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Experimental Investigations on the Shear load-bearing Capacity of Thermal Insulating Clay Unit Masonry Walls

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Abstract

Cyclic shear tests are a proven method for evaluating the load-bearing capacity and deformation behaviour of masonry walls under dynamic horizontal loads, such as those that occur during earthquakes. These tests reproduce the realistic behaviour of stiffening masonry walls in combination with floor slabs, as in real buildings the walls are subjected to both vertical and horizontal in-plane forces. The interaction with the reinforced concrete slabs leads to restraining effects which create counteracting bending moments and can be simulated using cyclic shear tests.

This paper describes cyclic shear tests on eight storey-high masonry walls made of highly thermally insulating clay units, which were carried out in the Laboratory for Structural Engineering at the University of Applied Sciences Regensburg (OTH), Germany. The failure mechanisms were documented by digital image correlation. The test results are then compared with the expected calculated resistance values of the current design approaches according to DIN EN 1996-1-1/NA. Based on the calculation and the failure patterns from digital image correlation, it was possible to show that the standard correctly depicts the failure of the shear wall in some cases. For the majority of the wall configurations analysed, however, the results are on the safe side and show additional load-bearing capacity reserves.

Keywords: static-cyclic shear tests; insulating clay unit; unreinforced masonry; shear capacity; in-plane loadings; Eurocode 6

Introduction

With the increasing importance of the energy transition, building physics aspects in multi-storey residential buildings, such as energy efficiency and thermal insulation, are moving more into focus. At the same time, modern buildings have to continue to fulfil high structural requirements while offering flexible usage options. This is reflected in the floor plan with few and short stiffening walls. Although masonry constructions are able to withstand high compressive forces, the verification of horizontal loads, particularly under earthquake loading, becomes more difficult.

This problem will be exacerbated in Germany in future by the introduction of Eurocode 8, which stipulates higher earthquake loads for certain regions. In addition, new areas will be subject to verification where no seismic verification was previously required.

Contrary to these current developments in the load assumptions for earthquake load cases and the preferred architectural transparency, the numerical verifications are still based on conservative, linear models that do not adequately reflect the actual load-bearing capacity and the failure mechanism. Axial force redistributions and the interaction of the stiffening walls with the concrete floors are often not considered in the verification process. These current challenges make it necessary to review or adapt conventional design methods to realistically map the behaviour of masonry walls made of thermally insulating clay units under horizontal loads.

Against this background, the German Federation of Industrial Research Associations (AiF) initiated the research project 'New approaches for the realistic design of masonry structures under horizontal loads. Three research institutes in Germany (IZF in Essen, OTH in Regensburg and CWE in Aachen) joined forces as project partners. The OTH in Regensburg assumed responsibility for experimental tests on storey-high walls made of highly insulating clay unit masonry. As part of these investigations, eight shear wall tests were carried out and analysed, which form the subject of this article. The focus is on analysing the failure mechanism and comparing the test results with the design approaches of the current standard DIN EN 1996-1-1/NA [1] to develop possible improvements and adjustments of the design approaches.

Experimental investigations

The test programme included tests W1, W2, W3, W4, W5, W6, W7 and W8. The superimposed load *N*, the moment distribution factor ψ and the wall length *l* were varied during the tests. The characteristic value ψ describes the moment distribution over the height of the wall. According to DIN EN 1996-1-1/NA Annex K [1], the interaction of the walls with the floor slabs results in a different moment distribution in the stiffening walls depending on the acting axial force and horizontal force on the wall. The moment distribution factor resulting in practice can be calculated using the two equations shown below from DIN EN 1996-1-1/NA [1] under the resulting eccentricity of the axial force at the bottom or top of the wall. Eq. (1) covers those cases where the eccentricity at the bottom of the wall is greater than at the top of the wall, while Eq. (2) is to be used if the eccentricity at the bottom of the wall is less than at the top of the wall.

$$\psi = \frac{1}{(1-\frac{e_0}{e_u})} > 0 \; with \; |e_u| > |e_o| \tag{1}$$

$$\psi = \frac{1}{(1-\frac{e_u}{e_o})} > 0 \; with \; |e_u| \leq |e_o| \tag{2}$$

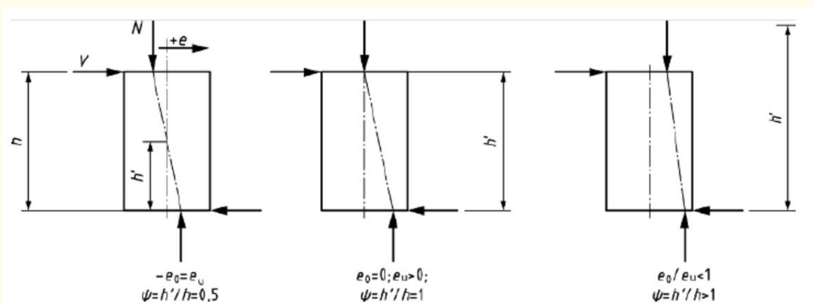


Figure 1: Examples of load eccentricities at the top and bottom of the wall [1].

