

Review article

Compression behavior of confined concrete masonry boundary elements



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ABSTRACT

The seismic performance of reinforced masonry (RM) walls can be enhanced by integrating confined boundary elements at the end zones of the wall. The evaluation of the compression behavior of the boundary elements is essential to the reliable assessment of displacement ductility and the seismic performance of the walls. Complementary to the experimental evaluation of the compression behavior, finite element numerical simulations are particularly useful in assessing the influence and sensitivity of various design parameters. In this study, experimental and numerical investigations are conducted to evaluate the compression stress-strain behavior of confined C-shaped reinforced concrete masonry block boundary elements (C-RMBEs). Compression tests are conducted on 16 full-scale confined C-RMBEs with different configurations of confinement reinforcement. A finite element modeling (FEM) procedure using the ABAQUS software is employed to simulate the compression behavior of a C-RMBE. The FEM procedure is validated with experimental results on a full-scale confined C-RMBE. Comparative compression stress-strain curves and damage progression are presented and discussed. The study shows the significance of the confinement reinforcement in the improvement of the compression strain capacity of the C-RMBE. Moreover, the proposed FEM procedure provides a good approximation of the compression stress-strain behavior in the elastic and inelastic regions and captures the influence of the confinement reinforcement ratio on the compression response of the C-RMBE.

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1. Introduction

Reinforced masonry (RM) shear walls are commonly used in medium-rise and high-rise masonry buildings as the lateral load resisting system to provide the lateral strength, stiffness and energy dissipation capacity required to resist seismic loading. RM walls are expected to exhibit an inelastic response during severe ground motions. Furthermore, the end zones of the RM wall will be subjected to cycles of tension and compression, arising from seismic overturning moments, as seen in Fig. 1. Therefore, ductile detailing of the horizontal and vertical reinforcements, especially at the end zones of the walls (toe region), is required. Typical rectangular RM walls would have only a single vertical reinforcement bar placed in the masonry block cells without violating the maximum reinforcement and spacing requirements specified in masonry design standards (e.g., MSJC-2013 [1] and CSA-S304 [2]). Consequently, this single bar disallows the placement of confinement hoops at the end zones of the wall, which are subjected to high inelastic strains during earthquake-induced cyclic loading (Fig. 1a). On the other hand, adding boundary elements at the wall ends allows the placement of at least four vertical reinforcing bars enclosed by hoops (Fig. 1b and c), thus enhancing the wall performance by providing core confinement to the wall ends through the reinforcement cage. As such, RM walls constructed with boundary elements at the end zones ensure stability under high compression loading and demonstrate an enhanced curvature ductility for the wall. The compression strain capacity can be increased using confinement reinforcement [3]. Therefore, compared to a rectangular wall, adding a confined boundary element will increase the width of the RM wall end. As such, the sustainable compression strain will be increased, and the compression zone depth will be

decreased [4]. Hence, adding boundary elements will enhance the curvature capacity of the wall cross-section.

The use of ductile reinforced concrete masonry shear walls with column-like boundary elements has been introduced in recent North American codes and standards for the design of masonry structures to improve the ductility capacity of walls. The US Building Code Requirements and Specifications for Masonry Structures (MSJC, 2013) [1] allows the use of confined boundary elements and only imposes some geometrical rules. The code requires testing to be conducted to verify that the detailing provided is 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, 2014) [2] also allows for the use of confined boundary elements. However, this design standard does not provide a correlation between specific detailing for confinement reinforcement and the corresponding improvement in the strain capacity and required testing and analysis to satisfy the level of required strain. Experimental studies that quantify the compression strain ductility of concrete masonry boundary elements are scarce in the literature.

The evaluation of the compression behavior of the confined end zones of an RM wall is a key component in the assessment of the curvature and displacement ductility capacities of RM walls. Recent experimental investigations on RM shear walls with confined boundary elements (e.g., [5,6]) have concluded that the geometry of the boundary element allows for more than one layer of vertical reinforcement enclosed by hoops, which delays or prevents buckling of the vertical reinforcement, confines the compression zone, and limits the damage at the ends of the RM wall. Integrating the boundary element enhances the seismic performance of RM walls by improving the compression strain capacity

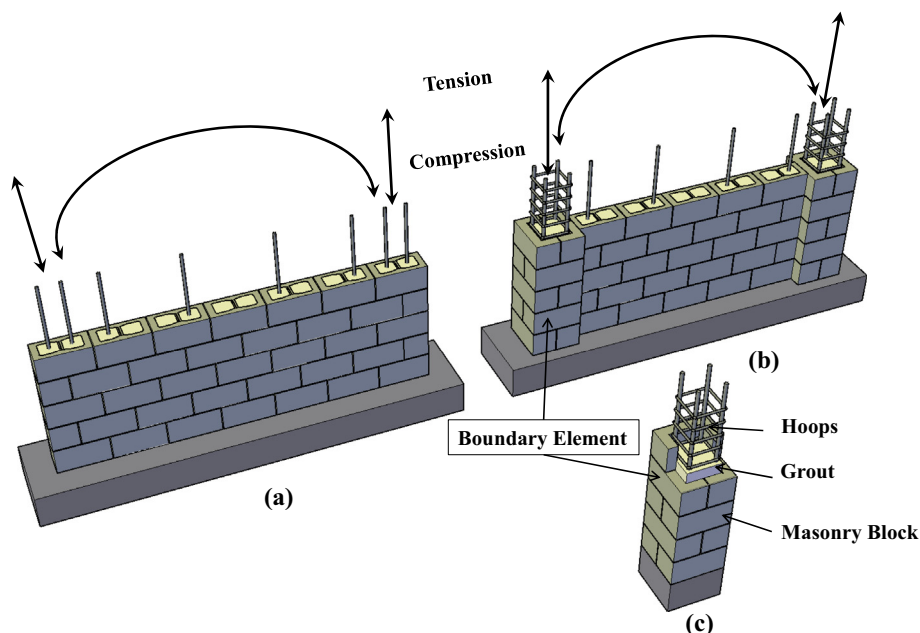


Fig. 1. Schematics of a masonry building: (a) Rectangular RM wall; (b) RM wall connected with boundary elements; and (c) RM boundary element.

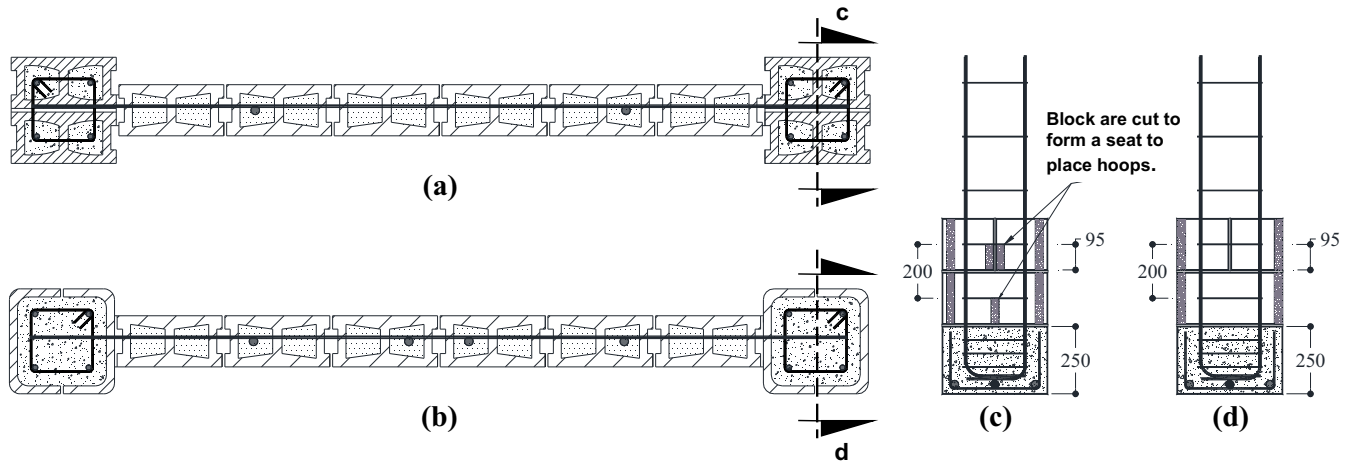


Fig. 2. Horizontal cross-section of an RM wall with boundary elements using (a) standard blocks and (b) C-shaped block units. Vertical cross-section of a boundary element using (c) standard blocks and (d) C-shaped block units.

and the corresponding curvature and displacement capacities. Abo El Ezz et al. [7] investigated the compression behavior of boundary elements with standard concrete blocks experimentally. However, the construction of boundary elements with standard concrete blocks requires cutting of the webs and shells of the blocks to accommodate the placement of the confinement hoops (Fig. 2a and c). On the other hand, C-shaped concrete masonry blocks can provide a more attractive alternative in terms of constructability, with a reinforcing steel cage placed prior to the construction of the boundary element without the need to cut the blocks (Fig. 2b and d). As such, there is a need for experimental and numerical investigations on the compression stress-strain behavior of confined boundary elements constructed with C-shaped concrete masonry blocks toward investigating their use in ductile RM walls with boundary elements.

