The Influence of Aggregates on Punching Shear Resistance of Slabs-Without Shear Reinforcement

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ABSTRACT: **The choice of aggregate type in producing reinforced concrete depends on the availability of the source sometimes and the intended concrete requirements like lightweight or normal aggregate concrete or high strength concrete. The punching shear resistance is being considered to be influenced by numbers of parameters including aggregate size and types. These parameters have not accounted in most of codes of design and have given a little attention by researchers. Most of available knowledge are based on outcomes from experimental works on beams. In this paper, the considerable slab tests without shear reinforcement are collected from literature in which aggregate types and sizes are given and they were failed in punching. The test results are compared to those calculated by ACI, EC2 and CSCT. The deficits of shear resistance are found clear where high compressive strength is combined with reinforcement ratio.**

Keywords: Aggregate interlocking, Aggregate size, Aggregate type, Crack width, Compressive strength of concrete and Punching shear strength.

I. INTRODUCTION

For the members without shear reinforcement, the shear capacity to resist shear force comes from the combine action of three mechanisms. Dowel action of the flexural reinforcement, aggregate interlock where the forces transmit through the interlocking of aggregate pieces in the cracks of concrete and ending with the resistance of the uncracked portion of the concrete where the stress is compression. The shear resistance by dowel action is limited to the tensile strength of concrete cover to the reinforcement. The aggregate interlocking with cement paste takes the role in transferring the shear through the concrete. For a cracked section, the fracture in aggregate causes the reduction in shear strength and as the transferring of shear becomes impossible. The rate of the shear strength reduction depends on the width of the cracks until the section fail in shear. The portion of contribution for theses mechanisms is not well established in literature, but the design equations for shear is most often represented in a form of empirical equations introducing various parameters.

The current codes of ACI [1] and EC2 [2] treat the shear strength as a function of the compressive strength of concrete only with a complete ignoring of the influence from the size and type of aggregate in predicting of the shear strength, however the critical shear crack theory (CSCT) by Muttoni [3] , which is the basis of current fib Model Code [4], considers the aggregate size only. The equations in CSCT treat the shear crack where the aggregate size is included in which the size is taken as a function of a rotation caused to the member when cracked.

The researchers are trying to confirm some influences from aggregate type and size on the behavior and shear strength of reinforced concrete members. There are experimental works on reinforced concrete beams aimed to investigate the influence of the aggregate size and types on the shear behavior. Some evidences came to light, like decreasing of maximum aggregate size causes inevitably fractures in concrete, Walraven and Stroband [5] investigates that when relatively wide cracks occur, and the maximum aggregate size has a considerable effect on the transfer of shear force. From Taylor [6] works on limestone aggregate showed that the shear strength decreased with increasing the compressive strength of concrete. Kawars [7] found that the punching shear strength is influenced by the aggregate type in small scale tests using wide range types of aggregate.

Regan et al. [8] studied the response from limestone and gravel on shear behavior in reinforced beams.It was observed that for beams with depth in the range of 500-700mm, the test strength are below the calculated strength and it was recommended to modify the limit of compressive concrete strength of $\leq 90MPa$ by EC2 to $\leq 50MPa$.

Fig.1 shows the shear strength predictions by EC2 against f_c' for simply supported beams with a range of aggregate types and $195mm \leq d \leq 298mm$. The beams with limestone and where $f_c' \leq 100MPa$ show a clear decrease in the V_u when f_c' increases.

