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PREDICTIVE SEISMIC SHEAR CAPACITY MODEL OF RECTANGULAR SQUAT RC SHEAR WALLS IN FLEXURAL AND SHEAR ZONES

D.T.W. Looi[(1)], R.K.L. Su[(2)]

[(1)] PhD candidate, The University of Hong Kong, dtwlooi@hku.hk
[(2)] Associate Professor, The University of Hong Kong, klsu@hku.hk

Abstract

This paper describes a comprehensive assessment of the seismic shear capacity of rectangular squat reinforced concrete walls through collection of up-to-date experimental database results. The theoretical shear capacity of wall specimens are first evaluated by axial-moment interaction curve in order to categorise the walls into shear, flexural-shear and flexural failure modes. It is shown that flexural failure is deemed possible for lightly reinforced squat walls of shear span-to-depth ratio around unity, in contrast with the commonly recognised shear failure merely governs by geometric aspect ratio or shear span-to-depth ratio of the wall. Important parameters contributing to shear strength i.e. shear span-to-depth ratio, axial load ratio, mechanical ratio of vertical and horizontal reinforcements and confinement effects are identified. Statistical multi-parameter regression analysis is adopted to formulate two reliable predictive seismic shear capacity empirical models distinctively for flexural zone and shear zone controlled walls. The proposed models predict the normalised shear stress with concrete cylinder strength, which is more robust compared to shear stress or shear force models. The models demonstrate more reasonable predictions compared to past shear capacity models with different functional forms recommended by other researchers and stipulated in design codes. The models were further affirmed through three squat reinforced concrete walls tested under high axial load. The merit of the proposed models lies within the coverage of the database especially for squat walls subjected to high axial load ratio, thus improves the accuracy of prediction. The high axial load characteristic of non-seismically detailed shear wall buildings with transfer structure in Hong Kong can be evaluated by this study in order to be resilient for future consideration of low-to-moderate seismicity action.

Keywords: shear walls; squat walls; high axial load; shear capacity; low-to-moderate seismicity

1. Introduction

The Southeast China region including Hong Kong is located in an intraplate area of low-to-moderate seismicity [1]. The lack of statutory seismic design codes in Hong Kong has exposed the risk of earthquake damage on many non-seismically detailed reinforced concrete (RC) shear walls with high axial loads induced by gravity. These shear walls may suffer from brittle shear coupled with axial failure due to rapid degradation of strength and stiffness during a rare earthquake event. The structural response of the lower part of a shear wall under high axial stress and lateral force is similar to that of a squat wall, measured by wall shear span-to-depth ratio. Such effects are more critical for walls above transfer structures (which is common in Hong Kong due to land scarcity), which concurrently suffering from gravity and lateral load shear concentration scenario [2, 3].

Research on the seismic behaviour of squat walls under high axial load is extremely rare as can be seen in the database collected by Wood [4]. Thus, this paper describes a comprehensive assessment of the seismic shear capacity of rectangular squat reinforced concrete walls through collection of up-to-date experimental database results with emphasis on axial load effects. The theoretical shear capacity of squat wall can be categorised into shear (Zone S) and flexural (Zone F) failure modes through simple axial-moment interaction curve. Statistical multi-parameter regression analysis is adopted to formulate two reliable predictive seismic shear capacity empirical models distinctively for Zone S and Zone F controlled walls. The proposed models predict the normalised shear stress with concrete cylinder strength (f_c') which is more robust compared to shear stress or shear force models. Simplified models using linear function graphs are presented to assist engineers in design office.

2. Shear strength of walls in design codes

The current structural concrete design codes in Hong Kong BD 2013 [5] was formulated with heavy referencing from BS 8110 [6], albeit with subtle alterations to suit the conditions specific to Hong Kong. However, there is no provision for shear design in shear wall. Hence, reference is drawn to other design codes. These other design codes generally take account of the contribution of concrete shear stress (v_c) and horizontal steel shear stress (v_s) via a 45-degree truss analogy in estimating the shear strength of shear walls. The higher tier strut-and-tie (STM) method which appears to be an alternative in squat RC wall D-region is excluded in the discussion, as the focus is given in some simple and general first tier shear strength estimation equations for both shear and flexural controlled squat walls.

2.1 ACI 318-14 [7]

Eq. (1) shows the detailed approach of in-plane shear strength calculation in accordance with Cl. 11.5.4.

$$V_n = V_c + V_s = \text{lesser}\left[\begin{matrix} 0.27\sqrt{f_c'}bd + \dfrac{Nd}{4L} \\ \left\{0.05\sqrt{f_c'} + \dfrac{L(0.1\sqrt{f_c'} + 0.2\dfrac{N}{bL})}{(\dfrac{M}{V} - \dfrac{L}{2})}\right\}bd \end{matrix}\right] + \rho_h f_{yh} bd \quad \le 0.83\sqrt{f_c'}bd \tag{1}$$

The denominator (M/V - L/2) in the second tier of Eq. (1) shall not apply if the value is negative. Provision for special structural wall in Cl. 18.10.4.1 is given in Eq. (2).

$$\begin{aligned} &V_n = V_c + V_s = (\alpha_c\sqrt{f_c'} + \rho_h f_{yh})bd \quad \le 0.83\sqrt{f_c'}bd \\ &\text{where} \quad \alpha_c = 0.25 \quad for\ H/L \le 1.5 \\ &\qquad\qquad \alpha_c = 0.17 \quad for\ H/L \ge 1.5 \end{aligned} \tag{2}$$

The λ factor for lightweight concrete is excluded for clarity. It is worth to note that the approach of NZS 3101 [8] is similar to ACI 318, except the limit to prevent strut crushing is set forth at $0.2f_c$' or 8 MPa rather than $0.83\sqrt{f_c}$'.