The majority of experimental, analytical and numerical works that have been conducted on the structural behavior of RM columns have focused on the evaluation of the ultimate compression strength rather than the evaluation of the post-peak strain capacity (e.g., [8–14]). On the other hand, considerable research has been conducted on the evaluation of the compression behavior of confined reinforced concrete (RC) columns, including the quantification of the post-peak strain capacity. In addition, analytical stress-strain models for confined RC columns have been proposed (e.g., [15–17]). The compression behavior of a confined RM column is not necessarily similar to that of RC columns since there are complex interactions between the mortar, grout, block and steel reinforcement.

Complementary to experimental investigations, a validated numerical model using a detailed FEM enables the investigation of the influence of the combination of different design parameters for the mortar, grout, masonry block, and steel reinforcement on the compression behavior of RM boundary elements. This article presents experimental and numerical investigations on the compression stress-strain curve characteristics of isolated unconfined and confined reinforced C-RMBEs, representative of the highly compressed end zones of RM walls. Full-scale C-RMBEs were constructed and tested under axial compression loading until failure. A three-dimensional FEM was developed using the software ABAQUS, therein considering nonlinear constitutive material models for the mortar, steel, grout, and masonry block; then, the model was validated with experimental results. In this study, the influence of different configurations of the confinement reinforcement on the compression behavior and a comparison between the experimental and numerical results, including the compression stress-strain curves and damage progression, are presented.

2. Experimental work

2.1. Boundary element construction and design

A total of 16 full-scale fully grouted C-RMBEs were constructed and tested. All the unconfined and confined C-RMBEs were constructed by professional certified masons. Each specimen has an individual designation, where the first letter, BE, represents a boundary element. The second letter, R or U, denotes a reinforced or unreinforced specimen, respectively. The numbers following the letters refer to the hoop bar size and the spacing of the hoops, as shown in Fig. 3. Typical dimensions of the unreinforced and reinforced C-RMBEs are shown in Fig. 3. The total length of the test unit is 1250 mm, with an effective (gage) length of 1000 mm. Each C-RMBE consists of five block layers placed on a concrete footing with dimensions of 400 mm × 400 mm × 250 mm (length × width × depth). Each course of the C-RMBE was composed of two block units placed together in alternating directions along its height. The blocks were joined together with 10 mm type-S mortar joints. The C-RMBE contained an internal vertical reinforcement of four bars of 20 M ($A_{sv} = 300 \text{ mm}^2$) that extended continuously from the base of the footing to over the height of the C-RMBE without lap splices. The horizontal reinforcement in the form of closed hoops was placed before the construction of the concrete blocks. The C-RMBEs had different volumetric reinforcing ratios, represented by the different diameters and spacings of the hoops. The hoop diameters were 10 M and 15 M at vertical spacing's of 100 mm and 200 mm (Table 1). The hoops were placed around a vertical reinforcement with outer dimensions of 230 mm × 230 mm. Fig. 4 shows the sequence of the placement of the ties and C-shaped concrete blocks of the C-RMBEs. The C-RMBEs were filled with a standard coarse grout mixed in the lab. The confinement ratio C_f is calculated as $C_f = \rho_s \sqrt{H/S}$, as shown in Table 1, where ρ_s is the volumetric ratio of the confinement reinforcement (the volume of the confinement hoops to the volume of the core concrete at a spacing S), H is the width of the confined core (230 mm in this case), and S is the spacing between the confinement hoops.

2.2. Material properties

The properties of the materials used for the construction of the C-RMBEs are summarized in Table 2. Type-S mortar was used to join different courses with an average thickness of 10 mm, as shown in Fig. 4. The proportions of the materials of the coarse

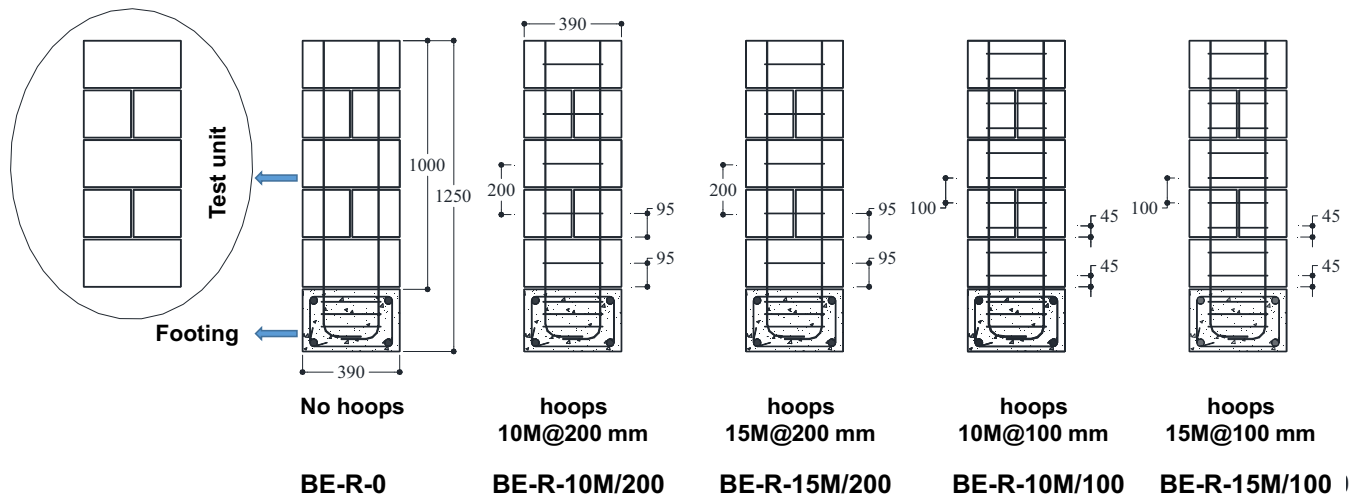


Fig. 3. Construction details of unreinforced and reinforced concrete C-RMBEs.

Table 1
Test matrix of concrete C-RMBE.

C-RMBE ID	Number of tested units	Vertical reinforcement	Transverse reinforcement (hoops)	Volumetric ratio of transverse reinforcement ρ_{sv}	Confinement ratio C_f
BE-U-0	2	–	–	0	0
BE-R-0	3	4–20 M	–	0	0
BE-R-10/200	3	4–20 M	10 M@200 mm	0.0089	0.01
BE-R-15/200	2	4–20 M	15 M@200 mm	0.0177	0.02
BE-R-10/100	3	4–20 M	10 M@100 mm	0.0177	0.028
BE-R-15/100	3	4–20 M	15 M@100 mm	0.0354	0.056



Fig. 4. Construction sequence of placing steel bars and blocks in the C-RMBEs.

Table 2
Tested material properties.

Material	Property	Compressive strength (MPa)	C.O.V.%
Masonry block	f_{bl}	15	3.5
Mortar	f_{mr}	13.7	10
Grout	f_{gr}	35	6.5
Reinforcement steel	f_y	420	7.5

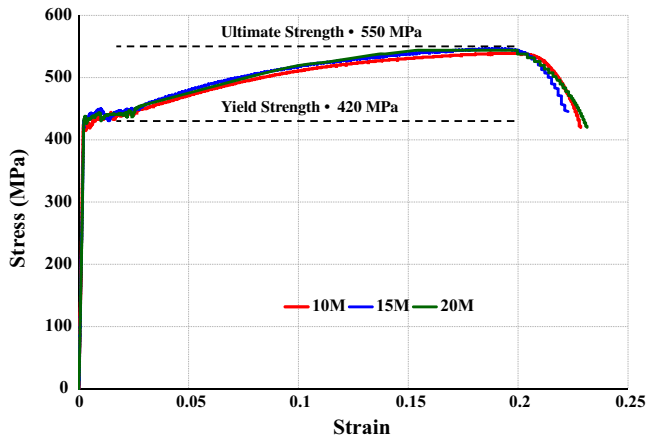
C.O.V. (coefficient of variation)

grout mix are summarized in Table 3. Quasi-static tension tests were conducted on five 600-mm-long tensile specimens for each diameter to determine the yield strength of the steel reinforcement used in the construction of the C-RMBE. The measured value of the yield strength was 420 MPa. The reinforcement characteristic stress-strain curves for three different diameters are shown in Fig. 5. Full-scale 70-mm-thick C-shaped concrete blocks 190 mm × 190 mm × 390 mm (depth × width × length) were used in the C-RMBE construction, as shown in Fig. 6. The average

Table 3

Grout mix proportions.

Element	Part
Portland cement	1
Fine aggregate	1.68
Coarse aggregate	1.68
Water	0.5

**Fig. 5.** Stress-strain curves for the reinforcement steel used in the construction of the C-RMBEs.

compressive strength based on the net area of three full-scale C-shaped concrete masonry block units was 15 MPa. The C-shaped concrete masonry block units were joined together with 10 mm of type-S mortar. Six 50 mm cubes were molded and tested under compression. The average compressive strength of the mortar at the beginning of testing of the C-RMBEs was 13.7 MPa. Course grout was used for grouting the C-RMBEs. The average grout compression strength of the six 100 mm × 200 mm (diameter × height) cylinders was 35 MPa when the C-RMBEs were tested. All materials used for the construction are available on the Montreal commercial market.

