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COMPUTATIONAL MODELLING OF HYBRID MASONRY SYSTEMS

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Abstract. In this work we investigate the seismic strength, behavior, and performance of the hybrid masonry structural system. The computational modeling efforts aim to characterize the inelastic behavior of hybrid masonry panels. In particular, we study the influence of the boundary conditions (gap or no gap, reinforced or bearing contact zone), the story shear, overturning moment, and the influence of the panel aspect ratio.

Several computational models with various levels of complexity are used in our study in an effort to identify the simplest model capable of capturing the salient features of these structural systems. A non-linear (plastic) constitutive model for the simulation of masonry is considered; this constitutive model is coupled with a damage mechanics model to simulate both the inelastic deformation of masonry in normal compression and tension and the damage due to cracking and micro-cracking.

In what we call type I hybrid masonry, the masonry does not make direct contact with the beams or columns of the steel frame. The frame and the masonry are connected only through connector plates. The hybrid masonry provides many advantages, such as improving the resistance to seismic loads, impeding the extent of the damage in the masonry and so on. There is no gap between beam and masonry for Type II, so the masonry shares the gravity load with the steel frame and benefits from the vertical compression. Type III is an extension of Type II systems with the addition of connectors along the sides of the panel, which resist vertical shear forces. For Type II and Type III hybrid masonry, because the beam is in contact with the masonry, another very important aspect needs to be considered for the numerical simulation: we require a contact formulation capable of modelling the transfer of normal and tangential forces between steel and masonry.

Preliminary computational results are presented in this paper that will be in the future correlated with laboratory test results from the large-scale tests done at the University of Illinois at Urbana Champaign.

1 INTRODUCTION

Reinforced masonry panels can be designed as stiff, strong and ductile panels, interacting with the surrounding steel frame to resist lateral seismic forces. In the hybrid masonry structural system, the panels are not only used to provide spatial functionality in a building, but they also be enhance the seismic performance. This structural system is designed in such a manner that steel frames are attached to masonry panels and will transfer part of the loading (e.g., gravity forces, story shears and overturning moments) to the masonry. The panel itself can be reinforced with horizontal and vertical bars. In what we call type I hybrid masonry, the masonry does not make direct contact with the beams or columns of the steel frame. The frame and the masonry are connected only through connector plates.

The hybrid masonry system improves the resistance to seismic loads and limits the extent of the damage. Because of the panel reinforcement, the deformation in the masonry is small. Two other systems, designated as Type II and Type III may be considered. There is no gap between beam and masonry for Type II, so the masonry share the gravity load with the steel frame and benefits from the vertical compression. Type III is an extension of Type II systems with the addition of connectors along the sides of the panel, which resist vertical shear forces. For the simulation of Type II and Type III hybrid masonry, due to the transmission of forces between the steel frame (beams) and the masonry panels through direct contact, a formulation capable of modelling the transfer of normal and tangential forces between steel and masonry needs to be considered for the numerical simulation.

For the numerical simulation we use FEAP [1], an open source finite element code, which provides a framework for finite element simulations; specific constitutive models and solution schemes are included via user functions. The formulations used in this study are a mix of FEAP original elements and user routines.

The rest of this paper is organized as follows: In the next section we briefly introduce various computational models that will be utilized in our study. The following section then introduces some preliminary numerical simulations. Through several parametric studies we investigate the influence of nonlinearity, of damage and of panel aspect ratio on the transfer of loads between the frame and the panel. The paper concludes with a summary of the important features of computational models for hybrid masonry, a discussion of the influence of nonlinearities and damage on the distribution of loads between the frame and the panels and a discussion of future work.

2 COMPUTATIONAL FRAMEWORK

An isotropic model is used to describe the linear elastic constitutive behavior of the steel frame. The formulation can account for shear deformation. Extensions to large deformations are also available (with or without shear effects). Through the built-in FEAP elements, we also have access to a more complex model for the frame that includes plasticity and also accounts for geometric nonlinearities. For this option the inelastic behaviour is accounted for in the bending and axial effects but the element retains elastic response in the transverse shear terms.

