COMPOSITE LOAD BEARING BEHAVIOUR OF UNREINFORCED AND REINFORCED WALL CONSTRUCTIONS MADE OF CLAY UNITS FILLED WITH STANDARD CONCRETE

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ABSTRACT

Masonry walls which are exposed to wind loads or earthquake action must have a sufficient shear strength, and, if necessary, also flexural strength in the direction of the shear walls. Recently, the required safety against structural failure due to wind loads or earthquake action has been tightened in the European and German standards. Consequently, the verification of sufficient load bearing capacity is only possible if a larger shear strength can be taken into account.

The massive type of construction consisting of clay units filled with concrete is in Germany presently mainly used for the manufacturing of walls with a high density and hence with good sound insulation properties. Since neither compressive nor shear strength play a major role in this context, this construction method has so far only been admitted for use in Germany either as masonry made of clay units filled with standard concrete (in this case only the unit cross section may be applied for load transfer) or as clay formwork units for unreinforced or reinforced concrete columns (here only the concrete cross section may be taken into account). Up to now, according to the German standards DIN 1053-1 and DIN 1045-1 as well as the National Technical Approvals, the considerable advantages of a composite system consisting of concrete and clay unit may not be taken into account for the structural analysis of masonry because the required basic investigations have been missing so far.

It was the aim of this research project to develop a model for the compressive and shear design of walls made of clay units filled with unreinforced standard concrete and clay formwork units with and without reinforcement. In doing so, the concrete cross sections and the clay unit cross sections should be considered as composite section. To determine the load bearing capacity of the composite system, extensive experimental tests and numerical calculations on the basis of Finite Element Methods were performed.

Keywords: Masonry, composite system, filled clay units, clay formwork units, Finite Element Method
INTRODUCTION
As a result of the increased design loads regarding the impacts on buildings it has become necessary to develop masonry with an increased load bearing capacity and to use existing load bearing reserves. A suitable approach is the presently applied method using clay units for which a National Technical Approval has been required up to now in Germany. Clay units have larger cavities which have so far been filled with mortar or flowable standard concrete. A vertical reinforcement can be placed in the cavities of the clay units. Furthermore, it is possible to place a horizontal reinforcement through recesses in the webs and shells of the clay units.

When designing building components made of clay units for concrete infill, presently either the load bearing capacity of the unfilled unit or that of the filling concrete may be applied depending on the respective National Technical Approval. A design taking into account the composite load bearing behaviour is not possible.

It was the aim of the research project to develop approaches for the compression and shear design of walls made of clay units filled with concrete with and without reinforcement which allow for the use of possible load bearing reserves due to the composite load bearing effect of clay units and concrete. For this purpose, extensive theoretic and experimental investigations were performed.

TEST PROGRAMME AND USED MATERIALS
The examinations were conducted on the following three clay units:
- Unit type A: clay unit for unreinforced concrete infill
- Unit type B: clay formwork unit for unreinforced or reinforced concrete infill
- Unit type C: DISWall clay formwork unit for unreinforced or reinforced concrete infill

The used clay units are shown in Figure 1. The dimensions of the units are approx. 500 mm x 240 mm x 249 mm (unit length x unit width x unit height).

![Figure 1: Used masonry units](image-url)
As filling concretes, a standard concrete (SC) which was compacted by poking as well as a Self-Compacting Concrete (SCC) whose properties were adjusted to be used in contact with the sucking material of the clay units, were used.

The load bearing behaviour of the composite system hollow clay unit/concrete under compressive and shear load was characterized applying FE-models and also analytic approaches. When modelling the building components, the single clay units filled with concrete were modelled in a smeared way. At first, effective characteristic values of the filled clay units had to be calculationally determined. Extensive experimental and numerical investigations were performed on single clay units filled with concrete to determine these effective characteristic values. Micro-models were developed in which the webs and shells of the clay units and the concrete were modelled in an FE-model. Prior to this, the properties of the single materials and of the bond had been experimentally determined. Applying the developed micro-models, the compression and tensile tests on the filled single clay units were numerically simulated. The investigations conducted to determine the characteristic properties of the single building materials clay unit and concrete are summarized in Brameshuber (2010). In the following, the investigations on the compressive load bearing and the fatigue behaviour as well as on the shear load bearing behaviour are presented.

