



Influence of confinement reinforcement on the compression stress–strain of grouted reinforced concrete block masonry boundary elements

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ABSTRACT

Reinforced masonry (RM) shear walls are widely used in medium- to high-rise masonry buildings as part of the lateral force resisting system to provide the lateral strength, stiffness and energy dissipation capacity required to resist lateral loads arising from earthquakes or wind. In the past few decades, there has been considerable advancement in the design of RM shear walls for new construction with variable types of confinement of the compressed boundaries of the wall for increasing ductility level. Recent codes and standards for design of masonry structures are introducing the use of Ductile Reinforced Masonry (DRM) Shear Walls with column-like boundary elements for the improvement of the ductility capacity of the walls. A key component in the evaluation of the ductility capacity of shear walls is evaluating the compressive response of their boundary elements. This paper presents an experimental and analytical investigation on the compression stress–strain behavior of fully grouted unconfined and confined reinforced concrete block masonry boundary elements. Full scale boundary elements were constructed and tested under concentric axial compression. An analytical stress–strain model was proposed that can be used in the design and assessment of DRM walls with boundary elements.

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1. Introduction

Moderately ductile reinforced masonry (RM) shear walls are considered an economical seismic force resisting system for masonry buildings in low- to moderate-seismicity zones. However, in regions with high seismicity, an economical RM shear wall system necessitates achieving higher ductility levels. Because RM shear walls are expected to undergo high inelastic response during severe ground motions, special consideration must be given to detailing of the horizontal and vertical reinforcement, especially at the ends of such walls in order to resist the high curvature ductility demands. Confinement of the wall end section is an efficient approach to enhance the curvature ductility capacity in RM shear walls. A key component in the evaluation of the ductility capacity of RM walls is the evaluation of compression stress–strain behavior of the confined ends. A number of different methods of confinement have been studied for application to masonry, including providing steel plates and seismic combs in the bed joints, steel rings and spirals around vertical reinforcing bars within the block cells and more recently the addition of confined boundary elements (Fig. 1).

Priestley and Elder [2] investigated the compressive stress–strain characteristics of reinforced concrete masonry containing steel confinement plates placed in the mortar bed joints. The researchers concluded

that the confinement plates produced a more gradual failure and improved the ductility of the concrete masonry prisms. Hart et al. [3] studied confinement reinforcement in concrete masonry prisms using seven different types of steel confinement reinforcement including Priestley plates, ties, closed wire mesh, seismic combs, steel ring cages, spirals, and spiral cages. The researchers concluded that confinement improved the displacement ductility and decreased the slope of the post-peak portion of the compressive stress–strain curve. Shing et al. [5] presented experimental work with prisms and walls demonstrating that ring, comb and spiral steel confinement reinforcement within the grouted cells of masonry units was effective in increasing the ultimate strain of the descending branch in masonry. Dhanasekar and Shrive [7] employed welded wire steel meshes to unreinforced masonry prisms which resulted in increased strength and enhanced post-peak behavior. Laursen and Ingham [6] integrated steel confinement plates for post-tensioned concrete masonry walls. More recently, Shedid et al. [1] and Banting and El-Dakhkhni [8] integrated confined boundary elements in the form of thickened wall ends that contained a double row of vertical reinforcement with closed hoops to improve the ductility and drift capacity of RM shear walls. Cyrier [9] investigated the ductility capacity of shear walls with integrated reinforced concrete boundary elements.

Recent codes and standards for design of masonry structures are introducing the use of Ductile Reinforced Masonry (DRM) Shear Walls with column-like boundary elements for the improvement of the ductility capacity of the walls. The 2013 Building Code Requirements and Specifications for Masonry Structures [10] allows the use of confined

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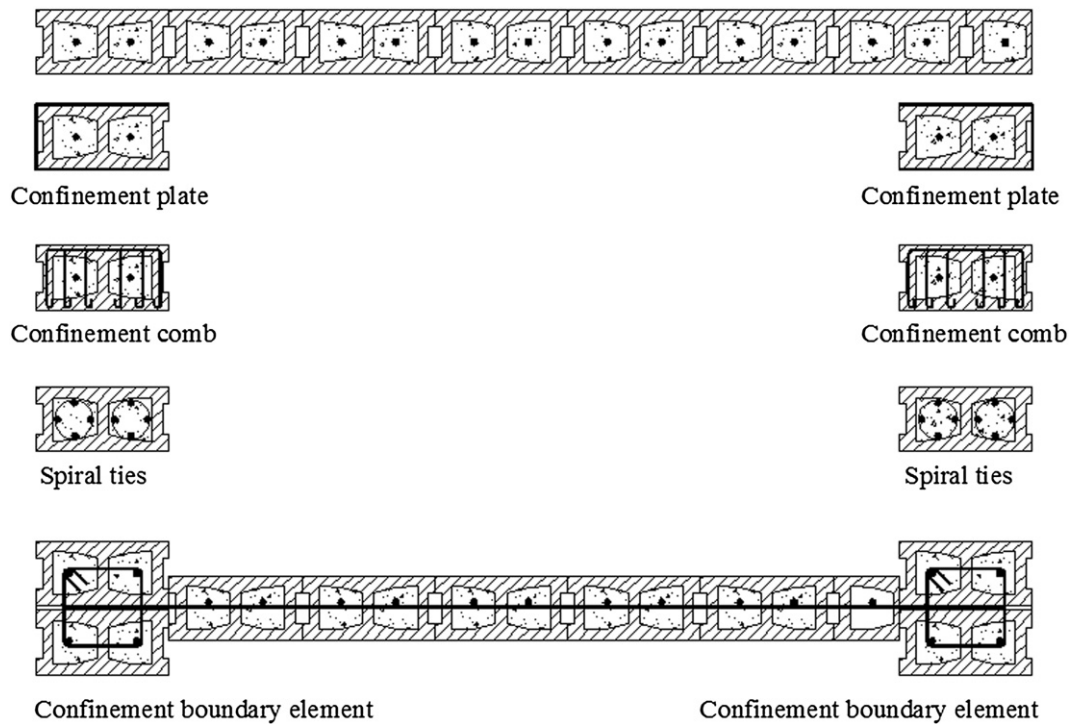


Fig. 1. Examples of confinement methods of reinforced masonry construction.

boundary elements by only imposing some geometrical rules but provides no guidelines for reinforcement detailing and requires that testing be conducted to verify that the detailing provided shall be capable of developing a strain capacity in the boundary element that would be in excess of the maximum imposed strain. The most recent Canadian Standard for the Design of Masonry Structures CSA S304-14 [12] also allows the use of confined boundary elements with prescriptive detailing requirements. However, these detailing requirements need more

experimental verifications for reliable application to RM shear walls. Such experimental studies are scarce in the literature.

This paper presents an experimental and analytical investigation on the compression stress–strain behavior of confined reinforced concrete block masonry boundary elements. Axial compression tests are conducted on 17 full-scale unconfined and confined boundary element columns with variable confinement ratios. A simplified analytical stress–strain model is proposed for unconfined and confined boundary

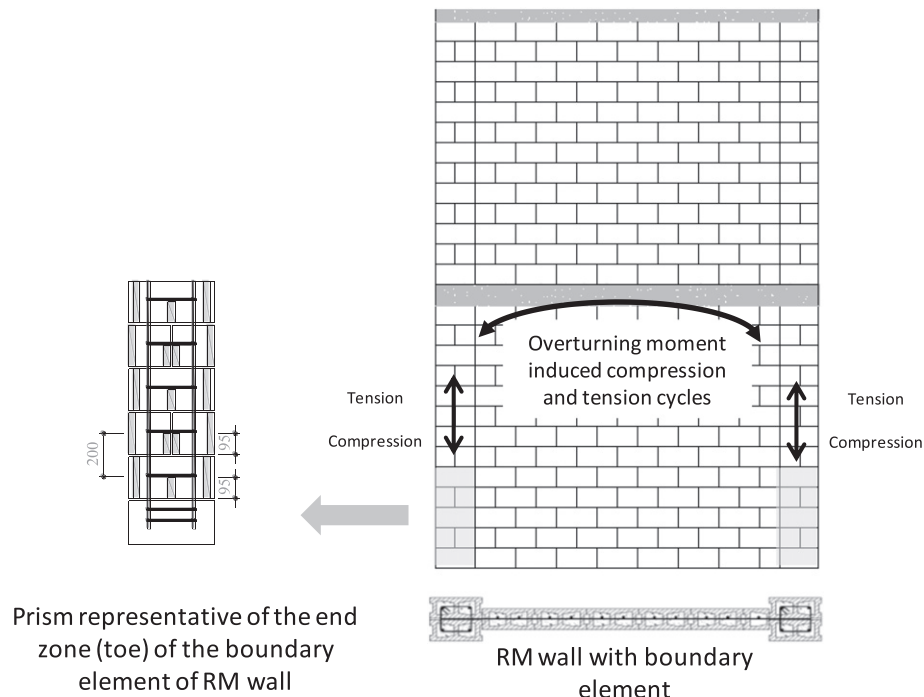


Fig. 2. Illustration of a RM wall with confined boundary elements and the tested prism that is representative of the end zone (toe) of the wall.

