ESECMaSE
Enhanced Safety and Efficient Construction of Masonry Structures in Europe

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Enhanced Safety and Efficient Construction of Masonry Structures in Europe

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Collective Research

Work Package N°: 9

D9.1 Proposals for an advanced design model of masonry under lateral loads for the implementation in Eurocode 6-1-1

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Table of contents

1. Introduction

The research project ESECMaSE has already lead to new knowledge about shear behaviour of masonry. This includes the results of the tests which were already carried out, the analytical and numerical studies and their comparison with the available experimental results. While the theoretical considerations were concentrated mainly on the static loading, the experimental implementation has been investigating the behaviour under cyclic loading too. The static loading can be seen as a special case of the cyclic loading.

The intention of this report is to collect the results about the shear behaviour under static loading for the purpose to propose an advanced design model of masonry subject to lateral loads for the implementation in Eurocode 6-1-1 (EC6). Therefore some already introduced or proposed normative regulations are taken as examples and thereupon compared with the analytical consideration from Work Package 4 and the proposed shear design model. The background of this proposal will be discussed and the universality will be checked.

Because of the major influence of the acting vertical load on the shear load capacity, this issue will also be discussed and a proposal for implementation in the code will be made. Within the scope of ESECMaSE the parameters of the used material were determined by means of small tests. As a result, the large scale experiments can be understood and recalculated. These values are of particular interest especially for the numerical recalculation. But the amount of the material data is not large enough for an universal definition of material properties for the normative work. A consideration of the already regulated normative values as well as values from the literature will be made.

In addition, the governing equations will have to be expand by the safety factors and compared with the collection of test results. This is necessary in order to ensure the global safety standards.

Finally, a text proposal for a future update of the Eurocode 6-1-1 will be given, merely a basis for discussion. The establishment of standards is a continually evolving process and the evaluation of the pending tests will be reflected in it.

2. State of standardisation

Most of the masonry codes define in first place the shear strength and maybe propose an equation for the calculation of the shear capacity of the wall in the second place. The strength is thereby defined as the minimum value of strength, due to different failure criteria. The main failure criterion, which is included in all standards, is the sliding. The second criterion follows from the tensile strength of the unit.

The geometry and the mechanical behaviour of the whole wall have to be taken into account by the designing engineer. This includes the absence of a tensile strength of masonry in vertical direction, or the neglecting of it, which leads to the formation of gaps and a partially compressed area. Other verification procedures, like the one for bending and compression, have also to been performed. The shear part of the standards usually does not refer to them.

In the following some normative regulations of the design procedure for shear are listed. Further procedures, like the Swiss or the Canadian approach are part of Deliverable 4.2 [12].

2.1. Australian Standard (AS 3700)

As an example of other non-European masonry standard the Australian Standard [1] should be cited.

The verification of unreinforced masonry members for shear forces has to be satisfied for each combination under minimum design compressive stress.

$$V_d \leq V_0 + V_1 \tag{1}$$

For AAC-masonry:

$$V_d \leq V_{A0} + V_{A1} \tag{2}$$

With

$$V_0 = \phi\, f'_{ms} A_{dw} \tag{3}$$

$$V_1 = k_v f_d A_{dw} \tag{4}$$

$$V_{A0} = \phi\, 0{,}67 f'_{ut} A_b \tag{5}$$

$$V_{A1} = k_v \; f_d \; A_b \tag{6}$$

Or in short notation:

$$V_d \leq \left(\phi \; f_{ms}^{'} + k_v f_d\right) A_{dw} \tag{7}$$

For AAC-masonry:

$$V_d \leq \left(\phi \; 0{,}67 f_{ut}^{'} + k_v \; f_d\right) A_b \tag{8}$$

Where:

k_v is the shear factor (friction coefficient);

$f_{ms}^{'}$ is the characteristic shear strength of masonry;

A_{dw} is the combined bedded area and grout area, if any, of a shear-resisting; portion of a member; for members of solid rectangular cross-section;

f_d the minimum design compressive stress on the bed joint under consideration but not greater than 2 MPa;

A_b the bedded area of a masonry cross-section;

ϕ the capacity reduction factor;

$f_{ut}^{'}$ the characteristic lateral modulus of rupture of masonry units.

