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SHEAR STRENGTH OF SQUAT REINFORCED CONCRETE WALLS UNDER CYCLIC LOADING

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ABSTRACT: Squat reinforced concrete (RC) walls are essential structural components in nuclear power facilities (NPP) and in many civil structures. An adequate prediction of the shear capacity of these elements is important for the seismic design and performance assessment of NPP and structures whose primary lateral force resisting system is comprised by squat walls. These walls have aspect ratios less than or equal to 2. Due to their geometry, squat shear walls tend to have shear-dominated behavior while exhibiting strong coupling between flexural and shear responses. This paper presents an evaluation of current expressions for the prediction of peak shear strength of squat RC walls available in US design codes and in the literature. A database is assembled with the results of moderate to large-scale experimental tests of rectangular cross-section walls with shear-dominated failures and subjected to cyclic loads found in the literature. Key parameters influencing the peak shear strength are identified and improved shear strength predictive equations are developed by calibration against the available data. Multiple-linear regression analysis is used to develop the predictive equations for the shear strength. It is found that the peak shear strength of such walls has not been adequately addressed by current US code equations (e.g. ASCE 43-05 and ACI 349-06 / ACI 318-14) since there is significant scatter on the predictions. The improved expression presented herein is intended to be used in the design and assessment of structures with RC squat walls. This paper discusses the advantages and disadvantages of the developed equation and provides recommendations for future research in this topic.

1. Introduction

Reinforced concrete (RC) shear walls are commonly used in building systems and other structures such as nuclear facilities as the primary lateral-force-resisting system. Shear walls are mainly categorized as two different types: tall (or slender) and squat (low rise, short) based on the aspect ratio (h_w/l_w) or the shear span (M/Vl_w). Walls with an aspect ratio less than or equal to 2 are considered as squat walls while walls with a higher aspect ratio are considered as slender walls. The seismic behavior of squat walls is shear-dominated and is characterized by very high lateral stiffness and strength, but also by relatively limited ductility and energy dissipation capacity. Thus, typical squat walls are prone to undesirable (non-ductile) shear failures characterized by sudden loss of strength and stiffness under lateral cyclic loading. The main shear failure mechanisms associated with squat walls are diagonal tension, diagonal compression (web crushing), sliding shear or a combination of the aforementioned (Paulay and Priestley, 1992; Gulec and Whittaker, 2009). In contrast, walls with aspect ratios higher than 3.0 would generally be flexure-controlled while walls with moderate aspect ratios between 1.5 and 3.0 (often called medium-rise walls) typically show a behavior influenced by both shear and flexure (ASCE 41-06).

Many equations are found in current US design codes (e.g., ACI 318, ACI 349, ASCE 43-05) and literature (e.g., Barda et al., 1977; Wood et al., 1990) for the prediction of the peak shear strength of reinforced concrete walls. However, comparison against experimental results show that these equations yield significantly scattered strength predictions for squat walls. The possible reasons for the scattered predictions derive from the limited data on which they are based. For example, ACI 318-14 equations are mostly based on data from moderate aspect ratio walls, and were modified by either imposing limits of reinforcement and/or incorporating a factor to include the strength characteristics of low aspect ratio walls (Cardenas et al., 1973). ASCE 43 equation is based on Barda's experiments (Barda et al., 1977) and was modified to provide a lower-bound solution when compared with some experiments available at that time. It is important to note that the US Code equations have remained basically unchanged for over 30 years while significant newer experimental data is available. Significant research (e.g. Carrillo et al, 2013; Gulec and Whittaker 2009; Massone, 2010; etc.) have been performed during the past years to address this situation, however the development of better equations to assess the peak shear strength of squat RC walls is still necessary, since the lateral strength and performance of these walls depends mostly on its shear strength. This paper presents an evaluation of current expressions for the prediction of peak shear strength of squat reinforced concrete walls available in US design codes and in the literature. A new equation to predict the peak shear strength of squat walls obtained from multivariable regression analyses using an assembled experimental database is presented to improve current peak shear estimates.

