A Simple Equation for Predicting the Punching Shear Capacity of Normal and High Strength Concrete Flat Plates

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***Abstract*—The aim of this research is to propose a new simple equation that can predict the punching shear strength of normal and high strength concrete flat plates. In this work test data for interior flat-plate slab-column connections subjected to concentric gravity loads were collected from literature and compared with the proposed equation. From test data available in the literature, a new simple equation for punching shear capacity due to gravity load as a function of concrete strength, slab reinforcement ratio, slab effective depth and the critical perimeter was developed. The punching shear resistance strength was evaluated using ACI 318-14, Eurocode 2, CSA-14 and the IS456 design code equations. The new equation was checked and compared with the known approaches to predict the punching shear for normal as well as high strength concretes by using the tests in the data bank available and it gave good correlation with reasonable standards of deviation and small coefficient of variation.**

Keywords— Punching shear strength; flat plates; high-strength concrete; interior column; building codes.

I. INTRODUCTION

Flat plate construction is very common in parking, office, and apartment buildings. Exclusion of the beams, drop panels, or column capitals in the structural system optimizes the story height, formwork, labour, construction time, and the interior space of the building. This makes flat plate construction a very desirable structural system in view of economy, construction, and architectural desires. However, from structural point of view, supporting a relatively thin plate directly on a column is significantly problematic due to the structural discontinuity. [1]

Punching shear failure disasters have occurred several times in the last decades. This type of failure is extremely dangerous and should be prevented. In 1995 June 30th a five story Sampoong department store in South Korea collapsed due to this sudden brittle failure. Where more than 500 people were killed and nearly 1000 were injured [2]. Also Pipers Row Multi-Story Car Park which was built in 1965 collapsed during the night of 1997 March 20th. Initial reports identified some of the factors which contributed to cause punching shear failure which is developed into a progressive collapse [3].

There are significant variations in the approaches used to assess shear resistance of reinforced concrete slab-column connections in the current major codes. Generally, all design codes adopt the simple "shear on certain critical perimeter" approach and involve only the most important parameters. The critical section for checking punching shear is usually situated a distance between 0.5 to 2.0 times the effective depth (d) from the edge of the loaded area [4].(0.5d for ACI, CSA, IS456 and 2d in EC 2)

The other important difference amongst codes is in the way they represent the effect of concrete compressive strength (f'_c) on punching shear capacity. Generally, these codes expressed this effect in terms of $(f'_c)^n$, where (n) varies from (1/2) in the ACI code to (1/3) in the European code. The further complication is the definition of the concrete compressive strength. The ACI and CSA codes use specified concrete strength f'_c. while the European and Indian codes use characteristic strength f_{ck}. Punching shear provisions of current major codes are illustrated in Table 4.

The main purpose of this study was to propose a new simple punching shear equation for both normal strength concrete (NSC) and also for high strength concrete (HSC) flat plates and to compare the proposed equation with the shear strength provisions of ACI 318-14 [5], Eurocode 2 [6], CSA A23.3-14 [7], IS456-2000 [8] for interior slab column connections without shear reinforcement.

II. RESEARCH SIGNIFICANCE

Punching shear is a very important issue specially in flat plate slabs. Errors in predicting the punching shear have shown to lead to catastrophic failure [9]. Different codes have discussed the punching shear provisions with great importance.

In recent years the use of high strength concrete (HSC) is becoming more and more popular. Increase uses of HSC is faster than the development of appropriate design code and recommendations. Several recent studies showed that HSC have different characteristics than NSC. As the use of HSC is becoming more popular, the importance of research on punching shear provisions for HSC is increased.

Even for NSC, to calculate the punching shear capacity according to most of the major codes, it is required to use several equations and need to deal with various factors. Thus, a simple equation is needed to predict the punching shear for both, the NSC and HSC.

Although some researchers including Rankin (1987, 2003), Sherif and Dilger (1996), Gardner and Shao (1996), El-Gamal and Benmokrane (2004), Ali S. Abdul Jabbar et. al (2012), Elsanadedy, H.M., Al-Salloum, Y.A. and Alsayed, S.H. (2013) proposed their own equations to predict punching shear as shown in Table 1 [4,10,11,19]. However, these equations are not studied in this research.

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TABLE I. FORMULAS PROPOSED BY OTHER RESEARCHERS FOR CONCRETE PUNCHING SHEAR CAPACITY OF HSC FLAT PLATES [4,10,11,19]

Researcher	**Concrete Punching shear Capacity (Units: N and mm)**
Modified Rankin (1987; 2003)	$V_c = 0.78\sqrt[3]{f_c'}\sqrt[4]{100\rho}\, b_0 d$
Sherif and Dilger (1996)	$V_c = 0.7\sqrt[3]{f_c'}\sqrt[3]{100\rho}\, b_0 d$
Gardner and Shao (1996)	$V_c = 0.79\sqrt[3]{f_c'}\sqrt[3]{\rho f_y}\sqrt{1+\frac{200}{d}}\sqrt{\frac{d}{b_0}}\, b_0 d$
El-Gamal and Benmokrane (2004)	$V_c = 0.33\sqrt[3]{f_c'}\left[0.5\sqrt[3]{\rho E}\left(1+\frac{8d}{b_0}\right)\right].b_0 d$ where E= modulus of elasticity of reinforcing material (MPa)
Ali S. Abdul Jabbar et. al (2012)	$V_c = 0.9\sqrt[3]{f_c'}\sqrt{\frac{f_{sy}\rho.d}{b_0}}\sqrt[4]{\frac{200}{d}}\, b_0 d$
Elsanadedy, H.M. et al. (2013)	$V_c = 0.1\sqrt[3]{f_c'}\sqrt{\rho f_y}\left(1+\frac{8d}{b_0}\right)\sqrt{\left(1+\frac{125}{d}\right)}.b_0 d$

III. DATA COLLECTION

Punching shear tests can be done on either a multi-panel structure or on isolated slab column connections. Multi-panel tests are time consuming, expensive and it is difficult to determine experimentally the shear and moments applied in individual connections. Isolated slab column connection tests have the problem that the boundary conditions may not represent connections in a continuous structure and the moment redistribution cannot occur in an isolated connection test. [12]

All comparisons in this study are with the results from tests on isolated specimens. Flat plates are widely used for floor construction in multi-story buildings, as such a significant amount of experimental research work has been done on the punching shear failure of concrete flat plates.

