A FORCE-BASED MACRO-MODEL FOR REINFORCED CONCRETE MASONRY WALLS SUBJECTED TO QUASI-STATIC LATERAL LOADING

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ABSTRACT

This paper presents a part of an ongoing research to investigate the system level behaviour of mid-rise buildings having reinforced concrete masonry (RCM) walls as their main gravity and lateral load resisting systems. Numerical models are developed and calibrated for individual walls to simulate their behaviour under lateral loads. Since multiple walls were to be incorporated in a single building model, it was essential that wall model formulations are simple and consume little computational time while capturing the wall behaviour with high accuracy. This is a complex procedure since these walls are composed of multiple materials with nonlinear behaviour while being subjected to reversed loading. OpenSees and Response-2000 software were used to develop the wall models to capture flexure and shear deformation as well as stiffness variation under cyclic loading. Walls were represented by 2D Force-Based frame elements provided in OpenSees and experimental results of ten RCM walls tested under quasi-static cyclic loading were used to verify the developed models. The development and verification of the technique were carried out in two phases. In the first phase, wall models were subjected to monotonic pushover lateral load and were calibrated to capture the backbone curve of the experimental cyclic loading with a high degree of accuracy. In the second phase the same models were refined to capture the hysteretic behaviour. Finally, recommendations and governing factors are given for the simulation of RCM walls under cyclic loading using OpenSees to capture combined flexure and shear behaviours.

KEYWORDS: *concrete masonry, shear walls, lateral load, cyclic, modeling, OpenSees*

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INTRODUCTION

This paper presents an approach for the numerical simulation of Reinforced Concrete Masonry (RCM) walls subjected to cyclic lateral loading using macro-models. The wall is simulated as a single force-based frame element. The model was calibrated against the experimental results of six RCM walls experimentally tested by Shedid et al. [1]. Such calibration required thorough investigation of the factors affecting the behaviour such as the plastic hinge length, the shear deformation of walls and the tension stiffening behaviour of RCM. Each of these factors is discussed herein under along with its effect on the model results.

EXPERIMENTAL VERIFICATION

The experimental results of six fully grouted RCM walls tested by Shedid et al. [1] were used to verify the modeling technique adopted in this paper. In their study, Shedid et al. tested all wall specimens under fully reversed displacement-controlled quasi-static cyclic loading; the walls were cycled up to 50% degradation in strength in order to obtain enough information on their post peak behaviour. The wall details and materials are shown in the following sections.

Walls Details

A series of three story (Phase I) and two story (Phase II) high half scale walls were constructed using half scale replicas of the full scale (20-cm) concrete blocks. All walls had a length of 1.8m and a height of 4.0m and 2.66m for three-story and two-story walls respectively. The test matrix is shown in Table 1 and wall reinforcement details are shown in Figure 1.

Table 1: Wall Specimen Details (Shedid et al., 2010)

Specimen	Wall Dimensions (Length X Height)	Vertical Reinforcement[a]	Horizontal Reinforcement[b]	Axial Stress (MPa)
W1	1802 x 3990 mm	19M10	1 D4 at 95mm[c]	1.09
W2		11M10		0.89
W3		11M10		0.89
W4	1802 x 2660 mm	19M10	2 D4 at 95mm	1.05
W5		11M10		0.88
W6		11M10		0.88

[a]Area of M10 bars = 100 mm^2
[b]Area of D4 bars = 25.4 mm^2
[c]Reinforcement in the first story, for the rest of the wall 1 D4 at 190mm is used

Material Properties

Grouted masonry prisms were constructed and tested in compression; the average prism compressive strength (f'm) was 16.5 MPa at a strain of 0.0016. Tension tests were conducted on reinforcement. An elastic plastic idealization for the stress strain curve resulted in an average yield strength of 495 MPa and 534 MPa for the M10 and D4 bars respectively with a Young's modulus of 200.6 GPa [1].

