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Division V

DESIGN OF MASONRY WALLS FOR THE OUT-OF-PLANE BEHAVIOR UNDER EARTHQUAKE LOAD

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ABSTRACT

Unreinforced masonry must withstand out-of-plane loads that occur in earthquake scenarios. While for ordinary building structures, static load-bearing capacity verification is sufficient in many cases, to drive safe nuclear energy there are high requirements on the earthquake-resistant design. The current design methods are not sufficiently developed for the out-of-plane behavior due to the high nonlinearity behavior. They contain considerable inaccuracies, which leads to an uncertain design. Especially in modern structural mechanics in reactor technology, a reliable calculation must be guaranteed. The rocking of masonry walls under earthquake loads represents a non-linear time-dependent process. It is therefore ideally represented by a non-linear, dynamic time-history simulation. However, discrete modeling of complete wall systems including bricks, mortar joints, boundary conditions, etc. is usually not feasible in practice. For the sake of simplicity and precision, an idealized model of an equivalent single-degree-of-freedom (SDOF) system can be used rather than the complex models. Therefore several authors developed methods to determine the force-displacement relationship of unreinforced masonry walls. Experimental investigations have shown that significant influencing factors are still neglected. Hence at the institute, a new model has been developed. In the present work, shaking-table tests have been conducted and compared to this analytical model.

INTRODUCTION

Masonry walls have very good characteristics for vertical load-bearing, but out-of-plane loads that occur in seismic events can only be absorbed to a limited amount. Out-of-plane loads are caused by wind, explosion, or earthquake. In particular, attention must be paid to seismic events. During the vibration of the building, Inertial forces perpendicular to the wall's plane will be generated that must be resisted by the wall.

In nuclear power plants, non-load-bearing walls are often built as unreinforced masonry. Due to high requirements, the structure, as well as substructures, have to be stable in the case of an earthquake. The collapse of partition walls can be a risk for safety-relevant structures and components, due to falling debris or sequential effects. Furthermore, slender non-load-bearing walls are vulnerable due to the lack of overburdened vertical loads. To avoid the collapse of the walls in advance, the out-of-plane capacity must be determined very precisely.

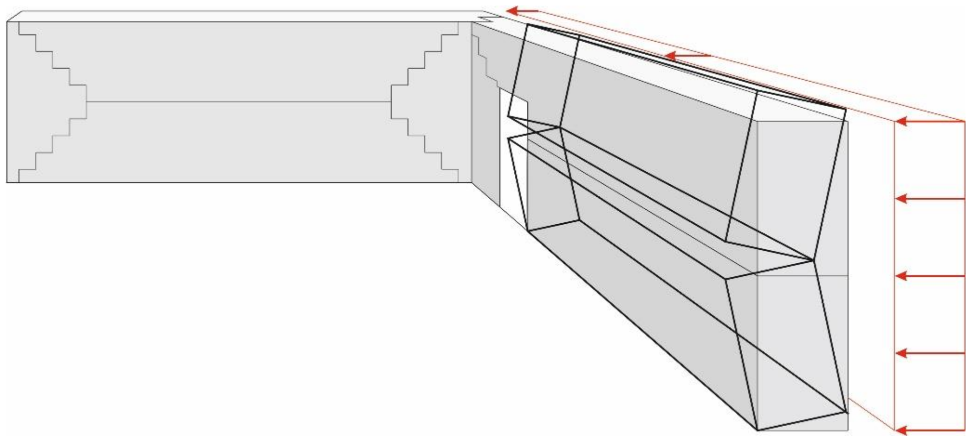

Figure 1. Out-of-plane deformation mode

Due to the earthquake and the vibration of the building, the wall is accelerating in the direction of the vibration. This can be idealized as a horizontal load. As a result, the out-of-plane behavior describes the failure out of the plane of the wall (Figure 1.). Due to the low and highly scattering tensile strength of masonry, this is not applied in most models. The load-bearing capacity is usually calculated from the dead weight and boundary conditions, such as overburden.

