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Simplified Strut-and-Tie Model for Shear Strength Prediction of Reinforced Concrete Low-Rise Walls

by Jimmy Chandra and Susanto Teng

In this study, an analytical model using the strut-and-tie concept was developed to predict reinforced concrete (RC) low-rise wall shear strengths. In the model, the failure mode considered was crushing of the diagonal compression strut. To accurately determine the strut area, a formula for calculating depth of compression zone at the bottom of wall was derived with the aid of nonlinear finite element analysis. A total of 100 RC low-rise wall specimens failing in shear obtained from available literature were used to verify the accuracy of wall strength predictions of the proposed strut-and-tie model. Furthermore, strength predictions from building codes and other analytical models were also included for comparison purposes. The analysis results show that the proposed strut-and-tie model is conservative and it has the lowest coefficient of variation as compared to other methods in predicting the shear strength of RC low-rise walls. In addition, the predictions of the proposed model are quite consistent and less scattered for wide ranges of wall height-length ratios and concrete compressive strengths.

Keywords: building code predictions; reinforced concrete (RC) wall shear strengths; strut-and-tie.

INTRODUCTION

The use of reinforced concrete (RC) walls has become increasingly popular nowadays due to their superior performance against lateral loads such as wind and earthquake loads.[1] In addition, not only for lateral loads, RC walls can also be used to resist gravity loads as well. Thus, it is important to be able to determine the strength of RC walls accurately to provide safe and economical design, as these are two major concerns for structural engineers. Previous studies by the authors[2,3] show that the flexural strength of RC walls can be reasonably well predicted using flexural theory for members subjected to axial load and bending moment. However, for the shear strength, empirical building code formulas[4,5] underestimate RC wall shear strengths by a significant margin, especially for high-strength concrete (HSC) walls, and the overall predictions are quite scattered. Therefore, there was a need to develop an analytical model based on rational theory to accurately predict the shear strength of RC walls.

The rational theory for predicting RC members' shear strength was developed in early 1900s based on the truss analogy.[6,7] The theory was further developed to predict the shear strength of RC members more accurately.[8,9] For RC low-rise walls having a height-length ratio (H_w/L_w) less than 2.5, many researches have been conducted to predict the shear strength.[10-13] All those theories are able to predict the shear strength of RC low-rise walls with certain accuracy. However, in their truss models, it was assumed that shear stress distribution over an entire wall cross section was uniform which is only valid for RC low-rise walls with boundary elements having H_w/L_w less than 1.0.[10,11] Moreover, the calculation of RC low-rise wall shear strengths using their models needs an iterative procedure to obtain a solution that satisfies equilibrium and compatibility conditions as well as constitutive law of materials. Thus, it may not be practical to be used by engineers to estimate the shear strength of RC low-rise walls.

In this study, an analytical model for predicting RC low-rise wall shear strengths was developed based on the strut-and-tie concept. RC low-rise walls having H_w/L_w less than 2.5 can be categorized as disturbed regions where a plane section does not remain plane. In this case, the strut-and-tie model is considered as a rational approach to predict the strength of disturbed regions.[14] Later on, experimental wall strengths obtained from available literatures were used to verify the accuracy of the proposed strut-and-tie model. In addition, strength predictions from building codes[4,5] and other strut-and-tie models[15,16] were included as well for comparison purposes.

RESEARCH SIGNIFICANCE

This study focused on the development of an analytical model based on the strut-and-tie concept to predict RC low-rise wall shear strengths. It is expected that the model could serve as a rational yet simple approach for predicting the shear strength of RC low-rise walls. Furthermore, the study conducted here provides a new formula for calculating the depth of the compression zone at the bottom of RC low-rise walls in which the assumption of plane section remains plane (linear strain distribution) is not valid. The formula was developed with the aid of nonlinear finite element analysis (FEA) using ATENA software.[17] This is important to accurately predict the shear strength of RC low-rise walls.

BUILDING CODES AND OTHER ANALYTICAL MODELS

ACI 318-19[4] and Eurocode 8[5] are two reference building codes that are adopted in many countries. As such, those two

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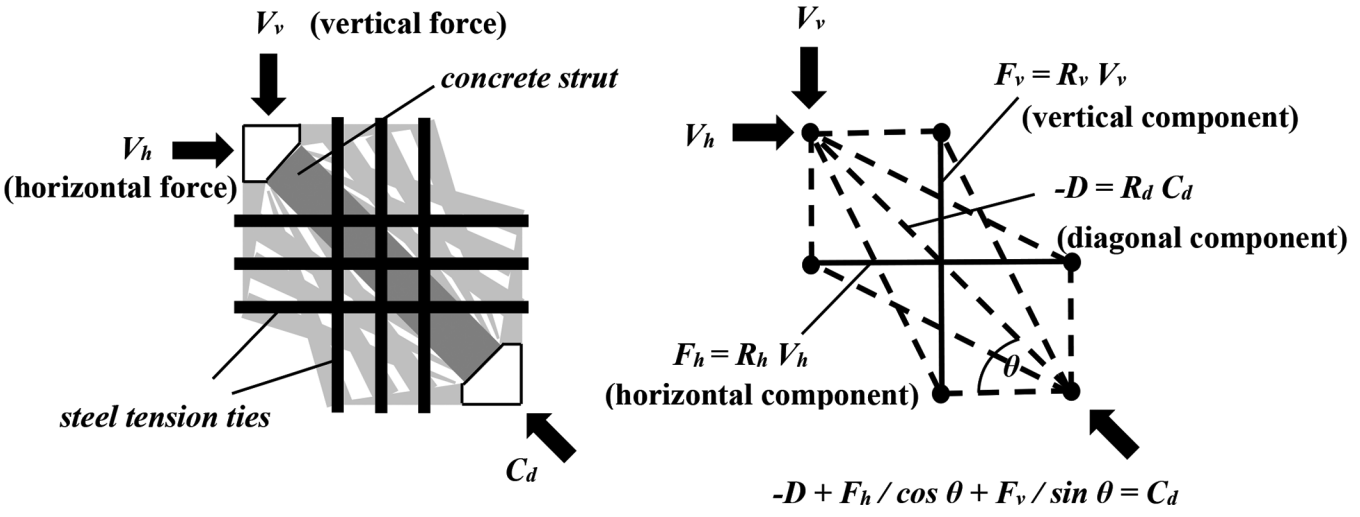

(*a) Disturbed stress field*

(b) Strut and tie idealization

Fig. 1—Strut-and-tie mechanisms proposed by Hwang and Lee.[15]

building codes and other strut-and-tie models proposed by other researchers[15,16] are reviewed briefly as follows.

ACI 318-19

According to ACI 318-19,[4] the nominal shear strength V_n of RC special structural walls can be calculated as follows

$$V_n = A_{cv}\left(\alpha_c \lambda \sqrt{f_c'} + \rho_t f_{yt}\right)$$

ACI 318-19 also states that the value of V_n shall not exceed $0.83A_{cw}\sqrt{f_c'}$ (in N).

Eurocode 8 (EN 1998-1:2004)

According to Eurocode 8[5] (EC8), the shear strength of RC walls subjected to earthquake loadings can be taken as the lesser value of shear resistance from two failure modes: 1) diagonal compression failure $V_{Rd,max}$; and 2) diagonal tension failures, either $V_{Rd,s}$ or V_{Rd}.

Diagonal compression failure of web due to shear—For the case of diagonal compression failure, the shear strength is calculated as follows

$$V_{Rd,max} = \alpha_{cw} b_w z \nu_1 f_{cd} / \left(\cot\theta + \tan\theta\right) \quad (1)$$

where the recommended value of α_{cw} is as follows

$$1.0 \text{ for non-prestressed structures} \quad (1a)$$

$$(1.0 + \sigma_{cp}/f_{cd}) \text{ for } 0 < \sigma_{cp} \le 0.25 f_{cd} \quad (1b)$$

$$1.25 \text{ for } 0.25 f_{cd} < \sigma_{cp} \le 0.5 f_{cd} \quad (1c)$$

$$2.5\,(1.0 - \sigma_{cp}/f_{cd}) \text{ for } 0.5 f_{cd} < \sigma_{cp} < 1.0 f_{cd} \quad (1d)$$

The recommended value for ν_1 is 0.6 $[1.0 - f_{ck}/250]$ (f_{ck} in MPa).

EC8 recommends that the values of cotθ and tanθ are taken as 1.0.