Fig. 1 from DIN EN 1996-1-1/NA [1] shows an example of the limiting case of a large degree of fixity with ψ = 0.5, where the moment is zero at the half of the wall height. The cantilever wall is shown in the centre, with the moment zero point at the top of the wall. A degree of fixity greater than 1.0, which often occurs with high axial forces, is visualised in the right-hand image. For the following experimental investigations, variant investigations of practical masonry buildings were carried out in advance by the project partner in Aachen to determine the degree of fixity realistically. In accordance with DIN EN 1996-1-1/NA [1], this definition results in the wall slenderness λ_v based on Eq. (3) and based on this, the shear distribution factor c, which considers the shear stress distribution in a differentiated manner.

$$\lambda_v = \psi \cdot \frac{h}{l} \tag{3}$$

Based on these theoretical considerations, the boundary conditions for the experimental investigations are defined below.

Wall geometry

For the shear wall tests, specific conditions were established concerning wall geometry. Walls were categorised as either short, with a length of 1.50 m, or long, measuring 2.50 m. This distinction aimed to examine how wall length affects failure mechanisms and load-bearing behaviour. To replicate boundary conditions typical of multi-storey residential buildings, walls were constructed with a storey-high height of 2.75 m. The chosen wall thickness of 365 mm aligns with standard requirements for thermal and sound insulation in multi-storey buildings. By varying wall geometry in length, a more nuanced analysis was possible, yielding valuable insights into the behaviour of walls subjected to shear stress.

Imposed vertical load

The vertical load level and the moment distribution factor ψ, representing typical conditions in multi-storey residential buildings, were determined through variant analyses conducted by RWTH Aachen [2].

These analyses show that, depending on wall length, the vertical stresses range from 0.1 MPa and 1.4 MPa, amounting to approximately 2.5 % to 30 % of the masonry compressive strength f_k as specified in the approvals. Within the scope of these investigations, the factor ψ for the moment distribution across the wall spans from 0.75 to 2.0, addressing a variety of load scenarios.

Materials

A clay masonry unit with small cores was selected as a representative building material for thermally insulating high precise brick masonry in accordance with the general German building inspectorate approval no. 17.1-1066 [3]. The main dimensions and material properties are outlined in Table 1.

The general-purpose mortar (NM) for the bed joint between the first row of bricks and the foundation consists of Formel Pro cement masonry mortar. The same cement masonry mortar is applied in the bed joint between the top row of bricks and the steel beam.

The 10 mm thick bed joint is formed with thin-layer mortar of mortar class M10. The material properties regarding the compressive and bending strength of the different mortar types are shown in Table 2.

Overlapping length

The overlapping length within the experimental investigation was taken to 40 % of h_u, the minimum value according to DIN EN 1996. Test specimen W2 was made of 50 % of h_u to investigate the resulting effect.

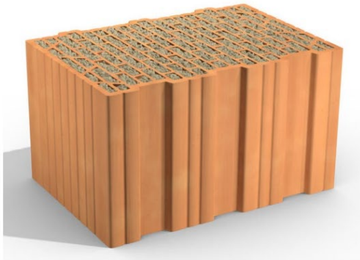
Perforation pattern	Dimensions		
	Length	[mm]	245.0
	Width	[mm]	364.0
	Height	[mm]	245.6
	Percentage of perforations		
	ρ	[%]	50.6
	Unit compressive strength f_b		
	f_b	[N/mm^2]	11.80
Average compressive strength of the masonry (with $h_{unit} / t = 6.8$)			
	f_{mean}	[N/mm^2]	5.70
Average modulus of elasticity of the masonry (secant modulus)			
	E	[N/mm^2]	4500

Table 1: Average values of the main dimensions and material properties of the vertically perforated brick used (taken from [2]).

Parameter [-]	Compressive strength (average) [N/mm²]	Bending strength (average) [N/mm²]
Thin-layer mortar with units with small coring	12.0	3.1
General purpose mortar (NM) with units with small coring	11.1	2.8

Table 2: Material parameters of the mortars used.

