BEHAVIOUR OF REINFORCED MASONRY WALLS MADE OF HOLLOW CLAY UNITS WITH CONCRETE INFILL UNDER COMBINED LOADINGS

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SUMMARY

Masonry made of hollow clay blocks with concrete infill shows a high load bearing capacity under compression loadings. For the application under dominant shear loadings – especially in shear walls in constructions subjected to seismic loadings – a suitable reinforcement system and adopted concrete was developed amongst others during the international research project DISWall. This paper deals with the numerical simulation of the behaviour under combined N-M-V-loadings. Also a design approach is recommended.

INTRODUCTION

Hollow clay blocks with concrete infill are commonly used for heavily loaded structural members in masonry structures and for walls with high demands on sound insulation. Within the running research project DISWALL this type of masonry is optimized using numerical and experimental investigations.

The relevant loadings for the mentioned types of constructions are in-plane combined N-M-V-loadings due to dead-, live- and wind- or earthquake loadings. The mentioned system consists of hollow clay blocks (dimensions of 500mm x 240mm x 250mm, l x b x h) and a concrete infill with a new developed type of reinforcement (see Figure 1).

The filling of the wall with concrete is done usually after the erection of the masonry wall in full storey height. As the use of vibrators is not possible with the present reinforcement, the application of self-compaction concrete was chosen to ensure a sufficient compaction of the concrete infill.

Figure 1. Masonry system with placed reinforcement mesh, here shown without the concrete infill

NUMERICAL MODELLING

Within this project the numerical modelling at the TU Munich was carried out to provide information concerning the structural behaviour of these types of walls and the suitable design process. Therefore the following parameters had been taken into account:

- characteristic of the several load types (vertical compression / in-plane bending / shear loadings; static or static-cyclic application of the horizontal loads)
- type of the structural system of the wall under in-plane loadings (shear-slenderness, boundary conditions i.e. cantilever system vs. full restraint)
- material parameters (bricks / reinforcement / horizontal & vertical)
- geometry of the wall

Simulating the ULS

For the design of the mentioned masonry walls generally the ultimate limit-state (ULS) is relevant. Therefore in the ULS under in-plane loadings numerical investigations using the finite element-method have been carried out.

The co-action between clay shells and concrete infill is affected by several parameters, e.g.

- effective adhesion (dependent on local moisture conditions)
- mechanical interlocking
- shrinkage of the concrete infill
- different stiffness of clay shell and concrete infill
- different strength / maximum strain under compression and tension

In the ultimate-limit-state, where high stresses and deformations especially under cyclic loadings appear, it can be assumed, that an extensive decoupling between the clay shells and the concrete infill occurs. In consequence, further the clay shells are not taken into account simulating the ULS.

FE-Model

The finite-element-mesh was applied parallel for the concrete and also the reinforcement. The types of elements were chosen to

- concrete: four-node, iso-parametric, membrane elements
- reinforcement: hollow, iso-parametric four-node membrane elements

In relation to the geometry of the concrete infill ($t_{collumn}$ / $l_{collumn}$) the regular element dimension was fixed to 25mm.

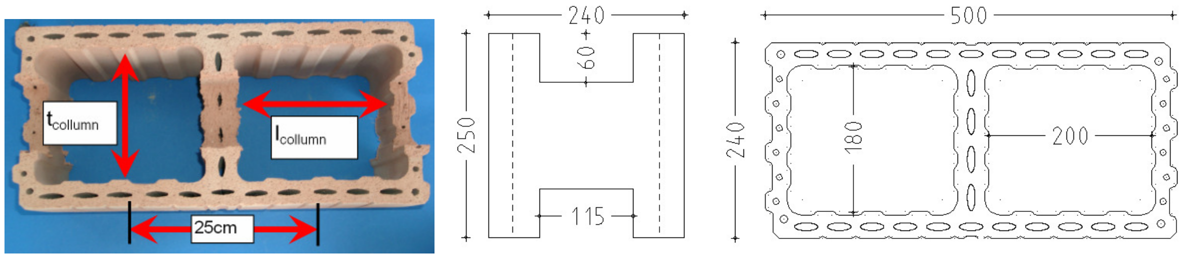

Figure 2. Detailed view on the hollow clay unit and dimensions of the concrete infill

The system can be characterized by vertical columns and horizontal bars. For the load application at the top of the wall and also at the base of the wall

a concrete beam was simulated.

