Punching shear resistance of high-strength concrete slabs

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ABSTRACT

The use of high-strength concrete in reinforced concrete slabs is becoming popular in Australia and other countries. Current design provisions of AS3600 and other major codes throughout the world are based on empirical relationships developed from tests on low-strength concrete. In this paper, the experimental results from 4 research studies are used to review the existing recommendations in design codes for punching shear failure of slabs. Design codes referred in this study are AS3600 and CEB-FIP MC 90. In AS3600 the punching shear strength is expressed as proportional to $f_c^{1/2}$*. However in CEB-FIP MC 90 punching shear strength is assumed to be proportional to* $f_c^{1/3}$*. It is shown that the present provisions in AS3600 are applicable up to 100 MPa.*

KEYWORDS

Punching shear; slabs; high-strength concrete; CEB-FIP model code; design standards

1. Introduction

Concrete with strengths above 50 MPa is currently used due to an increasing requirement for higher strengths and improved long-term properties. HSC is being utilised in many projects around Australia [1]. High-strength concrete members exhibit, in some instances, different failure mechanisms and simply extrapolating models and equations meant for normal strength to high-strength concrete may lead to unsafe designs. One of the reasons why some structural engineers are reluctant to use high-strength concrete is due to the lack of provisions in the Concrete Structures Standard, AS3600 [2], to address this issue.

The reinforced concrete flat slab system is a widely used structural system. Its formwork is very simple as no beams or drop panels are used. However, the catastrophic nature of the failure exhibited at the connection between the slab and the column has concerned engineers. This area (Fig.1) becomes the most critical area as far as the strength of flat slabs is concerned due to the concentration of high bending moments and shear forces. The failure load may be considerably lower than the unrestrained flexural capacity of the slab. A typical punching shear failure of a bridge deck during testing is shown in Fig.2. The use of high-strength concrete improves the punching shear resistance allowing higher forces to be transferred through the slab-column connection. In spite of the wide use, only a few research projects have been conducted on the punching shear resistance of high-strength concrete slabs. The empirical expressions given in design codes are based on the experimental results from slabs with concrete strengths between 15-35 MPa. Hence it is necessary to re-examine the applicability of the present punching shear design methods for HSC slabs, using the published data.