Diagonal tension failure of web due to shear—If $\alpha_s = M_{Ed}/(V_{Ed}L_w) \ge 2.0$, where M_{Ed} is the design bending moment at the base of the wall and V_{Ed} is the design shear force, the shear strength is given by $V_{Rd,s}$

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta \quad (2)$$

If $\alpha_s = M_{Ed}/(V_{Ed}\,L_w) < 2.0$, the shear strength is given by V_{Rd}

$$V_{Rd} = V_{Rd,c} + 0.75\rho_h f_{yd,h} b_{wo} \alpha_s L_w \quad (3)$$

Hwang and Lee's model

Hwang and Lee[15] proposed a softened strut-and-tie model for calculating the shear strength of RC walls. The model has the term "softened" because it takes into account the softening behavior of cracked concrete. In the model, the external forces were resisted by a combination of concrete compression struts and steel tension ties as shown in Fig. 1. There are three load paths—that is, vertical, horizontal, and diagonal components which are calculated according to their relative stiffness (R_v, R_h, and R_d), and these components are combined together to become the diagonal compression force acting on nodal zone C_d. The nominal capacity of the nodal zone can be calculated using Eq. (4a). Then, the shear strength of RC wall according to this model can be taken as the horizontal component of the diagonal compression force that is corresponding to the nominal capacity of the nodal zone

$$C_{d,n} = K\zeta f_c' A_{str} \quad (4a)$$

where K is strut-and-tie index, which is defined as follows

$$K = \frac{-D + \dfrac{F_h}{\cos\theta} + \dfrac{F_v}{\sin\theta}}{-D + \dfrac{F_h}{\cos\theta}\left(1 - \dfrac{\sin^2\theta}{2}\right) + \dfrac{F_v}{\sin\theta}\left(1 - \dfrac{\cos^2\theta}{2}\right)} \geq 1.00 \tag{4b}$$

and ζ is softening coefficient of cracked diagonal concrete strut, which in this model, it is calculated as $(3.35/\sqrt{f_c'}) \leq 0.52$.

Kassem's model

Kassem[16] proposed a strut-and-tie model and closed-form design formula for predicting the shear strength of squat walls. The model uses three shear-resisting mechanisms—that is, diagonal, horizontal, and vertical mechanisms—similar to Hwang-Lee's model.[15] In this model, a parametric expression to calculate the shear strength of squat walls was developed and calibrated using data of 645 walls obtained from literature. The design formulas developed are as follows (in SI units):

For walls with rectangular cross section

$$V_n = 0.27 f_c' \left[\psi k_s \sin(2\theta) + 0.11 \omega_h \frac{H'}{d_w} + 0.30 \omega_v \cot(\theta) \right] t_w d_w \leq 0.83 \sqrt{f_c'} t_w d_w \tag{5a}$$

For walls with flanged cross section

$$V_n = 0.47 f_c' \left[\psi k_s \sin(2\theta) + 0.15 \omega_h \frac{H'}{d_w} + 1.76 \omega_v \cot(\theta) \right] t_w d_w \leq 1.25 \sqrt{f_c'} t_w d_w \tag{5b}$$

PROPOSED STRUT-AND-TIE MODEL

In this study, an analytical model for predicting RC low-rise wall shear strengths was developed based on the strut-and-tie concept. The behavior of RC low-rise wall having a height-length ratio (H_w/L_w) less than 2.5 is dominated by shear mode[18,19] and it can be categorized as a disturbed region where a plane section does not remain plane and shear stress is not uniform within the wall panel. Thus, the strut-and-tie model is considered a more appropriate approach to predict the strength as compared to the sectional design model which includes concrete resistance to shear V_c due to tensile stresses in concrete.[14,20] In contrast to Hwang and Lee's softened strut-and-tie model[15] that uses three compression struts, the model developed in this study uses only one diagonal compression strut to be simple. Furthermore, the contribution of web reinforcement is accounted in terms of confinement effect to the diagonal compression strut.

Equilibrium of proposed strut-and-tie model

Initially, a typical RC low-rise wall with axial load P and lateral load V as displayed in Fig. 2 has reaction forces at the bottom of the wall—that is, horizontal reaction force that is equal to V, vertical reaction force, and bending moment that can be represented by a combination of tension force T and compression force C. To simplify the load transfer

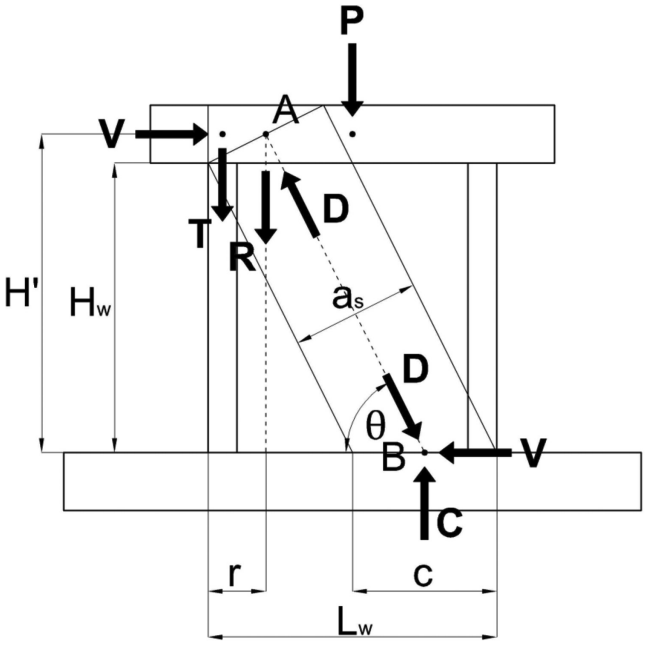

Fig. 2—Equilibrium of proposed strut-and-tie model.

mechanism, a resultant force R is used to replace the axial load P and tension force T in the equilibrium equation. The resultant force R and lateral load V are equilibrated at point A by diagonal compression force D and thus, it forms a strut-and-tie model. The diagonal compression force D is equilibrated at point B by compression force C and horizontal reaction force that is equal to V. The governing failure mode of the model is crushing of diagonal compression strut which represents shear failure of the wall web. The internal and external forces equilibrium of the model is described as follows

$$R = C = D\sin\theta \tag{6}$$

$$V = D\cos\theta \tag{7}$$

Determination of depth of compression zone at bottom of wall

In this model, depth of compression zone at the bottom of wall c as displayed in Fig. 2 has to be determined first before calculating the diagonal compression strut capacity. Initially, the authors calculated the depth of compression zone c based on flexural theory with the assumption of linear strain distribution along the wall cross section. Nevertheless, this assumption led to inaccurate predictions of RC wall shear strengths. This was because the assumption might not be valid for RC low-rise wall that can be categorized as disturbed region in which a plane section does not remain plane. Thus, in this model, the value of c is calculated using a formula that was derived using nonlinear FEA.

First, some parameters that influence the depth of the compression zone were identified. Based on flexural theory, these parameters are concrete strength f_c', vertical reinforcement area in the edge column or boundary element A_{sb}, and value of axial load P. Referring from the flexural theory for a member subjected to axial load and bending moment, it is

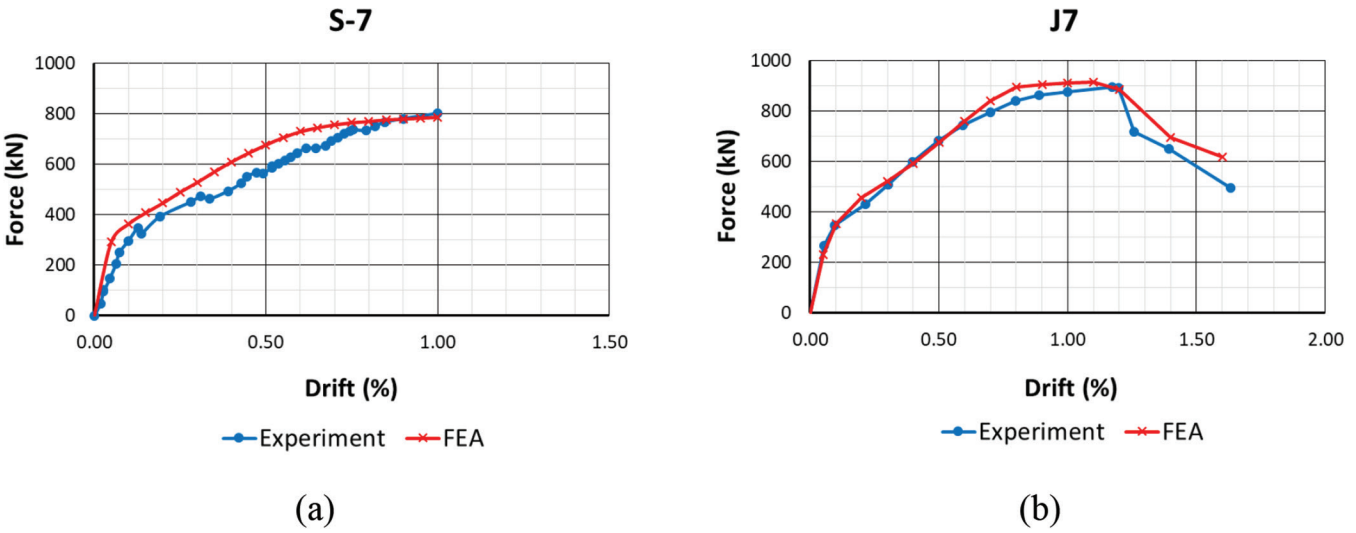

Fig. 3—Comparison of envelope curves between experimental result and finite element analysis (FEA): (a) Specimen S-7 tested by Gupta and Rangan[12]; and (b) Specimen J7 tested by Teng and Chandra.[22] (Note: 1 kN = 0.22 kip.)

clear that the value of c decreases if the value of f_c' increases. In contrast, the value of c increases if the value of A_{sb} or P increases. Moreover, the authors added shear span ratio or wall height-length ratio (H_w/L_w) as additional parameter that affects the value of c. This was because in similar cases of disturbed region—that is, deep beams—it was shown that the value of c increases if the shear span ratio decreases.[21]

Second, after identifying parameters influencing the value of c and their qualitative relationships, the following step was to determine quantitative relationships between these parameters and the value of c. The main objective was to express the value of c as a function of these parameters (f_c', A_{sb}, P, and H_w/L_w). For this objective, nonlinear FEA using ATENA software[17] was used to determine multiplication factors for each parameter. The software was used because its superior capability to perform nonlinear analysis of RC structures and it has advanced material models for concrete and steel reinforcement. In this study, RC wall specimen S-7 having H_w/L_w of 1.0 tested by Gupta and Rangan[12] and RC wall specimen J7 having H_w/L_w of 2.0 tested by Teng and Chandra[22] were used to validate the accuracy of the finite element model. The comparison of envelope curves between experimental result and FEA of those specimens is displayed in Fig. 3. It can be seen that the finite element model can predict well the force-drift relationship of those specimens.