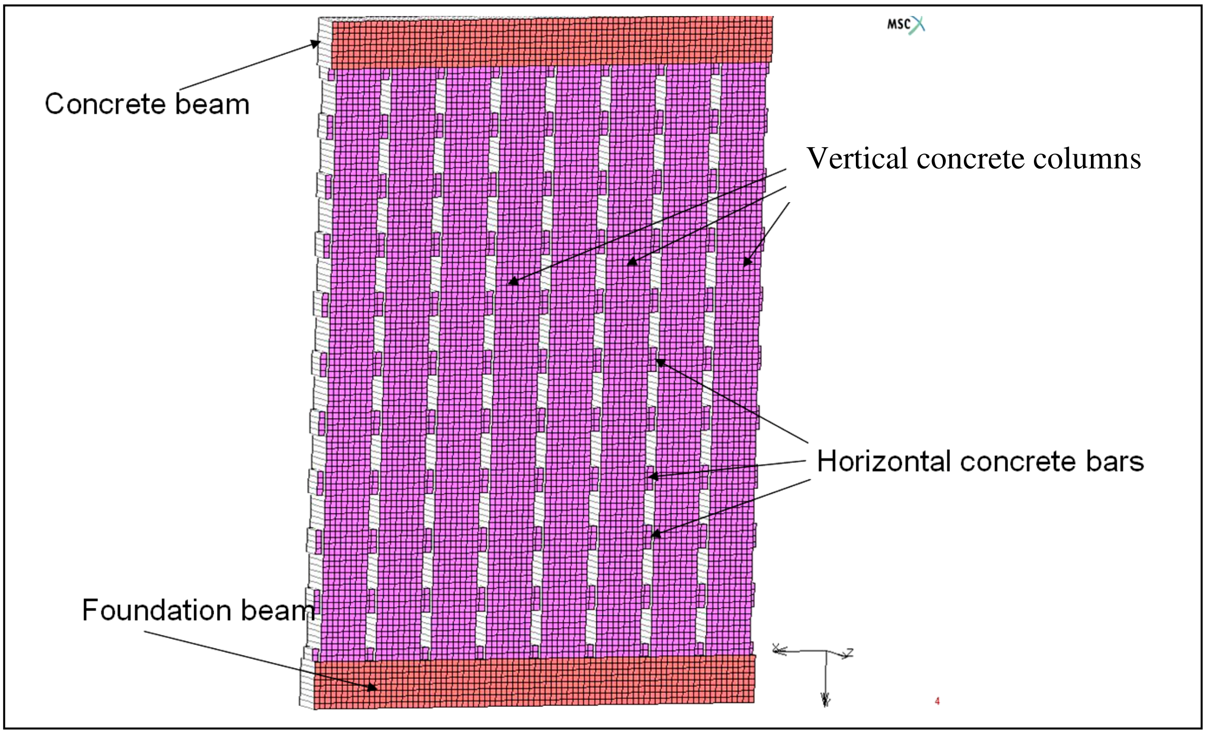

Figure 3. Finite-element-model of the investigated wall

The amount of reinforcement was taken to 4 ∅ 6 (A=113 mm²) in each vertical and horizontal direction (vertical concrete columns / horizontal concrete bars), i.e. # 452 mm²/m.

Material properties for the numerical investigation

The following material properties were chosen (mean values from preliminary testings):

Table 1. Material parameters of the concrete infill (SCC) for the numerical simulation

E-Modulus: E_{33} (=E)	29,340 N/mm²
Compression strength: f_c (=σ_c)	35.2 N/mm²
Tension strength f_t (=σ_{cr})	2.51 N/mm²
Fracture energy: G_F	88.2 Nm/m²