2.3. Test setup and instrumentation

The C-RMBE specimens were tested under compression using a displacement-controlled MTS machine (Fig. 7) at École Polytechnique de Montréal. The displacement-controlled setting enabled capturing of the post-peak descending branch of the stress-strain curve, which is needed to quantify the influence of the confinement reinforcement on the strain capacity of the C-RMBEs. The test setup consisted of a steel frame transferring the load to the strong floor supporting the test unit, a 12,000 kN hydraulic cylinder controlled by an MTS controller and a 50 mm rigid rectangular steel

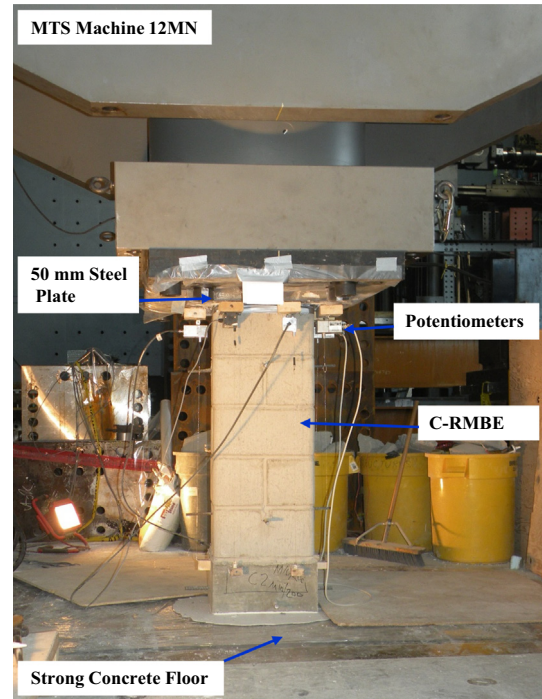
**Fig. 7.** Test setup.

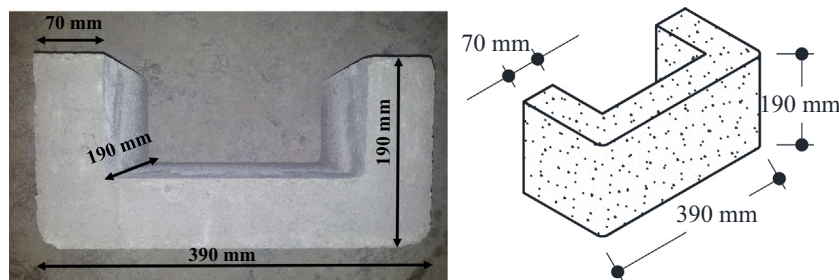
plate for transferring a uniformly distributed load to the test unit. High-strength gypsum was placed on the top and bottom of the C-RMBE to ensure that the top and bottom of the test unit were leveled and that the load was uniformly distributed on the specimen.

The rate of displacement was 0.45 mm per minute, and the load was recorded from an MTS controller attached to a data acquisition system. To measure the continuous deformation of the C-RMBE, the vertical displacement of the C-RMBE was measured using four cable-extension transducers (potentiometers) attached at the centerline of each side of the unit while being fixed underneath the bearing plate and at the top of the concrete footing. The gauge length of the potentiometers was 1000 mm, measured from underneath the bearing plate to the top of the concrete footing.

2.4. Experimental results

Fig. 8 shows the observed compression stress-strain curves for all test units, including unreinforced, vertically reinforced and reinforced confined C-RMBEs.

It can be seen that the vertically reinforced C-RMBE (BE-R-0) exhibited an increase in strength compared to the unreinforced C-RMBE (BE-U-0) due to the presence of the vertical reinforcement bars. The presence of the confinement horizontal reinforcement

**Fig. 6.** Full-scale C-shaped concrete masonry block unit.

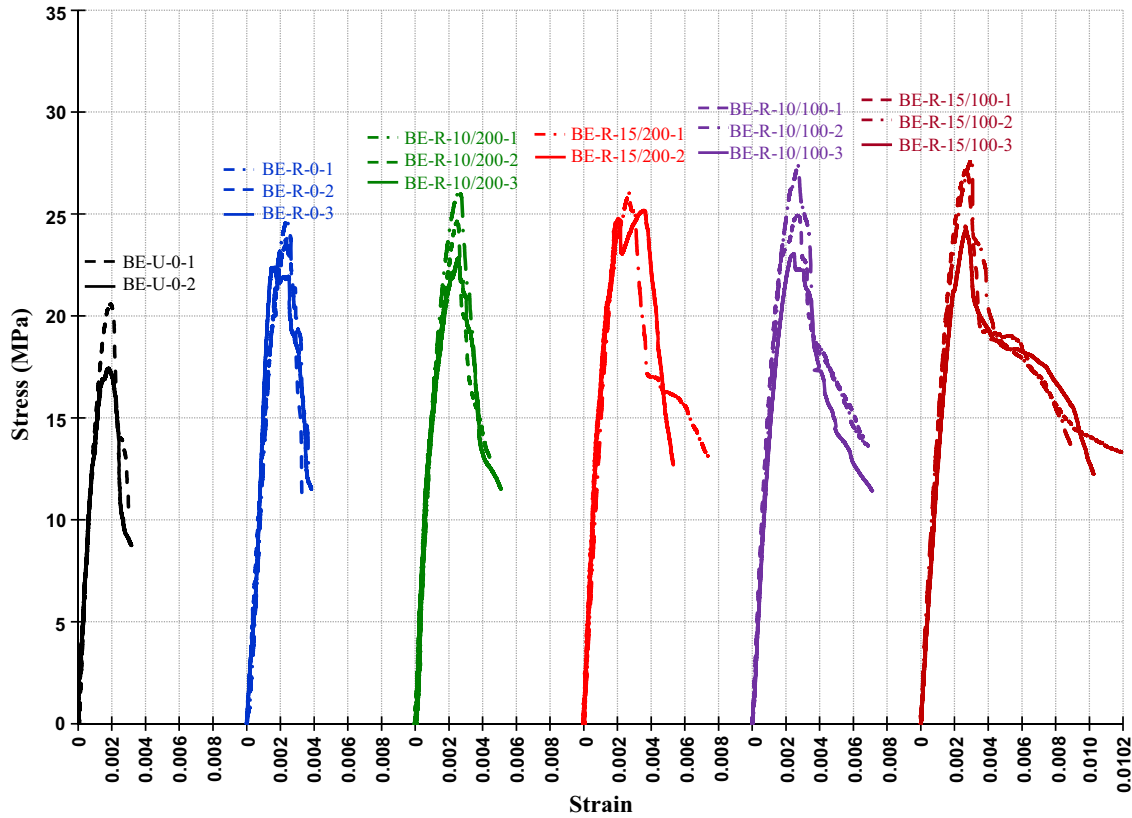


Fig. 8. Observed stress-strain curves for all test units, including unreinforced, vertically reinforced and reinforced C-RMBEs.

(hoops) produced a more gradual post-peak stress-strain behavior and improved the strain capacity of the C-RMBE. As the horizontal confinement reinforcement increased, the strain capacity of the C-RMBE increased. For the test units BE-R-10/200 with $C_f = 0.01$, the test results indicated that post-peak strain at 75% and 50% of the strength an increase of 29% and 45%, respectively, compared to the test units BE-U-0 (the unreinforced units). The test units BE-R-15/200 with $C_f = 0.02$ exhibited an increase of 37.5% and 100% in the post-peak strain at 75% and 50% of the strength, respectively, compared to the test units BE-U-0. In contrast, the post-peak strains at 75% and 50% of the strength for the test units BE-R-10/100 with $C_f = 0.028$ were higher by 58.4% and 122.5%, respectively, compared to the test units BE-U-0. The highest increases in the post-peak strain at 75% and 50% of the strength were obtained with test units BE-R-15/100 with $C_f = 0.056$, which resulted in an increase of 83% and 232%, respectively, compared to the test units BE-U-0.

C-RMBEs BE-R-10/100 and BE-R-15/100 exhibited a milder post-peak strain softening behavior compared to C-RMBEs BE-R-10/200 and BE-R-15/200 due to the presence of a higher confinement ratio. C-RMBEs with confinement reinforcement experienced an average drop in strength of 25% after reaching the C-RMBE's peak load. This is mainly attributed to the difference in the compressive strengths of the C-shaped concrete block (15 MPa) and the grout (35 MPa). The concrete block represents approximately 50% of the C-RMBE cross-sectional area that exhibits spalling after reaching the peak load.

3. Finite element analysis

The nonlinear finite element software package ABAQUS [18] was employed for the numerical simulation of the compression

stress-strain behavior of the tested C-RMBEs. Details of the assumptions concerning the constitutive laws of the materials that are used in this FEM are discussed in the following subsections.

3.1. Constitutive models

For simulating the behavior of grout, mortar, and masonry block, the damage plasticity model from ABAQUS was applied in this study. The plastic damage model requires the parameters of the concrete damage plasticity and the values of the modulus of elasticity, density and Poisson's ratio. Being cementitious material, the concrete damage plasticity parameters for the concrete block, grout and mortar were assumed to be the same [19]. The parameters of the concrete plastic damage that were used for the grout, mortar, and masonry block are the dilation angle (36°), the plastic potential eccentricity (0.1), the ratio of the strength in the biaxial state to the strength in the uniaxial state (1.16), the ratios of the distance between the hydrostatic axis and the compression meridian and the tension meridian, separately, in the deviatoric cross section (0.667), and the viscosity parameter that defines viscoplastic regularization (0) [20]. A perfect bond between the steel reinforcement and grout was achieved using the embedded regions available in the ABAQUS library.