The masonry panels are modeled with 2D plane strain or plane stress continuum elements. Constitutive models ranging from linear elasticity to large deformation plasticity are considered. The damage and the plastic deformation effects are included in a homogenized framework. From a numerical implementation point of view, the standard predictor corrector approach is used. In a first step, an elastic predictor is calculated. Then a plastic corrector is used to obtain the stress by an implicit Euler backward integration scheme. The stress can be expressed [2] as

$$\sigma_{n+1} = \sigma_n + D(\Delta\epsilon_{n+1} - \Delta\epsilon^p_{n+1}) = \sigma^{trial}_{n+1} - \Delta\lambda_{n+1} D \frac{\partial g}{\partial \sigma}|_{n+1}, \tag{1}$$

where subscripts indicate the step number, σ and ϵ are the stress and strain tensors, $\Delta\lambda$ is the increment of the plastic multiplier rate, and g is the plastic potential. Considering the additional equation enforcing the yield condition, we have a system of nonlinear equations having as unknowns the stress and the plastic multiplier rate that can be solved at every integration point using a local Newton-Raphson iteration. Different plastic potentials can be used to model masonry, such as the J2 or Mohr-Coulomb flow rule [2, 3].

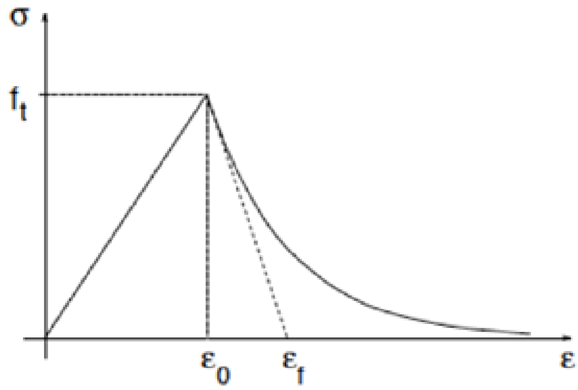

Figure 1: Uniaxial stress-strain curve based on an exponential softening model

Due to the brittle behavior of masonry, the computational framework should also include a damage mechanics model to simulate both the inelastic deformation of masonry in normal compression and tension and the damage due to cracking and micro-cracking. An effective simulation of the progressive deterioration of the mechanics properties of masonry panels under increasing loading can be obtained in the conventional framework of continuum damage mechanics. For this study, a scalar damage model is adopted with a single parameter, the damage coefficient d used to evaluate the effective stress $\sigma_d = (1 - d)\sigma$ (where σ_d is the stress in the damaged configuration, and σ is the stress corresponding to an undamaged state). A possible expression for d [4] that models the case of exponential softening is given below and requires the definition of two parameters, ϵ_0, the strain at the peak stress and ϵ_f a strain parameter controlling the initial slope of the softening branch (Figure 1).

$$d(\epsilon) = 0 \quad if \quad \epsilon < \epsilon_0 \tag{2}$$

$$d(\epsilon) = 1 - \frac{\epsilon_0}{\epsilon} exp(-\frac{\epsilon - \epsilon_0}{\epsilon_f - \epsilon_0}) \quad if \quad \epsilon_0 < \epsilon \tag{3}$$

In this context, d has the role of a reduction factor that accounts for the effect of damage in the material: $d = 0$, corresponds to a state where no damage is present in the material, while $d = 1$ represents a completely damaged state. In other words, d can be seen as the ratio of damaged area to the total cross sectional area [5].

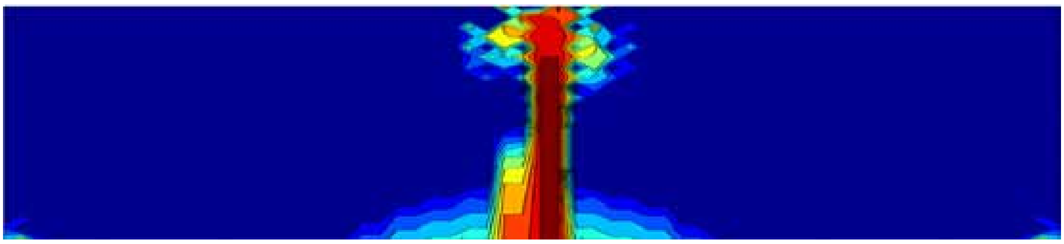

Figure 2: Distribution of damage in a simply supported unnotched beam subject to a concentrated load at mid span (blue: no damage to dark red: complete damage/ crack)

Figure 2 shows the distribution of damage in a simply supported beam under a concentrated load captured with our current implementation of such formulation (using a local damage model). This implementation is used for all numerical results that include damage presented in the next section. Work is currently underway to extend this model to a nonlocal model in order to avoid the mesh size effects that are characteristic for local formulations.