Despite the practical relevance in construction, which is supported by observed out-of-plane masonry damage during earthquake events, no uniform method has been established in literature and standardization. Therefore, different analytical, empirical and numerical approaches can be found. In practice, models from the literature (Paulay and Priestley (1992); Doherty et al (2002)) are used in addition to the methods from KTA 2201.3. The analytical model described in KTA 2201.3 is based on the arching effect of the masonry between stiff supports. In Paulay and Priestley (1992) and Doherty et al (2002) the wall is divided into two rigid blocks.

ANALYTICAL MODEL

Analytical investigations at the Technical University of Kaiserslautern have shown that the prediction of the out-of-plane load-bearing capacity by existing methods shows a strong scatter. Many parameters such as the crack height, position of the pivot point, and vertical stiffness are not or only considered in a simplified way.

Therefore, an analytical model is developed that is as close to reality as possible and includes all relevant influencing factors (Lönhoff 2021). For this purpose, the models according to Paulay and Priestley (1992) and Doherty et al (2002) are used as a basis and the arching effect is taken into account via a vertical stiffness of the support at the wall head. All important basic principles and calculation steps of the developed model can be found in the dissertation by Lönhoff (2021). The calculation of the out-of-plane bearing capacity with the newly developed model can be divided into two parts. First, the nonlinear force-displacement relationship for unreinforced masonry walls is determined. Secondly, the dynamic calculation is performed. The vibration mechanism of the masonry wall under earthquake loading represents a strongly nonlinear, time-dependent process. For this step, the wall is simplified to an equivalent SDOF model so a nonlinear time history calculation can be performed.

The model is based on two rigid blocks. The crack height, calculated using classical beam theory, defines the size of the blocks. The contact between the blocks is represented due to elastic springs. Furthermore, various external boundary conditions are taken into account. These include a possible centric or eccentric overburden and vertical stiffness at the wall head. The vertical stiffness represents, for example, a reinforced concrete ceiling.

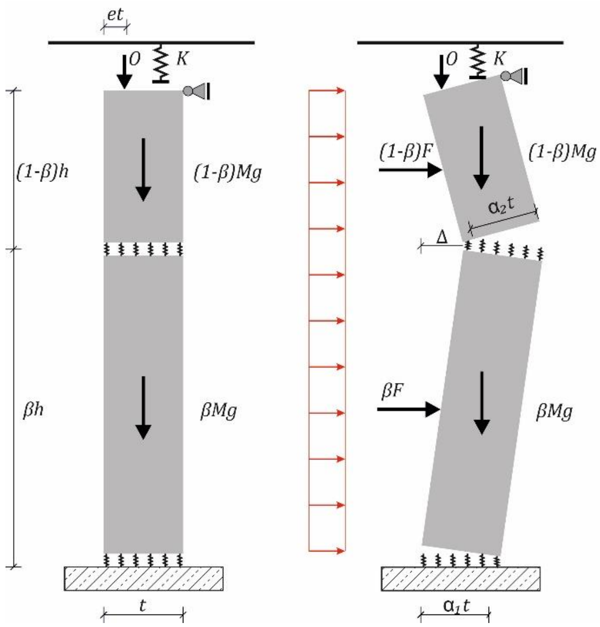

Figure 2. Analytical model according to Lönhoff (2021)

The principle of virtual work is used to determine the force-deformation relationship. The horizontal load leads to a deflection of the wall. This is countered by the self-weight, the overburden load, and the external boundary conditions. In a static consideration, all loads are in equilibrium for each deformation state. By incremental increasing the wall displacement and determining the virtual work, the nonlinear force-deformation relationship is obtained.

There are essentially two unknown variables in the model. On the one hand, the stiffness of the springs between the rigid blocks and the bottom, on the other hand, the damping. The stiffness of the springs hereinafter referred to as contact stiffness, describes, among other things, the degree of damage. If the wall behaves like rigid blocks, then the contact stiffness is very high. With increasing damage and for soft mortar and stones the value is reduced accordingly. Since the contact stiffness and the damping has no simple derivable basis, they must be determined experimentally for different walls.

EXPERIMENTAL SETUP

Extensive shaking table tests were carried out at the Technical University of Kaiserslautern (Helm et al 2022). The shaking table tests aim to verify the analytical model and quantify the contact stiffness and damping. For this purpose, walls were tested with synthetic and real-time history data. The acceleration and the resulting deformation of the wall were measured with acceleration and laser sensors. For the evaluation, the measured deformation of the wall is compared with the analytical calculation. A good match is considered a verification of the model. This also includes checking the contact stiffness and damping.

