13th International Brick and
Block Masonry Conference
Amsterdam, July 4-7, 2004

NUMERICAL MODELLING OF MASONRY UNDER SHEAR LOADING

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Abstract

Masonry is a unique building material due to its non-homogeneity. This leads to various shear failure modes directly related to the ratio of horizontal to vertical loads. This modes are also related to the used products. For new products the shear behaviour can change. On possibility for investigating the shear behaviour are numerical simulations. Therefore, it was decided to use micro-models to represent the shear-loaded masonry; i.e. stones, mortar and the contact surfaces were each modelled separately and the appropriate material properties were applied. One approach to modelling is the use of a wall section under constant stress. This is used when one wants to understand and calculate the failure of a single masonry unit. During the computation process, this approach relies on the inherent symmetry of the building material. Thus the time needed for calculating the results is minimised. Further this allows one to create a very sophisticated mesh. Thus one can investigate a great variety of load and material combinations during a short time frame. In turn, one can draw an exact shear failure curve while considering the plastic material behaviour. A second approach is to model an entire wall including the effects of bending. In this approach all the support conditions are considered with regard to the load capacity. This is of particular interest for modern masonry with large-sized masonry units.

Key Words

Shear loading, Shear-wall, Numerical simulation

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1 Introduction

In the new codes about earthquake and wind forces the applicable design loads have been increased for some regions in Germany. This is significant in the field of masonry, especially with regard to shear walls. In modern architecture buildings are frequently designed with open plan concepts, which result in an overall reduction of the available wall cross-section to be used as shear walls. Therefore, the resulting forces/stresses are higher and it is more difficult to transfer the loads to the ground.

Another reason for the investigation about shear loads/forces, is the large quantity of reforms in the field of masonry, which have led to a vast variety of products. Especially in the last thirty years the materials used for the masonry units and the mortar has changed greatly. The size and shape of the units as well as the thickness of the joints has also changed. The units have become larger in order to reduce the labour costs and thus to allow masonry to become a competitive construction material with concrete- and steel structures. For better thermal insulation qualities, the units have been modified to contain more cavities or a higher percentage of pores. This also affects the material properties. The end effect is an enormous variety of masonry unit and mortar combinations, which in turn impacts the shear load capacity.

The standardised experimental shear test set-up used in Germany to determine the shear-strength of masonry is complex and very expensive. Beyond this, the percentage of unsuccessful tests is very high. Further, the accurate interpolation of the results can be challenging.

The afore noted points prove a need for investigating the shear behaviour of masonry. Further, determining the load bearing capacity reserves and verifying the existing theories with respect to the new materials is necessary. In some cases, a new design-model will be required. An important starting-point is the implementation of computer simulations. With high-performance computers and current FE-programs the user has new options of how to investigate masonry.

2 Calculation methods today

2.1 Calculation method according to prEN1996-1-1

The design value at the ultimate limit state for the horizontal load applied to a masonry wall, V_{Sd}, shall be less than or equal to the design value of the horizontal resistance of the wall, V_{Rd}, such that:

$$V_{Sd} \leq V_{Rd} \tag{1}$$

The design value of the horizontal resistance is given by:

$$V_{Rd} = \ell_c \cdot t \cdot \frac{f_{vk}}{\gamma_M} \tag{2}$$

ℓ_c is the length of the compressed part of the wall, ignoring any part of the wall that is in tension

t is the thickness of the wall resisting the shear

f_{vk} is the characteristic shear strength of masonry, based on the average off the vertical stresses over the compressed part of the wall

γ_M is the partial safety factor for the material

The factor for the shear strength, f_{vk}, depends on the kind of shear failure. In prEN1996-1-1 two modes are defined. The first one is the failure due to sliding between the mortar and the masonry units. For masonry with unfilled head joints this factor is given by:

$$f_{vk} = 0{,}5 \cdot f_{vko} + 0{,}4 \cdot \sigma_d \quad (3)$$

f_{vko} is the characteristic initial shear strength of masonry, under zero compressive stress

σ_d is the design compressive stress perpendicular to the bed joints

The second failure method, is a tensile failure of the unit. It is represented only by a limit. The limit is :

$$0{,}045\, f_b \text{ or } f_{vlt} \quad (4)$$

f_b is the normalised mean compressive strength of a masonry unit

f_{vlt} is a set limit value

The smaller value of f_{vk} has to be used.