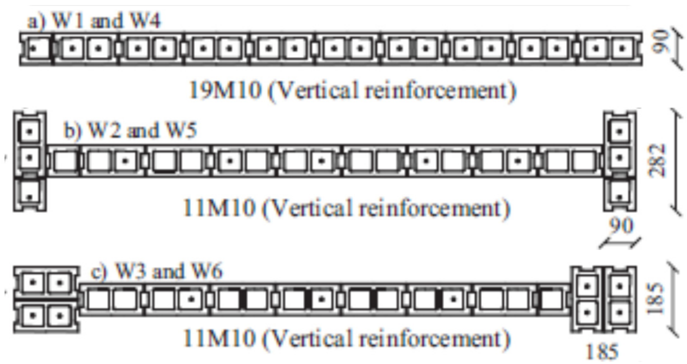


Figure 1: Wall Reinforcement Details (Shedid et al., 2010)

ANALYTICAL VERIFICATION

The verification of analytical models was carried out over two phases. In the first phase, initial models for the six walls were developed and subjected to a monotonic pushover displacement controlled lateral load. The load-displacement curves of the models were verified against the envelopes of the experimental push and pull cycles. The aim of this phase was to develop a modeling technique capable of capturing the overall behaviour of single walls.

In the second phase the analytical models developed in phase I were subjected to cyclic loading similar to that applied in the experiment. The analytical push and pull cycles were then verified against those experimentally acquired. The aim of this phase was to enhance the original modeling technique developed in phase I to capture the cyclic behaviour.

Modeling Approach and Element Type

Two approaches can be generally used for modeling of RCM walls, namely, micro-modeling and macro-modeling [2]. In micro-modeling the wall is discretized into a finite number of elements representing its constituent materials and their interface. This modeling strategy is used for investigating the local behaviour of elements, and is considered time consuming and impractical for capturing the global response.

In macro-modeling the wall is represented as a single frame element with material properties equivalent to the collective properties of the constituent elements (the masonry prism in this case). This modeling technique requires much less computational time and is adequate for investigating the global behaviour of elements. Hence; macro-modeling was the method of choice for this study. OpenSees [3] and Response-2000 [7] software were used to develop the macro-models for the walls.

Wall specimen is represented as a 2D force based (FB) beam-column element with distributed plasticity [3]. The FB formulation does not require meshing to capture the highly inelastic curvature distribution of the wall, i.e. the wall can be represented as a single element [4]. Each element is assigned fibre sections modeling the masonry and reinforcement parts of the wall cross section.

Material Models
Several material models are provided by OpenSees for concrete and steel. For this study, Thorenfeldt concrete model (Concrete06) was used, which depends on the compressive and tensile strengths, strain at compressive and tensile strengths and other factors which control the post peak curve of concrete; these factors were calibrated to fit the stress strain curve of tested masonry prisms as shown in Figure 2. Giuffre-Menegotto-Pinto model (Steel02) was used to model reinforcement. This is defined by the yield strength, initial elastic modulus, post-yield tangent modulus and other constants that control the transition from elastic to plastic zone [3].

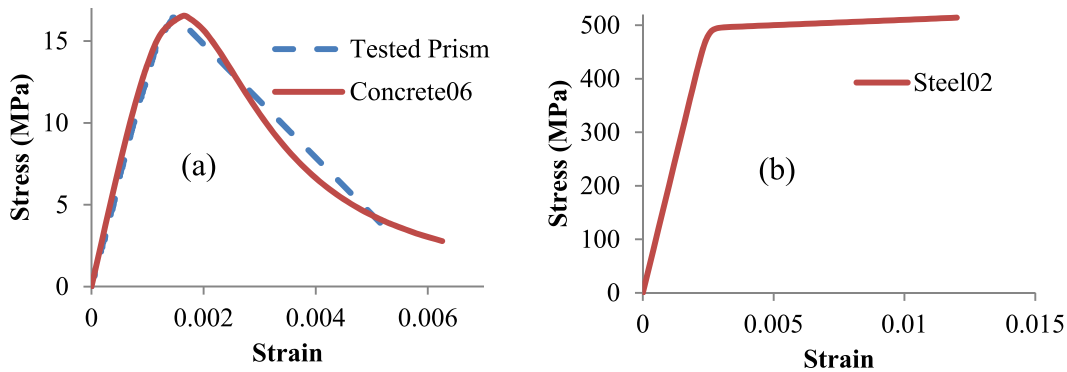


Figure 2: Grouted Masonry Material Model (Concrete06) (a), Steel Material Model (Steel02) (b)