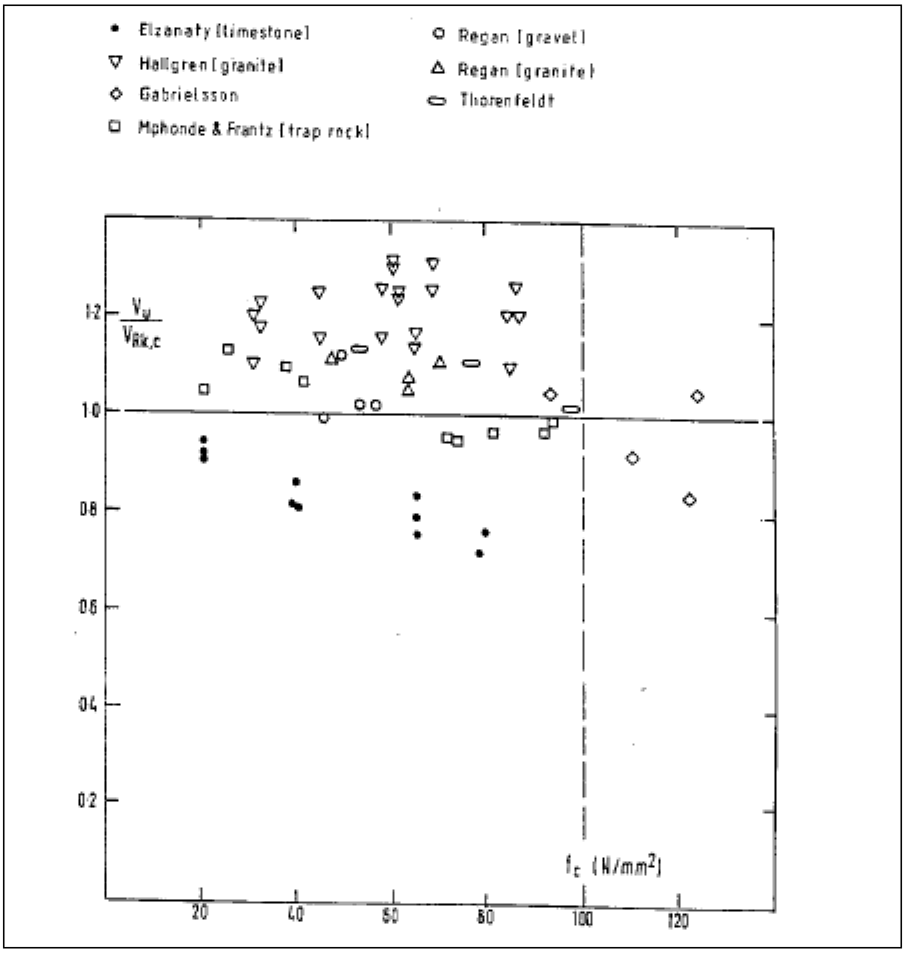


Fig. 1. $V_u / V_{RK,c}$ for various aggregates types in small beams (Duplicated from Ref.5)

Sagaseta [9] seems to conduct a research project completing the study by Regan et al. on the influence of aggregate fracture from different type of aggregates. It was confirmed the limit for concrete strength to EC2 as already recommended by Regan, the early aggregate fracture in beams with limestone rather than gravel resulted in obtaining the lower characteristic shear strength by 16%.

However, the experimental works by Sherwood et al. are aimed to investigate the aggregate size influence on shear strength in thick slabs. The specimens have been treated as wide and deep beams representing strip of thick slabs. It was found that ACI predictions are overestimated the actual strength due to the size effect of members, therefore it was recommended to including the size effect into the design equation.

On the other hand, there are contradictory in researcher's observation regarding the influence of aggregate type on shear strength. Walraven and Stroband [5] and Hamadi and Regan [10] conducted push-off tests and concluded that shear strength for HSC and LWAC at aggregate fracture becomes reduced. For beam specimens with stirrups, Hamadi and Regan used expanded clay lightweight aggregate and it was found a reduction in shear strength. Sagaseta and Regan tested beams without stirrups using limestone aggregate and showed a considerable reduction in shear strength due to the aggregate fracture, also confirmed that the compressive strength of concrete is lower in concretes with limestone aggregate than those with natural siliceous gravel. Sherwood et al. [11] observed that the shear strength of beams with NWC increased by 24% when the maximum aggregate size increased from 9.5 to 21mm and the rate of increasing becomes constant beyond an aggregate size of 25mm.While; there was no reduction in shear strength found in beams with LWAC and HSC by Walraven and Al-Zubi [12].

There are considerable tests on flat slab which are not studied intentionally to investigate the influence of aggregate types and sizes on shear punching shear behavior. So, this paper concerns this aspect with respect to predictions by the current codes of ACI, EC2 and the critical shear crack theory (CSCT) by Muttoni.
The equations by ACI, EC2 and CSCT are shown in Table. I.