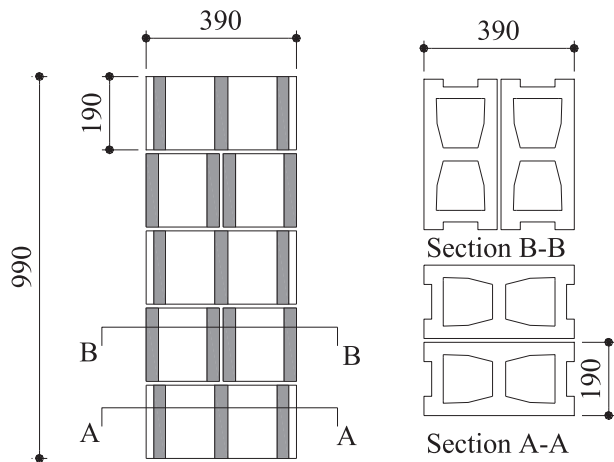


Fig. 3. Layout of the unreinforced concrete block masonry boundary element columns.

elements that can be used in the design and assessment of RM walls with boundary elements.

2. Test program

The main objective of this study is to provide necessary experimental support and verification for the confinement effects of the boundary element region located at the highly compressed end zone (toe of the wall) of flexural dominated RM walls. This objective was achieved through the evaluation of the compression stress–strain behavior of isolated boundary element prisms representative of highly compressed end zone. At the end zone, the boundary elements are subjected to cycles of axial compression and tension loading from the seismic induced overturning moment (Fig. 2). Therefore, confinement is required at the boundary elements in order to maintain the stability of the highly compressed boundary element, improve the strain and curvature ductility

as well as prevent out of plane buckling. More specifically, this study provided experimental evidence for the potential of improving the compression strain capacity of the boundary element. The compression strain capacity of the boundary element is a key parameter in the estimation of the curvature and rotation ductility of the walls to satisfy a specific design level of curvature demand in the most recent North American codes for design masonry structures [10,12].

A total of 17 full-scale fully grouted concrete masonry boundary element columns were constructed and tested that are representative of the end zone (toe) of RM walls. The boundary elements were constructed using standard concrete blocks (with actual width of 190 mm) stretcher units with 15 MPa nominal compressive strength. Typical dimensions of the unreinforced boundary element columns are shown in Fig. 3. Each boundary element column consists of 5 block layers with 2 blocks in each layer placed in alternating directions along the height. The blocks are joined together with 10 mm mortar joints. Fig. 4 shows the construction details for the reinforced and confined columns. All reinforced columns have an internal vertical reinforcement of 4 bars with nominal diameter of 19.5 mm (20 M). The test variables are the size and spacing of the transverse reinforcement (ties detailed as seismic hoops). A test group consisting of only vertical reinforcement bars is considered. The different sizes of the ties are varied between 10M and 15M bars and the spacing between 2 consecutive ties is alternate between 100 mm and 200 mm as shown in Table 1. The confinement ratio C_f in Table 1 is defined as $C_f = \rho_s \sqrt{H/S}$, where ρ_s is the volumetric ratio of the confinement reinforcement (the volume of confinement ties to the volume of core concrete at spacing S); H is the width of the confined core (185 mm in this case) and S is the spacing between the confinement ties.

Requirements for the minimum amount of vertical and transverse reinforcement in the boundary elements are provided in the CSA S304-14 [12]. The requirements for reinforcement of boundary elements in the CSA S304-14 [12] are not related to specific material strength parameters. On the other hand, the required detailing is based on the need to satisfy ductility requirement in the case that the seismic induced strain demand at the end of the RM wall is more than 0.0025. However,

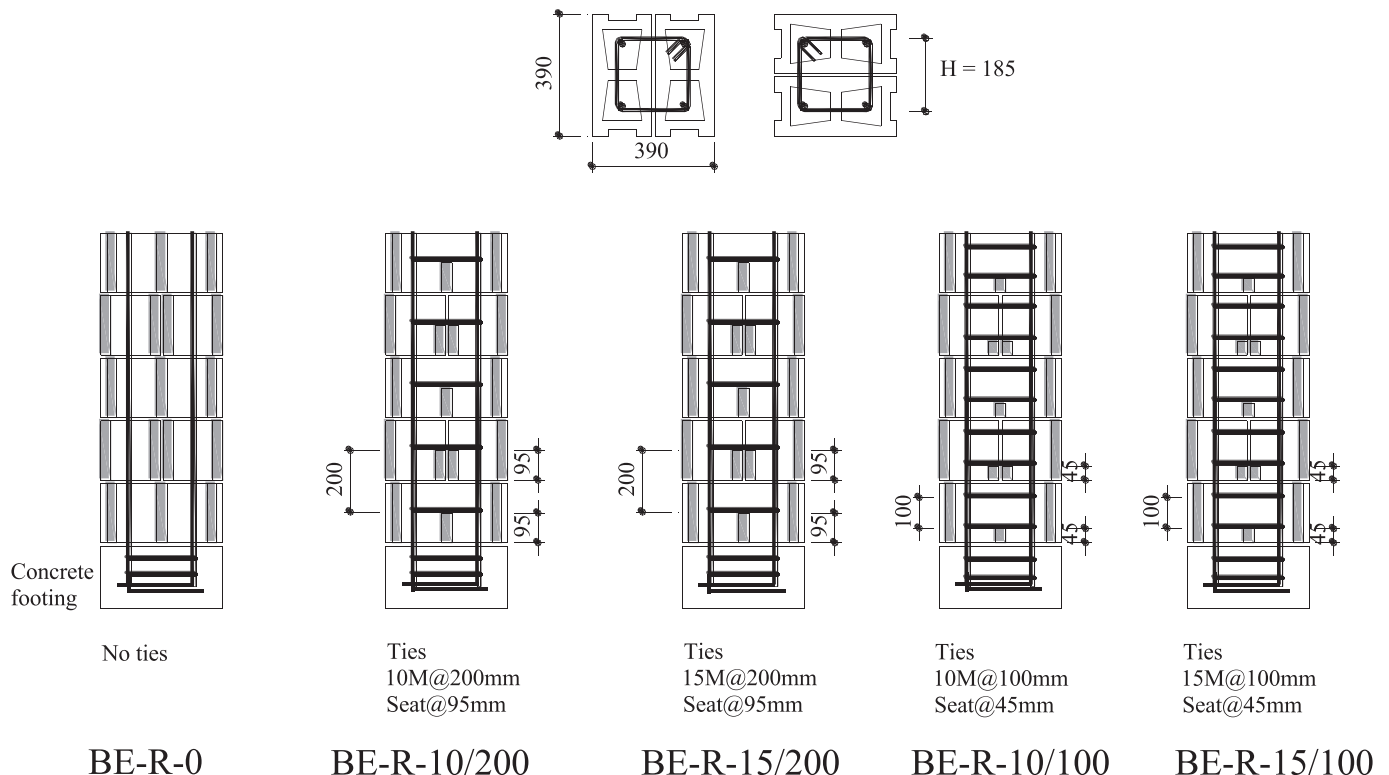


Fig. 4. Details for reinforced concrete block masonry boundary elements.

Table 1

Test matrix for concrete block masonry boundary elements.