2.2 Calculation method according to DIN 1053-1

In the current German masonry code there are three kinds of shear failure modes shown. However, only two are used for the calculations. The third failure mode (eq. 8) is the compressive failure of masonry. For the verification, the stresses determined via the various methods need to be compared.

$$\tau = \frac{c \cdot Q}{A} \leq \frac{zul\tau}{\gamma} \quad (5)$$

A is the area of the compressed part of the wall

Q is the shear load

c is a factor for the distribution of the shear stress, depending on the wall geometry

γ is the global safety factor

The maximum allowable shear stresses ($zul\tau$) are

$$zul\tau = \beta_{HS} + 0.4 \cdot \sigma \quad (6)$$

$$zul\tau = 0{,}45 \cdot \beta_{RZ} \cdot \sqrt{1 + \frac{\sigma}{\beta_{RZ}}} \quad (7)$$

$$zul\tau = \beta_R - \gamma \cdot \sigma \qquad \textit{not to be used} \quad (8)$$

β_{HS} is the value for the reduced initial shear strength

σ is the design compressive stress perpendicular to the shear in member

β_{RZ} is the value for the tensile strength of the unit

β_R is the compressive strength of the masonry

In the newest draft of the German code, eq. 6 and 8 are expanded with factors, which take into account the overlapping area between two units.

For eq. 7 a second formula is under discussion. However, there is not enough experience nor are there enough research results for an evaluation.

The theoretical background for eq. 6 and 8 as well as the drafted revisions, is the equilibrium of forces for one unit without head joints. Figure 1 illustrates this principle. The zone with the smaller compressive forces (σ_2) is to be used for the calculations. If σ_2 has a tension magnitude, which is higher than the bond strength, the joint will open.

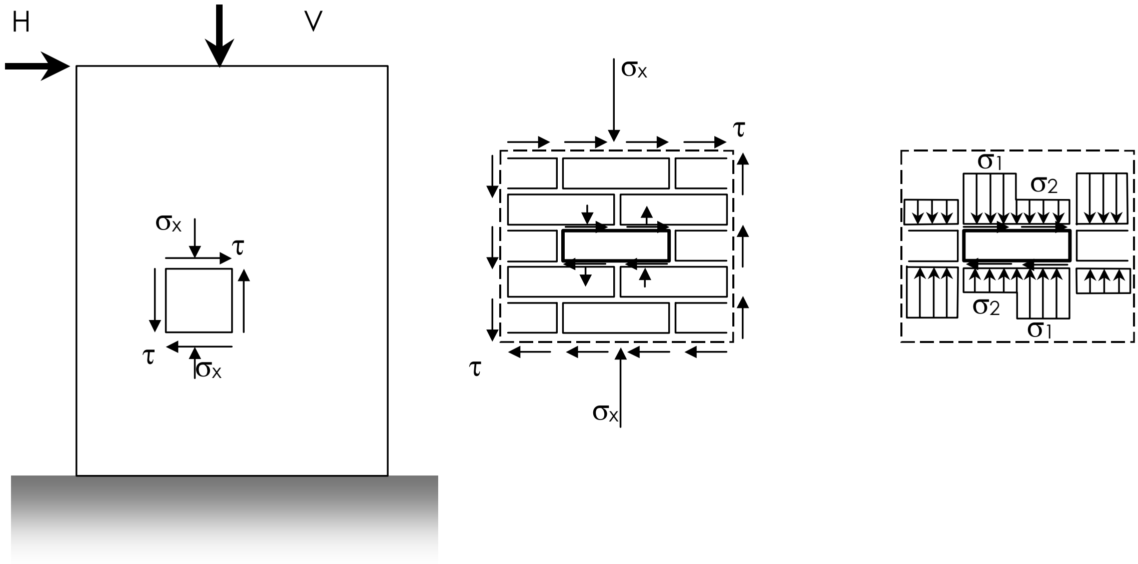


Figure 1 Principle of equilibrium of forces for one unit

For eq. 7 the first principal stress (tension) due to the maximum shear stress in the unit is calculated as 2.3 times the average shear stress, and lead to a tensile failure. This theory is based on Mann and Müller (1976).