PHASE I: PUSHOVER ANALYSIS
In this phase, each wall model was subjected to a monotonic displacement controlled top lateral load. The results of the analysis were verified against the push and pull envelopes of the experimental cyclic analysis. Figure 3 (a) shows the results of the initial model for W1, it showed very rapid strength degradation after reaching its peak, this behaviour was the same in all six wall models. It was obvious that the used model needed refinement to capture the actual behaviour. This required in depth study of three factors as described by the following sections.

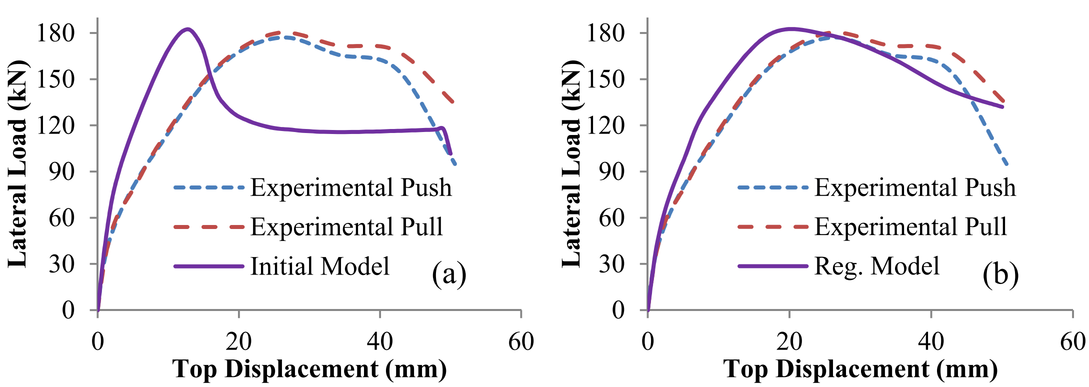


Figure 3: Pushover analysis results of initial wall model (a) and regularized wall model (b) for wall W1

Localization of Plasticity

The solution of force based elements is done through getting sectional forces at a finite number of integration points along the element length. In distributed inelasticity elements which undergo a softening behaviour, plastic strains tend to concentrate at the section which first plasticizes (bottom integration point in this case). This explains the rapid strength degradation and implies that as the weight of the first integration point decreases the element will show more abrupt strength degradation. This phenomenon is referred to as localization of plasticity [4].

Many regularization techniques are proposed in literature to overcome this phenomenon, the technique of choice for this study was the one developed by Coleman and Spacone (2001). They proposed modifying the concrete stress-strain relationship to maintain constant fracture energy after the initiation of strain-softening. They modified the descending branch of the Kent and Park concrete stress-strain model (Figure 4 (a)) based on the weight of the integration point using the equation (1) [4].

$$\varepsilon_{20} = \frac{G_c^f}{0.6 f_c^{'} L_{ip}} - \frac{0.8 f_c^{'}}{E_c} - \varepsilon_c \tag{1}$$

Where G^f_c is the fracture energy, L_{ip} is the weight of the integration point, ε_c is the strain at compressive strength and ε_{20} is the strain at 80% strength degradation (See Figure 4). The equation proposed above was used considering ε_{20} constant and L_{ip} variable in order to avoid changing the material model. Hence; the integration weight of the first integration point was changed to fit the material model.

