13TH CANADIAN MASONRY SYMPOSIUM
HALIFAX, CANADA
JUNE 4TH – JUNE 7TH 2017

REEVALUATION OF THE CURRENT NORTH AMERICAN SHEAR STRENGTH EQUATIONS

Dillon, Patrick B.[1] and Fonseca, Fernando S.[2]

ABSTRACT

The development of the TMS and CSA masonry shear strength equations is not well documented in the masonry literature. The TMS and CSA shear strength equations are based on an empirical model developed by the Technical Coordinating Committee on Masonry Research (TCCMaR). The TCCMaR model was based on two models: one developed by Blondet et al. and another developed by Anderson and Priestley. An analysis of these models has been conducted to replicate the original studies so as to better understand the development of the models and their potential limitations, which is an important consideration in evaluating whether changes to the TMS or CSA shear strength equations are warranted. The analysis conducted and presented herein confirmed that least squares regression was not used to determine the optimum coefficient values for the two models, leading to errors being introduced into the TCCMaR model coefficients. In addition, the form of the TCCMaR model is not the most representative form for masonry shear strength, which increases the errors in the model predictions. Furthermore, the TCCMaR model was neither designed nor analyzed for use with partially grouted walls. The combination of these factors increases the error, which results in model predictions having larger variability and uncertainty than they need to, particularly for partially grouted walls. All of the implicit errors in the TCCMaR model were carried over into the TMS and CSA shear strength equations, and it is not possible to mitigate all of the errors in the TMS and CSA equations by simply inserting modification factors or changing their coefficient values. The authors recommend that a new masonry shear strength model be developed to replace the TMS and CSA models.

KEYWORDS: *regression, reinforced masonry, shear strength, strength prediction modeling*

[1] Staff Engineer II, WDP & Associates Consulting Engineers, Inc., 335 Greenbrier Dr., Suite 205, Charlottesville, VA, 22901, USA, pdillon@wdpa.com

[2] Associate Professor, Department of Civil and Environmental Engineering, Brigham Young University, Provo, UT, 84602, USA, fonseca@byu.edu

INTRODUCTION

It has been shown that the shear strengths calculated using the TMS [1] and CSA [2] shear strength equations are more variable for partially grouted walls than for fully grouted walls [3]. The TMS and CSA shear strength equations are based on an empirical model developed by the Technical Coordinating Committee on Masonry Research (TCCMaR) [4]. The TCCMaR model was based on two models: one developed by Blondet et al. [5] and another by Anderson and Priestley [6]. These two models were developed based on experimental investigations that were conducted on fully grouted masonry panels during the 1970s and 80s. Much research has been conducted since that time, particularly involving partially grouted walls, which is not considered in the current shear strength equations.

The means by which empirical models are developed will affect how well the model predictions represent actual conditions. The development of the TMS, CSA, Blondet, and Anderson shear models is not well documented in the masonry literature. This article presents a study of the Blondet and Anderson models that was conducted to replicate the results of the original studies to better understand the development and potential limitations of the models. This understanding is an important consideration in evaluating whether the development of new models should be pursued.

BACKGROUND

Empirical models are equations that are be used to predict the behavior and performance of different materials. Empirical models are typically cast into an equation form that is more simple and usable than one purely based on mechanical theory. In many instances, a closed-form representation of mechanical theory is not possible, and a solution must be determined using numerical methods.

The development of empirical models typically has two phases: determination of the equation form and selection of coefficients. The equation form may be purely empirically based, developed from mechanical theory, or a combination of both. Once an equation form is selected for a model, the model coefficients can be determined from analysis of experimental data. Typically, multiple equation forms will be analyzed and compared using goodness-of-fit statistics to select the final form and coefficients that minimize the uncertainty of the predictions.

The use of empirical models, by nature, introduces modeling error into the predictions that would not be present in a purely mechanical theory-based model. Modeling error acts independently of and in addition to material uncertainty, increasing the total uncertainty of the predictions and the probability of obtaining unconservative predictions. The modeling error is influenced by both the selected equation form and by the selected coefficients.