3 Numerical modelling and simulation

3.1 Choice and specification of the numerical model

Two general approaches of simulating the stress behaviour of masonry under specified boundary conditions with FEM software are possible. The first one is the use of a macro-model. With this type of model the structure is modelled as a whole. The specific characteristics of the masonry, such as the units and the mortar, are covered via special material descriptions. The second approach is to model the unit, the mortar and the interface between the unit and mortar separately. The use of a micro-model is more appropriate for investigating the influence of different materials and geometries of the units and the mortar relative to their failure behaviours.
In a macro-model the description of the units and mortar interaction is limited by the use of the material-model. One such model was developed by Ganz (1985). In this model there are no dependencies defined between the tension failure of the unit and the overlapping area. This is only one reason for the decision to use the micro-model. Further, there are a lot more dependencies, which need to be analysed.

More details and examples for the use of macro- and micro-models are elaborated on in the article “Micro modelling for out-of-plane bending of masonry” in these conference proceedings.

The models were created with the ANSYS program. The masonry units and the mortar were modelled with SOLID65 elements. This is an eight-node 3-D structural solid-element. It is capable of cracking under tension forces and crushing under compressive forces. The maximum stress depends on the 3-D stress condition. For shear investigations, one also has to model the opening and slipping between units. In order to simulate this contact, interface-elements (CONTA173) were applied. These elements are able to transmit pressure and shear forces, while also being able to prevent tensional stresses. For the material a non-linear stress-strain curve was used.

3.2 Minimized FE-model

The enormous number of elements in a micro-model, in comparison to the macro-model, greatly increase the calculation time. Therefore the first rule of thumb for modelling is to minimize the zone to be examined. Following the principle illustrated in Figure 1 a FE-model has to be developed. Figure 2 shows such a model. The nodes at the top of the partial wall are connected with nodes at the bottom. The same procedure has to be followed for the right and left sides. In the third dimension, half the thickness of the wall is modelled. Thus the symmetry in this direction is used. This can be achieved by modelling a support at every node for the cross section.
The stress distribution in the model is equal to an infinitely large wall. That means, there is no influence of moments or concrete boundary beams, as would typically be the case in shear walls. This allows one to calculate the maximum shear stress and the failure mode due purely to shear.

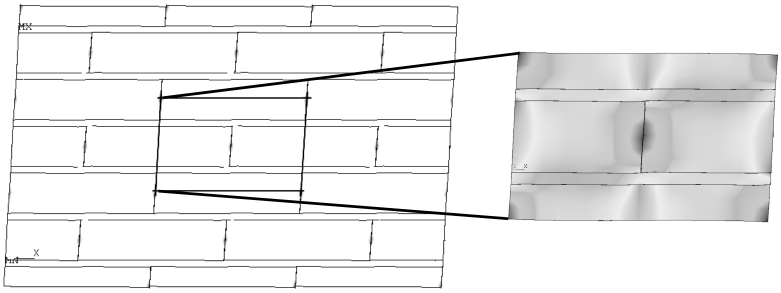


Figure 2 Minimized FE-model purely for shear

In Figure 3 some results of the shear strength in dependency of the vertical stresses are shown. For this example standard sized units (240x115x71 mm; KS f_b =15N/mm²) with general purpose mortar were used. Since the theory elaborated in section 2.2 is based on the same type of masonry, it possible to compare eq. 6-8 with the FE-results.

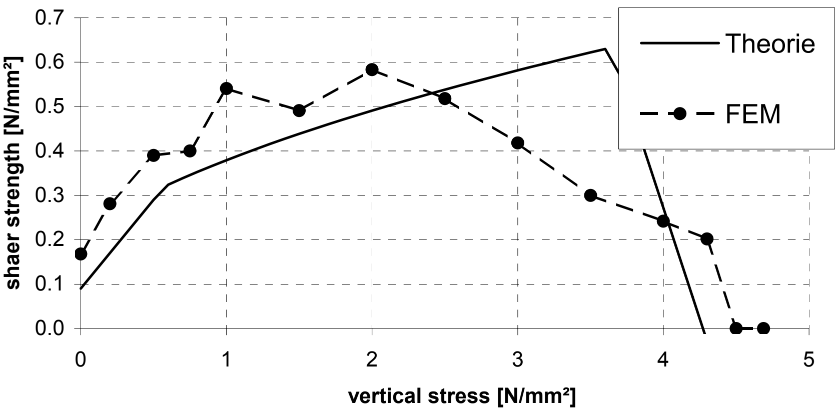


Figure 3 Comparison of FE-results with the theory (section 2.2)