INVESTIGATIONS ON THE COMPRESSIONAL LOAD BEARING BEHAVIOUR

The compressive load bearing behaviour was investigated on altogether 18 unreinforced and reinforced floor-to-ceiling wall specimens made of clay units filled with concrete. With one exception – see below – the walls were built using a customary thin layer mortar. The tests were performed on unit types A and B in combination with the standard concrete (SC) and the Self-Compacting Concrete (SCC). At walls made of unit type A and standard concrete (A/SC), the influence of the bed joint mortar on the compressive load bearing behaviour was additionally examined. For this purpose, the clay units of one series were dry stacked (A/SC/dry stacked). Unit type C was exclusively examined combined with SCC.

In three test series of the material combinations B/SC and C/SCC the walls were reinforced. On the one hand, a new developed reinforcement system (DISWall) was used which is built in layer by layer into the clay unit cavities and which consists of four ribbed steel bars (Ø 6 mm) every 130 mm in horizontal and vertical direction. In the cavities, the reinforcement runs in vertical direction and is embedded into the floor slab at the top and the bottom of the wall. Recesses in the masonry units are required for the horizontal reinforcement bars. On the other hand, a conventional reinforcement made of stirrup steel bar – 4Ø12 mm vertically in each cavity with stirrups Ø6 mm every 12.5 cm as well as a horizontal bar Ø14 mm in each masonry unit layer – was used (C/SCC/stirrup).

The test setup to determine the masonry compressive strength is illustrated in Figure 2. At the time of the wall tests, the concrete compressive strength of the cubes with an edge length of 150 mm manufactured in parallel to the wall was determined.
The compressive stress-strain curves of the walls with Self-Compacting Concrete have an almost similar course with the exception of Wall 2 at the combination C/SCC, Figure 3a). The walls with the standard concrete compacted by poking combined with the three examined clay units are clearly different. On the one hand, this is due to the partly lower standard compressive strength of the concrete, presumably however also to an inferior compaction of the standard concrete by poking. Even at investigations on reinforced walls, the strength and stiffness in the test series with standard concrete were lower than at the walls with SCC, see Figure 3 b).

The stress-strain curves of the unreinforced and reinforced walls made of unit type C with SCC only differ slightly, cf. Figures 3 a) and 3 b). The mean compressive strength of the walls could be increased by the new developed reinforcement system (DISWall) by approx. 5 % and by the stirrup steel bar reinforcement by approx. 23 % as compared to the unreinforced walls. Considering the lower concrete compressive strength of the reinforced walls, a larger influence of the reinforcement on the total load bearing capacity can be assumed. A comparable influence of the reinforcement on the load bearing capacity also
occurred at the walls made of unit type B with standard concrete. The comparison of tests series A/SC and A/SC/dry stacked shows no significant influence of the bed joint mortar on the total load bearing behaviour. The larger scattering of the results of test series A/SC/dry stacked is not necessarily due to the unmortared bed joints.

The investigations on the compressive load bearing behaviour showed that the loading capacity is decisively influenced by the concrete properties. Table 1 gives a survey of the results of the single test series on unreinforced walls and pillars. The cube compressive strength of the concrete and the results of the wall and pillar tests are stated as mean values. At the wall tests, the specimens had a slenderness of $\lambda = 10$, at the pillar tests of approx. $\lambda = 5$. According to Mann (1983), the factor 1.1 can be applied for masonry walls to convert between the slenderness $\lambda = 5$ and $\lambda = 10$. The calculated masonry compressive strength $\beta_{D,\text{mw, } \lambda=5}$ at a slenderness of $\lambda = 5$ as well as the ratio $\beta_{D,\text{mw, } \lambda=5} / \beta_{D,\text{concrete}}$ are also listed in Table 1.

