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Macro-Scale Numerical Modeling of Unreinforced Brick Masonry Squat Pier Under In-Plane Shear

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Abstract

Numerical modeling of brick masonry behaviour under different performance conditions has always remained a challenging task. Several modeling strategies have been developed for masonry, in general, through the course of time that have been simplified to speed up modeling and analysis duration. This ranges from a simplified strut model to a highly discontinuous micro-scale nonlinear model. With the current advent of high-speed computing and modeling tools, more realistic numerical modeling of masonry is now possible. In this paper, the strategy adopted is based on macro-scale modeling, where isotropic material properties are considered for the homogenous continuum. ABAQUS is used as a state-of-the-art finite element-based analysis and modeling tool. The Concrete Damage Plasticity (CDP) model is used for simulating inelastic material behaviour of brick and mortar, which is available in the ABAQUS library. This material model can be used in both implicit and explicit schemes of integration but the explicit procedure is highly preferred as it overcomes the convergence issues. Various parameters required for CDP modeling of brick and mortar are adapted from literature. The model is assembled in two parts, first part is modeled for masonry with both elastic and plastic properties, while the other part simulates a rigid beam at the top of the masonry part to create a uniform in-plane shear loading effect. The masonry part has been fixed at the bottom with free vertical ends, while horizontal in-plane displacement was applied to the top rigid beam. The load-displacement curves were generated from these models for monotonic push, to compare them with the envelopes of experimental results, loaded similarly. Since brick masonry is a highly disjointed material, it is a complicated procedure to develop an exact model *and predict its exact behaviour. However, the overall representative load-displacement curve developed numerically was in good agreement with the ones produced experimentally.*

Keywords : Macro-Scale, Masonry, Numerical Model, Squat Pier, Tension Stiffening

I. Introduction

Masonry is the oldest and most commonly used construction techniques for residential buildings. It may be used as load-bearing or non-load-bearing (architectural, partition, etc.) applications. In general, masonry is a composite of two different materials i.e. unit blocks (clay, concrete, etc.) and mortar (cement, lime, etc.). Mortar, by default, adds weak zones that have a high tendency to crack under extreme in-plane as well as out-of-plane loading conditions, for example, earthquake, blast, vehicle impact etc. These complexities in masonry make it significantly challenging to predict its failure modes, in any computational framework, about other conventionally use structural materials (concrete, steel, etc.).

There are several numerical models [I], [II], [III], [IV], based on different assumptions and characterized by different levels of details have been developed. The simplest method that is used to describe masonry comes from the mechanics of solids and allows the study of collapse through the limit equilibrium analysis. It assumes a masonry panel as a kinematic chain of rigid blocks that is described with Lagrangian displacement magnitude at one point [II]. Another possible approach for modeling masonry is the equivalent strut method (see Figure 1), which considers deformation in the elastic range possibly followed by inelastic. A method called POR [IV] was proposed, where the primary objective was to permit manual calculus to verify the masonry structure. This method was based on the shear resistance of plain masonry and simulating the story mechanism only. Masonry panels are also modeled with an approach of using macro-elements [V], for example, piers and lintels. This procedure was further improved [VI] and a kinematic nonlinear macro-element was proposed, shown in Figure 2.

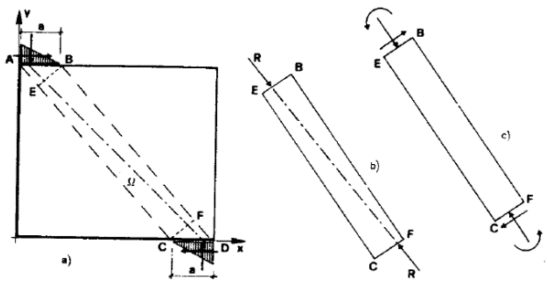

Fig 1. Masonry panel modeled through an equivalent strut [II][III]