In the case of the type I hybrid masonry, there is no direct contact between the masonry and the steel frame. The transmission of loads can only occur through the masonry-frame (beam) connectors in this case. Type II and III hybrid systems however, reduce or even eliminate the gaps and direct contact becomes possible. Two fundamentally different approaches can be used to model contact in the finite element framework. The traditional formulation uses the so-called node-to-surface approach where the contact constraints are enforced strongly at every node in contact. This approach is computationally efficient but numerically not very robust, in particular, in the presence of large sliding or of significant difference in the stiffness properties of the bodies in contact. An alternate approach relies on the use of a mortar formulation [6, 7]. Both formulations are available as user subroutines in our code.

3 NUMERICAL RESULTS

We present in this section some preliminary numerical results obtained for a reduced scale (1:5) model that will be used by our collaborators at the University of Illinois, Urbana-Champaign to calibrate the experimental setup for the full-scale systems testing planed for the near future. The first study that we performed investigated the effect

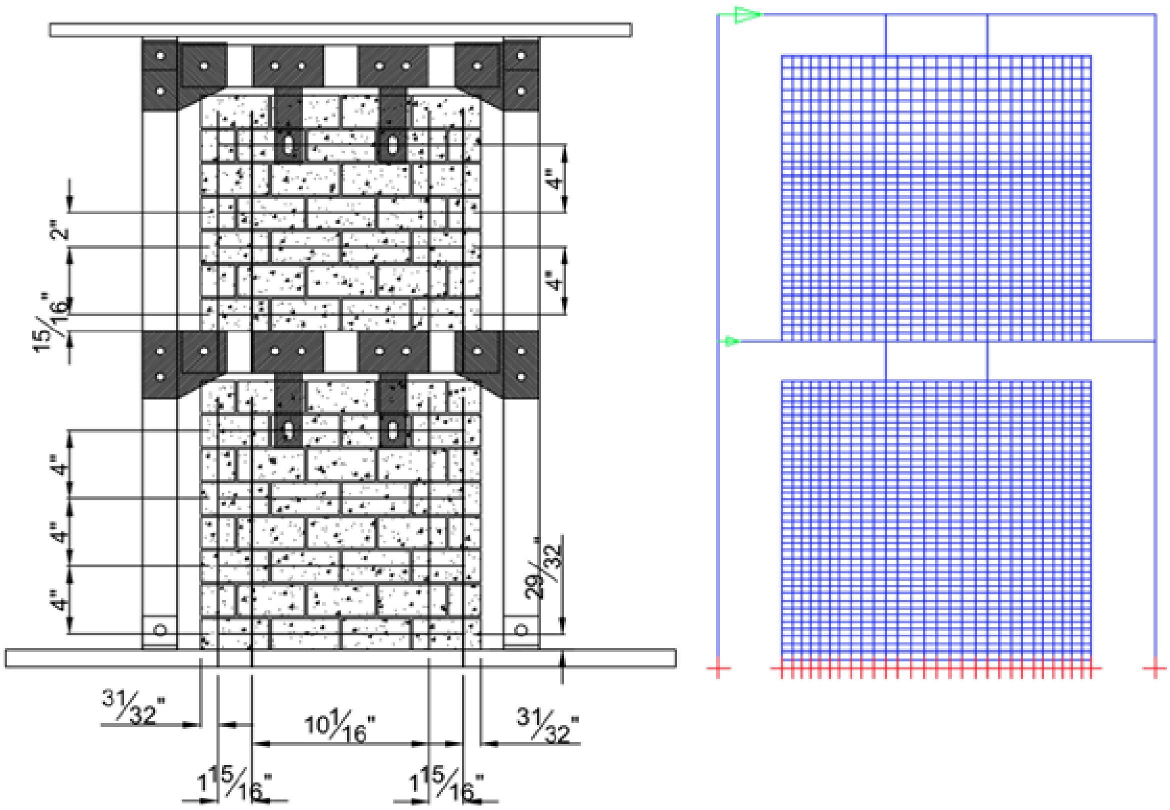

Figure 3: Reduced scale model of a type I hybrid masonry wall: Geometry (left) and finite element discretization (right)

of material and geometrical nonlinearities on the distribution of loads between the steel frame and the masonry panel. The structure presented in Figure 3 is of type I (no loads are transferred through direct contact). The connectors are in this example considered to transfer loads both vertically and horizontally. A controlled displacement loading sequence is considered. Three cases were simulated that consider different material behavior for steel and masonry: (1) linear elastic, (2) small deformation plasticity, and (3) large deformation plasticity.