2. Assembled Experimental Database

A database was assembled from experimental data of squat reinforced concrete walls tests found in literature (Table 1). The database considered only quasi-static cyclic, dynamic and hybrid-simulated dynamic loading experimental tests in order to minimize bias due to loading type in the shear strength estimate. It has been found that cyclic tests yield lower shear strength than monotonically loaded similar specimens. Moderate to large scale tests with thicknesses larger than 75 mm were used in order to reduce bias due to small scale and material properties. Also only walls with conventional normalweight concrete were included to minimize the bias due to reduced strength and stiffness of lightweight concrete or improved tensile stress-strain behavior of fiber-modified concrete. Walls with aspect ratios lower than 1.5 were selected in order to further minimize the possibilities of including flexure-controlled and mixed failure modes. Walls were considered to be shear-controlled by comparing the shear load associated with flexural failure ($V_{flexure}$) with the peak strength (V_{peak}) measured from test. A ratio of $V_{flexure}$ /V_{peak} higher than one suggests that the wall is expected to have a shear-controlled failure. Selected wall specimens were tested in a cantilever setup so that aspect ratio (h_w/l_w) is generally similar to M/Vl_w. Note that walls tested with restricted rotation at the top (simulating fixed-fixed boundary condition) will have an h_w/l_w of nearly twice M/Vl_w, representing boundary conditions found in wall-piers which are connected to very stiff elements at both ends. To eliminate bias due to boundary conditions only cantilever (fixed-free) tests were selected. Peak shear strength was taken as the average peaks from the first and third quadrants. Terzioglu tests data was obtained from Opazo (2012) and Gutierrez (2012). Walls with orthogonal (vertical and horizontal) reinforcement only were selected as it is conventionally found in US design and construction practice.

Table 1 presents the main properties of the wall specimens incorporated in the database, which are also used as parameters in the equations throughout the text. The parameters are the following: t_w is the thickness, h_w is the height, l_w is the length, and A_w is the cross-sectional area of the wall. In addition, d is the distance from the extreme compression fiber to the resultant tension force, ρ_{be} is the vertical boundary element reinforcement ratio (calculated by dividing the area of the reinforcement in the boundary element to the area of the cross section), ρ_v is the vertical web reinforcement ratio, ρ_h is the horizontal web reinforcement ratio, f'_c is the peak compressive stress of concrete, f_{ybe} is the yield strength of the vertical boundary element reinforcement, f_{yv} is the yield strength of the vertical web reinforcement, f_{yh} is the yield strength of the horizontal web reinforcement, and P is the applied axial force.

Table 1 – Assembled Squat Walls Database.