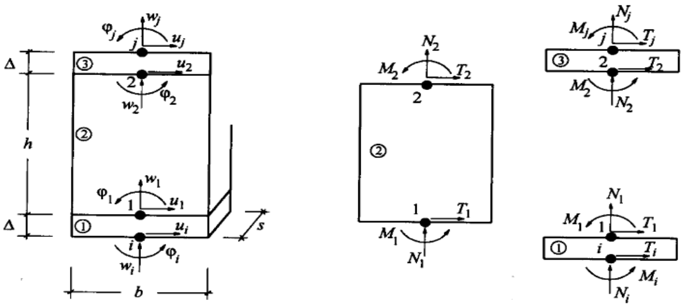

Fig 2. A kinematic model for the macro-element [VI]

With the recent developments of computational structural analysis mainly based on finite element method, that takes into account the composition of masonry, different micro and macro modeling techniques exists. These computational methods are mainly categorized into three groups, namely, detailed-micro, simplified-micro and macro models, illustrated in Figure 3 [VII]. The detailed micro-modeling is the one in which each component of the masonry wall is modeled separately in detail. This consequently provides a high level of accuracy but also increases the computational demand. In simplified micro-modeling, also known to be meso-scale modeling, brick units are assembled with an interface element of negligible thickness having properties that of mortar. This procedure is somewhere between micro and macro scale modeling with less computational demand relative to micro scale modeling. It has been suggested that the interface element between bricks can be modeled using a yield surface defined by tension and shear only [VIII]. This model has further modified [VII] by adding a compression cap to the yield surface. A two-dimensional meso-scale numerical model using rigid brick units was also developed [IX], to simulate the crack propagation in brick masonry. This study was further extended to a three-dimensional model [X] as well.

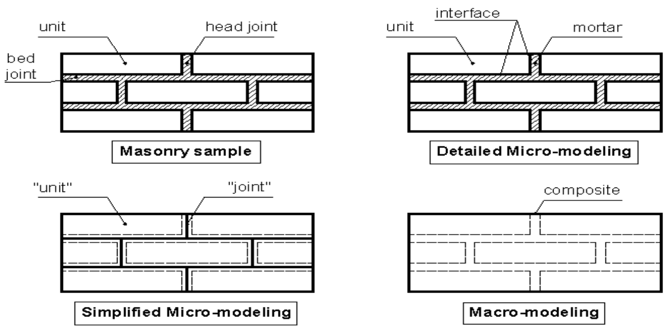

Fig 3. Modeling approaches for masonry

In the case of macro-modeling, the brick units and the mortar joints are smeared out in the continuum. The constituents are either handled mathematically by locating material within the geometry or the constitutive laws of structural elements are directly provided in terms of internal forces. Material properties necessary for macro-scale modeling may be isotropic or anisotropic depending on the level of required accuracy.

In this research, a brick masonry pier has been modelled as a simplified macro-scale three-dimensional continuum. A similar comparison [XII] has been done in the past for stone masonry walls. The material properties used in this research are isotropic and a Damage Plasticity Model has been used in ABAQUS to predict the behaviour under in-plane laterally loaded conditions. The primary aim was to validate the developed macro numerical model, by comparing its monotonic load displacement behaviour with the experimental envelope of cyclic load displacement curve of similar arrangement [XIII] by the same approach as used for stone masonry [XII] discussed earlier.

II. Numerical Model of Brick Masonry

The numeric model is based on the geometric and material model. Where, the material model has furthermore, different behaviours depending upon whether the straining is within elastic or inelastic range. This section highlights different parameters required to develop a numerical model. The loading scheme and method of analysis required to perform this study are also discussed in this section.

Geometric Model

The acquired experimental data [XII] including, geometry and material properties along with cyclic load-displacement curve have been used to develop and validate the macro numerical model. The experimental brick masonry panel was labeled as PII, and its dimensions were 1.36m x 1.28 m x 0.235 m (4.46 ft x 4.20 ft x 0.77 ft), with an aspect ratio of $h_p/l_p = 0.94$. In the ABAQUS CAE modeling environment, two parts were assembled, as shown in Figure 4, one is a rigid beam at the top, modeled with high stiffness and the second one is a brick masonry wall, modeled with typical masonry properties discussed below. Both parts are connected through tie constraints at the surface.