In Figure 3 the experimental setup is shown, which is mounted on the shaking table. The steel sections above and below the wall are filled with concrete. The vertical stiffness can be increased by installing springs in the upper steel section. The walls are further built in the setup aiming for realistic boundary conditions.

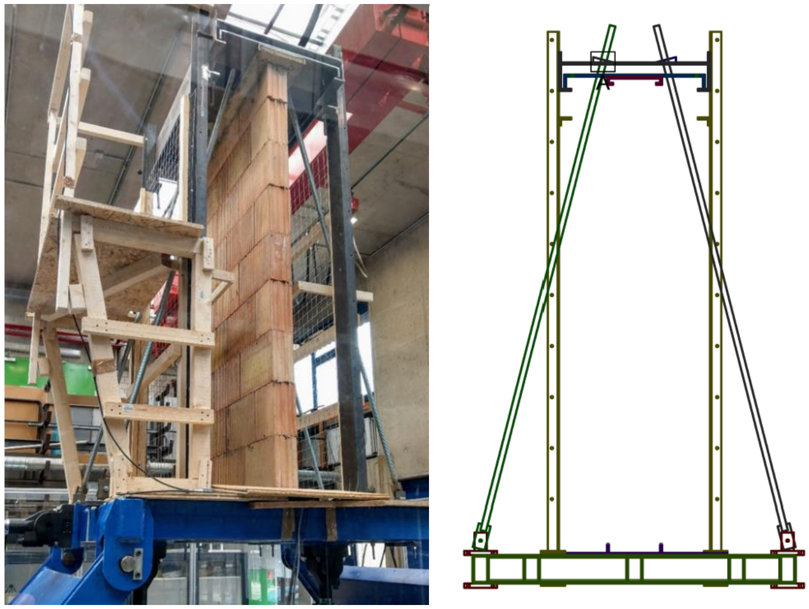

Figure 3. Experimental setup

In this series of tests, eight walls, designed with different wall thickness and boundary conditions, were examined. The investigated walls were made of vertically perforated bricks set in thin-bed mortar, with the first layer of bricks laid in standard masonry mortar.

Table 1. Overview of the tested walls

wall-no.	**dimensions [m]**	**slenderness h/t**	**weight [kg]**	**vertical stiffness [kN/m]**	**note**
1				1332	
2	1.115 x 0.115 x 2.22	19.3	285	1332	
3				2220	
4	1.21 x 0.24 x 2.26	9.4	550	0	
5	1.11 x 0.175 x 2.25	12.9	330	0	
6	1.115 x 0.115 x 2.26	19.7	300	0	
7	1.115 x 0.135 x 2.26	16.7	355	0	with plaster
8	1.115 x 0.138 x 2.26	16.4	405	0	with plaster

Six walls with a wall thickness without plaster of 11.5 cm were tested, as well as one wall each with 17.5 cm and 24 cm. The slenderness of the walls h/t varied between 9.4 and 19.7 at a wall height of 2.22 to 2.26 m. The first three walls were connected to the head by full-surface mortaring. The remaining walls were only held horizontally at the wall head so that it could rotate as freely as possible. Two walls were additionally plastered. The information is summarized in Table 1.

EXPERIMENTAL RESULTS

The first wall was connected at the wall head by full-surface mortaring. Since the wall was built in the test setup, a crack formed between mortar and concrete at the wall head at the beginning of the tests. The reason for this is the shrinkage behavior of the mortar. As a consequence, the wall behaved as a cantilever under small loads and deformations (Figure 4. a). At larger displacements, the wall head was supported due to friction (Figure 4. b). In this phase, the wall behaved according to the two-blocks model. After the deformations had decreased, the wall returned to its initial position as a cantilever (Figure 4. d).