3.2. Grout, mortar and masonry block material model

The stress-strain behaviors of the grout and concrete blocks under uniaxial compression are modeled using the stress-strain relationship proposed by Saenz's [21] equation ([22–25]).

$$f_c = \frac{E_c \varepsilon_c}{1 + (R + R_E - 2) \left(\frac{\varepsilon_c}{\varepsilon_{cc}} \right) - (2R - 1) \left(\frac{\varepsilon_c}{\varepsilon_{cc}} \right)^2 + R \left(\frac{\varepsilon_c}{\varepsilon_{cc}} \right)^3} \quad (1)$$

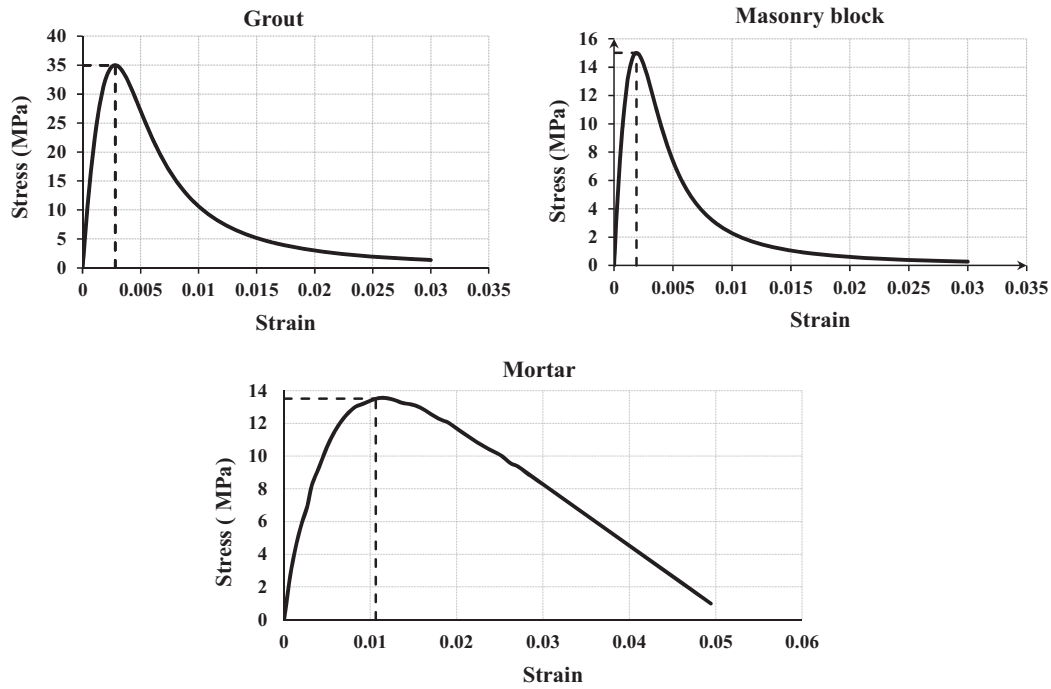


Fig. 9. Stress-strain behavior of concrete under uniaxial compression for the grout, masonry block, and mortar.

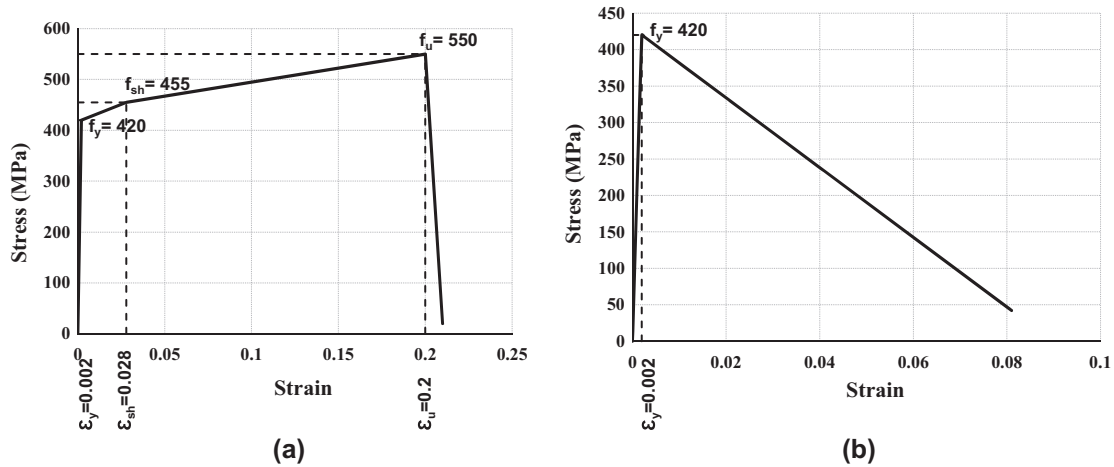


Fig. 10. Stress-strain relationship for reinforcing steel rebars in (a) tension and compression for a bar aspect ratio (S/D) of less than 8 and in (b) compression for a bar aspect ratio (S/D) of greater than 8.

$$R = \frac{R_E(R_{O'} - 1)}{(R_E - 1)^2} - \frac{1}{R_E}, \quad R_E = \frac{E_C}{E_0} \quad \text{and} \quad E_0 = \frac{f'_C}{\varepsilon_0} \quad (2)$$

$$\varepsilon_0 = 0.0014[2 - \exp(-0.024f'_C) - \exp(-0.140f'_C)] \quad (3)$$

where E_C is the initial modulus of elasticity, R is the ratio relation, R_E is the modular ratio, $R_{O'}$ is the stress ratio, R_E is the strain ratio, and E_0 is the secant modulus. The strain corresponding to the peak stress, ε_0 , was calculated to be equal to 0.0028 and 0.0019 for the grout and masonry block, respectively, based on plotting the stress-strain curve equations using Eq. (3) [18], where f'_C is the average compressive strength, $R_E = 4$, and $R_{O'} = 4$, where the last two values are as reported in [26]. The fitted mortar stress-strain curve used in the numerical model in this study is captured graphically from Kaushik et al. [27]. Kaushik et al. [27] obtained a compression stress-strain curve for mortar by testing mortar cubes 50 mm in size with a similar mortar strength that was used in this study. The stress-strain

curve of the mortar was obtained by averaging the data from nine specimens. The stress-strain curve under uniaxial compression is shown in Fig. 9. The ultimate compressive strengths of the grout, masonry block, and mortar are 35, 15, and 13.7 MPa, respectively, as recorded during the material testing.

3.3. Steel reinforcement

The plastic model from the ABAQUS software was applied to simulate the reinforcement stress-strain behavior. The material constitutive model of the steel reinforcement bars is assumed to behave in tension, with multi-linear stress-strain curves for hoops and the longitudinal bars, which have a bar aspect ratio (S/D) of less than 8 [28], where S is the spacing between the hoops and D is the diameter of the longitudinal bar. In this case, the stress-strain behavior in tension and compression is assumed to be similar [29] and was idealized from the as-tested curves shown in

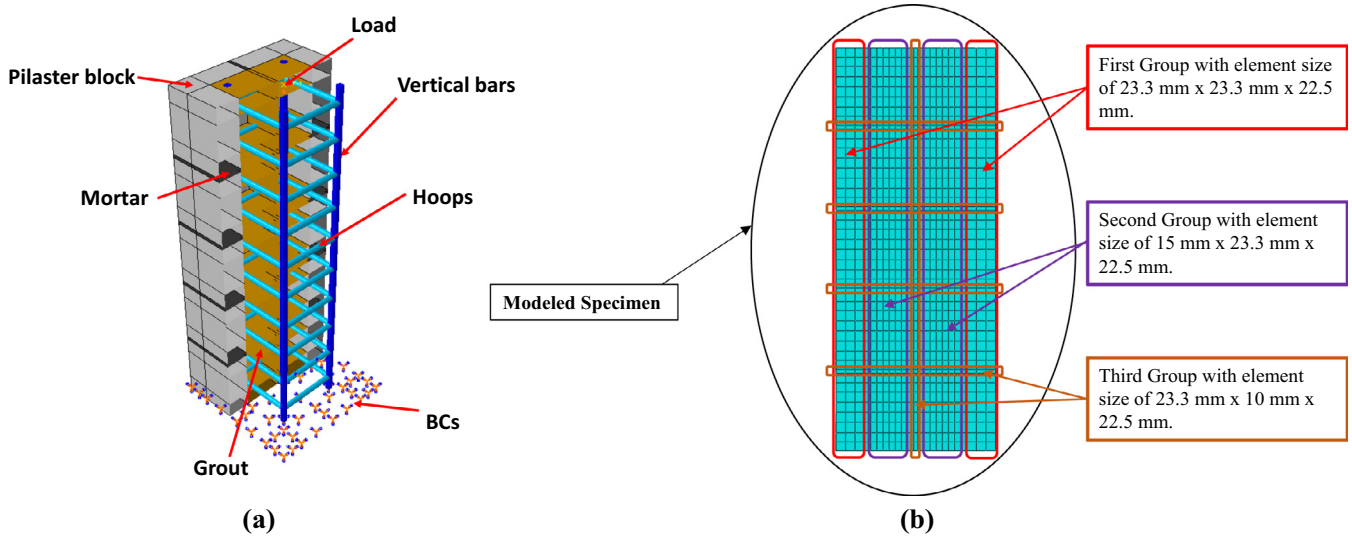


Fig. 11. RM C-RMBE modeled specimen: (a) typical geometry and finite element boundary conditions and (b) element mesh sizes.