Boundary element ID	Number of tested units	Vertical reinforcement	Transverse reinforcement	Volumetric ratio of transverse reinforcement ρ_s %	Confinement ratio C_f
BE-U-0	3	–	–	0	0
BE-R-0	2	4–20 M	–	0	0
BE-R-10/200	3	4–20 M	10 M@200 mm	0.008	0.008
BE-R-15/200	3	4–20 M	15 M@200 mm	0.018	0.019
BE-R-10/100	3	4–20 M	10 M@100 mm	0.018	0.023
BE-R-15/100	3	4–20 M	15 M@100 mm	0.038	0.051

the code imposed a limit on the strain capacity to be less than 0.008 and did not provide any guidelines to correlate specific detailing to confinement ratio and to the expected strain capacity of the boundary element. Therefore, in this study, the choices of the transverse reinforcement were varied to cover realistic confinement configurations for RM construction. The vertical reinforcement ratio and distribution satisfy the CSA S304-14 [12] requirements (Clause 16.10.2). All confined boundary elements (i.e. unit's BE-R-10/200, BE-R-15/200, BE-R-10/100, BE-R-15/100) satisfy tie requirements for compression reinforcement in the CSA S304-14 [12] (Clause 12.2.1).

The construction of the boundary element columns was conducted by certified masons. Type S-mortar prepared according to CSA A179 [11] was used between the different courses along the height of the columns with average thickness of 10 mm. The average compressive strength of the mortar was 12.9 MPa. The boundary element columns were then grouted with fine grout mixed according to the CSA A179 [11] with an average compressive strength of 25.6 MPa. The reinforcement bars used were grade 400W according to CSA G30.18 [15] with average yield strength of 400 MPa. In order to place the ties inside the concrete block columns, the blocks were cut to allow the ties to pass through them. The cuts were made according to the spacing between

the ties. For a tie spacing of 100 mm, the blocks were cut along their depth to a distance of 145 mm leaving only a 45 mm seat for the tie to rest on. For a tie spacing of 200 mm, the blocks were cut to a depth of 95 mm leaving another 95 mm as a seat for the ties (Fig. 5). Fig. 6 shows the placement of the ties and blocks of the boundary element. It should be noted that the cutting procedure might have caused a reduction in the stiffness and strength of the concrete blocks, which would then be expected to have an impact on the compressive strength of the boundary element as will be presented in the next sections.

Fig. 7 shows illustrations of the sequence of construction of the boundary element as follows: (a) the reinforced concrete footing is cast with required four (or more) vertical bars; (b) mortar is placed on the top of the footing and the first two blocks are added. It should be noted that one corner of the face-shell and the central web were removed from each block (by saw cutting) to accommodate the placement of the ties (as shown with the gray color in Fig. 7(b)); (c) ties are then placed on the seat that were formed by two blocks and are connected to the vertical bars; (d) and (e) represent the second round of the construction by placing the blocks and then the ties. The main difficulties that are related to the construction of this type of boundary elements are as follows: (a) the first difficulty is the process of cutting

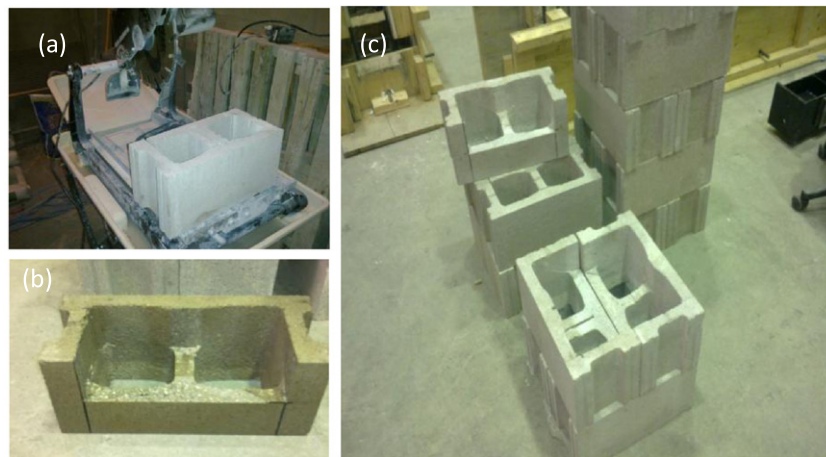


Fig. 5. Cutting procedure of the concrete blocks: (a) the concrete block in its original shape; (b) the concrete block after cutting the face-shell and the middle web; (c) the concrete blocks after cutting to form the required seat of the ties.

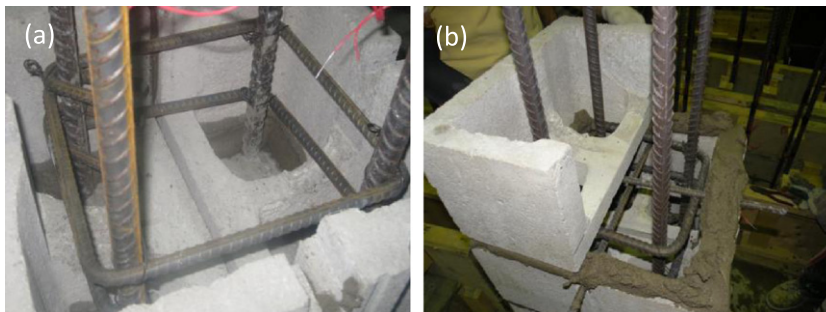


Fig. 6. Illustration of the placement of the ties and blocks in the confined boundary element.

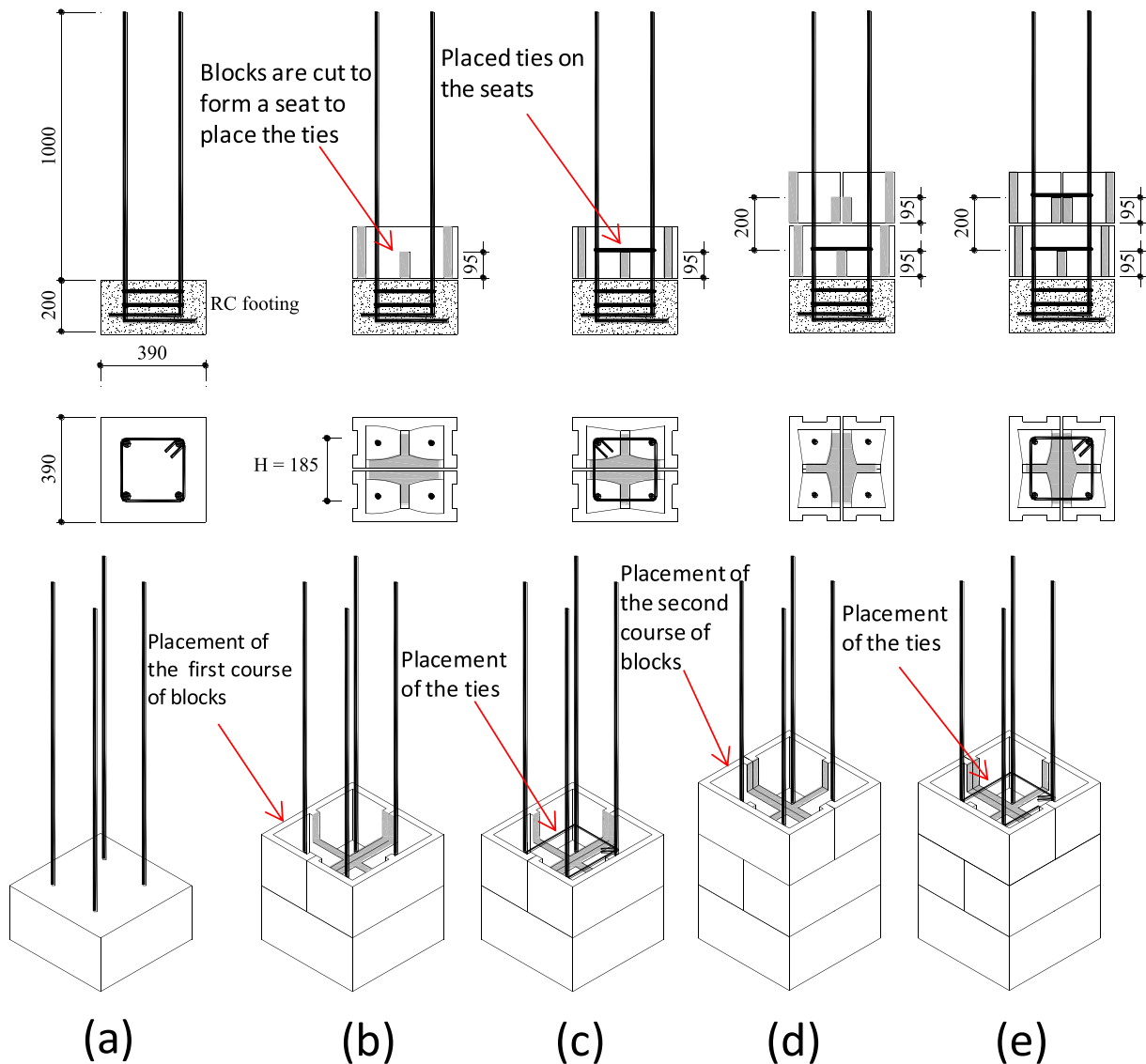


Fig. 7. Illustration of the sequence of construction of the boundary elements.

