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Evaluation of ACI Shear Provisions for Concrete Beams Without Web Reinforcement Using Stepwise Regression

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Abstract: Shear strength of concrete is one of the most critical design requirements in reinforced concrete design. Because of this special requirement the reinforced concrete researchers are continuously review the depended shear design formulas to predict better formulas with a high degree of confidence. In this research, based on an experimental data base from the literature with a total number of 175 beam specimens and using the stepwise regression a testing model is predicted to evaluate the ACI shear provisions for beams without web reinforcement. The squared coefficient of correlation R^2 of the predicted model is 0.934. The results show that the predicted model is adequate as a testing or an evaluation formula. The ACI provisions are found to be conservative for the full range of span to depth fć (a/d) ρ_w. While for width b and depth d, the ACI can be considered as conservative up to specific points and unsafe beyond specific points.

Key words: Shear strength, without web reinforcement, regression, stepwise, span to depth ratio, ratio, ACI

INTRODUCTION

Shear strength of concrete beam without web reinforcement is one of the most important concrete characteristics that should be evaluated carefully in the design of reinforced concrete structures. Although, it is unable to determine the accurately resistance of concrete to pure shearing stress, according to engineering mechanics if pure shear is produced in a member, a principal tensile stresses of equal magnitude will be produced on another plane. Because the tensile strength of concrete is less than its shearing strength, the concrete will fail in tension before its shearing strength is reached.

Shear strength of concrete: While bending is most often the critical failure mechanism for RC sections shear should always be considered. Shear forces are present wherever the bending moment in a member varies along its length, since:

$$V = \frac{dM}{dx} \quad (1)$$

Let's consider the behavior of an RC beam without shear reinforcement. The key to understanding the behavior of such a beam is to remember the following principle:

- Concrete is very weak in tension
- Within the beam there are principal planes along
- Which the tensile and compressive forces are a maximum

If cracks are to occur in a concrete section they are most likely to develop along the most heavily stressed tensile isoclines. Thus, the development of a tensile crack follows the following sequence.

The crack starts as a tensile crack which develops at right angles to the tensile face of the beam. As the load increases the tensile crack follows the path of the tensile isoclines and curves. This crack may develop in different ways each of which leads to shear failure, these are Diagonal tension failure: the diagonal crack extends over the entire depth of the beam, causing the beam to split into two parts. Final failure for this type of crack may be dowel failure. Shear tension or shear bond failure: if the applied load is close to the support then the diagonal crack may stabilize at some point but a secondary crack may develop along the tension steel, causing some loss of bond. As the steel begins to slip, it induces tensile stresses in the concrete adjoining the bars causing the concrete cover to split. This process continues until failure occurs due to loss of anchorage. Shear compression failure: if the diagonal crack does extend deeply into the compression zone then the concrete above the crack may fail due to crushing.

Shear transfer mechanism: Consider the beam which has a flexural/shear cracks running from the extreme tension fiber to the neutral axis. Shear stress transferred across this plane by a combination of the following three actions:

- Direct shear transfer by uncracked concrete in the compression zone

- Mechanical interlocking of the aggregate along the diagonal crack. This is called aggregate interlock or interface shear transfer
- Dowel action of the longitudinal tension steel

Shear is resisted by a combination of these three actions:

$$V_{cu} = V_{cz}+V_{ag}+V_d \quad (2)$$

The overall shear resistance of a section without shear reinforcement is a function of many parameters including the concrete properties and the section size properties in addition to the quantity of the flexural reinforcement. Extensive amount of experimental studies in this field had pointed out the main factors that affect the shear strength of beams with web reinforcement which are: concrete strength, width and depth of the beam cross section, span to depth ratio a/d and the longitudinal reinforcement ratio.

The ASCE-ACI Committee 426 (1973) Eq. 1 has classified failure modes of simply supported rectangular beams without web reinforcement based on the ratio a/d as follows.

Failure mode 1 (a/d>6): This is the case of very slender beams where reinforced concrete tend to fail in flexure even before the formation of inclined cracks.

Failure mode 2 (2.5<a/d<6): In this case some of the flexural cracks grow and may become flexural-shear cracks. The diagonal cracks may continue to the top and bottom faces of the beam and cause yield of the tension steel. The beam may split into two pieces at failure. The crack is called the diagonal tension crack.

Failure mode 3 (a/d<2.5): Which is the case in short beams where a diagonal crack may propagate along the tension steel causing splitting between the concrete and the flexural longitudinal reinforcement bars which is called shear-tension failure. The case where the diagonal crack results in crushing of the compression zone is called the shear-compression failure.

The fourth case is a special case for very short beams where (a/d<1). In this case different types of failure can be observed such as anchorage failure of tension steel, bearing failure, flexural failure, tension failure of arch-rib and compression strut failure.

Figure 1 shows the main three possible types of cracks that may be formed in a simply supported beam subjected to uniformly distributed load.

Data base: The literature database used in this research is composed of 19 sets of experimental results from 19 references (Morrow and Viest, 1957; Mphonde and Frantz, 1984; Ahmad *et al*., 1995; Elzanaty *et al*., 1986; Salandra and Ahmad, 1989; Thorenfeldt and Drangsholt, 1990; Kani *et al*., 1979; Xie *et al*., 1994; Adebar and Collins, 1996; Johnson and Ramiraz, 1989; Yoon *et al*., 1996; Islam *et al*., 1998; Kulkarni and Shah, 1998; Collins and Kuchma, 1999; Fonteboa, 2002; Cladera and Mari, 2005) with a total number of 175 tested beams without web reinforcement. Table 1 presents details about the experimental tests that were carried out during the last six decades (between 1957 and 2005) used in current study.

The dependent variables for the experimentally tested beams includes the geometrical properties in addition to the compressive strength of concrete and the percentage ratio of the longitudinal steel. The geometrical properties include the width of the beam section b which ranges

