PREDICTING THE BEHAVIOUR AND DESIGN OF SPECIAL DUCTILE SHEAR WALLS WITH CONFINED BOUNDARY ELEMENTS

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

The lack of a Ductile Shear Wall category of seismic force resisting system (SFRS) from the current Canadian masonry structures design standard CSA 304.1 as well as National Building Code of Canada (NBCC) puts masonry construction at a competitive disadvantage with reinforced concrete (RC). The lack of any such SFRS arises from historical perceptions of masonry performance as well as known limitations in traditional wall systems. To address these issues an experimental program has been conducted at McMaster University towards quantifying the behavior of fully-grouted reinforced masonry (RM) walls with confined boundary elements which contain a double layer of vertical reinforcement with lateral confinement stirrups. A total of 10 walls have been tested under reversed cycles of lateral displacement. Based on the experimental data gathered, a series of prescriptive design requirements are proposed in this paper to estimate wall stiffness and yield displacement necessary to estimate seismic demands. The necessary detailing and anticipated stress-strain behavior of the boundary element is also established. To facilitate construction the use of a new type of pilaster unit is proposed that will permit proper detailing and inspection of the reinforcement within a confined boundary element.

KEYWORDS: boundary elements, confinement, design codes, seismic design, shear walls

INTRODUCTION

The analysis to be presented in this paper is derived from three experimental programs which collectively reported the results of eleven half-scale concrete block structural walls tested under reversed cycles of quasi-static loading. Shedid et al. [1], reported on tests of two walls which were detailed with confined boundary elements to compare their performance with walls possessing a conventional rectangular cross-section as well as a small flange over two different wall heights. Banting and El-Dakahkhni [2] presented the results of three more walls tested to compare the effects of different levels of total applied axial load as well as changes in detailing, such as: the influence of inter-storey RC floors slabs and discontinuing the boundary element detail in upper stories of three storey walls. Finally, Banting and El-Dakhakhni [3] presented the results of five more walls with boundary elements that varied by their overall dimensions to compare walls that possess differing heights, lengths and aspect ratios as well as the reinforcement ratio in the web of the wall. All the test programs incorporated the same boundary element with the detailing depicted in Fig. 1. Their design was based on a RM column detailed for compression reinforcement [4], intended to improve in the stability of the compression region and to prevent buckling of the vertical reinforcement. The parameters that were compared between specimens are highlighted in Table 1 and include the wall height (h_w), wall length (ℓ_w), the height to length (aspect) ratio (A_R), the number of inter-story floor slabs ($IS^{\#}$), discontinuity of confinement detailing above the plastic hinge, the level of applied axial load (P_a), and the vertical reinforcement ratio (ρ_v). In addition, the horizontal reinforcement ratio in the plastic hinge region (ρ_h) is also given in Table 1, which was detailed to ensure a flexural failure of the walls. The same reinforcement bar sizes were used for all the walls, consisting of No. 10 bars (A_s = 100 mm^2, d_b = 11 mm) as the vertical reinforcement and horizontal reinforcement and lateral stirrups were comprised of D4 bars (A_s = 25.4 mm^2, d_b = 5.7 mm). The full experimental results for each wall can be found in the aforementioned references [1,2,3], however, the average peak load (Q_u), ultimate top wall drift associated with a drop in capacity to 80% Q_u (Δ_u) as well as the experimental displacement ductility (μ_Δ).

Table 1: Confined Boundary Element Wall Test Matrix and Experimental Results

	Wall									
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
h_w (mm)	**1,900**	**2,660**	2,660	**3,990**	2,660	3,990	3,990	3,990	3,990	3,990
ℓ_w (mm)	**1,235**	1,235	1,235	1,235	**1,805**	1,805	1,805	1,805	1,805	**2,665**
A_R	**1.53**	2.15	2.15	3.23	**1.48**	2.21	2.21	2.21	2.21	**1.5**
ρ_v (%)	0.69	**0.69**	**1.17**	0.69	0.56	0.56	0.56	0.56	0.56	0.51
ρ_h (%)	0.6	0.6	0.6	0.3	0.6	0.3	0.3	0.3	0.3	0.3
P_a (MPa)	0.89	0.89	0.89	0.89	0.89	**0.89**	**0.45**	0.45	**1.34**	0.89
$IS\#$	0	1 1		2	1	2	**2**	**0**	2	0
Q_u (kN)	179.2	129.7	177.6	92.7	238.7	155	142.8	141.0	203.4	308
Δ_u (%)	2.20%	1.78%	1.98%	3.36%	2.19%	2.37%	3.03%	3.11%	1.82%	1.54%
μ_Δ	7.1	5.9	5.5	8.6	14.6	10.3	12.1	13.0	6.7	9.1