The Australian Standard [1] gives some reduction factors, which are comparable to the partial safety factor for material in the Eurocode 6, to evaluate the strength capacity. The relation could be described by:

$$\phi = \frac{1}{\gamma_M} \tag{9}$$

Table 1 Capacity reduction factors from Australian Standard Table 4.1 in [1]

Type of masonry or accessory and action effect	**Capacity reduction factor(ϕ)**
(a) Unreinforced masonry	
(i) Compression	0.45
(ii) Other actions	0.60
(b) Reinforced and prestressed masonry	0.75
(c) Wall ties, connectors and accessories	
(i) Wall ties in tension or compression	0.95
(ii) Connectors across a joint in masonry	0.75
(iii) Accessories and other actions	0.75

For the second equation V_1 or V_{A1} the capacity reduction factor is included in the in the shear factor. So the values seems to be smaller then e.g. in EC 6 or the German standard DIN 1053.

Table 2 Shear factors (friction coefficients) from the Australian Standard Table 3.3 in [1]

Type of masonry	Location	k_v
Clay, concrete or calcium silicate	At mortar bed joints	0.3
AAC	At mortar bed joints	0.12
All	At membrane-type damp-proof courses, flashings and similar locations where the membrane is in contact with the unit, concrete or within the mortar and the damp-proof course membrane is of—	
	(b) polyethylene-and-bitumen coated aluminium	0.15
	(a) bitumen-coated aluminium or embossed polyethylene	0.3
	At interface of masonry with concrete	0.3
	At interface of masonry with steel	0.2
	At other locations	zero

In chapter 1.5.2.9 of the Australian Standard [1] the lateral modulus of rupture is descript as:

"The characteristic lateral modulus of rupture of masonry units (f'_{ut}) is the characteristic value of the flexural tensile strength of the masonry unit obtained when subjected to bending in the direction normal to the plane of the wall. In the absence of test data, the value of (f'_{ut}) is not allowed to exceed 0.8 MPa."

The characteristic shear strength f'_{ms} is defined in 3.3.4 of the Australian Standard [1]. For membrane-type damp-proof courses, flashings, it should be zero, or an appropriate value based on the results of tests. For shear in the horizontal direction in continuous horizontal mortar joints, for masonry constructed of other than AAC units, a value of $1.25 \cdot f'_{mt}$ (f'_{mt} = characteristic flexural tensile strength) but not greater than 0.35 MPa, nor less than

0.15 MPa. At a joint or interface confined by bonded reinforcement normal to the shear plane 0.35 MPa.

The Australian Standard [1] gives also values for the vertical direction.

2.2. Eurocode 6 (EN 1996-1-1)

The Eurocode 6 [5] is a joint development of some member states of the European Union. The intention is to unify the normative regulations for Europe. Therefore European committees were established to develop a proposal which has to be ratified by the national consortiums. The current version of the EC 6 could be used parallel to the national standards. The national standards will be entirely replaced in long-term.
A special feature of the Eurocode is the considering of country construction methods or building material properties. For this purpose possibilities were inserted at the appropriate points in the standardization text for national definitions. The concretions have to be made in the respective national annexes. Within a certain scope dissent design rules could also be define.

In chapter 6.2 of the Eurocode 6 [5] the verification of unreinforced masonry walls subjected to shear loading is regulated. At the ultimate limit state the design value of the shear load applied to the masonry wall, V_{Ed}, shall be less than or equal to the design value of the shear resistance of the wall, V_{Rd}.

$$V_{Ed} \leq V_{Rd} \tag{10}$$

The design value of the shear load has to be calculated in accordance to EN 1990 or DIN 1055-100. Typically it based on a characteristic loads multiplied by combination value and partial safety factors.

The design value of the shear resistance is given by:

$$V_{Rd} = f_{vd}\, l_c\, t \tag{11}$$

Where:

f_{vd} is the design value of the shear strength of masonry, based on the average of the vertical stresses over the compressed part of the wall that is providing the shear resistance;

t is the thickness of the wall resisting the shear;

l_c is the length of the compressed part of the wall, ignoring any part of the wall that is in tension. The length of the compressed part of the wall, l_c, should be calculated assuming a linear stress distribution of the compressive stresses.