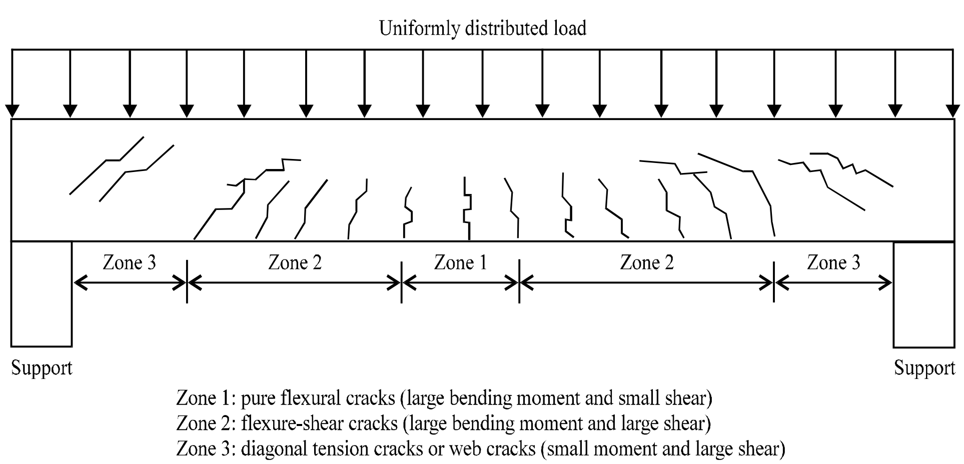

Fig. 1: Possible crack types in a simply supported beam

Table 1: Test results data base

Researches	ID	b	d	fc	ρ_w (%)	a/d	V_{test} kN
Beams without web reinforcement							
Morrow and	b40b4	305	368	34.8	1.85	2.76	155.7
Viest (1957)	b56b2	305	368	14.7	1.85	3.86	100.1
	b56a4	305	375	25	2.41	3.8	137.9
	b56b4	305	368	27.2	1.85	3.86	122.3
	b56e4	305	368	28.4	1.24	3.86	109
	b56a6	308	356	39.9	3.79	4	177.9
	b56b6	305	372	45.7	1.83	3.83	136.8
Morrow and	b113b4	305	365	32.6	1.87	7	104.3
Viest (1957)	b70b2	305	365	16.3	1.87	4.87	88.96
	b70a4	305	368	27.2	2.46	4.83	132.3
	b70a6	305	356	45	3.83	5	177.9
	b84b4	305	363	27.2	1.88	5.87	111.2
Kani *et al*. (1979)	1	150	137	28	2.75	5.39	28.7
	2	150	137	25	2.73	3.93	28.6
	3	150	137	25	2.8	3.02	32.7
	4	156	270	27	2.74	3	65.1
	5	151	270	27	2.84	4	55.4
	6	155	270	30	2.66	6.46	53.6
	7	156	543	26	2.77	4	93.1
	8	156	543	27	2.77	3.12	107.8
	9	156	543	26	2.72	6.84	84.6
	10	154	1090	27	2.71	3	164.4
	11	152	1090	30	2.72	3.98	158
	12	155	1090	27	2.7	7	153.6
	15	152	270	17	0.5	2.98	27.2
	16	152	270	17	0.5	3.53	24.5
	17	152	270	28	0.5	3.47	25.4
	20	152	270	35	0.5	2.57	33.6
	21	152	270	35	0.5	3.52	24.9
	23	152	270	17	0.8	3.96	30.2
	24	152	270	17	0.8	5.02	27.3
	28	152	270	17	0.8	2.48	35.6
	29	152	270	17	0.8	3.02	32.5
	30	152	270	17	0.8	2.99	32.8
	32	152	270	26	0.8	2.98	38.8
	33	152	270	26	0.8	4.03	33.6
	34	152	270	26	0.8	2.5	41.5
	35	152	270	26	0.8	2.53	44.6
	36	152	270	26	0.8	5.08	25.7
	37	152	270	26	0.8	5.05	27.9
	38	152	270	26	0.8	2.49	43.3
	39	152	270	26	0.8	2.49	39.4
	42	152	270	26	0.8	3.01	39.3
	43	152	270	26	0.8	3.96	32.6
Mphonde and Frantz (1984)	A0-3-3	152	298	22.6	3.36	3.6	64.6
	A0-3-3c	152	298	29.5	2.32	3.6	66.8
	A0-7-3a	152	298	40.9	3.36	3.6	82.16
	A0-7-3b	152	298	45.2	3.36	3.6	82.79
	A0-11-3a	152	298	81.4	3.36	3.6	89.69
	A0-11-3b	152	298	81.1	3.36	3.6	89.38
	A0-15-3a	152	298	88.4	3.36	3.6	93.45
	A0-15-3b	152	298	101.8	3.36	3.6	100
	A0-15-3c	152	298	99.8	3.36	3.6	97.84
	A0-3-2	152	298	22.4	3.36	2.5	77.77
	A0-7-2	152	298	49.1	3.36	2.5	117.9
	A0-11-2	152	298	86.2	3.36	2.5	111.3
Ahmad *et al*. (1986)	A8	127	208	60.8	1.77	3	48.92
	A1	127	203	60.8	3.93	4	57.83
	A2	127	203	60.8	3.93	3	68.95
	A3	127	203	60.8	3.93	2.7	68.95
	B1	127	202	67	5.04	4	51.21
	B2	127	202	67	5.04	3	68.95
	C1	127	184	64.3	6.64	4	54.28
	C2	127	184	64.3	6.64	3	75.63
	C3	127	184	64.3	6.64	2.7	68.95

Table 1: Countinue

Researches	ID	b	d	fc	ρ_w (%)	a/d	V_{test} kN
	C7	127	207	64.3	3.26	4	45.39
	C8	127	207	64.3	3.26	3	44.48
	C9	127	207	64.3	3.26	2.7	45.39
	B7	127	208	66.9	2.25	4	44.62
	B8	127	208	66.9	2.25	3	46.7
Elzanaty *et al.* (1986)	F11	177.8	273	20.6	1.2	4	44.81
	F12	177.8	273	20.6	2.5	4	54.48
	F8	177.8	273	39.9	1	4	45.97
	F13	177.8	273	39.9	1.2	4	46.35
	F14	177.8	273	39.9	2.5	4	64.93
	F1	177.8	273	65.5	1.2	4	58.69
	F2	177.8	273	65.5	2.5	4	67.21
	F9	177.8	273	79.2	1.6	4	63.67
	F10	177.8	273	65.5	3.3	4	78.53
	F15	177.8	273	79.2	2.5	4	68.3
	F6	177.8	273	63.4	2.5	6	61.9
Salandra and Ahmad (1989)	LR-2.59-NS	101.6	171.4	53.7	1.45	2.59	26.68
	LR-3.63-NS	101.6	171.4	52.1	1.45	3.63	21.79
	HR-3.63-NS	101.6	171.4	69.1	1.45	3.63	20.02
	HR-2.59-NS	101.6	171.4	66.8	1.45	2.59	29.8
Thorentfeldt and	B21	150	221	77.8	1.82	3	67.93
Drangsholt (1990)	B11	150	221	54	1.82	3	58.12
	B13	150	207	54	2.23	4	70.46
	B14	150	207	54	3.23	3	82.63
	B23	150	207	77.8	3.23	4	77.82
	B24	150	207	77.8	3.23	3	82.63
	B33	150	207	58	3.23	4	68.01
	B34	150	207	58	3.23	3	82.63
	B43	150	207	86.4	3.23	4	86.16
	B44	150	207	86.4	3.23	3	107.2
	B53	150	207	97.7	3.23	4	76.84
	B54	150	207	97.7	3.23	3	77.72
	B63	300	414	77.8	3.23	4	229.4
	B64	300	414	77.8	3.23	3	280.7
	B51	150	221	97.7	1.82	3	56.16
	B61	300	442	77.8	1.82	3	180.3
Jin-keun and Yon-Dong (1994)	CTL-1	170	270	53.7	1.87	3	70.68
	CTL-2	170	270	53.7	1.87	3	71.6
	P1.0-1	170	272	53.7	1.01	3	58.26
	P1.0-2	170	272	53.7	1.01	3	56.41
	P3.4-1	170	267	53.7	3.35	3	78.07
	P3.4-2	170	267	53.7	3.35	3	78.52
	P4.6-1	170	255	53.7	4.68	3	89.73
	P4.6-2	170	255	53.7	4.68	3	95.37
Jin-Keun and Yon-Dong (1994)	A4.5-1	170	270	53.7	1.87	4.5	66.55
	A4.5-2	170	270	53.7	1.87	4.5	63.8
	D142-1	170	142	53.7	1.87	3	41.03
	D142-2	170	142	53.7	1.87	3	39.34
	D550-1	300	550	53.7	1.87	3	226.1
	D550-2	300	550	53.7	1.87	3	214.5
	D915-1	300	915	53.7	1.87	3	299.2
	D915-2	300	915	53.7	1.87	3	332.1
Xie *et al.* (1994)	NNN-3	127	215.9	37.7	2.07	3	36.68
	NHN-3	127	215.9	98.9	2.07	3	45.72
Ahamad *et al.* (1994)	LNN-3	127	215.9	40.3	1.04	3	22.64
	LHN-3	127	215.9	89.1	2.07	3	43.39
Ahmad *et al.* (1995)	B7H	102	178	76.6	1.39	3.7	24.51
	B8H	102	178	79.3	1.39	3.7	19.79
Adebar and Collins (1996)	ST1	360	278	52.5	1.57	2.88	128
	ST2	360	278	52.5	1.57	2.88	119
	ST3	290	278	49.3	1.95	2.88	108
	ST8	290	278	46.2	1.95	2.88	81
	ST16	290	178	51.5	3.04	4.49	74.3
	ST23	290	278	58.9	1	2.88	90
Johnson and Ramiraz (1989) and	6	305	610	55.8	2.49	3.1	191.3
Yoon *et al.* (1996)	N1-S	375	655	36	2.8	3.23	249