A review of the literature revealed that only a few experimental studies are available on punching shear strength of high-strength slabs. The specimen's data was collected from the previous test results of isolated specimens conducted by Hallgren and Kinunen (1996) [13], Marzouk and Hussein (1991) [14], Tomaszewricz (1993) [15], Osman et al. (2000) [16], Ramdane (1996) [17], Ozden et al (2006) [18], Susanto T. et al. (2018) [19] with a total number of 38 specimens. These selected specimen samples for this study have compressive strength ranging between 70 to 119MPa. The reinforcement ratios were between 0.33 to 2.62 %. The effective depth ranged between 70 to 275mm. Table 2 presents details of HSC data. A large number of normal strength concrete specimens (243 specimen) test data are collected from literature (Susanto T. et al. (2018)) [19] where these data from 1956-2012. Table 3 presenting some of the details of NSC data from 243 samples.

IV. ANALYSIS OF DATA

In this study for the purpose of comparing code provisions safety factors have been removed from the equations. It might be important to note that this comparison is held between code provisions of interior circular or square columns of c1/c2 ratio equals to 1. It is generally accepted that the punching shear capacity of slab column connections results from concrete contribution and the contribution from shear reinforcement, if present. However, this study is limited for symmetrical flat slabs without shear reinforcement and connected to square or circular columns which means the aspect ratio is neglected and all other factors is taken to 1.0 for comparison reasons as noted before. So, to make the comparisons easier the provisions for the nominal concrete shear capacity V_c can be summarized in Table 4.

Eviews 10 statistic program is used to get the equation to predict the punching shear strength of interior column-slab connection of HSC flat plates. HSC specimens from literature conducted by different researcher [13,14,15,16,17,18,19] were selected for the regression analysis given earlier in Table 2. Investigations showed that the main variables affecting punching shear strength are: Concrete compressive strength (f'_c), Flexural Reinforcement ratio (ρ), Average effective depth (d), Column geometry (Critical perimeter b_0) [20]. These declared four variables have the most significant effect on flat plates without shear reinforcement.

Accordingly, the concrete punching shear strength can be expressed as follows:

$$V_c = C_1(f'_c)^{C2}(\rho/100)^{C3}(b_0 \times d)^{C4} \quad (1)$$

In the above equation C_1, C_2, C_3, C_4 are constants to be determined from the regression analysis. From the Regression analysis using Eviews 10 program, after cycles of iterations, it is found that C1=1.5, C2=0.5, C3=1/3, C4=1.0. The best-fit equation is as follows.

$$V_c = 1.5\left(\sqrt{f_c'}\right)\left(\sqrt[3]{\frac{\rho}{100}}\right)(b_0 \times d) \quad (2)$$

Where, V_c = predicted nominal punching shear stress (N)
f'_c = Concrete compressive strength (MPa)
d = average effective depth (mm)
ρ = flexural reinforcement ratio (%)
b_0 = critical perimeter at distance d/2 from column face
b_0 = 4(c+d) for square column
and π(c+d) for circular column
c = column diameter or width (mm)

The above equation has R^2 = 0.98. It should be illustrated that in the proposed equation, the critical section was assumed to be at a distance of $d/2$ from the column face because this value has been used to define the critical section in the ACI code since the 1960's. Therefore, the critical perimeter (b_0) is equal to $4(c + d)$ for square columns and $\pi(c + d)$ for circular columns.

It is important to emphasize that the suggested equation was concluded using punching shear strength testing database with specific physical and geometrical limits of the following: circular and rectangular columns with (c_1/c_2) ratio equals to 1.

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TABLE II. DETAILS OF HSC SPECIMENS FROM LITERATURE CONDUCTED BY DIFFERENT RESEARCHER

No.	Ref. [No.]	Slab ID	L (mm)	c (mm)	Column Shape	d (mm)	$f'c$ (MPa)	f_y (MPa)	ρ (%)	V_{exp} (kN)	Failure mode
1	[13]	HSC0	2540	250	C	200	90.3	643	0.8	965	P
2	[13]	HSC1	2540	250	C	200	91	627	0.8	1021	P
3	[13]	HSC2	2540	250	C	194	85.7	620	0.82	889	P
4	[13]	HSC4	2540	250	C	200	91.6	596	1.2	1041	P
5	[13]	HSC6	2540	250	C	201	108.8	633	0.6	960	P
6	[13]	HSC8	2540	250	C	198	95	634	0.8	944	P
7	[13]	HSC9	2540	250	C	202	84.1	631	0.33	565	FP
8	[14]	HS2	1700	150	S	95	70.2	490	0.84	249	P
9	[14]	HS6	1700	150	S	120	70	490	0.944	489	P
10	[14]	HS7	1700	150	S	95	73.8	490	1.19	356	P
11	[14]	HS9	1700	150	S	120	74	490	1.611	543	P
12	[14]	HS10	1700	150	S	120	80	490	2.333	645	P
13	[14]	HS11	1700	150	S	70	70	490	0.95	196	P
14	[14]	HS12	1700	150	S	70	75	490	1.524	258	P
15	[14]	HS14	1700	220	S	95	72	490	1.473	498	P
16	[14]	HS15	1700	300	S	95	71	490	1.473	560	P
17	[15]	nd95-1-1	3000	200	S	275	83.7	500	1.42	2250	P
18	[15]	nd95-1-3	3000	200	S	275	89.9	500	2.43	2400	P
19	[15]	nd115-1-1	3000	200	S	275	112	500	1.42	2450	P
20	[15]	nd65-2-1	2200	150	S	200	70.2	500	1.66	1200	P
21	[15]	nd95-2-1	2600	150	S	200	88.2	500	1.66	1100	P
22	[15]	nd95-2-1d	2600	150	S	200	87	500	1.75	1300	P
23	[15]	nd95-2-3	2600	150	S	200	90	500	2.49	1450	P
24	[15]	nd95-2-3d	2600	150	S	200	80	500	2.62	1250	P
25	[15]	nd95-2-3d+	2600	150	S	200	98	500	2.62	1450	P
26	[15]	nd115-2-1	2600	150	S	200	119	500	1.66	1400	P
27	[15]	nd115-2-3	2600	150	S	200	108.1	500	2.49	1550	P
28	[15]	nd95-3-1	1500	100	S	88	85.1	500	1.72	330	P
29	[16]	HSLW 1.0 P	1900	250	S	115	73.4	435	1	473.5	P
30	[16]	HSLW 1.5 P	1900	250	S	115	75.5	435	1.5	538.5	P
31	[16]	HSLW 2.0 P	1900	250	S	115	74	435	2	613.4	P
32	[17]	16	1700	150	C	95	99.2	650	1.28	362	FP
33	[17]	22	1700	150	C	98	84.24	650	1.28	405	P
34	[18]	HR1E0F0	1500	200	S	100	70.3	471	1.49	331	P
35	[18]	HR1E0F0r	1500	200	S	100	71.3	471	1.49	371	P
36	[18]	HR2E0F0r	1500	200	S	100	71	471	2.26	489	P
37	[19]	S11-090	2200	200	S	117	112	537	0.9	438.6	P
38	[19]	S11-139	2200	200	S	114	112	501	1.39	453.6	P