Subsequently, a parametric study using a typical wall specimen similar to the ones tested by Teng and Chandra[22] with varying parameters mentioned earlier was done to obtain the value of c at the peak loading condition of each specimen. For concrete strength, two values were used—that is, f_c' = 50 and 100 MPa (7.25 and 14.50 ksi). For vertical reinforcement area in the edge column or boundary element, two values were used—that is, A_{sb} = 1200 and 2400 mm^2 (1.86 and 3.72 in.^{2}). For axial force, three values of axial load ratio (ALR) were used—that is, 0.0, 0.1, and 0.2. For H_w/L_w, three values were used—that is, 0.4, 1.0, and 2.0. In addition, the authors also attempted to vary the boundary element width b_f—that is, 120, 250, and 500 mm (4.72, 9.84, and 19.69 in.)—to observe the relationship between c and b_f.

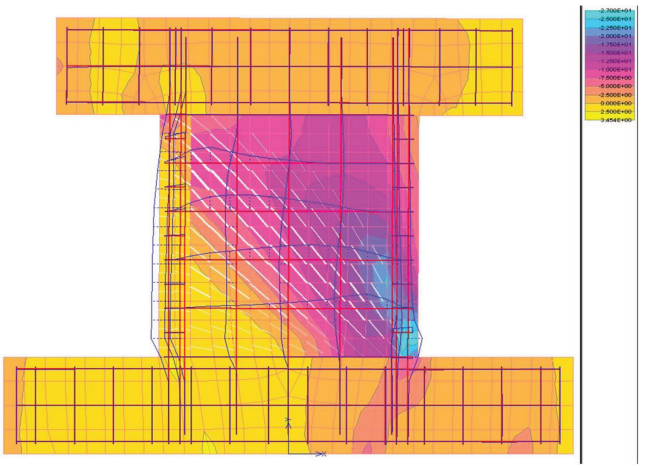

Fig. 4—State of normal stresses in vertical axis at maximum lateral load of typical wall specimen analyzed using ATENA software.[17]

An example of analysis results of a typical wall specimen is displayed in Fig. 4.

In total, 108 specimens were analyzed and the values of c obtained at the peak loading condition of each specimen were measured. These values were then plotted against varying parameters to obtain the quantitative relationships. These relationships can be seen in Fig. 5 to 7. The parameters f_c' and P are combined into one and normalized with wall web area to become $P/(f_c'A_w)$ because this is more frequently used as a parameter. From the figures, it can be seen that the value of c increases linearly with increment of $P/(f_c'A_w)$ and A_{sb}/A_w. In contrast, the value of c decreases exponentially with increment of H_w/L_w. These analysis results are consistent with qualitative relationships mentioned previously. Hence, the value of c can be expressed as follows

$$c = L_w\left(c_1 + c_2\frac{P}{f_c'A_w} + c_3\frac{A_{sb}}{A_w}\right)\left(\frac{H_w}{L_w}\right)^{c_4} \le d_w \qquad (8)$$

Equation (8) contains four constants that need to be determined. Constants c_2, c_3, and c_4 can be derived from Fig.

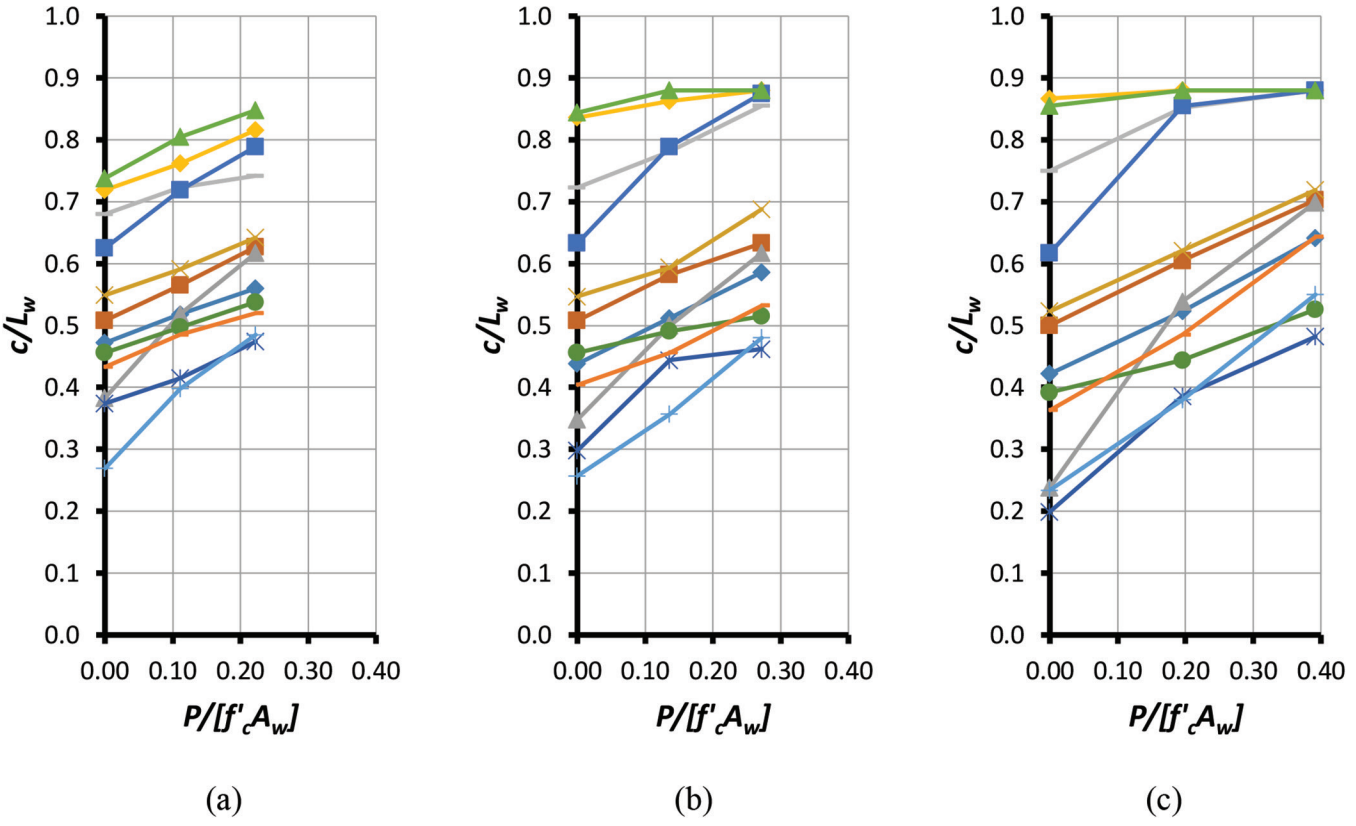

Fig. 5—Values of c/L_w *obtained from nonlinear FEA plotted against* $P/[f_c'A_w]$*: (a) cases for* b_f *= 120 mm (4.72 in.); (b) cases for* b_f *= 250 mm (9.84 in.); and (c) cases for* b_f *= 500 mm (19.69 in.).*