The curves show a discrepancy particularly for masonry under large compressive forces. However, there are also some reserves in the zones of minimal compressive forces. The tension failure of the unit due to shear under large compressive forces and the compressive failure of the masonry due to the lateral strain of mortar, are to be superposed. This is because the tension failure of the unit starts at the edges.

In Figure 4 the crack process for a vertical compressive stress of 3.0 N/mm² is documented. Most of the failure is due to tension (circles). The normal to the surface area of the circles is the direction in which the crack opens. The process starts with cracks due to shear. Then the lateral strain of the joint leads to the rupture of the masonry unit.

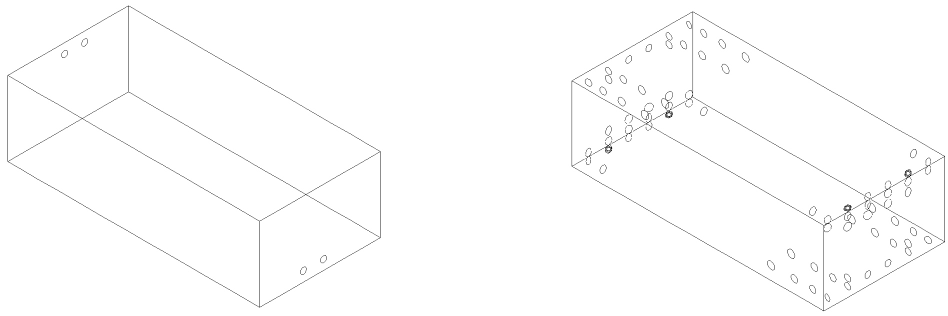


Figure 4 Cracks at the beginning (left) and at the end (right) of the crack process

In the following image on the left, the scale indicates the factor for the shear stresses prior to the first crack developing. This means the maximum shear stress in the masonry unit is about 1.5x the external average shear stress. This is less than the assumption stated in section 2.2. The right image represents the first principle stress (tension) prior to the first crack.

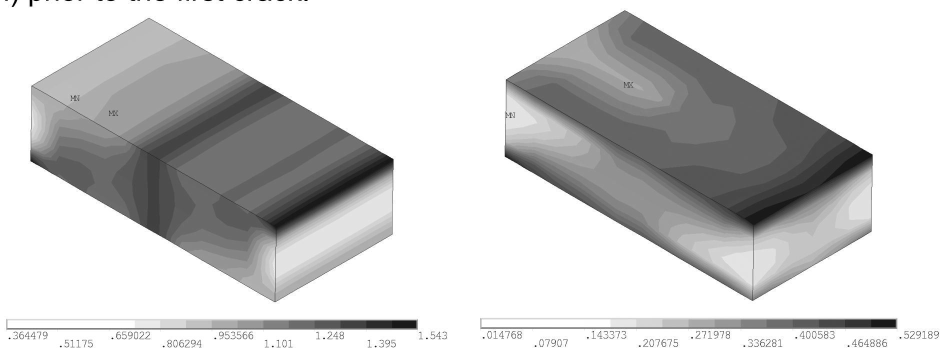


Figure 5 Factor of shear-stress (left) and of the 1st principle stress (right) in the unit prior to the first crack