Where, L = slab width or diameter (mm), c = column width or diameter (mm), d = average effective depth (mm), Column shape: S = Square; C = Circular, Failure mode: P = Punching failure; F = Flexural failure

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TABLE III. DETAILS OF NSC SPECIMENS FROM LITERATURE CONDUCTED BY DIFFERENT RESEARCHER

No.	Year	Slab ID	L (mm)	c (mm)	Column Shape	d (mm)	$f'c$ (MPa)	f_y (MPa)	ρ (%)	V_{exp} (kN)	Failure mode
1	1997	L5	1970	399	C	172	31.1	612	0.66	696	P
2	1997	L6	1970	406	C	175	31.1	612	0.65	799	P
3	1997	L7	1970	201	C	177	22.9	586	0.64	478	P
4	1997	L8	2470	899	C	174	22.9	576	1.16	1111	P
5	1997	L9	2470	897	C	172	22.9	576	1.17	1107	P
6	1997	L10	2470	901	C	173	22.9	576	1.16	1079	P
7	1998	H.H.Z.S.1.0	1900	250	S	119	67.2	460	1	511.5	P
8	2000	9	2600	250	S	150	26.9	500	0.52	408	P
9	2000	9a	2600	250	S	150	21	500	0.52	360	P
10	2000	NU	2300	225	S	110	30	444	1.11	306	P
11	2000	NB	2300	225	S	110	30	444	2.15	349	P
12	2000	P100	925	201	S	99	39.3	488	0.97	330	P
13	2000	P150	1190	201	S	150	39.3	464	0.9	582.7	P
14	2000	P200	1450	201	S	201	39.3	464	0.83	902.9	P
15	2000	P300	1975	201	S	300	39.3	468	0.76	1378.9	P
16	2000	P400	1975	300	S	399	39.3	468	0.76	2224	P
17	2000	1	2400	120	S	93	60.9	695	1.5	270	P
18	2000	2	1700	120	S	97	62.9	695	1.4	335	P
19	2004	L1b	1680	120	S	108	59	749	1.08	322.4	P
20	2004	L1c	1680	120	S	107	59	749	1.09	318	P
21	2004	OC11	2200	200	S	105	36	461	1.81	423	P
22	2006	NR1E0F0	1500	200	S	100	20.5	507	0.73	188	P
23	2006	NR2E0F0	1500	200	S	100	19	507	1.09	202	P
24	2006	HR2E0F0	1500	200	S	100	60.5	471	2.26	405	P
25	2008	1	2400	250	S	124	36.2	488	1.54	483	P
26	2008	7	3400	300	S	190	35	531	1.3	825	P
27	2008	30U	2300	225	S	110	30	434	1.11	306	P
28	2008	30B	2300	225	S	110	30	434	2.15	349	P
29	2008	65U	2300	225	S	110	67.1	445	1.18	443	P
30	2009	PG-1	3000	260	S	210	27.6	573	1.5	1023	P
31	2009	PG-6	1500	130	S	96	34.7	526	1.5	238	P
32	2009	PG-7	1500	130	S	100	34.7	550	0.75	241	P
33	2009	PG-11	3000	260	S	210	31.5	570	0.75	763	P
34	2010	S1	1500	152	S	127	47.7	471	0.83	433	P
35	2010	S2	1500	152	S	127	47.7	471	0.56	379	P
36	2012	A0	1050	200	S	105	21.7	492	0.66	284	P
37	2012	B0	1350	200	S	105	21.7	492	0.75	275	P
38	2012	C0	1650	200	S	105	21.7	492	0.7	264	P

Where, L = slab width or diameter (mm), c = column width or diameter (mm), d = average effective depth (mm), Column shape: S = Square; C = Circular, Failure mode: P = Punching failure; F = Flexural failure