the blocks to remove parts of one face-shell and the central web to prepare the seat where the tie will be placed. This process requires careful handling of the blocks. Alternatively, block manufacturers could consider the feasibility of producing special pre-cut blocks to be used in the construction of such boundary elements; (b) the second difficulty is related to the sequential process of placement and attaching the ties after the construction of each course of blocks which requires careful management with the masons during the construction process. However, this is somehow similar to the general procedure for placement of horizontal reinforcement in any RM wall construction.

3. Test setup and instrumentation

Fig. 8 shows the test setup for the compression tests on the confined boundary element columns. the test setup consists of: a reaction steel frame that transfers the load to the strong floor system; a 3000 kN hydraulic cylinder for the application of the axial compression force on the test units; a rigid steel beam and a rigid rectangular steel plate for uniform distribution of the axial load to the test units and a reinforced concrete footing supporting the test units. The loading cylinder is connected to a motor hydraulic pump for the control of the loading

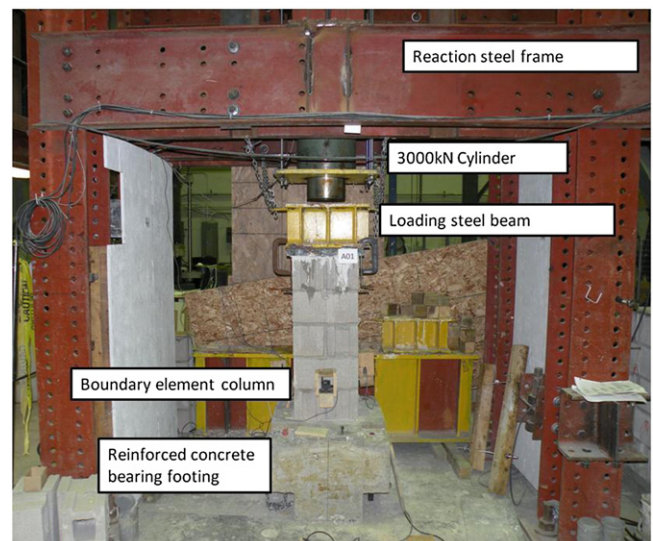


Fig. 8. Photograph illustration of the steel testing frame and the loading system.



Fig. 9. Observed failure mechanism of the unreinforced boundary element column.

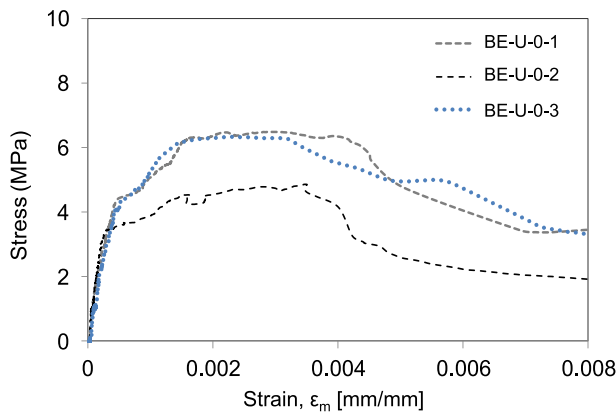


Fig. 10. Observed stress–strain curves for the unreinforced concrete masonry boundary element columns.

rate and the measurements of the load were recorded from the load cell attached to the cylinder. The vertical displacement of the boundary element columns was measured using 4 cable-extension transducers (potentiometers) attached at the four sides of the unit using 10M pre-embedded through bars in order to measure the continuous deformation of the units in the pre-peak and the post-peak loading range. The gauge length for the potentiometers was 600 mm measuring between the three middle blocks.

4. Test results for the unreinforced boundary elements (units BE-U-0)

Fig. 9 shows the observed failure mechanism of the unreinforced boundary element column at the end of the test. The failure started with vertical cracks after reaching maximum load followed by spalling of the face-shells and end webs initiated by the expansion of the grouted core columns. Fig. 10 shows the average observed stress–strain relationship for the unreinforced concrete block boundary element columns (test units BE-U-0). It can be observed that after a first cracking was recorded, the boundary elements continued to gain strength with reduced stiffness until reaching the peak strength. Contrary to typical unreinforced masonry prism tests with stacked concrete blocks, a sudden drop in the strength was not observed and a descending branch was then recorded with a mild slope until reaching 50% reduction of strength with an average strain of 0.005. The average maximum compressive strength was 5.6 MPa at an average strain of 0.002. On the other hand, the boundary element configuration has a significant effect on the post-peak behavior with softening the descending branch of the stress–strain curve after reaching the peak strength. Additionally, the increased value of the ultimate strain (50% reduction of strength) is of the order of 1.5 times the typically observed ultimate strain for unreinforced masonry prisms (0.003). It would be expected that the addition of horizontal reinforcement ties at the grouted core will lead to increased ultimate strain of the confined region as presented in the next section.

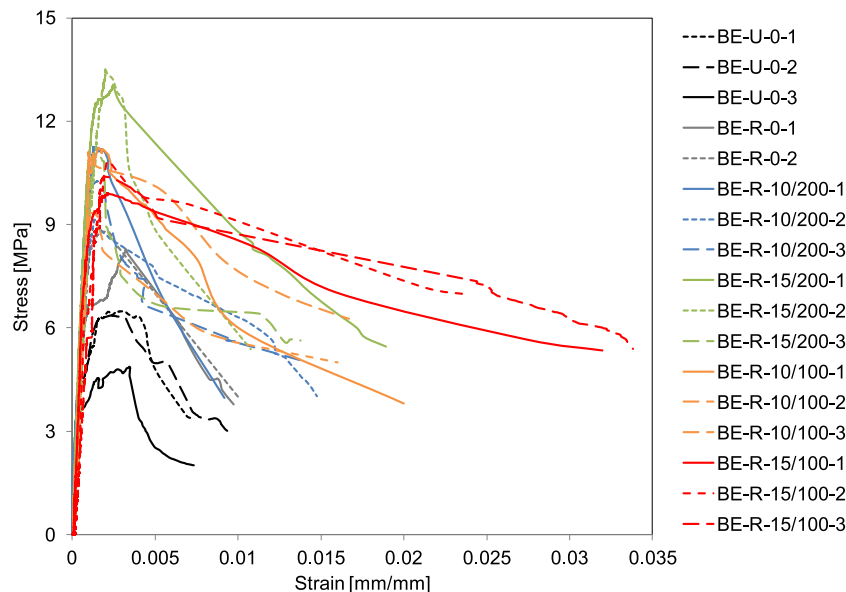


Fig. 11. Observed stress–strain curves for all test units including unconfined, reinforced and confined boundary elements.

5. Test results for reinforced boundary elements (units BE-R)

Fig. 11 shows the observed stress–strain curves for all test units including unreinforced, unconfined reinforced and confined reinforced boundary elements. It can be observed that the reinforcement and confinement improved the axial compression capacity and the strain ductility of the boundary elements. The compression strength of the confined boundary elements recorded an increase ranging from 1.6 to 2.4 times larger than the strength of unconfined boundary elements. Moreover, the confinement reinforcement improved the strain ductility of the boundary elements and induced more gentle strain softening slope with the increase of the confinement ratio. The strain at 50% strength degradation of the confined boundary elements recorded an increase

ranging from 2.2 to 7 times larger than the corresponding strain for the unconfined boundary elements.