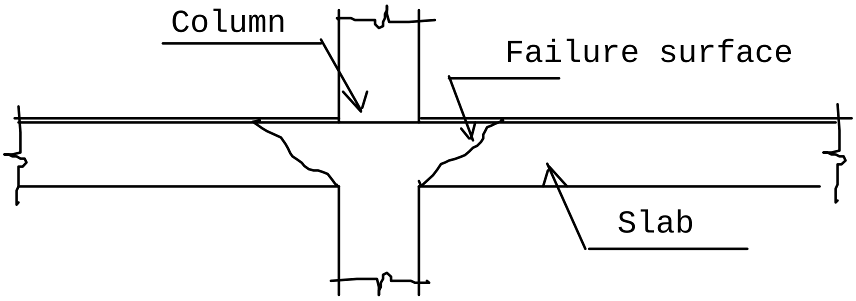

Fig. 1 - Punching failure surfaces of flat slab

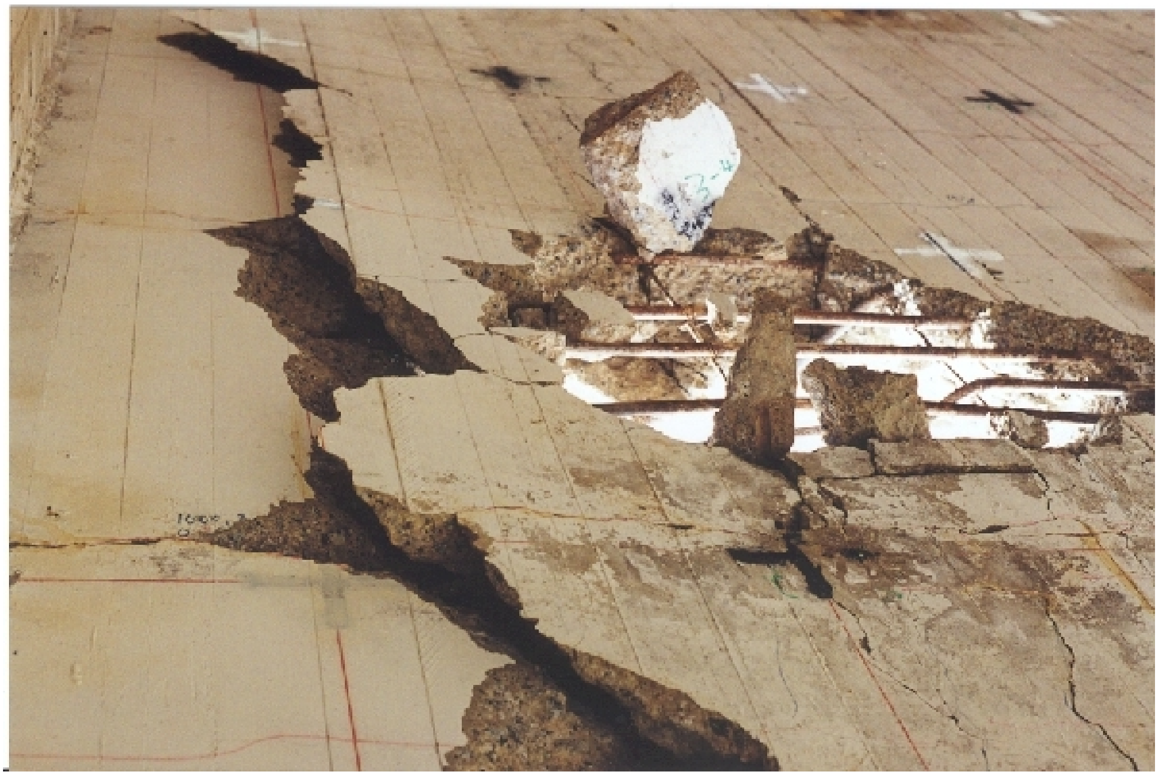

Fig. 2 - A typical punching shear failure of a bridge deck

2. Code Design Provision

Most codes present formulae, where the design punching load is a product of a design nominal shear strength and the area of a chosen control surface. Depending on the method used, the critical section for checking punching shear in slabs is usually situated between 0.5 to 2 times the effective depth from the edge of the load or the reaction. Influences of reinforcement, slab depth and other parameters are customarily governed by the application of different modification factors. The methods do not reflect the physical reality of the punching phenomenon, but can, when properly calibrated, lead to reasonable predictions [3].

Generally the punching shear strength values specified in different codes vary with concrete compressive strength f'_c and is usually expressed in terms of f_c^n. In AS3600 (Cl. 9.2.3) the punching shear strength is expressed as proportional to $\sqrt{f_c'}$. f'_c is limited to 50 MPa in the present code. The square-root formula in AS3600 is adopted from the ACI code [4]. ACI provisions for punching shear are derived from Moe's work on low strength concrete [5]. The ultimate shear strength for slabs without prestress is given by $V_{uo} = ud(f_{cv})$

where:

u= length of the critical perimeter, taken at a distance of d/2 from the column (mm) see Fig.2

f_{cv} = punching shear strength (MPa)

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

β_η = ratio of longest column dimension to shorter column dimension

In the comparison presented in this paper**,** the measured strength at the day of the test is substituted for f'_c.

In this study, CEB-FIP MC-90 model code [6] is also considered for comparison. Model Code is used by some engineers in Australia to design high-strength concrete members. In MC-90 the punching shear resistance, F_{sd} is expressed as proportional to $(f_{ck})^{\frac{1}{3}}$, Where f_{ck} is the characteristic compressive strength of concrete. The highest concrete grade considered in MC90 is C80, which corresponds to f_{ck} equal to 80 MPa. Influences of reinforcement and slab depth are also considered in this design code.

$$F_{sd} = 0.12\xi(100\rho f_{ck})^{\frac{1}{3}} u_1 d \qquad (2)$$

where:

$\xi = 1+\sqrt{\frac{200}{d}}$ is a size-effect coefficient

u_1= the length of the control perimeter at 2d from the column (Fig. 3)

