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Deliverable D6.2: Development of test methods for the determination of masonry properties under lateral loads in WP7, incl. test methods for European standardisation

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1. Introduction

The test of masonry wall panels on shear loadings can be carried out on different levels. As specified in the report of the work package 6.1 [3], the investigations can take place by applying a compression force on a masonry wall panel in a specified angle to the bed joints – e.g. diagonal tension test – or by the application of a horizontal load on the cap of a masonry wall panel with several possible boundary conditions at the cap of the wall and on the sides.

Generally an inhomogeneous distribution of the stresses in the panels can be found. The determination of the shear strength of masonry from the results is difficult, as the detection of the relevant failure mode and its location is not representative for a whole masonry wall.

2. Shear Design Process

2.1. Design process according prEN 1996-1-1

The design of masonry walls in the ultimate limit state is carried out according prEN 1996-1-1 [4] by comparison of the design shear load applied to the masonry wall, V_{Sd}, and the design value of the shear resistance of the wall, V_{Rd}.

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

The shear resistance is hereby simplifying determined by multiply the value of the shear strength of masonry based on the average of the vertical stresses by the compressed part of the wall and ignoring any part of the wall that is in tension.

$$V_{\mathrm{Rd}} = f_{\mathrm{vd}}\ t\ l_{\mathrm{c}} \tag{2}$$

The determination of the characteristic shear strength includes thereby a Mohr-Coulomb-friction law and a global limitation of $\leq f_{vlt}$ or $0.065 f_b$ to describe tension failure of the units.

$$f_{\mathrm{vk}} = f_{\mathrm{vko}} + 0{,}4\ \sigma_{\mathrm{d}} \tag{3}$$

Also the regard of unfilled perpend joints is described only global by reducing the initial shear strength f_{vk0} to 50% of the value for filled perpend joints. This empirical demand can't be justified mechanically / physically.

It has to be mentioned, that the determination of the input parameters, initial shear strength, friction coefficient and limit value f_{vlt} or $0.065f_b$ results from tests where only selective material properties can be obtained from. Other failure criteria or the co-action and interaction of failure modes, e.g. increasing compression strength in bi-axial compressed areas, can't be described by these simple tests on material properties.

Also the calibration of the design process with the mentioned input parameters to tests on full-scale walls showes a big lack of knowledge.

The current approach, to calculate the shear load capacity of masonry walls by taking roughly determined material properties and implementing them in a mechanical model in which the stress distribution in the wall is also considered also only roughly, leads to improper results.

2.2. Future design approach

In the future the shear resistance of the wall in formula (1), V_R, will base on the description of the total load bearing capacity of the wall taking the boundary conditions and other effects into consideration.

The determination of the shear resistance of the wall close to reality can be carried out by a new design model (to be developed in work-package 4) and / or by full scale tests. Both approaches have to cover the mentioned boundary conditions properly.

3. Boundary Conditions of Shear Walls in Common Structures

3.1. General

Regarding the masonry shear walls in whole structures, the boundary conditions at the cap and the bottom of the wall can be described by the three load parameters:

- normal force N (assumed to be constant along the height: $N_{cap} = N_{bottom}$)
- bending moment at the cap M_{cap} and the bottom M_{bottom}
- horizontal force V

The distribution of the stresses in the cross section is affected by the floor slabs. In the mentioned investigations relatively stiff floor slabs are assumed to distribute the horizontal and also the vertical forces, e.g. solid reinforce concrete slabs.

