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NUMERICAL SIMULATION OF UNREINFORCED MASONRY WALLS SUBJECT TO DYNAMIC OUT-OF-PLANE LOADING

L. Reindl, C. Butenweg, and T. Kubalski

Department of Civil Engineering, RWTH Aachen
Chair of Structural Statics and Dynamics
Mies-van-der-Rohe-Str. 1, 52074 Aachen, Germany
e-mail: masonry@lbb.rwth-aachen.de

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Abstract. *Out-of-plane failure is a very common failure mode of unreinforced masonry walls. After earthquake events in Germany (Albstadt 1978, Roermond 1992) damage due to out-of-plane loading was one of the main problems observed. Especially the failure of infill walls and gables was dominated by out-of-plane instability. Since these walls have a relatively low vertical load level, the out-of-plane stability is mainly governed by the slenderness, the thickness of the wall and the bending stiffness parallel and orthogonal to the bed joints, which depend on the bending stiffness of the masonry units, the mortar used, the interconnection between the units as well as the unit geometry and the connection to adjacent constructional elements. In general, current codes only provide limit values for the wall slenderness that are based on experimental results and engineering experience, and simple design concepts based on traditional, force-based design procedures. However, recent research has shown, that masonry walls subject to dynamic out-of-plane loading have deformation reserves exceeding the deformation capacity calculated using traditional, force-based design.*

In order to further investigate the deformation capability of dynamically out-of-plane loaded unreinforced masonry walls, a detailed 3D numerical model on the meso-scale has been developed using the FE toolkit ABAQUS. The model is able to reproduce typical failure modes of single wall panels with different boundary conditions and varying vertical load levels. It is used to investigate and classify the different influence parameters. The results shall be used to define new and refine existing constructional rules. Also, a simplified concept for a dynamic stability check shall be derived.

The paper gives an overview of the current state of the research. It addresses typical problems that arise in the development of numerical models for out-of-plane loaded masonry walls for seismic loading. The numerical model and the results from static and dynamic calculations are presented and - where possible - compared to experimental data.

1 INTRODUCTION

The behavior of out-of-plane loaded, unreinforced masonry wall panels subject to dynamic excitation such as seismic loading is very complex and – up to today – not yet entirely investigated and understood. In 1992 Paulay and Priestly [1] already described this subject accurately with the words:

„one of the most complex and ill-understood areas of seismic analysis“

The out-of-plane failure often happens at earthquake intensities with amplitudes much lower than the ones that are necessary to produce significant in-plane damage. Knowledge about the stability and vulnerability in the out-of-plane direction is therefore absolutely necessary.

Documented experimental investigations in this area of research were carried out as soon as 1802 (Rondolet [2]). Recent observations after different earthquake events have shown that in many cases wall failures due to out-of-plane loading resulted in greater damage than in-plane failure mechanisms.

Especially walls with low vertical load levels are vulnerable to out-of-plane loading. Out-of-plane damage can hence often be found in the upper stories and for poorly braced gable walls as shown in Figure 1.

Figure 1: Out-of-plane failure after the earthquake in Albstadt, Germany, 1978 (Photos: Peter Doll)

Traditionally, masonry is regarded as a brittle material with little ductility and is therefore considered to be especially vulnerable to its peak load bearing capacity resulting from the peak ground acceleration. Nowadays it is generally known that the calculated force-based loading capacity plays a subordinate role when it comes to the investigation of the dynamic stability of out-of-plane loaded wall panels. Recent results from current research projects have shown that after the calculated, static, force-based load bearing capacity is achieved, there are still very considerable reserves resulting from the deformation capabilities of the wall ([3], [4], [5]). Dynamically loaded walls are able to withstand accelerations that cannot be achieved with static loading. This discrepancy clearly reveals that the mechanisms leading to out-of-plane wall collapse are not yet entirely investigated and understood.

2 NON-LINEAR WALL BEHAVIOR

The highly non-linear behavior of unreinforced, masonry wall panels subject to out-of-plane loading is mainly governed by stability mechanisms while the calculated static load-

bearing capacity is of fewer relevancies. This complicates the description of the relevant failure modes and the associated kinematic model. Figure 2 shows a typical load-deflection curve of a one-way spanning out-of-plane loaded masonry wall panel.