For linear models, those which can be reduced to a linear combination of coefficients, the coefficients can be determined explicitly using least-squares regression. Additional methods exist for other, more complex models, but this study was limited to the former case. For any given linear model, the coefficients determined from least-squares regression minimize the modeling error. The use of coefficient values other than the optima will increase both the modeling error and the variance of the predictions.

In practice, the optimum coefficient values are not always used. The most common reason is due to rounding the coefficients to make the equation more presentable to users. Another reason may be that the coefficients were not determined explicitly. It is important to note that model coefficients are highly indeterminant. Changing one coefficient from its optimum value will not only increase the modeling error but will also change the optimum values of the other coefficients, which then must be redetermined. Furthermore, the increase in modeling error is more sensitive to changing some coefficient values than others. Final selection of the coefficients should be made by comparing the goodness-of-fit statistics from several possible combinations of coefficient values.

The TMS and CSA shear strength equation are both linear models that can be represented as linear combinations of the coefficients. The TMS shear equation can be represented by the linear form

$$V_n = \beta_1 \gamma_g \sqrt{f'_m} A_{nv} + \beta_2 \frac{M_u}{V_u d_v} \gamma_g \sqrt{f'_m} A_{nv} + \beta_3 \gamma_g P_u + \beta_4 \gamma_g \left(\frac{A_v}{s}\right) f_y d_v \tag{1}$$

where the SI coefficients are given by $\beta_1 = 0.33, \beta_2 = -0.145$, $\beta_3 = 0.25$, and $\beta_4 = 0.5$. The CSA shear equation can be represented by the linear form

$$V_r = \beta_1 \phi_m \gamma_g \sqrt{f'_m} \mathrm{b_w d_v} + \beta_2 \phi_m \gamma_g \frac{M_u}{V_f d_v} \sqrt{f'_m} \mathrm{b_w d_v} + \beta_3 \phi_m P_d + \beta_4 \phi_s A_v f_y \left(\frac{\mathrm{d_v}}{s}\right) \tag{2}$$

where the SI coefficients are given by $\beta_1 = 0.32, \beta_2 = -0.16$, $\beta_3 = 0.25$, and $\beta_4 = 0.6$. The variables f'_m and d_v are defined slightly differently in the two codes.

BLONDET ET AL. MODEL

Blondet et al. [5] presented two models for predicting the ultimate shear strength of fully grouted masonry. Of the two models, the second model was later used to develop the TMS and CSA equations. The authors assumed the ultimate shear stress could be represented by the form

$$v_u = v_{cr} + \frac{v_s}{2} \tag{3}$$

where v_{cr} is the cracking shear stress and the reinforcement component is represented by the form

$$v_s = \rho_h f_y \tag{4}$$

The authors assumed the cracking shear stress to be a function of the masonry tensile strength and axial load. Using Mohr's circle, the masonry tensile strength was assumed to be equal to the cracking strength at zero axial load v_{cr0}, which leads to the relationship

$$v_{cr}^2 = v_{cr0}^2 + \frac{2}{3} v_{cr0} f_a \tag{5}$$

where f_a is the vertical axial stress.

Blondet et al. assumed the cracking stress at zero axial load could be represented by the form

$$v_{cr0} = \left(\beta_1 + \beta_2 \frac{M}{Vd}\right)\sqrt{f'_m} \tag{6}$$

where the shear span ratio $\frac{M}{Vd}$ is less than or equal to unity. The authors developed separate model coefficients for concrete block and clay brick. Both models can be combined into a single linear model

$$v_{cr0} = \left[\left(\beta_{1\mathrm{BL}} + \beta_{2\mathrm{BL}} \frac{M}{Vd}\right)\gamma_{BL} + \left(\beta_{1\mathrm{BR}} + \beta_{2\mathrm{BR}} \frac{M}{Vd}\right)\gamma_{BR}\right]\sqrt{f'_m} \tag{7}$$

where γ_{BL} and γ_{BR} are given the value of 1.0 for concrete or clay masonry, respectively, and zero otherwise. The authors proposed the following model coefficients for Equation (7): $\beta_{1\mathrm{BL}} = 0.29$, $\beta_{2\mathrm{BL}} = -0.145$, $\beta_{1\mathrm{BR}} = 0.35$, and $\beta_{1BR} = -0.145$.