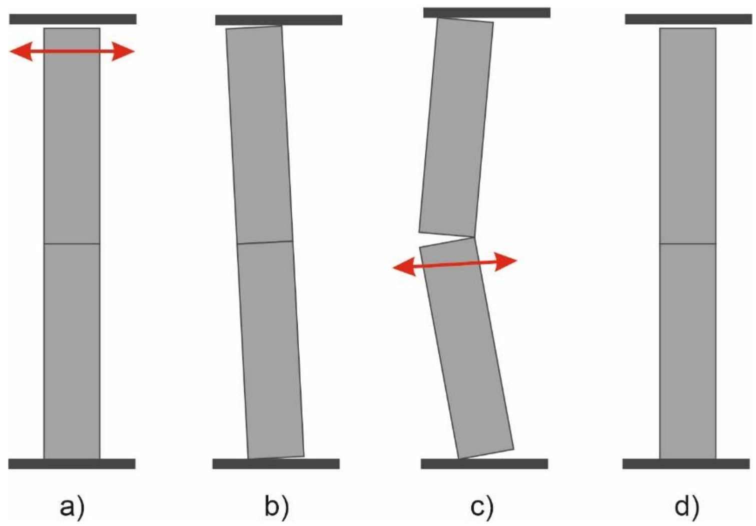

Figure 4. Alternating behavior between the cantilever and the two-blocks model

In Figures 5 and 6 two tests are shown as examples. In both tests, the behaviors were similar to the two-block model. Test No. 7 shows a free vibration of the wall with a deformation of 4 cm at the beginning. With this experiment, the period and damping can be analyzed. The period depends on the stiffness, or in this case on the nonlinear force-deformation relationship. Due to the nonlinearity, the time for a whole oscillation varies depending on the deformation. The model provides a good approximation here. The damping influences the decrease of the amplitudes. Effective damping of 4,8 % was found here.