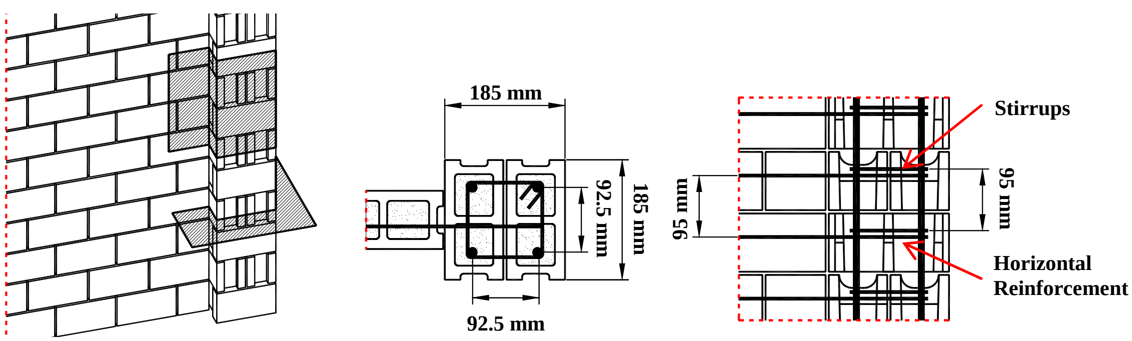

Figure 1: Confined Boundary Element Detailing of Half-Scale Walls

CONFINED BOUNDARY ELEMENT DETAILING

It has been established that confined masonry typically does not result in the same gains in strength levels that can be achieved with confined concrete [5,6]. However, it can be expected that softening of the descending branch of the stress-strain curve of confined masonry is possible such that an increase in the ultimate strain (ε_{mu}) used for ductility calculations is structural walls is justifiable. Tests on masonry prisms containing lateral confinement ties by Hart et al. [7] reported values of ε_{mu} = 0.0031 and ε_{mu} = 0.0043 for 200 mm and 100 mm tie spacing for a single layer of vertical reinforcement. Until recently, the Uniform Building Code [8] had specified a usable strain of masonry of masonry walls of ε_{mu} = 0.006 when they were confined by at least #3 ties at no more than 8” (200 mm) spacing. More recently, Shedid et al. [9] reported on compressive test results of 4-course masonry prisms, comprised of two stretcher units laid side by side in a square cross-sectional shape. It was reported that the addition of the stirrups around vertical rebar had the effect of softening the descending branch of the stress-strain relationship resulting in an average strain at 30% degradation in peak stress to be 0.0040, which represented a 51% increase over unreinforced prisms. Currently, the MSJC 2011 [10] allows increased values of ε_{mu} to be used in design when masonry is confined. However, it refrains from specifying any prescriptive confinement detailing method. Instead, this is left to the designer to verify such values through testing.

Due to the lack of any prescriptive detailing available for masonry confined with lateral stirrups, the requirements set out by the concrete structures design standard CSA A23.3 for design of ductile RC walls [11] has been selected as a basis to theoretically estimate ε_{mu} in confined boundary elements. The masonry confined boundary element is considered as the *area of concentrated reinforcement* with an effective gross length (ℓ_b) = 185 mm and a width (b_b) = 185 mm for an overall net area (A_g) = 32,610 mm^2 (after accounting for the loss of area from the frogged ends of the units). Four No. 10 vertical reinforcement bars are placed in the centre of the open webs of the overlapping units, which results in a centre-to-centre spacing of 92.5 mm. Lateral stirrups bent from the D4 bars into square stirrups were placed around the vertical bars. The outside-to-outside dimension of the confined core is therefore calculated as: 92.5 mm + (d_b of No. 10) + (2 × d_b of D4) = 115 mm, which represents a confined area (A_c) = 13,225 mm^2, equivalent to 38.6% of the gross boundary element area. Stirrups were placed at each course with a spacing (s_s) = 95 mm and, thus, ε_{mu} can be estimated from Eq. 2.2 derived from the CSA A23.3 [11]. A_s is the area of reinforcement of the stirrup along either axis (50.8 mm^2), h_c is the dimension of the confined core (115 mm) and k_n is a factor accounting for the number of bars in contact with the stirrup (for a square stirrup of four bars, k_n = 2.0 [11]).