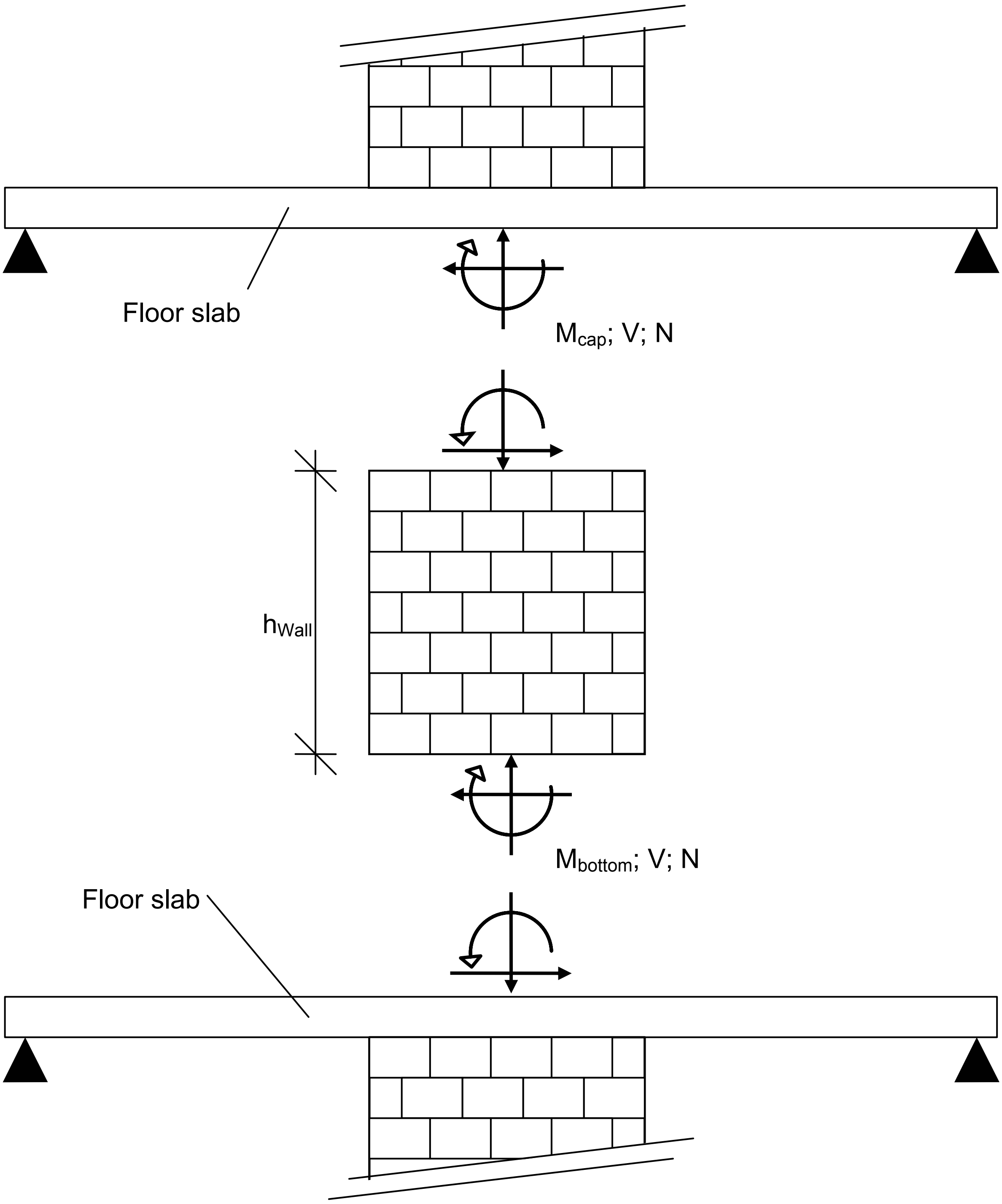

Figure 1: Loads acting on the shear walls with a rectangular cross section – boundary conditions of the wall to the floor slabs with internal forces - schematic diagram

Regarding the compatibility in the structure, the rotation of the wall at the cap and the bottom result in a restraint in the slabs due do plate bending.

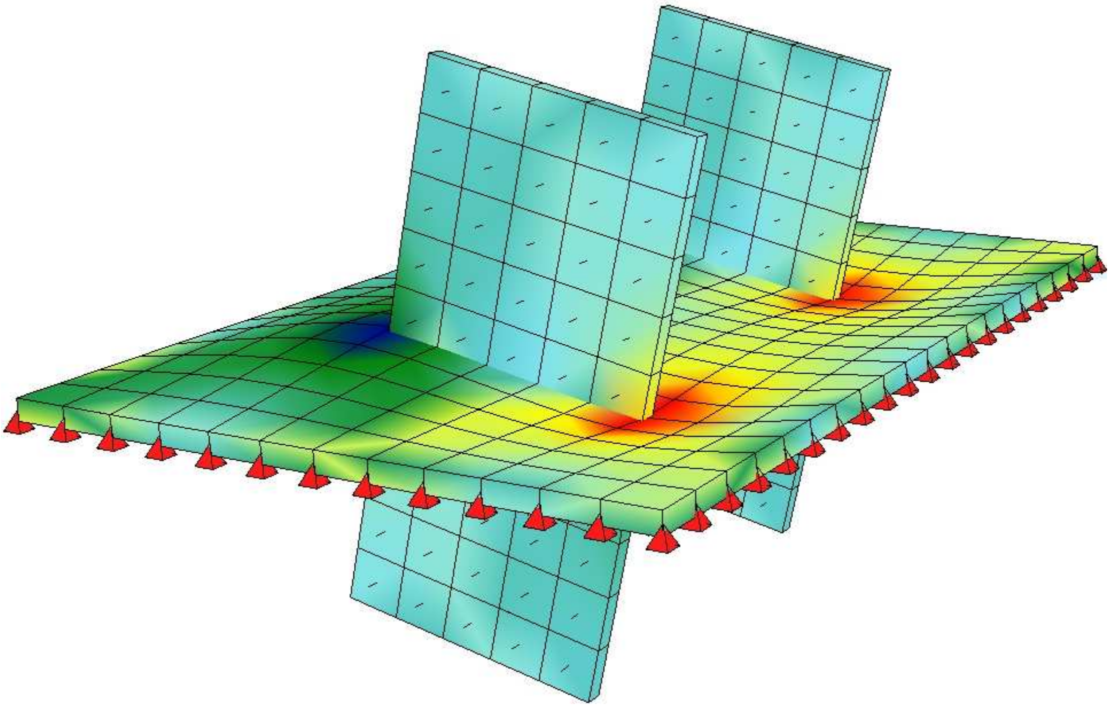

Figure 2: Restraint effect of two wall specimens in a four-side supported slab – qualitative distribution of the stresses at the surface of the structure from a linear-elastic calculation

The structural system can be reduced to the shear wall – running from the foundation to the top of the structure – and the floor slabs spanning to the cross walls. In most cases for the load bearing capacity of the structure, the shear wall in the 1st storey is relevant. In the upper storeys, due to the low vertical load level, under high horizontal loadings the walls start to gape and a ductile behaviour appears.