2.2 CSA A23.3-14 [9]

The Canadian Standard which advocated the modified compression field theory (MCFT) has significant role in the knowledge of shear. Eq. (3) which was replicated from Cl. 21.5.9.5.3 shows the unique parameter of β = 0.1 and θ = 45-degree in a simplified method to assess shear capacity of moderately ductile flexural controlled shear wall.

$$V_n = V_c + V_s = (\beta\sqrt{f_c'} + \rho_h f_{yh}\cot\theta)bd \quad \le 0.125 f_c' bd \tag{3}$$

The material resistance factor ϕ and λ factor for lightweight concrete are excluded for clarity.

2.3 GB50011-2010 [10] and JGJ3-2010 [11]

The mainland Chinese code for seismic design of buildings GB 50011 [10] and the Technical Specification for concrete structures of tall building JGJ3 [11] stipulate clauses for shear wall design by cross-referencing to its concrete design code GB 50010 [12]. The shear provisions of shear wall under eccentric compression can be estimated using Eq. (4), with shear limits imposed to account for concrete strength and aspect ratio. Interestingly, concrete tensile strength (f_t) is used as the parameter for estimating shear rather than concrete cylinder strength (f_c') or cube strength (f_{cu}). The maximum allowable ALR is limited to 0.2.

$$V_n = V_c + V_s = \frac{1}{0.85}\left[\frac{1}{a/L - 0.5}\left(0.4 f_t + 0.1\frac{N}{bd}\right) + 0.8\rho_h f_{yh}\right]bd \le \begin{bmatrix} \frac{1}{0.85}0.20\beta_c f_{cu}bd & \text{for } a/L \ge 2.5 \\ \frac{1}{0.85}0.15\beta_c f_{cu}bd & \text{for } a/L < 2.5 \end{bmatrix}$$

$$\begin{aligned} \text{where} \quad & (a/L)_{\min} = 1.5;\ (a/L)_{\max} = 2.2 \\ & \beta_c = 1.0 \quad \text{for} \le C50 \\ & \beta_c = 0.8 \quad \text{for} \ge C80 \\ & N \le 0.2 f_{cu}bd \end{aligned} \tag{4}$$

3. Shear strength of walls in literature

Krolicki et al. [13] proposed a physical shear strength model by revising the University of California at San Diego (UCSD) model [14], with the aim to improve the calculation of shear resistance of squat RC walls. Besides the common concrete and steel shear strength components, a third component i.e. axial compression is introduced in Eq. (5).

$$\begin{aligned} & V_n = (V_c + V_s) + V_p = (\alpha_p\beta_k\gamma_p\sqrt{f_c'} + \rho_h b h_{cr} f_{yh})bd + P\tan\xi \\ \text{where} \quad & \alpha_p = 3 - (a/L) \ge 1.0 \\ & \beta_k = 0.5 + 20\rho_v \le 1.0 \\ & \gamma_p = 0.29 \text{ (for low ductility); } 0.05 \text{ (for high ductility)} \\ & h_{cr} = \frac{L - c - c_0}{\tan\theta_{cr}} \le H; \quad c_0 \text{ is the concrete cover} \\ & \theta_{cr} = -7.5(a/L) + 45 \le 30^\circ \\ & \tan\xi = \frac{L-c}{2a}; \quad c \text{ is the length of compression at wall base} \end{aligned} \tag{5}$$

Empirical formulation as shown in Eq. (6) to predict peak shear strength of low aspect ratio rectangular RC walls was carried out by Gulec & Whitaker [15]. The proposed equation is only valid for aspect ratio of 1.0 or less and was simplified for design purpose. Attempt was given to include axial force in the last component in the numerator. All units are in imperial.

$$V_n = \frac{1.5\sqrt{f_c'}bd + 0.25\rho_{v,web}bl_{web}f_{yv} + 0.20\rho_{v,be}bl_{be}f_{yv,be} + 0.40P}{\sqrt{H/L}} \leq 10\sqrt{f_c'}bd \quad \text{(imperial unit)} \tag{6}$$

Experimentally calibrated code-form model was proposed by Sanchez-Alejandre & Alcocer [16] in Eq. (7) for squat RC walls.

$$\begin{aligned} V_n &= V_c + V_s = \left[(\gamma_{SA}\eta_v + 0.04\frac{P}{A})\sqrt{f_c'} + (\eta_h\rho_h f_{yh})\right]bd \\ \text{where} \quad \gamma_{SA} &= 0.42 - 0.08(a/L) \\ \eta_v &= 0.75 + 0.05\rho_v f_{yv} \\ \eta_h &= 1 - 0.16\rho_h f_{yh} \geq 0.20 \text{ MPa} \end{aligned} \tag{7}$$

The above shear strength Eqs. (5-7) are carefully selected from more recent literature as they qualified the criteria of squat walls and axial load consideration.