$$\varepsilon_{mu} = \frac{A_s A_c f_{y,h}}{15 k_n A_g f'_m s_s h_c} - \frac{1}{300} \tag{1}$$

Eq. 1 can be solved with a f'_m = 12.7, as was reported by Shedid et al. [9], to yield a value of ε_{mu} = 0.0039. Comparing the results of Eq. 1 with those reported by Shedid et al. [9] indicates that Eq. 1 provides a reasonably conservative result amenable to the reported data. Finally, as with RC design, the value of ε_{mu} determined from Eq. 1 would have to be limited within the confined boundary element of a wall to prevent against web crushing. Thus a check is required to ensure that the unconfined region is not subject to strains in excess of the unconfined strain limit ε_{mu} = 0.0025 as verified from similar triangles in Eq. 2.

$$\frac{\ell_c}{c} \geq \frac{\varepsilon_{mu} - 0.0025}{\varepsilon_{mu}} \tag{2}$$

Ultimately, the appropriate value of ε_{mu} for design of structural walls must be made with consideration of actual wall behavior with regard to displacement calculations since ε_{mu} must also be considered to act over an equivalent plastic hinge region, rather than just a the base of a wall. In conclusion, it is evident that the thickened boundary element would reduce and delay the typical failure mechanisms observed in traditional single wythe grouted masonry, whether unreinforced or reinforced. In the following sections, the appropriate value of ε_{mu} as it pertains to predicting wall behavior will be validated as it relates to actual wall test data.

EFFECTIVE ELASTIC STIFFNESS

The theoretical elastic stiffness (K_{gt}), and thus natural period, of a structure is dependent of the stiffness of individual walls that comprise the SFRS. The elastic stiffness of a cantilever RM wall subject to a point load at its top considering both flexure and shear deformations can be determined according to Eq. 3.

$$K_g = \frac{1}{\frac{h_w^3}{3E_m I_e} + \frac{kh_w}{0.4E_m A_e}} \tag{3}$$

Whereby, I_e is the effective moment of inertia of the wall reduced from the gross member properties (I_g) such as to consider the effects of cracking (m^4), E_m is the Young's modulus of the masonry in MPa, A_e is the effective area of the member (m^2), reduced from the effective gross area (A_g) in shear when cracking occurs (m^2), and k is a shape factor accounting for the distribution of shear stresses across a cross section. The value of k is typically taken as 1.2 for rectangular cross-sections, however, none of the walls tested meet this criteria. Therefore, adopting Hooke's law and assuming an isometric elastic material, k can be estimated with Eq. 4.

$$k = \frac{A_g}{I_g^2} \iint \frac{Q_m^2}{t^2} dxdz \tag{4}$$

Where, Q_m is the first moment of area and t is the width of the cross-section. Since Q_m and t_w are discontinuous along the wall cross-section due to the protruding boundary elements, solution to the double integral in Eq. 4 becomes quite cumbersome. Nevertheless, k can be solved for with Eq. 4 as 1.44, 1.35 and 1.29 for wall lengths of 1,235 mm, 1,805 mm and 2,660 mm, respectively. The theoretical gross section stiffness for flexure (K_{ft}) and shear (K_{st}) are given in Table 2 along with the total theoretical elastic stiffness (K_{gt}) determined from Eq. 3. Furthermore, a simplified approach to estimate the reduced cross-section stiffness is suggested by Paulay and Priestley [12] as well as in the Canadian concrete structures design code CSA A23.3 [11] based on the gross-section properties. The former suggests a reduction factor (α) given by Eq. 5 while the latter is based on an upper and lower bound of α given by Eqs. 6 and 7, respectively, where $I_e = \alpha I_g$ and $A_e = \alpha A_g$.

$$\alpha = \left(\frac{100}{f_y} + \frac{P_a}{f'_m A_g} \right) \tag{5}$$

$$\alpha = \left(0.6 + \frac{P_a}{f'_m A_g} \leq 1.0 \right) \tag{6}$$

$$\alpha = \left(0.2 + 2.5 \frac{P_a}{f'_m A_g} \leq 0.7 \right) \quad (7)$$

Whereby, the yield strength of the vertical reinforcement (f_y) is given in MPa, the total applied axial load on the wall (P_a) is given in MN and the compressive prism strength of the masonry (f'_m) is given in MPa and the gross area of the wall cross-section (A_g) is given in m^2. The theoretical stiffness of the cracked section representing an effective yield stiffness (K_{yt}) is determined with Eq. 5, 6 and 7 for each wall is presented in Table 2.