The design value of the shear strength of masonry hast to be calculated by:

$$f_{vd} = \frac{f_{vk}}{\gamma_M} \tag{12}$$

In the Eurocode 6 [5] in section 3.6.2 (1) as a binding definition is written: "The characteristic shear strength of masonry, f_{vk}, shall be determined from the results of tests on masonry". As a second source for the shear strength a database could be used. But it is also possible to calculate the shear strength. For these the EC 6 has foreseen the following equation:

$$f_{vk} = f_{vk0} + 0{,}4\sigma_d \tag{13}$$

Where:

f_{vko} is the characteristic initial shear strength, under zero compressive stress;
The initial shear strength f_{vko} have to be halve for unfilled perpend joint.

f_{vlt} is a limit to the value of f_{vk};

σ_d is the design compressive stress perpendicular to the shear in the member at the level under consideration, using the appropriate load combination based on the average vertical stress over the compressed part of the wall that is providing shear resistance;

f_b is the normalised compressive strength of the masonry units, as described in 3.1.2.1 of EC 6, for the direction of application of the load on the test specimens being perpendicular to the bed face.

For the tensile failure of the unit an upper limit of 0,065 f_b or 0,045 f_b for unfilled perpend joints is given. Instead of theses limit a value f_{vlt} could be defined in the National Annex of every country. Herewith it should be possible to take other effects into account e.g. a special influence of the tensile strength of the units and/or of the overlap in the masonry.

In shell bedded masonry, where the units are bedded on two or more equal strips of general purpose mortar, the initial shear strength f_{vko} has to be reduced by the relation of the width of the stripes to the width of the wall.

The initial shear strength of the masonry may be determined from either tests, like the characteristic shear strength, or from the values given in table 3.4 in EC 6 [5]. When a country decides to determine its values of f_{vko} from a database, the values may be found in the National Annex.

Table 3 *Values of the intitial shear strength of masonry, f_{vko} from the Eurocode 6 Table 3.4 [5]*

Masonry units	f_{vko} (N/mm²)			
	General purpose mortar of the Strength Class given		Thin layer mortar (bed joint ≥ 0,5 mm and ≤ 3 mm)	Lightweight mortar
Clay	M10 - M20	0,30	0,30	0,15
	M2,5 - M9	0,20		
	M1 - M2	0,10		
Calcium silicate	M10 - M20	0,20	0,40	0,15
	M2,5 - M9	0,15		
	M1 - M2	0,10		
Aggregate concrete	M10 - M20	0,20	0,30	0,15
Autoclaved Aerated Concrete	M2,5 - M9	0,15		
Manufactured stone and Dimensioned natural stone	M1 - M2	0,10		

The vertical shear resistance of the junction of two masonry walls may be obtained from suitable tests or the characteristic vertical shear resistance may be based on f_{vko} (Table 3).

The values of the partial factor for materials γ_M, which are used in equation (12), have to be defined in the National Annex of the country. Recommended values, given as classes that may be related to execution control (see also Annex A of EC 6 [5]) according to national choice, are given in the table below.

Table 4 Recommended values of the partial factor in Eurocode 6 chapter 2.4.3 [5]

Material		γ_M				
		Class				
		1	2	3	4	5
	Masonry made with:					
A	Units of Category I, designed mortar[a]	1,5	1,7	2,0	2,2	2,5
B	Units of Category I, prescribed mortar[b]	1,7	2,0	2,2	2,5	2,7
C	Units of Category II, any mortar[a, b, e]	2,0	2,2	2,5	2,7	3,0
D	Anchorage of reinforcing steel	1,7	2,0	2,2	2,5	2,7
E	Reinforcing steel and prestressing steel	1,15				
F	Ancillary components[c, d]	1,7	2,0	2,2	2,5	2,7
G	Lintels according to EN 845-2	1,5 to 2,5				

[a] Requirements for designed mortars are given in EN 998-2 and EN 1996-2.
[b] Requirements for prescribed mortars are given in EN 998-2 and EN 1996-2.
[c] Declared values are mean values.
[d] Damp proof courses are assumed to be covered by masonry γ_M.
[e] When the coefficient of variation for Category II units is not greater than 25 %.

In 6.2 (5) it is requested, that "The length of the compressed part of the wall should be verified for the vertical loading applied to it and the vertical load effect of the shear loads."