Table (1) Summary of design equations by codes of practice and CSCT

Code	Equation	Notation
EC2	$V_{Rk,c} = v_{Rk,c} u_1 d \le v_{Rk,\max} u_0 d$ $v_{Rk,c} = 0.18k(100\rho_1 . f_{ck})^{1/3}$ $k = 1+\sqrt{200/d} \le 2.0$ $v_{Rk,\max} = 0.24(1-f_{ck}/250)f_{ck}$	u_1 is the length of a perimeter to a column at 2d from it. u_0 is the length of the perimeter of the column. d mean effective depth of the reinforcement f_{ck} is the characteristic cylinder compression strength of the concrete is limited to $\le 90MPa$ ρ_1 is the ratio of flexural reinforcement in tension
ACI 308-5	$V_c = \begin{cases} \left(0.16(1+\frac{2}{\beta})\right)\lambda\sqrt{f_c'}b_o.d \\ 0.083\left(\frac{\alpha_s d}{b_o}+2\right)\lambda\sqrt{f_c'}b_o.d \\ 0.33\lambda\sqrt{f_c'}b_o.d \end{cases}$	β ratio of the long side to short side of column α_s is a coefficient, is 40 for internal columns, 30 for edge columns and 20 for corner columns b_o is a perimeter length at $d/2$ away from the face of the column d is the effective depth of the slab f_c' is the characteristic concrete strength and the maximum value of $\sqrt{f_c'}$ is limited to 8.3 MPa . b_o is the length of a perimeter constructed to obtain the minimum length without coming closer to the columns than 0.5d from it.
CSCT	$V = \frac{0.75 b_0 d\sqrt{f_c}}{1+15\psi\, d/(16+d_g)}$ $V_R = \left[\frac{\psi\, d\, E}{1.5 r_s f_y}\right]^{2/3} V_{flex}$ $V_{flex} = 2\pi m_R r_s / (r_q - r_c)$ $m_R = \rho f_y d^2\left[1-\frac{\rho}{2}\frac{f_y}{f_c}\right]$	ψ is the rotation of the slab (in radians) d_g is the maximum size of the aggregate, should be taken as 0 for high strength concrete V_{flex} is the yield-line flexural capacity of the slab m_R is the plastic moment of resistance at a yield line. The radii r_c , r_q and r_s are as shown in Fig. (1).

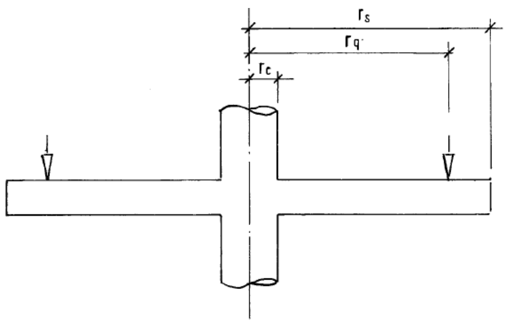


Fig. 2. Definition of r_c , r_q and r_s by CSCT

II. THE AVAILABLE TEST RESULTS

The collected test results are from Europe and North of America. The specimens are 36 square slabs by (Rizk and Marzouk [13], Rizk, Marzouk and Hussain [14], Moe [15], Regan [16] and Ramdane [17] and 44 circular slabs by (Kinnuen [18],Tolf [19],Hallgren [20], Marzouk [21] and Regan [22]). The types of aggregate are gravel, granite, crushed quartzite sandstone, sandstone, coarse sand, limestone and lay tag The aggregate sizes are 5, 6, 9.5, 10, 12,16,20,32 and 38 mm. Details of specimens are shown in Table (2). All slabs were loaded vertically except those by Rizk and Marzouk [13], and Rizk, Marzouk and Hussain [14] were loaded horizontally. The study concentrated on tests where failed in punching shear and those failed in flexure are omitted. However, the study includes tests with unknown aggregate types but for the consideration of their aggregate size influence on the punching shear behavior. The effective depth of the tests is varied between 64 mm to 619mm. The compressive strength of concrete is between 23.7MPa to 108.8MPa. The flexural reinforcement ratios are shown for all specimens which are varied between 0.33% to 1.58%.

Table 2. Summary of tests results from literature

Author	No. of tests	Agg. type	Agg. size (mm)	Effective depth of slab (mm)	Slab size and type	Col. Size and type (mm)	ρ %	f_y $\left(\frac{N}{mm^2}\right)$	f'_c $\left(\frac{N}{mm^2}\right)$
Kinnunen	2	G	16	101-201	700-1200S	120-240C	0.51-0.52	678-720	23.7-25.5
	1		38	619	4680-5820S	800 C	0.55	622	30.6
Tolf	4	G	16	98-100	1270C	125C	0.34-0.81	701-720	22.9-28.6
	4		32	197-200	2540C	250C	0.34-0.80	657-670	22.9-25.4
Hallgren	7	Gr.	20	194-202	2540C	250C	0.33-1.19	596-634	84.1-108.8
Marzouk	12	CQSS	20	70-120	1700C	150-300S	0.84-2.37	490	42-80
Rizk& Marzouk	3	Gr.	20	205-255	1900-2650S	250-400S	0.52-0.66	400	40-76
Rizk, Marzouk&Hussain	4	SS	20	262.5-312.5	2650S	400S	0.50-1.58	460	40-76
Moe	6	G	9.5	114	1830S	152-457S	1.14	328	26.6-27.6
			38			Varied S	1.06-1.52	328-482	20.8-35.2
Regan	1	CS	5	64	1800S	80C	0.98	480	9.0-42.8
	2	G	10	64-128	1800S	80C	0.98	480-485	
	20		20	64-128	1800-2745S 1500C	54-150C 200S	0.75-1.49	464-628	
Ramdane	3	LG	6-12	98	1700C	150C	0.58-1.28	550-650	44.6-68.4
	5	LS	10	98					26.9-108
	6	G	10-20	98-100					32.9-90.3