4. RC Wall experimental database

An attempt of classifying RC columns into Zone S and Zone F was reported in Zhu et al. [17]. A modified approach is extended here onto RC shear walls in view of their similar structural functions compared to columns. A database of 183 rectangular RC squat shear walls with complete information of peak shear and post-peak ductility tested under cyclic, dynamic and monotonic loading was collected. Conventional axial-moment (PM) interaction chart is first used to examine all the squat walls with shear span-to-wall length (*a/L*) ratio of less than 3. The theoretical moment capacity (M_f) obtained will be divided by the wall shear span to arrive at a theoretical shear capacity under flexural action ($V_f = M_f/a$). The ratio of V_f over shear obtained in experiment (V_f/V_{exp}) will then be computed. To cater for uncertainty (i.e. ill-informed results), variation of material parameters (i.e. ultimate strains of concrete and steel, Young's modulus of reinforcement) and design philosophy (i.e. concrete compression stress block), and a plus minus 10% band is imposed for a shear-flexural mode. Fig. 1 shows the classification of the database categorised the walls into Zone S and Zone F.

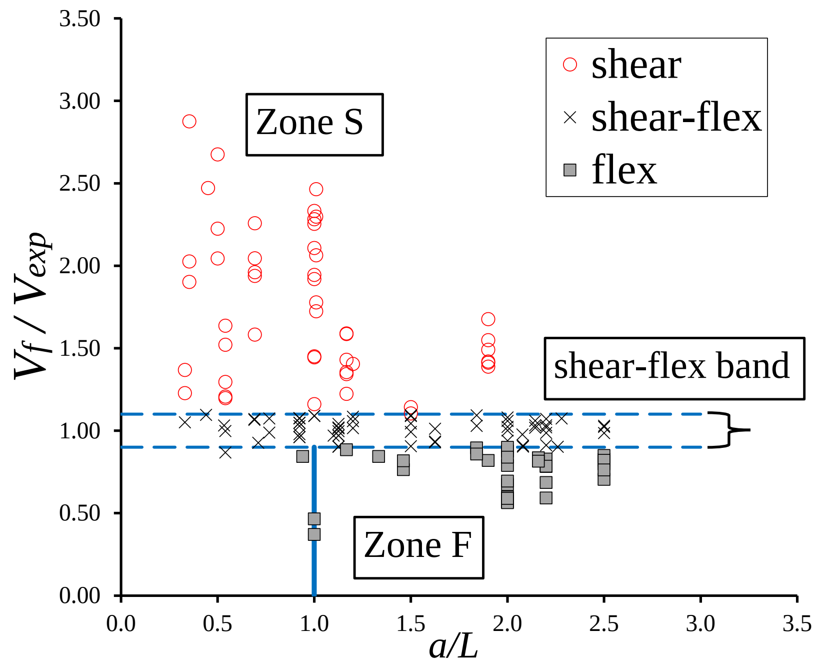

Fig. 1 – Database of rectangular RC squat walls categorised into Zone S and Zone F

Interestingly, it is shown that flexural failure (or ambiguous shear flexural failure) is deemed possible for lightly reinforced squat walls of shear span-to-depth ratio around unity, in contrast with the commonly recognised shear failure merely governs by geometric aspect ratio or shear span-to-depth ratio of the wall. In fact, a note in CSA A23.3 code Cl. 21.5.10.2 [9] pointed out that a squat wall can develop either a flexural or a shear mechanism.

Owing to the large database, only the summary of the range of data is shown in Table 1. Some more recent literature used in the database is listed in [18-20].

Table 1 – Range of data for wall specimens

Variable	Zone S	Shear-flex band	Zone F	All
Concrete cylinder strength, f_c'(MPa)	15.7-70.3	15.8-57.5	15.8-48.6	15.7-70.3
Axial load ratio, ALR = $P/A/f_c$'	0.00-0.40	0.00-0.50	0.00-0.38	0.00-0.50
Shear span-to-wall length ratio, a/L	0.4-2.6	0.4-2.5	0.9-2.5	0.4-2.6
Vertical reinforcement ratio in web, $\rho_{v,web}$ (%)	0.13-2.84	0.14-1.72	0.00-0.14	0.00-2.84
Horizontal reinforcement ratio in web, $\rho_{h,web}$ (%)	0.11-1.50	0.11-1.72	0.12-0.86	0.11-1.50
Vertical reinforcement ratio in boundary element, $\rho_{v,be}$ (%)	0.00-12.74	0.00-13.46	0.00-12.57	0.00-13.46
Hoop reinforcement volumetric ratio in boundary element, $\rho_{hoopbe,vol}$ (%)	0.00-4.34	0.00-2.67	0.00-4.34	0.00-4.34
Total vertical web reinforcement ratio, ρ_v (%)	0.25-4.25	0.39-4.60	0.13-2.49	0.13-4.60
Normalised shear stress ratio, v/f_c'	0.03-0.22	0.03-0.24	0.03-0.23	0.03-0.24

