APPLICATION OF THE NORMAL STRAIN-ADJUSTED SHEAR STRENGTH EXPRESSION (NSSSE) FOR STRENGTH PREDICTION OF FULLY-GROUTED REINFORCED MASONRY STRUCTURAL WALLS

Bennett Banting[1] and Wael El-Dakhakhni[2]

[1] Ph.D., Department of Civil Engineering, McMaster University, Hamilton, ON, Canada, bantinbr@mcmaster.ca

[2] Martini, Mascarin and George Chair in Masonry Design, Department of Civil Engineering, Hamilton, ON, Canada, eldak@mcmaster.ca

ABSTRACT

North American masonry design codes rely on a 45° cracked member assumption and truss analogy to estimate shear strength of members. Whereby shear strength is expressed as an algebraic summation of resistance offered by masonry, axial load and shear reinforcement. By contrast, the Modified Compression Field Theory (MCFT), which has gained a wide acceptance within the concrete design community, demonstrates that the 45° cracked member assumption can be overly conservative. Yet the MCFT or similar equilibrium-based approaches have often been thought of as incompatible with masonry due to the latter's complex anisotropic behavior. A methodology is proposed to accurately estimate the angle of inclination of shear cracking and the shear resistance offered by the horizontal reinforcement and the masonry compression strut accounting for aggregate interlock effects within a masonry macro-element. A design equation is proposed and its ability to accurately estimate the shear strength of a structural wall is verified through a comprehensive review of applicable test results from available literature. The proposed Normal Strain-adjusted Shear Strength Expression (NSSSE) was found to predict the shear strength of 57 wall tests reported in literature with a mean ratio of experimental to theoretical strengths of $V_{Experimental}$ / V_{Theory} = 1.16 (C.O.V. = 11.4%) and a 99% percentile of $V_{Experimental}$ / V_{Theory} = 0.86 marking a significant improvement over existing design code expressions that is also firmly grounded in a sound theoretical formulation.

KEYWORDS: concrete block, design codes, shear wall, shear strength

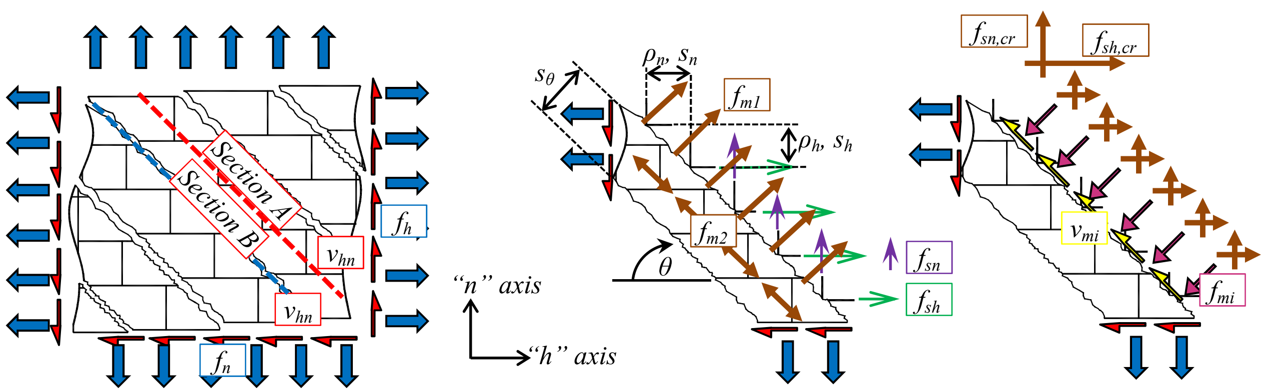
INTRODUCTION

The study presented herein focuses on addressing what is perceived to be excessive conservatism as well as a lack of theoretical basis for the current empirically derived masonry shear strength expressions. This is achieved by developing a mechanics-based approach to solve for force equilibrium and strain compatibility in cracked masonry macro elements. Thus, an explicit solution for the angle of inclination of shear cracking and resulting masonry shear strength contribution via aggregate interlock forces can be determined explicitly, rather than relying on a constant 45° crack model. Vecchio and Collins [1,2] developed a smeared crack model that could accurately predict the shear behavior of reinforced concrete (RC). Their *Modified Compression Field Theory* (MCFT) utilizes a series of material constitutive relationships, stress-strain compatibility equations and force equilibrium expressions to quantify the shear strength of RC elements. The difficulty in consolidating the inherent differences between reinforced masonry (RM) and RC materials precludes a theoretically sound adoption of MCFT to masonry in its current form. Nevertheless, the observations from [3,4] indicate that for both RM beams and shear walls, respectively, the MCFT has a potential for adoption in masonry shear design. However, a major hurdle to the application of MCFT to RM lies in inherent differences between RM, which is considered to be anisotropic and RC, which is generally regarded as isotropic. To overcome this, experimental tests on constituent masonry materials as well as masonry macro elements subjected to well defined states of stress and strain will be used from existing literature.