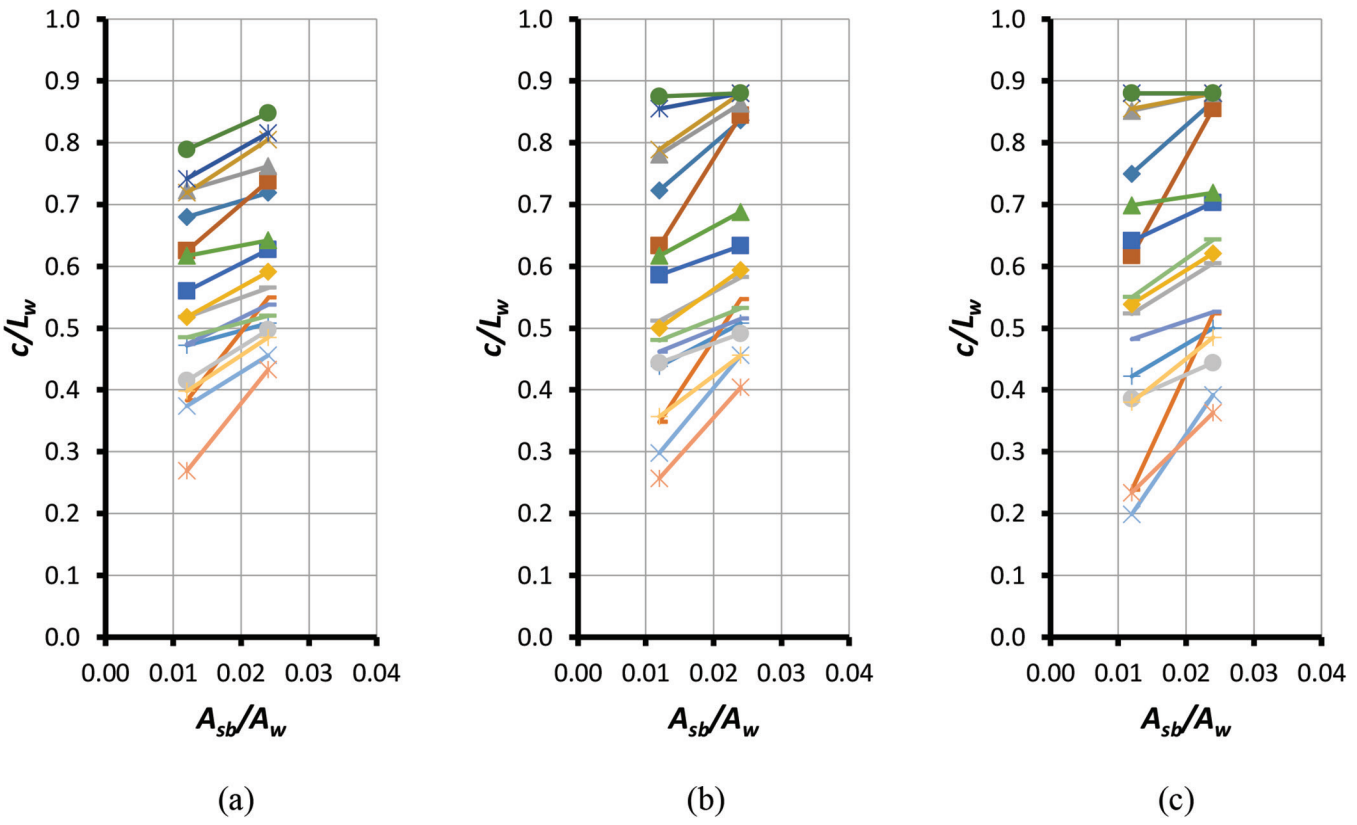

Fig. 6—Values of c/L_w *obtained from nonlinear FEA plotted against* A_{sb}/A_w*: (a) cases for* b_f *= 120 mm (4.72 in.); (b) cases for* b_f *= 250 mm (9.84 in.); and (c) cases for* b_f *= 500 mm (19.69 in.).*

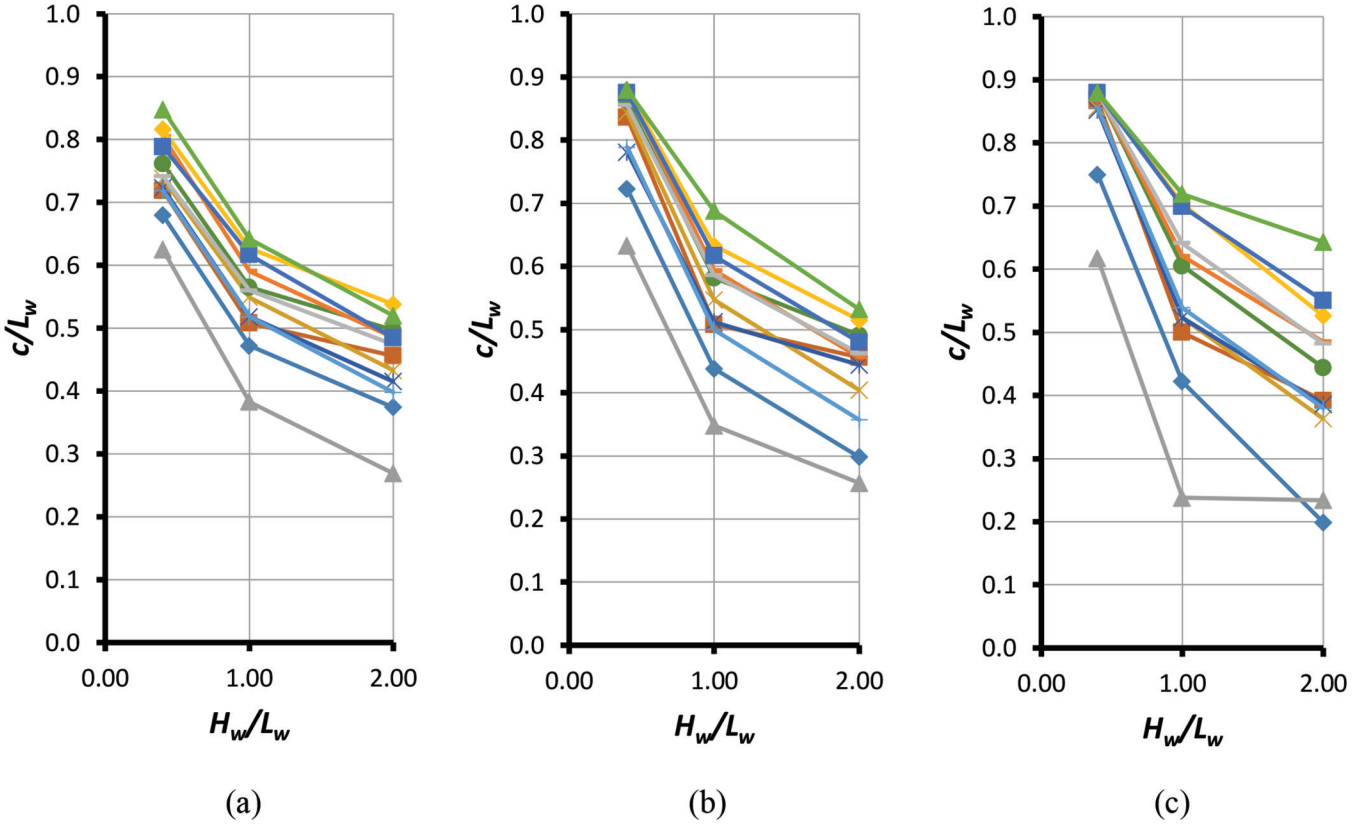

Fig. 7—Values of c/L_w *obtained from nonlinear FEA plotted against* H_w/L_w*: (a) cases for* b_f *= 120 mm (4.72 in.); (b) cases for* b_f *= 250 mm (9.84 in.); and (c) cases for* b_f *= 500 mm (19.69 in.).*

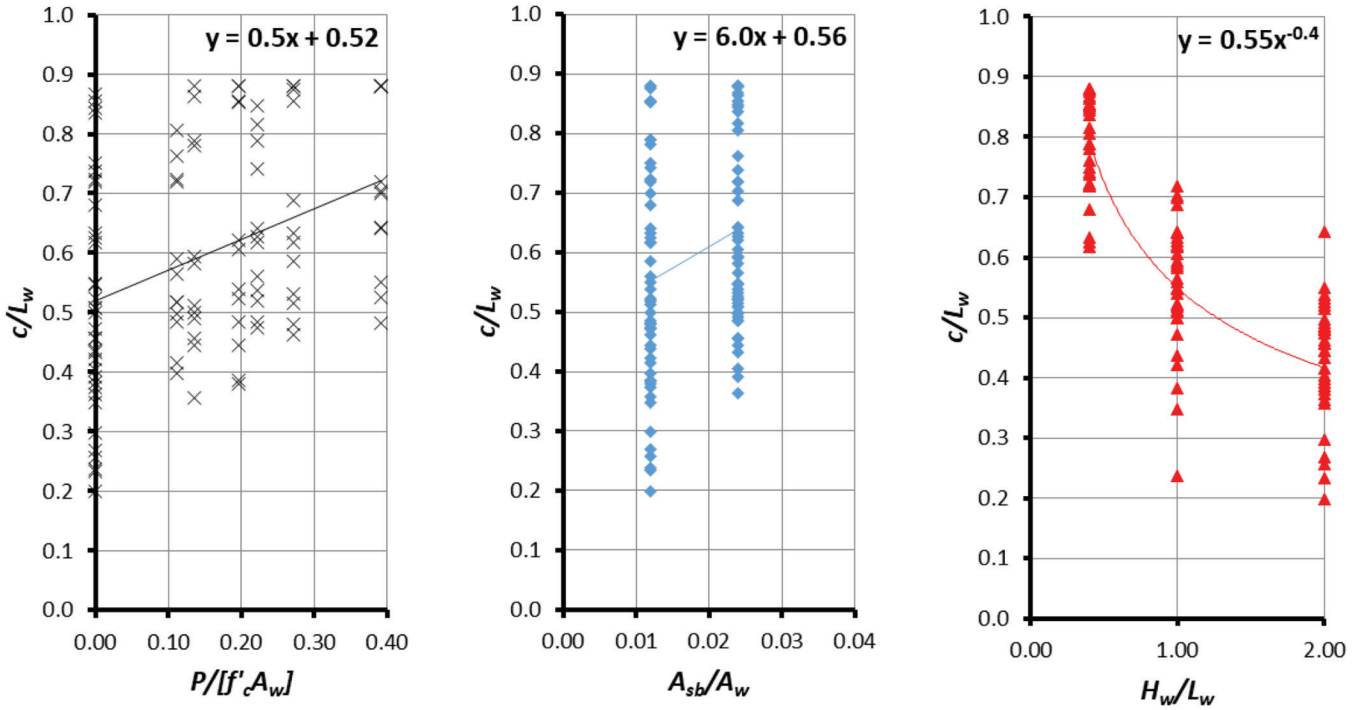

Fig. 8—Relationships between c *and varying parameters with average regression lines and their equations.*

5 to 7 by plotting regression lines for each data series. From the equations of the regression lines, the constants were obtained and then the average constant value from all data series was calculated. The average regression lines as well as the average constant values are presented in Fig. 8. From the figure, the values of c_2, c_3, and c_4 were determined as 0.5, 6.0, and –0.4, respectively. Subsequently, the value of c_1 was obtained by trial-and-error approach to achieve the most suitable values of c that were in good agreement with the values of c obtained from nonlinear FEA. Thus, the value of

c_1 was found to be 0.35. Moreover, from the nonlinear FEA, it was noted that the value of c should not be taken greater than effective depth of wall d_w. In this model, d_w is defined as the distance from center to center of the edge columns or boundary elements or it can be taken as 80% of wall length $0.8L_w$ in the case of an RC wall without edge columns or boundary elements.