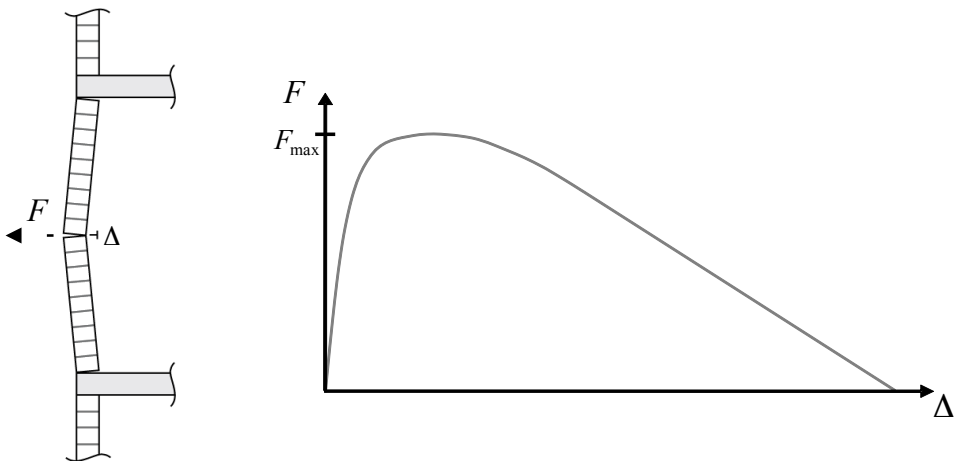

Figure 2: Qualitative load-deflection-curve of an out-of-plane loaded masonry wall panel

The wall panel exhibits a linear elastic behavior up to the point when the static force-based load bearing capacity is reached. The wall typically develops cracks around its mid-height as well as at its top and its base. It is split into single segments that rock around these pivot points which allows the wall to deform significantly until its point of instability.

It seems reasonable to take these apparently present non-linear reserves in the post-peak domain into account. Clearly, second order effects become more important due to the large deformations. A part from the physical nonlinearity, the geometric nonlinearity is increasingly important and has to be taken into account.

For now the ongoing challenge is the development of simple and feasible models that are able to reproduce this complex behavior and hence the derivation of a modern verification procedure.

3 STATE OF THE ART

The research focus of the past years in Germany was certainly put on the optimization and the determination of the behavior of masonry shear walls and accordingly on the development of a feasible concept for the approximation of in-plane capacity curves opening the way for a deformation based design concept. Consequently, usable verification concepts with a realistic depiction of the actual dynamic behavior of out-of-plane loaded masonry walls are missing until today.

Current design codes usually determine the bending capacity based only on the calculated, static load carrying capacity. E.g. in Eurocode 6 [6] two bending capacities parallel and orthogonal to the bed joints are calculated using the tensile capacity in each direction and the static moment. Beneficial influences of an existing vertical compression may be taken into account by increasing the tensile strength for bending parallel to the bed joints. It should be mentioned that Eurocode 6 [6] clearly excludes the use of this method for seismic design.

It is interesting to notice that Eurocode 8 [7] ignores the subject of the out-of-plane stability of load-bearing masonry walls. There are neither instructions for the determination of the seismic input onto the wall nor a concept for the approximation of the dynamic stability leaving it up to the structural engineer to find a practicable way for the seismic verification. Quite often the seismic input is hence determined by using the definition for non-structural compo-

nents that may be an acceptable approximation in many cases. However, for very important non-structural components and for components that represent a potential risk, a seismic verification using a realistic model and realistic response spectra that take into account the filtering effects of the structure is required. Feasible guidelines for this kind of verification are not included in Eurocode 8 [7].

Some limit values for structural shear walls are included in Eurocode 8 0. In detail these are limits for the thickness, the slenderness and the aspect ratio that are defined in dependence on the seismic zone. Moreover, the national annex of Germany requires all gable walls to be properly braced by transversal walls or pilaster strips if they are not positively anchored to the roof structure. This directive is probably based on the observations after the earthquake in Albstadt in 1978 where these walls were identified to be especially vulnerable to out-of-plane loading.

This clearly shows that further research into this field of seismic engineering is inevitable. The outcome of the research should be a new verification concept that allows for a quick and easy verification based on limit values and also provides guidance for a more advanced investigation of the dynamic stability using displacement-based approaches.

4 NUMERICAL SIMULATIONS

In order to contribute to the development of a feasible deformation-based design concept for unreinforced masonry walls subject to out-of-plane seismic loading, numerical models have been developed. The most recent model was developed using the FEM code ABAQUS [8] which provides powerful material models out-of-the-box. For the current research a 3D model on the meso-scale was chosen. It uses the concrete damage plasticity constitutive law for the bricks and a cohesive contact law to represent the bond between the single bricks [9].

