EXPERIMENTAL INVESTIGATION AND PREDICTION OF PUNCHING SHEAR STRENGTH OF HIGH STRENGTH CONCRETE SLABS

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ABSTRACT
This research presents an experimental program for investigating punching shear strength of slabs, consisting of 27 high and normal strength concrete slabs. The test data from the experiment are analyzed and divided into three series primarily concerned with the effects of three variables on the punching strength of high-strength (HS) concrete slabs: the concrete strength, the slab depth and the column size and shape. The predictions of typical approaches in the building code, ACI 318-11 are considered along with the approaches developed by Moe, Yitzaki, Herzog, Regan, and Rankin. All approaches are compared with the experimental results.
The experimental results indicated that shear strength for HS concrete slabs is proportional to $fc'^{(n)}$ where the power (n) had been found to be in the range of (0.5 to 0.33). The assumption that the critical perimeter is at a distance (1.5d) from the load area is found to be reasonable and the reinforcement ratio has a considerable effect on the shear strength especially when the slab depth is high.

A modified approach, modelled after the Rankin's approach, is proposed and verified by this experiment and other tests.

1. INTRODUCTION
High Strength Concrete (HSC) has been widely used in the construction industry due to increasing requirements and economical consideration for structures. Since HSC members not only have higher modulus of elasticity but also exhibit different failure mechanism from the normal concrete, it is unsafe to directly extrapolate the models and equations based on normal concrete to HSC members.

Punching in the vicinity of a column is a possible failure mode for reinforced concrete flat slabs. The undesirable suddenness and catastrophic nature of punching failure are of concern to structural engineers. In this respect, the use of high-strength concrete improves the punching strength of HSC flat slabs and allows higher forces to be transferred.

In the last 30 years, progressive researches have been carried out to propose the governing equations for the methods of analysis of punching shear. The analyses fall into two broad groups, (1) those in which the flexural strength or the amount of flexural reinforcement is the prime variable and (2) those in which the concrete compressive strength is the major factor.

In 1961, Moe (1) reported tests of twenty-eight 1.83 meter square slabs. The variables in Moe's tests are cleared through the following semi-empirical equation which was predicted form the experimental results.

Later in 1970, Herzog (2) derived a simple empirical formula to estimate the punching shear strength of slabs. He analyzed the results of fourteen previous investigators, and the main variables taken into consideration were flexural compressive steel ratio (ρ), steel yield strength (f_y) and compressive strength of concrete (f_c').

Regan (3) in 1981 developed an equation to calculate the punching shear capacity of reinforced concrete slabs. Regan's shear perimeter for rectangular columns was a rounded rectangle located (1.25 d) out from the column face, for circular columns it was the circular perimeter located (1.25d) from the column face.

In 1987, Rankin (4) developed a two-phase approach that classifies the punching failure as flexure and shear. First, the shear punching strength and the flexural punching strength are calculated. Then, the results are used to determine the failure mode.

In 1990, Gardner (5) reported tests of thirty circular slabs. The variables in Gardner's tests are concrete strength, steel ratio and slab thickness. He also made comparison with some researchers and code provision on punching shear capacity. Gardener concluded that the steel ratio in the region (3d) from the column should be of the order of 0.5 percent in each direction, and the spacing should be equal to the effective depth. He also found that the cube-root relationship between shear strength and concrete strength is preferable to the square-root relationship.

In 2001, Tuan [6] compared the value of punching shear calculated from Concrete Structures Standards of Australia (AS-3600), 1994[7], with twenty-nine tests results from four research studies. A considerable variety of concrete strengths, slab reinforcement ratios and slab depth is represented in these four approaches.

Tuan [6] concluded that the use of high strength improves the punching shear resistance allowing higher forces to be transferred through the slab-column connections. He also found that the comparison of experimental results shows that AS-3600 [7] formula is applicable up to 100 MPa.

2. MATERIALS

2.1 Cement; Ordinary Portland cements (OPC) was used in the experimental program. It is produced by Al-Sabe'a factory in Lebanon.

2.2 Fine aggregate (sand); Al-a'sela natural sand with maximum size of 4.75 mm was used throughout this work. The grading of the sand was conformed to the Iraqi specification No. 45/1984 [8].

2.3 Coarse aggregate; Crushed gravel from AL-Nibaey region was used throughout this work. According to the recommendations of ACI 211.4R-93 [9] for mix selection of high performance concrete, the maximum size of 10 mm (3/8 in.) for the crushed gravel was selected.

