Finite Element Modelling of Confined Masonry Wall under In-plane Cyclic Load

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Abstract. Confined masonry (CM) is an effective building construction technology for low-tech construction in seismic-prone regions. This paper focuses on developing a numerical model to study the in-plane seismic behaviour of CM wall. A half-scaled CM wall specimen is designed based on Mexican practice and tested under a combination of vertical load and in-plane cyclic load up to failure. Based on that a three dimensional (3D) finite element macro-modelling technique is developed using the commercial software package ABAQUS, where the masonry panel, RC tie-frame, and reinforcements are discretely modelled. The strength and stiffness degradation of masonry and concrete is defined using Concrete Damaged Plasticity (CDP), which is a damaged plasticity-based continuum constitutive model. All the input data in the numerical model are based on experimental test results and data obtained from past literature or inverse fitting, with justification. From the analysis, it is observed that the numerical results are in good agreement with the experimental. Therefore, the proposed model can be successfully practiced to model confined masonry walls.

1. Introduction
Confined masonry (CM) construction has emerged as a building technology that offers an alternative to both unreinforced masonry (URM) and infilled reinforced concrete (RC) frame construction in several countries or regions of high seismic risk including North and South American, European, African and Asian countries. The uniqueness of the CM building primarily lies in the construction technique, where masonry walls of the building are constructed prior to the casting of small RC tie-elements (columns and beams) around the wall and some other specific locations. The improved seismic performance is mainly due to this confinement and bond formation at wall-to-tie interface [1]. Although basic design guidelines of CM building have been addressed in various country codes, modelling and analysis of CM structure appears to be a challenging task. The finite element (FE) modelling method is a universally accepted analysis method, which provides opportunity to study the structure more thoroughly. FE analysis of masonry structures can be performed in two different level - micro or macro. Micro-modelling approaches are mostly suited for small structural elements in order to closely represent the heterogeneous states of masonry using the properties of each element and interface [2, 3]. However, macro-modelling approaches are used to represent the global structural behaviour, where the distinction between the individual units and the joints is not made and the material parameters are obtained from the masonry tests under homogeneous states of stress [4, 5, 6]. In this study, an experimental test was conducted on a typical CM wall and based on that a FE model was proposed and validated.
2. **Experimental study to understand the In-plane behaviour of CM wall**

A half-scaled model of a typical CM wall was considered for the study. The wall was designed and detailed (figure 1) as per Mexican guidelines [7]. Material properties, as given in table 1, were evaluated using relevant standards by conducting tests on masonry, concrete, and reinforcing bars. Masonry panel was constructed atop the foundation beam using solid clay bricks in English bond pattern with 1:1:6 (lime: cement: sand) mortar. Confining RC (frame) elements were constructed after the masonry panel. An RC slab of 75 mm thickness was constructed over the specimen to study its influence on the behaviour. As shown in figure 2a, the imposed (dead and live) load was applied on the test specimen by placing steel plates over the RC slab (around 17 kN) to simulate the loading of a typical two-story residential building on an exterior ground floor wall. The wall was tested under pseudo-static cyclic lateral loading applied at the slab level using servo-controlled hydraulic actuator of ±125 mm stroke length and 250 kN load capacity. The in-plane loading consisted of displacement-controlled slow cycle of gradually increasing story drifts from 0.10% till failure as per ACI 374.1-05 [8]. Three cycles of each displacement level were applied and the response was recorded using a data acquisition system. The obtained response of the specimen in the form of hysteretic for first cycle of every displacement level along with the envelope curve is shown in figure 2b.

![Figure 1](image1.png)  
**Figure 1.** Geometry and details of reinforcement of the considered CM wall.