Production of the test wall specimen

A total of eight shear wall tests were conducted, with each test wall constructed directly on the existing foundation in the test hall by skilled professionals from the brick industry. Before the production of the test wall specimens, the support beam was first aligned and placed in a 12 mm thick layer of normal mortar. The walls were then constructed in a stretcher bond without perpend joint mortar. To prevent joint failure between the first row of blocks and the foundation, the foundation surface was prepared as a rough joint. The bed joints were formed with thin-layer mortar of mortar class M10 and a capped application using the MauerTec system. Once the mortar has hardened, the load application beam with interlocking profiling was placed on the masonry wall and aligned using lasers. A 20 mm thick layer of normal mortar created the force-fit connection between the load application beam and the top of the wall.

Shear test procedure

In conventional multi-storey residential buildings, stiffening walls are subjected to axial forces as well as horizontal in-plane loads from wind or earthquake. The interaction of shear walls with the floor slabs results in relevant effects of fixity at the top of the wall, which generate back-rotating moments in real buildings. To capture this interaction of bending moment and axial force, static cyclic tests are carried out that follow the test specifications for the determination of the shear resistance of masonry walls [4].

The Laboratory for Structural Engineering is equipped with a 70 cm thick laboratory floor and a strong laboratory wall to facilitate the shear wall tests. The schematic diagram in Fig. 2 illustrates the test rig from the front.

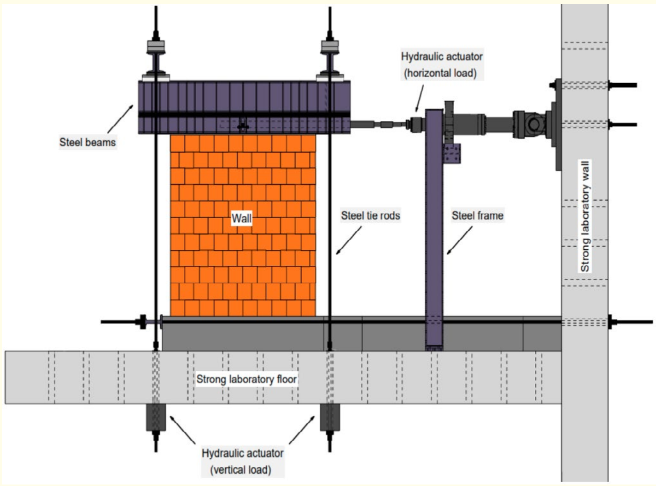


Figure 2: Test set-up.

To apply axial forces, shear forces and a bending moment, two vertical hydraulic actuators are arranged below the laboratory floor and a horizontal hydraulic actuator is arranged in front of the laboratory wall. The vertical force is transferred from the hydraulic actuator to the crossbars on the top of the wall of the test specimen via four tie rods. These crossbars are placed on roller bearings to prevent torsional forces on the wall. A Teflon plate is located below the roller bearing as a plain bearing, to reduce bearing friction and ensure horizontal displacement at the top of the wall. Elastomer bearings between the two steel beams at the top of the wall enable relative movement.

Additionally, reinforcing bars are welded to the bottom of the lower steel beam to ensure a sufficient bond between the masonry wall and the load application.

The cyclic horizontal in-plane force is applied via a steel beam lying on the masonry wall by coupling the horizontal actuator to the steel beam at half the length of the wall by means of a bolt. To ensure independent application of the horizontal force without affecting rotation or vertical displacement, a joint is located behind the horizontal actuator.

This test setup generates an in-plane bending moment in the wall. During testing, the vertical force is adjusted to maintain a consistent total load on the test specimen, while also simulating the effects of fixity from the torsional restraint of the overlying concrete slab.

The load history provides for the cyclic horizontal force to be applied to the wall head in a displacement-controlled manner. The sinusoidal load programme consists of three cycles, each lasting 60 seconds, with progressively increasing amplitude of the horizontal displacements. Meanwhile, the vertical force is applied in a force-controlled manner.

Measurement and data recording

During the experimental investigations, extensive data sets were collected. Vertical forces at the top of the wall were precisely measured using two load cells, while inplane deformations were captured with inductive displacement transducers. Additionally, detailed deformation data were recorded in all three spatial directions using an optical measuring system that employs digital image correlation for precise measurement and evaluation. These data acquisition methods provide a comprehensive understanding of the behaviour and failure of the test wall specimen.

Test results

This section shows the maximum shear resistance and the tested parameters of eight walls made of clay unit masonry under combined M-N-V loading (Table 3).