2.4 Superplasticizer; High range water-reducing admixture called SP-1 was used throughout the experimental work. The superplasticizer was produced by (Al-AZRAK Company, Jordan) and it is complied with ASTM C494 type A&F as described in the manual of the product.

2.5 Mixing water; Tap water was used for casting and curing all the specimens.

2.6 Steel reinforcing mesh; One size of normal strength steel wires was used. Wires of size ($\phi 2.5\, mm$) used as a bottom mesh reinforcement for the two phases of research (punching strength and long-term deflection) with 5 mm concrete cover. Yield strength of the wires was determined by tensile test. Results of test showed that the yield strength of the wires of ($\phi 2.5\, mm$) equal to 420 MPa. The number of wires in the punching panels was (15) wires in each direction.

2.7 Molds fabrication; Steel angles were used to fabricate the molds. Four profiled steel angles are assembled using bolts passing through holes in each corner. The assembled test frame is then made to be stood up on a steel base. The base plate is connected firmly to the frame by several bolts through the length of steel angles.

3. EXPERIMENTAL PROGRAM

3.1 Mix design

According to the recommendations of the ACI 211.4R 93 [9] several trial mixes were made. Reference concrete mixture was designed to give a 28-day characteristic compressive strength of 64 MPa. The cement content was 550 kg/m^3, water/cement ratio was 0.32, Vebe time was (6) second, superplasticizer was (1.4%) by weight of cement, and proportions of mix was found to be {1:1.21:1.8} by weight. Table (1) shows details of the mixes used throughout the experimental works.

3.2 Mixing, casting and curing procedure

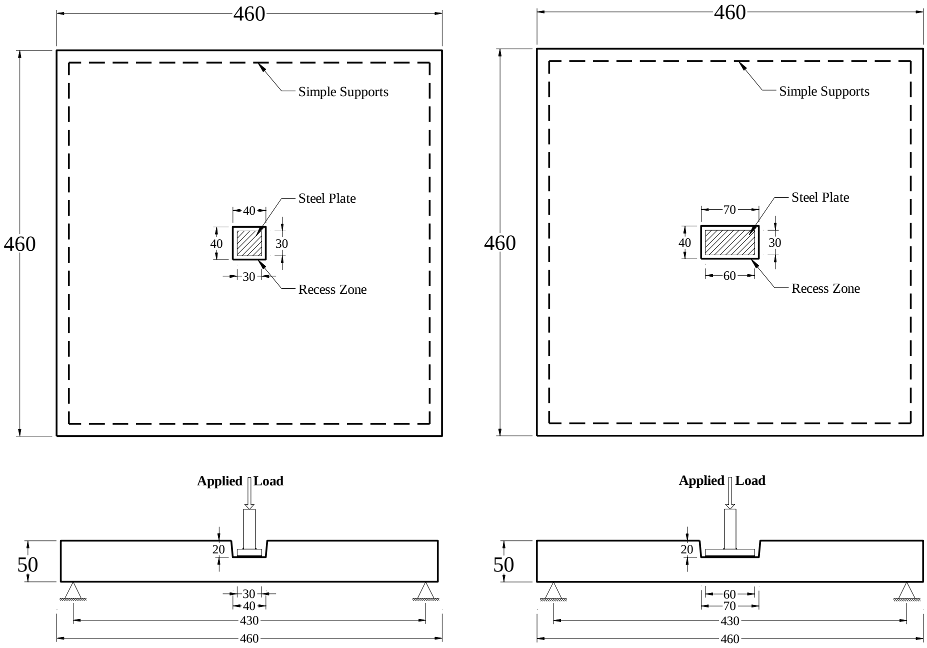
Corresponding to the different types of concrete mixes described previously, nine groups, each group consist of three panels of (460×460×50) mm were cast. These groups are marked as (HS1, HS2,…, and HS7) with two groups of normal strength concrete are marked as (NS1 and NS2), as shown in Table (1). Furthermore, corresponding to each slab, three companion (100 mm) cubes of concrete were cast. The mixing procedure according to the ACI committee 211 [9] was followed as described below:

A. Before mixing, all quantities were weighed and packed in clean containers.
B. The weighed superplasticizerwas added to the measured mix water taking into account the percentage of water contained in the weighted superplasticizer.
C. Saturated surface dry crushed gravel, dry sand, and cement were added to the rotary drum mixer of (0.1 m^3 volume capacity) and (15 r.p.m.) mixing speed. The rotary drum mixed the dry materials for several minutes before adding the water to the mix gradually during two minutes.