Capacity of diagonal compression strut

Capacity of the diagonal compression strut D_n is a product of effective strut strength $\zeta f_c'$ and the strut area A_{str} as described by

$$D_n = \zeta f_c' A_{str} \tag{9}$$

In this model, the value of effective strut strength is taken as recommended by Eurocode 2.[23] The code considers reduction of concrete strut strength due to tensile stresses that cause cracks in the concrete strut. Moreover, in this model, increment of concrete strut strength because of confinement effect from transverse reinforcement is also considered using recommendation by FIP Commission 3.[24] Thus, the softening coefficient for strut strength ζ in this model can be described as follows

$$\zeta = 0.6\,(1 - f_c'/250) \times 0.80\,(1 + 1.6\alpha_w\omega_w) \le 0.85 \tag{10}$$

where α_w and ω_w are defined as

$$\alpha_w = 1.6\frac{s}{t_w} \le 0.4 \tag{11}$$

$$\omega_w = 4\frac{\rho f_y}{f_c'} \tag{12}$$

Because the definition of transverse reinforcement herein that provides confinement effect to the concrete strut is the one that is perpendicular to the strut axis, it is needed to represent vertical and horizontal web reinforcement of the RC low-rise wall to be the transverse reinforcement of the concrete strut as defined by FIP Commission 3.[24] Therefore, in this model, the term ρf_y is represented as

$$\rho f_y = \rho_v f_{yv} \cos\theta + \rho_h f_{yh} \sin\theta \tag{13}$$

where θ is defined as

$$\theta = \tan^{-1}\left(\frac{H'}{L_w - r - 0.5c}\right) \tag{14}$$

In this model, the value of θ is limited to $31 \le \theta \le 59$ degrees.

The strut area A_{str} is defined as a product of strut depth multiplied by strut width. Strut depth a_s is the perpendicular projection of depth of compression zone at the bottom of wall c to the strut axis as displayed in Fig. 2, while strut width can simply be taken as the thickness of wall web t_w. Finally, the nominal wall shear strength due to crushing of diagonal compression strut V_n is defined as

$$V_n = D_n \cos\theta \tag{15}$$

An example of RC wall shear strength calculation using the proposed strut-and-tie model can be seen in the Appendix.

COMPARISON WITH EXPERIMENTAL RESULTS

To examine the accuracy of the proposed strut-and-tie model, experimental wall strengths of 100 specimens collected from past experiments on RC low-rise walls failing in shear[12,19,22,25-37] were compared with calculated shear strengths from the model. Subsequently, the predictions from the proposed strut-and-tie model were also compared with predictions from building codes[4,5] and other strut-and-tie models.[15,16] The analysis results are presented in terms of ratio of the experimental shear strengths to calculated shear strengths V_{exp}/V_n. The ratio below 1.00 means that the prediction overestimates the shear strength whereas the ratio above 1.00 means that the prediction underestimates the shear strength. These results are presented in Table 1. Moreover, the ratio was also plotted against H_w/L_w (refer to Fig. 9) and f_c' (refer to Fig. 10) to observe the variation of predictions as affected by those parameters.

From the statistical parameters of V_{exp}/V_n as presented in Table 1, it can be concluded that on average, all methods are conservative in predicting the shear strength of RC low-rise walls. Hwang and Lee's model[15] has the average value V_{exp}/V_n of 1.29, which is the closest to 1.00 but overestimates the shear strength of many RC low-rise walls (approximately 22 out of 100 specimens), whereas the proposed strut-and-tie model only overestimates seven out of 100 specimens. This means Hwang and Lee's model[15] needs a lower strength reduction factor (below 0.68) as compared to the proposed model (approximately 0.83) to ensure safe predictions of RC low-rise wall shear strengths. The proposed model has the average value V_{exp}/V_n of 1.35, which is the second closest to 1.00 and it has the lowest coefficient of variation (COV) of 0.19 as compared to other methods. This is definitely an advantage of the proposed model over other methods. Furthermore, as can be seen in Table 1, Eurocode 8[5] is indeed the most conservative method with average value V_{exp}/V_n of 2.13 and the code underestimates the shear strength of all 100 specimens collected in this study. Moreover, Kassem's model[16] has the highest COV of 0.37 with the average value V_{exp}/V_n of 1.40 while ACI 318[4] has similar average value V_{exp}/V_n (1.41) and slightly lower COV (0.35).

From Fig. 9 and 10, it can be seen that the predictions of the proposed strut-and-tie model are quite consistent and less scattered for various ranges of H_w/L_w and f_c', as compared to the predictions by other methods. From Fig. 9, it can be seen that ACI 318-19[4] is more conservative for walls with lower H_w/L_w while it is the opposite for Hwang and Lee's model[15] and Kassem's model.[16] From Fig. 10, except for Kassem's model,[16] it can be seen that the predictions of most methods are closer to 1.00 for walls with f_c' below 60 MPa (8.70 ksi) and they are more conservative for walls with f_c' above 60 MPa (8.70 ksi). In addition, the predictions of Eurocode 8[5] are quite scattered for various ranges of H_w/L_w and f_c', and there is no clear trend that can be observed from these figures.

Table 1—Ratio of experimental and calculated wall shear strengths

No.	Specimen ID	f_c', MPa	$P/(f_c'A_g)$	H_w/L_w	$\rho_v f_{yv}$, MPa	$\rho_h f_{yh}$, MPa	V_{exp}/V_n				
							ACI 318-19[4]	EC8[5]	Hwang-Lee[15]	Kassem[16]	Proposed model
Hirosawa[25]											
1	72	17	0.12	0.94	2.07	1.09	1.33	1.71	1.13	1.79	1.42
2	73	21	0.10	0.94	2.07	1.09	1.28	1.68	1.00	1.61	1.28
3	74	21	0.10	0.94	2.07	2.40	0.82	1.45	1.01	1.60	1.17
4	75	14	0.15	0.94	2.07	2.40	0.97	2.09	1.39	2.07	1.47
5	76	15	0.14	0.94	2.07	4.47	0.92	1.94	1.30	1.88	1.16
6	77	18	0.11	0.94	2.07	4.47	0.91	1.78	1.23	1.84	1.18
7	79	14	0.15	0.94	2.07	2.57	0.71	1.52	1.01	1.50	1.09
8	82	21	0.10	1.88	1.63	2.40	0.72	1.22	0.95	1.66	1.20
9	83	18	0.11	1.88	1.63	2.40	0.70	1.26	1.02	1.73	1.25
Barda et al.[26]											
10	B1-1	29	0.00	0.46	2.72	2.48	1.65	3.94	1.23	0.99	1.52
11	B2-1	16	0.00	0.46	2.76	2.50	1.51	3.45	1.72	1.06	1.39
12	B3-2	27	0.00	0.46	2.72	2.56	1.48	3.23	1.18	0.93	1.29
13	B6-4	21	0.00	0.46	1.24	2.48	1.25	2.72	1.39	1.22	1.33
14	B7-5	26	0.00	0.21	2.65	2.51	1.56	4.64	1.09	0.98	1.11
15	B8-5	23	0.00	0.96	2.64	2.48	1.24	2.24	1.82	1.02	1.57
Cardenas et al.[19]											
16	SW-7	43	0.00	1.00	3.44	1.12	1.30	2.06	0.88	1.45	1.03
17	SW-8	42	0.00	1.00	13.45	1.26	1.36	2.02	0.97	1.28	0.96
Corley et al.[27]											
18	B2	54	0.00	2.40	1.54	3.35	0.76	1.31	1.04	0.92	1.04
19	B5	45	0.00	2.40	1.46	3.16	0.91	1.56	1.27	1.14	1.30
20	B6	22	0.14	2.40	1.48	3.22	1.10	1.96	1.56	1.28	1.78
21	B7	49	0.08	2.40	1.42	3.08	1.18	2.05	1.11	1.07	1.40
22	B8	42	0.09	2.40	1.32	6.65	0.94	1.38	1.13	1.00	1.31
23	B9	44	0.09	2.40	1.34	2.91	1.25	2.17	1.12	1.12	1.49
24	B10	46	0.09	2.40	1.35	2.92	0.90	1.56	0.81	0.80	1.17
25	F1	38	0.00	2.40	1.58	3.73	0.90	1.45	1.41	1.20	1.51
26	F2	46	0.08	2.40	1.44	2.92	1.13	1.96	0.91	0.91	1.28
Maeda[28]											
27	MAE03	58	0.03	0.55	3.83	3.83	1.46	2.82	1.02	0.96	1.09
28	MAE07	58	0.03	0.55	6.42	6.42	1.52	2.38	1.10	1.10	1.11
Okamoto[29]											
29	W48M6	82	0.02	0.74	4.44	4.44	1.10	1.99	0.88	0.70	1.16
30	W48M4	82	0.02	0.74	4.12	4.12	1.12	1.97	0.86	0.68	1.14
31	W72M8	82	0.02	0.74	7.24	7.24	1.33	1.89	1.20	0.95	1.41
32	W72M6	82	0.02	0.74	6.65	6.65	1.30	1.93	1.17	0.93	1.38
33	W72M8	102	0.02	0.74	7.24	7.24	1.23	1.93	1.14	0.88	1.40
34	W96M8	102	0.02	0.74	9.41	9.41	1.44	2.04	1.33	1.03	1.49
Gupta and Rangan[12]											
35	S-1	79	0.00	1.00	5.45	2.89	1.11	1.58	0.99	0.61	1.03
36	S-2	65	0.07	1.00	5.45	2.89	1.96	2.24	1.32	1.06	1.55
37	S-3	69	0.13	1.00	5.45	2.89	2.28	2.28	1.23	1.21	1.53
38	S-4	75	0.00	1.00	8.00	2.89	1.58	2.16	1.43	0.82	1.32
39	S-5	73	0.06	1.00	8.00	2.89	2.10	2.43	1.42	1.10	1.49

