INTEGRAL MODEL FOR THE IN-PLANE LATERAL-LOAD CAPACITY OF URM (SHEAR) BEARING WALLS AND CALIBRATION WITH TEST RESULTS

T. KRANZLER

Bundesverband
der Deutschen Ziegelindustrie e.V.
Bonn, Germany

C.-A. GRAUBNER

Professor of Civil Engineering
Institute of Concrete and
Masonry Structures
Technische Universität Darmstadt,
Germany

SUMMARY

As one of the project partners of the European research project ESECMaSE (Enhanced Safety and Efficient Construction of Masonry Structures in Europe), it was the aim of the Institute of Concrete Structures and Materials to develop a new approach for the calculation of the in-plane lateral-load capacity of URM (shear) bearing walls. The outcome of a large number of FE-calculations and the comparison with partially modified theoretical design procedures at hand led to an integral approach, which considers the essential influence parameters and gives a good correlation with test results.

INTRODUCTION

Due to a strong increase in the design loads for wind and earthquakes in the European standards, it is evident that the structural design methods for load bearing masonry, particularly with regard to the in-plane lateral-load capacity of URM shear bearing walls will not longer be sufficient in the future. Additionally, the calculated in-plane lateral-load capacity of masonry structures on basis of the current standards seems to underestimate the ones observed in practice. The aim of the European research project ESECMaSE was the derivation of a new approach for the calculation of the in-plane lateral-load capacity of URM (shear) bearing walls.

In a first step, different theoretical models, design procedures, standards and the essential influences on the calculated in-plane lateral-load capacity of URM shear panels have been analysed in detail (see Graubner et al. 2005, Graubner and Kranzler 2005, 2006, 2007 and Kranzler 2008). The different design procedures take into account respective material properties, which are not always determined in a comparable and standardized manner. Finally, the safety requirements for masonry panels differ substantially between the analysed approaches.

The above-mentioned studies showed also that several problems occur and have to be handled well if the bearing capacity of horizontally loaded masonry panels is to be determined realistically. This applies for the structural system of the panels, particularly the ratio of restraint at the top and the bottom of the panel.

Another problem is the consideration of a realistic state of stress in the panel. In literature the stresses and the stress resultants mostly are calculated on basis of the theory of elastic beams

(Navier's theory). The material behaviour is simplified to be homogenous and isotropic and the Bernoulli hypothesis is assumed to be valid (plane sections remain plane). The material and structural inhomogenities, e.g. due to unfilled head joints then have to be considered by a theoretical model for the local shear strength in the decisive cross section. But the question is where the decisive cross section in the panel is and which model for the shear strength should be used there? Most of the theoretical models for the shear strength are derived from the equilibrium at a single unit in the panel centre and assume small units compared to the panel dimensions (e.g. Mann and Müller 1973, or Jaeger and Schöps 2004). In the middle of the panel, the assumption of a diagonal shifted stress block with similar state of stress for adjacent units is justified. But at the bottom and the corner areas of a panel (where the lateral-load capacity is mostly calculated), the resulting state of stress is not the same as the assumed one.
It is obvious that an integral suggestion for the calculation of the in-plane lateral-load capacity of URM (shear) bearing walls is strongly required.

MODELLING OF URM PANELS WITH FEM

General

The FE-investigations have been carried out with the Software Atena 2006. Therefore a micromodel has been chosen with a reduced level of detailing. This means that the mortar in the joints was not modelled explicitly. Instead of that, interface-elements have been used with a respective stiffness. The interface material is based on the Mohr-Coulomb criterion with tension cut off. The constitutive relation for a general two-dimensional case is given in terms of traction on interface planes and relative sliding and opening displacements. If stresses violate this condition, the surface collapses to a residual surface which corresponds to pure (dry) friction. Unfilled head joints (labelled SU) have been considered by non-coupling of the adjacent units. For filled head joints (labelled SV) the above described interface-elements have been used, but only under consideration of friction, i.e. that the cohesion (initial shear strength) in the head joints was neglected in general (adjacent units are in contact).
The units have been assumed as solid and homogenous with non-linear strain-stress-relationship.
By consideration of fracture mechanical parameters it was possible to analyse the behaviour of the masonry panels after the first cracks in the panel occurred.