Researcher	ID	l_w (mm)	t_w (mm)	h_w/l_w	ρ_{be} (%)	ρ_v (%)	ρ_h (%)	f'_c (MPa)	fy_{be} (MPa)	fy_v (MPa)	fy_h (MPa)	P (kN)	V_{peak} (kN)
Carrillo	MCN50C	2400	100	1.00	0.66	0.14	0.14	17.5	433	447	447	60	354
Carrillo	MCN100C	2400	100	1.00	0.95	0.29	0.29	17.5	430	447	447	60	454
Carrillo	MCS50C	2400	100	1.00	0.66	0.14	0.14	22	433	447	447	60	374
Carrillo	MCS100C	2400	100	1.00	0.95	0.29	0.29	22	430	447	447	60	454
Carrillo	MRN100C	5400	100	0.44	0.32	0.29	0.29	16.2	430	447	447	135	766
Carrillo	MRN50C	5400	100	0.44	0.22	0.14	0.14	16.2	433	447	447	135	670
Carrillo	MRN50mC	5400	100	0.44	0.22	0.12	0.12	20	433	605	605	135	777
Carrillo	MCN50mC	2400	100	1.00	0.71	0.12	0.12	20	430	605	605	60	329
Carrillo	MCN50C-2	2400	100	1.00	0.71	0.14	0.14	20	430	447	447	60	329
Carrillo	MCS50C-2	2400	100	1.00	0.71	0.14	0.14	27.1	430	447	447	60	321
Carrillo	MCNB50mC	2400	100	1.00	0.71	0.12	0.12	8.9	430	605	605	60	238
Carrillo	MRNB50mC	5400	100	0.44	0.22	0.12	0.12	8.9	433	605	605	135	612
Carrillo	MCN50mD	1920	80	1.00	0.77	0.11	0.11	24.7	411	630	630	38.4	234
Carrillo	MCN100D	1920	80	1.00	1.03	0.28	0.28	24.7	411	435	435	38.4	274
Terzioglu	SW-T2-S1-1	1500	120	0.50	0.45	0.67	0.67	19.3	437	473	473	0	793
Terzioglu	SW-T1-S1-2	1500	120	0.50	0.45	0.34	0.34	23.7	437	473	473	0	633
Terzioglu	SW-T2-S2-3	1500	120	0.50	0.45	0.67	0.67	25.8	437	473	473	0	666
Terzioglu	SW-T2-S3-4	1500	120	0.50	0.45	0.67	0.67	29	525	572	572	0	810
Terzioglu	SW-T3-S1-5	1500	120	0.50	0.06	0.67	0.67	32.1	572	572	572	0	385
Terzioglu	SW-T4-S1-6	1500	120	0.33	0.34	0.67	0.67	34.8	509	572	572	0	877
Terzioglu	SW-T5-S1-7	1500	120	1.00	0.84	0.34	0.67	35	536	572	572	0	709
Terzioglu	SW-T6-S1-8	1500	120	1.00	0.84	0.67	0.67	22.6	536	572	572	0	738
Terzioglu	SW-T1-S2-9	1500	120	0.50	0.45	0.34	0.34	24	525	572	572	0	565
Terzioglu	T1-N5-S1-10	1500	120	0.50	0.45	0.34	0.34	26.3	525	572	572	240	791
Terzioglu	T1-N10-S1-11	1500	120	0.50	0.45	0.34	0.34	27	525	572	572	480	796
NEES-UB	SW1	3048	203	0.94	0.00	0.71	0.71	24.8	0	462	462	0	1126
NEES-UB	SW2	3048	203	0.54	0.00	1.01	1.01	48.3	0	434	434	0	2372
NEES-UB	SW3	3048	203	0.54	0.00	0.71	0.71	53.8	0	434	434	0	1914
NEES-UB	SW4	3048	203	0.54	0.00	0.34	0.34	29	0	462	462	0	997
NEES-UB	SW6	3048	203	0.33	0.00	0.71	0.71	26.2	0	462	462	0	2207
NEES-UB	SW7	3048	203	0.33	0.00	0.34	0.34	26.2	0	462	462	0	1337
NEES-UB	SW8	3048	203	0.54	0.00	1.50	1.50	24.1	0	462	462	0	2632
NEES-UB	SW9	3048	203	0.54	0.00	1.50	0.71	29.7	0	462	462	0	2824
NEES-UB	SW11	3048	203	0.54	0.19	0.67	0.71	34.5	462	462	462	0	1871
Whyte	Wall 1	3048	203	0.54	0.00	0.71	0.71	35.5	0	464	464	0	1618
Whyte	Wall 2	3048	203	0.54	0.00	0.71	0.71	37.3	0	464	464	0	1705
Salonikios	MSW1	1200	100	1.50	0.34	0.57	0.57	26.1	585	610	610	0	196
Salonikios	MSW3	1200	100	1.50	0.25	0.28	0.28	24.1	585	610	610	202	173
Salonikios	MSW6	1200	100	1.50	0.34	0.57	0.57	27.5	585	610	610	0	187
Salonikios	LSW1	1200	100	1.00	0.34	0.57	0.57	22.2	585	610	610	0	262
Salonikios	LSW2	1200	100	1.00	0.25	0.28	0.28	21.6	585	610	610	0	185
Salonikios	LSW3	1200	100	1.00	0.25	0.28	0.28	23.9	585	610	610	201	252
Wiradinata	Wall 1	2000	100	0.50	0.20	0.71	0.21	25.0	435	435	425	0	531
Wiradinata	Wall 2	2000	100	0.25	0.20	0.71	0.21	22.0	435	435	425	0	685
Wasiewicz	Wall 3	2000	100	0.25	0.20	0.71	0.43	35	480	480	248	0	855
Pilette	Wall4	2000	100	0.50	0.20	0.71	0.80	33	480	480	480	0	401
Pilette	Wall 5	2000	100	0.50	0.20	1.15	1.15	27	480	480	480	0	545
Wasiewicz	Wall 6	2000	100	0.50	0.20	0.71	0.80	35	480	480	480	0	528
Mohammadi	Wall 7	2000	100	0.75	0.20	0.71	0.80	45.0	450	450	450	0	375
Mohammadi	Wall 8	1500	100	1.00	0.27	0.70	0.80	45.0	450	450	450	0	225
Synge	Wall 1	3000	100	0.50	0.15	0.81	1.68	27.2	300	300	380	0	775
Cardenas	SW-13	1905	76	1.00	0.00	2.93	0.98	43.4	0	448	455	0	632
Greifenhagen	M3	900	80	0.68	0.00	0.32	0.26	20.1	0	504	745	136	176
Greifenhagen	M4	900	80	0.68	0.00	0.32	0.26	24.4	0	504	745	76	135