TABLE IV. SUMMARY OF CODE PROVISION WITH PROPOSED EQUATION

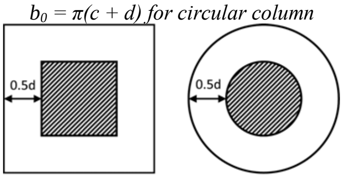
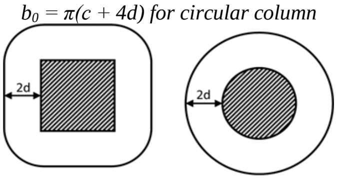
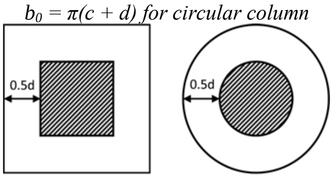
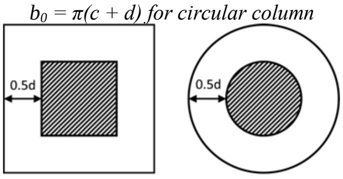
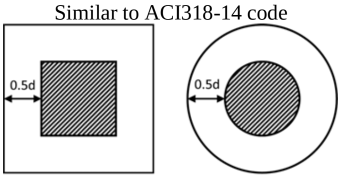
Code	Critical Perimeter	Nominal Shear Capacity
ACI 318-14	Located at 0.5*d* from the column's face $b_0 = 4(c + d)$ *for square column* $b_0 = \pi(c + d)$ *for circular column*  $b_o = 4(c+d)$ $b_o = \pi(c+d)$	$V_c = \min of \begin{cases} 0.33\sqrt{f_c'}b_0 d \\ 0.17\left(1+\frac{2}{\beta}\right)\sqrt{f_c'}b_0 d \\ 0.083\left(\frac{\alpha_s d}{b_0}+2\right)\sqrt{f_c'}b_0 d \end{cases}$
Eurocode 2	Located at 2d from the column's face $b_0 = 4(c + \pi d)$ *for square column* $b_0 = \pi(c + 4d)$ *for circular column*  $b_o = 4(c+\pi d)$ $b_o = \pi(c+4d)$	$V_c = \frac{0.18}{\gamma_c}\xi(100\rho_{av}f_{ck})^{\frac{1}{3}}b_0 d_{avg}$ $\xi = 1+\sqrt{\frac{200}{d_{ave}}} \leq 2.0$
CSA A23.3-14	Located at 0.5*d* from the column's face $b_0 = 4(c + d)$ *for square column* $b_0 = \pi(c + d)$ *for circular column*  $b_o = 4(c+d)$ $b_o = \pi(c+d)$	$V_c = \min of \begin{cases} 0.19\lambda\left(\frac{1+2}{\beta_c}\right)\sqrt{f_c'}b_0 d \\ \lambda\left(\frac{\alpha_s d}{b_0}+0.19\right)\sqrt{f_c'}b_0 d \\ 0.38\lambda\sqrt{f_c'}b_0 d \end{cases}$
IS 456	Located at 0.5*d* from the column's face $b_0 = 4(c + d)$ *for square column* $b_0 = \pi(c + d)$ *for circular column*  $b_o = 4(c+d)$ $b_o = \pi(c+d)$	$V_c = \tau_c b_0 d$ $\tau_c = k_s(0.25\sqrt{f_{ck}})$ $k_s = 0.5 + \beta_c \leq 1.0$
Author' s Proposed equation	Similar to ACI318-14 code  $b_o = 4(c+d)$ $b_o = \pi(c+d)$	$V_c = 1.5(\sqrt{f_c'}) \times \sqrt[3]{\frac{\rho}{100}}(b_0 \times d)$

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TABLE V. THE COMPARISON OF THE PROPOSED METHOD, ACI, EC 2, CSA AND IS 456 EQUATIONS WITH HSC EXPERIMENTAL DATA FOR THE SPECIMENS

Ref	No.	Specimen	$f'c$ (MPa)	Exp. (kN)	ACI (kN)	Euro (kN)	Canadian (kN)	Indian (kN)	Proposed (kN)	$V_{(exp)}/V_{(pred)}$				
										ACI	EC2	CSA	IS	Proposed
[13]	1	HSC0	90.3	965	887	989	1021	1008	806	1.09	0.98	0.95	0.96	1.20
	2	HSC1	91	1021	890	992	1025	1011	809	1.15	1.03	1.00	1.01	1.26
	3	HSC2	85.7	889	827	936	952	939	758	1.08	0.95	0.93	0.95	1.17
	4	HSC4	91.6	1041	893	1138	1028	1015	929	1.17	0.91	1.01	1.03	1.12
	5	HSC6	108.8	960	980	964	1129	1114	810	0.98	1.00	0.85	0.86	1.19
	6	HSC8	95	944	896	991	1032	1019	815	1.05	0.95	0.91	0.93	1.16
	7	HSC9	84.1	565	868	730	1000	986	587	0.65	0.77	0.57	0.57	0.96
[14]	8	HS2	70.2	249	257	293	296	293	238	0.97	0.85	0.84	0.85	1.05
	9	HS6	70	489	358	422	412	407	344	1.37	1.16	1.19	1.20	1.42
	10	HS7	73.8	356	264	334	304	300	274	1.35	1.07	1.17	1.19	1.30
	11	HS9	74	543	368	513	424	418	422	1.48	1.06	1.28	1.30	1.29
	12	HS10	80	645	383	596	440	435	497	1.69	1.08	1.46	1.48	1.30
	13	HS11	70	196	170	203	196	193	164	1.15	0.96	1.00	1.01	1.20
	14	HS12	75	258	176	243	203	200	198	1.47	1.06	1.27	1.29	1.30
	15	HS14	72	498	335	411	386	381	373	1.49	1.21	1.29	1.31	1.33
	16	HS15	71	560	417	473	481	474	465	1.34	1.18	1.17	1.18	1.20
[15]	17	nd95-1-1	83.7	2250	1577	1919	1816	1793	1736	1.43	1.17	1.24	1.26	1.30
	18	nd95-1-3	89.9	2400	1635	2351	1883	1858	2152	1.47	1.02	1.27	1.29	1.12
	19	nd115-1-1	112	2450	1825	2115	2101	2074	2009	1.34	1.16	1.17	1.18	1.22
	20	nd65-2-1	70.2	1200	774	1095	891	880	898	1.55	1.10	1.35	1.36	1.34
	21	nd95-2-1	88.2	1100	868	1181	999	986	1006	1.27	0.93	1.10	1.12	1.09
	22	nd95-2-1d	87	1300	862	1197	992	979	1017	1.51	1.09	1.31	1.33	1.28
	23	nd95-2-3	90	1450	877	1362	1009	996	1164	1.65	1.06	1.44	1.46	1.25
	24	nd95-2-3d	80	1250	826	1332	952	939	1116	1.51	0.94	1.31	1.33	1.12
	25	nd95-2-3d+	98	1450	915	1425	1053	1039	1235	1.59	1.02	1.38	1.39	1.17
	26	nd115-2-1	119	1400	1008	1305	1161	1145	1169	1.39	1.07	1.21	1.22	1.20
	27	nd115-2-3	108.1	1550	961	1447	1106	1092	1275	1.61	1.07	1.40	1.42	1.22
	28	nd95-3-1	85.1	330	201	315	232	229	236	1.64	1.05	1.42	1.44	1.40
[16]	29	HSLW 1.0 P	73.4	473.5	475	491	547	539	465	1.00	0.96	0.87	0.88	1.02
	30	HSLW 1.5 P	75.5	538.5	481	568	554	547	540	1.12	0.95	0.97	0.98	1.00
	31	HSLW 2.0 P	74	613.4	477	621	549	542	588	1.29	0.99	1.12	1.13	1.04
[17]	32	16	99.2	362	240	351	277	273	256	1.51	1.03	1.31	1.33	1.42
	33	22	84.24	405	231	347	266	263	246	1.75	1.17	1.52	1.54	1.65
[18]	34	HR1E0F0	70.3	331	332	421	382	377	371	1.00	0.79	0.87	0.88	0.89
	35	HR1E0F0r	71.3	371	334	423	385	380	374	1.11	0.88	0.96	0.98	0.99
	36	HR2E0F0r	71	489	334	486	384	379	429	1.47	1.01	1.27	1.29	1.14
[19]	37	S11-090	112	438.6	518	513	597	589	490	0.85	0.85	0.74	0.74	0.90
	38	S11-139	112	453.6	500	573	576	568	547	0.91	0.79	0.79	0.80	0.83
									Mean	**1.30**	**1.01**	**1.13**	**1.14**	**1.18**
									STD	**0.27**	**0.11**	**0.23**	**0.23**	**0.16**
									COV	**0.07**	**0.01**	**0.05**	**0.05**	**0.03**