**Table 1: Test results (mean values)**

<table>
<thead>
<tr>
<th>Unit type</th>
<th>Concrete</th>
<th>Specimen</th>
<th>$\beta_{D,\text{concrete}}$</th>
<th>$\beta_{D,\text{mw, } \lambda=10}$</th>
<th>$\beta_{D,\text{mw, } \lambda=5}$</th>
<th>$\beta_{D,\text{mw, } \lambda=5} / \beta_{D,\text{concrete}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>SC</td>
<td>Column</td>
<td>43.3</td>
<td>-</td>
<td>23.7</td>
<td>0.55</td>
</tr>
<tr>
<td>A</td>
<td>SC</td>
<td>Wall</td>
<td>35.3</td>
<td>16.5</td>
<td>18.2$^{1)}$</td>
<td>0.52</td>
</tr>
<tr>
<td>B</td>
<td>SC</td>
<td>Wall</td>
<td>44.2</td>
<td>18.9</td>
<td>20.8$^{1)}$</td>
<td>0.47</td>
</tr>
<tr>
<td>A</td>
<td>SCC</td>
<td>Wall</td>
<td>44.5</td>
<td>22.6</td>
<td>24.9$^{1)}$</td>
<td>0.56</td>
</tr>
<tr>
<td>B</td>
<td>SCC</td>
<td>Wall</td>
<td>37.0</td>
<td>23.8</td>
<td>26.2$^{1)}$</td>
<td>0.71</td>
</tr>
<tr>
<td>C</td>
<td>SCC</td>
<td>Wall</td>
<td>43.8</td>
<td>21.1</td>
<td>23.2$^{1)}$</td>
<td>0.53</td>
</tr>
</tbody>
</table>

1)$^{1)}$ calculated values

The ratio $\beta_{D,\text{mw, } \lambda=5} / \beta_{D,\text{concrete}}$ is approx. 0.54 except for the tests with unit type B. The values of test series B/SC fall below this value and those of test series B/SCC clearly exceed it. Basically, it can be assumed that the load bearing capacity of the walls with standard concrete is lower than with SCC because of the inferior compaction by poking.

The calculational assessment of the uniaxial compressive strength of the composite section at a slenderness of $\lambda = 5$ can be made at first considering the concrete cross section and the uniaxial concrete compressive strength. To convert the cube compressive strength (edge length 150 mm) into the cylinder compressive strength ($\lambda = 2$, $\varnothing$ 150 mm), which can approximately be seen as uniaxial concrete compressive strength, the factor 1.2 is applied. The result is the following equation:

$$\beta_{D,\text{mw, } \lambda=5} = \frac{A_C}{1.2 \cdot A_{\text{total}}} \cdot \beta_{D,\text{concrete}}$$

(A) concrete cross section

(A) total wall cross section

$\beta_{D,\text{concrete}}$ concrete compressive strength (cube, edge length 150 mm)
At a ratio concrete/total cross section of about 60%, a ratio of $\beta_{D,\text{mww},\lambda=5}$ / $\beta_{D,\text{concrete}} = 0.5$ results for the tested wall cross sections. This value characterizes the test results of the unreinforced walls in a sufficiently correct way and lies on the safe side except for test series B/SC.

**INVESTIGATIONS ON THE BEHAVIOUR UNDER SUSTAINED LOADS**

To characterize the load bearing and deformation behaviour of the composite building material under permanent load, tests were performed on six small masonry pillars, see Figure 4. For the examinations, unit type A was chosen combined with standard concrete since larger creep deformations were expected at the standard concrete. Moreover, at unit type A, there is no bond between the single concrete pillars so that the most unfavourable material combination could be assumed. At first, the short-term strength was determined on three pillars. The behaviour under different test loads was tested on three further specimens. One pillar was loaded with approx. 50%, the other with approx. 85% and the last one with approx. 90% of the mean short-term strength.

In Figure 5, the creep deformations of the specimen loaded with 20 N/mm² (approx. 85% of the short-term compressive strength) are illustrated for the first nine days under load. A crack formation and a local spalling of the clay unit shell could be observed two times during this period. The measuring points Z2 and Z3 showed a significant increase in deformations which can be ascribed to these failure processes. These failure processes increased in the course of the further testing time, the specimen however did not fail. After about 100 days under load, the pillar was loaded until failure. The result was a compressive strength of 26 N/mm² which ranges slightly above the mean short-term compressive strength presumably because of the post-hardening of the concrete. The compressive strength of the standard concrete of this specimen determined on cubes with an edge length of 150 mm at the time of the first loading of the pillar at the age of 63 days amounted to approx. 51 N/mm². Thus, the permanent load was approx. 40% of the concrete compressive strength at the time of the first loading.
masonry pillar under lower load remained free of damage whereas that under higher load showed increasingly serious damage patterns. Finally, it failed under permanent load.

![Graph](image)

**Figure 5**: Time-dependent creep deformations, pillar fatigue test

**INVESTIGATIONS ON THE SHEAR LOAD BEARING BEHAVIOUR**

The experimental investigations were conducted at the IZF Brick and Tile Research Institute Essen Regd. Four test series with three floor-to-ceiling wall specimens, each, were performed. In the tests, exclusively standard concrete was used since it was assumed that it led to a lower shear load bearing capacity due to its compressive strength and that thus the results lie on the safe side. As masonry units, unit types A and C were used. The test setup is displayed in Figure 6.