Table 1: Countinue

Researches	ID	b	d	fc	ρ_w (%)	a/d	V_{test} kN
	M1-S	375	655	67	2.8	3.23	296
	H1-S	375	655	87	2.8	3.23	327
Islam *et al*. (1998)	M100-S0	150	203	83.3	3.22	3.94	65
	M100-S3	150	203	83.3	3.22	2.96	96.9
	M100-S4	150	203	83.3	3.22	3.94	80.7
	M80-S0	150	203	72.2	3.22	3.94	58
	M80-S4	150	203	72.2	3.22	3.94	72.1
	M60-S0	150	207	50.8	2.02	3.86	45.5
	M60-S4	150	207	50.8	2.02	3.86	51.9
	M40-S0	150	205	34.4	31.19	3.9	55
	M25-S0	150	207	26.6	2.02	3.86	47.5
	M25-S3	150	207	26.6	2.02	2.9	56.5
Kulkarni and Shah (1998)	BRJL20-S	102	152	41.9	1.37	5	19.52
	B3NO15-S	102	152	43	1.37	4	22.66
	B3NO30-S	102	152	45	1.37	3.5	24.24
Collins and Kuchma (1999)	B100	300	925	36	1	2.92	225
	B100-R	300	925	36	1	2.92	249
	B100L	300	925	39	1	2.92	223
	B100L-R	300	925	39	1	2.92	235
	B100B	300	925	39	1	2.92	204
	BN100	300	925	37	0.75	2.92	192
	BN50	300	450	37	0.81	3	131.7
	BN25	300	225	37	0.88	3	72.9
	BN12	300	110	37	0.9	3.07	40
	SE100A-45	295	920	50	1.03	2.5	200.8
	SE100A-45-R	295	920	50	1.03	2.5	235.7
	SE50A-45	169	459	53	1.03	2.72	68.6
	SE50A-45R	169	459	53	1.03	2.72	80.5
Collins and Kuchma (1999)	SE100A-83	295	920	86	1.03	2.5	184
	SE50A-83	169	459	91	1.03	2.72	73.1
	B100H	300	925	98	1	2.92	193
	B100HE	300	925	98	1	2.92	217
	BH100	300	925	99	0.75	2.92	193
	BH50	300	450	99	0.81	3	131.7
	BH25	300	225	99	0.88	3	84.8
	BRL100	300	925	94	0.5	2.92	163
Fonteboa (2002)	V10HC	202	306	40.2	2.88	3.27	88.86
	V10HCS	203	306	46.77	2.87	3.27	100.5
	V10HRS	200	305	39.65	2.93	3.28	90.64
	V10HRS	199	305	41.45	2.93	3.28	83.88
Cladera and Mari (2005)	H50/1	200	359	49.9	2.24	3.01	99.69
	H60/1	200	359	60.8	2.24	3.01	108.1
	H75/1	200	359	68.9	2.24	3.01	99.93
	H100/1	200	359	87	2.24	3.01	117.9

from 101.6-375 mm, the effective depth d of the beam section which ranges between 110 and 1090 mm and the shear span to depth ratio a/d which ranges from 2.48-7. Note that all the included experiments are on slender beams only where a/d exceeds 2. While the concrete compressive strength and the longitudinal steel ratio are in the range of 14.7-101.8 MPa and 0.5-6.64, respectively. It should be referred here that some of the design codes specify lower and upper limits for the compressive strength of concrete used in all structural elements. The ACI 318 limits the structurally used compressive strength to a lower limit of 17 MPa and an upper limit of 70 MPa (Table 1).

The relationship between the experimental failure shear strength and the beams properties are shown in Fig. 2 through Fig. 6. It is shown from the observation of Fig. 2 that the shear strength of the tested beams increases as the width of the section increase. This relation can be noticed clearly by the observation of the high positive slope of the linear fit. Similar relation is noticed for the effect of effective depth of the section on the shear strength of the tested beams as shown in Fig. 3. Similarly, the linear fit between Vtest and d shows a noticeable positive slope indicating the strong effect of d