3. Current Expressions for Peak Shear Strength

The following equations are similar to the presented on the corresponding references but the names and subscripts of the parameters have been modified for the purpose of comparison and uniformity between equations and database. Also, the equations are presented in terms of nominal loads and nominal strength.

3.1. ACI 318-14

It should be noted that ACI 349 (Code Requirements for Nuclear Safety-Related Concrete Structures) and ACI 318 use essentially the same equations for strength prediction with the only difference that the lightweight concrete modification factor (λ) is not included since the use of lightweight concrete is not permitted in nuclear facilities.

Chapter 11 of ACI 318-14 presents the following set of equations for the calculation of wall shear strength:

$$V_n = V_c + V_s \le 0.83\sqrt{f_c^{'}}\, t_w d \tag{1}$$

where d shall be taken as $0.8l_w$ or determined by strain compatitbility analysis, V_c is taken as the lesser of the values obtained from Eq. (2) and (3) and V_s is calculated using Eq. (4).

$$V_c = 0.27\lambda\sqrt{f_c^{'}}\, t_w d + \frac{Pd}{4l_w} \tag{2}$$

$$V_c = \left[0.05\lambda\sqrt{f_c^{'}} + \frac{l_w\left(0.1\lambda\sqrt{f_c^{'}} + 0.2\dfrac{P}{l_w t_w}\right)}{\dfrac{M}{V} - \dfrac{l_w}{2}} \right] t_w d \tag{3}$$

$$V_s = \frac{A_v f_y d}{s} \tag{4}$$

Note that Eq. (3) shall not apply when ($M/V - l_w/2$) is negative.

Chapter of 18 ACI 318-14 presents the following equation for the calculation of wall shear strength:

$$V_n = A_w\left(\alpha_c \lambda\sqrt{f_c^{'}} + \rho_h f_y\right) \le 0.83 A_w \sqrt{f_c^{'}} \tag{5}$$

α_c = coefficient defining the relative contribution of concrete strength to nominal wall shear strength. Varies linearly from 0.25 for h_w/l_w =1.5 to 0.17 for h_w/l_w = 2.