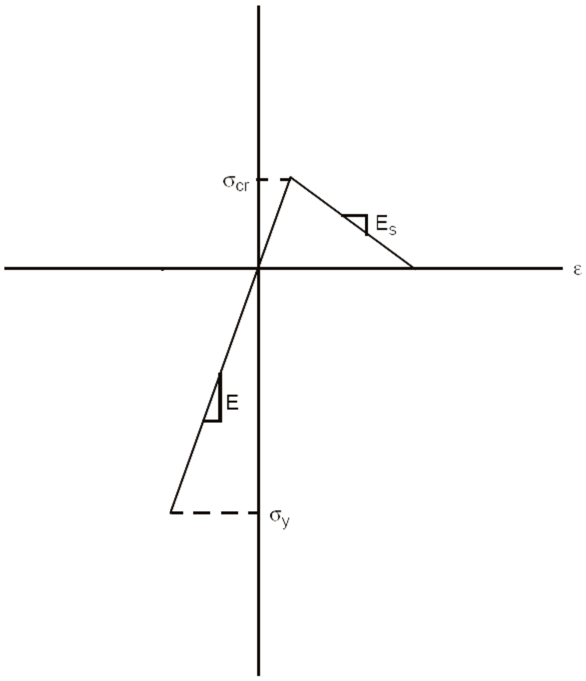

Figure 4. Non-linear stress-strain relation of the concrete infill within the FE-model

The tension softening modulus E_S was calculated from the fracture energy G_F and the type of finite element and the element dimensions. A crushing under compression was not assumed, as the level of stresses under the given conditions is too small.

Table 2. Material parameters of the reinforcement for the numerical simulation

E-Modulus: E	500,000 N/mm²
Yield strength: f_y	500 N/mm²
Ultimate strain: ε_u	0.25 %

For the reinforcement a perfectly plastic behaviour after exceeding the yield strain was assumed.

Parametric study

Within a parametric study

- the geometry (length: 1.0m, 2.0m and 3.0m),
- the boundary conditions (cantilever wall / full restraint) and
- the level of vertical compression stresses

were varied.

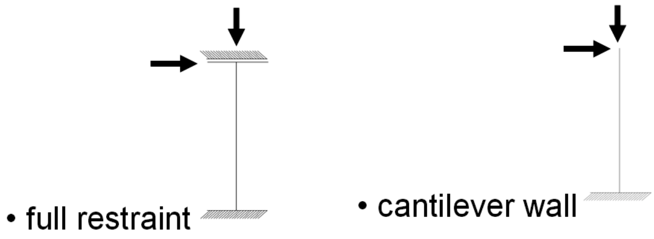

Figure 5. Boundary conditions of the shear walls for the numerical simulation

The material parameters were fixed and taken constantly (s. above). The calculations have been carried out under static-cyclic horizontal loadings where the vertical load remained absolutely constant.

In the following some results of a cantilever wall and a wall with full restraint at the top are shown (height of the wall: 2.5m; length of the wall: 2m; vertical load: N = 1000kN).

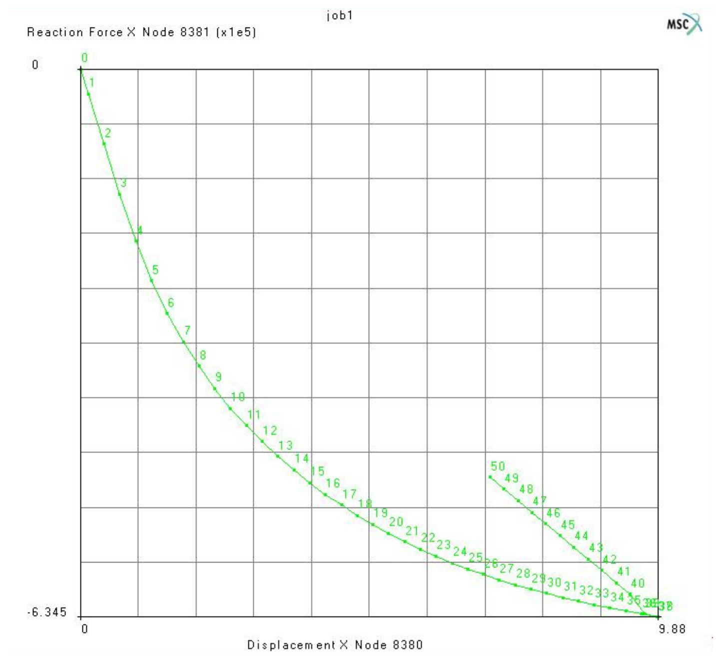

Figure 6. Force-displacement relation (N vs. mm) – cantilever system.