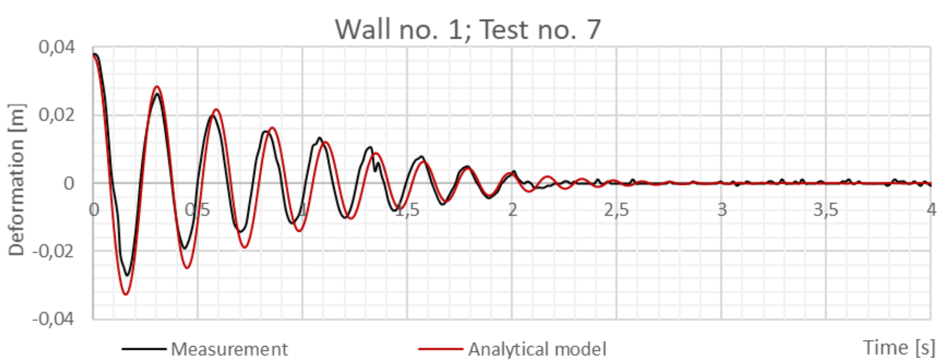

Figure 5. Free vibration test, measurement & calculation

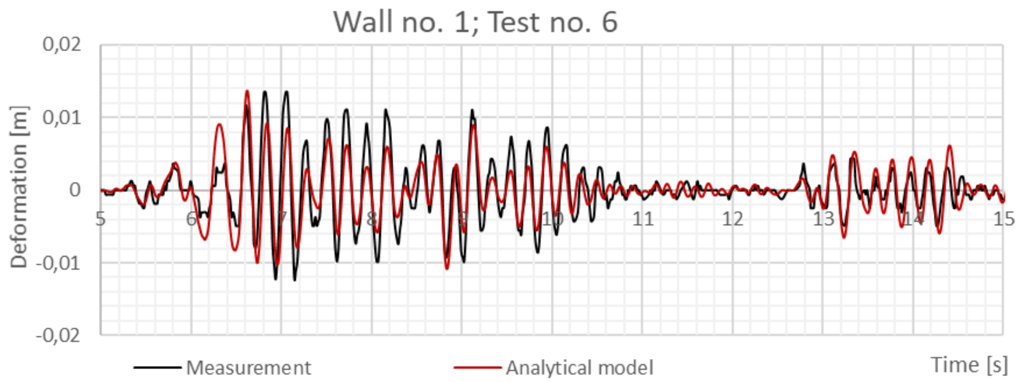

Figure 6. Time history vibration test (El Centro), measurement & calculation

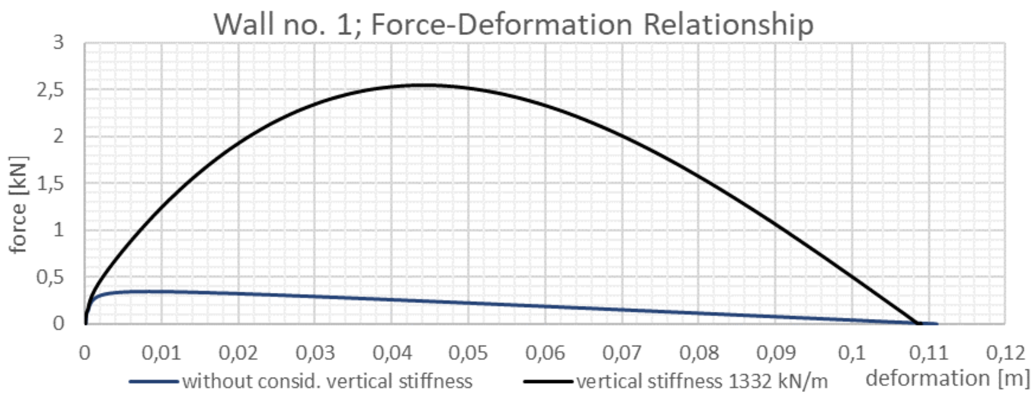

Figure 7. Force-deformation relationship due to the analytical model

The wall was tested with the well-known earthquake El Centro. The maximum deformation amounts to 1.4 cm at the crack height (Figure 6). The deformation was calculated with the nonlinear force-deformation relationship shown in Figure 7. The results show a good approximation between the measurement and the analytical model. Figure 7 represents the calculated force-deformation relationship without considering the vertical stiffness. It becomes clear how much the load-bearing capacity is influenced by this parameter.

Good results could be achieved with all walls, which are represented in Table 2. On average, the crack height is 0.57. This is consistent with the crack height of 0.593 according to the authors Rao et al (2017). The contact stiffness, a parameter explicitly defined for the analytical model, amounts between 0.13 1/m and 0.4 1/m. The effective damping is between 4.8 % and 9.1 %. Here, there seems to be a tendency towards higher damping with smaller slenderness. Further investigations are necessary to be able to make well-founded statements on this. Table 2. Additionally shows the largest tested earthquake without failure. To quantify the time history, the spectral accelerations in the period according to the model and the largest displacements measured in this test are given.

The main result of the experiment is the conclusion, the deformation due to the seismic action can be calculated well using the analytical model. This includes the determination of the nonlinear force-deformation relationship, the damping, and the idealization of the wall to an equivalent SDOF system. It also appears that, in addition to the influence of the wall geometry, the boundary conditions at the wall head are of significant importance. For the correct load-bearing capacity, these must be known.

Table 2. Summary of the test results

no.	**crack height**	**contact stiffness**	**effective damping**	**highest spectral acceleration without failure**		
				Sa	**at period**	**measured deformation**
	[-]	**[1/m]**	**[%]**	**[m/s²]**	**[s]**	**[m]**
1	0.67	0.3	4.8	15.6	0.40	0.029
2	0.56	0.33	5	13.0	0.40	0.013
3	0.56	0.14	6	61.9	0.22	0.079
4	0.56	0.4	8.8	19.2	0.44	0.083
5	0.56	0.17	9.1	18.0	0.52	0.060
6	0.45	0.19	6.1	6.1	0.52	0.081
7	0.67	0.19	8	8.2	0.62	0.091
8	0.56	0.13	5.6	5.83	0.60	0.076
ø	**0.57**	**0.23**	**6.7**			

CONCLUSION AND OUTLOOK

At the Technical University of Kaiserslautern, a model is being developed to evaluate the out-of-plane load-bearing capacity of unreinforced masonry walls, taking into account all significant influencing factors. The model contains two unknown parameters. One is the contact stiffness between the rigid blocks and the other is the damping. Experiments were carried out to verify the model and quantify these parameters. Masonry walls were tested on a shaking table with synthetic and real-time history data. The experimental results and evaluation show, that the deformation due to the seismic action can be calculated well using the analytical model. The contact stiffness, a parameter explicitly defined for the analytical model, amounts between 0.13 1/m and 0.4 1/m. The effective damping is between 4.8 % and 9.1 %.

For final verification of the analytical model, further geometries, types of masonry, and boundary conditions have to be tested and investigated.

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