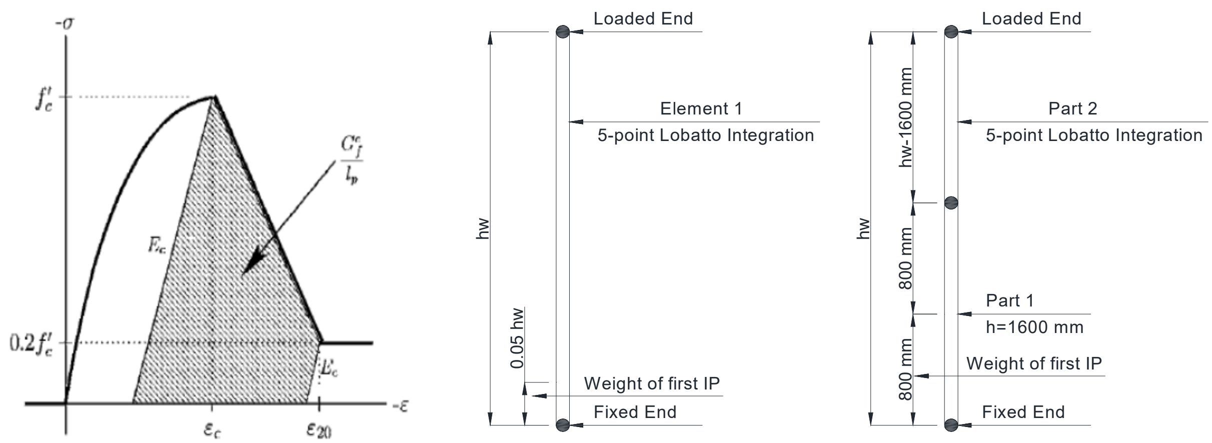
For the concrete model used here (Figure 2), ε_{20}=0.0057, G^f_c =40N⁄mm giving L_{ip} ≅800mm. Finally, a 7-point user defined integration method was used. The weight of the bottom integration point was set to 800 mm another point was added at 1600 mm with a weight of 800 mm so that the bottom part is based on a two-point Gauss Lobatto integration while the rest of the wall was divided based on the five-point Gauss Lobatto integration as shown in Figure 3. Figure 3 (b) shows the results of the regularized model for W1.

Tension Stiffening

One aspect that needed further investigation is the tensile strength and tension stiffening curve of grouted masonry. The tension branch in the chosen material model (Concrete06) is defined by the tensile strength, strain at tensile strength and a factor (b) that controls the post peak behaviour. The ACI 530-05 recommends a value of 1.72 MPa for the grout modulus of rupture [5]; this was used as an initial value for the tensile strength. Sokolov (2010) studied tensile behaviour of concrete and related it to the reinforcement ratio by equation (2) [6].

$$\sigma_{ct} = \begin{cases} E_c \varepsilon_{ct} & \varepsilon_{ct} < \varepsilon_{cr,EC2} \\ f_{t,EC2}\left(1 - 0.27 \ln\left(\frac{\varepsilon_{ct}}{\varepsilon_{cr,EC2}}\right) - 0.21\rho\right) & \varepsilon_{ct} > \varepsilon_{cr,EC2} \end{cases} \tag{2}$$

Where E_c is the concrete modulus of elasticity, ρ is the reinforcement ratio (%), $f_{t,EC2}$ and $\varepsilon_{cr,EC2}$ are the concrete tensile strength and cracking strain respectively according to EuroCode 2 [6]. This equation was used to modify the tension branch. f_t=1.72 MPa was used instead of $f_{t,EC2}$, and $\varepsilon_{cr}=f_t/E_m$ was used instead of $\varepsilon_{cr,EC2}$. The results of this calibration are shown in table 2 and Figure 5. The effect of this calibration on W1 is shown in Figure 6 (a).