Table 1 (cont.)—Ratio of experimental and calculated wall shear strengths

No.	Specimen ID	f_c', MPa	$P/(f_c'A_g)$	H_w/L_w	$\rho_v f_{yv}$, MPa	$\rho_h f_{yh}$, MPa	V_{exp}/V_n				
							ACI 318-19[4]	EC8[5]	Hwang-Lee[15]	Kassem[16]	Proposed model
40	S-6	71	0.13	1.00	8.00	2.89	2.59	2.60	1.40	1.37	1.62
41	S-7	71	0.06	1.00	5.45	5.45	1.52	2.05	1.41	1.12	1.56
Kabeyasawa and Hiraishi[30]											
42	W-08	103	0.09	1.18	5.75	5.75	1.48	1.93	1.35	1.10	1.89
43	W-12	138	0.09	1.18	5.75	5.75	1.46	1.95	1.21	1.02	1.99
44	No. 1	65	0.13	1.18	1.58	1.58	2.25	2.19	1.11	0.91	1.48
45	No. 2	71	0.12	1.18	2.75	2.75	1.90	1.93	1.18	0.99	1.55
46	No. 3	72	0.12	1.18	4.22	4.22	1.60	1.84	1.23	1.08	1.59
47	No. 4	103	0.14	1.18	4.22	4.22	1.84	1.88	1.22	1.11	1.70
48	No. 5	77	0.11	1.76	4.22	4.22	1.41	1.50	1.07	0.94	1.55
49	No. 6	74	0.12	1.18	9.31	9.31	1.45	1.86	1.26	1.09	1.34
50	No. 7	72	0.12	1.18	7.92	7.92	1.57	2.01	1.34	1.18	1.50
51	No. 8	76	0.11	1.18	11.52	11.52	1.66	2.13	1.45	1.25	1.45
Farvashany et al.[31]											
52	HSCW1	104	0.04	1.25	6.74	2.51	2.20	2.36	1.56	1.00	1.62
53	HSCW2	93	0.09	1.25	6.74	2.51	2.60	2.48	1.60	1.18	1.78
54	HSCW3	86	0.09	1.25	4.01	2.51	1.96	1.85	1.19	0.91	1.38
55	HSCW4	91	0.22	1.25	4.01	2.51	2.68	1.99	1.13	1.23	1.56
56	HSCW5	84	0.09	1.25	6.74	4.01	1.93	2.07	1.42	1.18	1.66
57	HSCW6	90	0.05	1.25	6.74	4.01	1.77	1.94	1.49	1.06	1.63
58	HSCW7	102	0.08	1.25	4.01	4.01	1.85	1.94	1.39	1.08	1.67
Burgueno et al.[32]											
59	M05C	46	0.08	2.25	6.54	8.14	1.85	2.68	2.46	1.68	1.62
60	M05M	39	0.09	2.25	6.54	8.14	2.14	3.23	2.76	1.89	1.81
61	M10C	56	0.06	2.25	7.00	8.71	1.56	2.19	2.22	1.46	1.39
62	M10M	84	0.04	2.25	7.00	8.71	1.53	2.09	2.43	1.62	1.51
63	M15C	102	0.03	2.25	7.07	8.80	1.27	1.77	2.09	1.45	1.37
64	M15M	111	0.03	2.25	7.03	8.75	1.38	1.98	2.33	1.65	1.54
65	M20C	131	0.03	2.25	6.44	10.69	1.11	1.72	1.92	1.43	1.35
66	M20M	115	0.03	2.25	6.44	10.69	1.34	1.95	2.27	1.59	1.49
Cheng et al.[33]											
67	M60	39	0.00	0.94	1.39	1.39	0.92	1.76	0.69	1.24	0.93
68	M115	38	0.00	0.94	1.21	2.41	0.68	1.14	0.68	1.23	0.83
69	H60	44	0.00	0.94	3.89	3.89	0.87	1.37	1.08	1.81	1.12
70	H115	44	0.00	0.94	3.30	3.30	0.88	1.39	0.99	1.70	1.13
71	H60X	42	0.00	0.94	3.89	3.89	0.88	1.41	1.10	1.85	1.14
Teng and Chandra[22]											
72	J1	103	0.05	1.00	1.71	1.71	2.85	3.25	1.62	1.29	1.65
73	J2	97	0.05	1.00	4.34	1.71	3.05	3.48	1.75	1.20	1.71
74	J3	111	0.05	1.00	1.71	4.34	2.09	2.36	1.71	1.51	1.87
75	J4	94	0.05	1.00	1.71	1.71	1.97	2.35	1.44	2.06	1.19
76	J5	103	0.05	2.00	1.71	1.71	1.73	4.36	1.07	0.88	1.07
77	J6	97	0.05	2.00	4.34	1.71	2.14	5.30	1.33	0.98	1.29
78	J7	111	0.05	2.00	1.71	4.34	1.46	2.58	1.23	1.24	1.52

Table 1 (cont.)—Ratio of experimental and calculated wall shear strengths

No.	Specimen ID	f_c', MPa	$P/(f_c'A_g)$	H_w/L_w	$\rho_v f_{yv}$, MPa	$\rho_h f_{yh}$, MPa	V_{exp}/V_n				
							ACI 318-19[4]	EC8[5]	Hwang-Lee[15]	Kassem[16]	Proposed model
Baek et al.[34]											
79	NS2	37	0.07	2.00	5.17	4.37	1.34	2.10	1.98	3.36	1.72
80	HS2	37	0.07	2.00	3.74	4.54	1.30	2.04	1.93	3.31	1.72
81	NS2L	37	0.07	2.00	3.10	2.16	1.40	2.57	1.31	2.46	1.43
82	HS2L	37	0.07	2.00	2.80	2.27	1.45	2.63	1.41	2.64	1.54
Baek et al.[35]											
83	NS1M	53	0.07	1.00	5.17	4.37	1.26	1.77	1.37	2.21	1.31
84	HS1M	53	0.07	1.00	4.67	4.54	1.17	1.64	1.28	2.06	1.22
85	NS0.5M	45	0.07	0.50	4.32	4.37	1.50	2.33	1.19	2.55	1.25
86	HS0.5M	37	0.07	0.50	3.87	4.54	1.54	2.40	1.29	2.74	1.29
Baek et al.[36]											
87	SW1	20	0.00	2.50	0.82	1.14	0.91	1.90	1.36	2.25	1.30
88	SW2	20	0.25	2.50	0.82	1.14	1.14	2.36	0.92	1.62	1.28
89	SW3	20	0.00	2.50	0.97	1.08	0.90	1.90	1.30	2.15	1.21
90	SW4	20	0.25	2.50	0.97	1.08	1.17	2.48	0.92	1.63	1.26
91	SW5	37	0.14	2.50	0.82	1.14	1.20	2.86	0.79	1.42	1.06
92	SW6	37	0.14	2.50	0.97	1.08	1.21	2.95	0.79	1.41	1.02
Hube et al.[37]											
93	WSL1	29	0.00	1.00	1.21	1.21	0.85	1.31	0.77	1.29	0.87
94	WSL3	29	0.00	1.00	1.56	1.56	1.01	1.50	1.02	1.70	1.13
95	WSL4	29	0.00	1.00	0.88	0.88	1.13	1.80	0.92	1.51	1.03
96	WSL5	29	0.00	1.00	0.89	0.89	1.00	1.58	0.82	1.34	0.91
97	WSL6	29	0.00	1.00	0.62	0.62	1.12	1.86	0.84	1.34	0.92
98	WSL7	29	0.00	1.00	1.51	1.51	0.89	1.34	0.87	1.46	1.00
99	WSL8	29	0.00	1.00	1.07	1.07	0.94	1.49	0.80	1.34	0.93
100	WSL9	29	0.00	1.00	1.12	1.12	1.07	1.67	0.92	1.54	1.07
Statistical parameters											
Minimum value							0.68	1.14	0.68	0.61	0.83
Maximum value							3.05	5.30	2.76	3.36	1.99
Average value							1.41	2.13	1.29	1.40	1.35
Standard deviation							0.50	0.70	0.41	0.52	0.25
Coefficient of variation							0.35	0.33	0.32	0.37	0.19

