Interaction of Masonry Walls with Slabs and other Elements in Building Structures - Influence on the seismic resistance -

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SUMMARY:
Within the joint research program ESECMaSE (Enhanced Safety and Efficient Construction of Masonry Structures in Europe) intensive experimental and theoretical studies have been performed in order to gain a better understanding with regard to the seismic resistance of masonry walls and masonry buildings. The results have enabled the development of improved and simplified methods to predict the in plane bearing capacity under both horizontal and vertical loading. Furthermore, the importance of the interaction of walls with slabs and other walls in a building structure could be identified. On the basis of experimental observations and numerical models, an engineering approach to assess this interaction has been developed. It considers the upward movement of short walls due to rocking ("vertical growth" of walls) and the resulting increase of normal force due to restraint by stiff slabs. Furthermore, the effect of rotational restraint at the wall top on the failure mechanisms (bending or shear) and the ultimate shear force can be considered in a straightforward manner.

Masonry walls, in plane bearing capacity, interaction, slabs," vertical growth"

1. INTRODUCTION

Within the framework of the European Research Project ESECMaSE it has been demonstrated by full size experiments that a remarkable additional potential of lateral load bearing capacity of masonry walls in buildings structures exists. The evaluation of the test results has shown that the actually existing static boundary conditions at the top and bottom of the wall play an extremely important roll with respect to both the lateral load bearing capacity and the deformation capacity. In order to exploit these reserves, the exact assessment of the interaction between boundary conditions and failure mechanisms of masonry walls is required. For this purpose, results of 74 tests on full size walls under combined vertical and lateral loading have been evaluated.

2. BOUNDARY CONDITIONS OF SHEAR WALLS IN A BUILDING STRUCTURE

There are a number of boundary conditions that influence the load bearing behavior as well as the deformation characteristics of masonry walls. The following questions with regard to the static boundary conditions of masonry shear walls have been identified that must be fully understood:

- Which degree of moment fixture at the top of a shear wall is being provided by the rest of the structural system?
- Which normal force act on the shear wall actually under consideration due to static loads within the framework of the structural load bearing system and how are they distributed along the length of the wall?
- Which influence comes from the „Growth“ of the shear wall – due to a possible rotation as rigid body (rocking) –with respect to the distribution of vertical forces between the load bearing walls within the considered story of a building structure?
- How does the connection between different walls (e.g. interior and exterior walls orthogonal to each other) influence the load bearing mechanisms and the boundary conditions of a wall?

It became evident that the degree of moment fixture and the effect of „Growth“ during lateral deformation exhibits the largest influence. These effects shall be examined in the following.

2.1 „Growth“ of a wall

Many of the tests performed at the University Kassel have shown that walls which tend to a rotational movement (rocking) during increased lateral deformation, that both the center and the top of the wall exhibit an upward movement in the wall plane. This vertical movement δ_y appears like a growth of the wall at the top. It leads to an increase of vertical loading on the “growing” wall and to a pertinent unloading with respect to the normal forces in adjacent walls. With increased lateral deflection this effects becomes more and more pronounced. Fig. 1 schematically depicts the rocking of a wall as (fictive) rigid body by an angle φ.