Abbreviations: Agg.: Aggregate, Co.; Column, C: Circle, S: Square, G: Gravel, Gr: Granite, CQSS: Crushed quartzite sandstone, SS: Sandstone, CS: Coarse sand, LS: Limestone and LG: Lytag

III. CODE PREDICTIONS AND ANALYSIS

Test results and calculations by ACI, EC2 and CSCT are summarized in Table 3. The ratios of test results to calculated loads show the lower value for those by ACI than the other two.

Fig. 3 and 4 (a,b and c) shows V_{Test}/V_{Calc} for AC1,EC2 and CSCT plotted against f_c for the slabs with gravel aggregate and a range of other aggregate types respectively, and have $64mm \le d \le 200mm$, as detailed in Table (1). The predictions of punching shear strength by ACI, as shown in (a) of both Figs. 3 and 4, are conservative for the most of tests and for all types of aggregate with V_{Test}/V_{ACI} above 1.20 .

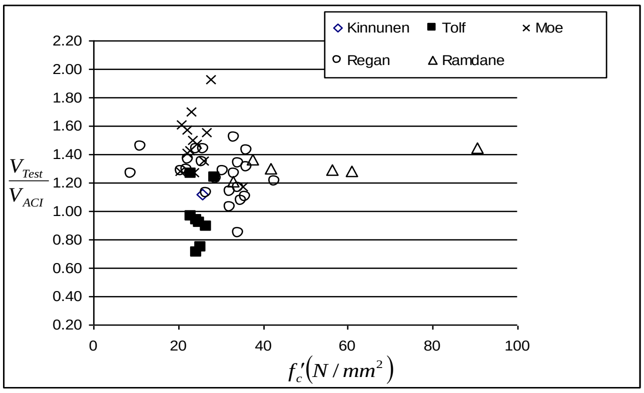
For gravel aggregates, there are problems with some tests by Tolf, the thicker slabs which have reinforcement ratio of 0.35% and 0.80% and the thinner slabs with reinforcement ratio of 0.35% showed low ratio of V_{Test}/V_{ACI}. However the most unsafe results are the thicker slabs with reinforcement ratio of 0.35% which obtained V_{Test}/V_{ACI} equal to 0.71-0.75. It is noted that the thicker slab showed lower strength than thinner for the similar compressive concrete strength and reinforcement ratio but larger aggregate size. There is only one test by Regan (V1) showed lower than unity and the reason could be the small column section which caused earlier punching. For other aggregates, all granite tests by Hallgren showed low V_{Test}/V_{ACI} between 0.80-1.00 and except in one test equal to 0.52% which has reinforcement ratio equal to 0.33% as it is much smaller among them. Also, slab 5 with lay tag aggregate by Ramdane showed V_{Test}/V_{ACI} lower than unity as it has lower reinforcement ratio than the other test.

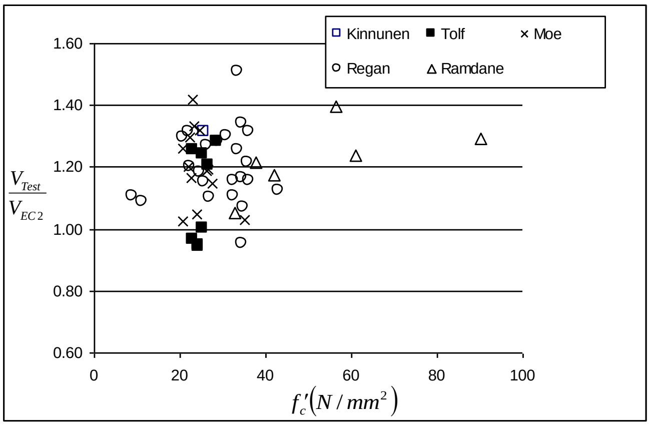
Fig. 3 and Fig. 4 in (b) showed that the resistance provided by EC2 has an overall in a good agreement with the actual resistances except thicker slabs by Tolf. For granite tests by Hallgren, the EC2 showed higher strength than the actual and this was not the case with Rizk's granite tests and the reason is not clear. However the differences between thicknesses in both cases are so small but the increasing of depth over 200mm makes the value of k below 2.0 and had its rule in decreasing the EC2 shear resistance below the actual resistance. This confirms the reduction in shear resistance when the depth increases. The amendment to the limit if concrete strength of $\leq 90MPa$ to $\leq 50MPa$ as proposed by Regan gives a safe prediction of V_{Test}/V_{EC2} above 1.12.