Before placing the concrete in the molds, steel wire mesh reinforcement for each slab were placed in the molds. Each slab is reinforced with one steel mesh of ($15\phi 2.5\, mm$ Each Way), distributed in the bottom face. Recess has been introduced to ensure the slabs fail in punching before flexure. Figure (1) shows details of the slabs used in this work.

3.3 Compressive Strength Testing

Based on the British Standard (BS1881-Part 4 1983), the compressive strength test was carried out using FORNEY compression machine on (100 mm cube) specimens. The compressive strength was considered as the average values for three specimens. The result of the test is shown in Table (1).



(a) Square Column (b) Rectangular Column

Figure (1) Details of the punching slabs under test [All dimensions in (mm)]

Table (1) Details types of mixes used, Average values of the compressive strength and Vebe test result

Ser. No.	Slab design ation	ID model design ation	Cement content kg/m^3	Sand kg/m^3	Gravel kg/m^3	Water content Kg/m^3	Super. % by wt. of cement	W/C Ratio	Cube comp. stren. MPa	Av. comp. Stren. MPa	Vebe time sec.
ONE	HS1	HS1-1	550	670	990	165	1.4	0.30	66.8	75.4	11
		HS1-2							74.5		
		HS1-3							84.8		
	HS2	HS2-1	542	640	930	178	1.44	0.33	55.0	64.0	7
		HS2-2							65.0		
		HS2-3							72.0		
	HS3	HS3-1	500	640	930	178	1.44	0.356	50.1	54.9	8
		HS3-2							51.4		
		HS3-3							63.2		
	NS1	NS1-1	350	640	930	178	---	0.51	39.0	36.1	5
		NS1-2							37.3		
		NS1-3							32.0		
TWO	HS4	HS4-1	550	600	925	165	1.4	0.30	89.0	94.2	13
		HS4-2							93.0		
		HS4-3							100.5		
	HS5	HS5-1	550	600	925	165	1.4	0.30	90.5	86.2	10
		HS5-2							86.0		
		HS5-3							82.0		
	HS6	HS6-1	550	600	925	204		0.37	65.4	61.7	8
		HS6-2							54.7		
		HS6-3							64.9		
	NS2	NS2-1	350	600	925	204	---	0.58	36.2	33	4
		NS2-2							33.5		
		NS2-3							29.2		
THR EE	HS7	HS7-1	550	670	990	154	1.4	0.28	64.6	62.3	13
		HS7-2							64.2		

		HS7-3							58.0		

3.4 Preparation of slab specimens

The slab specimens were cast in square pan-shaped form made of 3 mm mild steel plates with bolt-tied removable sides. With each three slabs three cubes were cast. Each batch was sufficient to cast three slabs and three cubes specimens. Having (27) slab specimens, the casting period lasted 35 days.The mix quality and specifications, the casting, and curing procedures were as mentioned above. After applying a thin layer of oil to the form entire surfaces, the slab specimen was cast in two layers each compacted using a table vibrator.
The recess zone was then made by inserting a wooden block, of dimensions identical with that of the recess zone, in the centre of the slab. The block was made such that it can be removed after setting and hardening of the specimen. The top surface of the specimen was well finished using steel trowel so that the upper surface of the wooden block is kept level with the concrete surface.
Twenty seven slab specimens were manufactured and tested upon which the test data and results of this investigation fall. All characteristics and details of these test specimens are listed in Table (2).