![Drawing of the test stand](image)

**Figure 6**: Shear tests on wall specimens

Always one day before manufacturing the wall specimens, the first unit layer was laid on the lower load transfer beam in a compensating layer made of standard concrete. The other unit layers were made using the thin layer mortar already applied at the investigations on the compressive load bearing behaviour. After a sufficient hardening time of the thin layer mortar, the concrete was placed and compacted by poking. All tests were performed in a displacement controlled way with an increase in the horizontal displacement of the wall top
by 1 mm/min. To ensure this, a horizontal displacement transducer was installed at the wall top which controlled and, if necessary, adjusted the piston travel of the horizontal cylinder. Other inductive displacement transducers installed at the test setup as well as at the specimen measured the respective deformations in order to take into account possibly occurring deformations of the test stand itself.

To be able to assess the composite load bearing behaviour it was necessary at first to make fundamental statements regarding the shear load bearing behaviour of the walls without concrete filling and thus to provide reference values. Therefore, in the first test series unfilled clay unit walls made of unit type A were investigated. Here, the opportunity arose to additionally examine the influence of the superimposed load on the shear load bearing capacity. In these three tests the superimposed load was 10 %, 30 % and 100 % of $\sigma_0$. The wall length was not varied and amounted to 2500 mm in all three tests. As was to be expected, the shear load bearing capacity increases and the maximum displacement of the walltop decreases with increasing superimposed load. At the lower superimposed load levels ($0.1\cdot\sigma_0$ and $0.3\cdot\sigma_0$) the specimen joints failed and under high superimposed load a tensile failure of the clay unit could be observed. In all cases, the test specimens leave the linear-elastic zone after about two third of the maximum load.

To investigate the influence of the wall length of filled clay masonry, in the second test series walls made of unit type A filled with standard concrete with the lengths of 1300, 2000 and 2500 mm were tested. In all tests, the superimposed load was $0.3\cdot\sigma_0$. The tests had been interrupted at a horizontal load of about 450 kN to avoid a failure of the testing equipment and not to endanger persons. It can be determined that the shear load bearing capacity increases by a multiple (at least by factor 4) and the displacements at the wall top amount to only about 25 % of the deformations of the unfilled walls.

The third test series conformed to the previous only that unit type C was used instead of unit type A. Also here, the tests had to be interrupted at a horizontal load of about 450 kN. Compared to the second test series, larger deformations and consequently a lower stiffness of the wall could be discerned by trend due to the lower strength of unit type C.

The fourth test series served primarily as comparison with the numerical simulations which were conducted simultaneously. As before, the wall lengths were varied between 1300, 2000 and 2500 mm. The superimposed load was 100 % of $\sigma_0$. As was to be expected, also here a shear fracture failure could not be effected. Due to the high superimposed load, the wall length has hardly any influence on the test result; the load-deformation curves are almost identical.

Parallel to the test series, a numerical model to simulate the shear failure of filled clay masonry was developed. It was the aim to verify this model by means of the existing shear tests to predict the ultimate loads of shear walls under changed boundary conditions which were not investigated in the test. For this purpose, the shear walls were modelled in a two-dimensional model to keep the computational effort within reasonable limits. The concept of the detailed modelling of masonry was adhered to in which the joints of the shear wall are linked by interface elements and the units by continuum elements. The reinforcement bars are modelled by bar elements. For the calculations the FE-programme OOFEM (Patzák (2000,
2001a, 2001b)), developed at the Technical University of Prague and available in the source code, was applied. The net used to model the shear walls is illustrated in Figure 7 a). In the net detail, Figure 7 b), the interfaces of the open head joints are displayed in red, the concrete interfaces in grey and the clay material interfaces in orange. The clay material interfaces represent the middle partition of the clay units, cf. Figure 1. The concrete interfaces model the concrete in the area of the joint of two units and in the area of the middle partition of the units. Bed joints were not provided because of the almost continuous concrete body. The continuum elements represent the composite body of the unit halves. The continuum elements were assigned effective material properties to characterize the fracture behaviour under tensile load which had previously been determined by means of a three dimensional simulation of the tensile and compressive tests on small composite specimens.