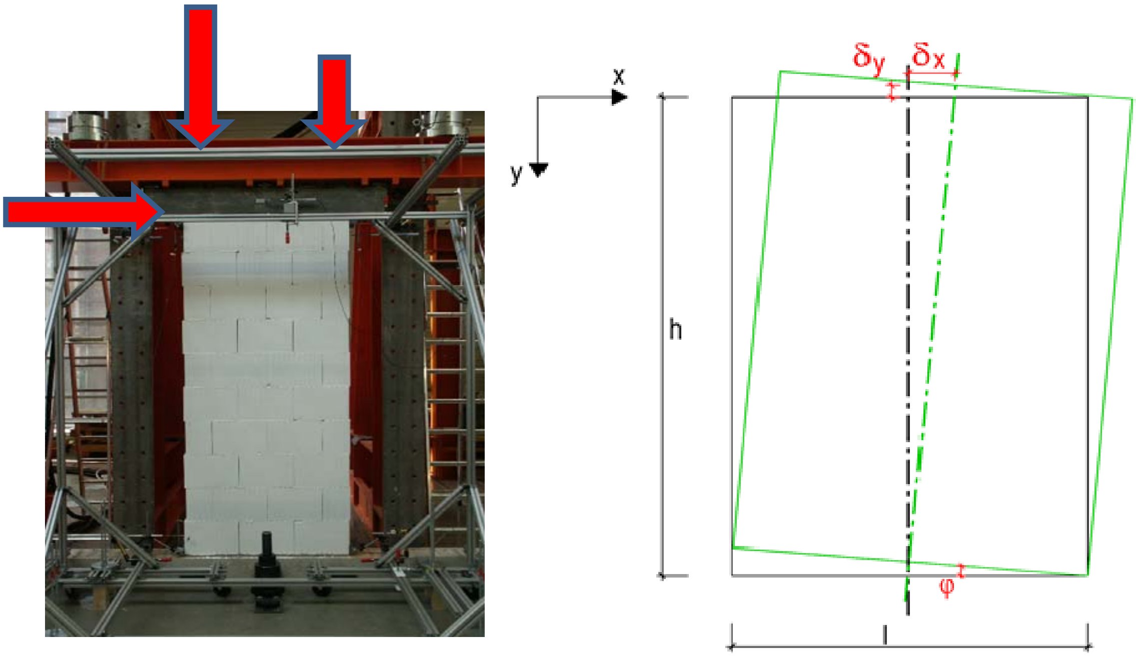

Figure 1. Rotational movement of a wall by an angle φ

The presented trigonometric relations lead to an expression for the relations between horizontal and vertical deflections at the wall top:

$$\delta_y = -\left|\frac{\delta_x}{2} \cdot \frac{l}{h}\right| \tag{1}$$

Fig. 2 shows both the results of equation (1) for an aspect ratio l/h = 2.0 and for δ_x between –20 mm and +20 mm as well as the experimental results for a masonry wall with identical aspect ratio in comparison. The experimental curve also contains the deflection due to vertical loading (normal force) as well as due to local compression at the wall toe(s) during rocking. Above 5 mm of lateral deflection it becomes evident that the experimental and the theoretical curve to rigid body movement appear as practically parallel to each other.

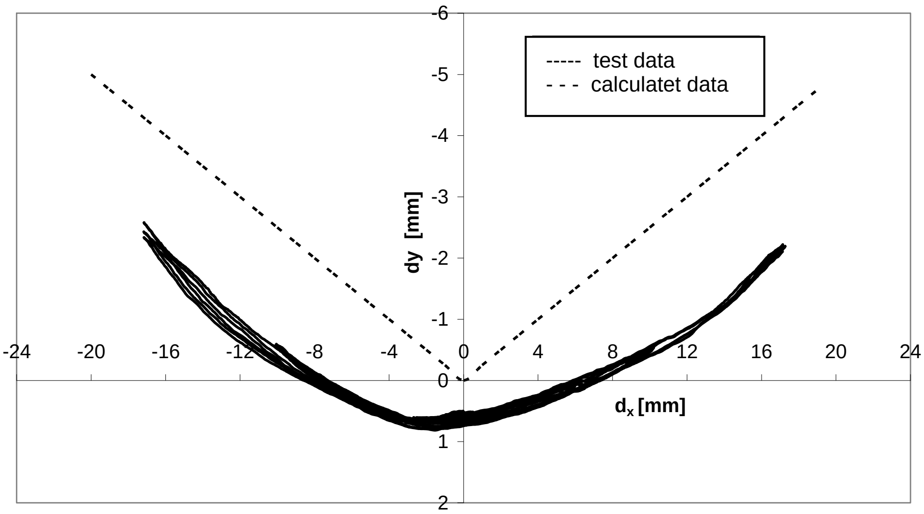

Figure 2. „Growth“ of a shear wall due to lateral deflection from experiment LAC-dm-125-110-1 after Fehling et al. (2008a)

Such a „Growth“ of a wall can lead to significant redistribution of normal forces between different walls in the same story. This should be accounted for when determining the horizontal load carrying capacity of those walls which are relevant or even decisive for the lateral load capacity of ta building.

Degree of Moment Fixture at Wall Top

The degree of moment fixture at the wall top can only be studied at the complete structural system. For this reason, numerical investigations on entire building structures have been performed within the joint research project ESECMaSE for a terraced house by Fehling et al. (2005) and for a multistory residential building by Schermer et al. (2005). Further studies by Löring (2005) and Elsche (2008) address the clamping effect due to the bending stiffness of the slabs. The investigations by Löring (2005) as well as the own investigation conclude that the degree of moment fixture depends largely on the wall length.