3.2. ASCE 43-05

ASCE 43 equation is based on the proposed by Barda et al. (1977) and was modified to account for both horizontal and vertical steel and provide a lower-bound solution when compared with some experiments available at that time. The shear strength is to be calculated as follows:

$$V_n = v_n d t_w \tag{6}$$

$$v_n = 0.69\sqrt{f_c^{'}} - 0.28\sqrt{f_c^{'}}\left(\frac{h_w}{l_w} - 0.5\right) + \frac{P}{4l_w t_w} + \rho_{se} f_y \le 1.67\sqrt{f_c^{'}} \tag{7}$$

$$\rho_{se} = A\rho_v + B\rho_h \tag{8}$$

where d shall be taken as $0.6l_w$ or determined by strain compatibility analysis.

Constants A and B constants are calculated as follows depending on the aspect ratio h_w/l_w:

$h_w/l_w \leq 0.5$	A = 1	B = 0
$0.5 < h_w/l_w < 1.5$	A = - h_w/l_w + 1.5	B = h_w/l_w – 0.5
$h_w/l_w \geq 1.5$	A = 0	B = 1

3.3. Barda et al. (1977)

Barda et al. (1977) proposed the Equation (9) based on monotonic and cyclic tests of 8 squat wall specimens with highly reinforced and well confined flanges.

$$V_n = \left(0.67\sqrt{f_c^{'}} - 0.21\sqrt{f_c^{'}}\frac{h_w}{l_w} + \frac{P}{4l_w t_w} + \rho_v f_y \right) t_w d \qquad (9)$$

For the calculations presented herein, d was assumed as $0.6l_w$.

3.4. Wood (1990)

Wood (1990) proposed the following semi-empirical expression to estimate the shear strength of squat walls:

$$0.5\sqrt{f_c^{'}} A_w \leq V_n = \frac{A_{vf}\, f_y}{4} \leq 0.83 A_w \sqrt{f_c^{'}} \qquad (10)$$

The equation was derived by using a shear-friction analogy and calibrating against experimental data from squat wall tests where, A_{vf} is the total vertical reinforcement area in the wall.

4. Statistical Approach and Methods

In order to eliminate bias due to load amplification factors and resistance reduction factors between codes the equations are modified to use nominal loads and nominal strengths. Also, the calculated peak and nominal shear strengths (V_{peak} and V_n) were then normalized with the gross area of the section (A_w) to eliminate the test scale differences between different experimental programs and all the parameters were worked in terms of stress instead of forces. The normalization of the shear force with A_w also allows for a fair comparison between the available equations and the developed equation since the shear stress is calculated based on different effective depth (*d)* definitions in the available expressions.

Multivariable linear regression analyses were performed to develop an empirical equation that better fits the included database. The goal was to lower the standard deviation and coefficient of variation of the predicted-to-measured shear strength ratio. The intent was also to obtain a mean predicted-to-measured strength ratio near to 1.0 and to reduce the percentage of over-predictions. A functional equation form was selected based on Barda’s proposed expressions and based on common ACI seismic design expressions and extended to account for the vertical, horizontal and boundary element reinforcement ratios on separate terms. The following equation includes the parameters that were found to produce a better correlation with the database.

$$V_n = \left(0.75 + 0.57\sqrt{f_c^{'}} - 0.50\sqrt{f_c^{'}}\frac{h_w}{l_w} + 0.61\frac{P}{A_g} + 0.36\rho_v f_{yv} + 0.12\rho_h f_{yh} + 0.5\rho_{be} f_{ybe} \right) A_w \qquad (11)$$

The aspect ratio (h_w/l_w) was found to produce better correlation with the included database (with cantilever test setup) than the shear span (M/Vl_w). The aspect ratio is a more practical parameter for design purposes, since it does not depend on structural analysis to calculate values for moment and shear.