Fig. 5. This curve consists of three segments and was idealized by three straight lines, as shown in Fig. 10a. The elastic and yield segments are linear. The strain hardening is also represented by a linear instead of parabolic curve. Whereas longitudinal bars have a bar aspect ratio (S/D) of greater than 8, the material constitutive model of steel reinforcement bars is assumed to behave in compression both linear elastically for buckling consideration and with the instability of the rebars, as shown in Fig. 10b [28]. In the figure, f_y , f_{sh} , and f_u represent the steel yield, strain-hardening, and ultimate stresses, respectively, and ϵ_y , ϵ_{sh} , and ϵ_u are the corresponding strains.

In ABAQUS, the linear elastic portion of the steel stress-strain curve is defined by the modulus of elasticity. However, the non-linear portion requires the true stress, f , versus the plastic strain, ϵ^p , relationship; this plastic strain must be calculated from the engineering stress-strain relationship. The steel reinforcement used in the FEM of the C-RMBE has a yielding stress of 420 MPa, modulus of elasticity of 200 GPa and Poisson's ratio of 0.3, as indicated in Fig. 5. A perfect bond was assumed between the steel reinforcement and the grout [30,31].

3.4. Geometry and boundary conditions

The FE models considered all the degrees of freedom of the nodes at the bottom of the RM C-RMBE to be restrained using the displacement and rotation commands available in the ABAQUS library, whereas the other nodes were considered to be free to rotate and translate in any direction. The boundary condition was applied at the top end of the RM C-RMBE geometry with a reference point at the middle to apply a concentrated load in increments using the static command available in the ABAQUS library (Fig. 11a).

To ensure that the nodes on the top of the geometry (i.e., all elements that are located at the top surface of the modeled specimen) have the same vertical displacement, and to ensure that the load is distributed uniformly, the multi-point connection (MPC) constraint commands available in the ABAQUS library were used.

3.5. Finite element type and mesh

3D simulations were conducted to obtain an accurate approximation of the overall compression behavior of the RM C-RMBEs. Three-dimensional eight-node solid elements were used for the grout, mortar, and masonry block. A beam element was used for

the reinforcement steel rebars. The perfect bond between the steel reinforcement and the grout was achieved using the embedded region available in the ABAQUS library. A fine mesh was used to provide more detailed recording of the elastic and inelastic behaviors ([31]), as shown in Fig. 11b. The C-RMBE has 28,800 solid elements and 32,435 nodes in total. Different element sizes were used within the single model (i.e., the model is discretized into small elements, and these elements are grouped into three groups based on element size). The first group, with an element size of 23.3 mm $\times$ 23.3 mm $\times$ 22.5 mm, is located at the corners of the masonry block along the height of the modeled specimen. The second group, with an element size of 15 mm $\times$ 23.3 mm $\times$ 22.5 mm, is located on the flange and web face of the masonry block along the height and depth of the modeled specimen. The third group, with an element size of 23.3 mm $\times$ 10 mm $\times$ 22.5 mm, is located on the bed and head mortar joints, as shown in Fig. 11b.

4. Validation of the numerical model

To verify the FEM approach of the C-RMBEs, six C-RMBE configurations, tested by the authors and as presented in the previous section, are modeled in this study using the ABAQUS software package. Table 4 shows the numerical and experimental results of the maximum stress f_{max} , strain at the peak stress ϵ_{max} , strain at 75% of the peak stress ϵ_{75} , strain at 50% of the peak stress ϵ_{50} , and strain ductility μ of the C-RMBE. The strain ductility is defined as the ratio between the strain at 50% of the peak stress on the descending curve and the strain at the peak stress, as shown in Table 4. These results are based on the calculated and measured stress-strain behavior of the C-RMBE. In the following subsections, a comparison of the experimental and numerical results of the compression stress-strain behavior is presented.

4.1. Comparison of results

The compression stress-strain relationships of the C-RMBEs and the mechanisms of damage obtained from the FEM analysis, compared to the experimental results, are presented in this section. Fig. 12 shows the average experimental compressive stress-strain curves and the FEM numerical curves of the C-RMBEs. It is clear that the initial elastic stage of the FEM calculated curves shows a good agreement with the experimentally measured curves. However, the FEM presents over-estimates in the post-peak behavior,

Table 4
Numerical and experimental results of the RM C-RMBEs for the observed maximum stress, strains at the peak stress, strains at 75% and 50% of the peak stress, and strain ductility.

C-RMBE ID	Maximum stress, f_{max} (MPa)			Strain at peak stress, ϵ_{max}			Strain at 75% of the peak stress, ϵ_{75}			Strain at 50% of the peak stress, ϵ_{50}			Strain ductility, μ	
	Experimental		FEM	Experimental		FEM	Experimental		FEM	Experimental		FEM	Experimental	FEM
	C-RMBE specimen	Ave. and COV. (%)		C-RMBE specimen	Ave. and COV. (%)		C-RMBE specimen	Ave. and COV. (%)		C-RMBE specimen	Ave. and COV. (%)			
BE-U-0-1	20.37	18.83	18.81	0.0019	0.00195	0.0018	0.0023	0.0024	0.0024	0.0030	0.0031	0.0033	1.59	1.83
BE-U-0-2	17.29	12%		0.0020	4%		0.0024	3%		0.0032	5%			
BE-R-0-1	25.6	23.80	23.40	0.0022	0.0023	0.0020	0.0031	0.0032	0.0030	0.0036	0.0037	0.0038	1.60	1.90
BE-R-0-2	24.00	8%		0.0024	4%		0.0035	7%		0.0037	3%			
BE-R-0-3	21.90			0.0023			0.0031			0.0038				
BE-R-10/200-1	27.5	25.32	25.35	0.0028	0.0026	0.0024	0.0033	0.0031	0.0029	0.0044	0.0045	0.0050	1.70	2.08
BE-R-10/200-2	24.62			0.0025	6%		0.0030	5%		0.0044	5%			
BE-R-10/200-3	22.5	13%		0.0026			0.0031			0.0048				
BE-R-15/200-1	26.15	25.53	25.59	0.0028	0.0029	0.0028	0.0032	0.0033	0.0036	0.0067	0.0062	0.0065	2.15	2.32
		3%						4%			11%			
BE-R-15/200-2	24.9			0.0030	5%		0.0034			0.0057				
BE-R-10/100-1	28.82	25.83	25.85	0.0023	0.0026	0.0027	0.0040	0.0038	0.0039	0.0069	0.0069	0.0080	2.71	2.97
BE-R-10/100-2	25.3	11%		0.0029	12%		0.0038	5%		0.0066	4%			
BE-R-10/100-3	23.4			0.0025			0.0036			0.0071				
BE-R-15/100-1	27.44	26.40	26.46	0.0031	0.0029	0.0030	0.0045	0.0044	0.0048	0.009	0.0103	0.0102	3.58	3.40
BE-R-15/100-2	27.35	7%		0.0029	5%		0.0044	2%		0.011	11%			
BE-R-15/100-3	24.30			0.0028			0.0043			0.011				