For typical building models with R/C slabs, short walls with a wall length of 1.00 m $\leq l_w \leq$ 1.50 m practically appear as fully clamped. With increasing wall length, the degree of fixture decreases. Furthermore, the investigations show that walls with asymmetric loading in wall plane exhibit degrees of moment fixture which depend on direction (sign) of lateral deflection.

Hence, it became clear that the degree of moment fixture of walls inside buildings depends on a number of factors, even for buildings with typical layout in plan. Furthermore, the whole band width of possible moment fixture degrees had to be expected in practice. The degree of moment fixture can change with changing lateral deflections and can be hardly predicted by usual means of structural analysis such as linear elastic theory.

However, it could be shown that a good estimate can be obtained nevertheless, as follows.

For this purpose the influence of the degree of moment fixture onto the shear capacity of the wall has be assessed. The experimental investigations by Magenes et al. (2008) in Pavia, Löring (2005) in Dortmund as well as the own tests performed at the University of Kassel contain comparative studies for walls with full clamping as well as with hinged conditions at the wall top.

As could be expected, there is no easy relationship between the degree of moment fixture and the lateral force capacity. In order to get an overview in this respect, the lateral force capacity of the investigated walls shall be depicted using a nondimensional wall coefficient k_w. As Fig. 3 shows, expectedly, the capacity of the walls with fixed condition at the top is larger than those for the hinged condition.

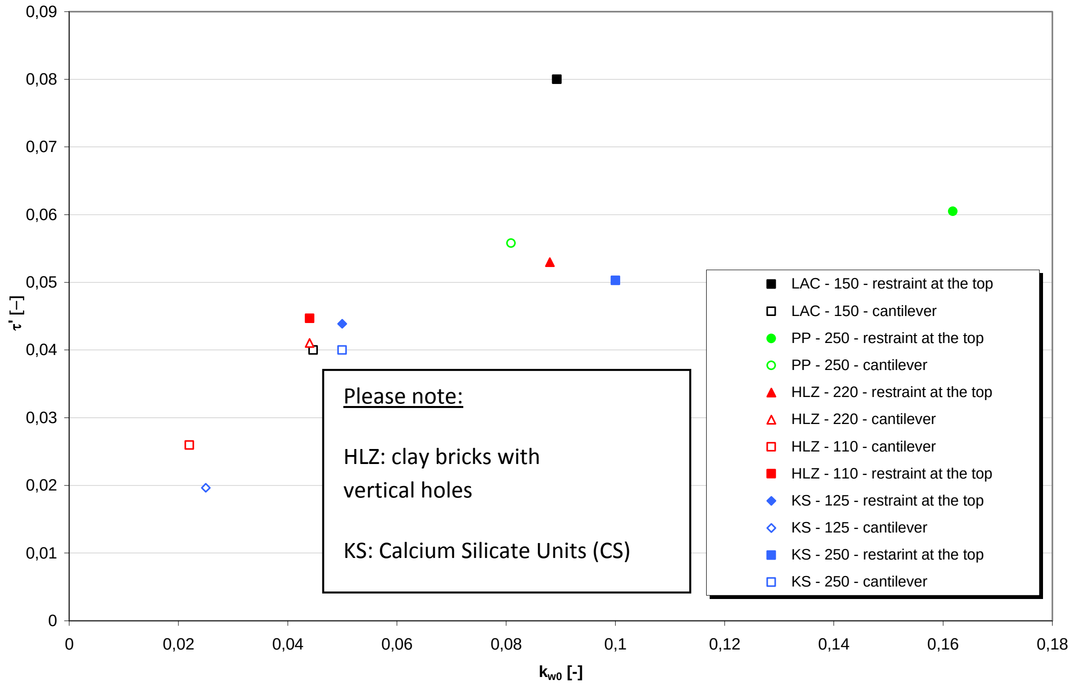

Figure 3. Influence of moment fixture (clamping) on the non-dimensional shear capacity

In order to respect both the degree of stress exploitation by normal force as well as the wall geometry (aspect ratio), the wall coefficient k_w has been defined as:

$$k_w = \alpha \cdot \left(\frac{l_w}{h_w} \right) \tag{2}$$

with the vertical stress exploitation

$$\alpha = \frac{\sigma_v}{f_k} \tag{3}$$

The nondimensional „shear“ capacity is

$$\tau' = \frac{H}{f_k \cdot A_w} \tag{4}$$

with:

σ_v normal stress due to vertical loading (normal force)
f_k compression strength of masonry
l_w wall length
h_w wall height
H maximum lateral force (“shear” force capacity)
A_w cross sectional area of wall

The position of the point of inflection (zero moment point) with regard to the wall height (= story height) can be characterized by the fixture degree ψ as it will be called in the following.
Thus, the fixture degree for full clamping both at the top and the bottom of a wall amounts to $\psi = 0.5$; for hinged condition at the top it will be $\psi = 1.0$.
In order to assess the influence of the fixture degree on the wall capacity, a further parameter β_0 is introduced which shall be called coefficient of clamping efficiency. It is defined by equation (5) and is depicted in Fig. 4.

$$\beta_0 = \frac{\tau'_{CA}}{\tau'_{DF}} \tag{5}$$

with:

τ'_{DF} = nondimensional shear capacity for full clamping at both sides (double fixed, $\psi = 1.0$)

τ'_{CA} = nondimensional shear capacity for clamping at bottom and hinge at top, both sides (cantilever, $\psi = 0.5$)

According to the test results it appears that very long walls $l_w \geq 3.0$ m and with more than $\geq 10\%$ exploitation of vertical stress exploitation, the influence of the fixture degree can be disregarded.

For short walls $l_w \leq 1.5$ m and low degrees of vertical stress exploitation α, as they are typical for the lateral stiffening of terraced houses, the fixture degree ψ has a significant influence on the lateral force capacity. This influence ranges until the factor of 2 for double fixed conditions, which means $\beta_0 = 0.5$.

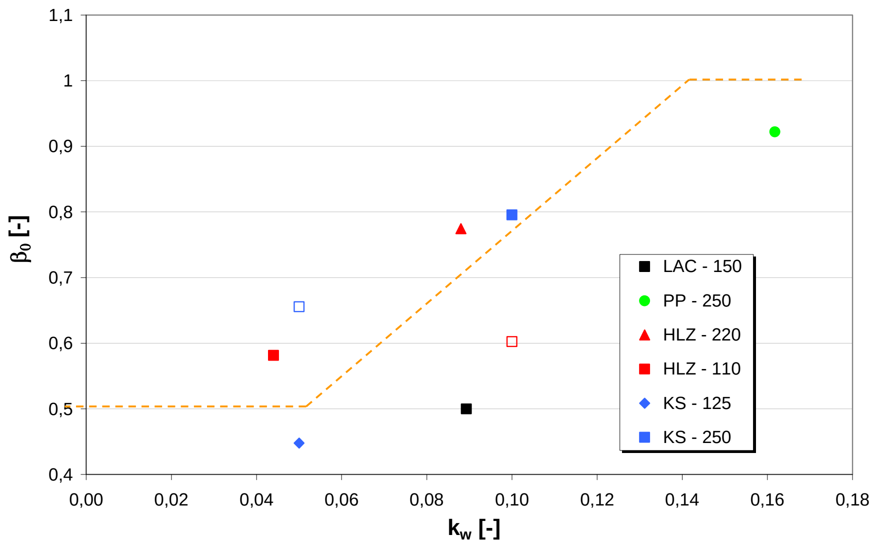

Figure 4. Influence of wall coefficient k_w on the coefficient of clamping efficiency β_0

As wall testes performed by Stürz (2009) have shown, that walls with rocking obtain „quasi plastic“ hinges since the normal force act as eccentric loads near the corners (e.g. upper left and lower right corner) on the wall. This leads to an antimetric moment distribution along the wall height, like for double fixed boundary conditions with $\psi = 1.0$. Furthermore, the maximum amount of bending moment is limited since the eccentricity of normal force cannot exceed half of the wall length as upper theoretical limit. However, the wall behavior is nonlinear elastic rather than plastic.

For limited vertical stress exploitation and short wall length, hence, the full clamping on both sides can be assumed. According to Fig. 5 this holds true for low values of the wall coefficient until $k_w = 0.06$.

In this range, toe crushing at the wall´s corners does not play a significant role, neither does shear failure. The maximum shear force is limited only by the possible mending moments.