Table 2 – Theoretical Effective Elastic Stiffness

	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	Wall 7	Wall 8	Wall 9	Wall 10
	Stiffness in kN/mm									
K_{gt}	72.1	31.3	31.3	10.4	105.0	30.2	28.4	28.4	28.4	65.8
K_{y5}	18.6	8.1	8.1	2.7	27.0	7.8	6.5	6.6	8.5	17.0
K_{y6}	47.5	20.6	20.6	6.9	69.0	19.8	17.9	18.0	19.8	43.3
K_{y7}	24.9	10.8	10.8	3.6	36.0	10.3	7.8	8.0	12.6	22.7

The average ratio of experimental uncracked gross stiffness to theoretical stiffness K_{gt} was determined as to be 1.44 (c.o.v. = 36.3%), representing a significant variation between observed and predicted stiffness of the walls prior to cracking. However, it is speculated that this likely due to the sensitivity of the instrumentation used to measure lateral displacements as well as the difficulty in establishing when cracked behavior occurs. Nevertheless, the use of Eq. 7 proved to be an accurate means of estimating the effective yield stiffness, yielding a ratio of theoretical to experimental yield stiffness of 1.21 (c.o.v. = 10.3%) which is reasonable for 'back of the envelope' calculations given the simplification of the approach. By normalizing the lateral stiffness degradation (K) of the walls with increased top drift by the theoretical yield stiffness K_{y7}, further defined as simply K_y, scatter in the behavior of the walls is significantly reduced, as evidenced by Fig. 2a for wall drift, and in Fig. 2b for the idealized ductility.

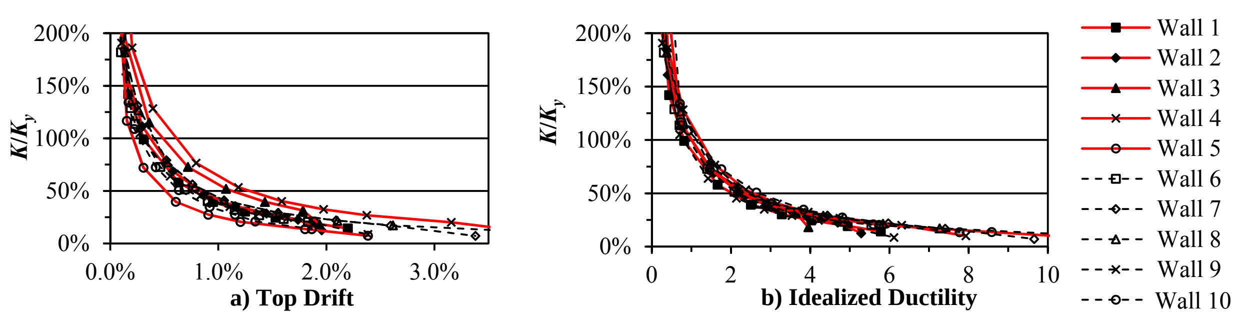

Figure 2: Normalized Experimental Stiffness (K) by Theoretical Yield Stiffness (K_y) versus: a) Top Drift and b) Idealized Displacement Ductility

EFFECTIVE YIELD DISPLACEMENT

For design purposes and initial wall sizing, it is useful to have an estimate of the yield displacement without requiring the extraneous work of conducting a thorough push-over analysis needed to solve for the idealized displacement ductility Δ_y* defined by [13] when experimental data is not available. Therefore, three theoretical values for the idealized yield displacement, necessary for ductility calculations, are proposed in this section which will be compared to the actual experimentally determined values.