A limited number of tests on concrete block panels (macro elements) are reported in the literature reviewed by the writers, with even fewer results that included steel reinforcement. Khattab [5] and Drysdale and Khattab [6], tested 36 unreinforced and reinforced concrete block macro elements under various bi-axial stress states. While, Tikalsky et al. [7] tested eight RM panels under a simultaneous axial compressive stress applied normal to the bed joints and a lateral tensile stress applied parallel to the bed joints and Liu et al. [8] tested a total of 86 unreinforced fully-grouted concrete block masonry square panels under varying principal stress ratios as well different angles. It is this test data that will subsequently be used towards establishing the necessary stress-strain relationships in masonry to ultimately derive an expression suitable for design.

EQUILIBRIUM OF FORCES AND STRAIN COMPATIBILITY IN CRACKED MASONRY MACRO ELEMENTS

A schematic view of a cracked masonry macro element is presented in Fig. 1a subjected to an average normal stress f_n along the axis normal to the bed joint (n), an average horizontal stress f_h along the axis parallel to the bed joint (or normal to the head joint) (h), and a shear stress v_{hn}. In Fig. 1a, the macro element is divided into identical crack-separated struts inclined at an angle θ at an average spacing of s_θ. The masonry struts are subjected to principal tensile stresses (f_1), which are oriented perpendicular to the cracks, and principal compressive stresses (f_2), which are oriented parallel to the struts. In addition, the vertical and horizontal reinforcing bars that are located along the n and h axes, respectively, of the macro element are smeared, resulting in reinforcement ratios of $\rho_{sn} = A_{sn} / s_n b_w$ and $\rho_{sh} = A_{sh} / s_h b_w$ where A_s, s_n, s_h and b_w are the area of a single bar (mm^2), the average spacing between the vertical bars (mm), the average spacing between the horizontal bars (mm) and the width of the masonry unit (mm), respectively.



a) Macro-Element **b)** Section A: Stress in a Strut **c)** Section B: Stress at a Crack

Figure 1: Equilibrium of Equivalent Stresses of a Cracked Masonry Macro Element

Through the assumption of a perfect bond, the state of strain within the macro element pictured in Fig. 1 can be determined assuming that the angles of principal stresses and principal strains coincide. This assumption has been experimentally verified [5] for masonry macro elements. The resulting state of strain of the macro element is expressed with a Mohr's circle in Fig. 2. The principal tensile strain (ε_1) represents the average tension strain acting perpendicular to the masonry struts and across the cracks, the principal compressive strain (ε_2) acting along the compression strut of the cracked masonry and the average strains along the vertical and horizontal axes are represented by ε_n and ε_h, respectively. Finally, the average total shear strain of the element is given by γ_{hn} in Fig 2.