According to Löring (2005) it can be concluded that for $k_w \geq 0.15$ the influence of moment fixture as described by the fixture degree ψ diminishes. The wall capacity will not be larger than for the boundary condition with a hinge at the top of the wall and will be governed by shear failure, not bending. Hence, for larger values of k_w, the influence of ψ can also be disregarded. This enables to use fully clamped conditions alternatively, as simplification, for the practical calculation of the lateral force capacity. Although clamping in reality does not exist for these walls, this does not lead to an overestimation of the capacity in these cases.

In Fig. 5 the results of Finite-Element models with respect to the fixture degree ψ and the coefficient of clamping efficiency β_0 is depicted. A similar result is obtained for both parameters, however, not identical.

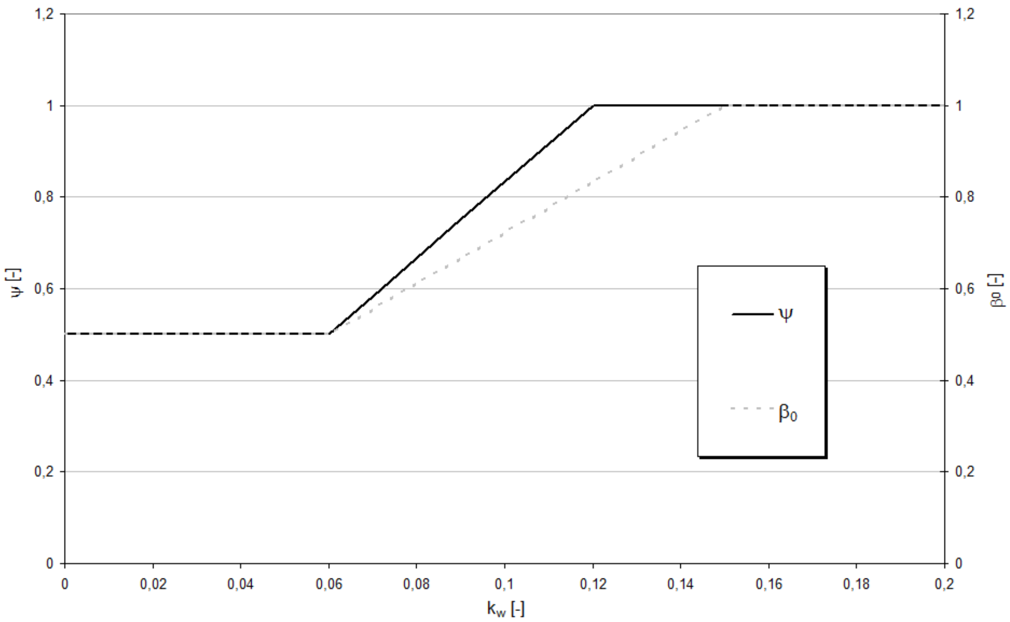

Figure 5. Dependency of fixture degree ψ and coefficient of clamping efficiency β_0 from wall coefficient k_w

In the intermediate range of $0.06 < k_w < 0.15$ the coefficient of clamping efficiency β_0 is not constant. There, the assumption of full fixation as actually existent for short walls as well as for (very) long walls (alternatively possible as simplification there) would not be conservative. In order to avoid an overestimation of lateral force capacity for the range of intermediate values of wall length, a correction factor β has been determined which relates the actual lateral force capacity to the value which would be obtained for full fixation (double fixed conditions, $\psi = 0.5$). Equation (6) contains the definition.

$$\beta = \frac{H(\psi)}{H(\psi = 0.5)} \tag{6}$$

Equations (7) and (8) deliver the values for ψ and β as linear approximations. This enables to determine the correction factor β according to equation (9):

$$\psi(k_w) = \tfrac{1}{6} + \tfrac{50}{6} \cdot k_w \qquad \text{(for } 0.06 \leq k_w \geq 0.12\text{)} \tag{7}$$

$$\beta_0(k_w) = \tfrac{1}{6} + \tfrac{50}{9} \cdot k_w \qquad \text{(for } 0.06 \leq k_w \geq 0.15\text{)} \tag{8}$$

$$\beta = 1 + 2 \cdot (\psi - 0{,}5) / (\beta_0 - 1) \quad (9)$$

In order to determine the correction factor in general, equations 7 and 8 can be put into equation 9. For 0,06 < k_w < 0,15 , the correction factor β can be obtained as a function of ψ:

$$\beta = 1 + 2 \cdot \left(\tfrac{50}{7} \cdot k_w - \tfrac{5}{14}\right) \cdot \left(\tfrac{50}{9} \cdot k_w - \tfrac{2}{3}\right) \quad \text{for } 0.06 < k_w < 0.12 \quad (10)$$

Fig. 6 depicts the correction factor β for the whole possible range of interest for the wall coefficient k_w. It can be seen that the maximum dip is less than 20 %. Hence, the maximum error due to this "shear valley" is small enough so that it would be possible to assume even the double fixture without severe overestimation.