$\rho = \sqrt{\rho_x \rho_y}$

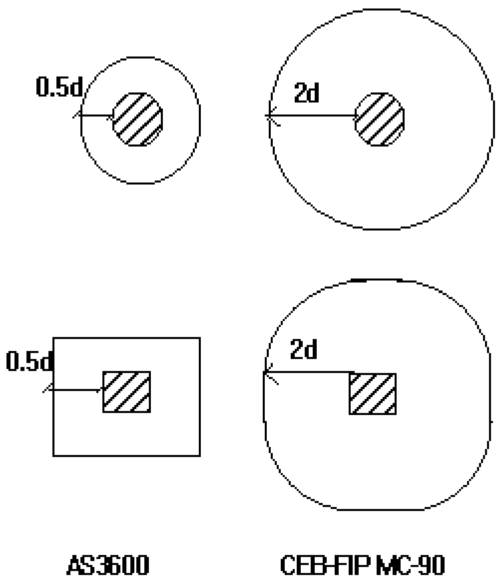

Fig. 3 - Control Perimeters specified in AS3600 and CEB-FIP codes

In the ultimate limit state the partial safety factor is 1.5. For the calculation of punching load capacity Eq. (2) is multiplied by 1.5, which gives Eq. (3).

$$F_{sd} = 0.18\xi(100\rho f_{ck})^{\frac{1}{3}} u_1 d \qquad (3)$$

In this study, measured concrete strength is taken as f_{ck}.

3. Comparison of Test Results with Code Predictions

A comprehensive literature review revealed that only a few experimental studies have been conducted on punching shear strength of high-strength slabs. These experimental results were used to check the validity of the punching shear strength formula given in AS3600 and CEB-FIP MC-90.

A total of 29 test results from four research studies conducted by Ramdane [7], Hallgren and Kinnunen [8], Marzouk and Hussein [9] and Tomaszewicz [10], were compared to values of punching strength calculated using AS3600. In all cases, tests were conducted on square or circular slabs supported by column stubs or loading plates. A brief description of the research studies is given below. A considerable variety of concrete strengths, slab reinforcement ratios and slab depths are represented in the various studies.

Ramdane[7] tested 18 circular slabs of 125 mm thickness and 1700 mm in diameter. They were divided into 3 groups in terms of main steel ratio with different concrete cylinder strengths varying from 32 to 102 MPa. The slabs were equally reinforced in orthogonal directions and were without shear reinforcement. The punching load was applied upward by a 550 kN hydraulic jack through a thick steel disk with a diameter of 150 mm situated in the centre beneath the slab. The reactions were provided by 12 high tensile steel rods equally spaced around a circle with a diameter of 1372 mm.

Hallgren and Kinnunen [8] tested 10 circular HSC slabs, supported on circular concrete column stubs. The total diameter of the slabs was 2540 mm and the diameter of the circle along which the load was uniformly distributed was 2400 mm. The slabs had a nominal thickness of 240 mm with an effective depth of 200 mm. The compressive strengths of HSC specimens were between 85 and 108 MPa. All slabs were provided with two-way flexural reinforcement consisting of deformed bars with a mean flexural reinforcement ratio of 0.003 to 0.012. Three slabs had shear reinforcement. The slabs without shear reinforcement are used in this paper for comparison.

Marzouk and Hussein [9] tested 17 square specimens to investigate the punching shear behaviour of high-strength concrete slabs. The structural behavior with regard to the deformation and strength characteristic of high-strength concrete slabs of various thicknesses and different reinforcement ratios (0.49-2.33%) were studied.

Tomaszewicz [10] tested 19 square flat slabs with orthogonal, equally spaced flexural reinforcement and without shear reinforcement. Slabs were supported along the edges and loaded at mid-span by a concentrated load to failure in punching. The variables in the test series were concrete strength (64-112 MPa), slab thickness (120, 240 and 320 mm) and reinforcement ratio. Parameters were chosen such that punching shear failure preceded flexural failure.

Table 1 shows the variables used for each study. Only the high-strength concrete slabs without shear reinforcement and failed in punching shear are used for the comparison.