4.1 Assembly

In order to be able to use a 3D model and to fasten up the numerical computations, a few assumptions and simplifications had to be made. All the constraints such as walls or ceilings were reduced to simple concrete beams, which represent the boundary conditions given by adjacent components, with a simplified linear elastic material law. The concrete beam at the top is able to perform vertical movements ensuring a constant vertical load level within the wall. The dimensions of the thin bed mortar joints are neglected.

4.2 Material model

The developed model on the meso-scale consists of individual bricks, which are able to interact between each other and the surrounding concrete beams. Two different materials were used for the bricks in the simulation: calcium silicate and autoclaved aerated concrete. The damaged concrete plasticity material model (CDP), which is included in ABAQUS by default, was used for the material idealization. The masonry bricks are assumed to fail either due to tensile cracking or compressive crushing. The CDP also considers the degradation of the elastic stiffness due to damage and accounts for stiffness recovery effects. It is hence suitable for simulating cyclic loading conditions such as earthquakes.

4.3 Interactions

Each surface needs to be able to interact in every direction with its neighboring surfaces. While pressure and friction can be directly transferred between the surfaces, traction as well as shear forces are delegated through the adhesive effect of the mortar. Since modern high-performance masonry walls in Germany are usually built using precision blocks in combina-

tion with thin-bed mortar joints, there is no need to explicitly model the mortar joints. Consequently, the mechanical properties of the joints were defined as interactions properties between the surfaces. Several interaction forms were used to model contact in the different directions.

Pressure can be transferred directly by using the 'hard contact' formulation which propagates forces only when the surfaces are in contact and prevents the penetration of surfaces as 'pressure overclosure' is avoided. Shear forces and traction forces are transferred using cohesive interaction behavior. The underlying concept is a traction separation law, which allows the definition of stiffness for each direction. With this, a small elastic deformation of the mortar joint was defined in combination with the specification of maximum strain, damage evolution and failure of the joint.

During the investigations, friction wasn't taken into account since friction failure is not considered relevant for the investigated walls.

4.4 Gravity

To obtain realistic results, gravity was applied to all models. When using gravity loads in explicit analyses, inertia effects must be taken into account. Otherwise it may occur that the wall bounces slightly, which can be shown by recording the vertical reaction forces at the bottom of the wall. The magnitude of the oscillations of the reaction forces varies depending on the walls' boundary conditions as well as the used material definitions and the used contact interactions. Especially the use of the 'hard contact' formulation amplifies the effect since overclosure of the surfaces is prevented. In order to eliminate such disturbing phenomena induced by the wall's inertia, the gravity load was applied in a separate analysis step where it was increased in small increments over a period of time.

4.5 Problems during numerical investigations

Within the scope of the of the FE-analysis 8-node-hexaeder elements type C3D8R were used for meshing the walls. During the calculations unrealistically large deformations occurred as shown in Figure 3.

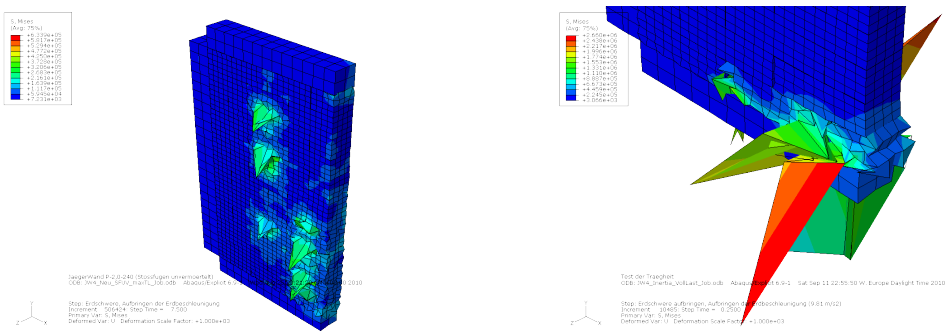

Figure 3: Hourglassing effects

These deformations were classified as hourglassing effects, which often happen when using plastic material behavior in dynamic response analysis combined with reduced integration in explicit analysis steps. Several measures for controlling the hourglassing effect were tested. As a result, a mesh refinement in combination with the use of 'enhanced hourglass control' was determined to be the most effective way to reduce such unrealistic deformations. Problems with the enhanced stiffness causing wrong results weren't encountered.