Fig. 12 shows the observed axial compression stress–strain relationships for separated test groups of reinforced concrete block boundary element test units. Table 2 shows the results of the maximum observed stress f_{max} , strain at maximum stress ϵ_{max} , ultimate strain at 80% strength degradation ϵ_{80} and the ultimate strain at 50% strength degradation ϵ_{50} . Fig. 13 shows the failure mechanisms at the end of the tests for all columns. The following observations were made based on the obtained stress–strain behavior of the reinforced boundary elements:

- For test units BE-R-0, the behavior was similar in the ascending branch of the stress–strain curve to the unreinforced units (BE-U-0)

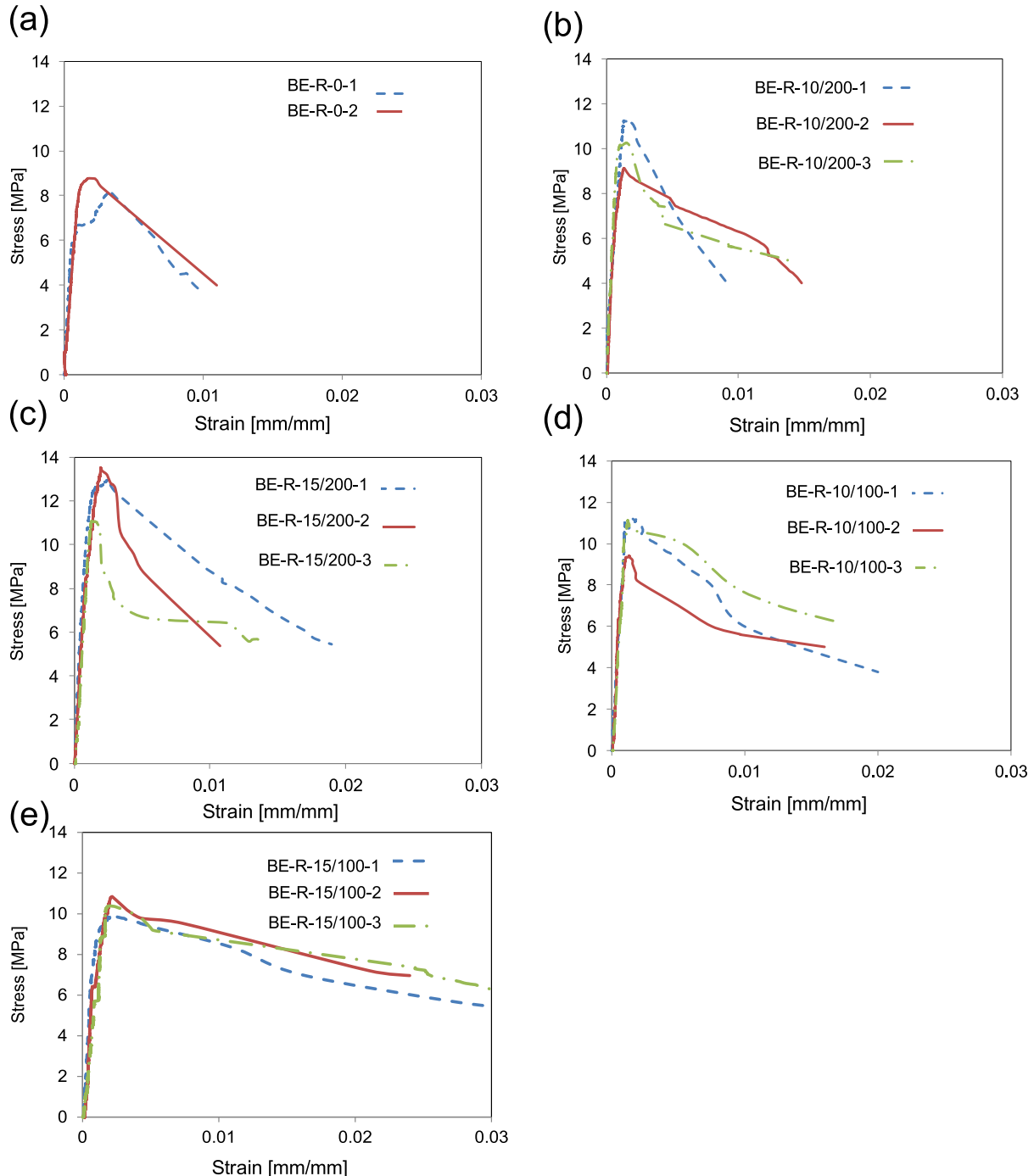


Fig. 12. Observed axial stress–strain relationships for reinforced concrete block masonry boundary element columns.

Table 2

Results for the reinforced units for maximum observed stress, maximum strain and strains at 80% and 50% strength degradation.

Test unit	Maximum stress		Maximum strain		Strain at 80% strength degradation		Ultimate strain at 50% strength degradation	
	$f_{\max}$ [MPa]		$\epsilon_{\max}$ [mm/mm]		ϵ_{80} [mm/mm]		ϵ_{50} [mm/mm]	
	Observed	Average and COV(%)	Observed	Average and COV(%)	Observed	Average and COV(%)	Observed	Average and COV(%)
BE-R-0-1	8.28	8.52 (4%)	0.003	0.0027 (12%)	0.004	0.0045 (16%)	0.0077	0.0078 (3%)
BE-R-0-2	8.77		0.0025		0.005		0.008	
BE-R-10/200-1	11.2	10.19 (10%)	0.0017	0.0015 (9%)	0.005	0.0053 (11%)	0.01	0.0090 (11%)
BE-R-10/200-2	9.13		0.0014		0.005		0.008	
BE-R-10/200-3	10.25		0.0015		0.006		0.009	
BE-R-15/200-1	13.04	12.52 (10%)	0.0024	0.0019 (20%)	0.009	0.0067 (31%)	0.015	0.0136 (16%)
BE-R-15/200-2	13.5		0.0019		0.006		0.011	
BE-R-15/200-3	11.04		0.0016		0.005		0.015	
BE-R-10/100-1	11.17	10.55 (9%)	0.0018	0.0014 (22%)	0.008	0.0083 (7%)	0.013	0.0144 (23%)
BE-R-10/100-2	9.38		0.0013		0.009		0.012	
BE-R-10/100-3	11.11		0.0012		0.008		0.0182	
BE-R-15/100-1	9.89	10.36 (4%)	0.002	0.0020 (5%)	0.013	0.014 (7%)	0.029	0.0283 (10%)
BE-R-15/100-2	10.83		0.0021		0.015		0.025	
BE-R-15/100-3	10.38		0.0019		0.014		0.031	

with an increase in the axial capacity due to the presence of the vertical reinforcement. At the maximum stress, vertical cracks started to form along the full length of the column followed by spalling of the face-shells of the concrete blocks. Strength degradation was observed after maximum load in the form of crushing of the grout columns and bar buckling. Test units (BE-R-0) reached an average of 50% increase in the maximum strength compared to test units (BE-U-0).

- For test units BE-R-10/200 and BE-R-15/200, the maximum strength recorded an average of 19% and 46% increase compared to tests units BE-R-0, respectively. This can be attributed to the effect of the confinement reinforcement on increasing the compressive strength of the grout core compared to the test units with only vertical reinforcement. After reaching the maximum strength, strength degradation started to occur in the stress–strain curve due to the spalling of the concrete face shell followed by vertical reinforcement buckling and crushing of the grout core. More gradual strain softening behavior and higher ultimate strains were observed for units BE-R-10/200 and BE-R-15/200 compared to units BE-U-0 and BE-R-0 due to the presence of confinement reinforcement.
- For test units BE-R-10/100 and BE-R-15/100, the maximum strength recorded an average of 23% and 21% increase compared to test units BE-R-0, respectively. On the other hand, milder strain softening was observed compared to units BE-R-10/200 and BE-R-15/200 due to the closer spacing of the confinement transverse reinforcement (100 mm) which resulted in an increase in the strain ductility after reaching maximum strength. Test units BE-R-15/100 showed more ductile behavior compared to units BE-R-10/100 which can be attributed to the larger diameter of the transverse ties (i.e. increase in the volumetric ratio of confinement reinforcement). The higher ultimate strain and gentle strain softening slopes are beneficial to increase the curvature ductility of RM shear walls.
- The maximum strength recorded for test units BE-R-15/200 is 18% higher than the test units BE-R-10/100 despite the fact that the confinement ratio of the latter is 20% higher than the former. This is mainly attributed to the weakening of the blocks induced by cutting portions of their webs and face-shells to accommodate the placement of the ties at closer spacing (Fig. 5). This can be explained in the following points: (1) according to Drysdale et al. [14], the compressive strength of grouted concrete masonry is highly correlated to the compressive strength of the concrete block and the failure mechanism is mainly related to the cracking in the blocks induced from the lateral expansion of the grout core. On the other hand, less correlation was observed between the compressive strength of the grouted masonry to the grout strength compared to the block strength especially if the grout strength is higher than the block strength; (2) for test units BE-