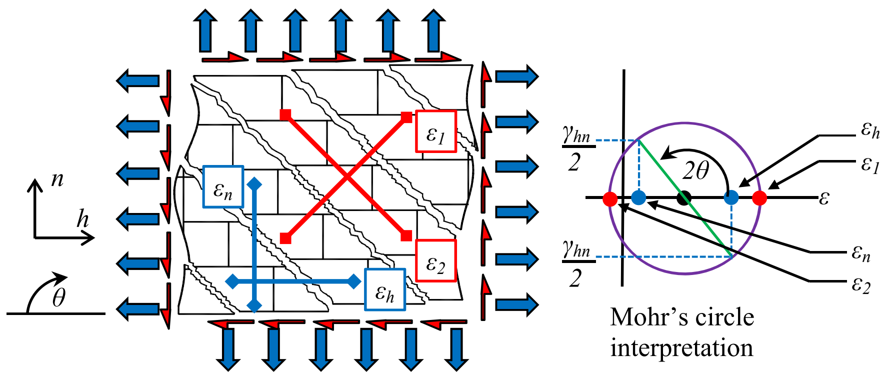


Figure 2: State of Strain in a Masonry Macro Element

CONSTITUTIVE RELATIONSHIPS FOR CRACKED MASONRY

It should be noted that the inclination of masonry compression struts by the angle θ means that, except for the special case of $\theta = 90°$, the characteristics of the compression strut is unlikely to resemble that of the typical uniaxial prism tests, specifically their strength, defined here as $f'_m(90°)$. Therefore, there is a need to consider the anisotropy of masonry construction in altering the compressive strength of masonry as a function of the load orientation with respect to the bed joint. To establish a relationship between $f'_m(90°)$ and $f'_m(\theta)$, the results from tests on large masonry panels loaded under pure compression with $\theta = 0°$, 22.5°, 45°, 67.5° and 90° will be utilized from [6,8]. Based on regression analysis of the data, the writers proposed the compressive strength-orientation interaction relationships for $f'_m(\theta)$ given in Eq. 1 and 2.

$$\frac{f'_{m(\theta)}}{f'_{m(90^o)}} = (4.74\times10^{-4})\theta^2 + (-2.43\times10^{-2})\theta + 0.883 \leq 1.0 \qquad \left[\theta \leq 45^o\right] \tag{1}$$

$$\frac{f'_{m(\theta)}}{f'_{m(90^o)}} = (2.66\times10^{-4})\theta^2 + (-3.04\times10^{-2})\theta + 1.58 \leq 1.0 \qquad \left[\theta > 45^o\right] \tag{2}$$

The combined effects of simultaneous compressive axial and lateral tension stresses would also be expected to be present in a compression strut of a masonry macro-element and would have an influence on $f'_m(\theta)$. To solve for equilibrium for a given state of stress it is necessary to develop a set of constitutive relationships that account for the effect of the angle θ as well as lateral tensile strains ε_1 on the masonry compression strut strength $f'_m(\theta)$. It has been shown that RM subject to axial compression shows a reduction in compressive strength capacity under the simultaneous application of lateral tensile forces [7] and that in addition to a reduction in strength caused by lateral tensile stresses, the angle by which the principal stresses are orientated will also cause a reduction in compressive strength [6]. Therefore, to consolidate this complex behavior into usable stress-strain relationships, the effect of θ as well as the lateral tension strains ε_1 must be considered within the compressive stress-strain relationship of the masonry compression struts within a macro element. In the first panel of Fig. 3 the stress-strain behaviour of masonry under pure compression is represented by the Hognestad relationship as it relates to the principal compressive strain (ε_2). The principal stress ratios which indicates the reduced compressive strength of a masonry strut when subjected to lateral tensile stress (ε_1) from ten RM panels reported by Drysdale and Khattab [6] (*DK*) and five RM panels reported by Tikalsky et al. [7] (*TAH*) at ultimate conditions are shown in the far right panel of Fig. 3. Whereby, the peak compressive stress (f_{m2}) reported in the respective studies are normalized by the converted strength $f'_m(\theta)$ using Eqs. 1 or 2, as applicable. The normalized strength is plotted against the ratio of ε_1 and the strain at peak compressive strength of masonry (ε_o) taken as 0.0018. This provides an upper limit to the peak masonry compressive strength when subject to lateral strains. The best fit curve is shown in Fig. 3 and its equation is given by:

$$f_{m2,\max} = \frac{f'_{m(\theta)}}{0.41 + 0.33\dfrac{\varepsilon_1}{\varepsilon_o}} \leq f'_{m(\theta)} \tag{3}$$

In the middle panel of Fig. 3 is a representation of the resulting stress-strain relationship of a masonry strut subjected to axial compression strains and lateral tension strains within a cracked macro element orientated at an angle θ.