Cracking Stress

Blondet et al. [5] did not describe how the coefficients for Equation (7) were obtained nor how values were derived for v_{cr0}. Since only two specimens from the examined dataset had zero applied axial load, the v_{cr0} values were most likely derived from experimental cracking and axial load values. The relationship in Equation (6) can be rearranged to solve for v_{cr0} in terms of the other variables:

$$v_{cr0} = \frac{1}{3}\left(\sqrt{f_a^2 + 9v_{cr}^2} - f_a\right) \tag{8}$$

In the present study, least squares regression was used to compute the model coefficients and goodness-of-fit statistics for the model in Equation (7). The original coefficients and the regressed coefficients determined in this study are presented in Table 1. A comparison of the coefficient values indicates Blondet et al. did not use least squares regression in determining their model coefficients.

Table 1: Coefficients and Goodness-of-Fit Statistics for Blondet et al. Cracking Model

	Blondet et al. [5]		**Regressed**	
	SI	USCS	SI	USCS
β_{1BL}	0.29	3.5	0.303	3.66
β_{2BL}	-0.145	-1.75	-0.055	-0.66
β_{1BR}	0.35	4.2	0.438	5.28
β_{2BR}	-0.145	-1.75	-0.233	-2.81
SD	0.436 MPa	63.3 psi	0.364 MPa	52.8 psi
R^2	0.8870		0.9306	

Shear Reinforcement

Equation (**1**) can be represent by the linear model

$$v_u = \beta_1 v_{cr} + \beta_2 v_s \tag{9}$$

Blondet et al. [5] assumed that only the reinforcement in the center part of the walls was effective in resisting shear such that only half of v_s contributed to the ultimate shear strength of masonry shear walls. In the current study, least squares regression was performed on Equation (9) to assess the validity of the assumed contribution of the horizontal reinforcement. A comparison of the coefficients and goodness-of-fit statistics is presented in Table 2. Based on the regression results, the ultimate strength of the wall is correlated to only 20% of the horizontal shear reinforcement strength. The 95% confidence interval for β_2 is given by the range $[0.0763, 0.3381]$. By statistical convention, the hypothesized value of 0.5 should have been rejected because it lies outside of the 95% confident interval.

Table 2: Coefficients and Goodness-of-Fit Statistics for Blondet Reinforcement Contribution

		Blondet	Regressed
β_1		1.0	0.9998
β_2		0.5	0.2072
SD	MPa	0.558	0.394
	(psi)	(80.9)	(57.1)
R^2		0.9413	0.9651

The coefficients in Table 2 were determined using both concrete and clay masonry. Following the approach of the original authors, the analysis was repeated to determine new reinforcement coefficients for concrete and clay masonry separately. The coefficients are presented in Table 3. The coefficient values are notably different both from each other and from the original values.

Table 3: Comparison of Coefficients and Goodness-of-Fit Statistics for Concrete and Clay Masonry

			Regressed	
		Blondet	Concrete	Clay
β_1		1.0	1.110	0.9394
β_2		0.5	0.2929	0.1807
SD	MPa	0.394	0.399	0.276
	(psi)	(57.1)	(57.9)	(40.0)
R^2		0.9413	0.9610	0.9849

Full Model

The full form of the Blondet et al. [5] model can be obtained by substituting Equations (4), (5), and (6) into Equation (3) to produce

$$v_u = \sqrt{\left(\beta_1 + \beta_2 \frac{M}{Vd}\right)^2 f'_m + \frac{2}{3}\left(\beta_1 + \beta_2 \frac{M}{Vd}\right) f_a \sqrt{f'_m}} + \beta_4 \rho_h f_y \tag{10}$$

The coefficients in Equation (10) cannot be determined explicitly using least squares regression. The form presented by Blondet et al. is different from the form later adopted by TCCMaR:

$$v_u = \left(\beta_1 + \beta_2 \frac{M}{Vd}\right) \sqrt{f'_m} + \beta_3 f_a + \beta_4 \rho_h f_y \tag{11}$$

The TCCMaR model [4] uses the form in Equation (7) for the masonry component; however, Equation (7) was originally intended to estimate the cracking strength of unconfined masonry, not the masonry contribution to ultimate shear strength. Nevertheless, the substitution of v_{cr0} in place of v_{cr} may not have had a significant effect on the model form.