Table (2) Detailed characteristics of slab specimen

Series No.	Slab designation	Identical model designation	f_{cu} MPa	Dimensions (mm)					
				Slab Thick.		Slab	Column		Recess Size
				Flexural thick.	Punching thick.		Cross Section	a / b	
ONE	HS1	HS1-1	75.4	43	26	460 × 460 × 50	30 × 30	1	40 × 40
		HS1-2		43.5	26.5				
		HS1-3		43.5	23.5				
	HS2	HS2-1	64.0	43	25				
		HS2-2		47.5	28				
		HS2-3		45	27				
	HS3	HS3-1	54.9	46	25				
		HS3-2		44.5	26.5				
		HS3-3		44	24				
	NS1	NS1-1	36.1	43.5	25.5				
		NS1-2		45	27				
		NS1-3		47	24				
TWO	HS4	HS4-1	94.2	45	28		60 × 30	0.5	70 × 40
		HS4-2		43.5	26				
		HS4-3		44.5	28				
	HS5	HS5-1	86.2	46.5	27.5				
		HS5-2		44	27				
		HS5-3		47	27				
	HS6	HS6-1	61.7	47	29				
		HS6-2		45	26.5				
		HS6-3		45	26				
	NS2	NS2-1	33.0	48	25				
		NS2-2		45.5	27				
		NS2-3		47.5	25				
THREE	HS7	HS7-1	62.3	44	44		40 × 40	1	Without recess
		HS7-2		43	43				
		HS7-3		47	47				

Note: $f_c^{/} = 0.83 f_{cu}$ for HSC > 50 MPa, $f_c^{/} > 0.8 f_{cu}$ and reaches to $0.89 f_{cu}$ for HSC=80 MPa [(10)]

3.5 Punching Test

All slabs were tested at age of 42 days. Before the day of testing, the slabs were taken out from the container of curing and cleaned to be ready for testing. All slabs were tested by universal testing M.F.L. machine model

(8551 MFL System) with maximum capacity of 30 Tons. The slabs were placed in the machine on a rectangular supported steel frame with clear span of (430 mm) as shown in Figure (2). The test was done at a rate of loading of 1.5kN/min. applied by a column load on each slab. A dial gauge is placed at the bottom face at center of slabs to measure the central deflections at each stage of loading. Due to presence of recess in the slabs, all slabs failed in punching.

Figure (2) Illustration of slabs during testing

4. RESULTS OF EXPERIMENTS AND FAILURE CHARACTERISTICS

The disagreement among specifications in the determination of the punching shear area and perimeter, the shear stresses, and the proper factor of safety against punching failure is reasonably expected since the exact location of the punching shear area cannot be observed in a slab.

4.1 Observed Crack Patterns

As the load was increased, radial cracks started to appear and extended from that perimeter towards the slab edges. At the same time the cracks increased in number at the central region of the slab. A complete sudden punching shear failure was occurred by increasing the load. Figure (3) represents some of tested slabs and their cracking patterns. No cracks were observed in the compression side of the slabs, except that which is observed around the loaded area at failure.

4.2 Load-Deflection Behavior of the Tested Slabs

Load-deflection curves under the centre of the loaded area for some of the twenty seven slabs which failed in punching are shown in Fig.(4) [15].

From these figures, it can be seen that concrete slabs made with HSC had reduced deflections at all loading stages over the ordinary concrete slabs and moreover, an increase in the ultimate load at failure is recorded.

4.3 Observation of Failure

Punching shear failure had occurred suddenly in all the tested slabs. There was no sign of warning before the occurrence of failure, except the rapid movement of the dial gauge.

4.4 Shape of the Failure Zone

It was observed that the shape of the failure zone in plan is ranging from a circle to a square with round corners. No significant difference in the shape of failure zone is found between NSC slabs and HSC slabs. The shape can be modelled similar to that proposed by the ACI 318[11]. Figure (3) shows shapes of the failure zones of the tested slabs.