4.1. Results and Discussion

In order to evaluate the suitability of the proposed equation, it is compared against several widely-used equations in terms of the predicted-to-measured strength ratio. Fig. 1 shows a graphic representation of the correlation of the calculated shear stress capacity vs the peak shear stress measured from tests for each of the equations discussed on section 3.

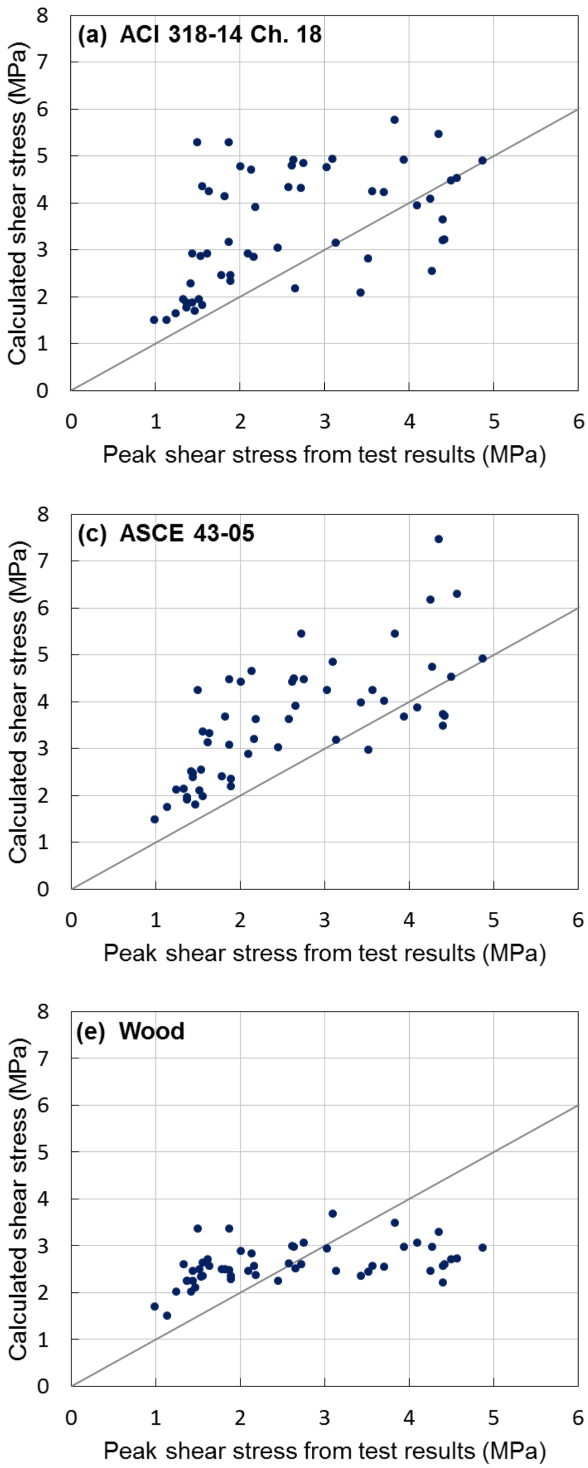

Fig. 1 – Correlation Between Calculated Nominal Shear Stress and Measured Peak Shear Stress Using Various Available Equations: (a) to (e); and the Equation Proposed in this Study (f)

The diagonal line represents a ratio of predicted-to-measured strength of 1.0 (exact prediction). Any point above the diagonal line represents an over-prediction of strength and vice-versa. The farther the point from the diagonal represents a larger error on the estimate. It can be noted from Fig. 1 that the equation proposed in this study (Eq. 11) reduces the scatter of the results significantly over the rest of the evaluated equations.