With this clause the bending of the shear wall along the wall (in plane) should be covert. The verification of it could be made by using the explanation of chapter 6.1.2 of EC 6"Verification of unreinforced masonry walls subjected to mainly vertical loading"

The general rule of design verification gives for vertical loads:

$$N_{\mathrm{Ed}} \leq N_{\mathrm{Rd}} \tag{14}$$

The design value of the vertical resistance is given by:

$$N_{\mathrm{Rd}} = \Phi \, t \, f_{\mathrm{d}} \tag{15}$$

Where:

Φ is the capacity reduction factor, ϕ_{i}, at the top or bottom of the wall, or ϕ_{m}, in the middle of the wall, as appropriate, allowing for the effects of slenderness and eccentricity of loading;

t is the thickness of the wall;

f_{d} is the design compressive strength of the masonry.

In case of the verification of shear walls in plane the reduction factor ϕ without the slenderness effect should be used. This is equals to ϕ_{i} at the top or bottom of the wall. Therefore the EC 6 suggests a rectangular stress block for the verification.

$$\Phi_{\mathrm{i}} = 1 - 2\,\frac{e_{\mathrm{i}}}{t} \tag{16}$$

The EC 6 has also the opportunity to verify low eccentric loads by using the bending tensile strength. For this the strength has to be defined by experiments or can be found in the tables mentioned in the notes.

Table 5 Values of the bending strength of masonry parallel to the bed joint, f_{xk1} from the Eurocode 6 chapter 3.6.3 [5]

NOTE 3 For masonry made with autoclaved aerated concrete units laid in thin layer mortar, f_{xk1} and f_{xk2} values may be taken from the tables in this note or from the following equations:

$f_{xk1} = 0{,}035\, f_b$, with filled and unfilled perpend joints

Values of f_{xk1}, for plane of failure parallel to bed joints

Masonry Unit	f_{xk1} (N/mm^2)			
	General purpose mortar		Thin layer mortar	Lightweight mortar
	$f_m < 5$ N/mm^2	$f_m \geq 5$ N/mm^2		
Clay	0,10	0,10	0,15	0,10
Calcium silicate	0,05	0,10	0,20	not used
Aggregate concrete	0,05	0,10	0,20	not used
Autoclaved aerated concrete	0,05	0,10	0,15	0,10
Manufactured stone	0,05	0,10	not used	not used
Dimensioned natural stone	0,05	0,10	0,15	not used

2.3. German Standard (DIN 1053-100)

The German Standard DIN 1053-100 [3] basically corresponds to the still valid previous version DIN 1053-1 [2]. The conversion of the global safety concept to the partial safety concept was main aim of DIN 1053-100.

The verification of shear is equivalent to the verification procedure in the Eurocode (see eq. (10)). The design value of loading has to be opposed to the resistance of member.

The design factor of the applied shear loading has to be defined on the basis of the semi probabilistic safety concept according to the information in EN 1990. Furthermore the vertical loading must be considered due to its positive effect with the partial safety factor of γ_E = 1.0 to calculate the load capacity.

The design value of the shear load resistance V_{Rd} results from:

$$V_{Rd} = \alpha_s \frac{f_{vk}}{\gamma_M} \cdot \frac{d}{c} \tag{17}$$

Where:

f_{vk} is the characteristic shear strength;

γ_M is the partial safety factor for the material;

α_s is the coefficient of the shear capacity. For the verification of shear walls under wind load it is considered to be α_s = 1,125 l or α_s = 1.333 l_c whereas the minor of both values is decisive. In all other cases $\alpha_s = l$ and $\alpha_s = l_c$ respectively must be applied.

l is the length the verifiable wall;

l_c is the length the compressed part of the wall, ignoring any part of the wall that is in tension;

d is the thickness of the verifiable wall;

c is the factor to consider the shear stress distribution over the cross section.

Der Faktor für die Berücksichtigung der Schubspannungsverteilung in der Wand ist nur in den deutschen Normen DIN 1053-1 bzw. DIN 1053-100 enthalten. Er ist in Abhängigkeit von der Wandgeometrie wie folgt zu bestimmt:

Only the German standard DIN 1053-1 and DIN 1053-100 respectively include the factor to consider the distribution of the shear stress along a wall. The factor must be defined depending on the geometry of the wall as follows:

$$c = 1{,}0 \quad \text{for } h_w/l \leq 1$$
$$c = 1{,}5 \quad \text{for } h_w/l \geq 2. \tag{18}$$

Where:

h_w is the height of the wall;

l is the length of the wall.