Calibration of the FE-Software with test results of storey high URM shear panels

Two well documented tests on storey high URM shear panels (taken from Jaeger and Schöps 2004) composed of calcium-silicate units with thin layer mortar and different vertical forces have been used for the calibration of the FE-Software. The main focus was set on the conformance of the experimental and calculated bearing capacity, a matchable crack pattern as well as a congruent load-deformation-curve. The determination of the relevant material parameters of the units, the mortar and the bond behaviour are documented in detail, so that they could be used directly for the calculation. Additionally required parameters have been taken from secondary literature and/or have been assumed. Figure 1 shows the structural test-setup (static-test) which was comparable with the test-setup planned to be used for the static-cyclic-tests within the ESECMaSE project (see Fehling and Stürz 2007) and the respective structural model.

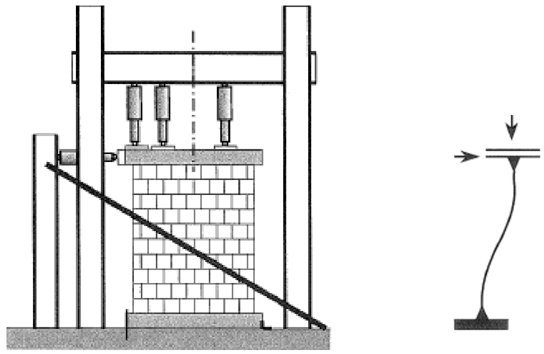


Figure 1. Structural system of the test-setup (Jaeger and Schöps 2004) and structural model which was used to calibrate the FE-Software

Figure 2 shows an acceptable match for the load deformation curves of the experimental tests and the FE-calculation with the chosen software tool. The differences of the ultimate horizontal loads are +10% for panel S1 and -5% for panel S2. The crack patterns also show a good match (see Figure 3).

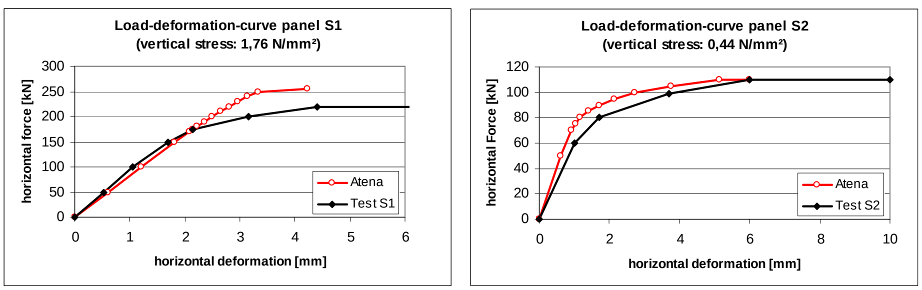


Figure 2. Comparison of the load-deformation curves resulting from tests and the respective FE-calculations

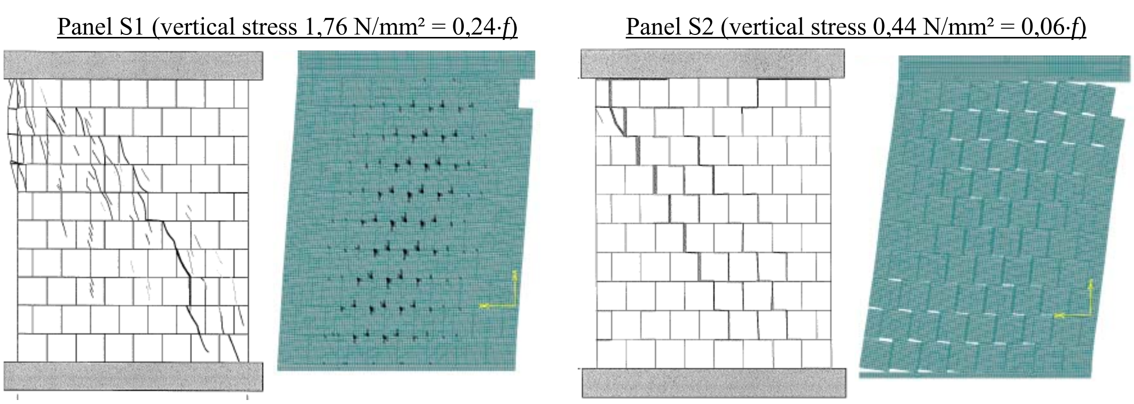
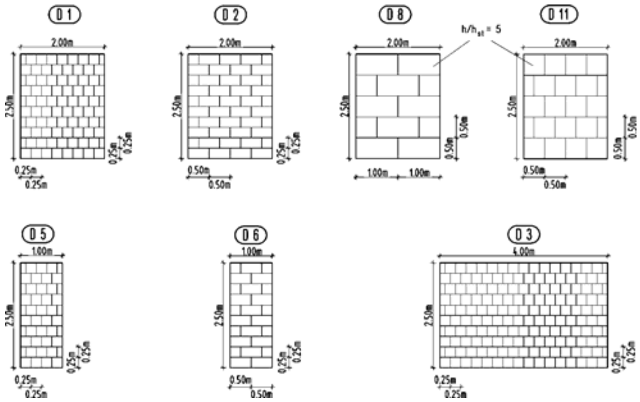