The fracture energy is represented by the shaded area (a)

Initial Model (b)

Regularized Model (c)

Figure 4: Kent and Park concrete stress–strain model (Coleman and Spacone, 2001) (a); Schematic diagrams for the initial wall model (b) and regularized wall model (c)

Table 2: Modified Tension Branch Factors

Wall	ρ (%)	σ_{ct} (MPa)[a]	ε_{cr}[b]	b
W1, W4	1.17	1.3	8.65E-05	0.86
W2, W3, W5, W6	0.55	1.52	1E-04	0.67

a Modified masonry tensile strength

b $\varepsilon_{cr}=\sigma_{ct}/E_m$

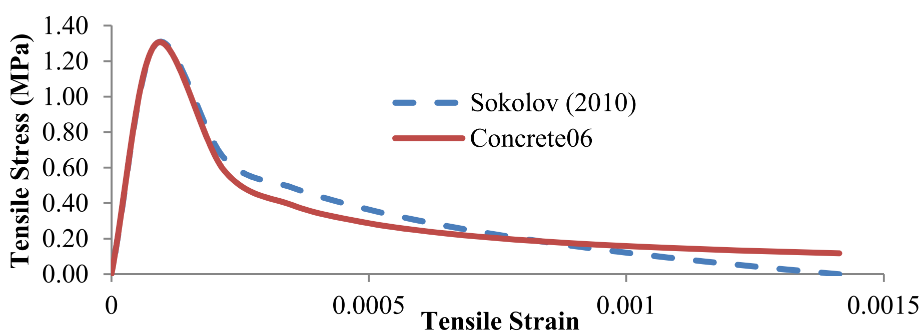


Figure 5: Calibration of grouted masonry tensile branch (ρ=1.17%)

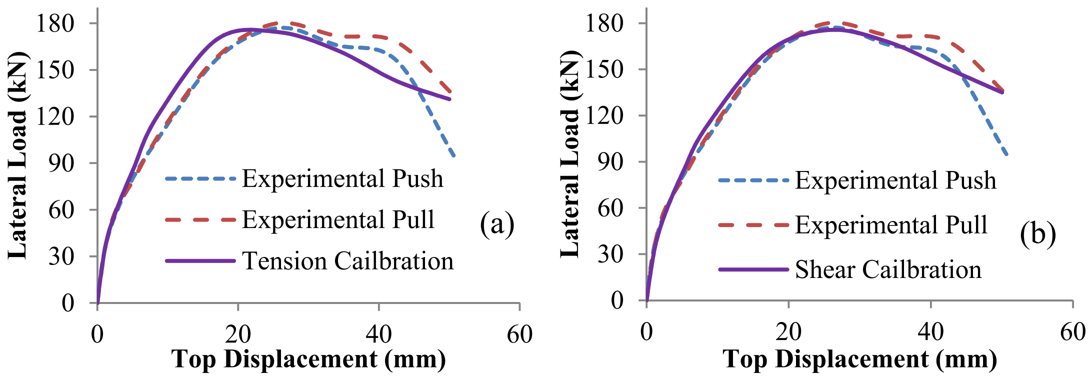


Figure 6: Pushover analysis results for W1 after calibrating tension stiffening (a) and shear deformations (b)

Shear Deformation

The FB element used to represent the walls calculates only flexural deformation. The last refinement of the model was to take into account the shear deformation. Two steps were followed to define shear behaviour for each wall; the first was to define the location of the section which would first undergo plastic shear deformation, the second was to idealize the shear-shear strain relation for such section into a bilinear relation.

For the first step, the full wall was modelled using Response-2000 [7] and subjected to the same loading conditions; the shear strain distribution over the wall height could be extracted for each loading step until reaching the initiation of plastic shear deformation. Figure 7 (a) shows the application of this step to W1.