5. Proposed predictive seismic shear capacity model

Statistical multi-parameter regression analysis is adopted to formulate two reliable predictive seismic shear capacity empirical models distinctively for Zone S and Zone F walls. The ambiguous shear-flex band data is incorporated into each of the zone for independent analysis in order to average out the uncertainty differences. The proposed models predict the normalised shear stress ratio (v/f_c') which is dimensionless and more robust (less sensitive) compared to shear stress or shear force models. Eq. (8) is the proposed predictive seismic shear capacity model for Zone S squat RC shear walls. Similar form of equation for Zone F squat RC shear walls is shown in Eq. (9).

$$\begin{aligned}
&\underline{\text{Zone S}}: \quad \frac{v}{f_c'} = 0.034 + A\,(ALR)^{1.3} + B\,\rho_v \frac{f_{yv}}{f_c'} + C\,\rho_h \frac{f_{yh}}{f_c'} + D\,\rho_{v,be} \frac{f_{ybe}}{f_{cc}'} \le 0.24 \\
&\text{where} \quad A = 0.283 - 0.084a/d \\
&\qquad\quad\;\; B = 0.4 - 0.15a/d \\
&\qquad\quad\;\; C = 0.5 - 0.2a/d \\
&\qquad\quad\;\; D = -0.08 + 0.06a/d
\end{aligned} \tag{8}$$

$$\begin{aligned}
&\underline{\text{Zone F}}: \quad \frac{v}{f_c'} = 0.015 + A\,(ALR) + B\,\rho_v \frac{f_{yv}}{f_c'} + C\,\rho_h \frac{f_{yh}}{f_c'} + D\,\rho_{v,be} \frac{f_{ybe}}{f_{cc}'} \le 0.24 \\
&\text{where} \quad A = 0.342 - 0.034a/d \\
&\qquad\quad\;\; B = 0.4 - 0.08a/d \\
&\qquad\quad\;\; C = 0.7 - 0.4a/d \\
&\qquad\quad\;\; D = -0.07 + 0.04a/d
\end{aligned} \tag{9}$$

The equation form has implicitly considered the component of shear strength contribution of concrete (v_c), axial compression (v_p) and steel reinforcement (v_s), with simple derivation shown in Eq. (10).

$$\begin{aligned}
&\text{General form}: \\
&\frac{v}{f_c'} = \text{constant} + A\,(ALR) + B\,\rho_v \frac{f_{yv}}{f_c'} + C\,\rho_h \frac{f_{yh}}{f_c'} + D\,\rho_{v,be} \frac{f_{ybe}}{f_{cc}'} \\
&\text{Contributing components}: \\
&v = [\text{constant}\ f_c'] + [A\,(ALR)\,f_c'] + \left[B\,\rho_v\,f_{yv} + C\,\rho_h\,f_{yh} + D\,\rho_{v,be}\,f_{ybe}\frac{f_c'}{f_{cc}'}\right] \\
&v = v_c + v_p + v_s
\end{aligned} \tag{10}$$

It should be noted that the reinforcement ratio (ρ_v, ρ_h and $\rho_{v,be}$) is in decimal (not percentage). The shear stress (v) and the shear span-to-depth ratio (a/d) is based on the assumption of shear depth, d = 0.8L, consistent with the recommendation in ACI 318 Cl. 11.5.4.2 [7] and CSA A23.3 Cl. 21.5.9.2 [9]. The concrete confined by hoops at boundary element is accounted for using the experimentally observed relationship proposed by Saatcioglu & Razvi [21] and is mathematically shown in Eq. (11).

$$\begin{aligned}
&f_{cc}' = f_c' + C_{conf}\ \rho_{hoopbe,vol}\ f_{yhoop} \\
&\text{where} \quad C_{conf} = \min\left[1, \sqrt{\left(\frac{l_{be}}{s_{v,hoop}}\right)\left(\frac{l_{be}}{s_{h,vbe}}\right)\left(\frac{1}{\rho_{hoopbe,vol}\dfrac{f_{yhoop}}{f_c'}}\right)}\right]
\end{aligned} \tag{11}$$

The empirical equations presented herein have uniquely identified shear span-to-depth ratio as a parameter coupled with ALR and mechanical reinforcing index of web and boundary element forming coefficient *A* to *D*. For application in design office, linear function charts are presented in Fig. 2a and Fig. 2b.