Intermediate values must be interpolated linearly.

Equations for the shear strength refer to a model by *Mann/Müller* (see [27]). At this only the failure of friction along the bed joints and the tensile failure of the unit were considered. A graph shows also the combined compressive failure but only as information. Normally this kind of failure becomes not decisive and it is covered by the verification of vertical loading with bending.

Also the gaping of the bed joints due to the rotation of the masonry units was not considered as it is not decisive concerning small bricks with an adequate large overlapping length.

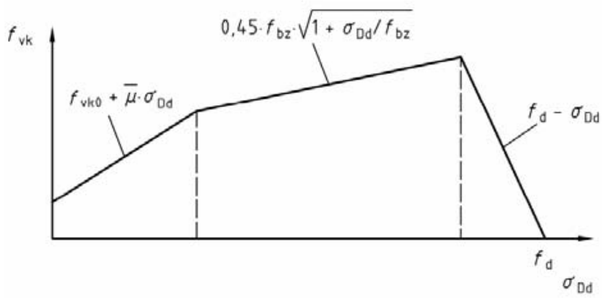


Figure 1 Schematic relation of shear strength and compressive stress (Figure 6 in [3])

In case of failure due to friction of the bed joint *Mann/Müller* give the following conditional equation:

$$f_{vk} = \frac{f_{vk0} - \mu \cdot \sigma_{Dd}}{1 + \mu \cdot \dfrac{2 \cdot h_{St}}{l_{St}}} \tag{19}$$

When applying a friction coefficient of μ = 0.65 and a ratio of h_{St}/l_{St} = 1/2, a decreased friction coefficient of $\bar{\mu}$ = 0.4 and a decreased initial shear strength of $f'_{vk0} = 0.606 \cdot f_{vk0}$ will result.

Thus the following equation is applied, when verifying shear loading with the exact method according to DIN 1053-100:

$$f_{vk} = f'_{vk0} + \overline{\mu} \cdot \sigma_{Dd} \ . \tag{20}$$

Mann/Müller [27] derive the tensile failure of the unit from the first principal stress (tension) in the middle of the unit. There a shear stress factor of 2.3 for the maximal shear stress in the unit was considered. The resulting equation is::

$$f_{vk} = 0.45 f_{bz} \sqrt{1 + \frac{\sigma_{Dd}}{f_{bz}}} \tag{21}$$

The minor value in the equations (20) and (21) is decisive for the verification.

Where:

f'_{vk0} is the decreased initial shear strength according to Table 6;

$\overline{\mu}$ is the decreased friction coefficient. The distribution of the stress in the bed joint along a unit is considered by decreasing. $\overline{\mu} = 0{,}4$ can be used for all types of mortar.

σ_{Dd} is the design compressive stress at the location of the maximal shear stress;

f_{bz} is the tensile strength of the unit. Using the following:

$f_{bz} = 0{,}025 \cdot f_{bk}$ for hollow bricks;

$f_{bz} = 0{,}033 \cdot f_{bk}$ for perforated bricks and units with grip holes or grip pockets;

$f_{bz} = 0{,}040 \cdot f_{bk}$ for bricks without grip holes or grip pockets;

f_{bk} is the characteristical value for compressive strength of the unit (classification of the unit's strength).

Table 6 Decreased initial shear strength f'_{vko} for several types of mortar according to DIN 1053-100

types of mortar	**NM I**	**NM II**	**NM IIa, LM 21, LM 36**	**NM III, TLM**	**NM IIIa**
f'_{vk0} $(N/mm^2)^{a)}$	0.02	0.08	0.18	0.22	0.26

a) The values f'_{vko} must be reduced by half for masonry with unfilled joints. In this case a joint is considered as filled, when a half or more of the wall thickness is filled.

The condition that the initial shear strength of unfilled joints must be reduced by half also exists in EN 1996-1-1. As the unfilled joint is included in the model by *Mann/Müller* in equation (19) in the decreased initial shear strength and the decreased friction coefficient respectively, a double decrease happens.

Table 1 in [3] shows the partial safety factor γ_M to define the design value of the load residence.