CONCLUSIONS

The authors have developed an analytical method based on the strut-and-tie concept to calculate the shear strength of reinforced concrete (RC) low-rise walls. The following conclusions can be made:

1. The proposed strut-and-tie model was verified with a total of 100 RC low-rise walls (wall height-length ratio [H_w/L_w] less than 2.5) failing in shear that were selected from available literature.[12,19,22,25-37] The analysis results show that the model is conservative in predicting the shear strength of RC low-rise walls with an average value of the ratio of the experimental shear strengths to calculated shear strengths V_{exp}/V_n of 1.35. While Hwang and Lee's model[15] has the average value V_{exp}/V_n of 1.29, which is the closest to 1.00, it overestimates the shear strength of 22 specimens whereas the proposed model only overestimates seven specimens. It is generally known that a strut-and-tie model serves as a lower-bound theory.[14]

2. As compared to building codes[4,5] and other strut-and-tie models,[15,16] the proposed strut-and-tie model has the lowest coefficient of variation (0.19) in predicting the shear strength of RC low-rise walls. This is clearly an advantage of the proposed model over other methods. In addition, the predictions of the proposed model are also quite consistent and less scattered for wide ranges of H_w/L_w and concrete compressive strengths.

AUTHOR BIOS

Jimmy Chandra *is an Assistant Professor and currently Head of Structural Engineering Laboratory at Petra Christian University, Indonesia. He received his Doctor of Philosophy degree from Nanyang Technological University, Singapore. His research interests include behavior and seismic performance evaluation of reinforced concrete structures.*

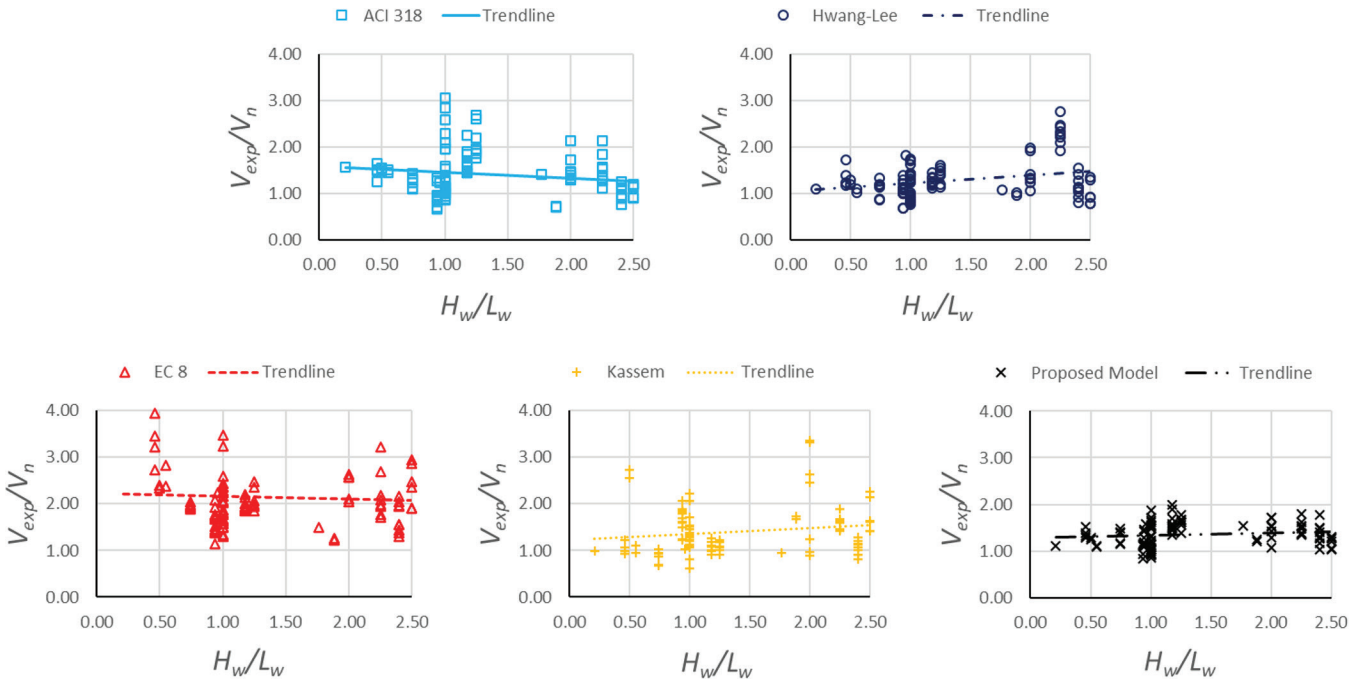

Fig. 9—V_{exp}/V_n *plotted against* H_w/L_w.

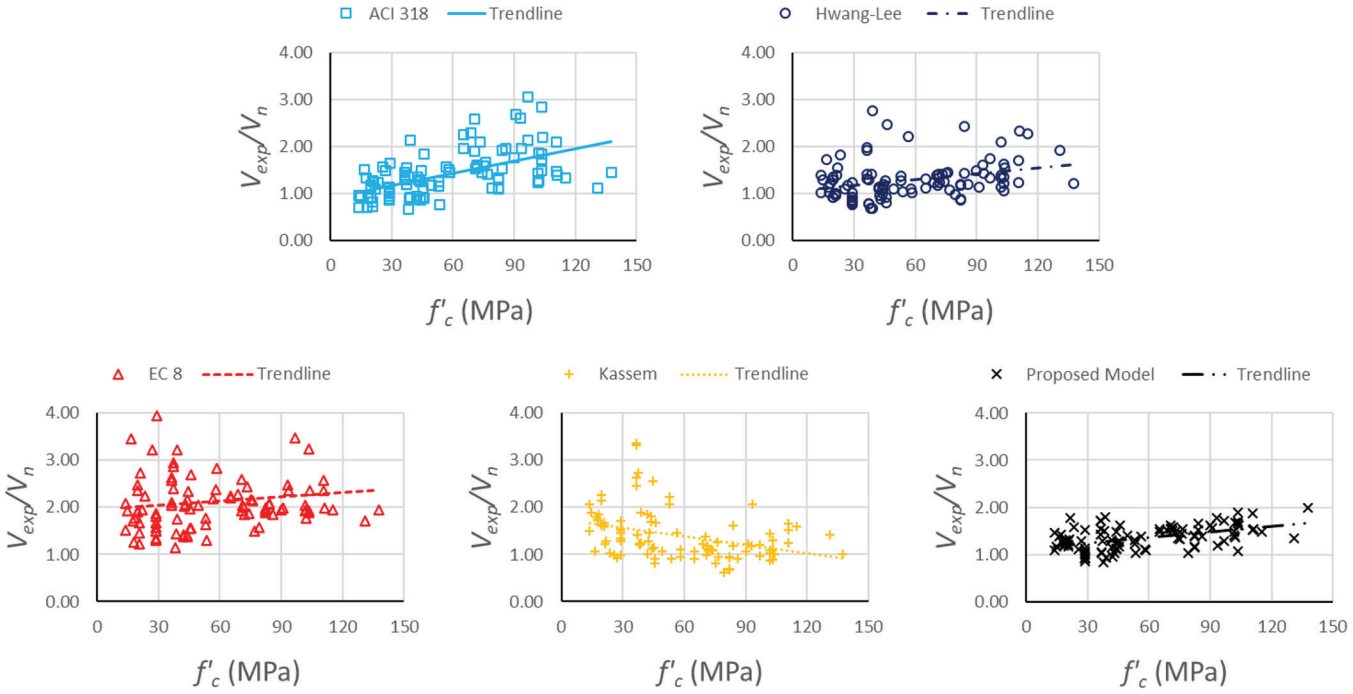

Fig. 10—V_{exp}/V_n *plotted against* f_c'. *(Note: 1 MPa = 145.04 psi).*

ACI member **Susanto Teng** *is an Associate Professor at Nanyang Technological University. He is a member of ACI Committee 435, Deflection of Concrete Building Structures; and Joint ACI-ASCE Committees 421, Design of Reinforced Concrete Slabs, and 445, Shear and Torsion. His research interests include behavior of structural concrete walls, shear strength of slabs, size effect in shear behavior of concrete members, computational modeling of concrete structures, and durability of marine concrete structures.*