Table 3. Summary of test results and calculations by ACI, EC2 and CST

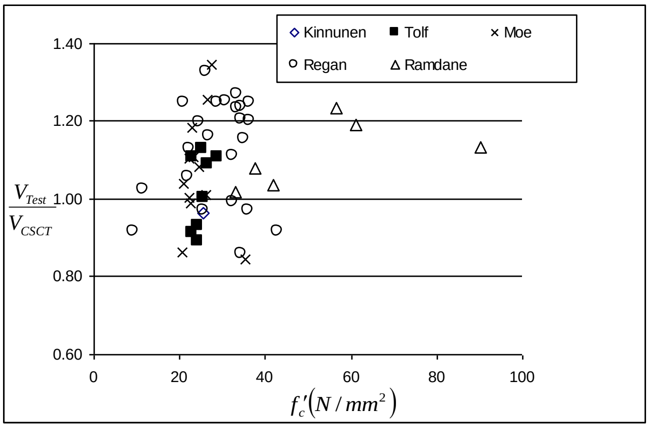
Author	Test	Agg. Types	V_{Test} (kN)	V_{ACI} (kN)	V_{CSCT} (kN)	V_{EC2} (kN)	$\frac{V_{Test}}{V_{ACI}}$	$\frac{V_{Test}}{V_{CSCT}}$	$\frac{V_{Test}}{V_{EC2}}$
Kinnunen	B2	G	185	165	192	141	1.12	0.96	1.32
	C2	G	573	631	635	547	0.91	0.90	1.05
	S1	G	5378	7012	5470	4607	0.77	0.98	1.17
Tolf	S1.1	G	216	175	195	168	1.24	1.11	1.28
	S1.2	G	194	154	175	154	1.26	1.11	1.26
	S2.1	G	603	643	677	637	0.94	0.89	0.95
	S2.2	G	600	620	658	620	0.97	0.91	0.97
	S1.3	G	145	163	133	120	0.89	1.09	1.20
	S1.4	G	148	161	131	119	0.92	1.13	1.24
	S2.3	G	489	658	487	487	0.74	1.00	1.00
	S2.4	G	444	627	477	471	0.71	0.93	0.94
Hallgren	HSC 0	Gr	965	1077	920	987	0.90	1.05	0.98
	HSC 1	Gr	1021	1077	921	991	0.95	1.11	0.97
	HSC 2	Gr	889	1026	867	927	0.87	1.03	0.96
	HSC 4	Gr	1041	1077	1080	1132	0.97	0.96	0.92
	HSC 6	Gr	960	1086	852	963	0.88	1.13	1.00
	N/HSC 8	Gr	944	1060	915	986	0.89	1.03	0.96
	HSC 9	Gr	565	1095	601	731	0.52	0.94	0.77
Marzouk	NS 1	CQSS	320	193	268	243	1.66	1.19	1.32
	HS 2	CQSS	249	246	254	238	1.01	0.98	1.05
	HS 7	CQSS	356	246	298	272	1.45	1.19	1.31
	HS 3	CQSS	356	246	317	286	1.45	1.12	1.25
	HS 4	CQSS	418	228	333	285	1.83	1.26	1.47
	HS 8	CQSS	436	342	420	386	1.27	1.04	1.13
	HS 9	CQSS	543	342	490	447	1.59	1.11	1.21
	HS 10	CQSS	645	342	567	493	1.89	1.14	1.31
	HS 12	CQSS	258	163	200	180	1.59	1.29	1.43
	HS 13	CQSS	267	163	213	191	1.64	1.25	1.40
	HS 14	CQSS	498	316	380	335	1.58	1.31	1.49
	HS 15	CQSS	560	396	445	385	1.41	1.26	1.46
RIZK	NS 4	Gr	882	755	777	780	1.17	1.14	1.13
	HS 4	Gr	1023	960	888	915	1.07	1.15	1.12
	HS6	Gr	1722	1763	1240	1414	0.98	1.39	1.22
RIZK &Hussain	HSS 1	SS	1722	1885	1392	1492	0.91	1.24	1.15
	HSS 3	SS	2090	1795	2080	2004	1.16	1.00	1.04
	NSS 1	SS	2234	1802	2250	2299	1.24	0.99	0.97
	HSS 4	SS	2513	2208	2756	2632	1.14	0.91	0.95
Moe	H 1	Gr	371	274	368	312	1.35	1.01	1.19
	S1/60	Gr	389	259	351	292	1.50	1.11	1.33
	S1/70	Gr	392	266	362	297	1.47	1.08	1.32
	S5/60	Gr	378	222	319	267	1.70	1.18	1.42
	R2	Gr	311	200	248	261	1.55	1.26	1.19
	M1A	Gr	433	269	417	344	1.61	1.04	1.26
Regan	I/2	Gr	176	129	156	146	1.36	1.13	1.20
	I/4	Gr	194	151	155	149	1.28	1.25	1.30
	I/6	Gr	165	129	132	127	1.28	1.25	1.30
	I/7	Gr	186	152	149	145	1.22	1.25	1.28