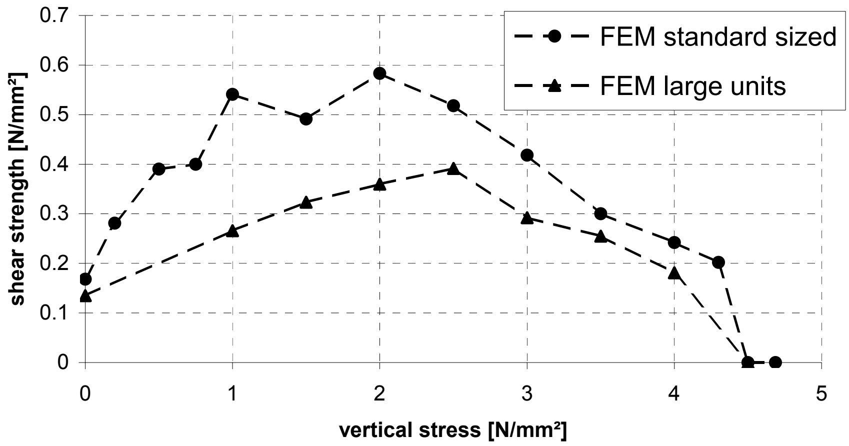


Figure 6 Comparison of different geometries of the units

By changing some of the model parameters, various influences were examined. A smaller overlapping area reduces the shear strength. If the tensile strength of the unit is increased, the shear strength also increases greatly. Thinner joints, with the same material, result in a better shear strength under large compressive forces. An example with large masonry units (h=240mm) is shown in Figure 6. In this case none of the head joints are filled.

3.3 Horizontal stresses in the head joints

Masonry with filled head joints has a higher shear strength. This is because the possible shear stresses in the head joints result in an additional equilibrium of force (see Figure 1). Thus the vertical stress-deviation is smaller. It should be noted that a pre-requisite is a good bond between the mortar and the units.

The vertical joint in Figure 7 is made with general purpose mortar, which is softer than the unit. The vertical stresses are much lower than in the masonry unit.

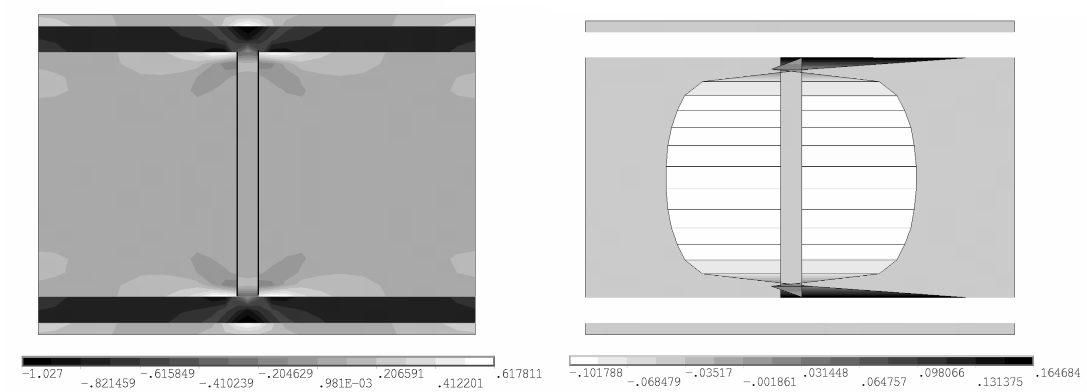


Figure 7 Horizontal stresses in the masonry (left) and at the contact area between the vertical joint and the mortar (right, tension is negative here)

The horizontal compression in the bed joint and the tension in the unit can be seen in the left image of Figure 7. The external vertical stresses, for this example, are 5 N/mm². In the right image, the pressure at the contact surface is shown (bed joints are turned off for visualisation purposes). Most of the head joint is in tension. The tension causes the mortar and the contact surface not to be able to transfer the shear forces as well.
With stronger mortar the tension in the head joints decreases. If the unit becomes softer, like autoclaved aerated concrete, the head joints are in compression.

3.4 Micro-model of a wall

The stress condition in a shear wall is different in every part of the wall. To see what the relationship or discrepancies between the theoretical model and a real masonry wall are, a model of a complete masonry wall is needed. This model can then be effectively compared with the model of the theoretical principles. To obtain the best comparison, the model of the wall should be a micro model. For the boundary conditions at the top and bottom of the wall a concrete beam is modelled.

A 2.0x2.5m wall is shown in Figure 8. The dark areas indicate high shear stresses. The highest shear stresses are located at the middle of the wall. A different geometry, i.e. proportions of the wall, could also have been used. Further, the rotational rigidity of the top beam can either be modelled as a free or fixed support condition.