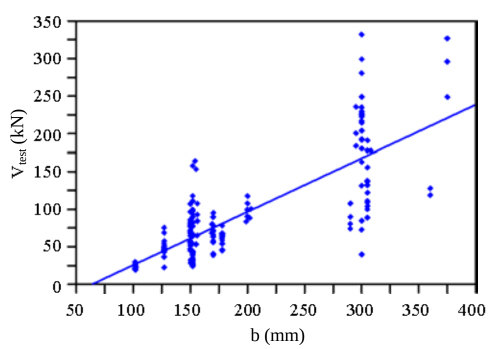

Fig. 2: V_{test}-section width relationship for the 175 beams

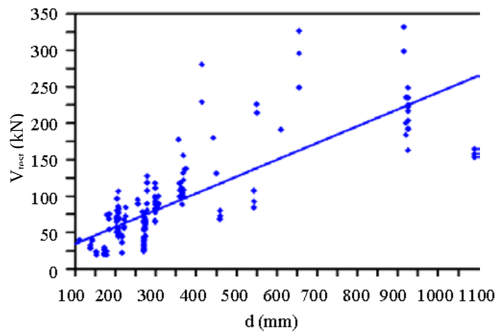

Fig. 3: V_{test}-effective depth relationship for the 175 beams

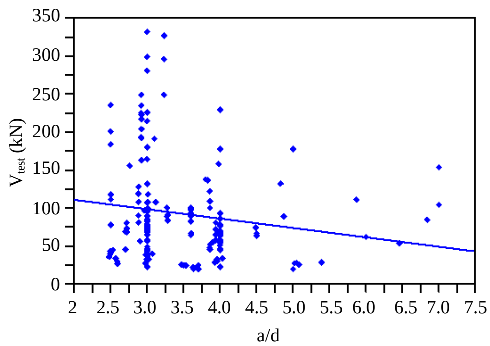

Fig. 4: V_{test}-a/d relationship for the 175 beams

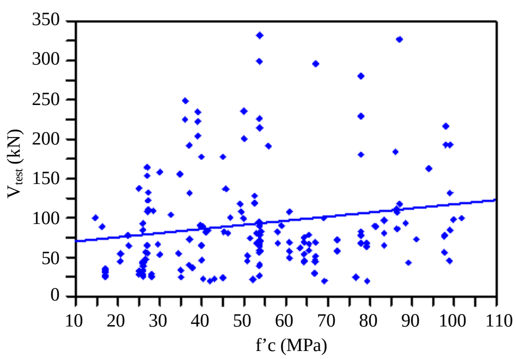

Fig. 5: V_{test}-compressive strength relationship

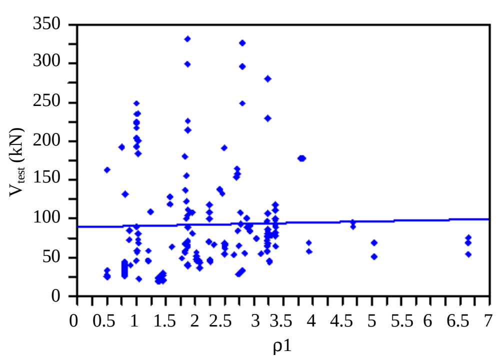

Fig. 6: V_{test}-steel ratio relationship for the 175 beams

on the shear strength of the tested beams. On the other hand, the opposite trend of the shear span to depth ration a/d as shown in Fig. 4. Figure 4 shows that the increase in a/d leads to a decrease in the shear strength of the tested specimens. However, the slope of the linear fit (negative slope) is not as high as for b or d and hence the effect of a/d is not as strong as of b or d. From the observation of Fig. 5 and 6, it can be noticed that the increase in compressive strength or longitudinal steel ratio ρ_w leads to the increase of the shear strength but of lower effect than the geometrical properties. Where the slope of the linear relationship between shear strength and geometrical properties is noticeable. While the slope of the linear fits shown in Fig. 5 and 6 are small and the linear fit tends to be horizontal with slope approaches to zero which reflects the less effect of these parameters compared to geometrical parameters. Although, it is noticed from the observation of Fig. 2-6 that each particular parameter cannot role the shear strength of the beams without being affected by the other parameters. This effect can be noticed from the sagging or the fluctuation in the shear strength-geometrical variables relationships with the increase of these variables.

MATERIALS AND METHODS

Using the stepwise multiple regression, five models were predicted. The predicted models are summarized in Table 2-6 show the predicted formula, coefficient of correlation R^2, squared R, standard error and coefficient of variation for each model.

As shown in Table 2, the first model is predicted using linear regression without interaction between the independent variables of the formula. The formula consists of six terms only which reflect the simplicity of the formula. However, the squared correlation coefficient R^2 is not very good which is only 0.885. Also, the standard error and the coefficient of variation of this model are the highest between the five predicted models which measures the higher dispersion of this model from the test results compared to the other models as shown in Fig. 7. The second model is linear model with the considering of the interaction between the five variables of the formula, thus the number of terms is higher compared to the first model. As shown in Table 3, the second model is composed of 7 terms which means that this model is more complex than the first one. However, comparing the R^2 of the two models, one can simply observe the higher degree of confidence of the second model. Where R^2 is 0.943 compared to 0.885 of the first

Table 2: Stepwise predicted formula: Model 1

Variables	Model 1: Linear regression
Formula	$V = b0-b1\times b+b2\times d+b3\times\sqrt{fc}+b4\times\rho l+b5\times a/d$
Coefficients	b0 = -98.60, b1 = 0.473, b2 = 0.162, b3 = 4.659, b4 = 14.15, b5 = -6.126

$R = 0.941$, $R^2 = 0.885$, SE = 22.787, COV = 24.672

Table 3: Stepwise predicted formula: Model 2

Variables	Model 2: Linear interaction regression
Formula	$V = b0+b1\times b\times d+b2\times b\times\rho l+b3\times\sqrt{fc}+b4\times\rho l\times a/d+b5\times d\times\rho l+b6\times d\times\sqrt{fc}$
Coefficients	b0 = -38.78, b1 = 0.0008, b2 = 0.0988, b3 = 8.482, b4 = -3.563, b5 = 0.036, b6 = -0.0114

$R = 0.971$, $R^2 = 0.943$, SE = 17.51, COV = 16.17

Table 4: Stepwise predicted formula: Model 3

Variables	Model 3: Simplified linear interaction regression
Formula	$V = b0+b1\times\rho l\times b\times d+b2\times a/d+b3\times b\times d+b4\times\sqrt{fc}\times a/d$
Coefficients	b0 = 38.03, b1= 2.75×10^{-4}, b2 = -15.15, b3 = 4.26×10^{-4}, b4 = 1.415

$R = 0.966$, $R^2 = 0.934$, SE = 18.68, COV = 17.25

Table 5: Stepwise predicted formula: Model 4

Variables	Model 4: Squared interaction regression
Formula	$b\times(a/d)^2+b7\times d^2\times\rho l+b8\times b\times\rho l\times\sqrt{fc}+b9\times b\times a/d+b10\times b\times d\times\sqrt{fc}$
Coefficients	b0 = 38.02, b1 = 0.000275,b2 = -15.15, b3 = 0.000426, b4 = 0.00383, b5 = -0.369, b6 = 0.00935, b7 = 6.022×10^{-5}, b8 = 0.0139, b9 = -0.104, b10 = 5.632×10^{-5}

$R = 0.983$, $R^2 = 0.967$, SE = 13.48, COV = 12.45

Table 6: Stepwise predicted formula: Model 5

Variables	Model 5: Full quadratic regression
Formula	$V = b0+b1\times b\times d+b2\times\rho l\times a/d+b3\times d\times\rho l+b4\times d^2+b5\times d\times\sqrt{fc}+b6\times b^2+b7\times b\times\sqrt{fc}+b8\times\rho l^2+b9\times\sqrt{fc}\times\rho l+b10\times fc$
Coefficients	b0 = -13.4, b1 = 0.00129, b2 = -2.644, b3 = 0.0603, b4 = -9.787×10^{-5}, b5 = -0.019, b6 = -0.000966, b7 = 0.0625, b8 = -2.757, b9 = 3.116, b10 = -0.549

$R = 0.979$, $R^2 = 0.959$, SE = 14.99, COV = 13.84

model. The better performance of the second model can be shown on the test-vs-predicted shear strength values as shown in Fig. 8 and can be measured using the error and variance measurements. The standard error and the coefficient of variation are 16.17 and 17.51, respectively. Figure 9 and 10 assures the lower variance of the second model predicted shear strength values from test values compared to the first model.

Table 7 shows comparison between the properties of the five predicted models. While Table 8 shows the $V_{test}/V_{pred.}$ and the mean, standard deviation and the coefficient of variation of V_{test}/V_{pred} of the five models. By the comparison of the first and second model data, the better behavior of the second model can obviously be noticed. Table 4 lists the third model and its statistical properties. The third model is a simplified linear with interaction mode. This simplification was taken out by introducing both the width b and the effective depth d of the beam section as one variable b×d. As a result this model became the simplest between the five models, where it is composed of five terms only. The dispersion of the predicted values of this model is much more better than the first model and is almost the same as the second model as shown in Fig. 11 and 12 and as listed in Table 7 and 8. The squared coefficient of correlation is 0.934 which is very close to that of the second model 0.943. From the comparison between the statistical

Table 7: Comparison between the five models

Model No.	No. of terms	R^2	COV
1	6	0.885	24.672
2	7	0.943	17.51
3	5	0.934	18.68
4	11	0.967	13.48
5	11	0.959	14.99