In order to numerically compare the observed behavior of the predictions with each equation from Fig. 1, Table 2 presents a summary of common central tendency and dispersion measures of the predicted-to-measured strength ratio, along with the percent of over-predictions. Mean and median values larger than 1.0 suggest that the equation tends to overestimate the strength. For example, ASCE 43-05 equation overestimates the strength, on average, by 49%. On the other hand, the standard deviation and coefficient of variation provide information on the dispersion (scatter) of the predicted-to-measured strength ratios.

Table 2 – Summary of statistics of the ratio of predicted shear strength to the measured peak shear strength for walls included in the database.

	ACI Ch. 18	ACI Ch. 11	ASCE 43-05	Barda	Wood	This study
Mean	1.47	1.20	1.49	1.35	1.17	1.03
Median	1.32	1.15	1.44	1.30	1.18	0.98
Minimum	0.59	0.50	0.79	0.70	0.50	0.60
Maximum	3.53	2.72	2.84	2.51	2.24	1.62
Std. Dev.	0.58	0.45	0.42	0.40	0.42	0.24
COV	0.40	0.37	0.28	0.30	0.36	0.23
% Over-predictions	79.6	64.8	88.9	79.6	61.1	46.3

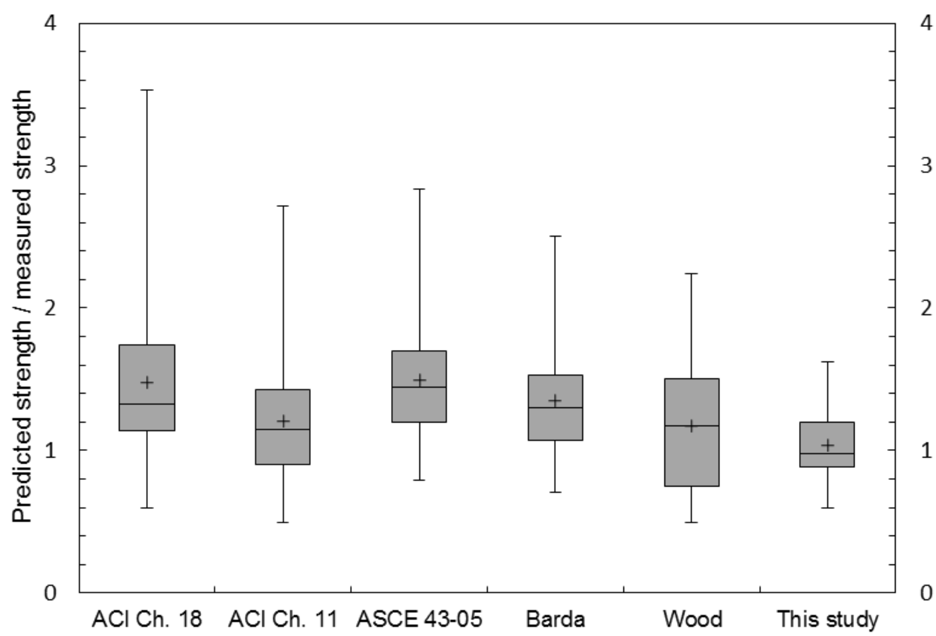

Fig. 2 – Distribution of the Ratio of Predicted to Measured Peak Strength

Fig. 2 shows a typical box and whisker plot comparing the distribution of the predicted-to-measured strength for each presented equation model. The graph shows the lower quartile (25th percentile), median, upper quartile (75th percentile), the extreme values (ends of whiskers) and the mean value marked with "+" symbol. It can be observed that the proposed equation produces a significantly improved estimate of the shear strength since the mean and average values are very close to 1.0 and all the dispersion measures indicate that the scatter is considerably reduced in comparison with the rest of the evaluated equations. It is also observed that the midspread (central 50% of the strength ratio observations) fall between 0.89 and 1.19, showing the smallest interquartile range. In the same manner, the proposed equation yields the lowest percent of over-predictions.