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TABLE VI. THE COMPARISON OF THE PROPOSED METHOD, ACI, EC 2, CSA AND IS 456 EQUATIONS WITH NSC EXPERIMENTAL DATA FOR THE SPECIMENS

Ref	No.	Specimen	$f'c$ (MPa)	Exp. (kN)	ACI (kN)	Euro (kN)	Canadian (kN)	Indian (kN)	Proposed (kN)	$V_{(exp)}/V_{(pred)}$				
										ACI	EC2	CSA	IS	Proposed
[19]	1	L3	31.1	530	374	437	431	425	281	1.42	1.21	1.23	1.25	1.89
	2	L4	31.1	686	562	597	647	639	482	1.22	1.15	1.06	1.07	1.42
	3	L5	31.1	696	568	602	654	645	484	1.23	1.16	1.06	1.08	1.44
	4	L6	31.1	799	588	617	677	668	499	1.36	1.30	1.18	1.20	1.60
	5	L7	22.9	478	332	459	382	377	280	1.44	1.04	1.25	1.27	1.71
	6	L8	22.9	1111	926	970	1067	1053	953	1.20	1.14	1.04	1.06	1.17
	7	L9	22.9	1107	912	959	1050	1037	941	1.21	1.15	1.05	1.07	1.18
	8	L10	22.9	1079	922	965	1061	1047	948	1.17	1.12	1.02	1.03	1.14
	9	H.H.Z.S.1.0	67.2	511.5	475	499	547	540	465	1.08	1.02	0.93	0.95	1.10
	10	9	26.9	408	411	404	473	467	323	0.99	1.01	0.86	0.87	1.26
	11	9a	21	360	363	372	418	412	286	0.99	0.97	0.86	0.87	1.26
	12	NU	30	306	266	341	307	303	270	1.15	0.90	1.00	1.01	1.13
	13	NB	30	349	266	426	307	303	337	1.31	0.82	1.14	1.15	1.04
	14	P100	39.3	330	246	297	283	279	238	1.34	1.11	1.17	1.18	1.39
	15	P150	39.3	582.7	436	514	502	495	412	1.34	1.13	1.16	1.18	1.41
	16	P200	39.3	902.9	669	769	770	760	615	1.35	1.17	1.17	1.19	1.47
	17	P300	39.3	1378.9	1244	1392	1432	1413	1112	1.11	0.99	0.96	0.98	1.24
	18	P400	39.3	2224	2308	2365	2658	2623	2063	0.96	0.94	0.84	0.85	1.08
	19	1	60.9	270	204	307	235	232	229	1.32	0.88	1.15	1.16	1.18
	20	2	62.9	335	220	321	254	250	241	1.52	1.04	1.32	1.34	1.39
	21	L1b	59	322.4	250	337	287	284	251	1.29	0.96	1.12	1.14	1.29
	22	L1c	59	318	246	333	284	280	248	1.29	0.95	1.12	1.14	1.28
	23	OC11	36	423	254	384	292	288	303	1.67	1.10	1.45	1.47	1.40
	24	HR2E0F0	60.5	405	308	460	355	350	396	1.31	0.88	1.14	1.16	1.02
	25	1	36.2	483	368	495	424	419	417	1.31	0.98	1.14	1.15	1.16
	26	7	35	825	727	887	837	826	777	1.13	0.93	0.99	1.00	1.06
	27	30U	30	306	266	341	307	303	270	1.15	0.90	1.00	1.01	1.13
	28	30B	30	349	266	426	307	303	337	1.31	0.82	1.14	1.15	1.04
	29	65U	67.1	443	398	456	459	453	412	1.11	0.97	0.97	0.98	1.07
	30	PG-1	27.6	1023	684	951	788	778	767	1.49	1.08	1.30	1.32	1.33
	31	PG-6	34.7	238	169	272	194	192	189	1.41	0.87	1.23	1.24	1.26
	32	PG-7	34.7	241	179	229	206	203	159	1.35	1.05	1.17	1.19	1.51
	33	PG-11	31.5	763	731	788	842	831	651	1.04	0.97	0.91	0.92	1.17
	34	S1	47.7	433	323	387	372	367	297	1.34	1.12	1.16	1.18	1.46
	35	S2	47.7	379	323	340	372	367	261	1.17	1.12	1.02	1.03	1.45
	36	A0	21.7	284	197	232	227	224	168	1.44	1.23	1.25	1.27	1.69
	37	B0	21.7	275	197	242	227	224	175	1.40	1.14	1.21	1.23	1.57
	38	C0	21.7	264	197	236	227	224	171	1.34	1.12	1.16	1.18	1.54
									Mean	**1.40**	**1.01**	**1.22**	**1.23**	**1.37**
									STD	**0.30**	**0.21**	**0.26**	**0.26**	**0.21**
									COV	**0.088**	**0.046**	**0.066**	**0.068**	**0.045**

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Therefore, this proposed equation should be updated by the same expert software program "Eviws 10" for its application to wide range beyond those mentioned here once testing database on HSC flat plates become available.