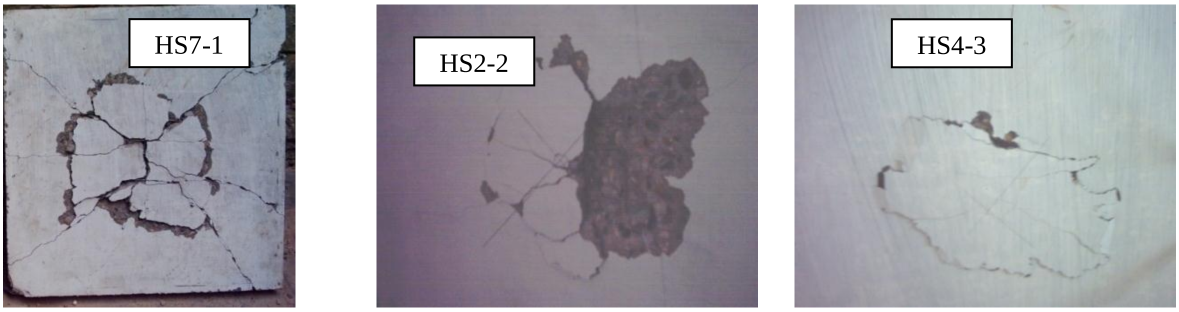
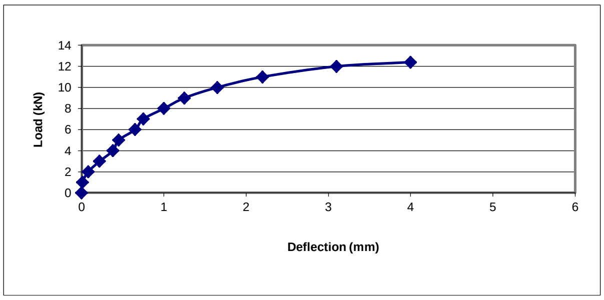
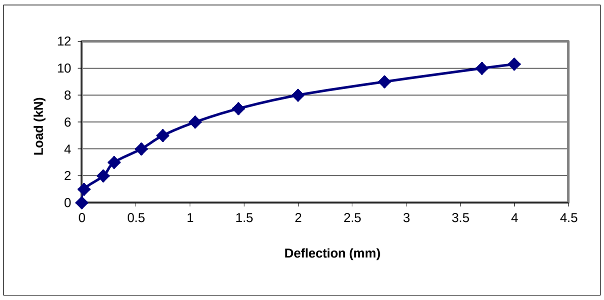


Figure (3) Sample of modes of failure of some tested slabs



Load-Deflection Curve for Slab HS1-1



Load-Deflection Curve for Slab HS3-3

Figure (4) Sample of Experimental Load-Deflection Curve for series one[15]

4.5 Size of the Failure zone

The areas of the punching failure zones and their perimeters were measured following same procedure by Ahmed [15]. Also the failure angles of the punching pyramid were measured by indicating the dimensions of crushed zone at the center line passing through the loaded area. It was observed that the angle of failure was about 18.24^{o} with respect to horizontal for slabs of group HS1. The angle was gradually decreased by increasing column side, and the angle was about 17.77^{o} in slabs of group HS4.

5. STRENGTH PREDICTION

The ultimate design load for the conventionally reinforced concrete slab specimen of all groups was calculated according to the ACI (318-11M) code [11] and six other formulae. Most of these methods defined a critical section from column face and gave a limiting value for the shear stresses.

The calculated loads are listed in Tables (3) to (11). The ACI (318-11M) code [11] formula gave predicted loads below the actual failure test load by 12.9%. In average, the predicted loads using the formulae of Moe [1], Yitzaki [12], Herzog [2], Regan [13], ACI [11] and Rankin [4] are lower than the actual failure load by 16.7%, 27.9%, 0.01%, 0.09%, and 0.03% respectively.

Table (3) Experimental and calculated load at failure of the slab of group HSI according to various formula (30×30 col. and f_{cu}=75.4 MPa)

Method	Average load (kN)	$\frac{Ave.calc.}{Ave.Exp.}$
Moe [1]	9.68	0.76
Yitzaki[12]	7.66	0.60
Herzog [2]	11.55	0.91
Regan [13]	10.77	0.85
ACI (318-11)[11]	10.01	0.79
Rankin [4]	11.34	0.90
Experimental	12.67	

Table (4) Experimental and calculated load at failure of the slab of group HS2 according to various formula (30×30 col. and f_{cu}=64 MPa)

Method	Average load (kN)	$\frac{Ave.calc.}{Ave.Exp.}$
Moe [1]	10.34	0.88
Yitzaki[12]	8.47	0.72
Herzog [2]	11.68	0.99
Regan [13]	11.14	0.95
ACI (318-11)[11]	10.14	0.86
Rankin [4]	11.48	0.98
Experimental	11.77	

Table (5) Experimental and calculated load at failure of the slab of group HS3 according to various formula (30×30 col. and f_{cu}=54.9 MPa)

Method	Average load (kN)	$\frac{Ave.calc.}{Ave.Exp.}$

Moe [1]	8.77	0.81
Yitzaki[12]	7.48	0.69
Herzog [2]	9.73	0.90
Regan [13]	9.57	0.88
ACI (318-11)[11]	8.43	0.78
Rankin [4]	9.57	0.88
Experimental	10.87	