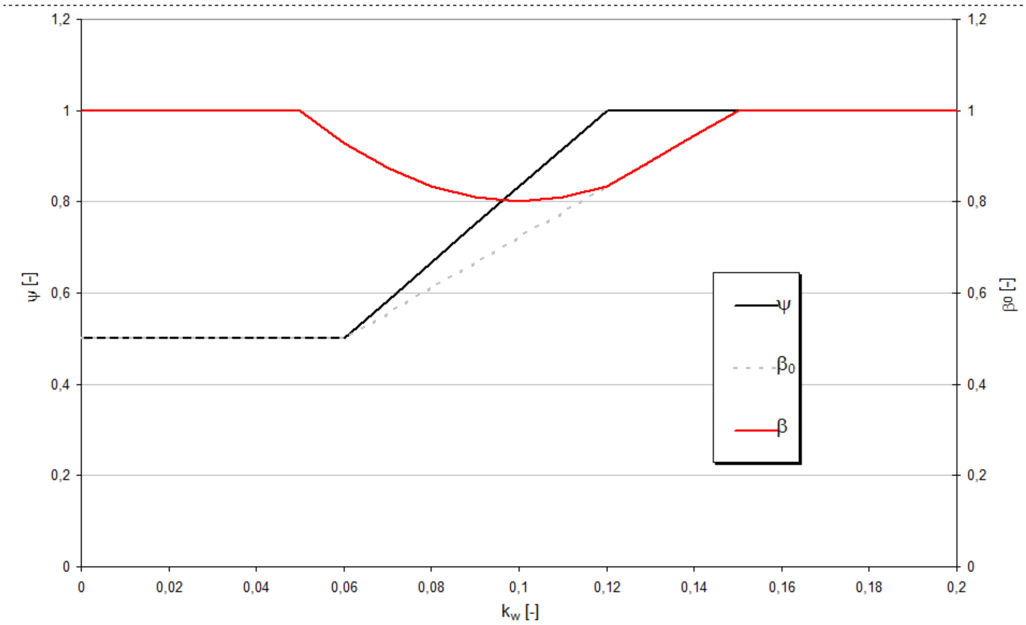

Figure 6. Correction factor β versus wall coefficient k_w

3. SEPARATION OF FAILURE MECANISMS OF MASONRY SHEAR WALLS

Due to in plane shear action, different failure mechanisms are possible for masonry walls. Three main groups of failure mechanisms can be discerned:

- Rocking
- Sliding failure
- Shear failure

Very often the failure is a mixture of different mechanisms with one dominating failure type. For discrimination between possible failure modes it is necessary to have a sufficiently large data base with experimental results. The test results from the works of Fehling et al. (2008a to 2008d), Magenes et al. (2008), Zilch et al. (2008a und 2008b) can be utilized for this purpose since they comprise a broad variety of parameters but also sufficient number of experiments with groups of identical boundary conditions. Hence, 73 experiments on full size walls could be evaluated based on their failure patterns and their force deformation hystereses.

Since both the vertical stress exploitation and the aspect ratio l/h significantly influence the type of failure, the capacity can be plotted versus the wall coefficient k_w, which is the product of stress exploitation and aspect ratio (see Fig. 7). Both are nondimensional values and, thus, allow for the comparison of a large number of experimental results for different typologies of masonry as well as for different loading and geometry. From this figure, a delineation between shear failure on the one hand as well as rocking and sliding on the other hand becomes feasible at k_w = 0.06. The modes rocking and sliding can be subsumed as rigid body modes of failure, since in these cases the wall moves as a whole diaphragm whereas shear failure means that a crack or failure band divides the wall internally. For sliding also the case of a stair case like crack can develop, but without different results in comparison for sliding of the whole wall. Hence, also this case is accounted for as rigid body mode.