The above suggested formula of Equation was utilized to assess the concrete punching shear capacity for the 38 HSC specimens and the ratios of the experimental-to-predicted punching shear capacity are then calculated and plotted as displayed in Figure (1). Comparison between experimental vs. predicted by the proposed equation for high strength concrete (HSC) specimens is presented in the Figure 1.

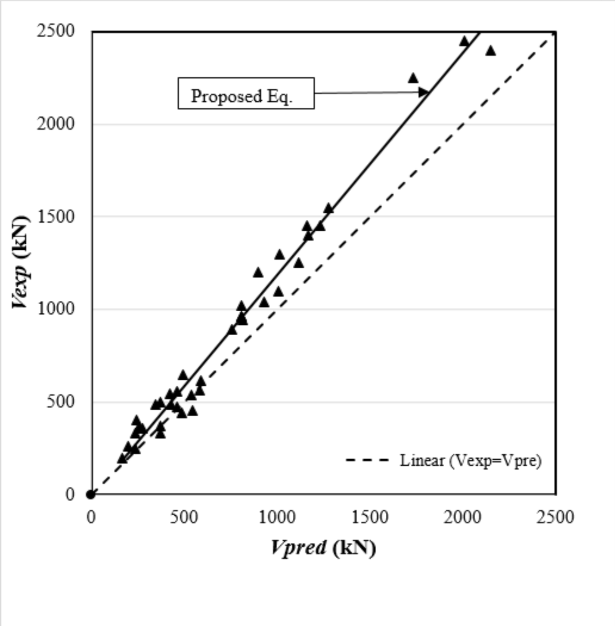

Fig. 1. Comparison of V_{exp} vs. V_{pre} for HSC specimens using proposed equation.

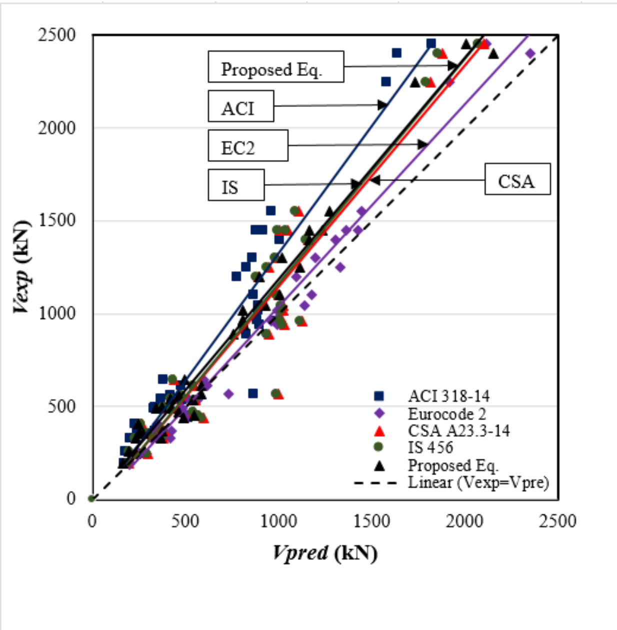

Fig. 2. Comparison of V_{exp} vs. V_{pre} for HSC specimens using proposed equation and different codes.

Table 5 summarize the comparison of different codes and proposed equation for high strength concrete, Also the statistical indicators the mean and standard deviations and the coefficient of variance are presented. Figure 2 showing comparison between experimental vs. predicted by different codes and the proposed equation for high strength concrete (HSC) specimens.

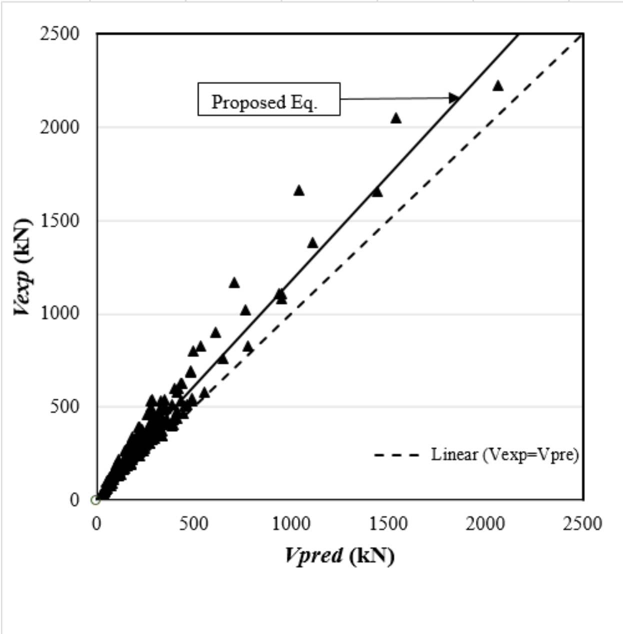

Fig. 3. Comparison of V_{exp} vs. V_{pre} for NSC specimens using proposed equation.