Table (6) Experimental and calculated load at failure of the slab of group NS1 according to various formula (30×30 col. and f_{cu}=36.1 MPa)

Method	Average load (kN)	$\frac{Ave.calc.}{Ave.Exp.}$
Moe [1]	7.98	0.78
Yitzaki[12]	7.56	0.74
Herzog [2]	8.09	0.80
Regan [13]	8.52	0.84
ACI (318-11)[11]	7.34	0.72
Rankin [4]	7.95	0.78
Experimental	10.17	

Table (7) Experimental and calculated load at failure of the slab of group HS4 according to various formula (30×60 col. and f_{cu}=94.2 MPa)

Method	Average load (kN)	$\frac{Ave.calc.}{Ave.Exp.}$
Moe [1]	13.36	0.79
Yitzaki[12]	10.85	0.64
Herzog [2]	19.20	1.13
Regan [13]	15.40	0.91
ACI (318-11)[11]	16.61	0.98
Rankin [4]	18.85	1.11
Experimental	17.00	

Table (8) Experimental and calculated load at failure of the slab of group HS5 according to various formula (30×60 col. and f_{cu}=86.2 MPa)

Method	Average load (kN)	$\frac{Ave.calc.}{Ave.Exp.}$
Moe [1]	12.90	0.87
Yitzaki[12]	10.70	0.72
Herzog [2]	18.15	1.23
Regan [13]	14.77	1.00
ACI (318-11)[11]	15.72	1.06
Rankin [4]	17.83	1.20
Experimental	14.80	

Table (9) Experimental and calculated load at failure of the slab of group HS6 according to various formula (30×60 col. and f_{cu}=61.7 MPa)

Method	Average load (kN)	$\frac{Ave.calc.}{Ave.Exp.}$
Moe [1]	11.96	0.91
Yitzaki[12]	10.63	0.81
Herzog [2]	15.38	1.17
Regan [13]	13.23	1.01
ACI (318-11)[11]	13.32	1.02
Rankin [4]	14.90	1.14
Experimental	13.10	

Table (10) Experimental and calculated load at failure of the slab of group NS2 according to various formula (30×60 col. and f_{cu}=33 MPa)

Method	Average load (kN)	$\frac{Ave.calc.}{Ave.Exp.}$
Moe [1]	9.01	0.78
Yitzaki[12]	9.40	0.81
Herzog [2]	10.31	0.89
Regan [13]	9.91	0.86
ACI (318-11)[11]	8.93	0.77
Rankin [4]	10.12	0.88
Experimental	11.53	

Table (11) Experimental and calculated load at failure of the slab of group HS7 according to various formula (40×40 col. and f_{cu}=62.3 MPa)

Method	Average load (kN)	$\frac{Ave.calc.}{Ave.Exp.}$
Moe [1]	26.93	0.92
Yitzaki[12]	22.17	0.76
Herzog [2]	29.00	0.99
Regan [13]	25.32	0.87
ACI (318-11)[11]	25.12	0.86
Rankin [4]	23.60	0.81
Experimental	29.23	

6. PROPOSED SHEAR STRENGTH PREDICTION

Based on present experiment, the relationship between the variables and the punching shear strength of HSC flat slabs could be defined as below:

$$P_{vs} \alpha \sqrt[4]{100\,\rho}\,.\sqrt[3]{f_c'}\,(c+d)\,d \quad \text{.......... (1)}$$

Compared with all approaches for predicting the punching strength, the Rankin approach gives good correlation with the test results, although it could not express the proper relationship between punching shear and concrete compressive strength. Therefore, a modified approach, improved from Rankin's, is proposed to predict the punching shear strength of HSC flat slabs.

$$P_{vs} = 3.12\sqrt[4]{100\,\rho}\,.\sqrt[3]{f_c'}\,(c+d)\,d \quad \text{.......... (2)}$$

This modified equation was derived using statistics (Statistica Software by using non-linear regression). It accurately shows that the punching shear strength is proportional to the cube root of concrete compression strength, the quadratic root of reinforcement ratio, the effective slab depth and the critical perimeter assuming the width of load area plus the effective slab depth each side. To verify the proposed approach, its validity is checked by comparing it with the present experimental work. Table (12) shows the comparisons of the test results for the present study with ACI code and the proposed approach.