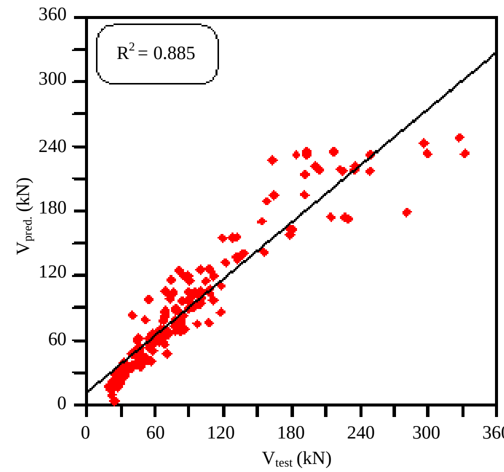

Fig. 7: Test-versus-predicted shear strength of Model 1

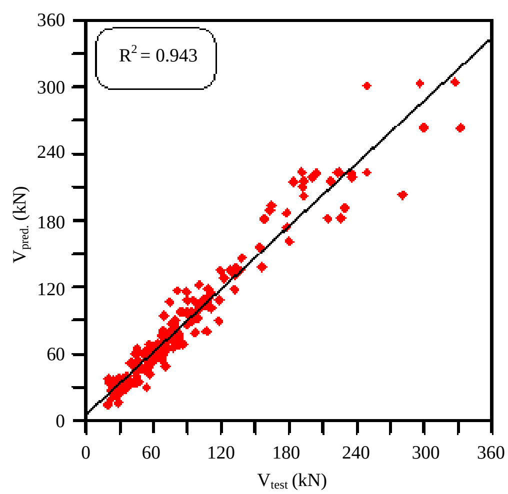

Fig. 8: Test-versus-predicted shear strength of Model 2

Table 8: Comparison between V_{test}/V_{Model} of the five models

Statistical parameters	V_{test}/V_{Model1}	V_{test}/V_{Model2}	V_{test}/V_{Model3}	V_{test}/V_{Model4}	V_{test}/V_{Model5}
Mean	1.030945	1.014431	0.989538	1.004963	1.020558
SD	0.226470	0.183557	0.175881	0.127957	0.167816
COV (%)	21.967200	18.094520	17.774030	12.732480	16.443560

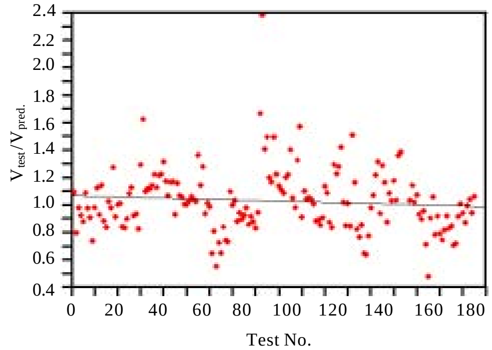

Fig. 9: $V_{test}/V_{pred.}$ of Model 1

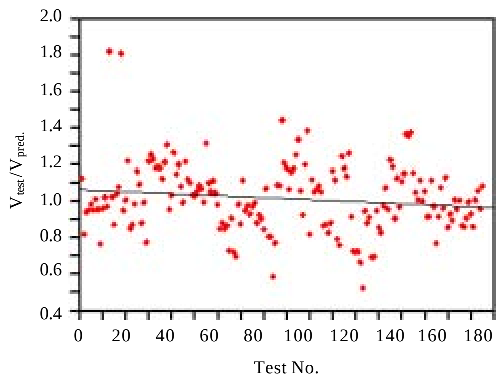

Fig. 10: $V_{test}/V_{pred.}$ of Model 2

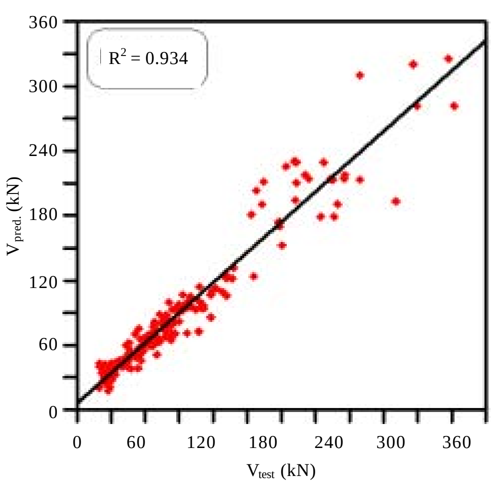

Fig. 11: Test-versus-predicted shear strength of Model 3

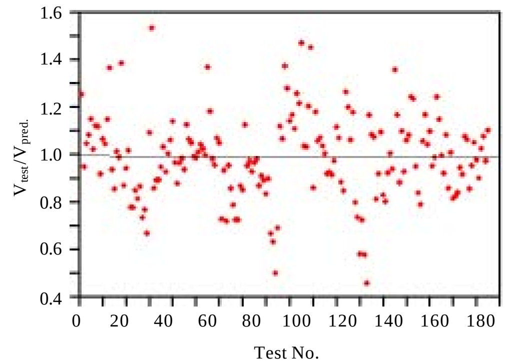

Fig. 12: $V_{test}/V_{pred.}$ of Model 3

measurements and taking into account the more simplicity of the third model, it can be drawn that the third model can be considered superior compared to the second one.

The fourth model is a squared with interaction one while the fifth model is a fully quadratic model. The fourth and fifth models and their statistical properties are shown in Table 5 and 6, respectively in addition to Table 7 and 8. From the statistical point of view, the fourth and fifth models are the best predicted formulas in this study. The R^2 of the fourth model is 0.967 which is the highest among the five models. While for the fifth model, the R^2 is about 0.96 which is better than those of the first three models. The variance of these two models are the lowest as shown from the SD and COV measurements listed in Table 7 and 8. Thus, the dispersion of the predicted values of these models are obviously less than of the first three models as shown in Fig. 13 through 16.

As shown in Fig. 15, the dispersion of the predicted values of the fourth model is very good. Where the $V_{test}/V_{pred.}$ values is almost ranges between 0.7 and 1.4. The standard deviation and the COV of these values are excellent which are 0.128 and 12.73. Thus, it is shown that from the statistical point of view, the fourth model is the best among the five models with highest R^2 and lowest SD and COV. But, on the other hand the simplicity of the predicted formula is a very important evaluation factor. The fourth and the fifth models are very complex and long formulas where each one contains 11 items which significantly alters the total evaluation of these models. Taking into account both the degree of confidence which represented by high R^2, low SD and low COV and the simplicity of the predicted formula, it can be concluded that the third model is more applicable formula with reasonable degree of confidence (R^2 = 0.934) than other models. Where the third model is composed of five items only which is much more simple and hence more applicable than the fourth and the fifth models in which the formula is composed of 11 items despite of the higher R^2 (0.967 and 0.959) of the fourth and fifth models, respectively.

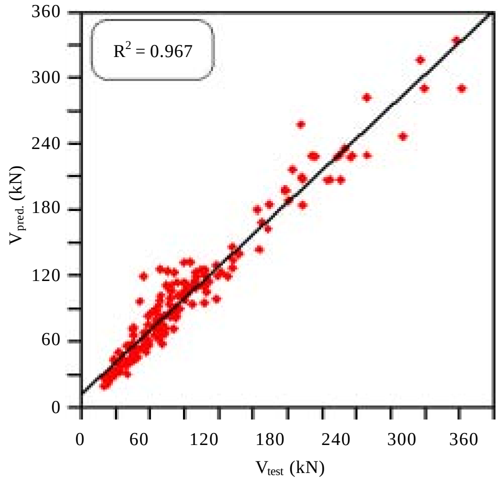

Fig. 13: Test-versus-predicted shear strength of Model 4

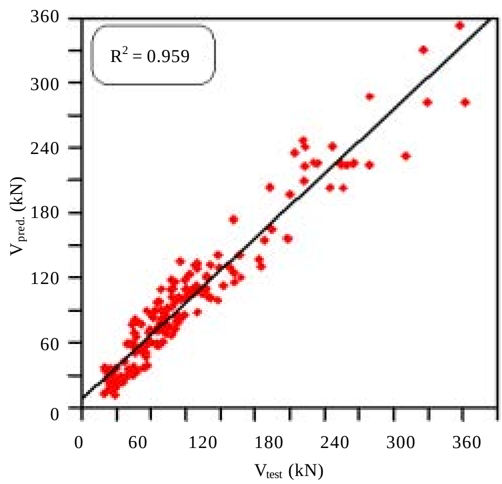

Fig. 14: Test-versus-predicted shear strength of Model 5

Evaluation of aci 318 shear provisions: The performance of the ACI 318 shear provisions for reinforced concrete beams without web reinforcement is evaluated in the section using two tests. The first is carried out using the test results directly from the literature review while the second test is performed using the selected model from the previous step (stepwise multiple regression).