![Figure 2](image2.png)  
**Figure 2.** Experimental study: (a) Test set-up, and (b) Hysteretic response with envelope.

| Table 1. Properties of the CM wall specimen. |
|--------------------------------------------|
| Compressive strength of concrete ($f_{ck}$) | 25MPa |
| Compressive strength of masonry prism ($f'_m$) | 5.2 MPa |
| Elastic modulus of masonry ($E_m$) | 2000 MPa |
| Yield strength of reinforcing bars ($f_y$) | 446MPa(3φ); 549MPa(6φ) |

Note: φ represents diameter of bar
3. Finite element (FE) modelling scheme
To capture the behaviour of the tested specimen with FE method, the selected discretization scheme and constitutive model are explained below:

3.1. Discretization scheme for masonry and RC tie-elements
In the present study, macro-modelling technique was considered for masonry to avoid material property definition of individual unit and joint. The FE analysis was carried out using the commercial software package ABAQUS [9]. 8 noded three-dimensional solid continuum elements with reduced integration (C3D8R) from the explicit element library were used for the concrete frame and masonry. Due to the selected reduced integration, hourglass control was activated to avoid zero-energy deformation.

3.2. Discretization scheme for reinforcement
Reinforcement bars were modelled using wire feature associated with 2 noded three-dimensional truss elements (T3D2). A proper bonding between reinforcement and concrete was ensured by applying embedded body constraints. The constraints allow the reinforcement nodes to displace exactly equal to the nodes of the surrounding concrete.

3.3. Constitutive model - concrete damage plasticity model (CDP)
In this study, the nonlinear behaviour of concrete and masonry were modelled using the CDP model (figure 3) selected from the ABAQUS material library [10]. The CDP model is designed to simulate the behaviour of concrete and other quasi-brittle materials, such as masonry, in any type of structure subjected to monotonic as well as cyclic loading. This model uses the concepts of isotropic damaged elasticity along with isotropic compressive and tensile plasticity to account for the multiple damage states. The evolution of yield surface is controlled by two hardening variables, i.e., equivalent plastic strain in compression and tension, which are linked to the compressive crushing and tensile cracking failure mechanisms, respectively. The transition from tension to compression and vice versa is another important aspect that is considered in modelling the response under cyclic loading. The cracks occurred during stress reversal from compression to tension, results in reduction of stiffness and these cracks are expected to close in the next stress reversal cycle. The change in load from tension to compression leads to partial recovery of the compressive stiffness. Also, total or partial recovery of tensile stiffness is expected when uncrushed or partially crushed concrete undergo tensile loading. Therefore, the stiffness degradation variable \( d \) depends on the stress state; recovery parameters \( w \), and the degradation (or damage) parameters \( d_c \) and \( d_t \) as shown in figure 3. \( w_c = 1 \) and \( w_t = 0 \) will be used for this FE analysis. \( d_c \) and \( d_t \) were determined using Park Pivot rule model [11]. Among the input parameters for CDP, the plasticity characteristics of the materials need different types of experimental tests and in absence of such data, the parameters were determined by calibrating with the experimental results, and by using common values recommended in the literature. The dilatancy in CDP model is controlled by the parameter ‘dilation angle’ in the plastic potential function. The values of dilation angle as 30° and 12° were found to be appropriate for concrete and masonry, respectively, in order to generate the wall behavior close to experimental result. The other parameters, \( e, \sigma_0/\sigma_{c1}, K \) and viscosity were taken with their default values from ABAQUS as: 0.1, 1.16, 0.667, and 0.00001, respectively. The compressive and tensile stress-strain relationships of materials are explained in the next section.

4. Calibration of material models
4.1. Masonry
The stress-strain curve specified for masonry in compression was developed using the parabolic model proposed by Kaushik et al. [12] as shown in figure 4a. The tensile behaviour of masonry was assumed to be linearly elastic till it reaches its tensile strength, followed by a linear softening curve as shown in figure 4b. The tensile strength is estimated as a fraction of its compressive strength and for the present study, the tensile strength was assumed as 8% of the compressive strength. The post-peak behaviour was modelled in such a way that the masonry completely loses its tensile strength at ultimate strain (equal to 10 times the strain corresponding to the peak tensile strength).
Concrete and reinforcement

The Kent and Park model of concrete under compression without confinement [13] was used for the compression behaviour of concrete as shown in figure 4c. It has to be noted that the unconfined concrete model was used because the steel stirrups were modelled separately in the FE program. The tensile response of concrete was modelled, as shown in figure 4d, based on the tension stiffening model proposed by Wahalathantri et al. [14]. Elastic-perfectly plastic model was used to simulate the uniaxial response of embedded steel bars.

Figure 3. Concrete Damage Plasticity (CDP) Model of ABAQUS [12].