To further evaluate the proposed equation, a plot of the ratio between the peak shear strength and A_w*$\sqrt{f'_c}$ is presented in Fig. 3. This figure shows that the upper bound of the ACI 318 (0.83*$\sqrt{f'_c}$) is a reasonable value to avoid non-ductile diagonal compression failure in squat shear walls with rectangular cross sections. It also suggests that the limit becomes more important as the aspect ratio decreases. In addition, a good correlation between the predicted and measured normalized peak shear strengths can be noted.

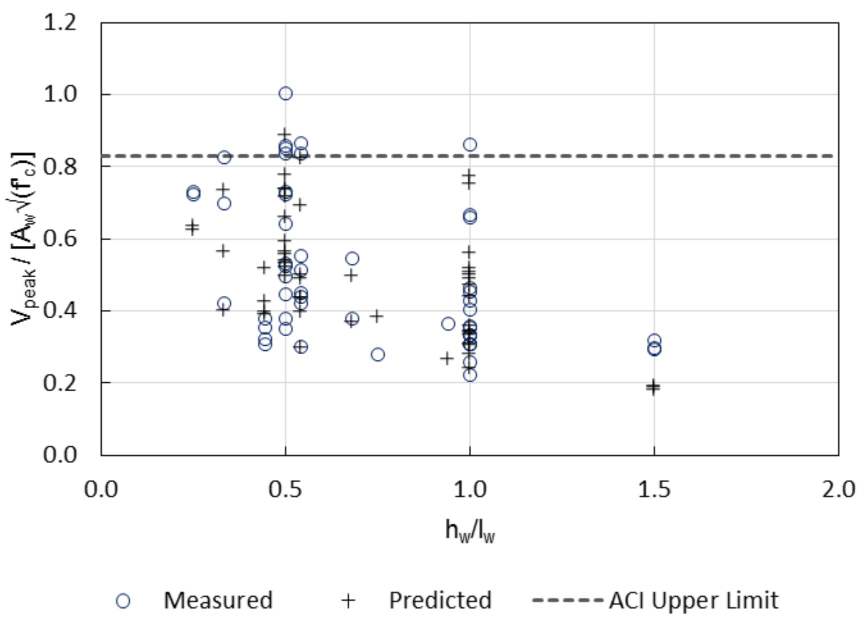

Fig. 3 – Variation of the Predicted and Measured Normalized Peak Shear Strength with Wall Aspect Ratio Compared to ACI 318 Upper Limit

5. Conclusions

This paper evaluated the peak shear strength of squat rectangular reinforced concrete walls from an assembled database from experimental walls tests found in the literature. The assembled experimental database considered only quasi-static cyclic, dynamic and hybrid-simulated dynamic loading squat wall tests. Several equations to predict peak shear strength that are available in the literature including the ones on several design standards (ACI and ASCE) were evaluated. From this evaluation, it was found significant scatter in the peak shear strength predictions among equations. A new equation to predict peak shear strength was proposed based on multivariable linear regression analyses considering the parameters that were found to produce a better correlation with the database. The new equation produces results of predicted-to-measured strength ratios with less standard deviation, lower coefficient of variation and lower percentage of over-predictions when compared with the other equations evaluated in this study. A good correlation from the predicted to measured peak shear strengths was obtained with the proposed equation. In addition, it was observed that the upper bound of the ACI 318 (0.83*$\sqrt{f'_c}$) is a reasonable value to avoid non-ductile diagonal compression failure in squat shear walls with rectangular cross sections and this limit becomes more important as the aspect ratio of the wall decreases.

6. Further Studies

Extend the database to include more RC squat wall experiments found in literature. The following tasks are underway: Develop predictive equations for walls with widened boundary elements (flanged and barbell) which can generally, reach higher shear stresses than rectangular cross sections. Develop predictive equations for the displacement capacity of squat walls and establish reasonable drift limits at yield, peak and different post-peak strength degradation stages for design purposes.

7. Acknowledgements

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