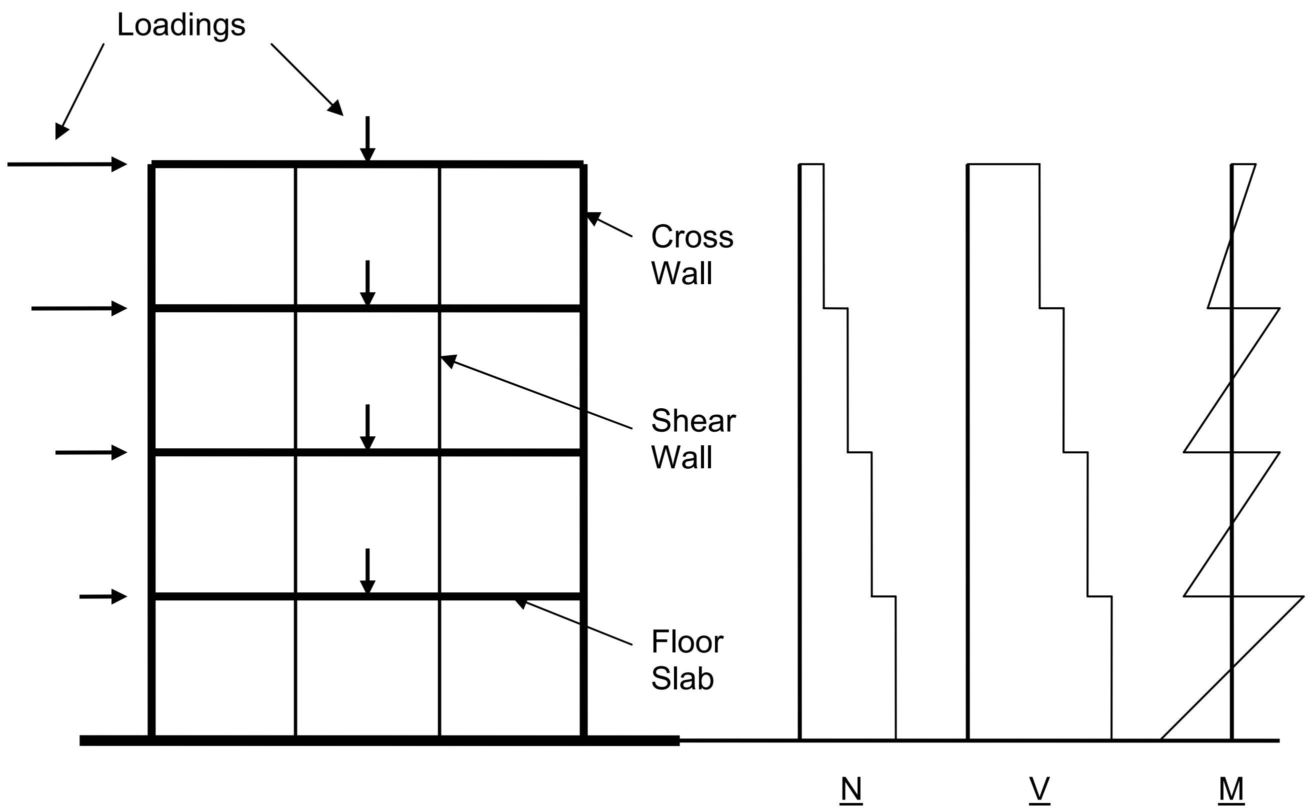

Figure 3: Plane structural system of the investigated shear wall in a multi-storey masonry structure with restraint in the floor slabs – characteristic distribution of the internal forces of the shear wall (schematic diagram)

3.2. Results from work-package 3

The spatial calculation of masonry structures taking non-linear material behaviour into consideration led to the description of the stress distribution in the shear walls in dependency of several parameters ([1]; [2]). It was found, that under high horizontal loads the compresses area of the walls was significantly reduced. The distribution of the total horizontal load in the structure to the shear walls depended significantly on their stiffness and its degradation.

The distribution of the normal stresses lead to the eccentricity e of the normal force N at the cap and the bottom of the wall. This can be generally converted to an axial normal force and a bending moment $M = N \cdot e$.

The eccentricity was found to be at the cap of the wall close to the eccentricity at the bottom of the wall – mirror-inverted to the centre of the wall. That means an almost absolute rigid restraint in the floor slabs occurred. The level of the restraint depended on the geometry of the wall, i.e. the stiffness of the wall in plane in comparison to the bending stiffness of the slab under plate loadings. For long walls the restraint effect was reduced significantly. On the other hand, in reality, the design of short walls is relevant and deciding, as the shear load bearing capacity of long walls is generally high and doesn't needed to be proved.

Also in some cases at walls with T-shaped cross sections or unsymmetric slab systems in the plan, also a ratio between moment at the cap and moment at the bottom $\left|\frac{M_{cap}}{M_{bottom}}\right| < 1$ the relevant shear walls in the 1st storey was found.

3.3. Conclusions for the shear test method

The tests have to be performed on full-scale masonry walls to cover size and other above-mentioned effects. The height and the length of the tested walls should be determined close to real structures. The tests should identify the shear load bearing capacity of the wall V_R as input parameter for the design process (see (1)).