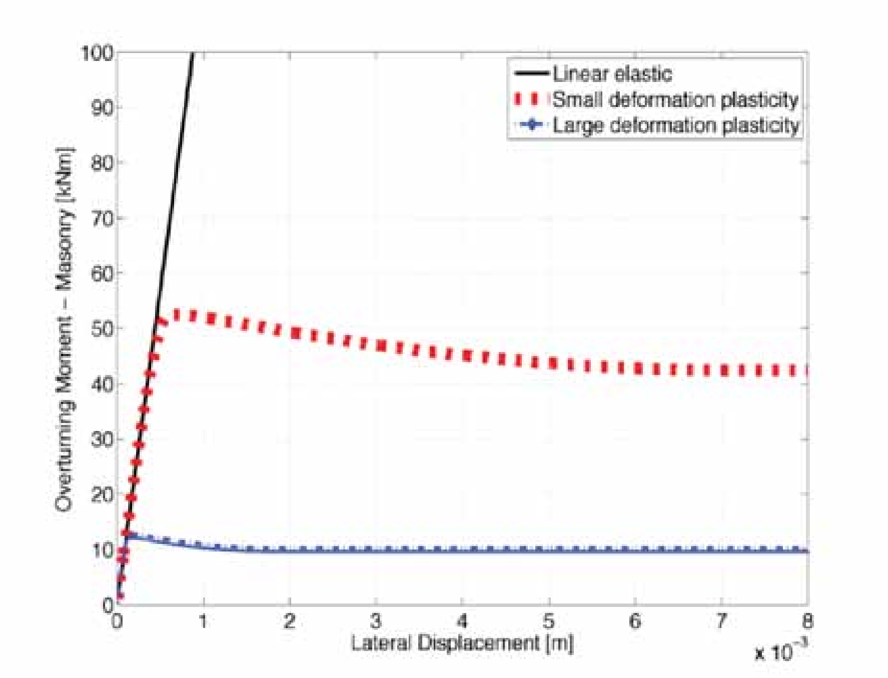

Figure 4: Overturning moment for the masonry panel

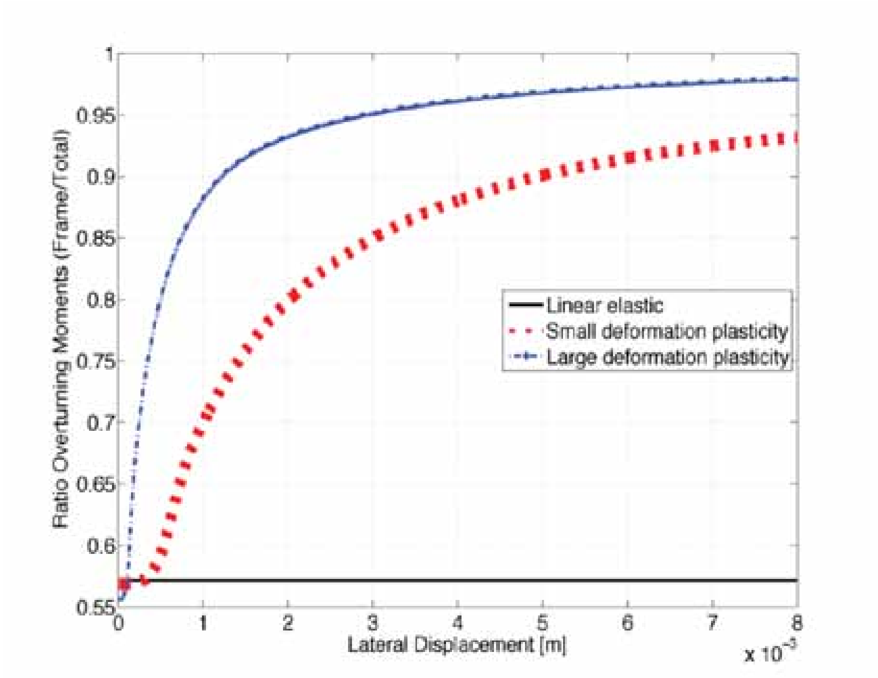

Figure 5: Ratio of overturning moment frame/masonry

Figures 5 and 4 show the total overturning moment of the masonry panel and the ratio of moments as functions of the lateral displacement at the second story beam. It is clearly seen that, while some loads are transmitted to the masonry panel in this system, there is a limited range of displacements for which the contribution of masonry in the overall behavior is significant. In the large deformation range, a significant percentage of the overturning moment has to be resisted by the steel frame. A more realistic assumption was used next, where it is assumed that the connectors transfer only horizontal loads to the masonry and that the masonry can undergo damage.