Figure 7: FE-net of the shear wall reinforced with steel bars

Figure 8 a) exemplarily shows the test setup of such a tensile test in the direction of the unit height for a composite specimen made of unit type C with SCC. The FE-net for the displacement controlled simulation of the test is illustrated in Figure 8 b) for the upper half of the composite specimen. At first, the tensile strength of the concrete core was reached under maximum load. The cracks localized in a band at about the height of the recess of the clay unit, Figure 8 c). With the growing de-strengthening of the concrete core, the tensile stress was increasingly transferred on the clay units. The ultimate failure occurred when the tensile strength of the clay unit was reached.

Figure 8: Tensile test in the direction of the unit height (test/simulation)
The measured and modelled stress-strain curves show a good correlation, see Figure 9. Also the simulation results for the tensile load in the direction of the unit length and for the compressive load conformed well to the test results. This applies also to the other clay units.

![Stress-strain curves](image)

**Figure 9: Stress-strain curves (Test/simulation)**

Based on these simulation results, an isotropic damage law to characterize the de-strengthening behaviour under tensile load was applied in the wall model for the continuum elements. The tensile strength was applied as mean value of the tensile strength in the direction of the unit length and unit height which was determined on small composite specimens. The fracture energy was equated with the fracture energy of the concrete. The effective elastic properties of the composite specimens were determined according to the homogenization method described in Hannawald (2007). The boundary conditions are selected in a way that the moment zero point remained in the centre of the wall under load. The total vertical load was 660 kN in the calculations discussed in the following. This corresponds to a superimposed load of 1.0\(\cdot\sigma_0\).

The load displacement curves (LDC) of the walls with the material properties of unit type C combined with SCC with and without steel bar reinforcement are shown in Figure 10. The horizontal force is plotted over the horizontal displacement in the upper right edge of the shear wall. In both cases, i.e. at the reinforced as well as the unreinforced wall, the limit of the elastic zone can clearly be discerned by the bending of the LDC. The LDC of the reinforced wall runs horizontally at the maximum load of approx. 600 kN. The calculation was interrupted because the minimum incremental motion of the displacement difference for convergence had to be reduced to the minimum value of \(10^{-6}\) mm and thus a noteworthy load increase was no longer possible. The LDC of the unreinforced wall shows a de-strengthening behaviour. Here, with 540 kN, the maximum load ranges below the value of the reinforced wall by approx. 10 %.
Figure 10: Simulated load displacement curves for the shear wall 2.5 x 2.5 x 0.24 m³ with C/SCC (1.0·σ₀ corresponds to 660 kN)

The failure mechanism of the shear walls is discussed as measurement for the load by means of the distribution of the strain tensor, Figures 11 and 12.

Figure 11: Simulation results C/SCC with steel bar reinforcement (1.0·σ₀)

Figure 12: Simulation results, C/SCC unreinforced (1.0·σ₀)

In each left Figure a), strain concentrations (red) in the continuum elements of the composite material occur at the limit of the elastic zone where the open head joints of the unit layer above and below adjoin. In the area of the open head joints of the considered unit layer, the
continuum elements are lightly loaded (blue). With increasing horizontal force, the strain concentrations cover the entire unit height. There, the tensile strength of the composite material is exceeded and the strains increase as a result of the de-strengthening. These strain concentrations are to be interpreted as smeared crack. The “crack” runs coming from below up to half the unit height on the left of the clay material interface into the upper unit half on the right. In each right Figure b), the state of maximum load is shown. In the case of the unreinforced wall, Figure 12 b), it can be discerned how the deformations localize in a diagonal crack band and thus lead to a failure of the shear wall. This crack band runs approximately diagonally through the unit halves. When comparing the test results to the simulation calculations, a satisfactory correlation in the linear zone of the load deformation curves was yielded.

SUMMARY
The investigations on the axial compressive strength and on the shear load bearing behaviour showed that the load bearing and deformation behaviour of filled clay walls is significantly influenced by the concrete properties – in this case particularly by the concrete compressive strength. As compared to unfilled masonry, the shear load bearing capacity could be increased by a multiple while the deformations were smaller.

To determine the effective material properties of the filled clay units, extensive experimental and numerical investigations on filled masonry units had to be performed. Applying the determined smeared material laws, the shear tests conducted at the IZF Brick and Tile Research Institute were numerically simulated.

The comparison of the non-linear behaviour in the shear test failed because presently there is no test stand in Germany which can transmit the occurring shear forces without being damaged. The simulated maximum loads thus represent non-validated predictions.

REFERENCES