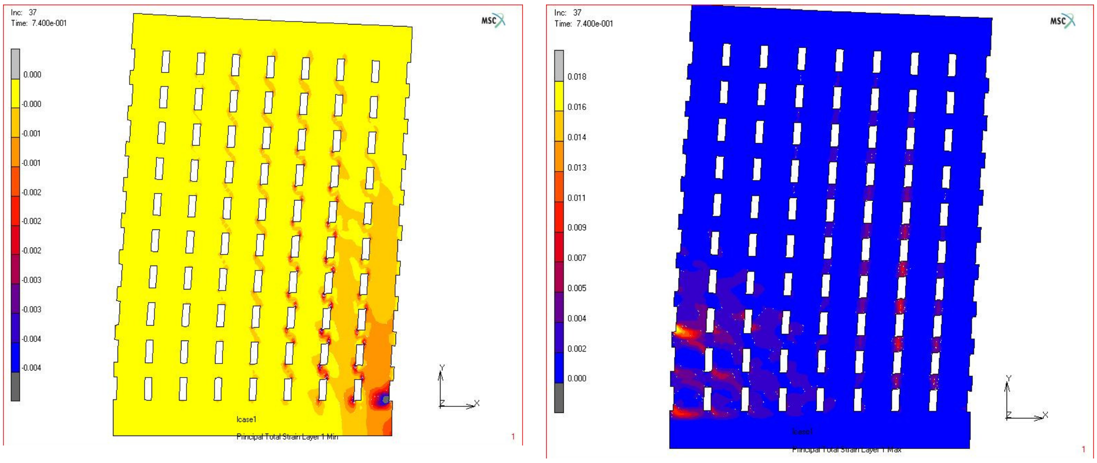

Figure 7. Compression and tension strains – cantilever system.

It is clearly observed, that the dominant bending of the cantilever wall leads to a yielding of the reinforcement (locally high strains, i.e. plastic deformations) and a rotation of the top of the wall. The force-displacement diagram shows a maximum force of about 650kN at 10mm.

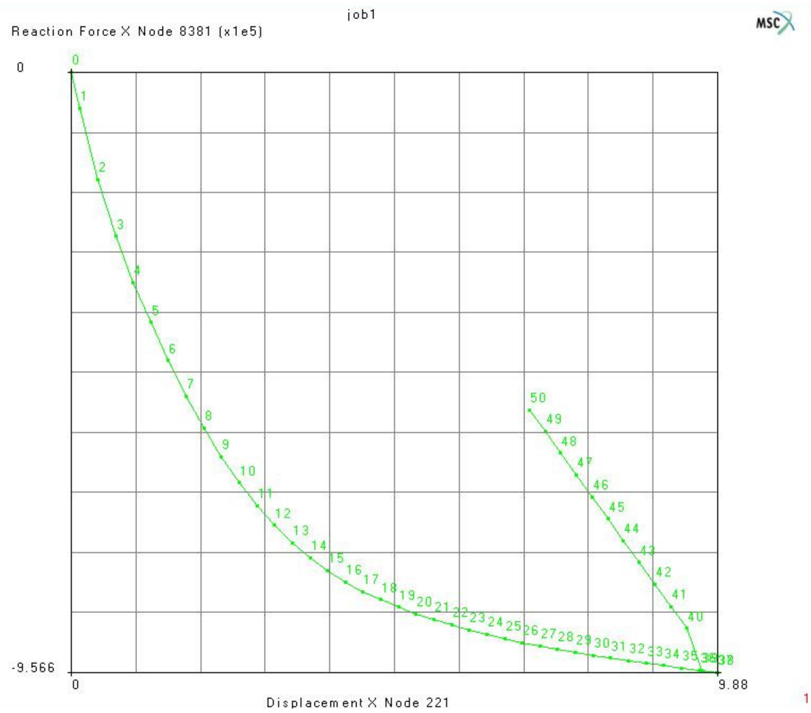

Figure 8. Force-displacement relation (N vs. mm) – full restraint system.

Regarding the strains in the deformed system below it is found, that the dominant shearing of the walls with full rotation restraint at the top leads to a kind of splitting between the vertical columns. The local stress analysis showed that this splitting can be described as a ductile behaviour as the induced forces don't exceed the load bearing capacity of the existing horizontal reinforcement bars.
The force-displacement diagram shows a maximum force of about 980kN at 10mm.