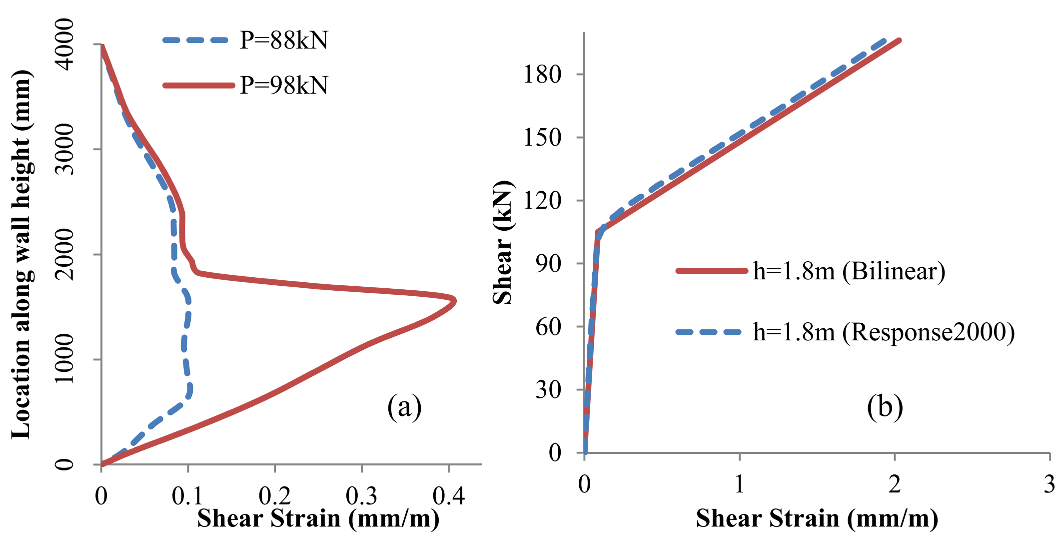


Figure 7: Shear Strain Distribution over W1 Height (Response-2000) (a), Bilinear Idealization of Shear Behaviour for W1 at a height of 1.8m (b)

It can be seen from the Figure that the first section to undergo inelastic shear deformation was at a height of around 1.8 m at a lateral load of approximately 100 kN. For the second step, only the section at which plastic shear deformations were first encountered was modeled with Response-2000. Incremental shear force and bending moment were applied to the section depending on its location. Finally, the shear-shear strain relation for this section was extracted and idealized to a bilinear relation. Figure 7 (b) shows the application of this step to W1.

Hence, the bilinear relation was applied to all integration points below this section using section aggregator in OpenSees. It is interesting to note that many modeling techniques apply shear deformations to walls by introducing an inelastic shear spring at a height (ch_w) referred to as the wall center of rotation, where (c) is the ratio of the wall height to its center of rotation measured from its bottom. In their attempt to evaluate the factor (c) using experimental results Vulcano et al. (1988), Orackal et al. (2004) and Massone and Wallace (2004) all suggested a value of c=0.4. This is consistent with the location of plastic section for shear in this study except for W4 as shown in table 3. Figure 6 (b) shows the effect of this refinement on W1.

Table 3: Shear Parameters for Wall Models

Wall	$P_{Plastic\ Shear}$ (kN)	$h_{Plastic\ Shear}$ (m)	$c = h_{PS}/h_w$
W1, W2, W3	100	1.8	0.45
W4	150	1.6	0.60
W5, W6	140	1	0.38

PHASE II: CYCLIC ANALYSIS

Phase I set the basis for developing a sound wall model which captured the behaviour under pushover lateral loading accurately. The aim of phase II was to calibrate the model to capture the wall behaviour under cyclic lateral loading.

The six wall models were subjected to quasi-static cyclic lateral loading similar to which was performed experimentally. The results of the initial models were characterized by wider loops than the experimental results and nearly no strength degradation as shown in Figure 8 for W1. Studying the behaviour of both masonry and steel components at each loading cycle and comparing them to the experimental observations, it was found that some parameters needed to be calibrated.

The concrete model used (Concrete06) defines the cyclic behaviour of the material by two factors, α1 and α2 for behaviour in compression and tension respectively. Initial values used for these factors were α1=0.32, α2=0.08 as suggested in OpenSees manual. In addition, it was found that a crushing strain lesser than that used in the pushover analysis was needed. An upper bound for the tensile strain was also a controlling factor. The crushing strain and maximum tensile strain were applied using the "MinMax" material available in the OpenSees library. Table 4 shows the final values used for the concrete parameters.