The masonry component in the Blondet et al. model, given in Equation (5), can be rearranged as

$$v_{cr} = \sqrt{v_{cr0}^2\left(1+\frac{2}{3}\frac{f_a}{v_{cr0}}\right)} = v_{cr0}\sqrt{1+\frac{2}{3}\frac{f_a}{v_{cr0}}} \tag{12}$$

For cases where $\left|\frac{f_a}{v_{cr0}}\right| \leq \frac{3}{2}$, the square root term in Equation (12) can be approximated by using the first two terms of the Taylor series expansion:

$$v_{cr} = v_{cr0}\left[1+\frac{1}{2}\left(\frac{2}{3}\frac{f_a}{v_{cr0}}\right)\right] = v_{cr0} + \frac{1}{3}f_a \tag{13}$$

By substituting Equations (6) and (13) into Equation (10), the Blondet et al. model can be approximated by the simplified model

$$v_u = \left(\beta_1 + \beta_2\frac{M}{Vd}\right)\sqrt{f'_m} + \beta_3 f_a + \beta_4\rho_h f_y \tag{14}$$

where β_1 and β_2 are provided in Table 3, $\beta_3 = \frac{1}{3}$, and $\beta_4 = \frac{1}{2}$. The simplified model form is identical to the form of the TCCMaR model, only the coefficient values differ.

ANDERSON & PRIESTLEY MODEL

Anderson and Priestley [6] presented a model to predict the ultimate shear strength of masonry walls. It appears that the authors partially based their model on the model proposed by Shing et al. [7], but with some terms modified or omitted. Their model has the form

$$V_u = \beta_1 k_1 k_2\sqrt{f'_m}A_g + \beta_2 A_v f_{yv} + \beta_3 P_u + \beta_4\frac{A_h}{s}f_{yh}d_v \tag{15}$$

where k_1 is the aspect ratio coefficient, k_2 is the ductility coefficient, A_v is the area of vertical reinforcement in the middle third of the wall, and A_h/s is the area of horizontal steel per unit spacing.

Anderson and Priestley assumed that shear strength was correlated with the wall aspect ratio by a coefficient k_1. All of the walls in their examined dataset had aspect ratios greater than unity. The general consensus amongst researchers is that shear strength is correlated to the shear span ratio and not to aspect ratio; nevertheless, since there was not enough variation in the dataset to determine the influence of aspect ratio, they remove the k_1 term from consideration in their analysis. Since wall ductility was not considered in their study, they similarly removed the k_2 term from their analysis.

Methodology

Anderson and Priestley [6] used an optimization-based procedure to determine the coefficient values for their model. The objective of their procedure was to minimize the deviation of V_u/V_t from unity. They performed the optimization using a trial-and-error approach by varying each of the four coefficient values until the model deviation was minimized. Given the highly indeterminate relationship between all of the coefficients, the approach used was likely cumbersome, especially considering that all four values could have been determined explicitly in a single step using multivariate regression.

The Anderson model for concrete masonry was reanalyzed in the current study using multivariate regression to determine the optimum coefficient values. Both the original and regressed sets of values and goodness-of-fit statistics are presented in Table 4. The coefficient values obtained by the original study for concrete masonry are reasonably close to the regressed values. This similarity can be attributed to the use of an optimization approach in the original study.

Table 4: Comparison of Coefficients and Goodness-of-Fit Statistics for the Anderson and Priestley Model

	Anderson & Priestley [6]		**Regressed**	
	SI	USCS	SI	USCS
β_1	0.24	3.28	0.275	3.32
β_2	0.0		-0.011	
β_3	0.25		0.196	
β_4	0.5		0.481	
SD	32.8 kN	7.37 kip	28.8 kN	6.47 kip
R^2	0.995		0.996	

The regression analysis determined the contribution of the vertical reinforcement parameter to be insignificant, which agrees with the results from the original study. Repeating the regression with the vertical reinforcement parameter omitted resulted in a negligible change to the coefficient values for the three remaining parameters.