R-10/100, larger cutting in the webs and face-shells of the blocks was required to accommodate the placement of the ties at 100 mm spacing as compared to the test units BE-R-15/200 at 200 mm spacing (Fig. 5); (3) two interacting mechanisms affected the behavior: an increased compression strength of the confined portion of the grout by confinement reinforcement that is opposed to weakening of the blocks and the corresponding increase in the lateral expansion stresses from the unconfined portion of the grout on the face-shells. These mechanisms limited the efficiency of the confinement reinforced of the grout core in the enhancement of maximum strength of the boundary elements. On the other hand, the confinement reinforcement improved the strain ductility of the boundary elements and induced more gentle strain softening slope which is more important for increasing the curvature ductility of RM shear walls.

6. A proposed confinement model for boundary elements

In order to be able to predict the inelastic response of RM shear walls, it is necessary to have detailed information on the complete stress–strain characteristics of its reinforced boundary elements including compressive strength and its corresponding strain, slope of the falling branch of the stress–strain curve after peak strength, and ultimate compression strain. An important part of the design process of RM structural walls with boundary elements is the definition of suitable stress–strain curves that takes into account the compressive behavior of unconfined blocks and the influence of the confinement reinforcement.

6.1. Stress–strain model for unconfined boundary elements

First, a simplified stress–strain model is proposed for modeling the compressive behavior of unreinforced concrete block masonry boundary elements without confinement ties. The model is similar to the Kent–Park model [13] with some modifications. The model of the unreinforced boundary elements is broken into three portions; a parabolic rising curve as in Eq. (1), a plateau with constant stress as in Eq. (2) and a linear falling branch as in Eqs (3) and (4). The model is detailed as follows (Fig. 14):

$$f_m = f_{\max-u} \left(\frac{2\epsilon_m}{\epsilon_{\max1}} - \left(\frac{\epsilon_m}{\epsilon_{\max1}} \right)^2 \right) \quad \text{for } \epsilon_m < \epsilon_{\max1} \quad (1)$$

$$f_m = f_{\max-u} \quad \text{for } \epsilon_{\max1} < \epsilon_m < \epsilon_{\max2} \quad (2)$$

$$f_m = f_{\max-u} (1 - Z(\epsilon_m - \epsilon_{\max2})) \quad \text{for } \epsilon_m > \epsilon_{\max2} \quad (3)$$

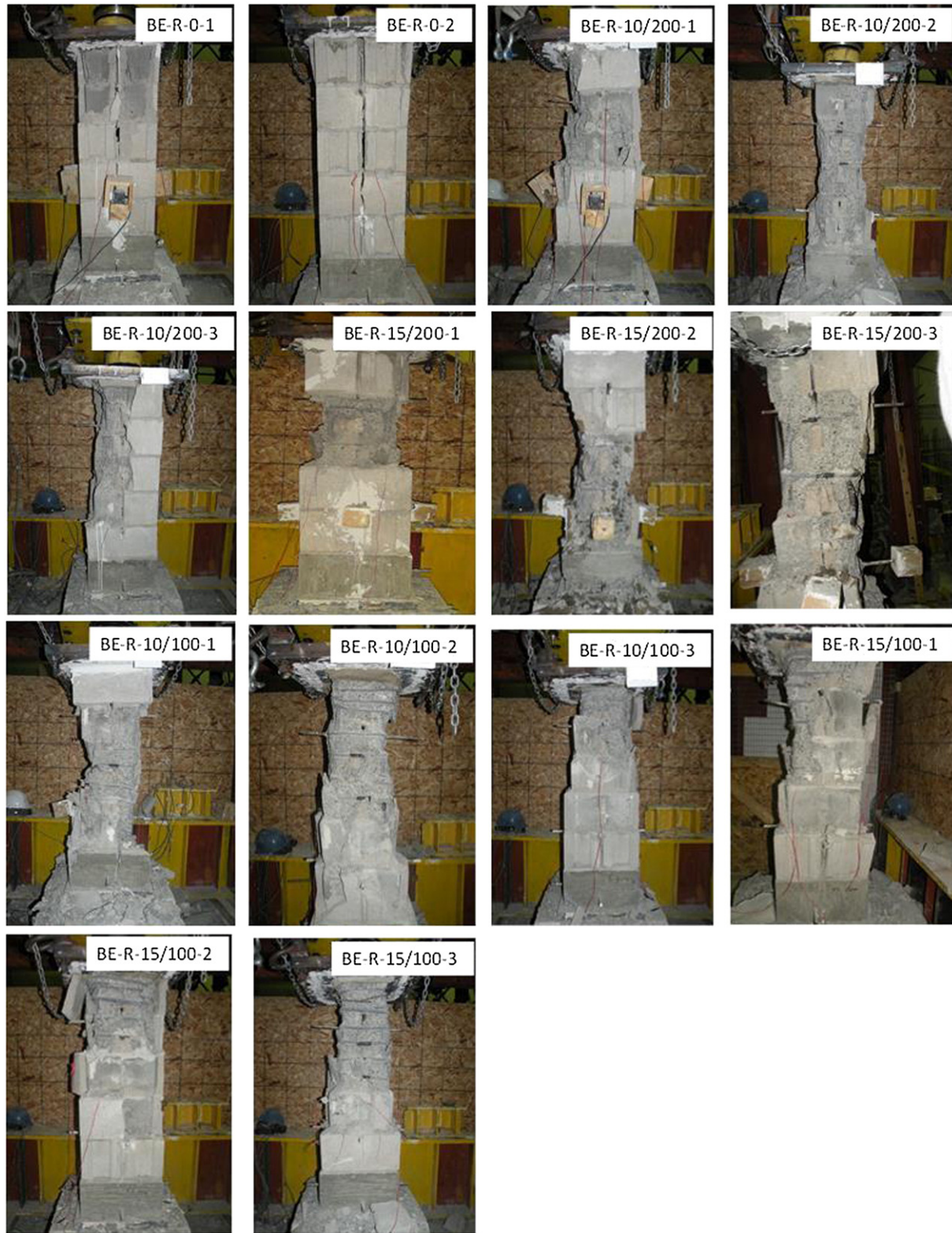


Fig. 13. Photograph illustrations of the failure mechanisms of the reinforced concrete block masonry columns at the end of the tests.

$$Z = \frac{0.5}{\varepsilon_{50u} - \varepsilon_{\max 2}} \quad (4)$$

where: f_m is the compression stress and ε_m is the corresponding compression strain; $f_{\max-u}$ is the peak compressive stress for the unreinforced element and $\varepsilon_{\max 1}$ is the corresponding strain (with average value of 0.0010

from test results); $\varepsilon_{\max 1}$ is the strain at the end of the plateau region with an average value of 0.003, Z is the slope of the post-peak part of the curve and ε_{50u} is the strain at 50% reduction of strength ($50\%f_{\max-u}$) with average value of 0.005 from test results. Fig. 14 shows fairly good agreement between the experimentally observed stress–strain curves of test units BE-U-0 and the corresponding analytical curve based on average observed maximum unconfined maximum strength (5.6 MPa).

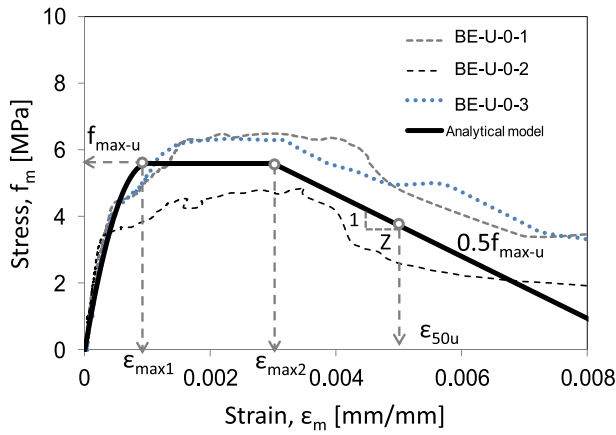


Fig. 14. Proposed analytical stress-strain curves for the unreinforced concrete masonry boundary element columns.