The first simplified estimate of the experimental effective yield displacement Δ_y*, given in Table 3, is referred to as Δ_{y1}*, can be arrived at based on the previously selected theoretical yield stiffness, K_y, which passes through the experimental yield displacement at the experimental yield load (Q_{ye}). On average, the ratio of Q_{ye} to the ultimate wall strength (Q_{ue}) was 74.8% (c.o.v. = 5.5%). Therefore, Δ_{y1}* can be solved for as: $\Delta_{y1}* = Q_{ue}/K_{y1}$, as given in Table 3, based on a bilinear interpretation of the yield displacement or the walls. This gives a reasonable and conservative estimate of Δ_{y1}*, with a ratio to the experimentally determined idealized yield displacement of 90.6% (c.o.v. = 16.5%). Alternatively, Priestley et al. [13] provides an even simpler means to estimate Δ_y* based on a yield curvature equal to 2.10 ε_y/ℓ_w, where ε_y is the yield strain of the vertical reinforcement of 0.0025 for these test specimens. The resulting idealized yield displacement is thus determined as Δ_{y2}* and given in Table 3 with a resulting ratio to the experimental value of 100.8 % (c.o. v. = 18.7%). The final estimate of Δ_{y3}* given in Table 3 is based on a push-over analysis to determine the point where the seel first yields, and then amplifying this by the ratio of yield strength to ultimate strength of 75% resulting in a ratio to the experimental value of 111.3 % (c.o.v. = 16.5%). In conclusion, for the extra effort required to estimate the yield displacement considering push-over analysis there is little benefit over adopting a more simplified approach such as given by Δ_{y1}* *and* Δ_{y2}*.

Table 3: Theoretical Estimates of the Effective Yield Displacement

	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	Wall 7	Wall 8	Wall 9	Wall 10
	Effective Yield Displacement as Top Drift									
Δ_y*	0.38%	0.37%	0.49%	0.50%	0.21%	0.33%	0.35%	0.36%	0.39%	0.42%
Δ_{y1}*	0.41%	0.43%	0.55%	0.64%	0.25%	0.40%	0.47%	0.35%	0.35%	0.35%
Δ_{y2}*	0.27%	0.38%	0.38%	0.57%	0.26%	0.39%	0.39%	0.39%	0.39%	0.39%
Δ_{y3}*	0.25%	0.35%	0.36%	0.52%	0.23%	0.35%	0.33%	0.33%	0.36%	0.33%

DESIGN STANDARD REQUIREMENTS OF MASONRY SHEAR WALLS WITH CONFINED BOUNDARY ELEMENTS

Based on the requirements for *Ductile Walls* in the CSA A23.3 RC Structures Design Code, the following modifications are suggested towards the prescriptive requirements for Special Ductile Masonry Shear Walls with Confined Boundary Elements. Firstly, one major difference between RC and RM shear wall detailing is the use of double leg stirrups as shear reinforcement and using two layers of vertical reinforcement within the web of the wall. Any amenable detailing within masonry would be very difficult to achieve within the realities of construction. Presently, the commentary of the American Concrete Institute (ACI) 318-11 Concrete Standard states that the requirements for two layers of web reinforcement stems from the observation that "*the probability of maintaining a single layer of reinforcement near the middle of the wall section is quite low*" [14]. Therefore, this requirement appears to be in place as a measure of quality control during construction or due to the fact that in most cases RC walls require double legged shear reinforcement to satisfy strength requirements, more so than for any strictly theoretical motivation. Such a concern is also minimized within masonry construction because of the nature of the units are such that rebar can seldom practically be placed close to the edge of the units. Within the CSA A23.3, the requirement for a double layer of vertical web reinforcement may be waived if the design shear force can entirely be resisted by the concrete strength, a similar requirement could be readily and conservatively applied to masonry as well.

Unlike their RC counter-part, RM shear walls with confined boundary elements would most likely be constructed using standard pilaster units (390 mm × 390 mm) at the wall ends as depicted in Fig. 3, thus resulting a barbell shaped cross-section. However, such a configuration would overcome the limitations in the stirrup spacing that would occur due to the presence of webs in typical units shown in Fig. 1.