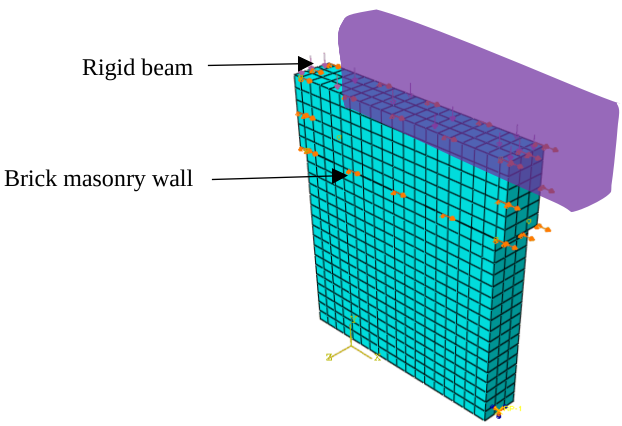

Fig 4. The assembled masonry model

Both, rigid beam and brick masonry are modeled with linear (first order) brick element with reduced, designated by C3D8R in ABAQUS. With the testing of both fine and coarse meshed models, the global seed size of 0.0762 m (3 in) is used to mesh this model

III. Material Model

Elastic Properties

Elastic modulus of the top beam was assumed to be very high E_b=20 GPa (2900.76 ksi) to account for a stiff shear transferring system (i.e., RCC Beam) at the top of the masonry wall. Although various material properties for the brick masonry are available from [XII], these models were not used because they were based on linear, bi-linear or trilinear models while Concrete Damage Plasticity (CDP) model, used in ABAQUS, requires the tabulated no-linear stress-strain history for masonry. Therefore, the compressive strength and modulus of elasticity were estimated using empirical relationships illustrated by Hemant et al.[XIII].

Hemant et al. presented a generalized relation to estimating the compressive strength of masonryf'_m, as follows.

$$f'_m = k\, f_{cb}^{\alpha}\, f_{cj}^{\beta} \tag{1}$$

Where k, α and β are the constants based on experimental data. From the available experimental results [XII] the values of 0.63, 0.49 and 0.32 are adopted, for k, α and β respectively. Using equation 1 the compressive strength for our model is estimated to be 4.78 MPa (694.5 ksi) which is a close estimate of the experimentally [XII] measured value of 4.54 MPa (658.1 psi).

The modulus of elasticity has been calculated using equation 2 provided by Tomazevic [XIV].

$$E_m = k\, f_m \tag{2}$$

Where k is an empirical constant with a value ranging from 200 to 2000 for clay brick masonry [XIV]. Hamant et al. [XIII] used k=550 for their study. In this study, the value of k=850 has been numerically selected for the model, using the back analysis approach. An elastic comparison is shown in Figure 5 and Figure 6 for different values of k value of k.

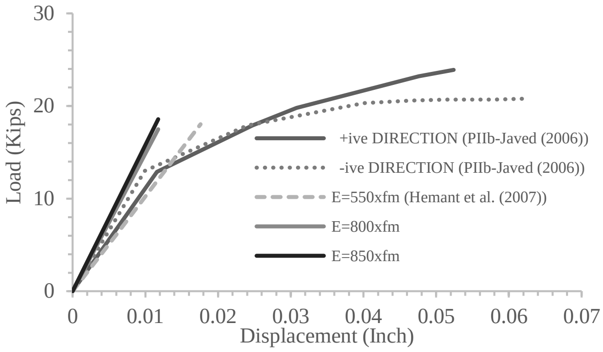

Fig 5. Elastic comparison with results of PIIb[XII]

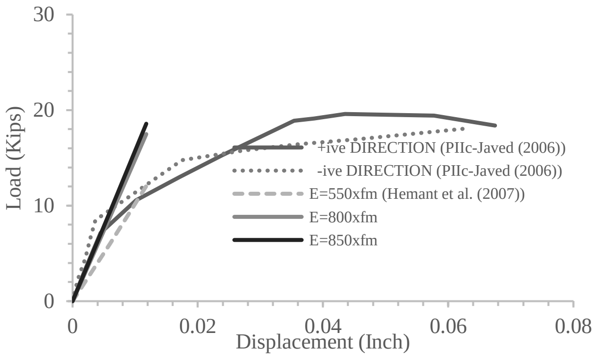

Fig 6. Elastic comparison with results of PIIc[XII]

The elastic parameter used for the numerical model is summarized in Table 1, where density and Poisson's ratio are acquired from the literature [XXII].