The vertical on the test specimen applied forces describe the vertical load and also the cap moment.

For representing the distribution of the bending moment and eccentricities, the ratio between $\left|\frac{M_{cap}}{M_{bottom}}\right|$ is fixed to be 1.0 – mirror-inverted to the centre of the wall. That means that the eccentricity of the resulting force in the mid height of the wall has to be zero and no relative rotation between cap and bottom of the wall should occurs.

The conversion of different ratios to the proposed ratio can be done by modifying the geometric properties, e.g. by reducing the length of the tested wall.

The following dependencies can be noticed:

$$M_{cap} = N \cdot e = V \cdot \frac{h_{Wall}}{2} = -M_{bottom} \tag{4}$$

$$|R| = \sqrt{|N|^2 + |V|^2} \tag{5}$$

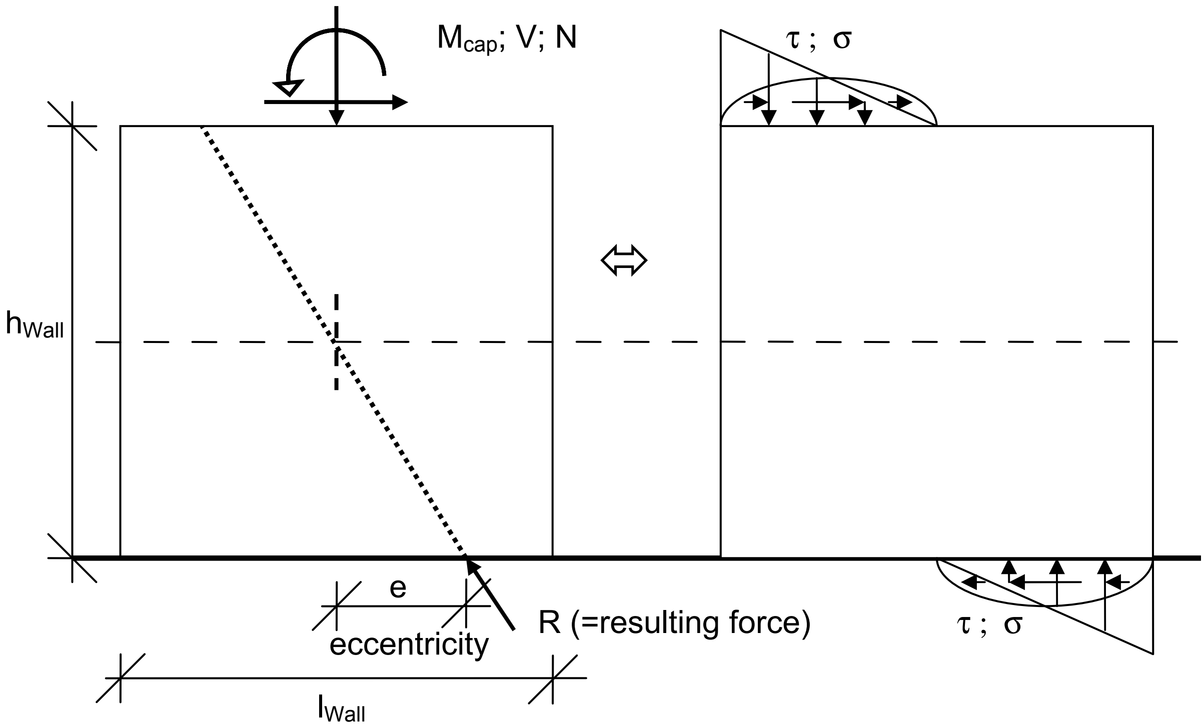

Figure 4: Load and stress state on the idealized shear-wall - schematic diagram

The mechanical system can be reduced to the following:

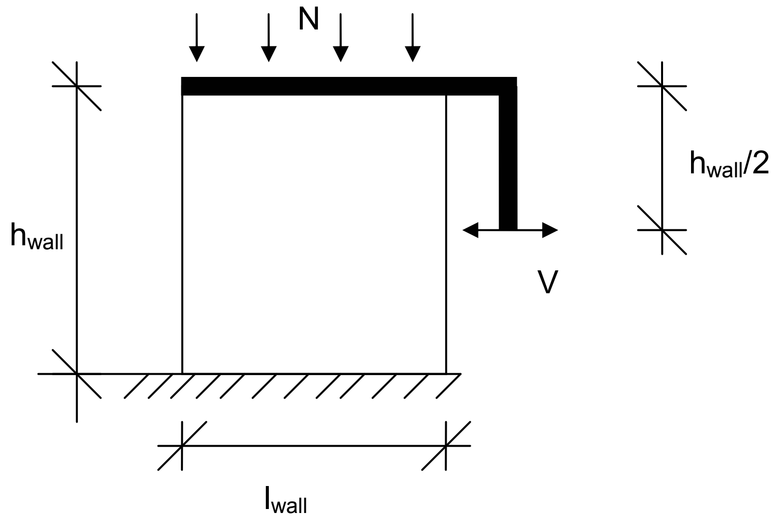

Figure 5: Mechanical system of the load application of the idealized shear wall - schematic diagram

On the sides of the walls no load, e.g. like vertical tangential shear stresses proposed in the tests of Müller (s. deliverable 6.1, figure 8), is applied at all.