Figure 3. Comparison of crack patterns resulting from tests and the respective FE-calculations parameter study with the calibrated FE-software

Figure 4 gives an overview of the investigated types of panels and the relevant chosen material parameters. By calculating the standardized horizontal bearing capacities of the single panels $v = V/(t \cdot l \cdot f)$ in dependence of the standardized vertical force $n = N/(t \cdot l \cdot f)$ it was also differed between a full restraint at the top and the bottom of the panel (labelled E) and a cantilever system (labelled K). The standardized normal force at the top of the panels has been chosen in a practically relevant range between $0{,}03 < n < 0{,}25$.



Parameter	Value
Compressive strength f	7,3 N/mm²; 12,2 N/mm²
Tensile strength f_t	1,12 N/mm²; 0,2 N/mm²
Starting elastic modulus E	3687 N/mm²
Poisson's ratio υ	0,12
Initial shear strength of bed joints (= cohesion) f_{vo}	0,673 N/mm²
Friction coefficient bed joint (μ)	0,6
Tensile strength perpendicular to the bed joint f_{x1}	0,29 N/mm²
specific weight	0,018 MN/m³
Compressive strain at compressive strength ε_C	5 mm/m
Friction coefficient head joint (μ)	0; 0,6

l,h: length, height of panel
l_{st},h_{st}: length, height of unit
t: depth (thickness) of panel and unit
u: overlapping length of the units

Figure 4. Overview of the investigated panels and main input-parameters for the FE-calculations

Figure 5 shows the results of the FE-calculations in excerpts. The different types of failure were found by analysis of the stress- and crack patterns. They are labelled as follows:

Bd: Bending (flexural failure) due to exceedance of the compressive strength at the bottom of the panel
Kl: Gaping of the single units in the bed joints (Typical crack pattern shown in Figure 3, right)
S: (Diagonal) tensile failure of the units (Typical crack pattern shown in Figure 3, left)

The shear slenderness λ_v is calculated in dependence of the height h and the length l of the panel as follows:

$$\lambda_v = \psi \cdot \frac{h}{l} \tag{1}$$

With: ψ = 0,5 (full restraint at the top and the bottom)
= 1,0 (cantilever system)