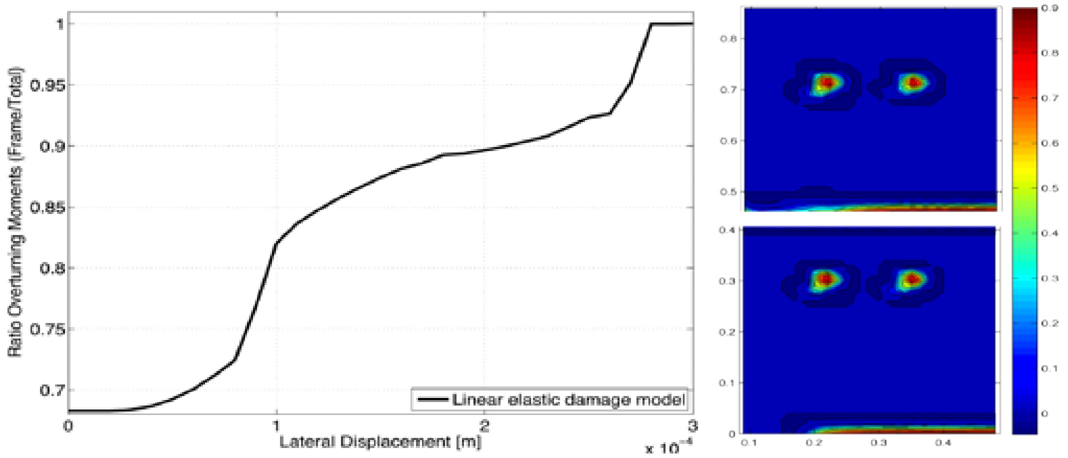

Figure 6: Ratio of overturning moment frame/masonry (left) and distribution of damage (right)

Figure 6 indicates that at relatively small lateral displacements, the ability of the masonry panel to provide resistance to the overturning moment is lost and quite rapidly the whole load needs to be supported by the frame. At this stage, we can only use such simulations for qualitative analyses since most material parameters that we used as input are approximate values for simple masonry. When experimental data will become available from the hybrid systems testing, the material parameters will be calibrated and quantitative conclusions will be drawn.

4 CONCLUSIONS

A computational framework for the analysis of hybrid masonry systems was established. The effect of nonlinearity on the structural response of a small scale hybrid masonry wall was investigated in this preliminary study. We studied both material nonlinearities (by using elasticity and plasticity models) and geometrical nonlinearities by also incorporating the effect of large deformations. The influence of damage in the masonry panel on the overall distribution of the overturning moment was investigated. The crack pattern is significantly influenced by the refinement of the mesh. Such mesh size effects associated with the use of local damage formulations require the investigation/ implementation of a nonlocal damage algorithm to alleviate the stress softening problem. Future simulations of type II and type III systems will also require use of contact formulations.

REFERENCES

[1] Taylor, R. L., *FEAP A Finite Element Analysis Program* (2006).

[2] Lourenco, P.B., *Analysis of Masonry Structures with Interface Elements* (1994).

[3] Zucchini, A., Lourenco, P.B., A micro-mechanical homogenization model for masonry: Application to shear walls, *International Journal of Solids and Structure* (2009), 871–886.

[4] Jirasek, M. , Non-local damage mechanics with application to concrete, *Failure, degradation and instabilities* (2004), pp 683–707

[5] Faleiro, J., Oller, S., Barbat, A.H., Plastic-damage analysis of reinforced concrete frames, *International Journal for Computer-Aided Engineering and Software* (2010), Vol. 27, No. 1, 57-83.

[6] Wriggers, P., *Computational Contact Mechanics*, Springer, 2006.

[7] Yang, B., Laursen, T. A., Meng, X, Two Dimensional Mortar Contact Methods for Large Deformation Frictional Sliding, *International Journal of Numerical Methods in Engineering* (2005), Vol 62, 1183–1225