5 SPECIMEN

5.1 Static specimen

In order to test the quality of the model, static tests were carried out and compared to real life specimen found in literature. In these tests a horizontal distributed load was applied to the wall's back while the wall's horizontal displacement was recorded. An example of the wall's geometry is shown in Figure 4 whereas Table 1 gives an overview of the tested walls' dimensions. All four walls are made of AAC precision blocks in combination with a thin-bed mortar.

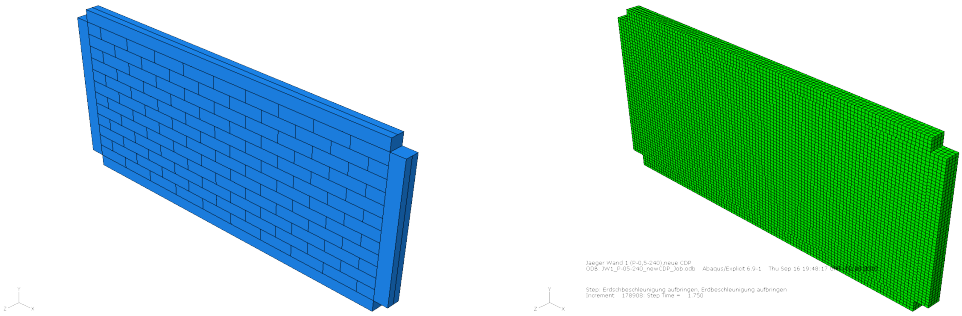

Figure 4: Example of unmeshed and meshed tested wall

	H/L	Height [m]	Thickness [mm]	Supports
JW1	0,5	3,0	240	4-sided
JW2	0,5	3,0	175	4-sided
JW3	1,0	3,0	240	3-sided
JW4	2,0	3,0	240	3-sided

Table 1: Overview of tested static walls

5.2 Dynamic / cyclic Specimen

	H/L	Height [m]	Thickness [mm]	Material	Supports	Acceleration
W1a	2,0	3,0	240	AAC	1-sided	ZVL 8
W1b	2,0	3,0	240	AAC	1-sided	El Centro
W1c	2,0	3,0	240	CS	1-sided	El Centro
W2a	1,5	1,5	110	CS	2-sided	El Centro
W2b	2,0	3,0	240	CS	2-sided	ZVL 8
W2c	2,0	3,0	240	CS	2-sided	ZVL 8
W3a	1,0	3,0	240	AAC	3-sided	ZVL 15
W3b	1,0	3,0	240	CS	3-sided	El Centro
W4a	1,0	3,0	240	AAC	4-sided	ZVL 8
W4b	1,0	3,0	240	CS	4-sided	ZVL 8

Table 2: Overview of tested dynamic walls

For research on the behavior of masonry walls subjected to dynamic loading, the geometry of the static walls was used while the distributed load was changed to a horizontal gravity load. The gravity was only distributed to the bricks in order to give conclusion to an equiva-

lent distributed load. Two artificial accelerations (ZVL 8, ZVL 15: $a_{max} \approx 0{,}8 m/s^2$) as well as the El Centro accelerogram record ($a_{max} \approx 3{,}0 m/s^2$) were used to simulate different input motions. The geometries of the tested walls were derived from the static specimen. Different types of walls such as parapet walls or vertically spanned walls as well as 3-sided and 4-sided constraint walls were tested. A full list of the specimen tested under dynamic gravity load is given in Table 2. The dimension of the bricks as well as the properties of the mortar were chosen according to the experimental test results provided by [10] and [11].