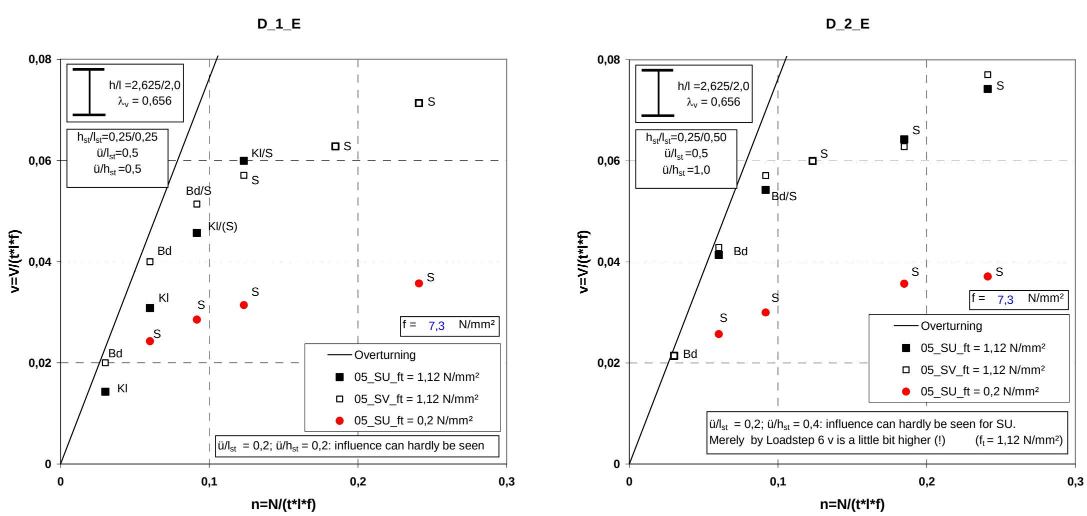


Figure 5. Results of the FE-calculations (panels D_1_E,left and D_2_E, right

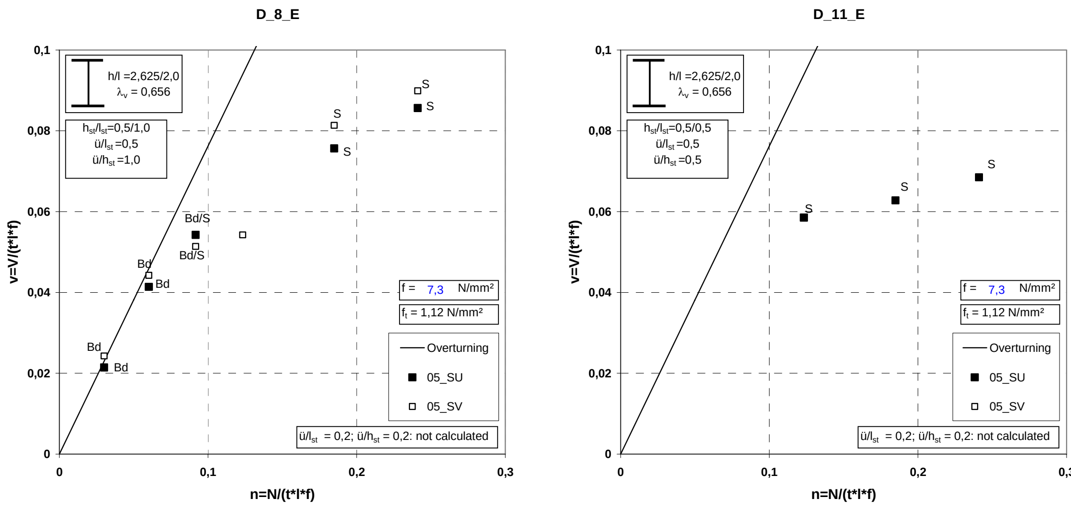


Figure 6. Results of the FE-calculations (panels D_8_E, left and D_11_E, right)

The results of the FE-calculations gave the following main information:

- The theoretical failure criterion “overturning” is not reached, because for low normal forces “bending/diagonal compression failure” or “gaping of the single units” is always decisive.
- Failure due to bending is one of most important failure criteria, in particular for slender panels and/or low normal forces when gaping does not occur.
- With increasing normal forces diagonal tensile failure of the units was observed.
- Whether the head joints are unfilled or whether the adjacent faces of the units are closely abutted together has an significant influence on the criterion “gaping of the single units” and (in case of very compact panels) on the criterion “diagonal tensile failure” for unit geometries $h_{st}/l_{st} \geq 1,0$.
- If the results of the panels D_1_E and D_2_E are compared, (see Figure 5), only a very small influence of the h_{st}/l_{st}-ratio (resp. u/h_{st}) and the execution of the head joints on the failure criterion “diagonal tensile failure” can be detected. In contrast to that, if the units are very big in relation to the panel size, a significant influence of these parameters can be seen if the panels D_8_E and D_11_E are compared (see Figure 6). The bearing capacity of the panel D_8_E is significantly higher than the capacity of D_11_E.
- A significant decrease of the bearing capacity can be seen, if the tensile strength of the material is reduced from 1,12 N/mm² to 0,2 N/mm².
- “Friction (sliding) failure” was not relevant in the FE-calculations as the analysed panels had a quiet large shear slenderness in combination with the very high value for the considered initial shear strength.