Test wall panel [-]	Wall Length [m]	Bonding overlap l_{ol}/h_u [-]	Average imposed load N [kN]	Moment distribution factor ψ [-]	Max. horizontal force V [kN]
W1	2.5	0.4	411	1.5	107
W2	1.5	0.5	739	1.0	125
W3	2.5	0.4	719	2.0	123
W4	1.5	0.4	222	0.75	100
W5	2.5	0.4	924	2.0	192
W6	1.5	0.4	370	0.75	130
W7	2.5	0.4	205	1.0	91
W8	1.5	0.4	370	1.0	97

Table 3: Results of the shear tests on walls of precision small-cored units (wall thickness t = 365 mm, wall height = 2.75 m).

Calculation of the expected horizontal force according to DIN EN 1996-1-1/NA

In the course of the investigation, the experimentally determined values are categorised in the normative context in accordance with DIN EN 1996-1-1/NA [1]. To enable a more comprehensive analysis, characteristic strength values are converted to mean values by multiplying them by a factor of 1.2. Additionally, existing results from small component tests, such as masonry compressive strength, are included in the analysis.

In this chapter, the expected horizontal force according to the standard is calculated by applying the normative design approaches. In this context, different types of failure are considered in a differentiated manner, including friction failure (FR) (stepped crack pattern in the bed and perpend joints), diagonal tension failure (DT) of the unit and flexural failure (FL).

Strength parameters

A comparison between test and design result requires a detailed characterisation of the shear strength values. The following parameters according to Table 4 are required:

$f_{b,mean}$	$f_{mean,masonry}$	$f_{bt,cal,mean}$	$\mu_{mean,cal}$	$f_{v,mean,0,cal}$
[N/mm²]	[N/mm²]	[N/mm²]	[-]	[N/mm²]
11,8	5,7	0,37	0,48	0

Table 4: Strength parameters.

The results of the compressive strength of the unit $f_{b,mean}$ are based on own experimental investigations. The compressive strength of the masonry is based on experimental investigations conducted by the project partner IZF in Essen [2].

For the tensile strength $f_{bt,cal,mean}$ of vertically perforated bricks a proportion of 2.6 % of the compressive strength of the unit $f_{b,mean}$ is assumed in accordance with DIN EN 1996-1-1/NA [1].

The coefficient of friction μ reduced from 0.6 to 0.4 in accordance with DIN EN 1996-1-1/NA [1] is used for the friction failure criterion. In addition to the coefficient of friction μ, the adhesive shear strength $f_{v,mean,0}$ is required for the friction failure criterion. It is assumed in the test under cyclic loading that partial opening of the bottom bed joint can occur when the previously suppressed areas are relieved. Therefore, the cohesion is neglected on the safe side, as the degradation due to gaping because of the preceding cycles can be assumed.

Verification according to DIN EN 1996-1-1/NA

In this section, the maximum possible horizontal force is calculated at mean value level in accordance with DIN EN 1996-1-1/NA [1] for the eight tested wall configurations. The lower value of friction failure (Eq. (4)) and diagonal tensile failure (Eq. (5)) is determined as the calculated shear strength.

$$f_{vlt,1} = f_{v,mean,0} + \mu_{mean} \cdot \sigma_{D,mean} \tag{4}$$

$$f_{vlt,2} = 0{,}45 \cdot f_{bt,cal,mean} \cdot \sqrt{1 + \frac{\sigma_{D,mean}}{f_{bt,cal,mean}}} \tag{5}$$

The compressive stress $\sigma_{D,mean}$ is the mean normal stress acting in the supressed area which is calculated according to Eq. (6) from the quotient of the acting normal force at mean value level N_{mean} and the supressed cross-section.

$$\sigma_{D,mean} = \frac{N_{mean}}{t \cdot l_{cal}} \tag{6}$$

The calculated wall length l_{cal} according to Eq. (7) is the supressed length of the wall panel resulting from the linear calculation with a triangular normal stress distribution (Fig. 3 centre). No increase was made from l_{cal} to $l_{c,lin}$.

$$l_{cal} = min\begin{cases} l \\ \frac{3}{2} \cdot \left(1 - 2 \cdot \frac{V}{N} \cdot \lambda_v\right) \cdot l \end{cases} \tag{7}$$

The shear stress distribution factors c were determined according to the wall slenderness in accordance with Annex K of DIN EN 1996-1-1/NA [1]. The values for the wall slenderness λ_v and the shear stress distribution factor c are shown for all eight wall test specimens in Table 5. According to Fig. 1 from Chapter 2, test specimen W1, W3 and W5 with a degree of fixity ψ > 1.0 can be assigned to the moment distribution of the right-hand drawing file. The test specimen W2, W7 and W8 represent a cantilever wall with a moment distribution in the centre section of Fig. 1. The test specimen W4 and W6 have a moment distribution factor ψ = 0.75 and a moment zero point at 0.75 times the wall height.