	II/1	Gr	825	772	715	770	1.07	1.15	1.07
	II/2	Gr	390	309	316	310	1.26	1.23	1.26
	II/3	Gr	365	313	295	313	1.16	1.24	1.16
	II/4	Gr	117	77	92	78	1.52	1.27	1.51
	II/5	Gr	105	78	87	78	1.34	1.21	1.34
	II/6	CS	105	80	84	80	1.30	1.25	1.32
	III/1	Gr	197	152	186	150	1.29	1.06	1.32
	III/2	Gr	123	97	134	93	1.26	0.92	1.32
	III/3	Gr	214	194	220	176	1.10	0.97	1.21
	III/4	Gr	154	106	150	113	1.46	1.03	1.36
	III/5	Gr	214	159	220	185	1.35	0.97	1.15
	III/6	Gr	248	205	270	220	1.21	0.92	1.13
	V/1	Gr	170	201	198	142	0.84	0.86	1.20
	V/2	Gr	280	274	282	253	1.02	0.99	1.11
	V/3	Gr	265	234	238	229	1.13	1.11	1.15
	V/4	Gr	285	200	237	246	1.43	1.20	1.16
Ramdane	3	LS	169	177	161	161	0.96	1.05	1.05
	5	LG	190	251	204	206	0.76	0.93	0.92
	6	LS	233	281	241	236	0.83	0.97	0.99
	12	LS	319	265	275	285	1.20	1.16	1.12
	13	LS	297	225	247	254	1.32	1.20	1.17
	14	Gr	341	266	276	286	1.28	1.24	1.19
	15	LG	276	282	286	303	0.98	0.96	0.91
	16	LS	362	281	333	343	1.29	1.09	1.06
	21	Gr	286	221	243	276	1.30	1.18	1.04
	22	Gr	405	281	314	357	1.44	1.29	1.13
	23	Gr	341	264	244	276	1.29	1.40	1.24
	24	LG	270	228	248	251	1.19	1.09	1.08
	25	Gr	244	202	232	240	1.21	1.05	1.02
	26	Gr	294	216	242	272	1.36	1.21	1.08