INTEGRAL MODEL FOR THE IN-PLANE LATERAL-LOAD CAPACITY

General

The outcome of the FE-calculations was compared with the theoretical design procedures given in literature to find out their limits of applicability. A differentiation between the observed types of failure has been made.

Failure due to bending (flexural failure)

The calculations have shown, that for small normal forces most of the observed bearing capacities can be described/modelled with the equation for bending (flexural) failure by assuming a stress block for the strain-stress-relationship (equation (2)), notwithstanding the fact, that the observed failure mode often looks like diagonal tensile failure. It strongly depends on the shear slenderness λ_v and will mainly become decisive for tall specimens and/or panels with low vertical loads. The overlapping length, the type of the head joints (filled or unfilled) and the size of the units compared to the size of the panel have no influence on the load-bearing capacity, if this criterion governs.

$$v_{bending} = \frac{1}{2 \cdot \lambda_v} \cdot \left(n - n^2\right) \tag{2}$$

Failure due to gaping of the single units

This failure mode only occurs for masonry with unfilled head joints. Because of the occurring stress peaks near the head joints, which lead to a successive failure due to exceeding of the tensile strength perpendicular to the bed joints, this material parameter is not considered in equation (3). Furthermore this parameter is a very uncertain value, particularly in case of alternating loads. With the approach it is possible, to describe the already known positive effect of large (high) units in relation to the panel height. Equation (3) is independent of any material parameter and the shear slenderness. For masonry with filled head joints, or adjacent faces of the masonry units closely abutted together, gaping does not occur.

$$v_{gaping} = \frac{1}{2} \cdot \left(\frac{1}{h_{st}} + \frac{1}{h}\right) \cdot l_{st} \cdot n \tag{3}$$

Friction failure (sliding)

The investigations have shown, that even if a local failure due to friction in the middle of the panel appeared and if no diagonal tensile failure occurred, the minimum horizontal load could be increased up to

$$v_{friction} = \mu \cdot n \tag{4}$$

Equation (4) is independent from the construction of the head joints and the overlapping length u. How far additional initial shear strength (cohesion) can be considered was not found out. Because of the alternating horizontal loads the consideration of the cohesion has to be questioned anyway. If the masonry panel has some sort of damp proof course or slip joint, this fact has to be considered by using a respective friction coefficient in this joint.

Diagonal Tensile failure

Two theoretical models for calculating the shear strength have been analysed in detail. Therefore it was necessary for both, to compare their assumptions for the state of stress at the single unit and to answer the question, where (and how) in the panel these formulations should be used. By analysing the decisive unit it was found, that after a modification of an

equation, in the original given by (Jaeger and Schöps 2004), acceptable results were received, in particular if an uncracked cross section is assumed.

$$v_{tensile} = \frac{1}{c} \cdot \overline{f_{bt}} \cdot \left(F^*\right)^{-2} \cdot \left(\sqrt{1 + \left(F^*\right)^2 \cdot \left(1 + \frac{n}{\overline{f_{bt}}}\right)} - 1 \right) \quad \text{if } h/h_{st} > 5 \qquad (5)$$

$$v_{tensile} = \frac{1}{c} \cdot \overline{f_{bt}} \cdot \left(2 \cdot F^* \cdot \left(\frac{u}{h_{st}}\right)\right)^{-2} \cdot \left(\sqrt{1 + \left(2 \cdot F^* \cdot \left(\frac{u}{h_{st}}\right)\right)^2 \cdot \left(1 + \frac{n}{\overline{f_{bt}}}\right)} - 1 \right) \quad \text{if } h/h_{st} \leq 5 \qquad (6)$$

With: $c = 1{,}0 \leq 0{,}5 + \lambda_v \leq 1{,}5$

$\overline{f_{bt}} = f_{bt} / f$

$F^* = F \cdot \left(0{,}63 + 0{,}45 \cdot f_{bt}\right)$ [f_{bt} in N/mm²]

F = 2,0 (thin layer mortar)

= 1,7 (general purposed mortar)

An influence of different u/h_{st}-ratios was only observed in case of small h/h_{st}-ratios (D_8 and D_11, see Figure 6). In these cases equation (6) has to be used instead of equation (5).