Test wall panel	W1	W2	W3	W4	W5	W6	W7	W8
Wall slenderness $\lambda_v = \psi \cdot \frac{h}{l}$ [-]	1,65	1,83	2,20	1,38	2,20	1,38	1,10	1,83
Shear distribution factor c [-]	1,33	1,42	1,50	1,19	1,50	1,19	1,05	1,42

Table 5: Overview of wall slenderness and shear distribution factor.

For the verification of the bending compression zone, the normal stress $\sigma_{flexural}$ acting in the supressed area is calculated by applying the stress block (Fig. 3 right) according to Eq. (8).

$$\sigma_{flexural} = \frac{N_{mean}}{t \cdot l_c} \tag{8}$$

The supressed length of the stress block is calculated according to Eq. (9).

$$l_c = \min \begin{cases} 0{,}9 \cdot l \\ \left(1 - 2 \cdot \frac{V_{mean}}{N_{mean}} \cdot \lambda_v\right) \cdot l \end{cases} \tag{9}$$

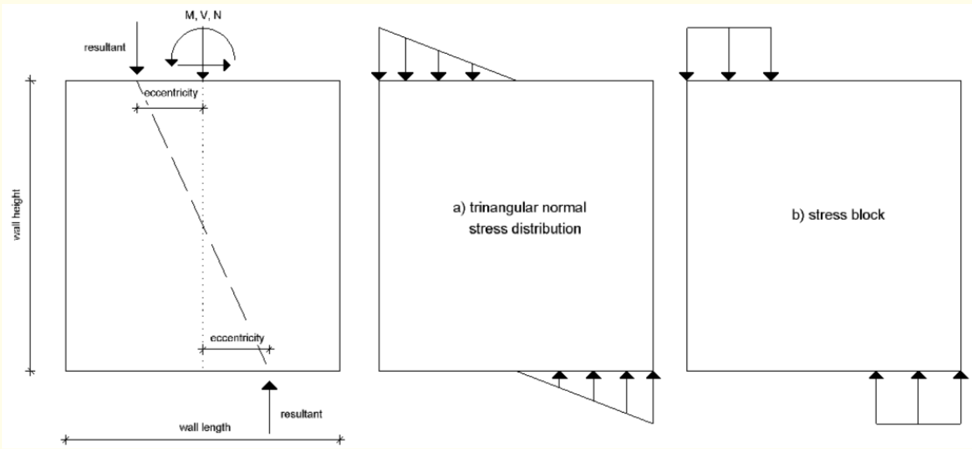


Figure 3: Exemplary course of the normal stress distribution under combined M-N-V interaction for a) triangular normal stress distribution b) stress block.

Since the verification of the shear load-bearing capacity has always be carried out under combined normal force and bending load (M-N-V interaction) and the action and resistance depend on each other via the determination of the supressed length, the determination of the calculated load-bearing capacity is only possible in an iterative way. The quality of the design equations in the standard (DIN EN 1996-1-1/NA [1]) can therefore be assessed by comparing the iteratively determined maximum calculated load-bearing capacity with the results of the experimental investigations. This comparison between test and design result is part of the following chapter.

The results of the maximum horizontal force to be expected on average in accordance with DIN EN 1996-1-1/NA [1] are shown in Table 6 together with the associated calculated failure mechanism. Here it is clear that for all test specimens, diagonal tensile failure (DT) is the decisive type of failure.

Test wall panel	W1	W2	W3	W4	W5	W6	W7	W8
$V_{R,mean,EC6/NA}$ [kN]	95	113	119	66	138	89	81	68
Failure Criterion [-]	DT	DT	DT	DT	DT	DT	DT	DT

Table 6: Overview of the calculated horizontal forces to be expected in accordance with DIN EN 1996-1-1/NA [1] with representation of the calculated failure criterion.

Comparison of the test values with the design approaches of DIN EN 1996-1-1/NA

In this chapter, the expected shear load-bearing capacities $V_{R,mean,EC6/NA}$ previously calculated in accordance with DIN EN 1996-1-1/NA [1] are compared directly with the horizontal forces $V_{exp,mean}$ achieved in the test. The comparison of the test values for each wall test specimen always considers the maximum horizontal force. The results, showing the utilisation between test and design results, are shown in Table 7.