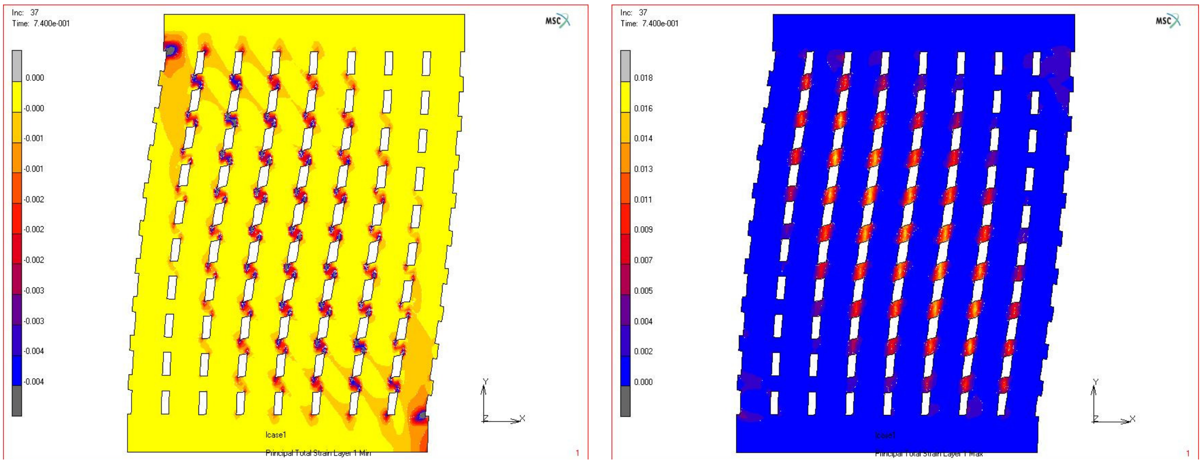
Figure 9. Compression and tension strains – full restraint system.

DESIGN

For the design process in the ULS for the mentioned reinforced masonry system assumptions from RC has been adopted. A parametric study has been carried out where the geometric of the wall, the structural system, and the loadings were varied and the material properties prefixed.
Below the strain and stress distribution of masonry walls under combined in-plane loadings is shown.

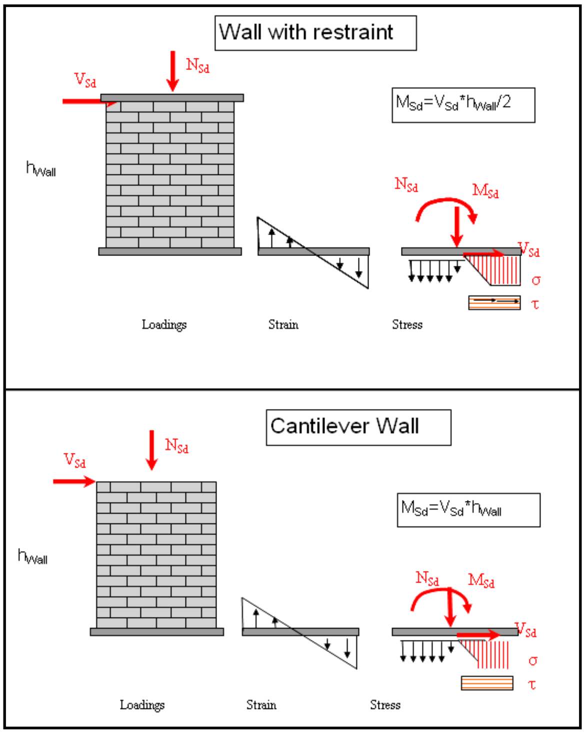

Figure 10. Investigated structural systems with corresponding strain and stress distribution in the relevant cross section at the base of the wall based on the design model.

Design model

The design approach focuses on the consideration of each vertical column *i*, where the vertical strain ε_i and the corresponding stresses are assumed to be constant.

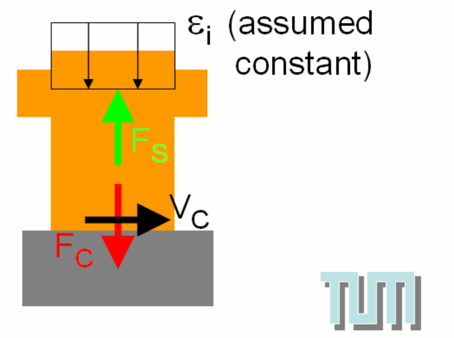

Figure 11. Single column "i" under tension F_S resp. compression F_c force and acting shear force V_c.