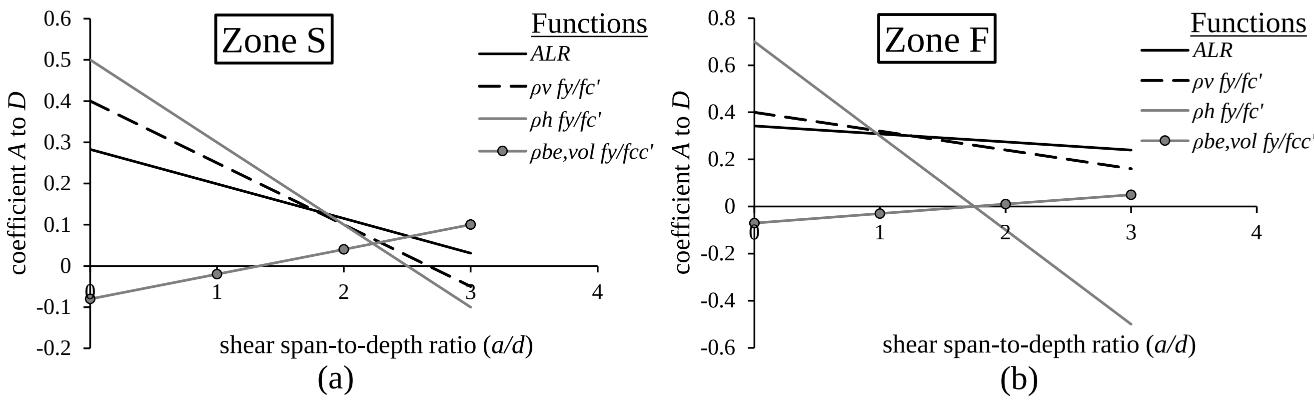

Fig. 2 – Linear functions of proposed model in Zone S and Zone F

6. Comparison of the proposed model with design codes and the literature

The proposed models in Eqs. (8-10) are corroborated with the collected database in Fig. 3a-3b to ensure near to unity mean and median predictions with minimum coefficient of variation (COV) for Zone S and Zone F.

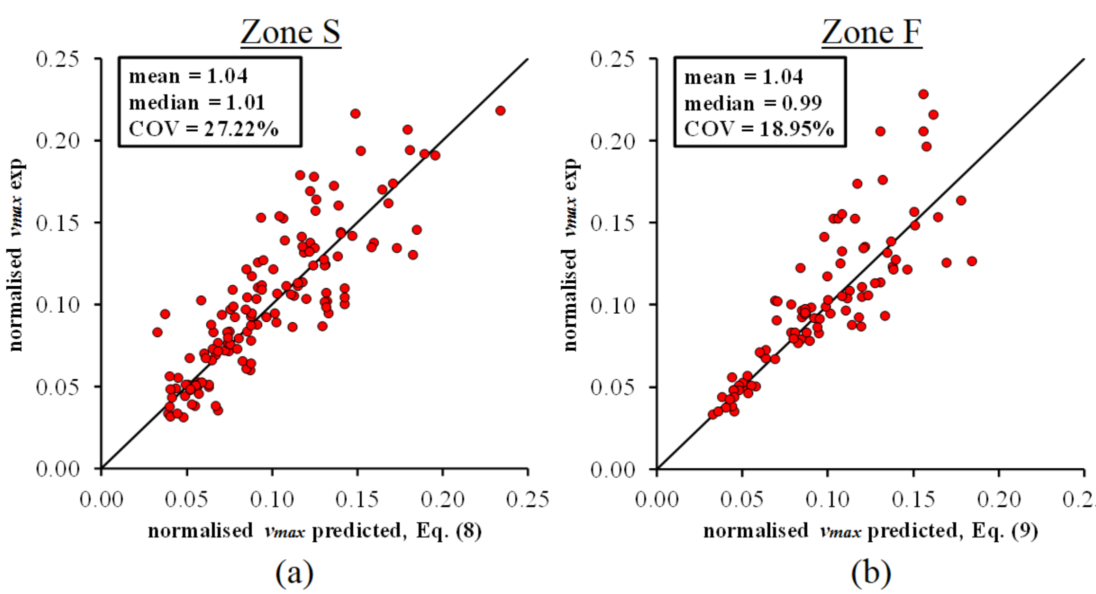

Fig. 3 – Corroboration of experimental to proposed prediction of shear strength in Zone S and Zone F

The design codes introduced earlier are compared to the collected experimental results, illustrated in Fig. 4a-4c for Zone S and Fig. 4d-4f for Zone F.