Following conditions have to be fulfilled additionally:

- The resulting normal force has to remain absolutely constant during the experiment. That means, that mechanical fixations to reduce the cap rotation are not suitable at all, as in these systems, additional parasiteric vertical forces are activated caused by the unavoidable uplift of the wall under bending. Though, the vertical load application has to be ensured by force controlled actuators.
- As the bending moment depends on the horizontal load V, the eccentricity varies during the test and a variable controlling system has to ensure equation (4).
- The distribution of the vertical stresses should be linear. Depending on the eccentricity of the vertical force partially areas without compression should be covered (s. Figure 4).
- For the standard determination of the shear load bearing capacity of a wall, the horizontal load has to be increased continuously in the experiments till the collapse of the wall is reached. To cover also the post-crack behaviour, the application has to be carried out displacement controlled.
- Generally also cyclic horizontal loadings (compression / tension) should be possible with alternating sign of V to investigate the behaviour e.g. under seismic loadings.
- The horizontal load application has to ensure a constant cap-displacement along the wall. The adjusting shear stress distribution at the cap of the wall depends on the vertical stresses in the compressed part and the geometric properties of the wall in general.

  To ensure this demand, a very high longitudinal stiffness of the directly on the cap of the wall placed beam is necessary.

4. Proposed Test Method for the European Standardisation

4.1. Application of the horizontal load

For the horizontal load application, a beam (labelled as part B) with high longitudinal stiffness is recommended. Here either a HEB-girder (e.g. 2 parallel HEB140) or a steel girder with solid section is possible. The beam has to be placed in a cement-mortar bed at the cap of the wall. To avoid sliding, at the bottom side of the beam some interlocking elements has to be welded on.

The application point of the horizontal displacement has to be placed ideally at the mid of the wall length. To avoid unintentional restraint effects, a hinge in form of a round steel bar can be placed running through the beam. The load is carried from the hydraulic actuator to the hinge by 2 U-girders running parallel on both sides of the cap beam.