By the comparison between Table 8 and 9, it can be obviously noticed that the ACI equations are of lower adequacy to predict the shear strength of reinforced beams without stirrups than the predicted models in this research. The highest SD and COV of the V_{test}/V_{pred} of the predicted models are those of the first model which are 0.226 and 21.96, respectively. While for the selected model Model 3, the SD and COV are 0.176 and 17.77 only. On the other hand the V_{test}/V_{pred} of the ACI 318 equations are of higher dispersion than any of the predicted models. The SD and COV are 0.414 and 32.51 for Eq. 11-3 and 0.341 and 27.04 for Eq. 11-5 as listed in Table 2. Figure 17-20 show the high dispersion of the ACI equations from the test shear strength values. It is shown in Fig. 17 and 18 that the ACI 318 Eq. 11-5 is of less dispersion than Eq. 11-3. Figure 19 and 20 assures this where it is shown the V_{test}/V_{pred} values ranges between about 0.4-2.6 for Eq. 11-3 while for Eq. 11-5, V_{test}/V_{pred} values ranges between about 0.5 to about 2.3. Similarly, the better performance of Eq. 11-5 can be observed simply by the comparison of SD and COV values of V_{test}/V_{pred} of these two equation as listed in Table 9. The reason of the superiority of Eq. 11-5 is due to the consideration of more effective parameters in this equation while in Eq. 11-3 lower number of effective parameters were considered to simplify the equation which in turns affected the adequacy of this equation. As we mentioned previously

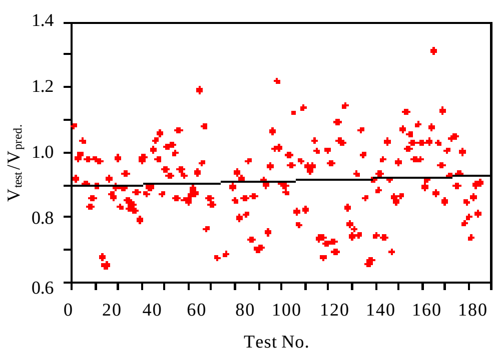

Fig. 15: $V_{test}/V_{pred.}$ of Model 4

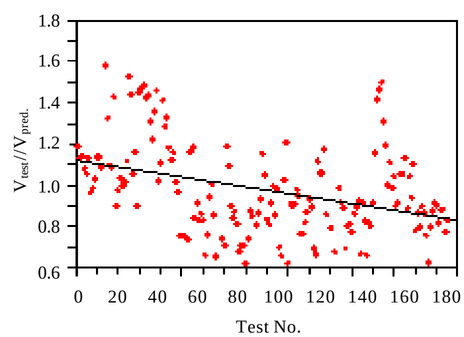

Fig. 16: $V_{test}/V_{pred.}$ of Model 5

Table 9: Comparison between V_{test}/V_{ACI} and V_{Model3}/V_{ACI} for ACI Eq. 11-3 and 11-5

Statistical parameters	$V_{test}/V_{ACI\,11\text{-}3}$	$V_{test}/V_{ACI\,11\text{-}5}$	$V_{Model\,3}/V_{ACI\,11\text{-}3}$	$V_{Model3}/V_{ACI11\text{-}5}$
Mean	1.273439	1.260708	1.303342	1.292079
SD	0.414062	0.340874	0.401676	0.330625
COV (%)	32.515290	27.038310	30.818940	25.588630

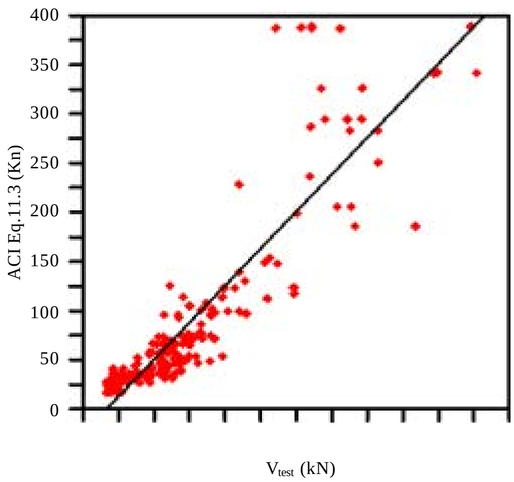

Fig. 17: Test vs. ACI Eq. 11.3 predicted strength

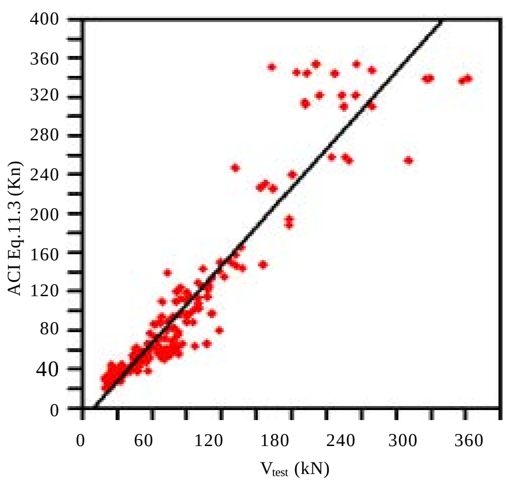

Fig. 18: Test vs. ACI Eq. 11.5 predicated shear strength

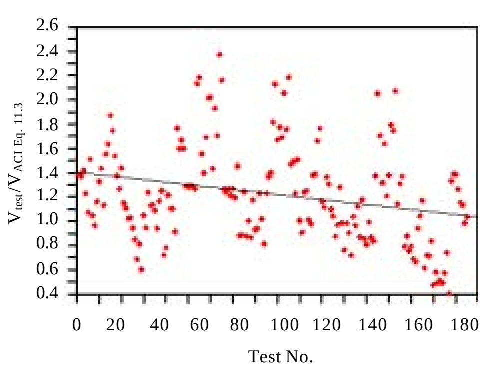

Fig. 19: $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-3

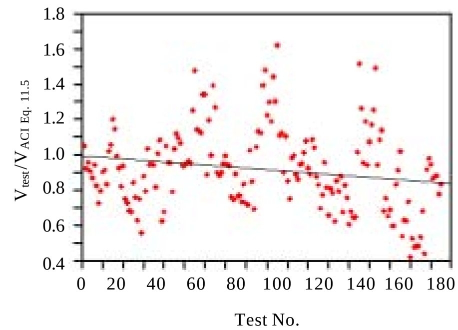

Fig. 20: $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-3

in this study, the second evaluation method to evaluate the adequacy of the current shear (without web reinforcement) provisions of ACI 318 is the using the proposed model in this research which is the third model. Figure 21 plots the V_{test}/V_{pred} of Eq. 11-3 against those of Model 3. As shown in the Fig. 21, the distribution of V_{test}/V_{pred} values of model 3 is very good where the mean value which is represented by the blue line is almost 1.0 which indicates the high degree of confidence on Model 3 as an evaluation test. This conclusion can be strengthened by the comparison of the SD and COV of V_{test}/V_{ACI} and $V_{Model\,3}/V_{ACI}$ values which is listed in Table 9. The SD is 0.414 for V_{test}/V_{ACI} while for $V_{Model\,3}/V_{ACI}$ the SD is 0.401 which is very close to that of V_{test}/V_{ACI}. Similarly, the COV of V_{test}/V_{ACI} and $V_{Model\,3}/V_{ACI}$ are 32.51and 30.82, respectively.