Table 1: Elastic Properties

	Concrete Top Beam	Masonry Wall
Density (kg/m^3)	2400	1887
Modulus of Elasticity (MPa)	20000	4070.39
Poisson's Ratio	0.15	0.2

Inelastic Properties

The Concrete Damaged Plasticity (CDP) material model is used to model the inelastic behaviour of masonry. CDP plasticity parameters are also assumed from the literature [XXV] and are summarized in Table 2.

Table 2: Plasticity Parameters

ψ	ϵ	f_{b0}/f_{c0}	K_c	Viscosity Parameter
45	0.1	1.3	0.5	0

Where, where ψ (in Degrees) is the dilation angle, ϵ is the eccentricity defining the hyperbolic flow potential behaviour. f_{b0}/f_{c0} is a factor by which equal biaxial compressive yield stress varies from initial uniaxial compressive yield stress. K_c parameter is based on the stress invariant in a three-dimensional stress state.

For compression stress-strain behaviour of masonry is based on the theoretical method [XV] by using Eq. 3 and Eq. 4, which are used to find f_m^1 , the pre-peak stress profile in masonry and f_m^2, the post-peak stress profile in masonry, shown in Figure 7.

$$f_m^1 = f_i + (f'_m - f_i)\left(\frac{2\varepsilon_m}{\varepsilon'_m} - \frac{\varepsilon_m^2}{\varepsilon'^2_m}\right)^{0.8} \tag{3}$$

$$f_m^2 = f'_m + (f_{mid} - f'_m)\left(\frac{\varepsilon_m - \varepsilon'_m}{\varepsilon_{mid} - \varepsilon'_m}\right)^2 \tag{4}$$

In Eq. 3 and Eq. 4, f'_m is the ultimate strength, f_i is the initial stress that starts from zero. f_{mid} is half of the ultimate strength. ε'_m is the peak strain at crushing strength while ε_{mid} is the strain corresponding to f_{mid}, i.e. is assumed to be 2.25 times ε'_m according to [XIII]. Figure 7 is the acquired stress-strain from equations which was used to define compression behaviour in ABAQUS.

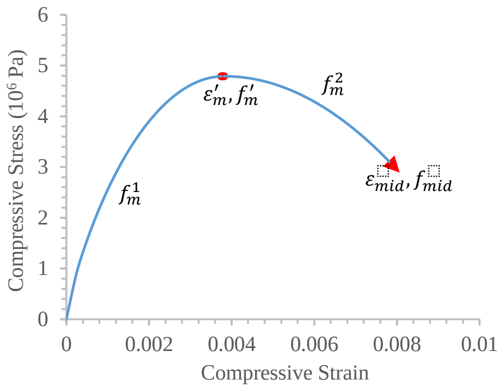

Fig 7. Empirically estimated compressive curve

The tensile strength of masonry is estimated from the equation provided by Tomazevic [XIV], which suggests that the tensile strength of masonry is 5.5% of its compressive strength. Backes [XXIII] has experimentally studied the post cracking stiffening behaviour of unreinforced masonry, wherefrom we can develop an approach to use tension stiffening property. Belarbi et al. [XVIII] and Ghiassi et al. [XIX], presented a mathematical expression for tensile behaviour of masonry and other quasi brittle material as illustrated by Eq. 5 and Eq. 6.

$$\sigma_t^1 = E_c\varepsilon_t \tag{5}$$

$$\sigma_t^2 = f_{cr}\left(\frac{\varepsilon_{cr}}{\varepsilon_t}\right)^c \tag{6}$$

Where f_{cr} is the cracking strength, ε_{cr} is the strain at fracture strength and c is the stiffening parameter that defines the post cracking sharpness of the model curve. Here c=0.2 is used for this study to represent the experimental curve presented by Backes [XXIII].