<u>Failure of the corner unit due to tension triggered by friction failure in the centre of the panel</u>

Further calculations show an additional failure type (see Figure 7). In this case a local sliding begins in the middle of the wall. This leads to a rearrangement of the principal stresses to the corner of the panel. In case of failure, either the maximum load according to equation (4) (global friction failure) was reached, or the unit in the corner collapses because of exceeding of its tensile strength. If neither friction (sliding) failure nor failure of the unit in the corner occurs, the horizontal load can be increased up to the maximum value according to bending failure. The investigations have shown that it was not necessary to develop an additional equation to describe the horizontal bearing capacity for this failure criterion because the respective capacities are already covered by the equations (2), (3), (4) and (5).

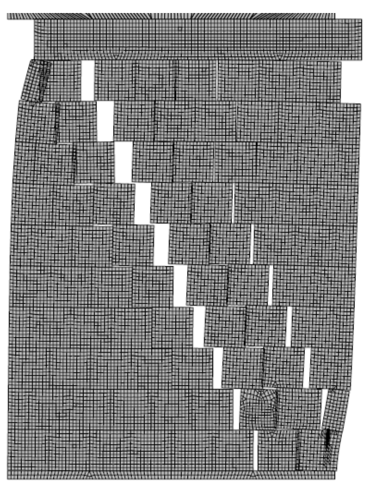


Figure 7. Crack pattern for failure of the corner unit triggered by friction failure in the centre

VERIFICATION AND CALIBRATION WITH TEST RESULTS

Comparison of test results with the uncalibrated approach

The developed suggestion for the calculation of the in-plane bearing capacity of URM shear panels (minimum of the equations (2), (3), (4) and (5)) have been compared with the maximum horizontal load determined in tests on storey high masonry panels. Additionally to the aim to recalculate these loads, it was checked, whether the decisive type of failure according to the calculation corresponds to the observed failure modes in the test. Since no test results of panels made with large units ($h/h_{st} \leq 5$) were available, equation (6) could not verified.

Considering the fact, that only the horizontal tensile strength of the units was at hand in most cases, the comparison and the calibration of the model with the test results of solid units (calcium-silicate and autoclaved-aerated-concrete units) has been carried out using the horizontal tensile strength of the units $f_{bt,hor}$. For these types of units, the difference between the horizontal tensile strength $f_{bt,hor}$ and the actually required diagonal tensile strength $f_{bt,\alpha}$ is negligible. In case of the high perforated clay units the actually required diagonal tensile strength is significantly higher than the horizontal tensile strength. Taking the horizontal tensile strength into account would lead to a significant underestimation of the test results. Furthermore, the horizontal tensile strength is a large scattering parameter in case of perforated clay units. That is the reason why it was suggested by Zilch and Grabowski 2005 to apply the splitting tensile strength perpendicular to the bed joints $f_{bt,sp}$ in equation (5).

If the necessary material parameters were not determined within the respective research reports, they have been assumed realistically.