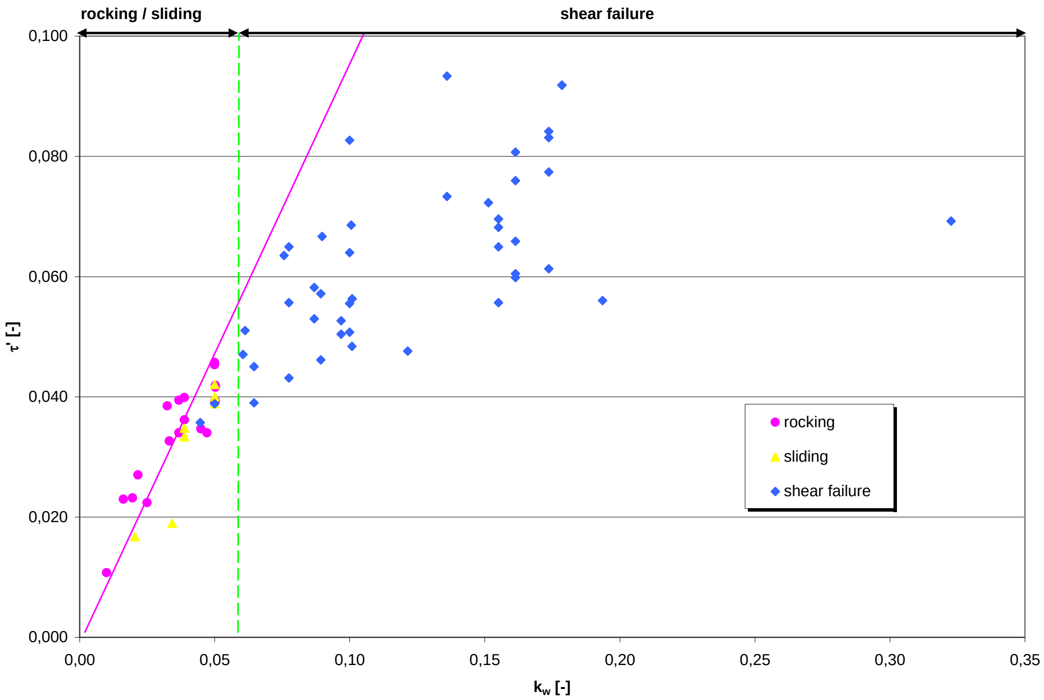

Figure 7. Nondimensional representation of lateral force capacity versus wall coefficient

In this way, the discrimination between rigid body failure modes and internal failure based on k_w. The authors believe that in this way (method a) a better delineation between different failure modes can be performed than with just comparing nominal failure loads form different mechanisms (method b). Since the estimation of the failure load level for one or more failure mechanisms may be burdened by a certain epistemic error, the wrong failure mode can be selected when just applying method b as many design codes do.

Looking at the two types of rigid body failure, it becomes clear that sliding can only be governing for longer walls with $l_w \geq \mu \cdot h_w$. Thus, a clear differentiation between sliding and rocking is possible based on the friction coefficient. Comparing the tests with sliding failure, different limits could be developed for the different types of masonry. A flow chart for the discrimination between the failure modes is given in Fig. 8.