Anderson and Priestley also developed model coefficient values for clay masonry but did not publish the dataset from which the coefficients were determined. They simply stated that their analysis showed no correlation for the horizontal reinforcement parameter and a high degree of variability, which they presumed was due to a lack of clay masonry data. They repeated their analysis while maintaining the same coefficient values for the axial load and horizontal reinforcement parameters from their prior analysis of concrete masonry. In their second analysis of clay masonry, they computed $\hat{\beta}_1$ to be 0.12.

DISCUSSION

Model Form

The commentary to the 1997 NEHRP Provisions [4] explains that the TCCMaR equation

$$v = \left(\beta_1 + \beta_2 \frac{M}{Vd}\right)\sqrt{f'_m} + \beta_3 \sigma_c + \beta_4 \rho_h f_{yh} \tag{16}$$

was a combination of the Blondet et al. [5] and Anderson and Priestley [6] equations. The masonry component was based on the Blondet form. Based on research performed by Matsumura [8], shear strength appears to be inversely related to the shear span ratio. The $\left(\beta_1 + \beta_2 \frac{M}{Vd}\right)$ form adopted by

Blondet et al. and subsequently by the TCCMaR appears to be a linear approximation of the inverse relationship.

The NEHRP commentary explains that the form $\beta_3\sigma_c$ from the Anderson and Priestley model was selected for the axial load component because it was simpler than the form proposed by Blondet et al. and the correlations were similar [4]. Based on this explanation, it appears that the simplified form of the Blondet equation, Equation (14), was not considered in the TCCMaR study and that the similarity in forms is just coincidental. If this simplified form had been considered, it is possible that the value $\beta_3 = 0.33$ may have been selected to maintain theoretical consistency with the Mohr's circle relationship used by Blondet et al. The horizontal shear reinforcement component was the same for both studies, so it can be ascribed to both studies.

Coefficient Values

The rationale for the selection of values for the $\hat{\beta}_1$ and $\hat{\beta}_2$ coefficients is absent from the NEHRP commentary. Given that the coefficient $\hat{\beta}_2$ in the Blondet model was constant for both concrete and clay masonry, it appears that this value was adopted directly into the TCCMaR model.

The coefficient value $\beta_3 = 0.25$ for the axial load component was taken directly from the Anderson and Priestley study. Regression results of the Anderson model presented herein shows that a value of 0.20 would have been more representative. Regardless, the results of the Anderson model should not have been directly translated into the TCCMaR model because they do not share the same model form.

The selected horizontal shear reinforcement coefficient $\beta_4 = 0.5$ was the same for both studies. In the Blondet model, the coefficient value was assumed by the authors. Regression results for the Blondet model, presented in Table 2, show that the selected value of 0.5 was unconservative. The coefficient value of 0.5 appears to be valid for the Anderson model and dataset. In either case, the form of the TCCMaR model differed from both of the previous studies. The β_4 value from the previous studies should not have been directly assigned to the TCCMaR model; it should have been derived expressly for the TCCMaR model using the TCCMaR dataset.

In the final version of the TCCMaR model, it appears the values for the $\hat{\beta}_2$, $\hat{\beta}_3$, and $\hat{\beta}_4$ coefficients were assigned and not derived. By assigning values to three of the unknown coefficients, determining a value for the remaining unknown coefficient $\hat{\beta}_1$ could be performed explicitly by computing β_{1i} for each of the n specimen and finding the mean of all the $\hat{\beta}_{1i}$ values:

$$\hat{\beta}_1 = \frac{1}{\mathrm{n}}\sum_{i=1}^{n}\left(\frac{v + 1.75\frac{M}{Vd}\sqrt{f'_m} - 0.25\sigma_c - 0.5\rho_h f_{yh}}{\sqrt{f'_m}}\right)_i \tag{17}$$

The TCCMaR model was originally evaluated using the dataset assembled by Fattal and Todd [9]. By reevaluating Equation (17) using the Fattal dataset, the solution for $\hat{\beta}_1$ was computed to be 3.96. The value was rounded up to 4.0 in the final TCCMaR model for simplicity.