Figure 8 shows the comparison of the maximum in-plane bearing capacity determined in tests on storey high URM shear panels (v_{obs}) with the capacities on basis of the uncalibrated approach (v_{cal}). Values v_{cal}/v_{obs} larger than 1,0 are an overestimation of the test results, whereas values smaller than 1,0 yield an underestimation. For masonry panels made of calcium-silicate units, the uncalibrated approach matches the test results very well (Mean value $(v_{cal}/v_{obs})_m$ = 0,98, coefficient of variation COV = 0,12). Taking the theoretical model of Mann/Müller into account (design at the bottom of the panel) would lead to a significant underestimation of the test results on masonry made of calcium-silicate units ($(v_{cal}/v_{obs})_m$ = 0,79, COV = 0,26). In case of panels made of clay units the approach shows an acceptable match, together with a slight underestimation of the bearing capacities ($(v_{cal}/v_{obs})_m$ = 0,87, COV = 0,16). Anyhow, the uncalibrated approach leads to higher (and more realistic) bearing capacities than a calculation on basis of the theoretical model of Mann/Müller ($(v_{cal}/v_{obs})_m$ = 0,76, COV = 0,17). In case of autoclaved aerated concrete units, the uncalibrated approach leads to a similar scatter of the values ($(v_{cal}/v_{obs})_m$ = 1,03, COV = 0,25) as a calculation on basis of the theoretical model of Mann/Müller ($(v_{cal}/v_{obs})_m$ = 1,00, COV = 0,25). The greater number of unshaded marks show that the theoretically decisive failure mode and the observed ones in the tests do not correspond. For autoclaved-aerated-concrete units the uncalibrated approach shows a strong overestimation in particular for the criterion “diagonal tensile failure”. Nevertheless, the uncalibrated approach already matches better with the test results ($(v_{cal}/v_{obs})_m$ = 0,94, COV = 0,21) than a calculation on basis on the theoretical model of Mann/Müller at the bottom of the panels ($(v_{cal}/v_{obs})_m$ = 0,84, COV = 0,26).

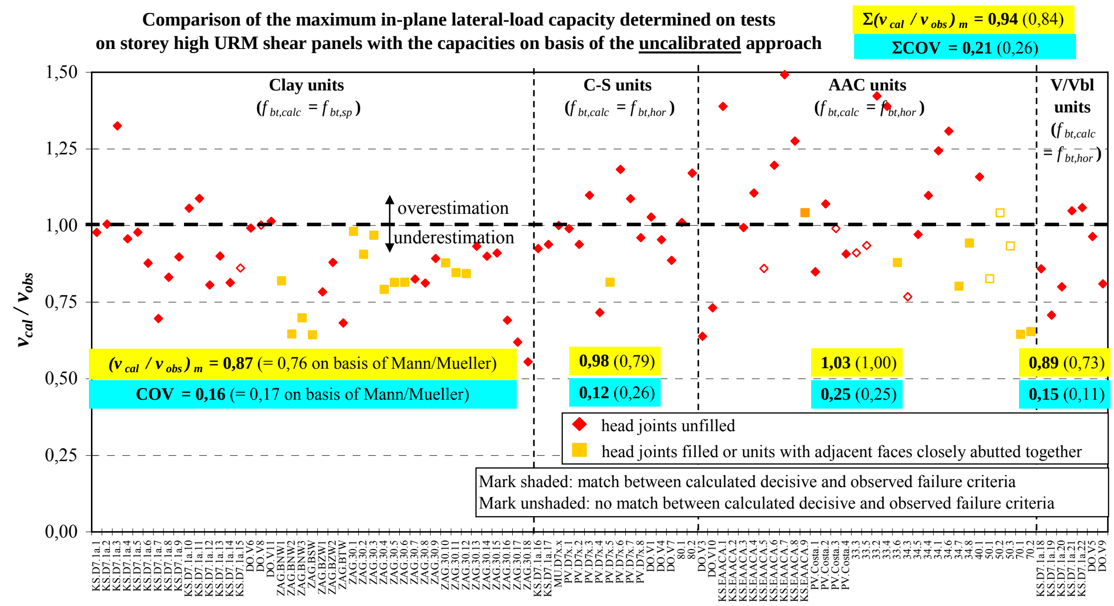


Figure 8. Comparison of the maximum in-plane bearing capacity determined on basis of tests on storey high URM shear panels with the capacities on basis of the uncalibrated approach

<u>Final approach after calibration on test results</u>

Analysing the test results and their comparison with the uncalibrated approach firstly leads to the conclusion that the thickness of the bed joints does not have the assumed effect on the in-plane-bearing capacity. Considering all test data, the approach for diagonal tensile failure performs best with $F = 1{,}9$, independently from the thickness of the bed joints.