Table 2 compares the experimental ultimate loads (P_{test}) of the slabs to the values predicted by AS3600 and CEB-FIP MC-90 as given by Eqns. (1) and (3) respectively. In these expressions, the limits with respect to the concrete strength have been ignored. The capacity reduction factor is assumed to be equal to 1. The mean and standard deviations for all the slabs are also given. Fig.4 shows the ratios between test results and the failure loads predicted by different formulae plotted with respect to the concrete strength. The concrete strengths for the test results considered in this study vary from 54 to 120 MPa. As seen only two points from AS3600 fall below the safety margin with one result for a slab with a concrete strength of 108 MPa. Therefore the AS3600 formula [Eq. (1)] can be considered to be applicable up to 100 MPa. However the ratios between observed and calculated loads clearly show that AS3600 is less conservative for the HSC slabs and high scatter is found. As AS3600 provisions are similar to ACI provisions, these conclusions are applicable to ACI 318-95. Generally CEB-FIP formula is less conservative and may be unsafe for some cases. Therefore if the CEB-FIP code formula [Eq. (2)] is used to calculate the punching shear strength, the concrete strength limit of 80 MPa should be maintained.

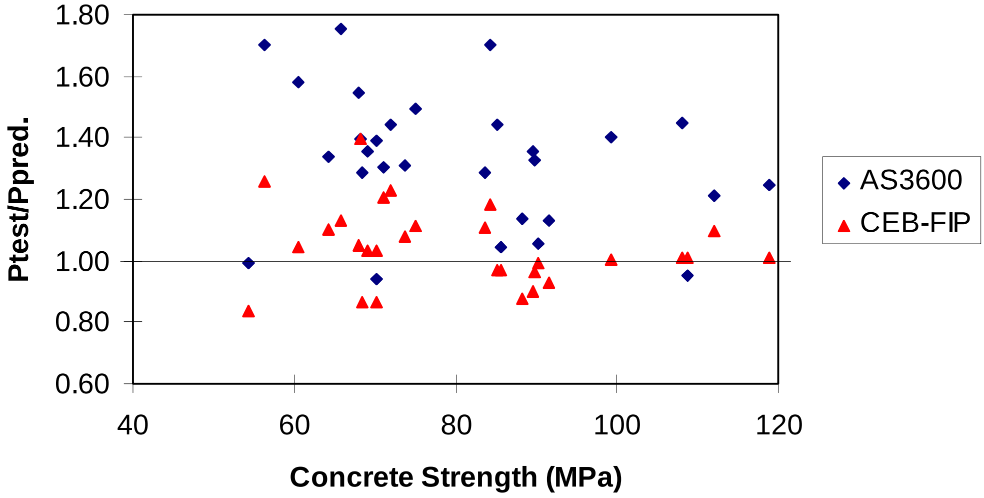

Fig. 4 - Ratios of experimental and predicted shear strengths

4. Conclusions

1. The use of high-strength concrete improves the punching shear resistance allowing higher forces to be transferred through the slab-column connection. However current design provisions of AS3600 for punching shear are based on empirical relationships developed from tests on low strength concrete. Hence it is necessary to re-examine the applicability of these provisions for HSC.

2. Generally the punching shear strength values specified in different codes vary with concrete compressive strength f'_c and is usually expressed in terms of f_c^n. In AS3600 the punching shear strength is expressed as proportional to $\sqrt{f_c'}$. However in CEB-FIP MC 90 punching shear strength is assumed to be proportional to $\sqrt[3]{f_c'}$.

3. The experimental results from 4 research studies are used to review the existing recommendations in AS3600 for punching shear failure of slabs. A brief description of these projects is given and the experimental results are summarised. The comparison of experimental results show that the AS3600 formula is applicable up to 100 MPa. However

the ratios between observed and calculated loads clearly show that AS3600 is less conservative for the HSC slabs. As AS3600 provisions are similar to ACI provisions, these conclusions are applicable to ACI 318-95.