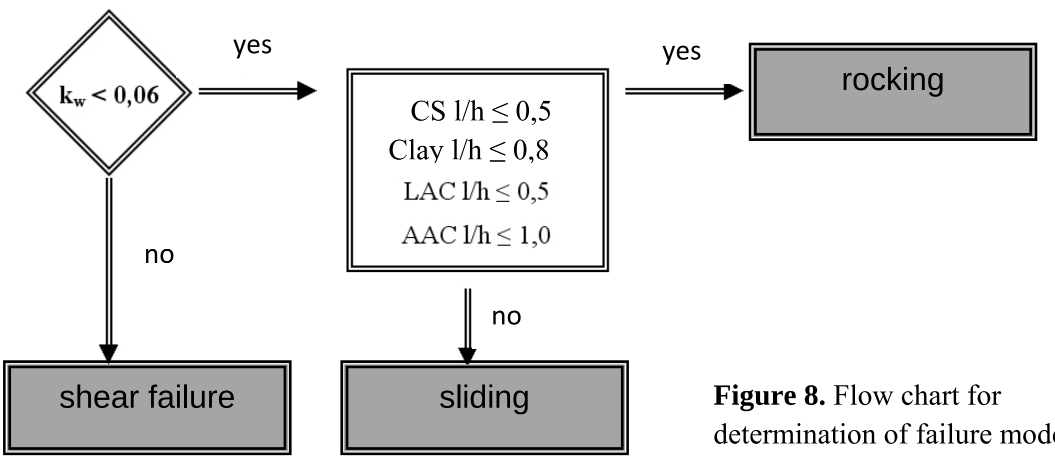

Figure 8. Flow chart for determination of failure mode

3. INTERACTION OF BOUNDARY CONDITIONS AND FAILURE MODE

The effect of wall "growth" in vertical direction has been described previously. The interaction with the rest of the load bearing structural system depends on different parameters like

- failure mode and stiffness of shear wall
- failure mode of adjacent walls
- position of wall in ground plan
- bending stiffness of story slab.

Since it is impossible to take these influences into account independently, a complete structural model is required to assess the "growth" effects. In general, this will require a finite element model taking into account the nonlinear effects of rocking as well as of load dependent stiffness of slab in bending and walls under combined action of vertical and horizontal loads.
In a simplifying manner, the shear walls may be treated as I-shaped structures (or macro elements) with moment stiff corners. The transverse walls may be modeled as line supports without load bearing capacity in tension (see Fig. 9). The support of the I-shaped macro elements also must consider that no forces can be transferred in tension.

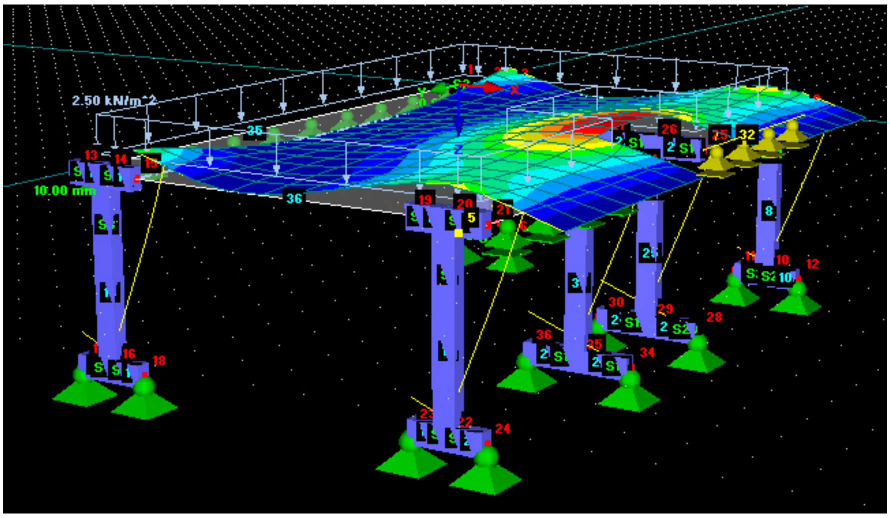

Figure 9. Isometric view of one story finite element model for a terraced house

Such a simplified model can represent the failure mode rocking of shear walls very well and, thus, can be employed to calculate the redistribution of normal forces between different walls in the story under consideration. However, it must be made sure, that the rocking mode is still in effect even for the higher normal forces after force redistribution. It can become necessary, to reiterate the model depending on the normal force level to be expected in all walls finally.

More detailed information as well as a comparison with the experimental test results for the pseudo dynamic tests within ESECMaSE on full scale terraced houses at JRC Ispra can be found in Stürz (2011).

4. Conclusions

It has been shown that the assumption of full moment fixture for both the wall top and bottom can be justified for masonry walls in buildings with stiff slabs. An overestimation of lateral load capacity is limited to less than 20 % and occurs for intermediate values of the nondimensional wall coefficient k_w. This degree of error is uncritical in comparison to the uncertainties which have to be expected for masonry in general.

Based on data from more than 70 tests on full size walls, a method has been developed to determine the governing type of failure based on wall geometry and vertical stress exploitation. By this, it is now possible to identify those walls which can be expected to exhibit rocking and, hence, may attract higher vertical loads as would be obtained from linear elastic analysis. Increased vertical loads lead to increased lateral load capacity for walls in rocking mode and thus are a main reason for the observed reserves with regard to seismic resistance. Simple finite element models may be used in order to assess the amount of redistribution of normal forces between adjacent walls.

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