Similar behavior is noticed for the relationship between Vtest/VACI and VModel 3/VACI of ACI Eq. 11-5 as shown in Fig. 22 and listed in Tbale 9. It is shown in Fig. 21 and 22 that the Vtest/VACI is mostly higher than 1.0 reaching values of 2.4 or 2.5. This reflects that the ACI equations are mostly conservative formulas. However, it is shown that in some points the Vtest/VACI are less than one reaching values if about 0.4 which means that these equations are unsafe in these regions. To discuss this point more extensively and to find the parameters that affect the degree of safety of the ACI equations, the Vtest/VACI are plotted against the different parameters that influence the actual and predicted shear strength of beams without web reinforcement as shown in Fig. 23-32.

From the observation of Fig. 23 and 24, it can be shown that the mean of $V_{test}/V_{Model\,3}$ ranges from about 0.9 to about 1.1 as b increases the mean increase. This means that a moderately small difference from the test results exists. Figures indicates that Model 3 is a little un safe for b values <240 mm while it is safe for b values >240 mm. This degree of variance from 1.0 can be considered as ineffective on the degree of confidence of Model 3 to be used as a testing model. The scatter of V_{test}/V_{ACI} from the $V_{test}/V_{Model\ 3}$ is obvious for both Eq. 11-3 and 11-5 as shown in Fig. 23 and 24. It is shown that the red line which represents the V_{test}/V_{ACI} values is above the blue line which represents the $V_{test}/V_{Model\ 3}$ values for all values of b less than about 300 mm for both Eq. 11-3 and 11-5. Thus, it can be said that the ACI equations are conservative for b <270 mm. Oppositely, ACI equations can be considered un safe as b exceeds 360 mm where V_{test}/V_{ACI} values decrease continuously to values <0.9.

The relationship between $V_{test}/V_{Model\ 3}$ and the effective depth of the beams d is excellent, where as shown in Fig. 25 and 26 the $V_{test}/V_{Model\ 3}$ is almost 1.0 for the whole range of d. The relationship between V_{test}/V_{ACI} and d tends to follow the same behavior as that with b. Where the ACI equations are observed to be conservative starting from the lower values of d and up to a certain limit, then tend to be un safe beyond this limit. The ACI equation can be considered as conservative for d values <600 mm and unsafe as d exceeds 750 mm. In the region between 600 and 750 mm, the ACI equation can be considered as reasonable where no effective over estimation of lower estimation is observed in this region. For the effect of the beam width b on the ACI equation behavior, similar region can be noticed which extent between b equals to about 270 mm to b equals about 360 mm. These three regions (conservative, reasonable and unsafe regions) can be noticed by the observation of Fig. 23-26.

Figure 27 and 28 show the effect of the shear span length to depth ratio a/d on the V_{test}/V_{pred} of ACI equations. It is shown that for both Eq. 11-3 and 11-5, the behavior of V_{test}/V_{ACI} which is represented by the red line seems to be steady with slope nearly equal to zero (the slope is 0.007 for Eq. 11-3 and is 0.031 for Eq. 11-5). The value of V_{test}/V_{ACI} for Eq. 11-3 ranges from about 1.3 to about 1.25 as a/d increases while for Eq. 11-5, V_{test}/V_{ACI} ranges between 1.2 and 1.4 as a/d increases. Thus, it can be concluded that the change in a/d values has no noticeable effect on the ACI prediction of the shear strengths. Hence, the ACI equations can be considered as conservative formulas when plotted against a/d. Similar behavior is noticed for the fc-V_{test}/V_{ACI} relationship. This steady behavior refers to the less important effect of this parameter

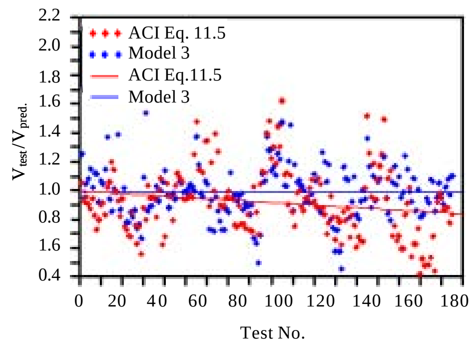

Fig. 22: $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-5 and Model 3

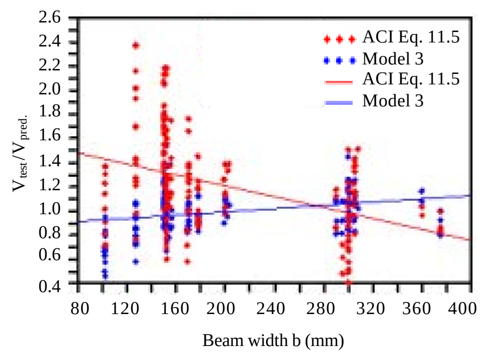

Fig. 23: The effect of (b) on $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-5 Model 3

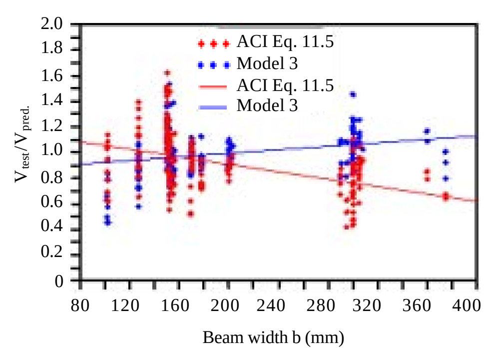

Fig. 24: The effect of (b) on $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-3 Model 3

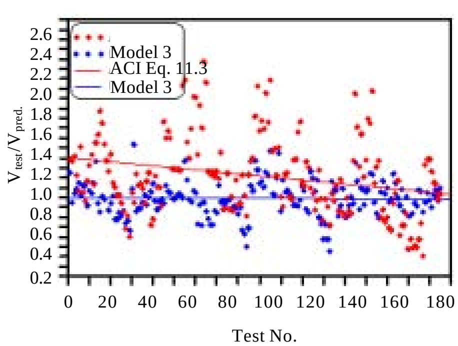

Fig. 21: $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-3 and Model 3

Model 3

on the equation. Referring to Fig. 2-5, it is shown that the fc and the a/d have not the same effect on the shear strength of the tested beams as that of the b and d.