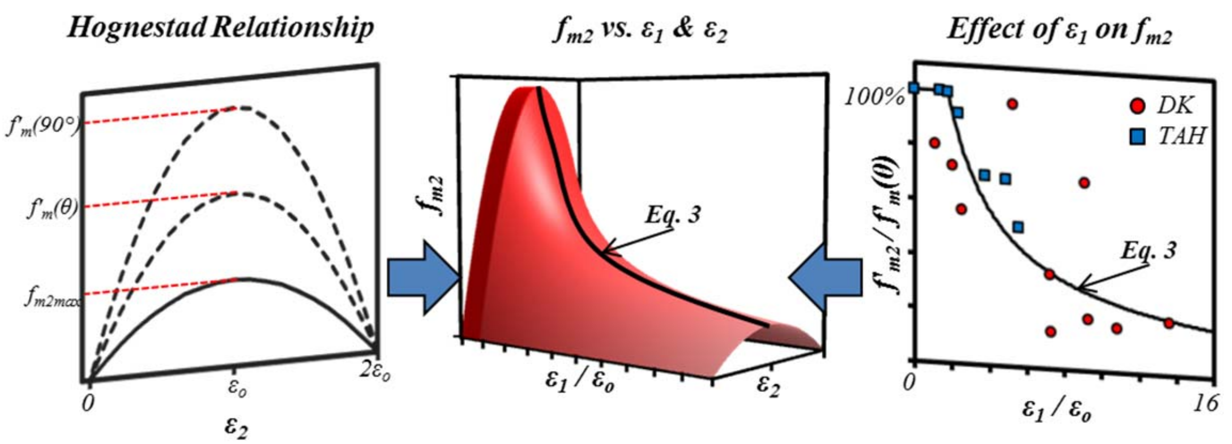


Figure 3: Stress-Strain Relationship for Masonry Subject to Lateral Tension

To solve for stress equilibrium of the macro elements, the tensile stress capacity of the masonry strut must also be considered. Prior to cracking, the tensile strength of masonry is assumed to act linearly elastic, however, the capacity of the uncracked masonry strut to carry the tensile stress f_{m1} is expected to soften as ε_1 increases and cracks form. To account for this effect, Eq. 4 was proposed by [5], where $f_{cr(\theta)}$ is the strength determined experimentally [6,8] and given in Eq. 5.

$$f_{m1} = \frac{f_{cr(\theta)}}{1+400\varepsilon_1} \leq f_{cr(\theta)} \tag{4}$$

$$\frac{f_{cr(\theta)}}{\sqrt{f'_{m(90^o)}}} = \left(-3.93\times10^{-5}\right)\theta^2 + \left(3.49\times10^{-3}\right)\theta + 0.212 \tag{5}$$

If the average crack width (w) between the compression struts exceeds the size of protruding aggregates along the crack surface, no contact, and hence no friction, will exist. It can be seen then that the aggregate interlock contribution of mortar along bed joint cracks, which is comprised of very fine aggregates, will be dwarfed by the contributions of the concrete block and grout which are comprised of larger aggregate sizes, when cracks penetrate the grout and the faceshells of the masonry units. An upper bound ($v_{c,max}$) to the maximum shear that may be transferred across an open crack in RC as developed by [2] based on the work presented by [9] has been adapted for use with masonry as follows:

$$v_{mi} \leq v_{m,\max} = \frac{0.18\sqrt{f_{m,av}}}{0.31 + \frac{24w}{(a_{g,av}+16)}} \tag{6}$$

Whereby, $a_{g,av}$ is taken as a weighted mean of the standard aggregate sizes in constituent masonry materials. In this context it is also proposed that the weighted mean of the masonry material bond strength (f_{av}) based on the volumetric ratio of each material, rather than the typical uniaxial prism strength $f'_{m(90°)}$ for masonry is used. This is due to the fact that prism strength is not an accurate reflection of the strength of the cement matrix which aggregates are boned within due to failure mechanisms associated with masonry assemblages. The final term required in the determination of shear transferred by aggregate interlock is the crack width ($w = s_\theta \times \varepsilon_1$). The average crack spacing within a masonry macro element (s_θ) can be estimated with Eq. 7 substituting in average crack spacing parameters s_{hc} and s_{nc}, measured along the horizontal and vertical axes, respectively.

$$s_\theta = \frac{1}{\frac{\sin(\theta)}{s_{hc}} + \frac{\cos(\theta)}{s_{nc}}} \tag{7}$$

Whereby, the average crack spacing measured as the vertical distance between horizontal cracks is given by s_{hc} and the horizontal distance measured between cracks forming normal to the bed joint is given by s_{nc}. The exact value of the crack spacing s_θ is a function of the potential planes of weakness within RM which may be related to the horizontally measured spacing between vertical reinforcement that runs normal to the bed joint (s_n). It also may be related to the measured space between head joints aligned within a running bond pattern (s_{hj}) taken as half the nominal block length. In the vertical direction these planes of weakness may be dictated by the spacing between horizontal reinforcement that runs parallel to the bed joint (s_h) or the nominal bed joint spacing (s_{bj}) taken as the nominal block height.