Test wall panel	W1	W2	W3	W4	W5	W6	W7	W8
$V_{exp,mean}$ [kN]	107	125	123	100	192	130	91	97
$V_{R,mean,EC6/NA}$ [kN]	95	113	119	66	138	89	81	68
$V_{exp,mean}/V_{R,mean,EC6/NA}$ [-]	1,13	1,11	1,03	1,52	1,39	1,46	1,12	1,43

Table 7: Comparison between experimental results and design calculations according to DIN EN 1996-1-1/NA [1].

In the case of the highly thermally insulating small-chamber brick used, the standardised approaches according to DIN EN 1996-1-1/NA [1] with the average material parameters used, describe the results of the tests on the safe side. On average, the design approaches of DIN EN 1996-1-1/NA [1] for vertically perforated bricks underestimate the load-bearing capacity by 27 % (value range 3 % to 52 %). No test value was over-estimated by the calculated prediction.

For a more detailed analysis, the test results are categorised below in the failure curves in accordance with DIN EN 1996-1-1/NA [1]. In addition, the decisive failure pattern based on the measurement used digital image correlation is shown to verify the failure mechanism.

For better comparability, the normal and horizontal forces (N_{mean} and V_{mean}) are standardised based on the test specimen cross-sectional area (t · l) and the mean value of the masonry compressive strength f_{mean} according to the following equations (Eq. (10) & Eq. (11)):

$$n_{mean} = \frac{N_{mean}}{t \cdot l \cdot f_{mean}} \tag{10}$$

$$v_{mean} = \frac{V_{mean}}{t \cdot l \cdot f_{mean}} \tag{11}$$

It was found that for all the wall configurations analysed, the criterion of diagonal tensile failure is decisive. As the comparison of the calculation with the failure diagrams on the following two pages makes clear, in many cases the fracture pattern cannot reflect the type of failure that is decisive for the calculation.

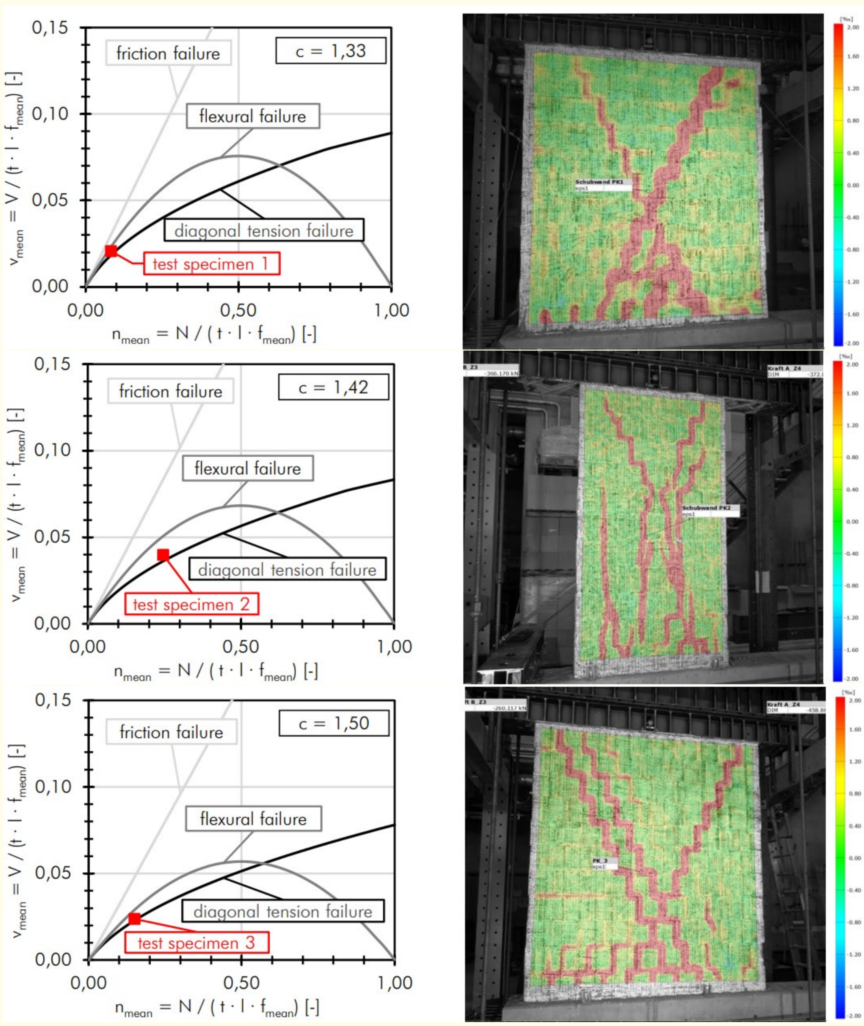
Test specimen W1 has a load-bearing reserve of 13 % beyond diagonal tension failure. The failure pattern is characterised by staircase-shaped cracks in the perpend and bed joints, but also cracks through the stones are visible. This indicates a mixed failure as the final cause of failure.