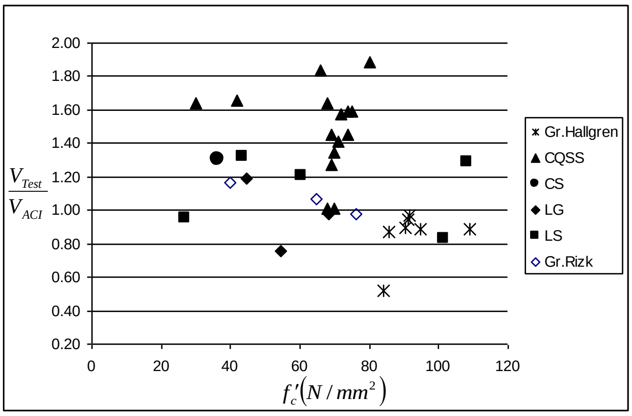


(b) $\frac{V_{Test}}{V_{EC2}}$ for tests with aggregate type of gravel

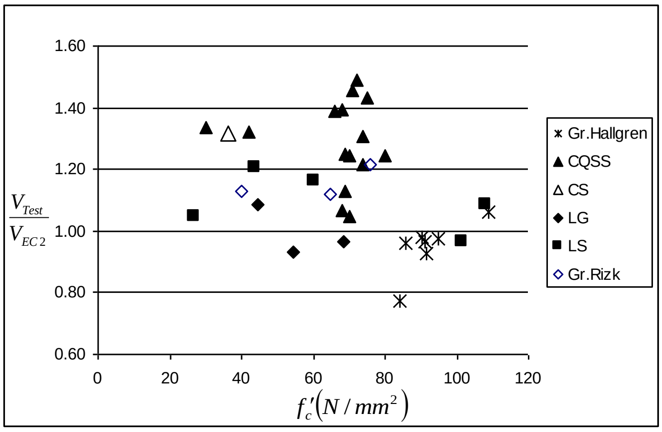


(c) $\frac{V_{Test}}{V_{CSCT}}$ for tests with aggregate type of gravel

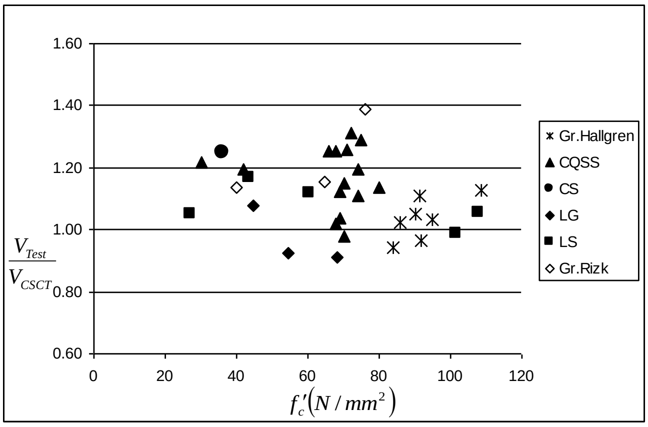
Fig. 3 Comparison of $\frac{V_{Test}}{V_{Calc.}}$ for gravel aggregate for slabs with thickness <200mm



(a) $\frac{V_{Test}}{V_{ACI}}$ for tests with various types of aggregate



(b) $\frac{V_{Test}}{V_{EC2}}$ for tests with various types of aggregate



(c) $\frac{V_{Test}}{V_{CSCT}}$ for tests with various types of aggregate

Fig. 4 Comparison of $\frac{V_{Test}}{V_{Calc.}}$ for other aggregate types for slabs with thickness <200mm

For $10mm$ sandstone aggregates as shown in Fig. 5, tests with low reinforcement ratio equal to 0.58% and compressive concrete strength of 26.9 MPa is not safe only by ACI, while the one with reinforcement ratio equal to 0.58% and compressive concrete strength of 101.6 MPa is underestimated by all present equations. However, the increasing of reinforcement ratio to 1.28% gives good predictions by all equations for compressive concrete strength between 43.6 MPa-108MPa.

It is worthy to discern the test results for slabs with crushed quartzite sandstone from other aggregates, as they obtained secure higher shear strength than the design equations presented here. The average V_{Test}/V_{Calc} for ACI, EC2 and CSCT respectively are 1.49, 1.29 and 1.17, and the C.O.V. is 0.09, 0.08 and 0.14. So, the parameters like size effect and aggregate size in EC2 and CSCT respectively have significant on their evaluations to be better than those by ACI.

The reason could be explained as the stiffness of crushed quartzite sandstone behaves stronger to fracture under the load and this is obviously not accounted in design equations. The results for tests with $d > 200mm$ are shown in Fig. 5. The V_{Test}/V_{Cal} ratios for ACI and EC2 are unsafe for tests with gravel aggregate. Tests with sandstone showed that the low reinforcement ratio is the reason for ACI overestimation rather than other equation as shown in Fig.6. While for overestimation by EC2 and CSCT for tests with d= 312.5mm is not definitely obvious, but could the reduction of the factor k in EC2 due to thicker slab than 200mm be a reason and the higher reinforcement ratio in equations by CSCT resulted in higher shear strength than the actual strength. For 20mm size of granite aggregates by Hallgren and Rizk, only one test (HSC9) by Hallgren showed the worse results among them and it presumably due to load arrangement or other laboratory fault.

The calculated strengths by ACI for tests with granite aggregates for the range of $194mm \le d \le 255mm$ are below the actual strength. Putting a limit of $f_c \le 50MPa$ instead of $\le 68MPa$ similar to EC2 would be a simple solution as it gives the V_{Test}/V_{ACI} above 1.06, however this restriction does not prevent the lower ratio at 60% for the test with very low reinforcement ratio.

There is no evidence from the collected data for the clear role of maximum sizes of aggregate in producing high or low concrete strengths. The grave is the most aggregate in which has various maximum sizes considered here, therefore the test strengths are plotted in Fig.7 to find all size performance. The maximum size of 20 mm performed the highest strength, then 10mm and 16mm where the reinforcement ratio is high. The 32mm size performed low strength where reinforcement ratio is low, even though they preformed poorly compared to slabs with 16mm in the same series of experimental work. There is no chance to decide on 38mm size as there is a constant reinforcement ratio in all slabs.