Fig. 15 shows a comparison of the idealized analytical stress-strain curves for the unreinforced boundary element columns and the standard prisms. The stress-strain model for the standard prisms is based on the proposal of [5] for a modification of Kent–Park model to match the experimental results on concrete masonry prisms. The boundary element configuration affected the post-peak descending part of the curve and increased the ultimate strain compared to the standard prism stacked configuration.

6.2. Stress-strain model for confined boundary elements

Experimental investigations have shown that analytical curves in the post-peak strain softening range developed to model the behavior of confined concrete in compression need to be modified to better reflect the behavior of grouted confined concrete masonry [4,5,16]. This is mainly attributed to the composite nature of grouted masonry where there are interactions between block face-shells and webs, grout and mortar. Grouted concrete masonry fails due to a mechanism generated by the tendency of the grout to expand laterally and the tendency of the face-shell to deform outward under the actions of axial compressive loading Dhanasekar and Shrive 2002 [7]. More specifically, previous experimental results showed that the post-peak strain softening range of the stress-strain curve is steeper for confined concrete masonry than for reinforced confined concrete. Therefore, researchers developed confinement models specific to grouted concrete masonry for different confinement schemes based on the analysis of experimental results. Shing et al. [5] modified the analytical function developed by Kent and Park [13], originally developed for concrete columns confined

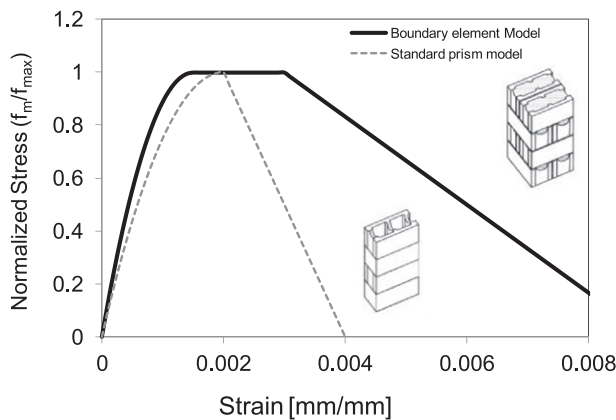


Fig. 15. Analytical stress-strain curves for the unreinforced concrete masonry boundary elements and the standard prisms.

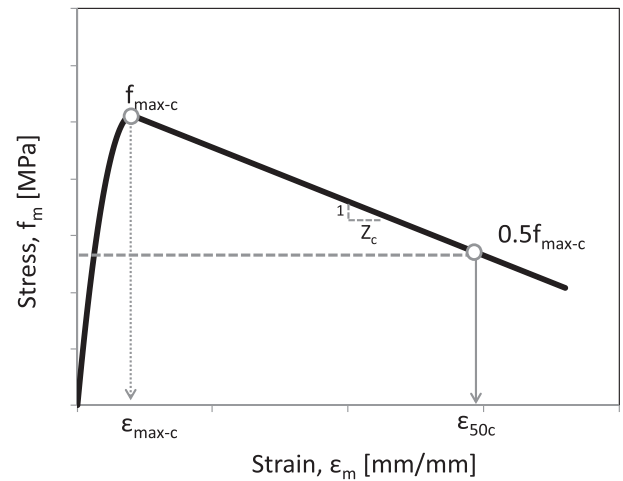


Fig. 16. Illustration of the proposed stress-strain model for the confined boundary elements.

with rectangular ties, to better reflect the effect of confinement of grouted concrete masonry with comb and spiral cage confinement in the slope of the post-peak strain softening range. The Kent and Park model provides simple analytical function of the stress-strain curve including the confinement effect based on few input parameters (e.g. the concrete strength and the confinement ratio). This simple formulation is desirable for practice-oriented design and assessment procedures. A similar approach was adopted in this paper where the ascending parabolic pre-peak analytical formulation is similar to Kent and Park [13] and replacing the peak confined concrete strength with the peak confined concrete masonry strength of the boundary element (Eq. (5)). On the other hand, the post-peak strain softening part (linear) was modified based on the experimental results of the RM boundary element in this study (Eqs (6) and (7)). The model is detailed as follows (Fig. 16):

$$f_m = f_{\max-c} \left(\frac{2\varepsilon_m}{\varepsilon_{\max-c}} - \left(\frac{\varepsilon_m}{\varepsilon_{\max-c}} \right)^2 \right) \quad \text{for } \varepsilon_m < \varepsilon_{\max-c} \quad (5)$$

$$f_m = f_{\max-c} (1 - Z_c (\varepsilon_m - \varepsilon_{\max-c})) \quad \text{for } \varepsilon_m > \varepsilon_{\max-c} \quad (6)$$

$$Z_c = \frac{0.5}{\varepsilon_{50c} - \varepsilon_{\max-c}} \quad (7)$$

where: f_m is the compression stress and ε_m is the corresponding

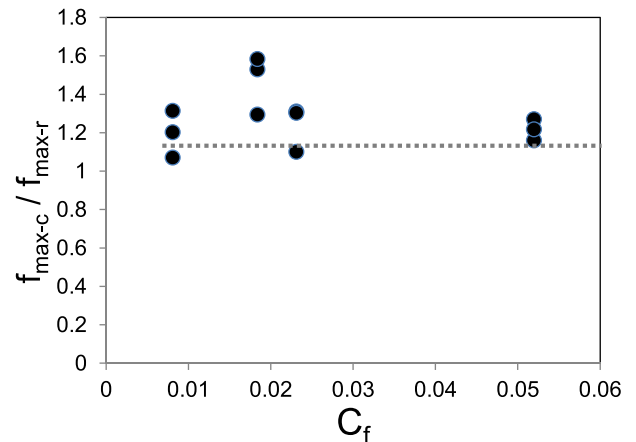


Fig. 17. Correlation between the strength enhancement ratio and the confinement ratio based on test results.

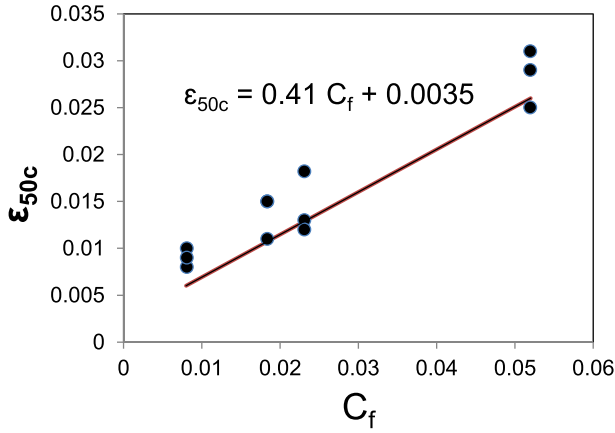


Fig. 18. Correlation between the strain at 50% strength degradation and the confinement ratio based on test results.

compression strain; $f_{\max-c}$ is the peak compressive stress for the confined reinforced boundary element and $\varepsilon_{\max-c}$ is the corresponding strain (with average value of 0.002 from test results); Z_c is the slope of the post-peak part of the curve and ε_{50c} is the strain at 50% reduction of strength for the confined boundary element. The value of $f_{\max-c}$ can be computed from the corresponding strength of a reinforced boundary element and taking into account the contribution from vertical steel reinforcement by the following equations:

$$f_{\max-c} = 1.15f_{\max-r} \quad \text{for } C_f > 0.008 \quad (8)$$

$$f_{\max-r} = \frac{[0.85f_{\max-u}(A_g - A_{st}) + f_{sy}A_{st}]}{A_g} \quad (9)$$