The mean value of (V_u / V_{exp}) for the present study is 0.85 and the coefficient of variation is 11.5%. This coefficient of variation corresponds to an improvement of approximately 5% over the next coefficient of variation (16.7%) given by ACI.

Table (12) Comparison of experimental and predicted punching strength

Series No.	Slab designation	ID of model	$P_{u})_{exp}$ kN	$V_{proposed}$ kN	V_u / V_{exp} Proposed - Eq.(2)	V_u / V_{exp} ACI
ONE	HS1	HS1-1	12.4	11.14	0.90	0.85
		HS1-2	13.1	11.54	0.88	0.83
		HS1-3	12.5	9.25	0.74	0.70
	HS2	HS2-1	11.5	9.82	0.85	0.78
		HS2-2	10.6	12.1	1.14	1.04
		HS2-3	13.2	11.31	0.86	0.78
	HS3	HS3-1	10.5	9.33	0.89	0.79
		HS3-2	11.8	10.385	0.88	0.78
		HS3-3	10.3	8.65	0.84	0.75
	NS1	NS1-1	10.2	7.42	0.73	0.72
		NS1-2	12.1	9.347	0.77	0.64
		NS1-3	11.17	7.52	0.67	0.65
TWO	HS4	HS4-1	18.8	13.75	0.73	0.92
		HS4-2	16.4	12	0.73	0.93
		HS4-3	15.8	13.75	0.87	1.09
	HS5	HS5-1	14.5	12.92	0.89	1.11
		HS5-2	15.7	12.5	0.80	0.99
		HS5-3	14.2	12.5	0.88	1.10
	HS6	HS6-1	12.8	12.73	0.99	1.16
		HS6-2	13.0	10.8	0.83	0.98
		HS6-3	13.5	10.43	0.77	0.91
	NS2	NS2-1	11.1	7.87	0.86	0.79
		NS2-2	12	9.07	0.91	0.80
		NS2-3	11.5	7.87	0.83	0.73
THREE	HS7	HS7-1	29.2	26.78	0.92	0.84
		HS7-2	28.4	25.64	0.90	0.82
		HS7-3	30.1	30.33	1.01	0.92
Mean					**0.85**	**0.87**
Coefficient of variation					**0.115**	**0.167**

7. PREVIOUS EXPERIMENTAL PROGRAMS

The experimental program conducted by Marzouk and Hussein [16] was based on a series of practical configurations of a conventional slab-column system. Seventeen specimens were subjected to concentric vertical loading. The specimens were simply supported along all four edges to simulate the lines of contra flexure. Such specimen represented the region of negative bending moment around an interior column. These specimens were divided into four groups in order to investigate important parameters of flat slabs.

Full details of the test slabs are given in Table (13). Reinforcing bars consisted of Grade 400 steel conforming to CSA standards with actual yield strength of 490 MPa. The concrete mix was designed to produce a 28-day strength of 70 MPa.

The specimens were loaded monotonically through the centre stub column. The load was applied with a hydraulic actuator that has a maximum capacity of 670 KN. During the test, the slabs were carefully inspected. Extensive measurements of the strains were used at key locations on the flexural bar and surface of concrete slabs. Linear variable displacement transducers (LVDT) were used to measure the deflection of the centre of the slabs, and dial gages were used to measure the vertical displacements at the slab corners. Others explained programs [17,18] are shown together with Marzouk and Hussein's in Table (13).