4. Generally CEB-FIP formula is less conservative for HSC slabs and may be unsafe for some cases.

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Table 1 - Test variables

	Type	Diameter/width (mm)	f_c (MPa)	Column Dia./width (mm)	Slab Depth (mm)	Slab Rft %
Ramdane [7]						
slab 5	Circular	1372	54.4	150	125	0.58
slab 12	Circular	1372	60.4	150	125	1.28
slab 15	Circular	1372	68.4	150	125	1.28
slab 16	Circular	1372	99.2	150	125	1.28
slab22	Circular	1372	84.2	150	125	1.28
slab 23	Circular	1372	56.4	150	125	0.87
Hallgren and Kinnunen [8]						
HSC0	Circular	2400	90.3	250	240	0.8
HSC2	Circular	2400	85.7	250	240	0.8
HSC4	Circular	2400	91.6	250	240	1.2
HSC6	Circular	2400	108.8	250	240	0.6
Marzouk and Hussein [9]						
HS2	Square	1500	70.2	150	120	0.84
HS7	Square	1500	73.8	150	120	1.19
HS3	Square	1500	69.1	150	120	1.47
HS4	Square	1500	65.8	150	120	2.37
HS5	Square	1500	68.1	150	150	0.64
HS12	Square	1500	75	150	90	1.52
HS13	Square	1500	68	150	90	2
HS14	Square	1500	72	220	120	1.47
HS15	Square	1500	71	300	120	1.47
Tomaszewicz [10]						
nd65-1-1	Square	2500	64.3	200	320	1.42
nd95-1-1	Square	2500	83.7	200	320	1.42
nd95-1-3	Square	2500	89.9	200	320	2.43
nd115-1-1	Square	2500	112	200	320	1.42
nd65-2-1	Square	2200	70.2	150	240	1.66
nd95-2-1	Square	2200	88.2	150	240	1.66
nd95-2-3	Square	2200	89.5	150	240	2.49
nd115-2-1	Square	2200	119	150	240	1.66
nd115-2-3	Square	2200	108.1	150	240	2.49
nd95-3-1	Square	1100	85.1	100	120	1.72

Table 2 - Comparison of experimental and predicted shear strengths

	Exp. (kN)	AS3600	CEB-FIP	Exp /AS3600	Exp /CEB-FIP
Ramdane [7]					
slab 5	190	191.5	227.9	0.99	0.83
slab 12	319	201.8	306.3	1.58	1.04
slab 15	276	214.7	319.1	1.29	0.86
slab 16	362	258.6	360.8	1.40	1.00
slab22	405	238.2	341.8	1.70	1.19
slab 23	341	200.5	271.3	1.70	1.26
Hallgren and Kinnunen [8]					
HSC0	965	913.5	975.1	1.06	0.99
HSC2	889	851.7	915.4	1.04	0.97
HSC4	1041	920.1	1120.0	1.13	0.93
HSC6	960	1010.0	950.2	0.95	1.01
Marzouk and Hussein [9]					
HS2	249	265.2	288.7	0.94	0.86
HS7	356	271.9	329.2	1.31	1.08
HS3	356	263.1	345.4	1.35	1.03
HS4	418	238.3	369.7	1.75	1.13
HS5	365	261.2	261.3	1.40	1.40
HS12	258	172.8	231.9	1.49	1.11
HS13	267	172.7	253.7	1.55	1.05
HS14	498	345.3	404.8	1.44	1.23
HS15	560	430.0	465.1	1.30	1.20
Tomaszewicz [10]					
nd65-1-1	2050	1532.7	1863.1	1.34	1.10
nd95-1-1	2250	1748.7	2032.5	1.29	1.11
nd95-1-3	2400	1812.3	2486.1	1.32	0.97
nd115-1-1	2450	2022.8	2237.6	1.21	1.09
nd65-2-1	1200	861.4	1163.5	1.39	1.03
nd95-2-1	1100	965.6	1254.6	1.14	0.88
nd95-2-3	1250	921.3	1390.4	1.36	0.90
nd115-2-1	1400	1121.6	1384.9	1.25	1.01
nd115-2-3	1550	1069.0	1533.8	1.45	1.01
nd95-3-1	330	228.8	340.9	1.44	0.97
Mean				1.33	1.04
Std. Dev.				0.22	0.13