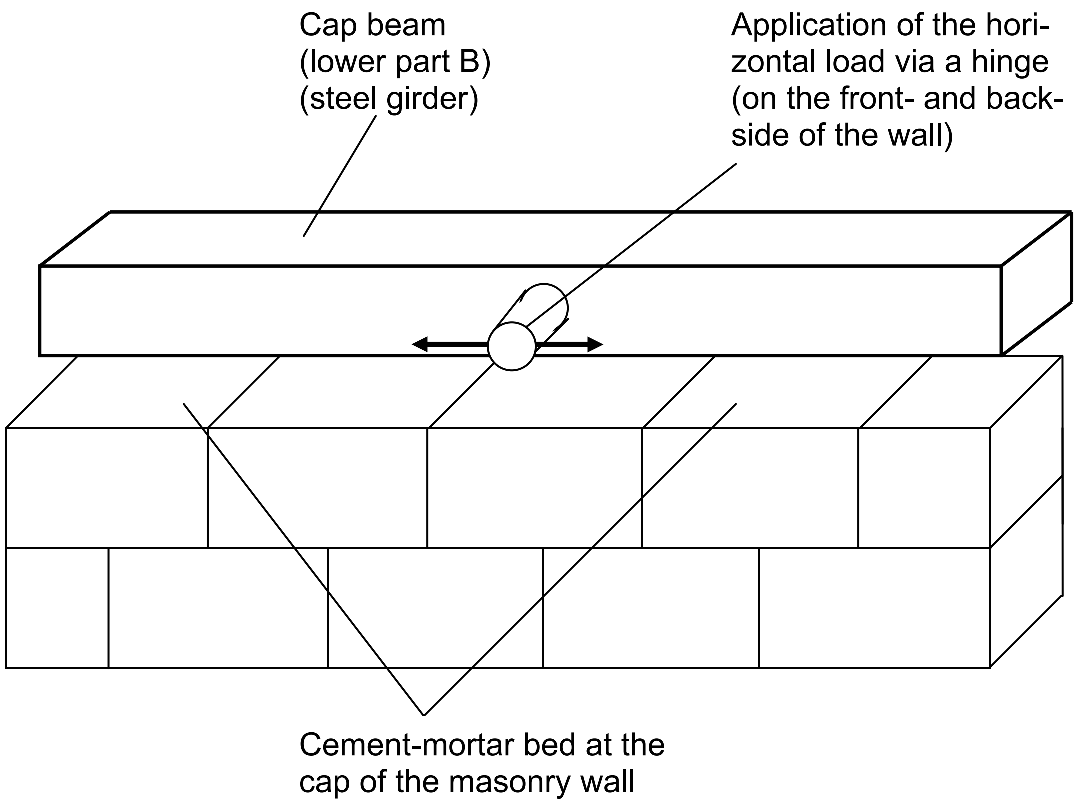

Figure 6: Application of the horizontal load to the cap of the wall

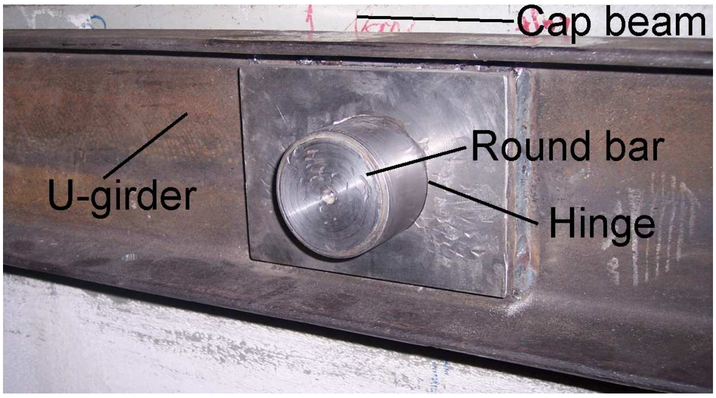

Figure 7: Detail of the hinge construction – round bar and strengthened U-girder

4.2. Application of the vertical load

To apply a normal force with a defined eccentricity at least two hydraulic actuators are needed. As no tension force can be carried from the vertical hydraulic actuator to the wall, the arrangement of the forces in the plan view is deciding for the maximum possible eccentricity , i.e. $M_{max}=N*e_{max}$.

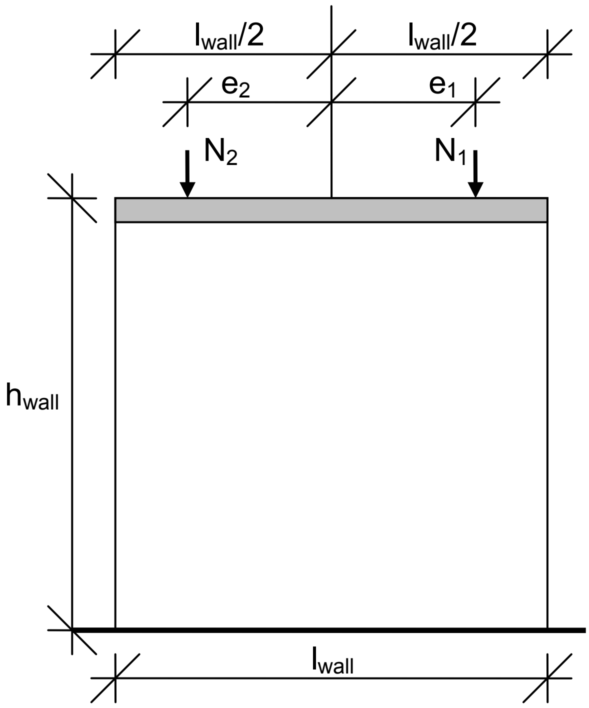

Figure 8: Application of the vertical load and the cap moment on the masonry wall using two hydraulic actuators - schematic diagram

For a linear distribution of the vertical stresses a soft interface layer between a upper stiff beam (labelled as part A) and the wall is proposed. Therefore the use of unreinforced elastomeric stripes with the cross section of 50mm (length) x 40mm (height) is suitable.

For the execution type *Calenberg Compactlager S70*, t = 2 x 20mm is recommended. The form-factor is S = 1.0 and the compression modulus E_D = 11.7 N/mm² (initial stiffness). For the calculations assuming a wall thickness of 17.5cm the spring constant is determined to c = 2600 kN/mm. The allowable stress is 14.8 N/mm², i.e. 129.5 kN per stripe, where a deformation of 2 x 6mm = 12mm appears (differing from the determination using the initial stiffness due to rubber deformation characteristics).

