} \quad (3-5)$$

where b_o is the perimeter of the critical section located at a distance of $2d_{av}$ from the face of the column and f_{ck} is the characteristic strength used in European codes. In using Equation (3-5) f_{ck} cannot be taken greater than 50 MPa. The nominal punching shear strength has been determined by multiplying the Eurocode 2 design expression coefficient of 0.12 by the concrete partial safety factor, γ_c , of 1.5 to give the 0.18 coefficient in Equation (3-5). The average reinforcement ratio, ρ_{av} , is limited to a maximum of 0.02. The Eurocode 2 includes a size effect depending on the value of d_{av} . The term within the square brackets in Equation (3-5) must be less than or equal to 2.0. The shear strength is a function of the average flexural reinforcement ratio in the slab and the shear strength varies as the cube root of the concrete compressive strength. The size effect can be written as:

$$0.5 \left[1 + \sqrt{\frac{200}{d_{av}}} \right] \leq 1.0 \quad (3-6)$$

It is noted that the Eurocode 2 has the same general form as the German Code (DIN 1045-1).

British Code BS 8110 (BS 8110 1997)

The expression for the nominal punching strength is:

$$V_c = 0.79 \sqrt[4]{\frac{400}{d_{av}}} \left[100 \rho_{av} \frac{f'_c}{25} \right]^{1/3} b_o d_{av} \quad (3-7)$$

where b_o is the perimeter of the critical section located at a distance of $1.5d_{av}$ from the face of the column and ρ_{av} is limited to 0.03. In using Equation (3-7) the concrete compressive strength cannot be taken greater than 40 MPa. The influence of size is included in the term:

$$\sqrt[4]{\frac{400}{d_{av}}} \leq 1.0 \quad (3-8)$$

EXPERIMENTAL PROGRAMS ON SIZE EFFECT

It is difficult to gather experimental data solely on the size effect. Many of the experimental programs reported in the literature have varied not just the thickness of the slab, but other parameters such as concrete strength, the reinforcement ratio ρ and the

Punching Shear in Reinforced Concrete Slabs 43

column size. For example, keeping the reinforcement ratio constant while increasing the slab thickness leads to a decreasing ratio of the flexural capacity to shear capacity. Having a smaller flexural capacity increases flexural cracking prior to failure and reduces the punching shear capacity. The results of many of the testing programs are also affected by other factors such as having extremely small specimens, having extensive yielding of the flexural reinforcement prior to punching shear failure or experiencing bond failures of the flexure reinforcement.

The test series on six reinforced concrete slabs tested by Li (2000) provide some useful data on the size effect. The overall thickness, h , of the slabs varied from 135 mm to 550 mm. The slabs were tested upside down by loading the centrally located high-strength concrete (80 MPa) square columns and reacting against neoprene pads on HSS sections sitting on the strong floor. Figure 3-1 shows the testing of Specimen P500, with an average effective depth of 500 mm. The flexural reinforcement and the overall dimension of the test slabs varied as shown in Table 3-1. Adequate anchorage of the flexural reinforcement was ensured by welding steel plates on the ends of the bars. All of the slabs were cast from the same batch of ready-mix concrete and the average concrete compressive strength of the slab concrete was 39.4 MPa.

Figure 3-2 shows the variation of the normalized shear stress versus the average effective depth for this series. The normalized shear stress was determined from the shear stress calculated based on the ACI Code critical shear periphery and dividing by $\sqrt{f'_c}$. It is noted that the flexural reinforcement ratio was constant at 0.76% for the three thickest slabs, with ρ varying from 0.83% to 0.98% for the thinner slabs (see Table 3-1). It is clear from Fig. 3-2 that there is a size effect for punching shear for slabs thicker than about 200 mm.

The test series by Nylander and Sundquist (1972), on the other hand, shows decreasing normalized stresses for slabs with an increase in effective depth from 95.5 mm to 201 mm. These results indicate that there is an influence of slab thickness on the shear stress at failure, even for effective depths less than 200 mm. Figure 3-3 shows the variation in the normalized shear stress ($v/\sqrt{f'_c}$), based on the ACI critical shear periphery, as a function of the average effective depth for these tests, including one test by Tolf (1988) with a thickness of 619 mm. The slabs were supported by circular columns having diameters of 120, 240 and 800 mm. The concrete strengths varied from 23.2 to 30.6 MPa. The results shown in Fig. 3-3 have been separated into two groups; one with an average reinforcement ratio of 0.76% and the other for an average reinforcement ratio of 0.53%. The slabs containing the higher ρ values had slightly higher capacities than the tests with lower values of ρ , as expected. The trend of the data clearly shows a size effect with the normalized shear stress decreasing with increased values of d_{av} . Similar results were observed by Birkle (2004) who tested slabs with thicknesses of 160, 230 and 300 mm, and Regan (1986) who varied the slab thickness from 80 to 250 mm. The normalized shear stresses at failure for both these test series are presented in Fig. 3-4. It is to be noted that out of Regan's 23 tests only the tests of the Group II Series which focussed on size and depth of the specimens are considered. Also, the tests with varying maximum aggregate size are not included.