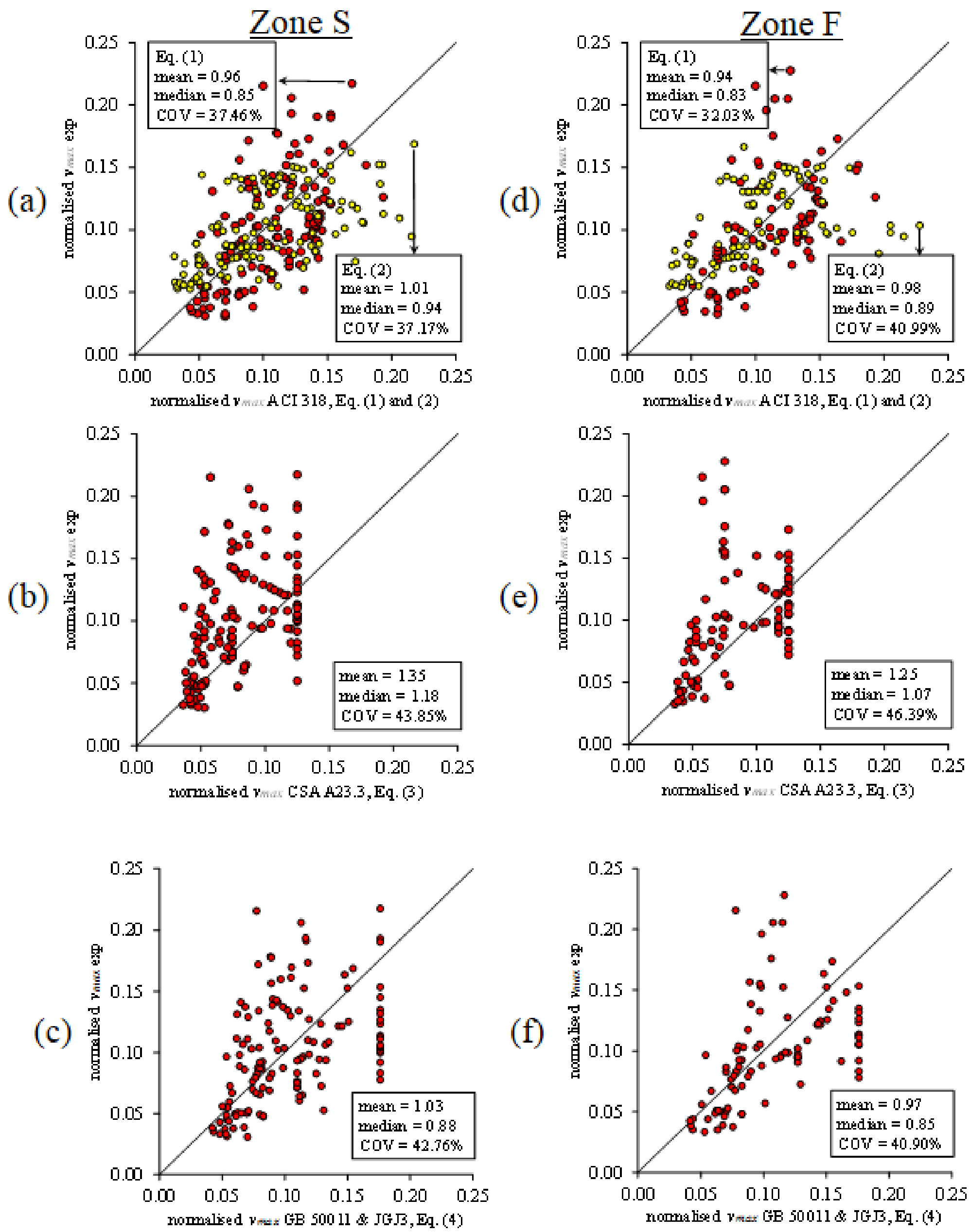

Fig. 4 – Comparison of experimental to code prediction of shear strength in Zone S and Zone F

The ACI 318 code [7] using either the detailed calculation method for a non-seismically designed wall, or provisions for earthquake resistant wall gave satisfactory prediction, although the latter shows a better mean and median. The CSA A23.3 MCFT approach [9] shows generally conservative results, probably due to the imposed strut compression limit. A possible refinement can be done by carrying out more rigorous method using the recommendation in Cl. 21.5.9 to account for shear resistance of inside and outside of plastic hinge. It is however noted that the inelastic rotational demand of the wall is required as an input. The results of GB 50011 code [9] or JGJ3 [11] thus far show good mean but fair median and COV.

Similarly, some recent proposals by other researchers are compared with the collected database. At a glance in Fig. 5a and Fig. 5d, the results of Krolicki et al. [13] appear to be scattered. The main uncertainty lies in the estimation of γ_p for low and high ductility, which is extraordinarily sensitive. The empirical formulation of Gulec & Whittaker [15] which was tailored strictly for squat walls with aspect ratio of 1.0 or less gave reasonably good prediction in Zone F (see Fig. 5e) compared to Zone S (see Fig. 5b). Although the database collected in [15] might well have been included as part of the database in this paper (which will affect the outcome of the empirical analysis), it is however noted that the flexural and shear critical characteristics are coupled in one Eq. (6). The proposal by Sanchez-Alejandre & Alcocer [16] appears to be less scattered generally but deviates from the mean and median values (see Fig. 5c and Fig. 5f). It is more ideal that some minor modifications to be carried out to refine Eq. (6) and Eq. (7) if they were to be used for squat wall shear strength estimation.