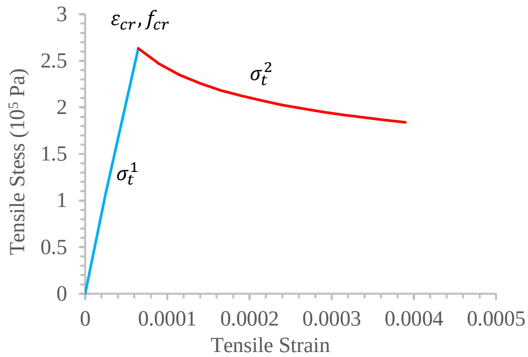

Figure 8: Empirically estimated tensile curve

The mathematical relationships for estimation of crushing and cracking strains of masonry, proposed in ABAQUS documentation [XX] and [XXI], were used to evaluate and input crushing and cracking strains for our model.

Analysis of Procedure

Among the two major analysis procedures available in ABAQUS, the explicit dynamic analysis procedure is the most used procedure for highly discontinuous numerical models. It can be used to solve the equation of motion either in dynamic or quasi-static problems. As compared to implicit procedures, the explicit methods require no iterations and hence demand less computational time. This procedure is derived from a central difference integration scheme where the time increment is the only constraint that must be controlled. To have a conditionally stable central difference operator, the time increment (Δt) should be less than the stability limit (Δt_{max}). The stability limit for the operator is given in terms of the highest eigenvalue in the system as

$\Delta t \leq {}^{2}/_{\omega_{max}}$ when damping is neglected and

$\Delta t \leq {}^{2}/_{\omega_{max}} \cdot (\sqrt{1+\xi_{max}^2} - \xi_{max})$ when damping is considered.

Where ξ is the fraction of critical damping in the highest mode. In this research, an explicit dynamic analysis procedure has been used with the value of Δt=0.2582 msec.

III. Loading and Boundary Conditions

The explicit analysis method requires the load to be defined in the time domain. Initially, the masonry wall was preloaded under constant compression from the top through the rigid beam, afterwards, lateral in-plane shear is induced in the form of displacements applied horizontally at the top rigid beam. These pre-compression and lateral displacements are applied in two distinct steps. A pre-compression load of 0.421 MPa (61 psi) is applied at the top with a ramp amplitude up to 30sec, as shown in Figure 9, to damp any dynamic effects.

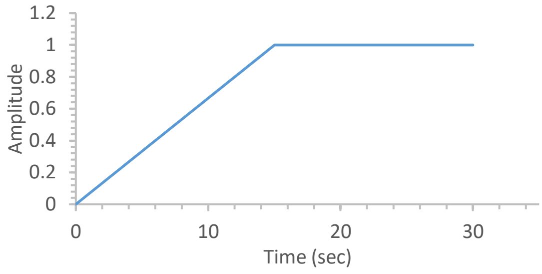

Fig 9. Amplitude applied to pre-compression

The total lateral in-plane monotonic displacement of 8 mm (0.32 in) was applied on a top rigid beam using the ramp function shown in Figure 10 for the duration of 272 sec. The loading rate was kept at 0.03mm/sec which was according to the tests performed by Tomazevic [XXII].

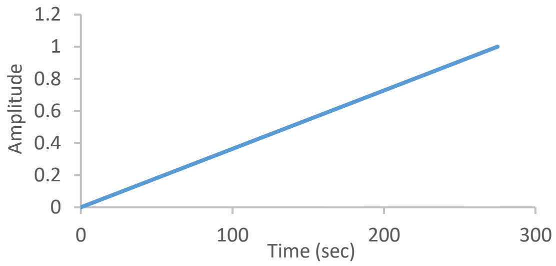

Fig 10. Amplitude applied to lateral displacements

The base of the masonry wall model was assigned an ENCASTRE (fixed) boundary condition. This boundary condition was applied along with the pre-compression step, before the application of displacement in the lateral direction.

IV. Results and Discussion

The load-displacement results obtained from ABAQUS are compared with two results i.e. PIIb and PIIc In [XII], three piers were tested for the type labeled as PII. and are illustrated in Figure 11 and Figure 12 respectively. Wherefrom the overall trend of behaviour was found in good agreement with the envelopes of experimental results.

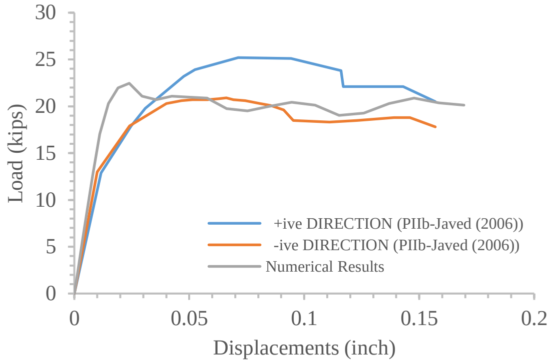

Fig 11. Comparison of numerical results with PIIb experimental envelopes.