SIMPLIFIED NSSSE FOR CODE ADOPTION

Bentz et al. [10] proposed the Simplified Modified Compression Field Theory (SMCFT) as a relatively simple, but accurate, means to estimate the peak shear strength in RC members with simple hand calculations and minimal iteration. In a similar manner, the equilibrium equations derived through NSSSE of a masonry macro element can be put into terms amenable to shear strength expressions designers are familiar with, with a masonry strength parameter β and a crack inclination angle θ, where shear strength (v) is comprised of masonry (v_m) and reinforcement (v_s) terms:

$$v_n = v_m + v_s = \beta\sqrt{f'_{m(90^o)}} + \rho_{sh} f_{shy} \tan\theta \tag{8}$$

With the appropriate substitutions, the masonry strength components considering the shear transferred at the crack interface can be rearranged to solve for the β parameter which may be simplified into Eq. 9.

$$\frac{f_{cr(\theta)} \tan\theta}{\sqrt{f'_{m(90^o)}}\left(1+400\varepsilon_1\right)} = \beta \le \frac{0.18\sqrt{f_{av}}}{\sqrt{f'_{m(90^o)}}\left(0.31 + \dfrac{24\varepsilon_1 s_\theta}{a_{g,av}+16}\right)} \tag{9}$$

For design purposes, it would be beneficial to know f_{av} beforehand as a function of $f'_{m(90°)}$ since the latter is typically specified prior to the start of the design process. Therefore a new parameter, referred to as *the homogenized strength factor* (J), is proposed. The parameter gives the ratio between the average material strength of the aggregate interlock mechanism and the prism strength, and is given by:

$$J = \frac{f_{av}}{f'_{m(90^o)}} \tag{10}$$

The value of f_{av} should be based on a more thorough analysis considering actual material properties and material testing. However, adopting typical values for masonry it can be shown that J would be expected to range from 1.5 – 3.0 or can conservatively be taken as $f'_{m(90°)}$ (J = 1.0) instead. To reduce the iterative nature of NSSSE, a second parameter, defined as *the crack spacing and aggregate size factor*, λ, is necessary and is given by the following:

$$\lambda = \frac{24(s_{nc}+s_{hc})}{a_{g,av}+16} \tag{11}$$

The following values are suggested based on typical masonry construction: $a_{g,av}$ = 7.0 mm (for RM with coarse grout) and $a_{g,av}$ = 3.5 mm (for RM with fine grout). Finally, it is necessary for solution to Eq. 9 to have a means of relating shear wall behavior to the needed normal strain (ε_n). This may be simplified for walls where flexural deformations are anticipated to be significant (h_e/ℓ_w > 1.0), such that the normal strain can be taken directly as the tensile reinforcement strain in a shear wall, which it can be solved for as:

$$\varepsilon_n = \frac{(Vh_e / d_v) + (V) - (0.5P_{axial})}{0.8\rho_{sn} b_w \ell_w E_s} \qquad (h_e/\ell_w > 1.0) \tag{12}$$

Where, V is the shear force applied to the wall, P_{axial} is the axial load, ρ_{sn} is the vertical (flexural) reinforcement ratio, b_w is the width of the wall, ℓ_w is the length of the wall and E_s is modulus of elasticity for the steel reinforcement. However, in cases when shear deformations are anticipated to be significant in cantilever walls ($h_e/\ell_w \leq 1.0$) it can be shown that Eq. 12 can be modified to reflect the fact that it is less likely that flexural reinforcement will yield:

$$\varepsilon_n = \frac{(Vh_e / d_v) + (V) - (0.5P_{axial})}{2\rho_{sn} b_w \ell_w E_s} \qquad (h_e/\ell_w \leq 1.0) \tag{13}$$

Finally, for walls subject to double curvature Eq. 13 can be modified to account for the increased influence of the applied axial load as such:

$$\varepsilon_n = \frac{(Vh_e / d_v) + (V) - (P_{axial})}{2\rho_{sn} b_w \ell_w E_s} \qquad (\textit{Piers with double curvature}) \tag{14}$$