Figure 4. Stress-strain properties specified in CDP Model: (a), (b) Masonry, and (c) (d) Concrete.

5. Abaqus analysis procedure

To simulate the experimentally obtained behaviour of the considered wall in the numerical model, a three-dimensional (3D) FE macro-modelling strategy was adopted as shown in figure 5. Masonry panel, RC confining frame, and reinforcements were discretely modelled as different units. The masonry modelled at macro level was defined using macro-masonry properties and the confining RC elements were modelled considering the tension stiffening effect using the CDP model as explained earlier. The steel reinforcement was modelled as an embedded truss element located within the concrete using perfectly bonded constraint. The contact between the masonry panel and RC confining frame was simulated by assigning suitable interface properties. The interface was modelled with frictional properties to account for tangential slip and hard contact was assigned in the normal direction to avoid penetration of finite elements. The mesh was continuously refined to achieve convergence and a finer mesh size of 25 mm was chosen for this study. The boundary conditions were simulated in an idealised manner; the bottom face of the bottom beam was assumed to be fixed using the ‘Encastre’ boundary condition. To model the pseudo-static lateral load phenomenon, in-plane cyclic displacement-controlled loading was applied at the center of the top beam and cyclic analysis was carried out in ABAQUS explicit solver. To maintain the pseudo-static condition, the lateral displacement was applied at a reduced rate, ensuring the kinetic energy of the model does not become more than 5% of the total internal energy.
The analysis was carried out in three steps in ABAQUS. First, the initial step is mandatory and default, in which the boundary condition was initialised. In the next step, gravity load was applied along with pre-compression load, which was followed by the in-plane loading stage.

5.1. Sensitivity study
Numerical simulations are parameter sensitive. Therefore, a sensitivity study was carried out to understand influence of some parameters. As shown in figure 6a, it is observed that the results are quite stable for the mesh size below 30mm. Hence, a mesh size of 25mm, which discretise confining elements into 5 parts, was selected considering computational power required into account. Similarly, dilation angle of masonry was selected as 12 (figure 6b). At masonry panel and RC confining element interface, different permutations of possible interface scenarios were tested. As shown in figure 6c, the interfaces were modeled with penalty friction with frictional coefficient of 0.8.

Figure 5. Numerical model.

Figure 6. Sensitivity study for: (a) Mesh, (b) Dilation angle, and (c) Boundary condition.

6. Validation of the proposed FE model
The load-displacement relation obtained from the FE analysis was compared with the experimental as shown in figure 7. The pre- and post-peak behaviour of the numerical model matched the experimental results quite well. The estimated maximum in-plane strength from the numerical model (average value 63.6kN) almost matched with the experimental (average value 56.1 kN) with an error of 13.5%.

Moreover, figure 8 shows the equivalent plastic strain contour (PEEQ) in the wall obtained from the FE analysis and it was compared with the crack pattern in wall obtained from the experimental study. Dotted lines were drawn for visualization of major crack as per simulation. In the FE model, wall was modelled as a continuum (single unit) using CDP model from ABAQUS library. The discontinuity in the form of mortar joints was not explicitly considered while modelling. The obtained PEEQ signifies the diagonal cracking of wall, which provides a reasonable concordance with its experimental.
Therefore, the validation study confirmed that the developed FE macro model could effectively predict the lateral capacity and crack pattern of CM wall.

![Figure 7](image1.png)  
**Figure 7.** Comparison of numerical lateral load-displacement behaviour with experimental.

![Figure 8](image2.png)  
**Figure 8.** Comparison of plastic strain obtained from FE simulation with experimental crack pattern

7. Summary and conclusion
Confined masonry constructions have been documented as an alternative to URM and infilled RC structural system. For having a better understanding of the behaviour of CM structure, a solid CM wall with aspect ratio 1.5 was designed using Mexican guidelines and tested in the laboratory under pseudo-static cyclic lateral loading. Based on that, a 3D finite element model was developed using macro-modelling approach. The developed numerical model was validated with the experimental study. Therefore, the proposed numerical model is able to capture the nonlinear response and cracking pattern of CM wall subjected to cyclic lateral loading. It will be helpful in understanding the effect of parametric variations, such as, geometric variation, material variation, and other possible uncertainties, which will reduce the number of required experimental tests by many folds.

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