As aforementioned, any values other than those determined through least squares regression will introduce additional modeling error into the model predictions. In the current study, regression was performed on the TCCMaR model using the Fattal dataset to determine the optimum coefficient values. These optimum coefficients were compared to the TCCMaR values to examine how much error the assumed values introduce into the predictions. All but one of the optimum coefficient values for the TCCMaR model are notably different from the values that were ultimately used in the TCCMaR model, as shown in Table 5. It appears that the similarity in the $\hat{\beta}_2$ values is completely coincidental.

Table 5: Comparison of Model Coefficents and Goodness-of-Fit Statistics for the TCCMaR Model

	TCCMaR [4]		Regressed	
	SI	USCS	SI	USCS
β_1	0.332	4.0	0.466	5.62
β_2	-0.145	-1.75	-0.146	-1.75
β_3	0.25		0.090	
β_4	0.5		0.156	
SD	0.467 MPa	67.7 psi	0.328 MPa	47.6 psi
R^2	0.957		0.978	

It appears that the TCCMaR model significantly over-represents the strength contributions from the axial load and horizontal reinforcement and under-represents the contribution of the masonry. This result appears consistent with two recent studies [10][11] that both concluded that the contribution of the horizontal shear reinforcement is much smaller than is represented by the TCCMaR, TMS, and CSA shear models. It appears that the principal contribution of shear reinforcement is to resist the opening of shear cracks, thus permitting the masonry to transfer stresses through the panel via strut action and crack friction [11].

Throughout the various analyses presented herein, the coefficient values were highly sensitive to slight changes to model form and to the experimental dataset used. This lack of robustness suggests that the form given by the three models is not the best at modeling the ultimate shear strength of reinforced masonry panels.

Vertical reinforcement

The TCCMaR study considered whether to include a term for the vertical reinforcement in the final model. In the original study, the contribution of the vertical steel was assumed to have a coefficient of 0.25. Results indicated that the correlation of the model was not as good when the vertical reinforcement term was included, so the contribution of the vertical steel was excluded from the model. The methodology used to determine whether to include the vertical reinforcement term was not appropriate for determining whether to include or exclude a parameter in the linear model. An appropriate method would have been to perform a multivariate regression of the model with and without the parameter and perform an analysis of variance (ANOVA) between the full and reduced models to determine a p-Value for the parameter. Modern regression tools automatically calculate p-Values for all parameters.

In the current study, the vertical reinforcement contribution was evaluated by comparing regression results of the reduced model given in Equation (16) and the full model given by

$$v = \left(\beta_1 + \beta_2 \frac{M}{Vd}\right)\sqrt{f'_m} + \ \beta_3\sigma_c + \beta_4\rho_h f_{yh} + \beta_5\rho_{vi} f_{yvi} \qquad (18)$$

where ρ_{vi} is the ratio of the interior vertical reinforcement (excluding the jamb, or flexural, reinforcement) and f_{yvi} is the yield strength of the interior reinforcement. The coefficients and p-Values for the two models are presented in Table 6. The p-Value for the vertical reinforcement is 0.069, which is statistically significant at the $\alpha = 0.10$ level. Although the p-Value for the vertical reinforcement is not as statistically significant as the other parameters in the full model, the difference is not enough to justify its exclusion from the final model.

Based on the results of the regression analysis presented in Table 6, the contributions of the vertical and horizontal reinforcement are similar. This suggests that vertical reinforcement is nearly as effective as horizontal reinforcement in increasing the ultimate shear strength of masonry panels. This similarity in strength contribution supports the previous hypothesis that the principal contribution of reinforcement is to resist the opening of shear cracks, permitting the masonry itself to transfer the shear stresses through the panel [11]. Given the assumed 45° inclination angle of shear cracking, vertical and horizontal reinforcement will cross shear cracks at similar angles and would be similarly effective in resisting the opening of shear cracks.