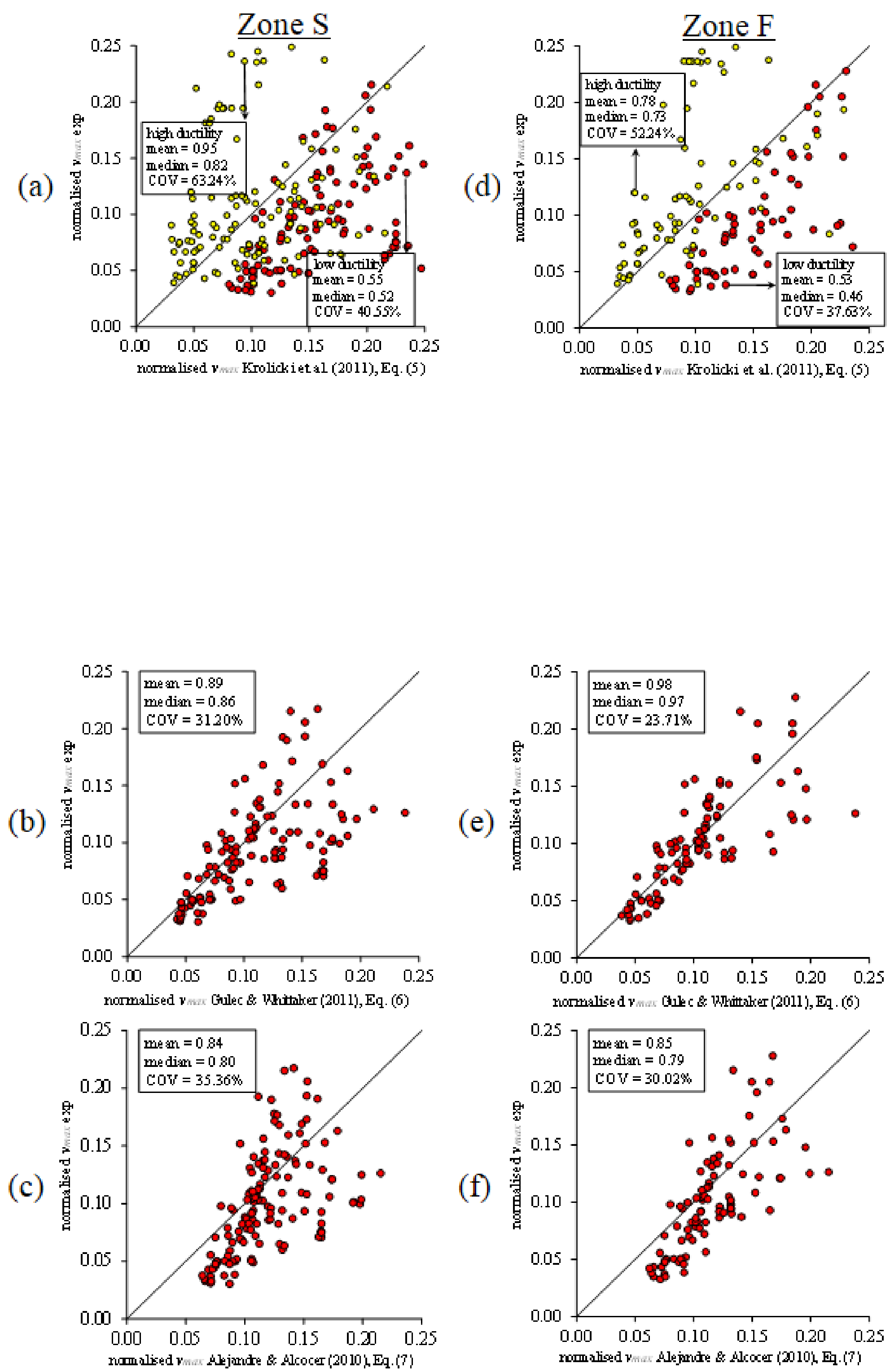

Fig. 5 – Comparison of experimental to literature prediction of shear strength in Zone S and Zone F

Thus far, most of the simplistic past shear capacity models with different functional forms recommended by other researchers and stipulated in design codes do not correlate well for both the Zone S and Zone F database. To further validate the proposed models, experimental verification was carried out on two planar RC squat walls under high ALR.

7. Further research – experimental verification

Three rectangular squat RC walls under ALR of approximately 0.2, 0.3 and 0.4 were tested under reversed cyclic loading to axial failure. The dimension of the wall specimens is fixed at 800 mm x 800 mm x 80 mm thick, with shear span of 950 mm at the horizontal level of actuator (see Fig. 6). The specimens were reinforced with 2% 10 mm diameter high tensile ribbed (T10) vertical steel and 1.4% 8 mm diameter plain (R8) horizontal steel. (without boundary element). The concrete cylinder strength f_c' ranges from 27.6 MPa and 29.1 MPa, and steel yield strength f_{yv} is 601 MPa and f_{yh} is 289 MPa respectively for T10 and R8 bars.