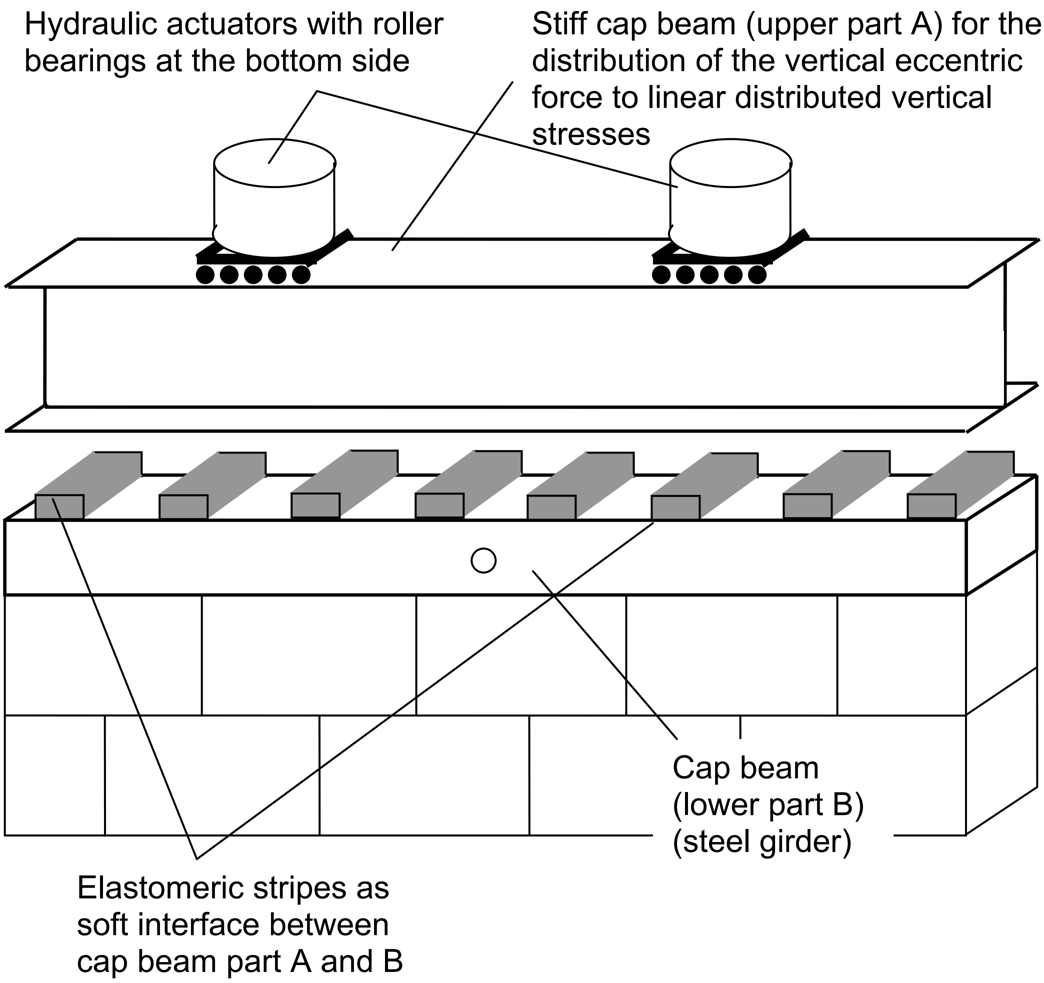

Figure 9: Application of the vertical load and the cap moment on the masonry wall using two hydraulic actuators - schematic diagram

For the cap beam a HEB240-steel girder (upper part A) and two parallel HEB 140-steel girders (lower part B) could be used.

4.3. Controlling of the load application

The load application is proposed to be force controlled by two vertical hydraulic actuators and displacement controlled for the horizontal load.

The vertical forces N_1 and N_2 have to be controlled independently to ensure the constant normal force $N = N_1 + N_2$ and also the cap Moment (s. Figure 8).

$$M_{cap} = N_2 \cdot e_2 - N_1 \cdot e_1 = V \cdot \frac{h_{Wall}}{2} \qquad (6)$$

4.4. Numerical investigations

Based on the proposed test method several non-linear finite-element calculations have been carried out on a plane system. Only the upper half of the wall was regarded with the two cap beams and the soft layer in between. The dimensions of the wall were 2.5m (length) x 1.25m (half height) x 17.5cm (thickness) and the Young-modulus 6,000N/mm². The contact between the cap of the wall and the cap beam B and also the contact between cap beams A and B was covered by springs without any tension strength. The stiffness between the cap of the masonry wall and the girder B was taken to be very stiff and the contact between girder A and B by springs with the above determined spring constant c = 2600 kN/mm. Totally 10 stripes were applied with a constant distance of 25cm. The vertical load was separated to two single forces N_1 and N_2 (s. Figure 8 and equation (6), applied as point load on the cap beam A without any distribution length) covering also the cap moment.

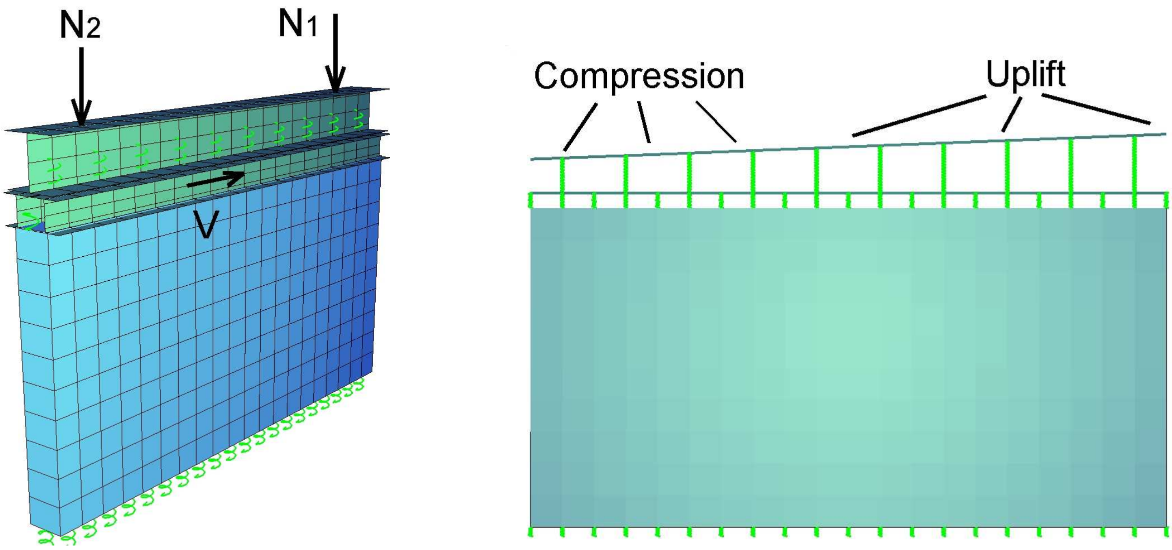

Figure 10: Isometric view to the finite-element-model with the loads (left) and uplift effect under loadings (right)

The following loads were applied:

- V = 230 kN
- M_{cap} = 230kN x (1.25m + 0.05m) = 299kNm
  (load application in the middle of the girder B)
- N = 300 kN;
  $e_1 = e_2$ = 1.0m
  => N_1 = 0.5 kN; N_2 = 299.5kN