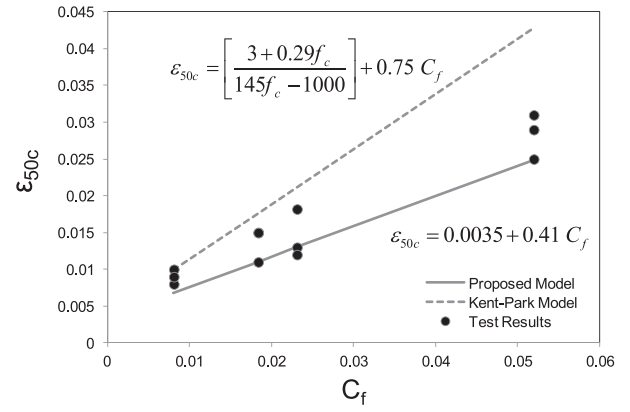


Fig. 20. Comparison of the increase in the strain at 50% strength degradation with confinement ratio from the proposed model, Kent-Park model and test results.

where: $f_{\max-r}$ is the average strength of the vertically reinforced boundary element; A_g is the gross cross-section area of the column; A_{st} is the cross-sectional area of the vertical steel; and f_{sy} is the yield strength of the vertical steel. The correlation between $f_{\max-c}$ to $f_{\max-r}$ presented in Eq. (8); is based on a regression analysis of the test results presented in this paper as shown in Fig. 17. The ultimate strain for confined boundary elements ε_{50c} can be obtained from the following equation which is based on the regression analysis with the confinement ratio from test results. (see Fig. 18).

$$\varepsilon_{50c} = 0.0035 + 0.41 C_f \quad \text{for } C_f > 0.008 \quad (10)$$

Fig. 19 shows that the proposed analytical model provides reasonable approximation of the average observed stress–strain behavior for the confined concrete block masonry boundary elements. The

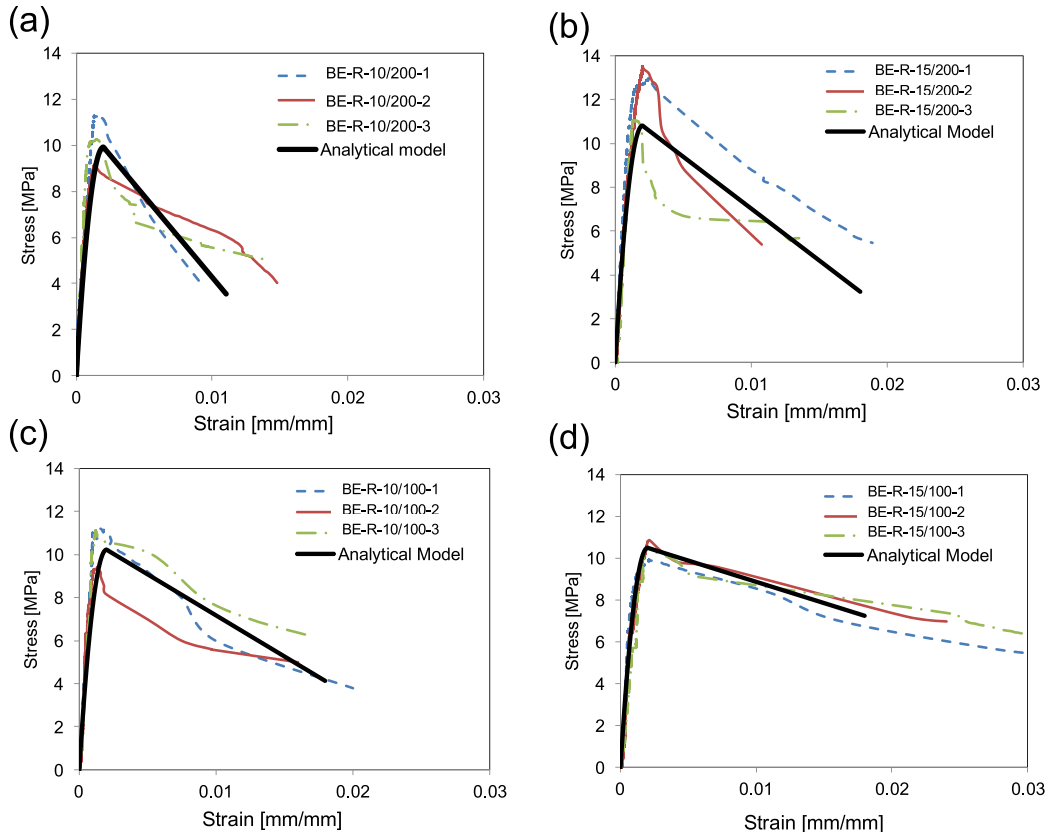


Fig. 19. Analytical and observed stress–strain curves for the confined concrete masonry boundary element columns.

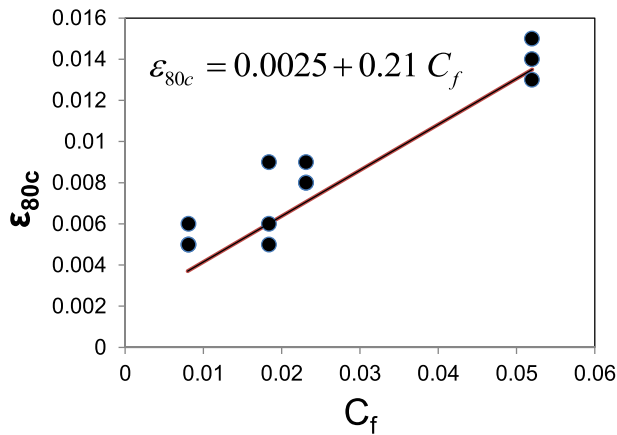


Fig. 21. Correlation of the confinement ratio with the strain at 80% strength degradation.

proposed analytical model for the stress–strain relationships of confined boundary element is particularly useful when conducting numerical simulation of the behavior of reinforced concrete block masonry with boundary elements under lateral loading.

Fig. 20 presents a comparison of the predicted-to-observed strain at 50% strength degradation (which controls the slope of the descending range) using the proposed model and the Kent–Park model for confined concrete (Eq. (11)). It should be noted that the intercept of Eq. (11) represents the unconfined concrete strain (ϵ_{50}) which is a function of the concrete cylinder compression strength (f_c). The f_c in this study is represented by the grout cylinder compression strength (25.6 MPa). The slope of Eq. (11) represents the effect of the confinement on increasing the strain capacity for concrete. The Results show that the Kent–Park model moderately overestimates the strain capacity at lower confinement ratios. On the other hand, larger overestimation is observed with increasing confinement ratios.

$$\epsilon_{50c} = \left[\frac{3 + 0.29f_c}{145f_c - 1000} \right] + 0.75 C_f \quad (11)$$

Fig. 21 shows the correlation between the confinement ratio and the strain corresponding to 80% strength degradation. This strain limit is typically defined in masonry codes and standards as the design strain. The developed correlation is particularly useful for estimation of the required confinement ratio that satisfies a specific strain demand from seismic loading. The ultimate strain for confined boundary elements ϵ_{80c} can be obtained from the following equation which is based on the regression analysis with the confinement ratio from test results.

$$\epsilon_{80c} = 0.0025 + 0.21 C_f \quad \text{for } C_f > 0.008 \quad (12)$$

It should be noted that the most recent Canadian standards for masonry design [12] provides a limit on the design strain not to exceed 0.008. The results of the presented experimental investigation validated the limiting strain imposed by the code and provided a possible increase of the design strain with adequate confinement ratios.

7. Conclusions

This paper presents an experimental and analytical investigation of the compressive stress–strain behavior of unconfined and confined concrete block masonry boundary element columns that are an integral part of special ductile reinforced masonry shear walls. A reliable

prediction of the deformation and ductility capacity of these walls requires accurate quantification of the behavior of the boundary element columns. The observed compression behavior of unconfined and confined boundary elements was investigated experimentally under displacement control for full identification of the pre-peak and post-peak behavior. Contrary to typical unreinforced masonry prism tests, a sudden drop in the strength was not observed and a descending branch was then recorded with a mild slope until reaching 50% reduction of strength with an average strain of 0.005. The investigated boundary element configuration with confinement reinforcement has a significant effect on the post-peak behavior with softening of the descending branch of the stress–strain curve and providing more strain ductility with the increase in the confinement ratio. A good correlation was observed between the strain gain ratio and the confinement ratio. A simplified analytical model is proposed for idealization of the compressive stress–strain behavior of concrete block masonry boundary elements which showed reasonable approximation of the observed behavior from testing. The proposed model is useful in the estimation of strain ductility and consequently the curvature ductility of reinforced masonry walls with boundary elements which can lead to more economical design in seismic regions.

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