In cases where the normal strain is determined to be a negative or where members are of a very low aspect ratio (i.e. piers of $h_e/\ell_w \leq 0.50$) such that the plane strain assumption is no longer valid, ε_n may be conservatively taken to be zero. With the normal strain defined above, a relationship between θ and β in terms of level of normal strain ε_n and the parameters λ and J can be derived. Through regression analysis using the range of variables expected within shear wall design the following relationships have been determined:

$$\theta = \left(\frac{2000J + 4000 - \lambda}{50(18J + 75)} \right) \left(\frac{50}{(1 + 220\varepsilon_n)} \right) \tag{15}$$

$$\beta = \left(\frac{4000J + 2000 - \lambda}{500(3.5J + 1.5)} \right) \left(\frac{1}{4(1 + 1800\varepsilon_n)} \right) \tag{16}$$

Incumbent upon the NSSSE analysis carried out for RM walls was the presumption that failure occurred upon yielding of the horizontal reinforcement. If failure occurs prior to this, then the horizontal reinforcement would be assumed to have a tensile strain just below its yield strain (e.g. $\varepsilon_h \leq 0.002$ for 400 MPa reinforcement). Similarly, the principal compressive strain could be conservatively taken as just prior to the peak masonry strength (i.e. $\varepsilon_2 = \varepsilon_o = 0.0018$), then a relationship between shear strength and normal strain ε_n can be solved for. By also assuming a normal strain equal to that just prior to yielding of vertical reinforcement (e.g. $\varepsilon_n \leq 0.002$ for 400 MPa reinforcement) a limiting shear stress of $0.26f'_{m(90°)}$ can be determined directly. Accounting for the fact that reinforcement yield strengths may actually vary significantly from 400 MPa and possible ε_o deviation from 0.0018, a limit to the maximum design shear stress of $0.15f'_{m(90°)}$ is proposed. This is also consistent with the current maximum shear stress limit of $0.25f'_c$ adopted by RC design standards considering the reduction in compressive strength of masonry with the angle θ (i.e. the minimum masonry strength is approximately 0.6 $f'_{m(90°)}$ and 0.25×0.6 = 0.15). In the next section the preceding simplification of the NSSSE will be verified with the available database on RM structural wall tests.

SIMPLIFIED NSSSE VERIFICATION: RM WALLS

A survey of the available literature has resulted in a total of 77 reliable RM wall and pier tests all subject to reversed cycles of quasi-static loading. However, unlike previous design code expressions, the NSSSE was explicitly derived for use with concrete block masonry and, as such, brick masonry construction will not be considered. This resulted in a total of 57 tests on concrete block masonry structural walls and piers collected from seven sources [4,11,12,13,14,15,16]. The physical parameters for each wall have been used as reported in literature, however, for cases where material testing data was not available a value of J = 1.5 was used. For each wall, the predicted shear strength was determined using Eq. 17 and was compared with the corresponding average peak shear strength from both directions of loading. To account for the fact that θ may deviate from 45°, a check was also necessary to ensure that the assumed height of the crack does not exceed the height of the wall in determining the shear resistance of the reinforcement resulting in:

$$V_n = \beta\sqrt{f'_{m(90^o)}}\,b_w d_v + A_{sh} f_{shy}\frac{d_v \tan\theta}{s_h} \le 0.15 f'_{m(90^o)}\,b_w d_v; \quad d_v \tan\theta \le h_w \tag{17}$$

The physical parameters for each wall were first identified based on the reinforcement detailing, aspect ratio and boundary conditions according to the procedures previously laid out. The shear strength (V_n) is solved for with Eq. 17 as part of an iterative solution process, since both θ and β are a function of V_n by Eq. 15 and 16. A summary of all the test walls' ratios of experimental to theoretical shear strength for existing design code expressions [17,18,19] are presented in Fig. 4 compared with the solutions from NSSSE. It is not an easy task to directly compare each of these code expressions to NSSSE because of the way they were individually empirically calibrated. Since the applied axial load, and the steel and masonry experimental strengths were known for the walls, additional load or material strength reduction factors were not included in calculations. In addition, the shear strength reduction factors (ϕ) adopted by the MSJC and the NZS 4230 have also not been used so that the expressions may be compared to the CSA S304.1 and the Simplified NSSSE.