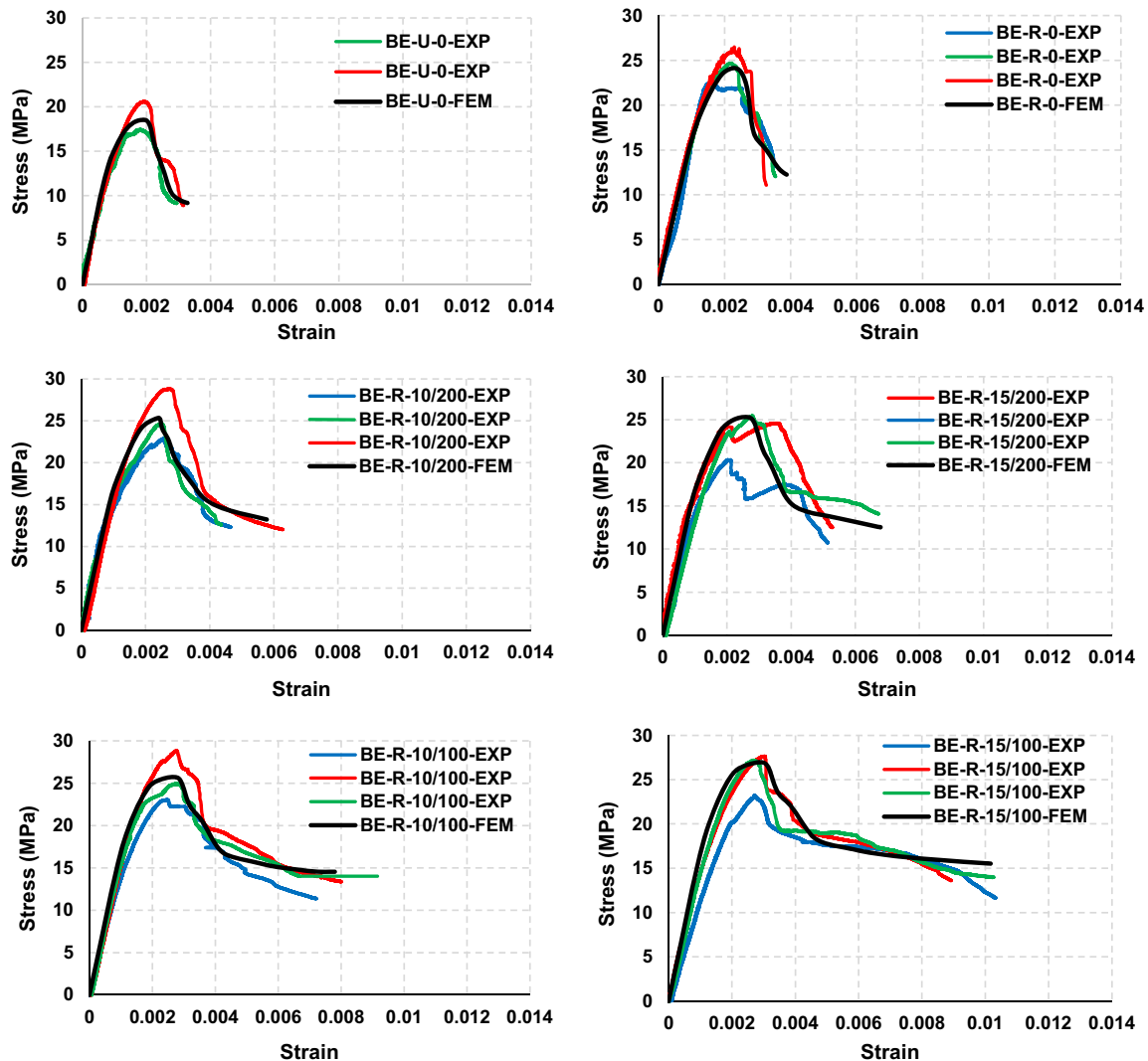


Fig. 12. Average stress-strain curves of unreinforced and reinforced C-RMBE obtained by the numerical and experimental work.

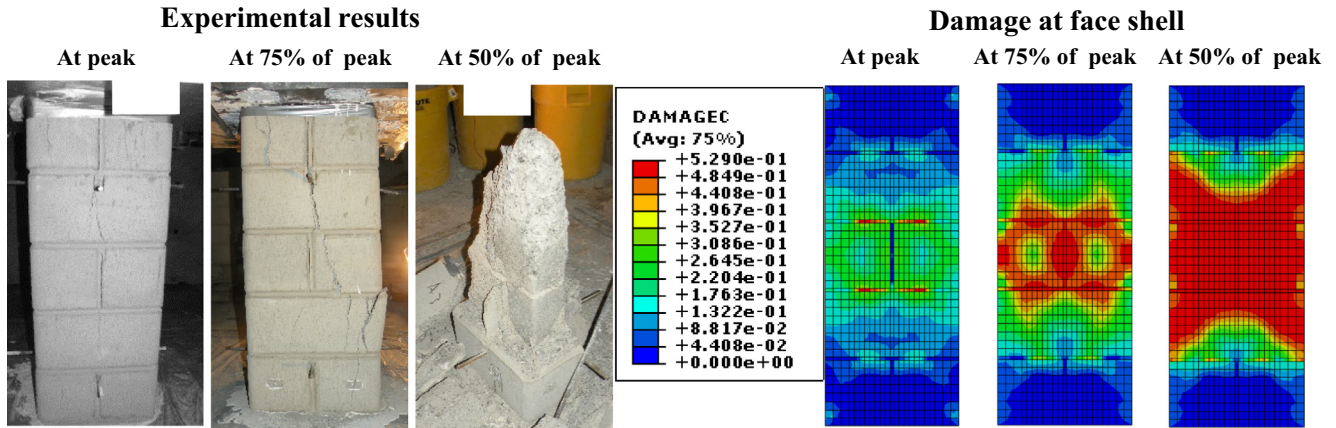


Fig. 13. Comparison between the damage results from the FEM and experimental tests for the unreinforced C-RMBE.

which can be attributed to the assumption of a perfect bond between the steel reinforcement and the grout and the perfect bond assumption between the block units, grout and mortar. However, the overall trends of the stress-strain curves obtained from the FEM show satisfactory agreement with the experimental results, and in general, the trends are quite similar.

4.2. Definition of compression damage

The compression damage parameter, d_c , is shown in the output as DAMAGEC, which is used in the concrete damage plasticity model as a numerical indicator of material degradation in the post-peak range. The damage parameter values varied from zero, representing undamaged material, to one, which represents a total loss of strength. The damage variable, d_c , was computed using the following equation (Batikhia and Alkam [32], Jankowiak and Lodygowski [33] and Yu et al. [34]):

$$d_c = (\sigma_{cu} - \sigma_c) / \sigma_{cu} \quad (4)$$

where σ_{cu} is the compression ultimate strength and σ_c is the compression stress along the descending stress-strain curve. The attainment of a 50% reduction in the strength of the materials ($d_c > 0.5$) is used as an indicator of the extensive damage and spalling of the materials.

4.3. The unreinforced C-RMBE (Units BE-U-0)

As shown in Fig. 13, the unconfined unit failed in a brittle manner, as initiated by vertical splitting cracks of the face shells of the C-shaped concrete blocks, with an increase in the density of crack-

ing followed by spalling of the face shells. At the ultimate load, the strength of the test unit decreased quickly, and the cracking increased due to the crushing of the grouted core.

As shown in Fig. 12, the average maximum compressive stress of the test unit was 18.83 MPa, and the corresponding maximum strain was 0.00195. It can be observed that the descending branch reached 50% of the strength with an average strain of 0.0031. Fig. 13 shows the comparison of the damage result from the FEM analysis and the damage from the experimental tests, where the damage parameter (d_c) provides an indication of the damage propagation in the C-RMBE after the attainment of the peak load.

4.4. The vertically reinforced block C-RMBE (Units BE-R-0)

The vertically reinforced C-RMBE test units showed the first signs of failure immediately before experiencing the maximum compression load on the rising curve of the compressive stress-strain relationship. When the maximum load was reached, the vertically reinforced C-RMBE (BE-R-0) exhibited a similar cracking pattern as the unreinforced C-RMBE. However, the resulting 1.27-fold increase in strength compared to the unreinforced units (BE-U-0) was due to the presence of the vertical reinforcement, as shown in Fig. 12. The post-peak behavior was similar to that of the unreinforced units, with a steep slope after the peak load. Fig. 14 shows the damage of the vertically reinforced C-RMBE and a comparison with the damage pattern recorded from the numerical simulation.

The damage mechanisms of the units are as follows: vertical splitting cracks were observed at the peak loads, followed by spal-

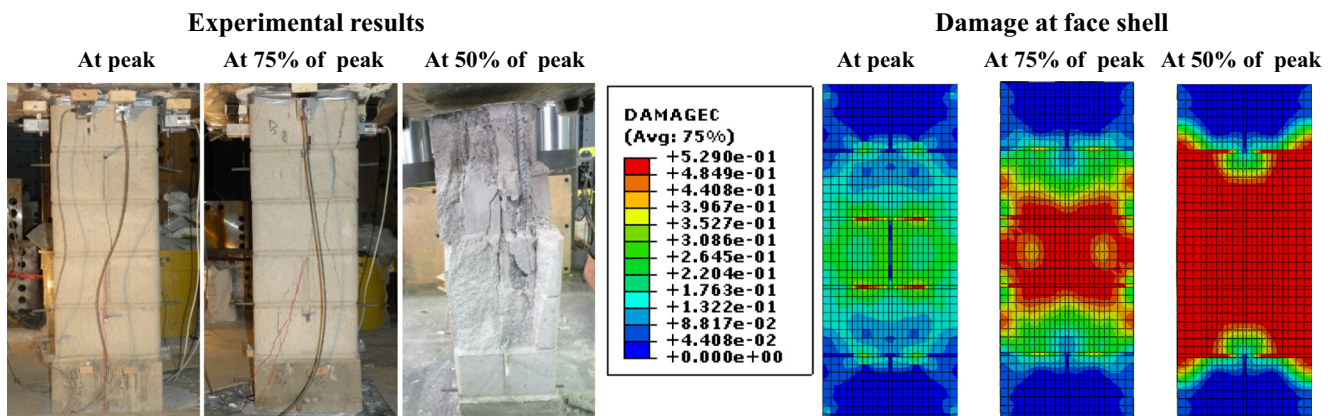


Fig. 14. Comparison between the damage results from the FEM and the experimental tests for the vertically reinforced C-RMBE.

ling of the face shells and simultaneous core crushing and vertical bar buckling. Fig. 14 shows the damage parameter, d_c , for the face shell of the concrete blocks. The extensive damage and spalling of the concrete blocks are indicated by the red color range, with an average value of d_c of 0.5. On the other hand, the post-yield stress in the longitudinal steel bars was used as an indicator for buckling. At the peak load, the yield stress was attained for the vertical bars. In the post-yield range, a reduction in the stress in the vertical bars was observed and provided an indication of bar buckling. This is compatible with the modeling assumptions shown in Fig. 10b for the stress-strain curves of the vertical bars in compression because the ratio of the spacing between the hoops and the diameter of the longitudinal bars is greater than 8. As seen in this figure, when the yield point is attained, the bars become unstable. Thus, the stress in the steel decreases linearly as the strain increases. Such a behavior has been observed experimentally, as reported by Yalcin and Saatcioglu [28]. The average maximum compressive stress of the test unit was 23.80 MPa, and the corresponding maximum strain was 0.0022. As shown in Fig. 12, the descending branch reached 50% of the strength, with an average strain of 0.0037.