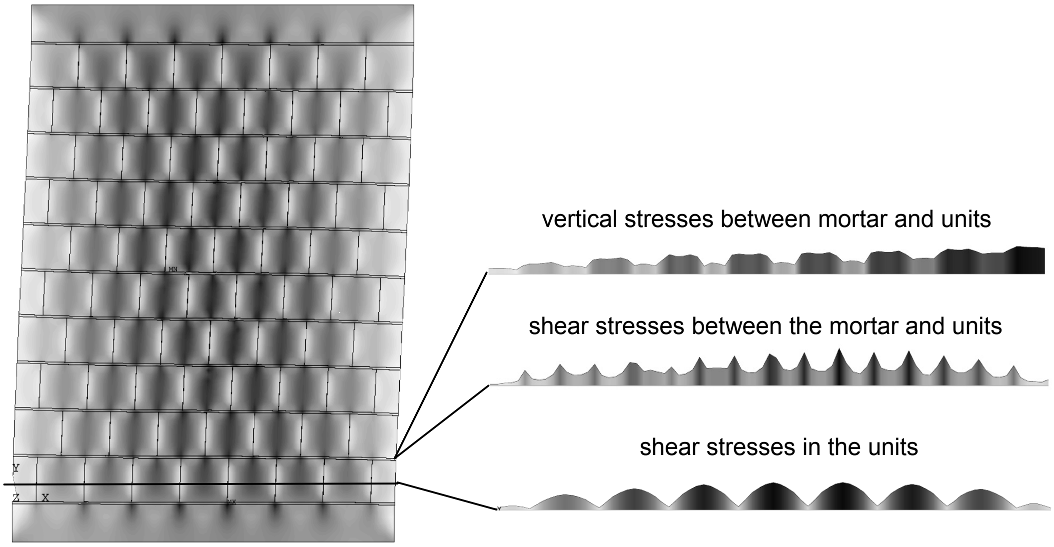


Figure 8 Distribution of the stresses in the wall

For indications about the tension failure of the wall, it is important to consider the peaks in the shear stress curve. The curve of the vertical stresses of the contact between the mortar and the masonry unit shows the same principle of masonry under shear forces as in Figure 1.

Figure 9 shows a wall with the following dimensions, 1.0x1.25m. The thickness, proportions and masonry unit sizes are the same as for the full-scale wall. The area of the cross section, the length and the height have the ratio of 0.5. The vertical load is 528 kN (2.2 N/mm²) for the semi-scaled wall. Initially, the calculations also indicated that the horizontal load capacity would be halved. This is to be confirmed by experimental tests.

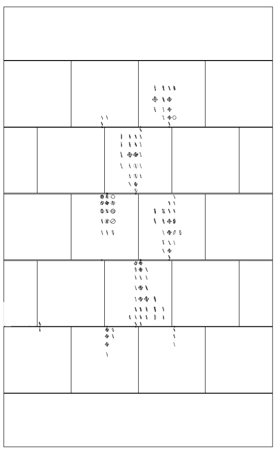


Figure 9
Cracks in a semi-scaled wall (only the wall dimensions) with thin layer mortar

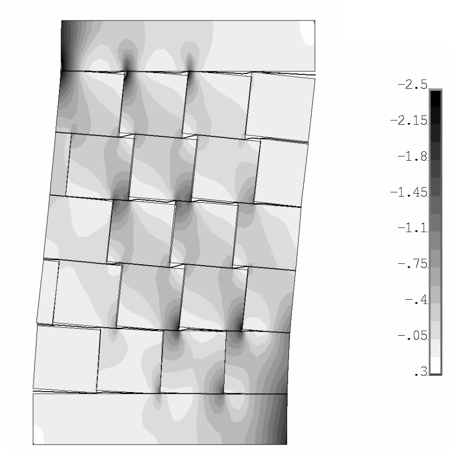


Figure 10
Vertical stress under low compression

The cracks started above or under intersections of the head- and bed joints causing the unit to break into two pieces. The point of the first failure in the wall depends on the boundary conditions (fixed or free at the top of the wall) and on the vertical load.

Figure 10 shows the same wall with a smaller overlapping length and a small vertical load (0.22 N/mm²). At the bed joints, contact to the units was lost in some regions. The failure mode for such a load combination is sliding or over-turning of the units. The mode depends on the bond pattern of the units.