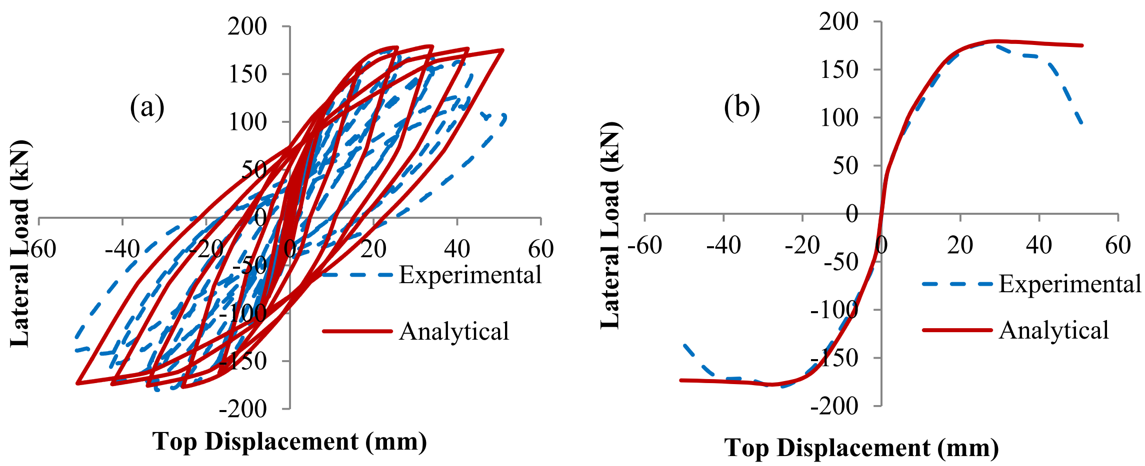


Figure 8: Initial Cyclic Behaviour of W1. Hysteresis Loops (a) Envelope (b)

For the steel model (Steel02), its cyclic behaviour is defined by three factors, R0, cR1 and cR2. The initial values used as recommended by OpenSees were R0=20, cR1=0.925, cR2=0.15. These factors were calibrated to modify the Pinching behaviour of steel hence solve the wide loops seen in the hysteresis. In addition a buckling strain and a fracture strain were assigned to the steel model using the "MinMax" material.

The buckling strain is assigned as a minimum compressive strain after which the steel material is assumed to lose all of its resistance either in tension or in compression. When this was first applied it was found that the model loses strength much more rapidly than the experimental results. It was hypothesized that this is due to the fact that when a bar buckles it retains a fraction of its strength, opposed to fracture after which a bar has no contribution. Hence; it was decided that a bar would be modeled as two fibres each having a percentage of the bar area; one of them is assigned the buckling strain while the other is assigned only a fracture strain. The optimum area for the part assigned a buckling strain was 25-30% of the bar area. Table 4 shows the final values used for the steel parameters.

Table 4: Cyclic Parameters for Concrete06 and Steel02 Material Models

Material	Concrete06				Steel02				
Parameter	α1	α2			R0	cR1	cR2		
Definition	Parameter for compressive plastic strain definition	Parameter for tensile plastic strain definition	Crushing Strain	Maximum Tensile Strain	Parameters to control the transition from elastic to plastic branches.			Buckling Strain	Maximum Tensile Strain
Value	0.03	1e-5	-0.004	0.015-0.025[a]	10	0.95	1	-0.005	0.1

[a]Maximum values for flanged walls

Using the above mentioned factors greatly enhanced the model results and nearly matched them to the experimental results. Figure 9 shows the results of this enhancement for W1. The results of the cyclic analysis for all walls are shown in table 5.