Test specimen W2 shows a similar load reserve of 11 %. The cracks run through the blocks themselves as well as trough perpend and bed joints, which corresponds well with the calculated diagonal tension failure.

In test specimen W3, a load-bearing reserve of 3 % was observed beyond the diagonal tension failure. The failure pattern is characterised by staircase-shaped cracks in the perpend and bed joints, while diagonal tension failure is only partially visible. This failure pattern corresponds more to friction failure and is therefore not fully compatible with the standard-compliant calculation.

At 52 %, test specimen W4 shows the highest load-bearing capacity reserve beyond diagonal tension failure. The failure pattern rather indicates a friction or flexural failure, as staircase-shaped cracks occur in perpend and bed joints without signs of diagonal tension failure. The classification in the failure diagram lies between the boundary lines of friction and flexural failure, which would fit well with the observed crack pattern.

For test specimen W5, the load-bearing reserve is 39 %. The crack pattern shows a combination of cracks in perpend and bed joints as well as through the units themselves, with a concentration at the bottom of the wall. It is presumably a mixed failure of diagonal tension failure and flexural failure here, which basically corresponds with the calculation under considerable load reserves.



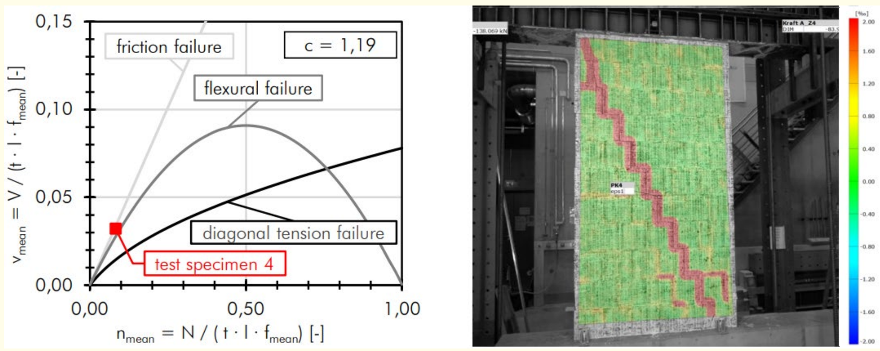
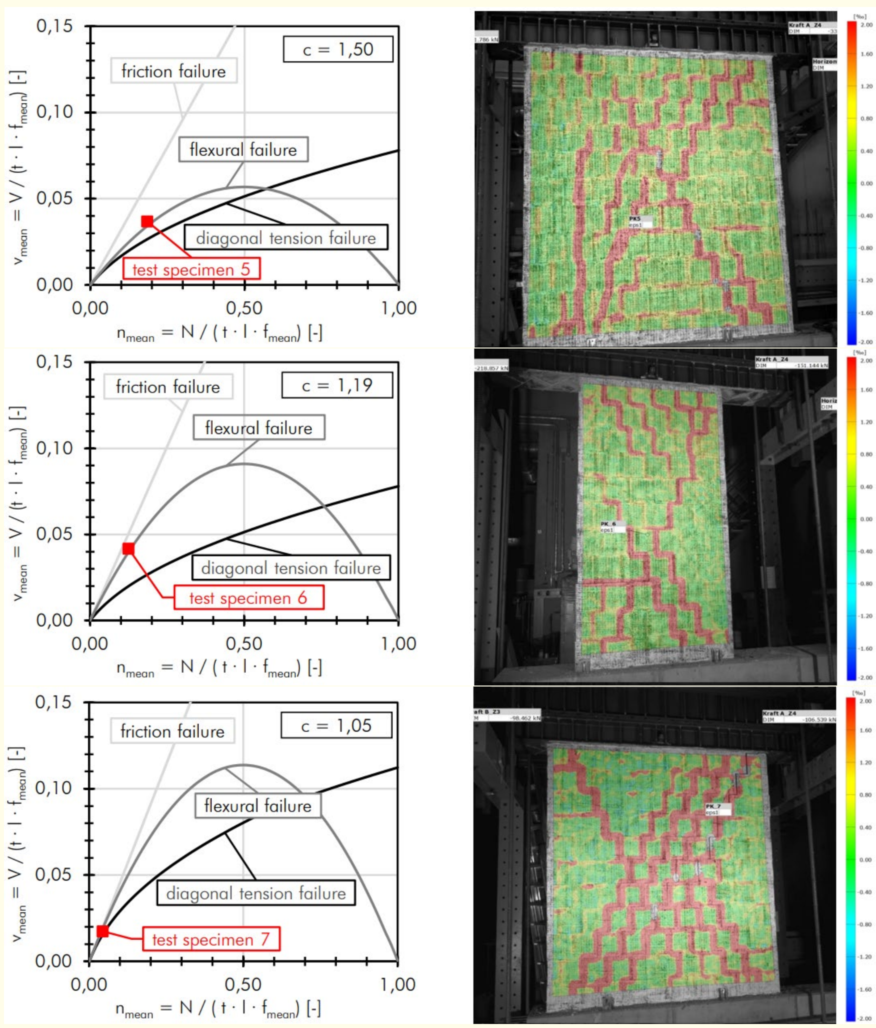