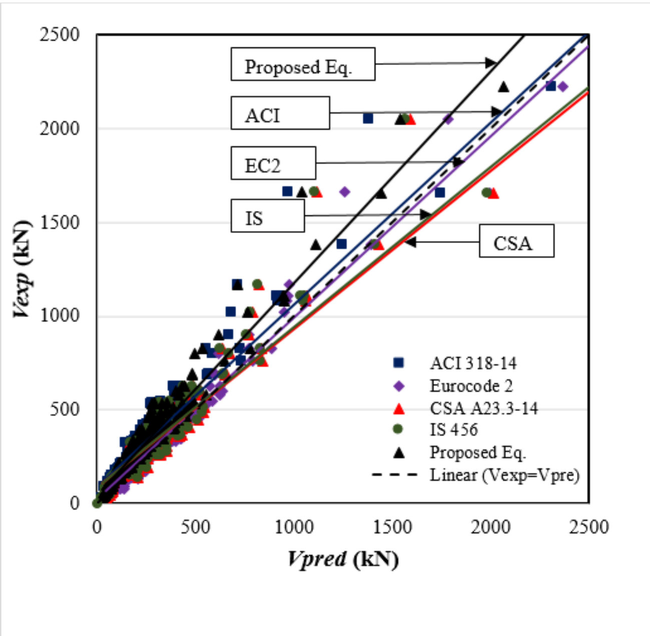

Fig. 4. Comparison of V_{exp} vs. V_{pre} for HSC specimens using proposed equation and different codes.

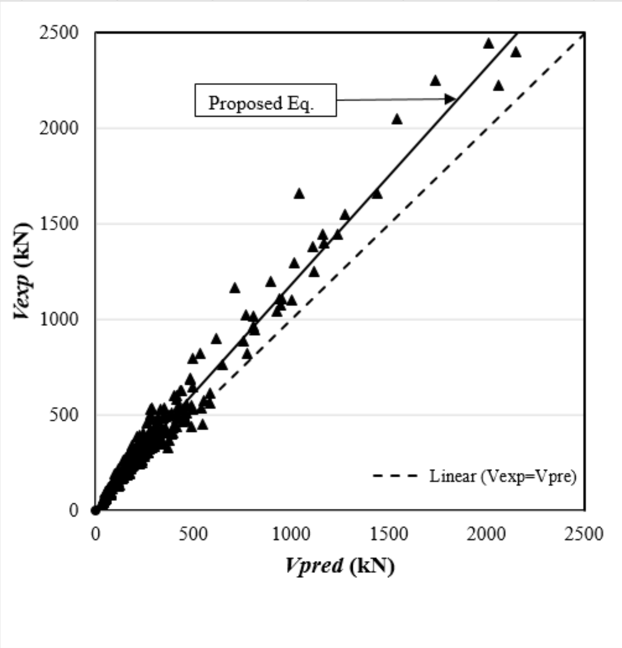

Fig. 5. Comparison of V_{exp} vs. V_{pre} for HSC and NSC specimens using proposed equation.

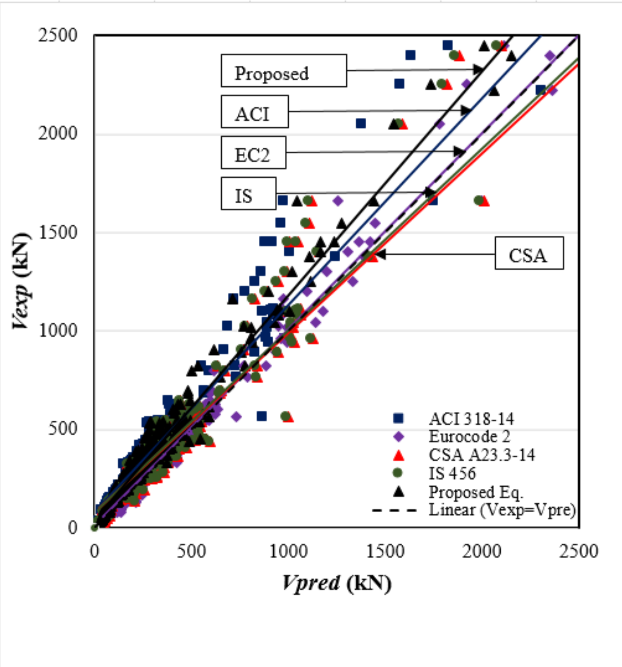

Fig. 6. Comparison of V_{exp} vs. V_{pre} for HSC specimens using proposed equation and different code.

Although, the basis in deriving the proposed equation was estimating the punching shear for high strength concrete flat plates, we might extrapolate it for use with normal or low strength concrete (NSC). Therefore, to evaluate and assess the applicability of this proposed equation for NSC a sample of different slabs tested by different researchers with compressive strength less than 69 MPa were used. The prediction of shear strength for normal strength concrete (NSC) specimens by proposed equation and different codes are calculated and the means, standard deviations and coefficients of variation are given in Table 6. The statistic parameters: mean, standard deviation and the coefficient of variation were used for comparing with ACI, EC2, CSA, IS provisions in Table 6. Figure 3 shows a comparison between experimental vs. predicted by the proposed equation for normal strength concrete (NSC) specimens. Also in Figure 4 shows a comparison between experimental vs. predicted by different codes and the proposed equation for normal strength concrete (NSC) specimens.

281 existing published data for normal and high strength concrete specimens is used to evaluate the accuracies and safety of the ACI 318-14, Eurocode 2, CSA A23.3-14, IS 456 and proposed equation for punching shear. Table 7 shows (mean, standard deviation and coefficient of variation) fot the comparison of the proposed equation and ACI, Eurocode 2, CSA and IS 456. In all cases, test was conducted on square or circular slabs supported by column stubs or loading plates. Figure 5 shows a comparison between experimental vs. predicted by the proposed equation for normal and high strength concrete (NSC and HSC) specimens. Also in Figure 6 shows a comparison between experimental vs. predicted by different codes and the proposed equation for normal and high strength concrete (NSC and HSC) specimens.