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NOTATION

A_{cv}	=	gross area of concrete section bounded by web thickness and length of section in direction of shear force considered
A_{cw}	=	area of concrete section of individual vertical wall segment considered
A_g	=	wall gross cross-sectional area
A_{sb}	=	total area of vertical reinforcement in one boundary element
A_{str}	=	area of diagonal concrete strut
A_{sw}	=	cross-sectional area of shear reinforcement
A_w	=	wall web area
a_s	=	depth of diagonal concrete strut
b_f	=	width of boundary element

b_w = minimum width (thickness) of wall between tension and compression chords
b_{wo} = width of wall web
C = compression force in compression zone
C_d = diagonal compression force acting on nodal zone
$C_{d,n}$ = nominal capacity of nodal zone
c = depth of compression zone at bottom of wall
D = compression force in diagonal strut
D_n = nominal strength of diagonal concrete strut
d_w = effective depth of wall
F_h = tension force in horizontal tie
F_v = tension force in vertical tie
f_c' = concrete cylinder compressive strength
f_{cd} = design value of concrete compressive strength
f_{ck} = characteristic compressive cylinder strength of concrete at 28 days
f_y = specified yield strength of reinforcement.
f_{yb} = yield strength of vertical reinforcement in boundary element
$f_{yd,h}$ = design value of yield strength of horizontal web reinforcement
f_{yh} = yield strength of horizontal shear reinforcement
f_{yt} = yield strength of transverse reinforcement.
f_{yv} = yield strength of vertical shear reinforcement
f_{ywd} = design yield strength of shear reinforcement
H_w = height of wall
H' = distance measured from point of application of external shear force to wall base
K = strut-and-tie index
k_s = ratio of depth of compression zone at wall base to effective depth of wall
L_w = wall length
M_{Ed} = design bending moment at base of wall
P = axial load applied at top of wall
R = resultant force of external axial force and tension force in tension tie
R_d = wall shear ratio resisted by diagonal mechanism
R_h = wall shear ratio resisted by horizontal mechanism
R_v = wall shear ratio resisted by vertical mechanism
r = distance measured from point of application of resultant force to nearest wall edge
s = spacing of shear (web) reinforcement
T = tension force in tension tie
t_f = thickness of boundary element
t_w = thickness of wall web
V = applied external shear force
V_c = concrete contribution to overall shear strength
V_{Ed} = design shear force
V_{exp} = experimental wall shear strength
V_n = nominal shear strength of RC wall
V_{Rd} = shear resistance of a member with shear reinforcement
$V_{Rd,c}$ = design shear resistance of a member without shear reinforcement
$V_{Rd,max}$ = design value of maximum shear force which can be sustained by member
$V_{Rd,s}$ = design value of shear force which can be sustained by the yielding shear reinforcement
z = inner lever arm, which is taken as 0.8 L_w (L_w is wall length)
α_c = coefficient defining relative contribution of concrete strength to nominal wall shear strength which may be taken as 0.25 for $H_w/L_w \le 1.5$, 0.17 for $H_w/L_w \ge 2.0$, and varies linearly between 0.25 and 0.17 for H_w/L_w between 1.5 and 2.0; where H_w/L_w is height-to-length ratio of wall
α_{cw} = coefficient taking account of state of stress in compression chord
α_w = coefficient taking account of confinement effect of web reinforcement to concrete strut strength, related to spacing of web reinforcement
λ = modification factor reflecting reduced mechanical properties of lightweight concrete, all relative to normal weight concrete of same compressive strength
θ = angle between concrete compression strut and wall axis perpendicular to shear force (Eurocode 8)
θ = angle of diagonal compression strut with respect to horizontal axis (Hwang-Lee's model, Kassem's model, and proposed model)
ν_1 = strength reduction factor for concrete cracked in shear
ρ = reinforcement ratio
ρ_b = ratio of vertical reinforcement in boundary element
ρ_h = average horizontal web reinforcement ratio
ρ_t = ratio of area of distributed transverse (horizontal) shear reinforcement to gross concrete area perpendicular to that reinforcement
ρ_v = average vertical web reinforcement ratio
σ_{cp} = mean compressive stress, measured positive, in concrete due to design axial force
ω_h = horizontal web reinforcement index which can be defined as [$\rho_h f_{yh}/f_c'$]
ω_v = vertical web reinforcement index which can be defined as [$\rho_v f_{yv}/f_c'$]
ω_w = coefficient taking account of confinement effect of web reinforcement to concrete strut strength, related to ratio of web reinforcement
ψ = a non-dimensional function which can be defined as [0.95 − $f_c'/250$] (f_c' in MPa)
ζ = softening coefficient of concrete in compression

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APPENDIX

An example of RC wall shear strength calculation using the authors' proposed strut-and-tie model is given herein. A specimen taken from Teng and Chandra[22] is used—that is, Specimen J5. The procedure is given as follows (in SI units):

Specimen J5 data:

Concrete compressive strength, $f_c' = 103.3$ MPa
Wall gross cross-sectional area, $A_g = 196{,}000$ mm^2
Axial load applied at top of wall, $P = 1012$ kN (compression)
Wall height, $H_w = 2000$ mm
Wall length, $L_w = 1000$ mm
Thickness of wall web, $t_w = 100$ mm
Width of boundary element, $b_f = 500$ mm
Thickness of boundary element, $t_f = 120$ mm
Ratio of vertical reinforcement in boundary element, $\rho_b = 0.0388$
Yield strength of vertical reinforcement in boundary element, $f_{yb} = 630$ MPa
Ratio of vertical shear (web) reinforcement in wall, $\rho_v = 0.0028$
Yield strength of vertical shear reinforcement, $f_{yv} = 610$ MPa
Ratio of horizontal shear (web) reinforcement in wall, $\rho_h = 0.0028$
Yield strength of horizontal shear reinforcement, $f_{yh} = 610$ MPa
Experimental wall shear strength, $V_{exp} = 595.76$ kN

Calculation of nominal shear strength (V_n) according to proposed strut-and-tie model:

1. Calculate c using Eq. (8) and the corresponding A_{str}

$$c = L_w\left(0.35 + 0.5\frac{P}{f_c'A_w} + 6\frac{A_{sb}}{A_w}\right)\left(\frac{H_w}{L_w}\right)^{-0.4} \le d_w$$

$$c = 1000\left(0.35 + 0.5\frac{1{,}012{,}000}{103.3\times 100{,}000} + 6\frac{2328}{100{,}000}\right)\left(\frac{2000}{1000}\right)^{-0.4}$$

$$c = 408.23 \text{ mm} \le 880 \text{ mm (OK)}$$

Calculating T assuming yielding of reinforcement:
T_1 from vertical reinforcement in boundary element

$$T_1 = \rho_b \times b_f \times t_f \times f_{yb}$$

$$T_1 = 0.0388 \times 500 \times 120 \times 630$$

$$T_1 = 1466.64 \text{ kN}$$

T_2 from vertical web reinforcement that is in tension

$$T_2 = \rho_v \times (L_w - c - t_f) \times t_w \times f_{yv}$$

$$T_2 = 0.0028 \times (1000 - 408.23 - 120) \times 100 \times 610$$

$$T_2 = 80.58 \text{ kN}$$

Calculating r by taking wall edge in tension as reference point

$$r = \frac{T_1 \times \text{arm}_1 + T_2 \times \text{arm}_2 + P \times 0.5L_w}{T_1 + T_2 + P}$$

$$r = \frac{1466.64\times 0.5\times 120 + 80.58\times\left[120 + 0.5\times(1000 - 408.23 - 120)\right] + 1012\times 500}{1466.64 + 80.58 + 1012}$$

$$r = 243.31 \text{ mm}$$

Calculate θ using Eq. (14)

$$\theta = \tan^{-1}\left(\frac{H'}{L_w - r - 0.5c}\right)$$

$$\theta = \tan^{-1}\left(\frac{2200}{1000 - 243.31 - 0.5 \times 408.23}\right)$$

$$\theta = 75.9 \text{ degrees}$$

Then, take $\theta = 59$ degrees
Calculating A_{str}

$$A_{str} = a_s \times t_w$$

$$A_{str} = c \times \sin\theta \times t_w$$

$$A_{str} = 408.23 \times \sin 59° \times 100$$

$$A_{str} = 34{,}992.14 \text{ mm}^2$$

2. Calculate ζ using Eq. (10):
Calculating α_w using Eq. (11)

$$\alpha_w = 1.6\frac{s}{t_w} \le 0.4$$

$$\alpha_w = 1.6\frac{200}{100}$$

$$\alpha_w = 3.2$$

Then, take $\alpha_w = 0.4$
Calculating ω_w using Eq. (12)

$$\rho f_y = \rho_v f_{yv} \cos\theta + \rho_h f_{yh} \sin\theta$$

$$\rho f_y = 0.0028 \times 610 \times \cos 59° + 0.0028 \times 610 \times \sin 59°$$

$$\rho f_y = 2.34$$

$$\omega_w = 4\frac{\rho f_y}{f_c'}$$

$$\omega_w = 4\frac{2.34}{103.3}$$

$$\omega_w = 0.09$$

$$\zeta = 0.6\,(1 - f_c'/250) \times 0.80\,(1 + 1.6\alpha_w\omega_w) \le 0.85$$

$$\zeta = 0.6\,(1 - 103.3/250) \times 0.80\,(1 + 1.6 \times 0.4 \times 0.09)$$

$$\zeta = 0.30$$

3. Calculate D_n using Eq. (9)

$$D_n = \zeta f_c' A_{str}$$

$$D_n = 0.30 \times 103.3 \times 34{,}992.14$$

$$D_n = 1077.18 \text{ kN}$$

4. Calculate V_n using Eq. (15)

$$V_n = D_n \cos\theta$$

$$V_n = 1077.18 \times \cos 59°$$

$$V_n = 554.79 \text{ kN}$$

Thus, $V_{exp}/V_n = 595.76/554.79 = 1.07$.