Table 6: Evaluation of Vertical Reinforcement Parameter for the TCCMaR Model

	Full Model			Reduced Model		
	SI	USCS	p-Value	SI	USCS	p-Value
$\hat{\beta}_1$	0.506	6.10	0.000	0.466	5.62	0.000
$\hat{\beta}_2$	-0.257	-3.09	0.002	-0.146	-1.75	0.006
$\hat{\beta}_3$	0.100		0.017	0.090		0.033
$\hat{\beta}_4$	0.150		0.035	0.156		0.031
$\hat{\beta}_5$	0.103		0.069	--		--
R^2	0.979			0.978		

CONCLUSIONS

Based on the reanalysis of the models presented by Blondet et al. [5], Anderson and Priestley [6], and TCCMaR [4], it appears that many errors were introduced into the TCCMaR model coefficients. Several of the coefficients were simply assumed or assigned rather than being derived explicitly, which lead to non-optimum values being used in the final model. Specifically, the contributions of the axial load and horizontal shear reinforcement appear to be inflated in the final TCCMaR model, which suggests that they have a greater contribution to shear strength than they actually do. These errors in the coefficient values increase the modeling error and inflate the variation and uncertainty of the model predictions compared to optimum coefficient values.

It appears that the form of the TCCMaR model is not the most representative form for masonry shear strength. Misspecification of the model introduces additional modeling error into the predicted values.

An example of one model misspecification is that the model excluded the contribution of the vertical reinforcement when there wasn't a statistical basis to do so. It appears that other misspecifications are likely because the coefficient values were observed to be highly sensitive to the experimental dataset used in analysis. In addition, the model was not designed or analyzed for use with partially grouted walls, which further increases the modeling error when used for partially grouted walls.

All of the implicit errors in the TCCMaR model carried over into the TMS and CSA equations. It is not possible to resolve all of the errors in the TMS and CSA equations by adding factors or changing coefficient values. Given the large number of masonry shear specimens have been tested in the past two decades, particularly partially grouted masonry, and the increased computing power that is now available, it should be possible to develop an improved shear strength model. In light of the observations made during this study, it is recommended that a new masonry shear strength model be developed to mitigate the errors in the TMS and CSA models.

REFERENCES

[1] Committee 402/602 (2016). *Building Code Requirements for Masonry Structure (TMS 402-16)*, The Masonry Society, Longmont, CO.

[2] CSA (2014). *Design of Masonry Structures (S304-14)*, Canadian Standards Association, Mississauga, ON.

[3] Dillon, P. B. and Fonseca, F. S. (2016). "Partially grouted masonry shear strength uncertainty." *Proc. 13th Canadian Masonry Symposium*, Halifax, NS.

[4] National Earthquake Hazards Reduction Program (1997). *NEHRP recommended provisions (FEMA 302) and commentary (FEMA 303) for seismic regulations for new buildings and other structures*, Federal Emergency Management Agency (FEMA), Washington, DC.

[5] Blondet, J. M., Mayes, R. L., Kelly, T., Villablanca, R., and Klinger, R. E. (1989). "Performance of engineered masonry in the Chilean earthquake of March 3, 1985: Implications for U.S. design practice." *Tech. Rep. 89-2*, University of Texas at Austin, Austin, TX.

[6] Anderson, D. L. and Priestley, M. J. N. (1992). "In plane shear strength of masonry walls." *Proc., 6th Canadian Masonry Symposium*, Saskatoon, SK, 223-234.

[7] Shing, P. B., Schuller, M., and Hoskere, V. S. (1990). "In-plane resistance of reinforced masonry shear walls." *J. Struct. Eng.,* 116(3), 619-639.

[8] Matsumura, A. (1987). "Shear strength of reinforce hollow unit masonry walls—differences between partially grouted walls and fully grouted walls." *Proc., 4th North American Masonry Conference*, Los Angeles.

[9] Fattal, S. G. and Todd, D. R. (1991). *Ultimate strength of masonry shear walls: Predictions vs. test results, Tech. Rep. NISTIR 4633*, National Institute of Standards and Technology, Gaithersburg, MD.

[10] Oan, A. and Shrive, N. (2015). "The effect of reinforcement embedded in bond beams in concrete masonry walls." *Proc., 12th North American Masonry Conference*, Denver, CO.

[11] Dillon, P. B. (2015). *Shear Strength Prediction Methods for Grouted Masonry Shear Walls*, PhD dissertation, Brigham Young University, Provo, UT.