4.5. The reinforced confined C-RMBE (BE-R-10/200 and BE-R-15/200)

Test units BE-R-10/200 and BE-R-15/200 were confined by hoops of 10 M@200 mm and 15 M@200 mm, respectively. As shown in Fig. 12 and Table 4, the confinement reinforcement had a significant effect on the post-peak behavior and produced a more gradual post-peak stress-strain curve. A slight 1.06-fold increase in the strength compared to the vertically reinforced units was

observed. Fig. 15 shows the comparison between the damage progression between the experimental and numerical results. In the experimental work, the damage mechanisms of the units were as follows: vertical splitting cracks at the peak load followed by spalling of the face shells and simultaneous core crushing and buckling of the vertical bars. The test unit BE-R-10/200 confined by hoops obtained a maximum strength of 25.32 MPa, resulting in a 34.5% higher strength compared to unreinforced C-RMBE test units BE-U-0 due to the presence of the confined reinforcement hoops. The strains at 75% and 50% of the strength were 0.0031 and 0.0045, respectively, which represent increases of 29% and 45%, respectively, compared to the unreinforced units (BE-U-0) due to the effects of the higher hoop confinement. For the unit BE-R15/200, the maximum strength of 25.53 MPa resulted in a maximum of 35.6% higher strength compared to test units BE-U-0. The strains at 75% and 50% of the strength were 0.0033 and 0.0062, respectively, resulting in an average increase in the strain of 37.5% and 100%, respectively, compared to the unreinforced units (BE-U-0) due to the effects of the higher hoop confinement.

4.6. The reinforced confined C-RMBE (BE-R-10/100 and BE-R-15/100)

The C-RMBE BE-R-10/100 and BE-R-15/100 behaved similarly to their counterparts C-RMBE BE-R-10/200 and BE-R-15/200, with a milder strain softening and gradual post-peak behavior due to the presence of the higher confinement ratio. The strain ductility after obtaining the maximum strength was increased due to the closer spacing of the confinement transverse reinforcement (i.e., 100 mm instead of 200 mm). Fig. 16 shows the comparison

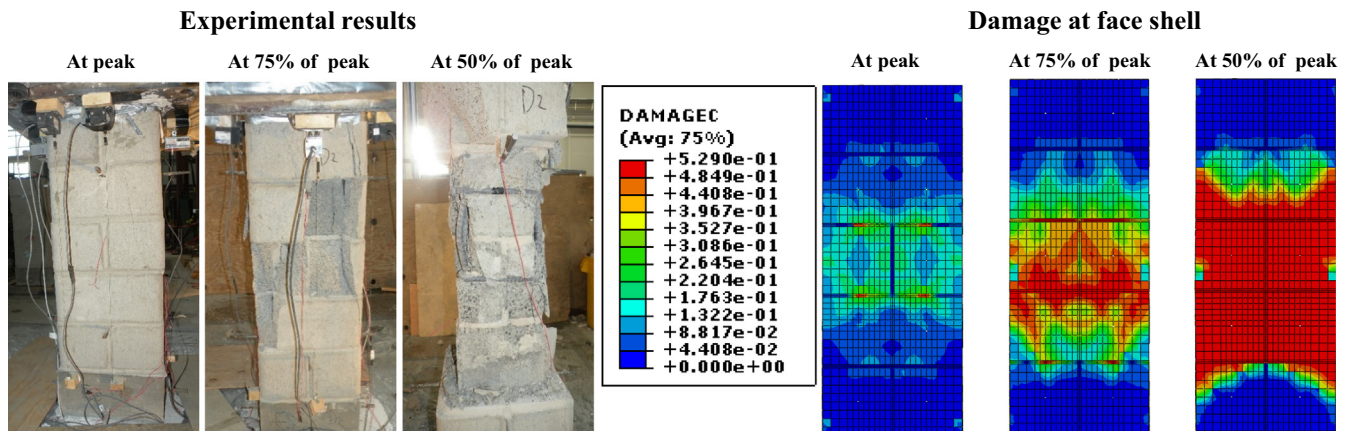


Fig. 15. Comparison between the damage result from the FEM and the experimental tests for C-RMBE BE-R-10/200 and BE-R-15/200.

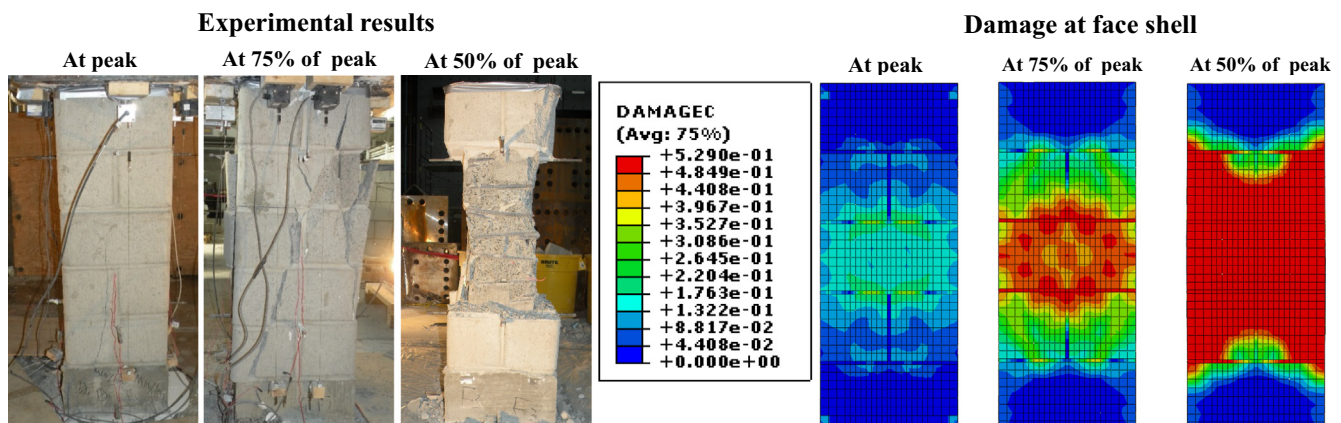


Fig. 16. Comparison between the damage results from the FEM and the experimental tests for C-RMBE BE-R-10/100 and BE-R-15/100.

between the damage obtained from the FEM predictions and the experimental tests. From Fig. 12, it can be observed that the test unit BE-R-10/100 confined by hoops obtained a maximum strength of 25.83 MPa, representing a 37.2% higher strength compared to test units BE-U-0 due to the presence of the confined reinforcement hoops. The strains at 75% and 50% of the strength were 0.0038 and 0.0069, respectively, representing increases of 58.4% and 122.5%, respectively. For the unit BE-R15/100, the maximum strength reached was 26.4 MPa, resulting in a 40% higher strength compared to test units BE-U-0. The strains at 75% and 50% of the strength were 0.0044 and 0.0103, respectively, representing average increases of 83% and 232% compared to the unreinforced units BE-U-0.

5. Effect of confinement ratio

Fig. 17 shows the correlation between the confinement ratio and the ratio of the maximum strength of the confined units to the unreinforced units based on the numerical and experimental results. For each unit, the peak stress of the confined unit is normalized by the peak stress of the unconfined unit, where $f_{\max,c}$ is the average strength of the confined C-RMBE and $f_{\max,u}$ is the average strength of the unreinforced C-RMBE. It can be observed that the confinement introduces an increase in the axial strength in the range of 1.19–1.46 compared to the unreinforced units. On the other hand, an increase in the confinement reinforcement provided an increase in the range of 1.25–1.87 and 1.42–3.55 for the 75% and 50% strains, respectively, compared to the unreinforced units and had a significant effect on the post-peak behavior, with softening of the descending branch of the stress-strain curve, as shown in Figs. 18 and 19.

The effect of the confinement ratio on the compressive strain is substantially more pronounced than that on the compression strength. In addition, the post-peak compressive strain behavior of the confined C-RMBE depends on the confinement ratio of the transverse reinforcement. As the confinement ratio increases, the strains at 75% and 50% of the peak stress increase, and the severe deterioration of the C-RMBEs after the peak stress is reduced. Hence, the strain ductility increases, leading to an increase in the wall's curvature ductility. From the regression analysis presented in Fig. 17, the observed trend representing the line of the value of $f_{\max,c}$ compared to the $f_{\max,u}$ based on the experimental average, the numerical results and 95% of the data above it, which is determined by subtracting 1.645 times the standard deviation from the experimental average, are given by the following equations:

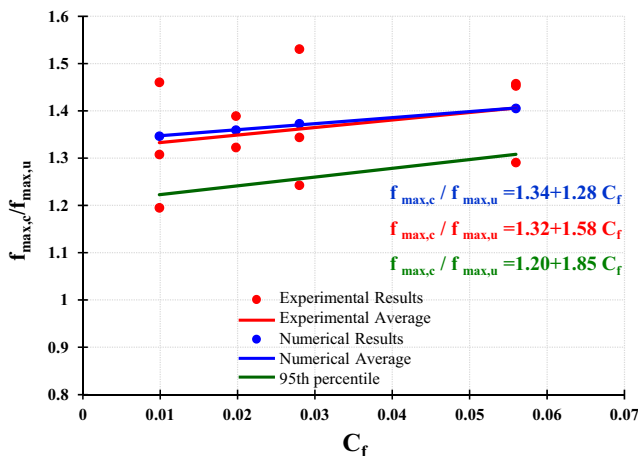


Fig. 17. Correlation between the strength enhancement ratio and the confinement ratio based on experimental tests and numerical results.