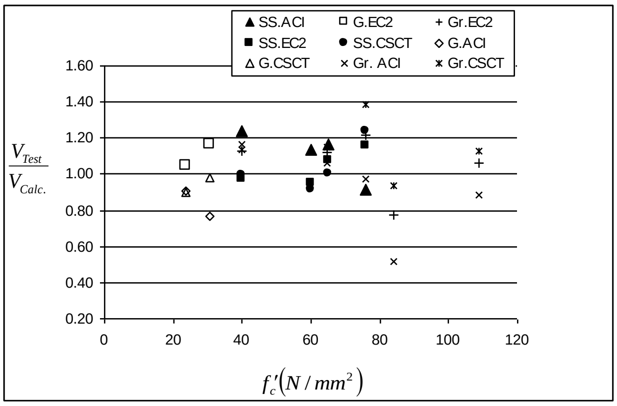


Fig . 5 Comparison of $\frac{V_{Test}}{V_{Calc.}}$ for thickness $\ge 201mm$

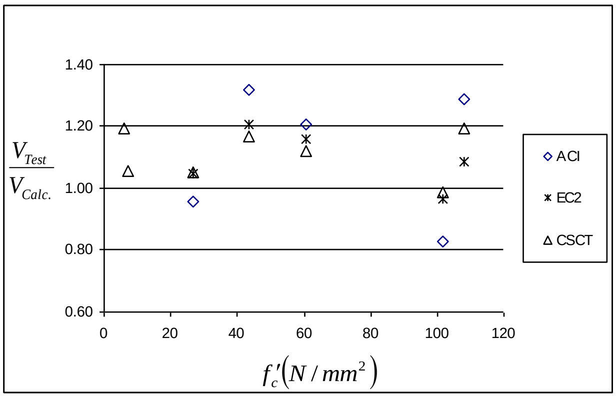


Fig. 6 Comparison of $\frac{V_{Test}}{V_{Calc.}}$ for limestone aggregate

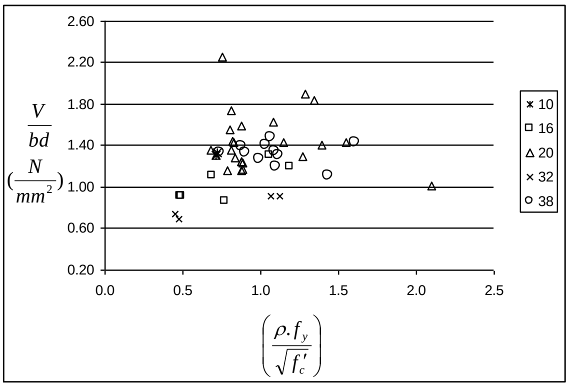


Fig. 7 Strength of slabs with gravel aggregate

CONCLUSIONS

A reasonbale databse of slabs without shear reinforcement have been assesed for the influence from the aggregate on their punching shear strenhth. The assessment is worked according to the current Code of design ACI and EC2 and the method of CSCT by Muttoni . The assessment focused on type and size of aggregate and the compressive strength of these tests.
The conclusions and recommendations of this pqaper are summarised as followings:

1. Test results from literature for slabs without shear reinforcement and loaded centrally through a column or a plate show that type of aggregate and its size has significant influences on shear resistance.

2.The influence of aggregate is related to other parameters like size effect of slabs and flexural reinforcement ratio. For slabs with gravel, granite and lay tag, the lower reinforcement ratio up to 0.51% produced lower shear strength but it is not true for sandstone aggregate.

There is no clear evidence to find whether there is a limit for size of aggregate to be helpful in producing a low strength and high strength. However, the 20mm size of gravel showed the best performance than other sizes, while there are no enough tests to examine the aggregate size by other types of aggregate.

3.The ACI code performed rather poorly than EC2 and CSCT, as it is conservative for most cases and underestimated the shear strength for thick slabs with low reinforcement ratio. The EC2 predictions for granite aggregate are unexpected therefore the proposed limit of $f_c \leq 50N/mm^2$ by Regan is recalled to overcome this deficiency by the Code. Although, imposing this limit on other tests cause about 1.15 times the strength to those without this limit.

4. It is worthwhile to undertake further researches in considering the effects from type and size of aggregates in shear strength equations. This is possibly by assessing the equations by producing coefficients representing these effects
in Codes' equations for shear strength

5.An intensive experimental works could be necessary to assess the influence from size and type of local aggregates in Iraq on the punching shear strength. This is possibly by producing coefficients representing these effects through a wide-range experimental works using the local aggregates from various sources in Iraq.

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Notations

d is the thickness of slab

f_c' is the characteristic compressive strength of concrete

f_y is the yield strength of steel reinforcement

ρ is the ratio of the area of flexural reinforcement to the are of cross section of concrete

V_u is the ultimate shear force

HSC is the high strength concrete

LWAC is the light weight aggregate concrete

NWC is the normal weight concrete

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