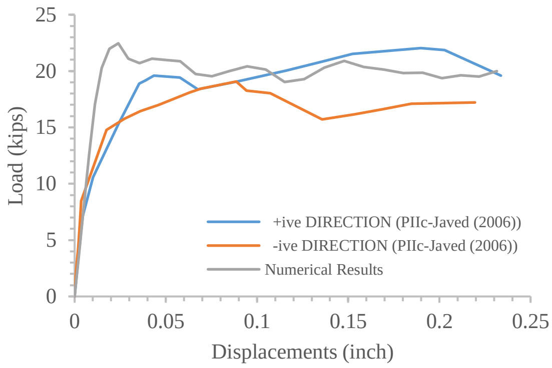

Fig 12. Comparison of numerical results with PIIc experimental envelopes.

From experiments, in PIIb the first significant crack was observed at a lateral load of 123.6 MPa (17.94 kips) in the negative direction while the diagonal cracks were observed at 159.8 MPa (23.16 kips) in the positive direction. The maximum lateral load was 173.7MPa (25.18 kips) in a positive direction and 143.6 MPa (20.82 kips) in a negative direction. In the case of PIIc, lateral load at the first significant crack and the diagonal crack was the same i.e. 18.84 kips and the maximum lateral load was 151.8 MPa (21.94 kips) in a positive direction and 131.4 MPa (19.04 kips) in a negative direction.

Numerically it can be observed from Figure 11 and 12 that the maximum lateral load is 155 MPa (22.46 kips). Since the model is macroscale therefore damages in the section can be observed in terms of maximum principle plastic strains, see in Figure 13.

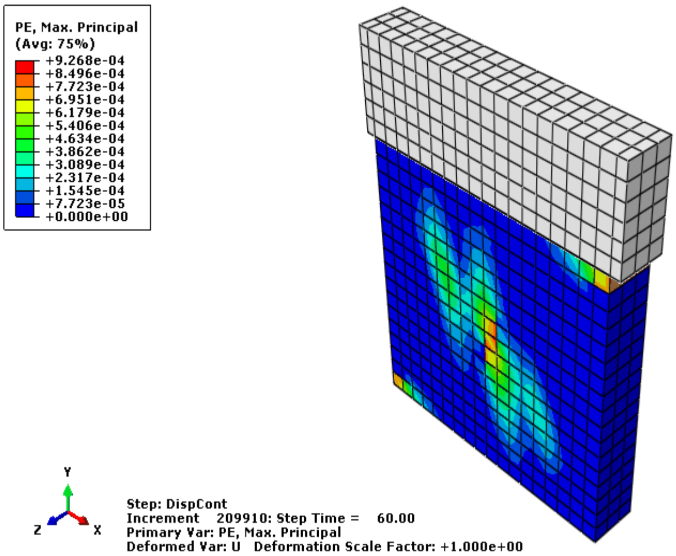

Fig 13. Maximum Principle plastic strains at maximum lateral load

V. Conclusion

This manuscript described a three-dimensional macroscale numerical strategy for brick masonry under in-plane shear. Model from Javed [XII] was selected and was simulated for a lateral load in a monotonic direction. Numerical load-displacement results were compared to experimental envelope curves and the overall behaviour was found in good agreement. From the study, it was revealed that the stress-strain curve for tensile behaviour played an important role and that the diagonal damage is greatly dependent on it. A tension stiffening analogy was developed and used to model diagonal damage behaviour of brick masonry. The model also showed certain levels of plastic damages at two alternate corners showing flexural cracks. Furthermore, the explicit procedure was found very helpful for a highly discontinuous material model like the one used in this study i.e. Concrete Damaged Plasticity. The modeling strategy offers efficient and a relatively reduced computational demand. More work, however, is still needed to further enhance the computational efficiency of the developed numerical modeling strategy.

Conflict of Interest:

Authors declared : No conflict of interest regarding this article

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