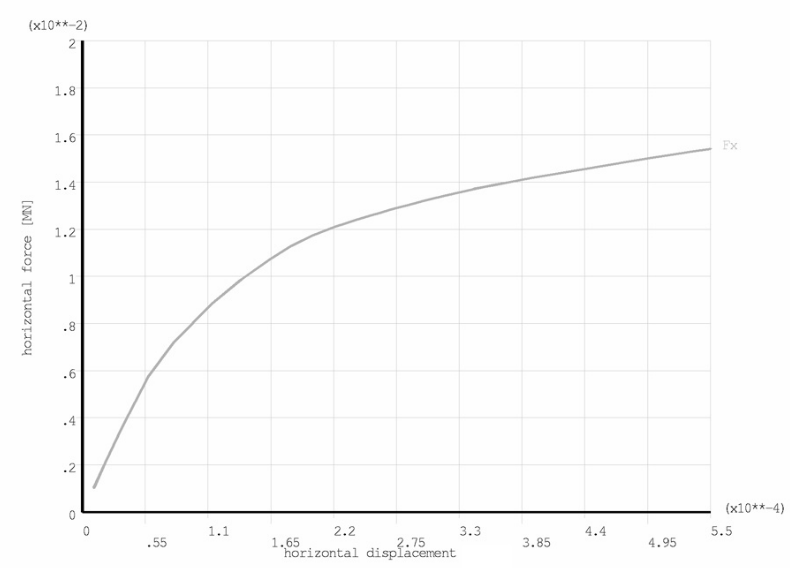
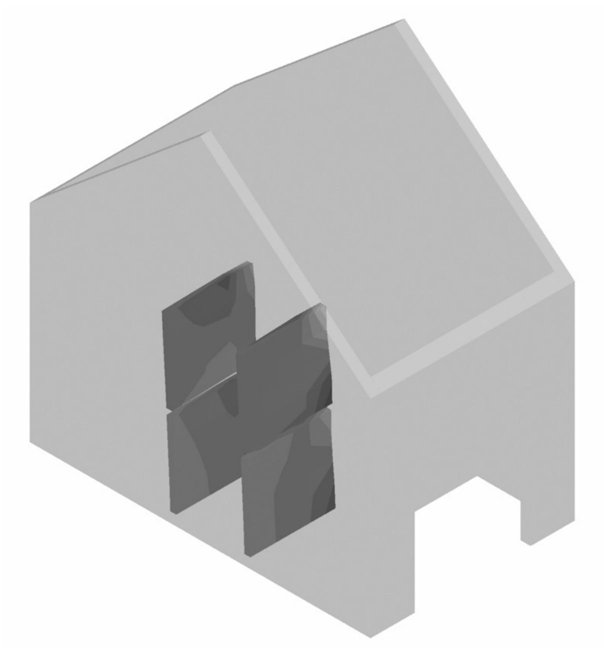


Figure 11 Curve for horizontal force and displacement at the top of the wall

In Figure 11 the force-displacement curve of the wall and load combination of Figure 10 is shown. The stiffness in the horizontal direction decreases as the horizontal loads increase. This is relevant for earthquake actions. However, it should be noted that this is only the case for small vertical loads.

3.5 Building model



For modelling an entire building or structure the micro-model is not appropriate. The calculating time rises dramatically. A macro-model should be used for complete building investigations. In this example, the modelled building is a town-house. This type of house has the greatest demands for masonry shear walls. In Figure 12 the house is shown under earthquake forces. The vertical stresses in the shear walls are shown. Black zones indicate the greatest compressive stresses. One can see, that the top of the wall is partially moment resistant.

Figure 12 Vertical stresses in shear walls in a town-house

4 Conclusion

With the FEM calculations the principle of the shear failure can be represented. However, there is a need for a modification of some of the equations. For shear walls

under small compressive forces unused reserves exist. For large compressive forces a new equations needs to be developed.

A simplification of the shear tests by using fewer courses is possible. Nonetheless, this should be verified experimentally. The main aim is to replace the standard shear test with numerical calculations.
Another question is, how many courses are needed for the existing theory. Further, the implementation of FEM investigations should be expanded for the modelling of elements/units with dimensions, which are half the length or height of a wall. The existing theory is not valid for such elements.

Acknowledgements

This research project was carried out with the financial support of the Federal Office for Building and Regional Planning Germany.

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