In general, for each individual test program, the Simplified NSSSE had an average $V_{Experimetnal}$ / V_{Theory} closer to 1.0 as well as a lower the coefficient of variation (C.O.V.) than that of other expressions. Of particular interest are the results from [12], which have been discounted in some more recent shear expression derivations simply because of abnormally high strengths. While the Simplified NSSSE also observed a relatively high average strength ratio of 1.18, it was substantially lower than the other shear strength expressions. The overall average results from the concrete block wall specimens indicate that the Simplified NSSSE had the lowest shear strength ratio of $V_{Experimetnal}$ / V_{Theory} = 1.16 and the lowest C.O.V. of 11.4%. The 99th percentile strength ratios for the simplified NSSSE expression was 0.86, indicating that no additional empirical reductions factors are required.

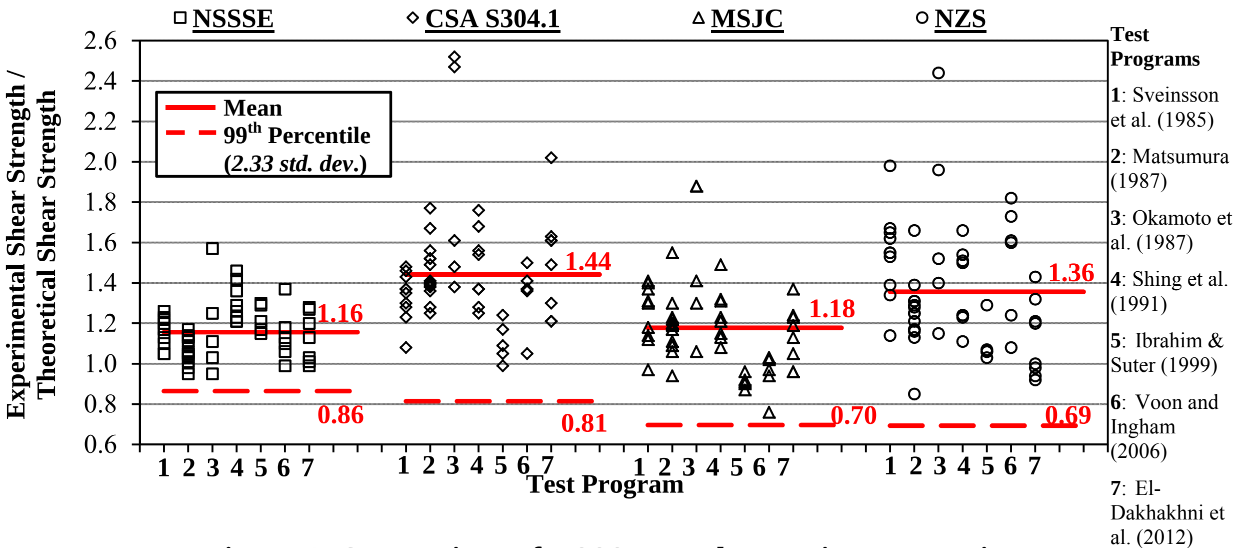


Figure 4: Comparison of NSSSE to other Design Expressions

CONCLUSIONS

Masonry has historically suffered from overly conservative shear design expressions that have been developed by empirical curve fitting of a relatively small experimental database. As such, empirical strength reduction factors were needed to ensure conservatism in design. To overcome the reliance on empirical reduction factors and arbitrary limits, a rational expression, derived from first principles, was developed in which constitutive masonry relationships are presented considering the composite and anisotropic nature of masonry. Overall the Simplified NSSSE provides a sufficiently conservative, more accurate and more precise prediction for the shear strength of RM structural walls and piers compared to current code expressions. The Simplified NSSSE also provides an engineering feel and physical sense of RM walls' characteristics, which would subsequently enhance the designers' confidence in their designs and facilitate better understanding of the factors influencing RM shear wall behavior.

ACKNOWLEDGEMENTS

Financial support has been provided by the McMaster University Centre for Effective Design of Structures (CEDS) funded through the Ontario Research and Development Challenge Fund (ORDCF) as well as the Natural Sciences and Engineering Research Council (NSERC) of Canada. The continuous support of the McMaster Masonry Research Group by the Ontario Masonry Contractors Association (OMCA), the Canada Masonry Design Centre (CMDC), and the Canadian Concrete Masonry Producers Association (CCMPA) is gratefully acknowledged.