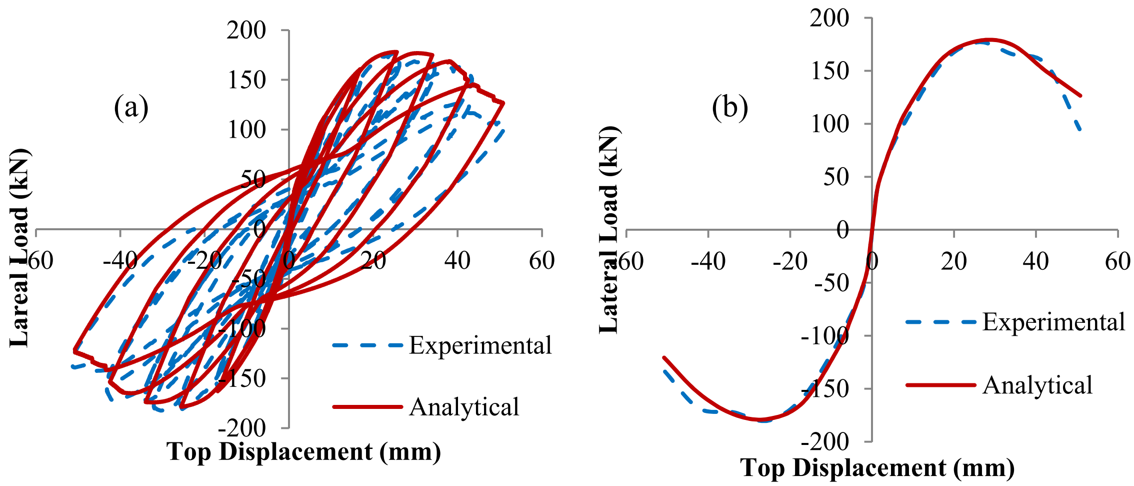


Figure 9: Refined Cyclic Behaviour of W1. Hysteresis Loops (a) Envelope (b)

Table 5: Results of Cyclic Analysis for all Wall Specimens

Specimen	Loading Direction	Experimental				Analytical			
		P_y (kN)[a]	P_u (kN)[b]	Δ_y (mm)[c]	Δ_u (mm)[d]	P_y (kN)[a]	P_u (kN)[b]	Δ_y (mm)[c]	Δ_u (mm)[d]
W1	Push Pull	101 110	177 180	8.5	25.1 25.3	113 114.5	178 178.5	8.5	25.6
W2	Push Pull	121 123	151 154	10.5	31.5	129 129	152 150	10.5	32
W3	Push Pull	110 106	152 147	9.2	36 36.1	115 116	147.5 147	9.3	36
W4	Push Pull	160 162	265 267	3.5	14.1 12.5	145 147	267.5 266	3.5	14
W5	Push Pull	185 183	245 239	5	21.2 25.1	184.5 183.5	235 230	5	25
W6	Push Pull	173 169	241 234	4	24.1 24	172 175	230 227	4.5	24

[a]Lateral Load at First Yield
[b]Lateral Load at Maximum Wall Resistance
[c]Displacement at First Yield
[d]Displacement at Maximum Wall Resistance

CONCLUSIONS

This paper presents a simplified technique for modeling Reinforced Concrete Masonry walls under cyclic lateral loading using OpenSees and Response-2000 software. The modeling technique was applied to all six wall specimens. The tested wall specimens covered a wide range of wall aspect ratios and configurations which made the technique more reliable. In Phase I, each refinement step had a consistent effect as follows:

- Regularizing the weight of the bottom integration point solved the rapid strength degradation caused by localization of plasticity
- Modifying the tension model for masonry based on the reinforcement ratio enhanced the initial stiffness of the model and better captured the strength
- Applying elasto-plastic shear-shear strain relations to the walls added the shear deformation component which resulted in a closer match between analytical and experimental response

Phase II showed that the modeling technique is adequate for capturing the cyclic behaviour. Certain values were recommended for the factors controlling the cyclic behaviour of masonry and steel material models, and a method was proposed to model the buckling of reinforcement. The maximum error obtained from all six models for yield load, maximum load and deformation at maximum load were 9.3%, 3.9% and 8.8% respectively. The results showed that the technique predicted the wall behaviour with a high degree of accuracy, in addition to consuming very little modeling and computational time.

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