In columns with tension strains, no shear component can be taken. The internal vertical tension force F_S is here driven by the effective strain and the given amount of reinforcement.

In columns with compression strains, the allowable shear component V_c is calculated in dependency of the amount of reinforcement and the vertical compression force F_c (s. equations below taken from Eurocode 2).

$$V_{Rd,c} = \left(C_{Rdc} \cdot k \cdot \sqrt[3]{100 \cdot \rho_l \cdot f_{ck}} + k_1 \cdot \sigma_{cp}\right) \cdot b_w \cdot d$$

with a minimum of (1)

$$V_{Rd,c} = \left(v_{\min} + k_1 \cdot \sigma_{cp}\right) \cdot b_w \cdot d$$

Where: $C_{Rdc} = 0.18/\gamma$; $\sigma_{cp} = \dfrac{N_{Ed}}{A_c} \leq f_{cd}$; $\rho_l = \dfrac{A_{sl}}{b_w \cdot d} \leq 0.02$; $k_1 = 0.15$; $k = 1 + \sqrt{\dfrac{200}{d}} \leq 2.0$;

$v_{\min} = 0.035 \cdot k^{2/3} \cdot \sqrt{f_{ck}}$

b_w is the width of the column; d is the effective depth of the cross-section

The allowable compression force F_C also taken from Eurocode 2 with e_{c3} = -1.75‰, e_{c3u} = -3.50‰, γ_C = 1.5 and α_{CC} = 0.85.

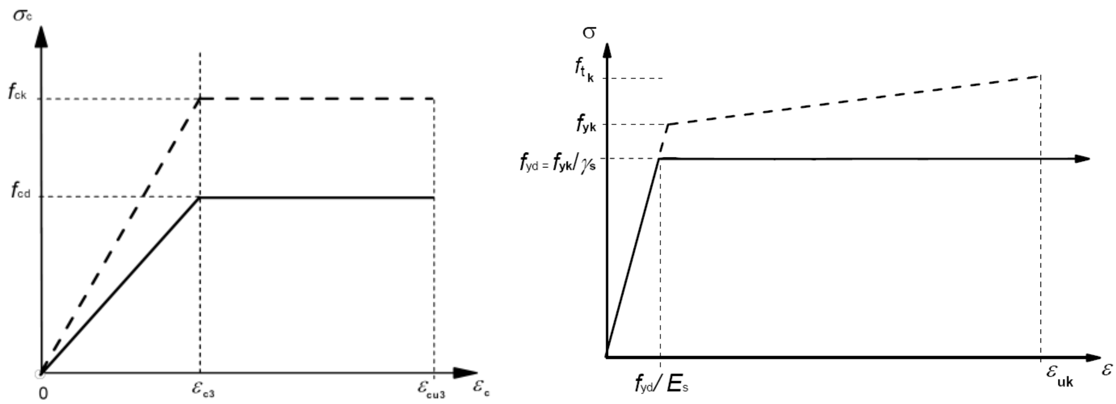

Figure 12. Concrete under compression (left) and reinforcement under tension (right).

For the reinforcement the stress-strain relation under tension was taken to f_{yk} = 500 N/mm², E_S = 200.000 N/mm², e_{uk} = +25‰ and γ_S = 1.15.

The integration over all columns *i* leads to the resulting internal forces N_{Rd}, V_{Rd} and the in-plane bending moment M_{Rd}, in dependency of the given strain distribution along the cross section.