6 RESULTS

6.1 Static specimen

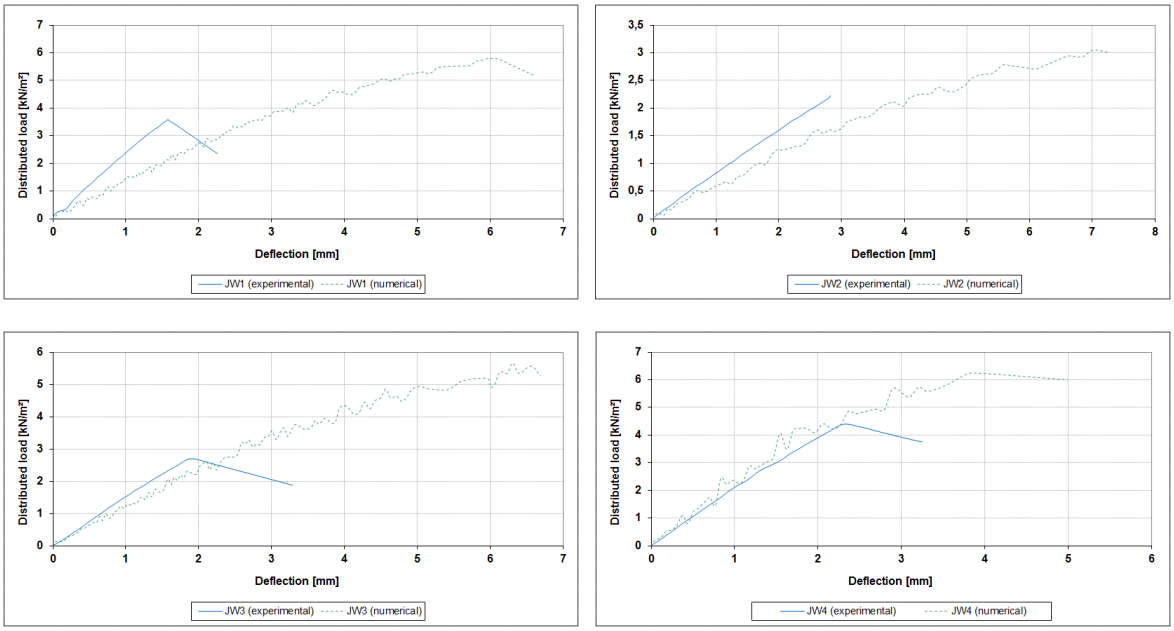

Figure 5: Results of static specimen

Figure 5 gives an overview of the numerical simulations compared to experimental tests found in [11]. In general the results show a good correlation considering the initial stiffness. However, big differences are present for the load bearing capacity and the deformation capability. This may be caused by several reasons. Firstly, the numerical model did not accurately reproduce the boundary conditions of the experimental tests, where the wall was built into a steel frame instead of being held by concrete beams. Moreover, there was no vertical load applied in the experiments while there was a concrete beam at the top charging the wall in the numerical simulations. It is also questionable whether or not the CDP constitutive law correctly described the material behavior since detailed information on the used bricks are not available. There is also a difference in the way the deflection was recorded. While the maximum displacement was used in the numerical model, the experimental testing setup had fixed reference points that did not necessarily provide the exact same deflections.

The failure mechanisms of the numerical walls give the expected results as shown for example in Figure 6. Failure at the 4-sided walls started about mid-height in the middle of the wall whereas the failure of the 3-sided walls started about mid-height of the free side. Crack propagation proceeded to the edges of the constraint sides.

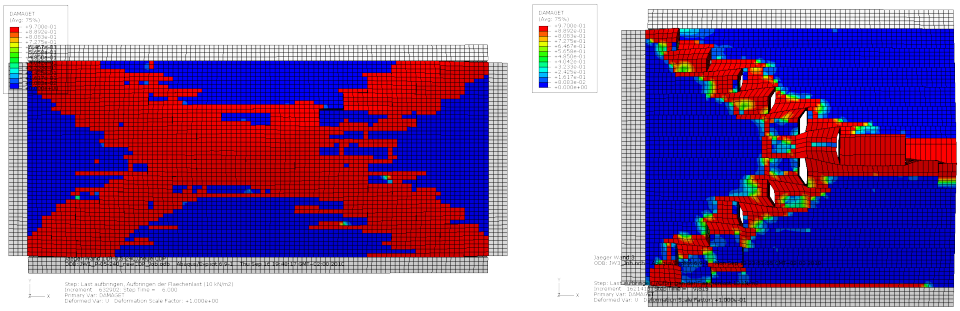

Figure 6: Failure of wall JW1 and wall JW3

6.2 Dynamic specimen

The different wall types showed specific buckling behavior resembling the typical behavior of such walls subjected to dynamic loadings. As an example the results of the parapet wall W1a and the results of the vertically spanned wall W2b are presented. These two walls were subjected to the same excitation. The parapet wall started quickly to buckle. The top of the wall deflected the most as shown in figure 5 while the joints at the bottom of the wall started to open slightly. Complete failure of the wall wasn't achieved, but small plastic strains occurred at about height of the bottom joints. The maximum deflection is about 4 mm as can be seen in Figure 7.