The criterion diagonal tensile failure in case of panels made of clay-, calcium-silicate- or concrete units should only be taken into account if the shear slenderness of the panel is smaller than $\lambda_v \leq 1{,}5$. The bearing capacity may be calculated from:

$$v_{tensile} = \frac{1}{c} \cdot \overline{f_{bt,cal}} \cdot \left(F^*\right)^{-2} \cdot \left(\sqrt{1 + \left(F^*\right)^2 \cdot \left(1 + \frac{n}{\overline{f_{bt,cal}}}\right)} - 1\right) \tag{7}$$

With: $\overline{f_{bt,cal}} = f_{bt,cal} / f$

$f_{bt,cal} = f_{bt,sp}$ (tensile splitting strength for perforated units with percentage of perforation > 25%)

$= f_{bt,hor}$ (uniaxial (horizontal) strength for solid units with percentage of perforation ≤ 25%)

$$F^* = 1{,}9 \cdot \left(0{,}63 + 0{,}45 \cdot f_{bt,cal}\right) \approx 1{,}20 + 0{,}85 \cdot f_{bt,cal}$$ [f_{bt} in N/mm²]

For panels made of autoclaved aerated concrete units equation (7) has to be modified. Equation (8) has to be considered only if the shear slenderness of the panel is smaller than $\lambda_v \leq 1{,}0$:

$$v_{tensile} = \frac{1}{c} \cdot \overline{f_{bt,hor}} \cdot 2 \cdot \left(F^*\right)^{-2} \cdot \left(\sqrt{1 + \frac{\left(F^*\right)^2}{4} \cdot \left(1 + \frac{n}{\overline{f_{bt,hor}}}\right)} - 1\right) \tag{8}$$

Figure 9 shows the comparison of the final calibrated approach with the test results. The final approach matches the test results of all kinds of units very well with a relatively low coefficient of variation (COV = 0,16).

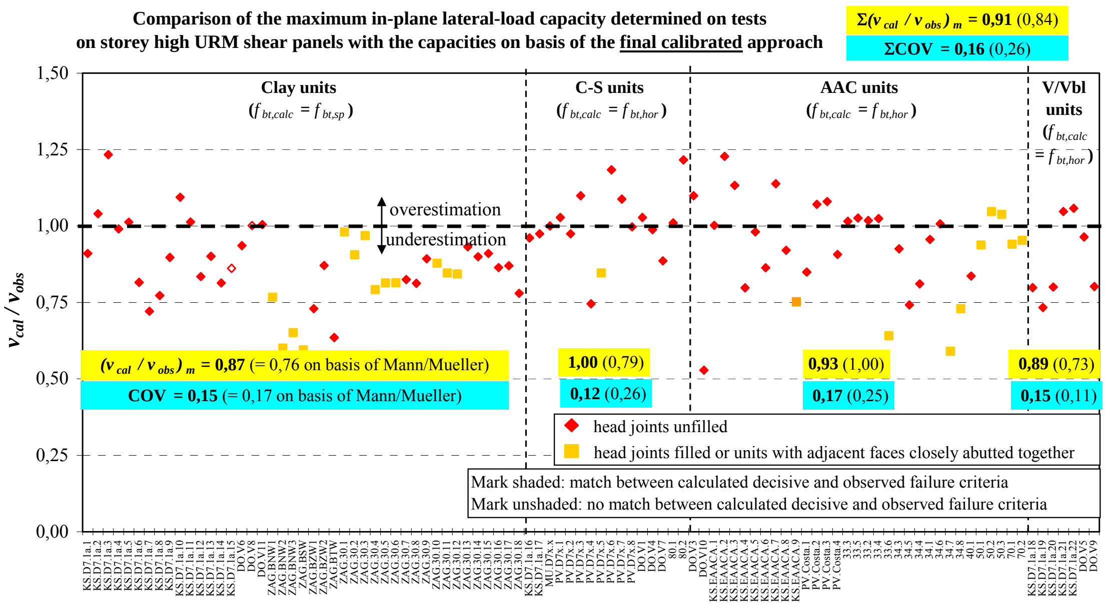


Figure 9. Comparison of the maximum in-plane bearing capacity determined on test on storey high URM shear panels with the capacities on basis of the final calibrated approach

CONCLUSIONS

The new integral approach performs very well, especially after calibration. The model works without consideration of the uncertain parameters tensile strength perpendicular to the bed joints and initial shear strength (cohesion). The investigations have also shown, that some of the developed equations are valid for all types of units (bending and gaping), whereas others (friction and tensile) should be modified in terms of the types of units to get better results.

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