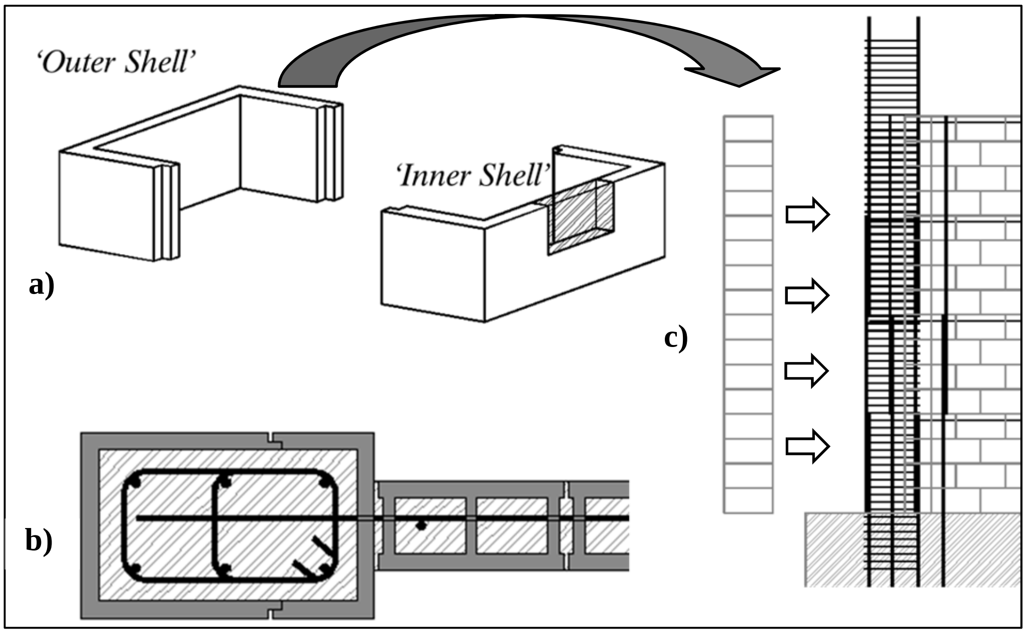

Figure 3: Pilaster Unit Alternative to Confined Boundary Element: a) Interlocking Pilaster Block, b) Confined Cage Placed within and c) Cage in Place in Wall to Allow Ties, Inspection and Finally Placement of Outer Shell

Regardless of the type of unit selected as the confined boundary element, it would have to permit shear reinforcement to pass through and develop its full strength as well create a continuous grout connection with the web of the wall. In addition to typical shear strength calculation, the connection between the wall and boundary element would also have to be designed to preserve shear flow in the wall as also typically required in flanged wall design. The use of a pilaster unit would give a designer greater flexibility to satisfy tie spacing requirements for buckling prevention of the reinforcement. Based on the requirements for RC design, the following tie spacing requirements are proposed:

Such that tie spacing shall not exceed the smallest of

a) six longitudinal bar diameters;
b) 24 tie diameters; or
c) one-half of the least dimension of the member

For a wall possessing a typical pilaster unit as a boundary element this would entail a minimum tie spacing of 68 mm (for 10M longitudinal bars as ties of at least 2.8 mm in diameter), 96 mm (15M bars with at least 4 mm ties), 117 mm (20M bars with at least 4.9 mm ties) or 151 mm (25M bars with at least 7.5 mm ties). Given these requirements for buckling prevention, it is obvious that open units or units with severely depressed webs would be required in the confined boundary element. Consequently, as allowed by the CSA A23.3, wire mesh reinforcement could also be used based on an equivalent area. Other than these identified areas, the same prescriptive design details could be followed as used in RC Ductile Shear wall design within a RM Special Ductile Shear wall with confined boundary elements.

CONCLUSIONS

Estimates of stiffness and displacement using existing expressions correlated well with measured parameters. These could be integrated within the prescriptive design requirements set out for a new category of masonry shear wall as a means estimating seismic demands. Finally, the practical obstacles of detailing a confining boundary element containing lateral ties around multiple layers of vertical reinforcement were also described. The requirements currently applied within the design of reinforced concrete shear walls could be adapted within the context of masonry detailing. As indicated, this would likely require the adoption of pilaster of other custom units to permit tie spacing cot conducive with the modular height of the units. In addition, allowing for a highly ductile masonry shear wall category would also likely necessitate the need to greater scrutiny and oversight regarding reinforcement detailing. Therefore, a boundary element unit that could be placed after the reinforcement was tied as indicated in Fig. 3 is proposed.

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. Provision of mason time by Ontario Masonry Contractors Association (OMCA) and Canada Masonry Design Centre is appreciated. The supply of half-scale blocks by the Canadian Concrete Masonry Producers Association (CCMPA) is gratefully acknowledged.

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