TABLE VII. COMPARISON OF STATISTICAL RESULTS OF DIFFERENT CODES AND PROPOSED EQUATION

HSC Specimen					
	ACI 318-14	***EC2***	***CSA A23.3-14***	***IS 456:2000***	***Proposed Eq.***
Mean	1.30	1.01	1.13	1.14	1.18
STD	0.27	0.11	0.23	0.23	0.16
COV	0.07	0.01	0.05	0.05	0.03
NSC Specimen					
	ACI 318-14	***EC2***	***CSA A23.3-14***	***IS 456:2000***	***Proposed Eq.***
Mean	1.40	1.01	1.22	1.23	1.37
STD	0.30	0.21	0.26	0.26	0.21
COV	0.088	0.046	0.066	0.068	0.045
NSC and HSC Specimen					
	ACI 318-14	***EC2***	***CSA A23.3-14***	***IS 456:2000***	***Proposed Eq.***
Mean	1.39	1.01	1.21	1.22	1.35
STD	0.29	0.20	0.26	0.26	0.22
COV	0.087	0.041	0.065	0.067	0.047

V. DISCUSSION

According to above analysis based on the available data of specimens from different experimental studies, the relation between the variables and the punching shear strength of interior slab-column connection of HSC flat plates could be defined as below:

$$V_c \propto \left(\sqrt{f_c'}\right)\left(\sqrt[3]{\rho/100}\right)(b_0 \times d)$$

Compared with the given provisions of major codes (ACI, EC2, CSA, IS) this proposed formula based on regression analysis gives good correlation with test results.

Therefore, the new proposed equation reasonably shows that the punching shear strength in HSC flat plates is proportional to the square root compressive strength but it is also shows that the influence of the flexural reinforcement ratio is proportional to the cubic root. On the other hand, it was found that the punching strength is proportional to the effective depth multiplied by the critical perimeter which is in accordance with the definition on the ACI code.

In case, using HSC the ACI 318-14 shows the most conservative and scatter results among other design codes as evidenced by the highest mean of 1.30 and standard deviation of 0.27. While EC2 prediction shows the least difference to the experimental values with the lowest mean of 1.01 and least scatter with a standard deviation of 0.11. On the other hand, proposed equation is better than ACI code and close to CSA A23.3-14 and IS 456:2000.

In case, using NSC the ACI 318-14 shows the most conservative and scatter results among other design codes as evidenced by the highest mean of 1.40 and standard deviation of 0.30. While EC2 prediction shows the least difference to the experimental values with the lowest mean of 1.01 and least scatter with a standard deviation of 0.21. On the other hand, proposed equation is conservative, similar to ACI code with mean of 1.37 and standard deviation of 0.21. Canadian and Indian code with mean 1.22, 1.23 and standard deviation of. 0.26, 0.26 respectively.

In case, using NSC and HSC together the ACI 318-14 shows the most conservative and scatter results among other design codes as evidenced by the highest mean of 1.39 and standard deviation of 0.29. While EC2 prediction shows the least difference to the experimental values with the lowest mean of 1.01 and least scatter with a standard deviation of 0.20. On the other hand, proposed is conservative similar to ACI code with mean of 1.35 and standard deviation of 0.22. Canadian and Indian code with mean 1.21, 1.22 and standard deviation of. 0.26, 0.26 respectively. The statistical analysis is summarized in Table 7.

It must be noted that the ACI 318-14 places the upper limit of $\sqrt{f_c'}$ by 8.33MPa which means it is limited to a concrete with a compressive strength of about 69MPa. This limitation in the American code is due to the fact that these provision developed mostly from tests on low and normal strength concrete flat plates. Therefore, this study uses 70MPa or more as a HSC flat plate specimens.

From the previous summary, it is clear that the European design code consider the influence of the flexural reinforcement on the punching capacity of slab-column connections either directly or indirectly, and in most cases it is increased in proportion to the cubic root of the flexural reinforcement ratio. This capacity is also increased in proportion to the cubic root of the compressive strength; $V_c \propto \sqrt[3]{f_c'}$. The American code, CSA standard and IS 456 do not reflect the influence of flexural reinforcement ratio, but the capacity is increased in proportion to square root of the compressive strength; $V_c \propto \sqrt{f_c'}$.

In this study the code provisions for punching capacity, all the limitations on the magnitude of concrete compressive strength, flexural reinforcement ratio and size effect are ignored in the comparison given. No distinctions was made between f'_c and f_{ck}.

However, the proposed design equation is limited to flat plates with depth not more than 300 mm and concrete compressive strength below 120 MPa. For slabs exceeding these limits, experimental validation is required. The proposed equation is only meant for typical interior columns.

Moreover, the validity of different code approaches to predict the punching shear strength were checked and compared with the new presented formula. It is found that the new proposed equation can be used for high strength or normal strength or both of them with a reasonable accuracy. This equation for all strength levels studied, gave conservative results compared to ACI 318, EC2, CSA and IS456 codes.

Generally the investigations show that the punching shear predictions of ACI 318-14 is conservative and with large scatter compared with Euro-02 code formula which is more accurate and with smaller coefficients of variation. The CSA and the IS456 code seemed to be in good correlation with experimental results.

The provisions of North American codes ACI 318-14, CSA A23.3-14 which are based on same expression do not include the influence of flexural reinforcement. The proposed equation which is based on regression analysis by using an expert software program depends on results of specimens from different studies and includes the influence of flexural reinforcement ratio.

VI. CONCLUSIONS

This Parametric study showed that the significant parameters that primarily affect the punching shear behavior of slab-column connection of flat plates are concrete compressive strength, slab effective depth, flexural reinforcement ratio and critical perimeter. Despite being a simple equation, the new proposed equation includes the main four significant parameters ($f'c$, ρ, d and b_0) can predict the punching shear capacity with a conservative and reasonable accuracy compared with other available design codes.

1. The proposed equation is found proportional to one half the square root of the compressive strength and to the cubic root of the reinforced ratio, and to effective depth and assumed that the critical perimeter at a distance *0.5d* from the column face as it defined in the ACI 318-14 code. The punching shear strength of plates is a function of the flexural reinforcement ratio, ρ and the investigation shows it is proportional to power 1/3. And a power of 1/2 for $f'c$. This proposed equation is in good agreement with other code equations yet it has a very simple form and is valid for a wide range of normal as well as high strength concrete flat plates. The proposed equation has the following form;

$$V_c = 1.5\left(\sqrt{f_c'}\right)\left(\sqrt[3]{\frac{\rho}{100}}\right)(b_0 \times d) \qquad (in\ SI\ Units)$$

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