$$N_{Rd} = \Sigma\, N(i) \tag{2}$$

$$V_{Rd} = \Sigma\, V(i) \tag{3}$$

$$M_{Rd} = \Sigma\, N(i)*x(i) \tag{4}$$

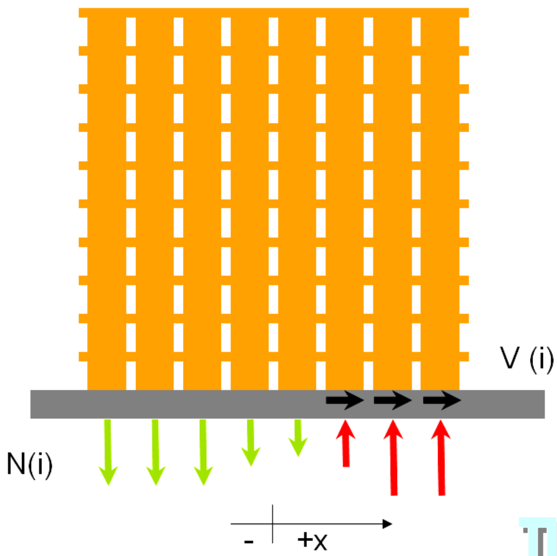

Figure 13. Vertical *N(i)* and horizontal forces *V(i)* in the columns *i* and corresponding *x*-ordinate.

Results

In the following the results of a calculation is shown. In dependency of the vertical compression force N_{Sd} and bending moment M_{Sd} the design shear resistance of the wall V_{Rd} can be checked in the relevant cross section (generally at the base relevant).

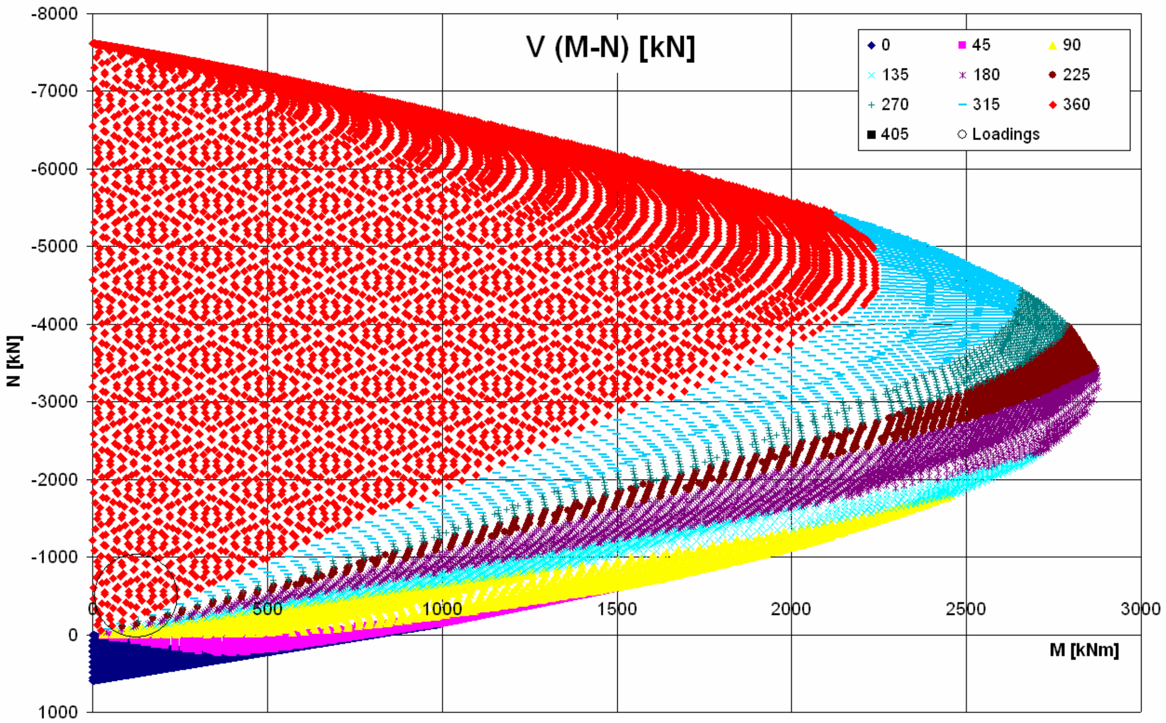

Figure 14. Shear load bearing capacity (V_{Rd}: 0 – 360kN) of a masonry wall in dependency of the acting vertical compression force N_{Sd} and corresponding the in-plane bending moment M_{Sd} (f_{ck}=35N/mm² / l = 3.0m / reinforcement: 113mm² a 250mm horizontal and vertical).

EXPERIMENTAL INVESTIGATIONS

All results have to be checked with the results on full-scale tests. As the experimental investigations will start in autumn 2007, these results will be published on the home-page of the research group (http://diswall.dic.unipd.it/) and at the IBMAC.

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