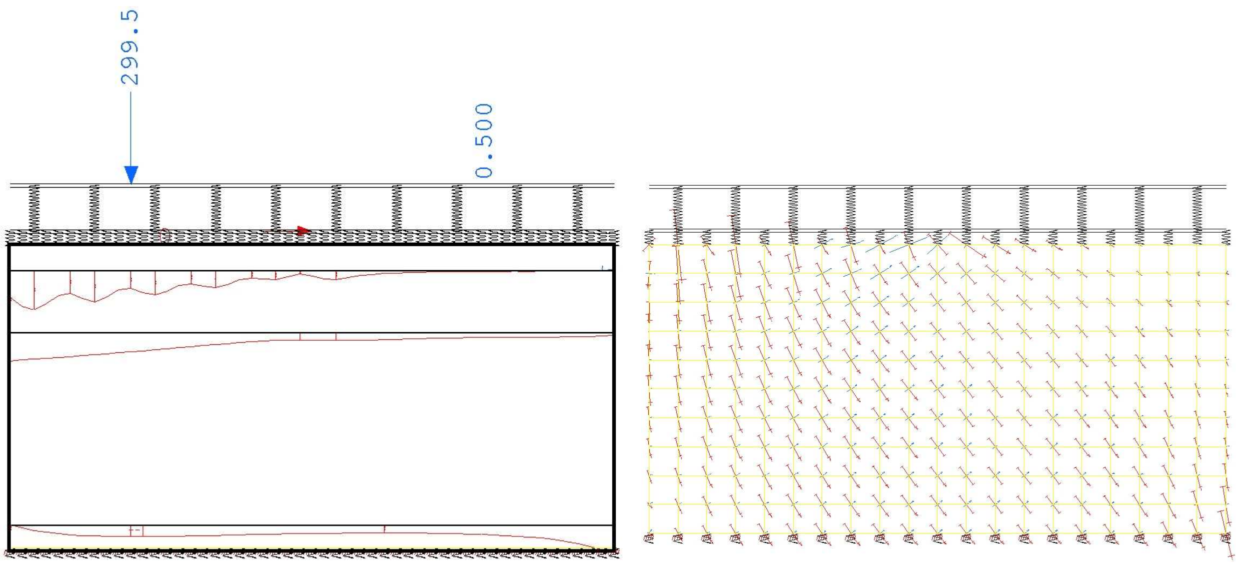

Figure 11: Results of the non-linear calculations: vertical stress distribution perpendicular in the given sections (left) and trajectories (right)

The spring forces and their position are given below.

Number	1	2	3	4	5	6	7	8	9	10
Position [m]	0.125	0.375	0.625	0.875	1.125	1.375	1.625	1.8756	2.125	2.375
Force [kN]	92.0	75.7	58.9	41.7	24.5	7.3	0	0	0	0

Table 1: Spring forces (forces in the elastomeric stripes) from the FEM-calculation

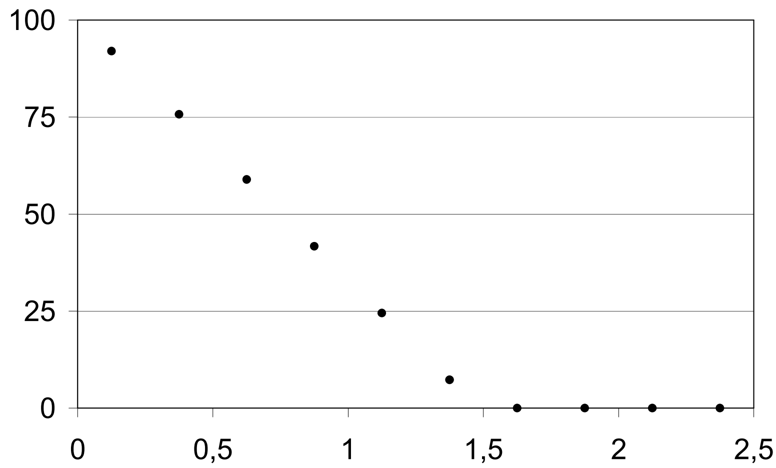

Figure 12: Distribution of the spring forces (forces in the elastomeric stripes) in dependency of the location in the plan

Regarding the distribution of the stresses (Figure 11) it can be found, that in a distance of about 12.5 cm from the cap (left corner at the top of the wall) the local effects of the load application can be clearly detected but in a distance of 37.5cm these effects are totally distributed to a smeared and almost linear characteristic.

The numerical investigations on recommended test method showed that the stress distribution is close to the above given demands.

5. Test Programme

For the tests in the following work-packages the following parameters should be varied and their effects on the load bearing behaviour investigated:

- Materials (units / mortar / overlapping length / length to height-ratio)
- Geometry of the wall (length to height-ratio)
- Normal stress (to be given in relation to the allowable normal stress according the conventional design process)

6. Appendix

[1]: Deliverable D3.1: Analysis of Terraced House

[2]: Deliverable D3.2: Analysis of Apartment House

[3]: Deliverable D6.1: Study on suitability of existing test methods

[4]: prEN 1996-1-1 (2003): Eurocode 6: Design of masonry structures: Part 1-reinforced and unreinforced masonry structures; CEN TC250 – 2003-03.