Figure 4: Comparison of the test values with the design approaches according to DIN EN 1996-1-1/NA [1] (test specimens 1 – 4).



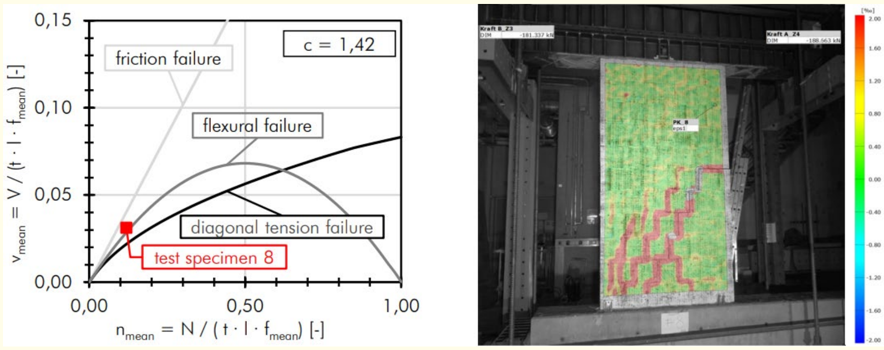


Figure 5: Comparison of the test values with the design approaches according to DIN EN 1996-1-1/NA [1] (test specimens 5 – 8).

Test specimen W6 shows a load reserve of 46 %, whereby the failure pattern is dominated by cracks in perpend and bed joints, which are concentred at the bottom of the wall. The observed failure does not correspond to the calculated diagonal tension failure, but rather shows a tendency towards flexural failure, which, however, should not be decisive according to the calculation.

Test specimen W7 has a load-reserve of 12 % and shows staircase shaped cracks in the perpend and bed joints, which is typical of friction failure. Diagonal tension failure cannot be identified here in the failure pattern. As the horizontal forces in this area of the diagram are low, the failure curves are relatively close to each other, making it difficult to strictly separate the forms of failure.

Test specimen W8 shows a high load-bearing reserve of 43 % with a failure pattern that shows staircase-shaped cracks in the perpend and bed joints as well as cracks through the units. The failure pattern matches the calculation, but shows additional reserves close to the flexural failure curve.

The comparison of the failure pattern with the calculation according to DIN EN 1996-1-1/NA [1] shows that the curve for the diagonal tension failure would have to be set higher in most cases in order to realistically map the existing load-bearing reserves. This indicates that an increase in the standardised factor of 2.6 % of the compressive strength f_b for vertically perforated bricks could be useful here in order to take diagonal tension failure into account more accurately in the calculation.

An adjusted factor would better utilise the existing load-bearing capacity reserves and enable a more economical design of masonry structures under horizontal loads.

Summary

In this article, the results of eight shear tests on storey-high high-precise brick masonry walls made of highly thermally insulating clay bricks with small cores were presented and analysed. The aim of the comprehensive investigations was to analyse the load-bearing behaviour of the thermal insulating high precise brick masonry under inplane shear loading and realistic boundary conditions and to check the quality of the current design approaches in the standard. It was found that the design approaches of the current standard DIN EN 1996-1-1/NA [1] tend to represent the tests on the safe side and that there are considerable load reserves in some cases.

A comparison of the calculated results with the failure patterns from digital image correlation showed that the failure mechanism of the diagonal tensile failure predicted by the standard was not reproduced for all test specimens. The failure pattern often showed a mixed failure, which manifested itself in cracks through the units as well as staircase-shaped cracks through perpend joints and bed joints.

Against the background of these findings, the development of an improved design concept that utilises the existing load reserves of up to 50 % and adequately considers the frequently occurring mixed failure seems sensible. This adaptation would contribute to a more accurate and realistic representation of the load-bearing behaviour of thermal insulating clay unit masonry under horizontal loads.

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