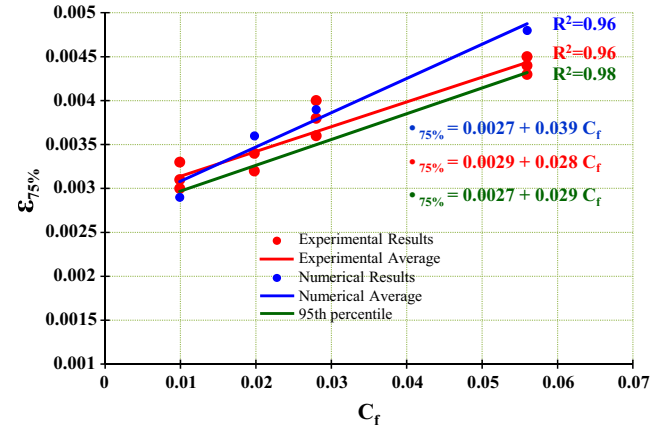


Fig. 18. Correlation between the strain at 75% strength reduction and the confinement ratio based on the experimental tests and numerical results.

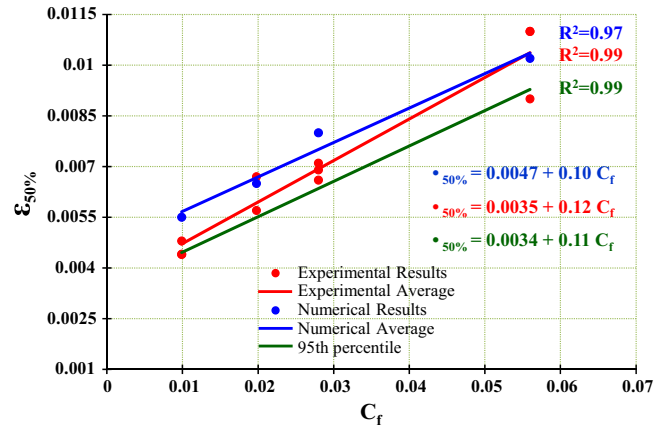


Fig. 19. Correlation between the strain at 50% strength reduction and the confinement ratio based on the experimental tests and numerical results.

$$f_{\max,c}/f_{\max,u} = 1.34 + 1.28C_f \quad \text{for } C_f \geq 0.01 \quad (\text{Numerical results}) \quad (5)$$

$$f_{\max,c}/f_{\max,u} = 1.32 + 1.58 C_f \quad \text{for } C_f \geq 0.01 \quad (\text{Experimental average}) \quad (6)$$

$$f_{\max,c}/f_{\max,u} = 1.20 + 1.85 C_f \quad \text{for } C_f \geq 0.01 \quad (95\text{th percentile}) \quad (7)$$

The values of the strains at 75% ($\epsilon_{75\%}$) and 50% ($\epsilon_{50\%}$) of the peak stress of the C-RMBEs are obtained from equations based on the regression analysis with the confinement ratio. Eq. (8) ($R^2 = 0.96$ and $\text{RMSE} = 4.71 \times 10^{-8}$) and Eq. (9) ($R^2 = 0.97$ and $\text{RMSE} = 1.12 \times 10^{-7}$) represent the numerical results, where R^2 is the coefficient of determination and the RMSE is the residual mean square error. Eq. (10) ($R^2 = 0.96$ and $\text{RMSE} = 3.67 \times 10^{-8}$) and Eq. (11) ($R^2 = 0.99$ and $\text{RMSE} = 3.81 \times 10^{-4}$) represent the experimental average. Eq. (12) ($R^2 = 0.98$ and $\text{RMSE} = 3.22 \times 10^{-8}$) and Eq. (13) ($R^2 = 0.99$ and $\text{RMSE} = 4.15 \times 10^{-4}$) represent the line that considers 95% of the experimental results above it, which is determined by subtracting 1.645 times the standard deviation from the experimental average, as shown in Figs. 18 and 19.

$$\epsilon_{75\%} = 0.0027 + 0.039 C_f \quad \text{for } C_f \geq 0.009 \quad (\text{Numerical results}) \quad (8)$$

$$\varepsilon_{50\%} = 0.0047 + 0.10 C_f \quad \text{for } C_f \geq 0.009 \quad (\text{Numerical results}) \quad (9)$$

$$\varepsilon_{75\%} = 0.0029 + 0.028 C_f \quad \text{for } C_f \geq 0.009 \quad (\text{Experimental average}) \quad (10)$$

$$\varepsilon_{50\%} = 0.0035 + 0.12 C_f \quad \text{for } C_f \geq 0.009 \quad (\text{Experimental average}) \quad (11)$$

$$\varepsilon_{75\%} = 0.0027 + 0.029 C_f \quad \text{for } C_f \geq 0.009 \quad (95\text{th percentile}) \quad (12)$$

$$\varepsilon_{50\%} = 0.0034 + 0.11 C_f \quad \text{for } C_f \geq 0.009 \quad (95\text{th percentile}) \quad (13)$$

The current North American masonry codes apply a displacement-based method for the evaluation of the inelastic rotation demand and capacity of shear walls. CSA-S304 [2] estimates the rotation capacity of shear walls based on a maximum compression strain of 0.0025. If the rotation demand from seismic loading exceeds the rotation capacity, the compression region of the wall must be detailed to produce an increased compressive strain capacity in the masonry either by adding boundary elements or using another technique that can be demonstrated through testing. CSA-S304 [2] does not provide a correlation between the specific detailing for confinement reinforcement and the corresponding improvement in the strain capacity and thus requires testing and analysis to satisfy the strain demand requirements. Similarly, the MSJC-2013 [1] code requires testing to be conducted to verify that the provided confinement detailing is capable of ensuring a strain capacity in the boundary element that would be in excess of the maximum imposed strain. The conducted research in this paper provides a contribution to bridge the gap in knowledge on the correlation between the detailing of confinement reinforcement and the ratio to the increase in the compression strain. Such correlations were developed in this paper. These correlations can be used to update the design methods in masonry standards by calculating the required confinement ratio based on the imposed strain demand corresponding to the earthquake-induced design inelastic rotation demand.

6. Conclusions

The prediction of the deformation and ductility capacity of RM walls with boundary elements requires an investigation of their compression behavior. This article presented experimental and numerical investigations of the effects of the confinement ratio on the compression stress-strain behavior of RM boundary elements at the ends of RM shear walls. Full-scale C-RMBE test units were constructed and tested under axial compression loading until failure. An FEM procedure was developed and validated with the experimental results. The FEM considered the specific stress-strain characteristics of the grout, mortar and concrete blocks. In this study, the influence of different configurations of the confinement reinforcement on the compression behavior, as well as a comparison between the experimental and numerical results, including the compression stress-strain curves and damage progression, was presented. The results showed that increasing the confinement reinforcement ratio significantly affected the post-peak behavior. This was indicated by a softening of the descending branch of the stress-strain curves resulting in an increase in the post-peak strain at 75% and 50% of the compressive strength of approximately 1.25–1.87 times and 1.42–3.55 times that of the unreinforced units, respectively. Regression equations were devel-

oped to correlate the confinement ratio with the corresponding improvement in the strain capacity. The proposed FEM procedure provided a good approximation of the compression stress-strain behavior in the elastic and inelastic ranges and captured the influence of the confinement ratio on the compression response of the C-RMBE. The obtained results from this study provide greater insight into the complete compression stress-strain behavior of confined concrete masonry boundary elements, an area with limited study in the literature. This is particularly important since recent masonry codes and standards in North America do not provide guidelines on how to correlate specific detailing for confinement reinforcement to the corresponding improvement in the compression strain capacity.

7. Recommendations for future research

The following lists some recommendations for future research toward improving the knowledge on the seismic behavior and design of RM shear walls with boundary elements:

- Extending the experimental and numerical methodology developed in this paper to investigate the effects of design variables, such as the compressive strength of the constituent materials, and to update the proposed regression equations with additional data.
- Testing of full-scale shear walls with different configurations of boundary elements to develop recommendations for their seismic design.
- Developing numerical models for shear walls with boundary elements to evaluate the effects of confinement at the boundary elements on the ductility capacity of the wall. This can be performed using the validated FEM approach applied in this study.
- Based on these experimental and numerical studies, correlations between displacement ductility, inelastic rotation and the required compression strain at the boundary elements can be developed. These correlations will help in providing recommendations for confinement reinforcement for achieving a specific level of ductility.

Acknowledgments

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