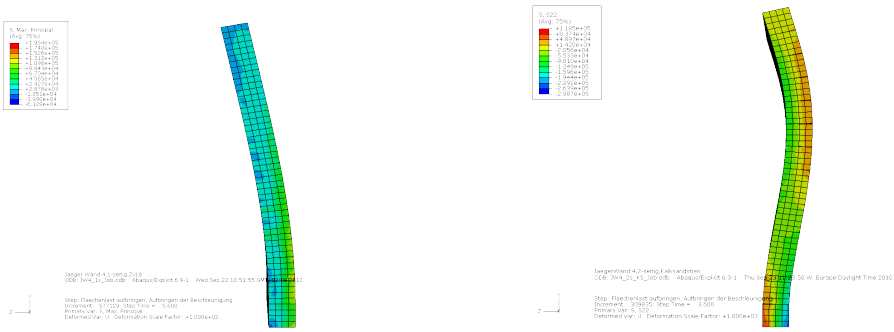

Figure 7: Comparison of (scaled) deformation of wall W1a (left) and wall W2b (right)

The vertically spanned wall has been supplied with an additional vertical load due to the top concrete beam. As a consequence of the added supply, the achieved deformations were smaller, just up to about 0,23 mm as shown in Figure 8. The biggest deflection occurred about mid-height of the wall. No plastic deformations were produced during this conducted test. The upper half of the wall bends due to the excitation.

Again, the results of the presented model show a good approximation of the overall behavior while the maximum deflections differ significantly when compared to experimental results found in [5]. In an attempt to verify the model, on of the walls tested in [5] was modeled. It showed maximum deformations significantly smaller than in the experiment. However, it has to be noted, that the construction type used in the experiments (general purpose mortar) is different from the one that the numerical model is intended for. It is therefore not surprising that the numerical model of the current German construction type with thin bed mortar and modern high performance masonry units has a greater stiffness and hence smaller deformations.

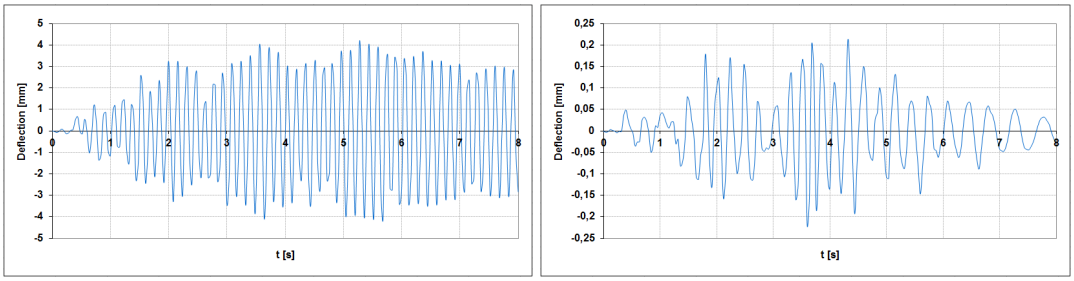

Figure 8: Comparison of deflection of wall W1a (left) and wall W2b (right)

In closing it can be said that the presented model is capable of modeling the failure modes and initial stiffness of an out-of-plane loaded wall made of modern high performance masonry sufficiently well. However, there are still great deficiencies when it comes to predicting the load bearing capacity and the deformation capability.

In its current state, the model cannot yet be used to do parametric tests in order to derive and verify e.g. load-deflection curves. It is already being used to study the typical cracking patterns as a function of the wall and unit geometries and hence the kinematics of cracked wall panels.

7 DISCUSSION

The presented numerical model is able to realistically simulate the typical failure modes of out-of-plane loaded unreinforced masonry walls. It provides a strong basis for comprehensive parametric studies on the influences of the wall geometry, brick dimensions and boundary conditions. Further enhancements are necessary to improve the quality of the results and hence the scope of the model.

The finite element code ABAQUS provides satisfactory results while providing sophisticated material models for brittle material and cohesive contact out of the box.

It is evident, that there is still a high demand for research in the field of seismic verification concepts for out-of-plane loaded unreinforced masonry walls. Basic simplified approaches are already available in European codes but they are very limited and cannot be applied to arbitrary wall configurations. A clear definition for walls not satisfying the provided limit values are still missing.

The development of practicable and efficient verification concepts should result in a mulit-stage verification concept.

A more detailed specification of limit values for thickness, slenderness, etc. in dependence on the seismic hazard should be provided along with detailed information on the design of the contact zone between walls and slabs. These values need to be defined for structural as well as non-structural components.

For walls exceeding the limits simplified approaches for a traditional force based design as well as for a typical dynamic stability check is necessary. This requires guidelines for definition of the seismic input as well as a concept for a simplified estimation of the kinematic equivalent system.

The presented numerical model along with future improvements will contribute to this development.

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