Table (13) Comparison of experimental and predicted punching strength

Ser. No.	Slab No.	Comp. Strength (MPa)	Effective length, a (mm)	Column size (mm)	Average depth (mm)	Steel (ρ)	Exp. Ult.load (kN)	$V_{prop.}$	V_u / P_{exp}	
									Propo.	ACI
Marzouk and Hussein (1991) [16]	NS1	42	1500	150	95	1.473	320	286	0.89	0.63
	HS1	67	1500	150	95	0.491	178	175	0.98	1.42
	HS2	70	1500	150	95	0.842	249	286	1.15	1.04
	HS7	74	1500	150	95	1.193	356	328	0.92	0.75
	HS3	69	1500	150	95	1.473	356	338	0.95	0.72
	HS4	66	1500	150	90	2.370	418	348	0.83	0.56
	HS2	30	1500	150	120	0.944	396	319	0.80	0.60
	HS5	68	1500	150	120	0.640	365	356	0.98	0.97
	HS6	70	1500	150	120	0.944	489	422	0.86	0.74
	HS8	69	1500	150	120	1.111	436	438	1.00	0.82
	HS9	74	1500	150	120	1.611	543	492	0.91	0.68
	HS10	80	1500	150	120	2.333	645	554	0.86	0.60
	HS11	70	1500	150	70	0.952	196	173	0.88	0.87
	HS12	75	1500	150	70	1.524	258	232	0.90	0.69
	HS13	68	1500	150	70	2.000	267	240	0.90	0.63
	HS14	72	1500	220	95	1.473	498	440	0.88	0.68
	HS15	71	1500	300	95	1.473	560	544	0.97	0.75
Ramdane (1996) [17]	slab5	54.4	1372	150	100	0.580	190	194	1.02	1.29
	slab12	60.4	1372	150	100	1.280	319	335	1.05	0.81
	slab15	68.4	1372	150	100	1.280	276	349	1.26	1.00
	slab16	99.2	1372	150	100	1.280	362	395	1.09	0.91
	slab22	84.2	1372	150	100	1.280	405	374	0.92	0.75
	slab23	56.4	1372	150	100	0.870	341	277	0.81	0.73
Tomaszewicz (1993) [18]	nd65-1-1	64.3	2500	200	290	1.420	2050	1995	0.97	0.74
	nd95-1-1	83.7	2500	200	290	1.420	2250	2178	0.97	0.77
	nd95-1-3	89.9	2500	200	290	2.430	2400	2551	1.06	0.75
	nd115-1-1	112	2500	200	290	1.420	2450	2400	0.98	0.82
	nd65-2-1	70.2	2200	150	210	1.660	1200	1136	0.95	0.70
	nd95-2-1	88.2	2200	150	210	1.660	1100	1226	1.11	0.86
	nd95-2-3	89.5	2200	150	210	2.490	1250	1364	1.09	0.76
	nd115-2-1	119	2200	150	210	1.660	1400	1355	0.97	0.78
	nd115-2-3	108.1	2200	150	210	2.490	1550	1452	0.94	0.67
	nd95-3-1	85.1	1100	100	95	1.720	330	300	0.91	0.69
Mean									**0.962**	**0.793**
Coefficient of Variation									**0.105**	**0.231**

The mean value of (V_u/V_{exp}) for the present study and other experiments [Tables (12) and (13)] is (**91.4%**) and the coefficient of variation is found 12.4%. While the mean value of (V_u/V_{exp}) for ACI is (**82.6%)** and the coefficient of variation is 20.5%. Therefore, the adoption of this proposed approach for predicting the punching strength of HSC flat slabs would provide a more accurate prediction. Furthermore, it considers and efficiently uses the reinforcement and the concrete.

8. CONCLUSIONS

The following conclusions can be drawn based on the results of this work.

1. There is an increase in shear strength with the increase of compressive strength, this increase is found to be (47 %) in rectangular loading area when the compressive strength increased from (33 MPa) to (94.2

MPa). However, this increase is found to be (25 %) in square loading area when the compressive strength increased from (36.1 MPa) to (75.4 MPa).

2. It was observed that the shape of the failure zone in plan is ranging from a circle to a square with round corners.
3. The size of the failure zone decreased by decreasing compressive strength. This indicates the major effect of high compressive strength punching shear strength of slabs. The size of the failure zone increased by increasing column dimension ratio. The average area has increased (12 %) when the compressive strength increased from (36.1 MPa) to (75.4 MPa) for square loading. The increase in area is found to be (21 %) in rectangular loading area when the compressive strength increased from (33 MPa) to (94.2 MPa).
4. The failure angles of the punching pyramid were measured by indicating the dimensions of crushed zone at the centre line passing through the loaded area. It was observed that the angle of failure was about 18.24^0 with respect to horizontal for slabs of group HS1. The angle was gradually decreased by increasing concrete strength to about 17.77^0 in slabs of group HS4, the angle was decreased to about 17.89^0 in slabs of group HS5 by increasing column side ratio, and the angle was about 20.67^0 in slabs of group HS7.
5. A modified approach, improved from Rankin's, is proposed to predict the punching shear strength of HSC flat slabs (Eq.4.2). This modified equation accurately shows that the punching shear strength is proportional to the cube root of concrete compression strength, the quadratic root of reinforcement ratio, the effective slab depth and the critical perimeter assuming the width of load area plus the effective slab depth each side.

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