The effect of the longitudinal steel ratio ρ_w on the $V_{test}/V_{pred.}$ of ACI 318 equations and Model 3 is shown in Fig. 31 and 32 for Eq. 11-3 and 11-5, respectively. Figures shows the very good behavior of Model 3 where the mean of $V_{test}/V_{pred.}$ which is represented by the blue line is almost 1.0 within the full studied range of ρ_w. On the other hand, the V_{test}/V_{ACI} of the both equation shows high scatted from the V_{test}/V_{pred} of Model 3. Figures show that the ACI Eq. 11-3 is conservative for ρ_w values exceeding 1.0 which is less than the minimum steel ratio prescribed by the ACI 318 provisions. As ρ_w

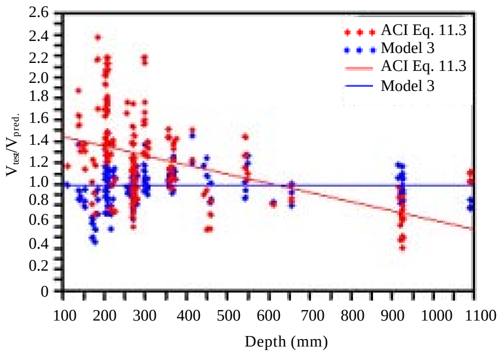

Fig. 25: The effect of (d) on $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-3 and Model 3

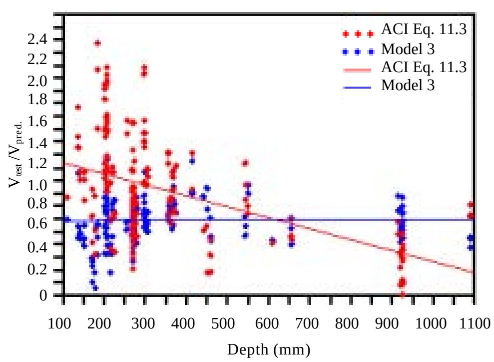

Fig. 26: The effect of (d) on $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-5 and Model 3

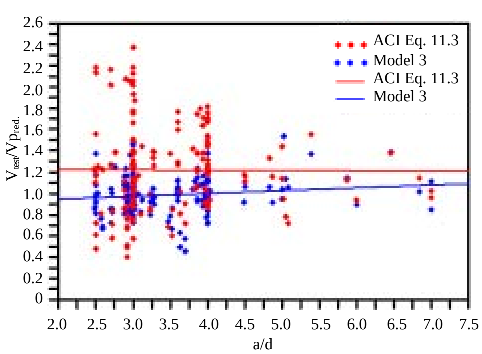

Fig. 27: The effect of (a/d) on $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-3 and Model 3

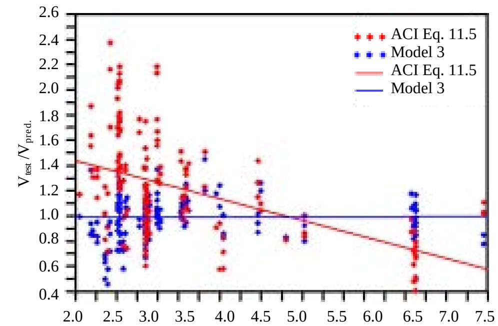

Fig. 28: The effect of (a/d) on $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-5 and Model 3

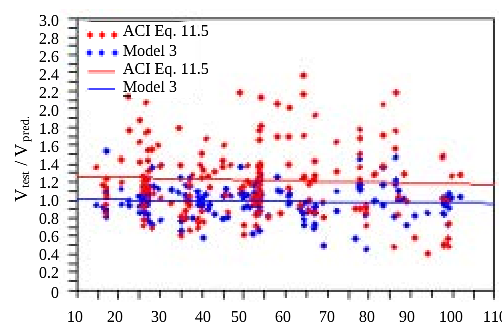

Fig. 29: The effect of 'fc' on $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-3 and Model 3

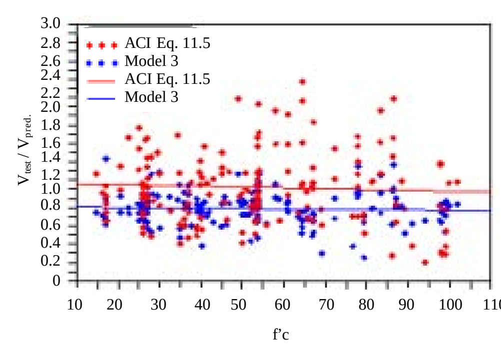

Fig. 30: The effect of 'fc' on $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-5 and Model 3

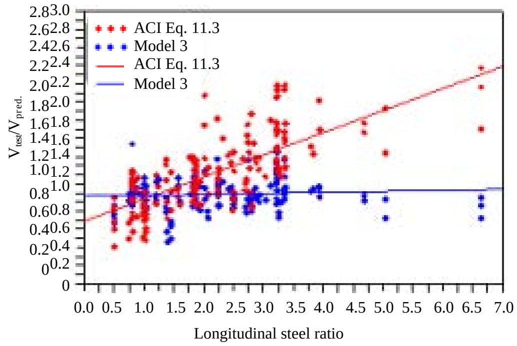

Fig. 31: The effect of 'ρ_w' on $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-3 and Model 3

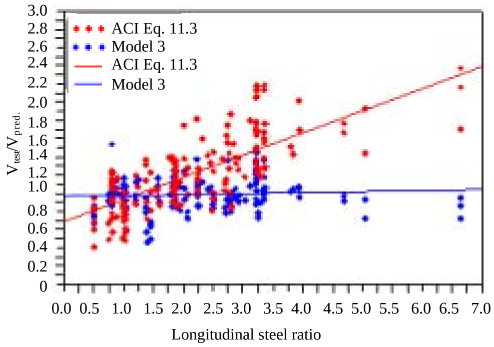

Fig. 32: The effect of 'ρ_w' on $V_{test}/V_{pred.}$ of ACI 318 Eq. 11-5 and Model 3.7

increase, the Eq. 11-3 becomes more conservative. This is an expected result because of the absence of the steel ratio P_w in Eq. 11-3 where Eq. 11-3 is a simplified formula takes into account only the geometrical properties of the section and the strength of concrete. Similar behavior is noticed for Eq. 11-5 but with lower degree of conservation.

CONCLUSION

Within the limits of the studied range of parameters and depending on the studied range of test data, the following conclusions can be drawn. The width and the effective depth of the beam sections are of higher effect on the shear strength of concrete beams without web reinforcement than the shear span to depth ratio, compressive strength of concrete and longitudinal steel ratio. Using a literature test data base of 175 concrete beams, five stepwise multiple regression models are predicted. The coefficient of correlation R for all models exceeds 0.94. Comparing the correlation coefficients, SD and COV and taking into account the degree of simplicity of the five models, a simplified linear with interaction model is chosen to evaluate the ACI shear provisions. The R^2 of this model is 0.934 and the COV is 18.68. The predicted model is found to be obviously better than the current ACI equations to predict the shear strength of beams without web reinforcement. The mean, SD and COV of the $V_{test}/V_{pred.}$ are 1.27, 0.41 and 32.51 for ACI Eq. 11-3 and 1.26, 0.34 and 27.04 for Eq. 11-5. While for the predicted model, the mean, SD and COV of the $V_{test}/V_{pred.}$ are 0.99, 0.17 and 17.77, respectively. The behavior of the $V_{test}/V_{pred.}$ of the predicted model with the studied parameters is found to be very good. Where the mean of $V_{test}/V_{pred.}$ is almost still very close to 1.0 with the full range of the studied variables. Comparing the ACI Eq. 11-3 and 11-5 with the predicted testing model, The ACI equations are conservative for all values of b less than about 270 mm and un safe for b greater than about 360 mm. Similarly, the equations can be considered as conservative for d <600 mm and un save when d exceeds 750 mm. The ACI equations are noticed to be conservative for the studied range of compressive strength and shear span to depth ratio. Also, the ACI equations are conservative for all longitudinal steel ratios exceeds the minimum prescribed by ACI provisions. The gap between the predicted testing model and the ACI equations increases as ρ_w increase.

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