REFERENCES

1. Vecchio, F. J. and Collins, M. P.(1982) "The response of reinforced concrete to in-plane shear and normal stresses." Publication No. 82-03, Department of Civil Engineering, University of Toronto, Canada.
2. Vecchio, F. J. and Collins, M. P. (1986). "The modified compression-field theory for reinforced concrete elements subjected to shear." ACI Journal, 83(2), 219-231.

3. Sarhat, S. R., and Sherwood, T. G. (2011). “Shear design of reinforced masonry beams.” Proc., 11th North American Masonry Conference, Minneapolis, USA, Paper 3.04-2 on USB Flash Drive.
4. El-Dakhankhni, W. W., Banting, B. R. and Miller S. C. (2012). “Seismic performance parameters quantification of shear-critical reinforced concrete masonry squat walls.” Submitted for publication, J. Struct. Eng.
5. Khattab, M. M. S. (1993). “In-plane behaviour of grouted concrete masonry under biaxial states of stress.” Ph. D. in Civil Engineering-Thesis, McMaster University, Hamilton, Canada.
6. Drysdale, R. G. and Khattab, M. M. (1995). “In-plane behavior of grouted concrete masonry under biaxial tension-compression.” ACI Journal, 92(6), 1-13.
7. Tikalsky, P. J., Atkinson, R. H. and Hammons M. I. (1995). “Compressive strength of reinforced masonry under lateral tension.” J. Struct. Eng., 121(2), 283-289.
8. Liu L., Tang, T. and Zhai X. (2006). “Failure criteria for grouted concrete block masonry under biaxial compression.” Advances in Structural Engineering, 9(2), 229-239.
9. Walraven, J. C. (1981). “Fundamental analysis of aggregate interlock.” Journal of the Structural Division, 107(11), 2245-2270.
10. Bentz, E. C., Vecchio, F. J. and Collins, M. P. (2006). “Simplified modified compression field theory for calculating shear strength of reinforced concrete elements.” ACI Journal, 103(4), 614-624.
11. Sveinsson, B. I., McNiven, H. D., and Sucuoglu, H. (1985). “Cyclic loading tests of masonry piers – Volume 4: Additional tests with height to width ratio of 1.” Report No. UCB/EERC-85-15, Earthquake Engineering Research Center, University of California Berkeley, USA.
12. Okamoto, S., Yamazaki, Y., Kaminosono, T., Teshigawara, M. and Hirashi, H. (1987). “Seismic capacity of reinforced masonry walls and beams.” Proceedings of the 18th Joint Meeting of the U.S.-Japan Cooperative Program in Natural Resource Panel on Wind and Seismic Effects: NBSIR 87-3540, National Institute of Standards and Technology, Gaithersburg, USA, 307-319.
13. Matsumura, A. (1987), “Shear strength of reinforced hollow unit masonry walls.” Proceedings of the 4th North America Masonry Conference, Paper No. 50, Los Angeles, USA, 50-1 - 50-16.
14. Shing, P. B., Noland, J. L., Spaech, H., Klamerus, E. and Schuller, M. (1991). “Response of single-storey reinforced masonry shear walls to in-plane lateral loads.” U.S. – Japan Coordinated Program for Masonry Building Research Report No. 3.1(a)-2, University of Colorado at Boulder, USA.
15. Ibrahim, K. S., and Suter, G. T. (1999). “Ductility of concrete masonry shear walls subjected to cyclic loading.” Proc., 8th North American Masonry Conference, Austin, Texas, USA, Paper 3.04-2 on USB Flash Drive.
16. Voon, K. C. and Ingham, J. M. (2006). “Experimental in-plane shear strength investigation of reinforced concrete masonry walls.” J. Struct. Eng., 132(3), 400-408.
17. Masonry Standards Joint Committee. (MSJC). (2011). “Building Code Requirements for Masonry Structures.” TMS 402/ASCE 5/ACI 530, The Masonry Society, American Society of Civil Engineers, Boulder, New York/American Concrete Institute, and Detroit, USA.

18. Canadian Standards Association (CSA). (2004). "Design of masonry structures." CSA S304.1-04 (R2010), CSA, Mississauga, Canada.
19. Standards Association of New Zealand (SANZ). (2004). "Design of reinforced concrete masonry structures." New Zealand Standards Association (NZS) 4230:2004, CEN, Wellington, New Zealand.