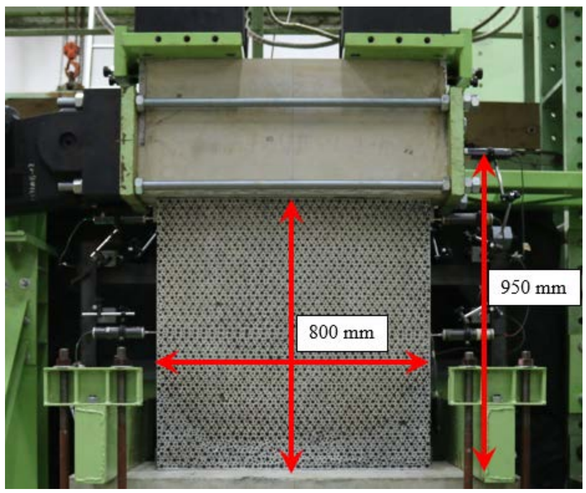

Fig. 6 – Typical squat RC wall test setup

Table 2 shows the calculated shear strength compared to the one obtained from experiment. Coefficient *D* is omitted as the wall contains no boundary elements. The results show excellent prediction of the proposed model compared to the wall tested. More tests are in queue to be carried out on Zone F and walls with boundary elements, in order to further examine the robustness of the proposed models.

Table 2 – Verification of prediction model compared to tested high ALR squat walls

Specimen	M_f (kNm)	V_f (kN)	V_{exp} (kN)	**Zone**	f_c' (MPa)	ρ_v (%)	ρ_h (%)	f_{yv} (MPa)	f_{yh} (MPa)	$(v/f_c')_{exp}$
ALR 0.2	310	326.3	245	S	27.6	2	1.4	601	289	0.173
ALR 0.3	335	352.6	255	S	28.0	2	1.4	601	289	0.178
ALR 0.4	380	400.0	256	S	29.1	2	1.4	601	289	0.171
	H/L	***a/d***	***A***	***B***	***C***	***ALR***	$\rho_v f_y/f_c'$	$\rho_h f_y/f_c'$	$(v/f_c')_{model}$	$(v/f_c')_{exp}/(v/f_c')_{model}$
ALR 0.2	1	1.48	0.159	0.178	0.204	0.22	0.43	0.15	0.161	1.07

ALR 0.3	1	1.48	0.159	0.178	0.204	0.38	0.42	0.15	0.183	0.97
ALR 0.4	1	1.48	0.159	0.178	0.204	0.42	0.41	0.14	0.186	0.92

8. Summary and conclusions

An up-to-date experimental database of rectangular squat RC shear walls was collected. The theoretical shear capacity of wall specimens is first evaluated by PM interaction curve in order to categorise them into Zone S or Zone F. Two reliable predictive seismic shear capacity empirical models distinctively for Zone S and Zone F were proposed, calibrated through statistical multi-parameter regression analysis on the database. The proposed models encapsulated the contribution of axial load, shear span-to-depth ratio and various mechanical reinforcing indexes to predict the normalised shear stress with concrete cylinder strength which is more robust than the prediction of shear stress or shear force. The robustness of the normalisation to eliminate variation of concrete strength is evident in the database results summarised in Table 1, where the narrower range 0.03-0.24 is obtained rather than a wider range of shear force. The STM diagonal strut and node failure (which is often observed in squat wall failure) had warranted the use of concrete strength as a function in shear strength prediction [22]. These models demonstrated more reasonable estimations compared to past shear capacity models with different functional forms recommended by other researchers and stipulated in design codes. Three squat RC walls under 0.2, 0.3 and 0.4 high ALR were experimentally tested and the shear strength values were compared to the proposal, which yields an excellent agreement. The merit of the proposed models lies within the coverage of the database especially for squat walls subjected to high ALR, thus improves the accuracy of prediction. The high axial load characteristic of non-seismically detailed shear wall building with transfer structure in Hong Kong can be evaluated by this study in order to be resilient for future consideration of low-to-moderate seismicity action.

9. Acknowledgement

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10. Notation

H and *L* = wall height and wall length respectively

V_c = maximum shear force contributed by concrete

V_n = total maximum shear force

V_s = maximum shear force contributed by stirrups

a = shear span (*M/V*)

b = wall thickness

d = effective wall length for shear depth calculation, assumed as 0.8 *L*

f_{cc}' = confined concrete strength (unit MPa)

f_{ybe} = vertical steel reinforcement yield strength in boundary element (unit MPa)

f_{yh} = horizontal steel reinforcement yield strength (unit MPa)

f_{yhoop} = hoop reinforcement yield strength (unit MPa)

f_{yv} = total vertical steel reinforcement yield strength (unit MPa)

$f_{yv,\ web}$ = vertical steel reinforcement yield strength in web area (unit MPa)

l_{be} = boundary element length along in-plane direction of wall

l_{web} = web length along in-plane direction of wall

$s_{v,hoop}$ = vertical distance of hoop in boundary element

$s_{h,vbe}$ = horizontal distance of vertical reinforcement in boundary element along in-plane direction of wall

ρ_v = vertical steel reinforcement ratio

ρ_h = horizontal steel reinforcement ratio

$\rho_{hoopbe,vol}$ = hoop steel reinforcement volumetric ratio

$\rho_{v,be}$ = vertical steel reinforcement ratio in boundary element

$\rho_{v,web}$ = vertical steel reinforcement ratio in web

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