Table 7 Partial safety factors for material (table 1 in [3])

	γ_M	
	usual loading	**unusual loading**
masonry	$1.5 \cdot k_0$	$1.3 \cdot k_0$
bond, tensile and compressive residence of wall ties	2.5	2.5

In Table 7 is:

k_0 a factor to consider differnet partial safety factors γ_M of walls and „short walls“ according to DIN 1053-1:1996-11, 2.3. The rule is:

k_0 = 1,0 for walls;

k_0 = 1,0 for „short walls“ which consist of one or more separate units with a part of holes of less than 35 % and are not weaken by grooves or holes;

k_0 = 1,25 for all other „short walls“.

Short walls are walls with a cross section of less than 1000 cm². To verify shear walls under wind load normally a γ_M of 1.5 must be used.

The verification of an eccentric compressive load without the influence of slenderness (bending) is equivalent to EC 6. The equations in DIN 1053-100 ([3] chapter 8.9.1) are identical to the equations (14) to (16). But other then the EC 6 in chapter 8.9.1.2 the DIN 1053-100 refers clearly to the verification of wind panels.

The following is given to define the eccentricity:

$$e = \frac{M_{Ed}}{N_{Ed}} \tag{22}$$

The design value for wind panels has to be defined as follows:

$$M_{Ed} = 1.5 \cdot H_{Wk} \cdot h_w \tag{23}$$

Where:

H_{Wk} is the characteristic value of the resulting wind load, regarding the cross section which has to be verified;

h_W is the lever arm of H_{Wk}, regarding the cross section which has to be verified;

N_{Ed} is the **design value** of the normal load in the cross section which has to be verified.

Additionally the existing normal load eccentricities must be considered. The partial safety factor for the load is included due to the factor 1.5

Besides it is necessary that the calculated strain at the tensile loaded side is not larger than 0.1‰ with eccentricities larger than *l*/6. Thus an uncracked cross section should be indirectly ensured to take the initial shear strength into account for verifying friction at different directions of loading. If it is not possible to verify the tensile strain for unusual load combinations, the initial shear strength should be omitted when defining shear strength at ultimate limit state.

2.4. Latest Proposal for DIN 1053

The analytical approach for shear was enhanced in Germany on the basis of the model of *Mann/Müller* over the past years. Thus an addition for irregular bond (staircase bond) was made by *Simon* [33]. Another essay about shear failure of large-sized masonry by *Jäger/Schöps* [22] discusses the regular bond and facture behaviour with partial failure of the joints. As it is verified experimentally and numerically that failure in case of tensile failure of the unit begins at the kerb of the unit, a new equation for unit failure was developed as well.

The extended proposal for shear strength was discussed for the German standard and the national annex of the European masonry code. In the following a short description should be given.

The shear strength is given by the minimum of the main three failure cases:

$$f_{vk} = c \cdot \min \begin{cases} f_{vk,g} \\ f_{vk,f} \\ f_{vk,u} \end{cases} \tag{24}$$

The failure cases are:

<u>Gaping of single units</u>

$$f_{vk,g} = \max \begin{cases} \boxed{(f_t + \sigma_d) \cdot \dfrac{l_{ol}}{h_b}} \\ \dfrac{f_t \cdot (l_b - l_{ol}) + \sigma_d \cdot l_b}{2 \cdot h_b} \end{cases} \tag{25}$$

Where:

f_t is the characteristic initial shear strength. For masonry made of general purpose mortar and light weight mortar $f_t = 0{,}05\ N/mm^2$ and for masonry of thin layer mortar $f_t = 0{,}15\ N/mm^2$ should be applied;

σ_d is the design compressive stress at the location of the maximal shear stress;

l_{ol} is the overlapping length;

h_b is the height of the brick;

l_b is the length of the brick.

Friction failure

$$f_{vk,f} = \max \begin{cases} \boxed{\dfrac{(f_{vk0} + \mu \cdot \sigma_d)}{1 + \mu \dfrac{h_b}{l_b - l_{ol}}}} \\ f_{vk0} \cdot \dfrac{l_{ol}}{l_b} + \mu \cdot \sigma_d \end{cases} \tag{26}$$

Where:

f_{vk0} is the characteristic initial shear strength;

μ is the friction coefficient.

Tensile failure of units

$$\boxed{f_{vk,u} = \frac{f_{bt,cal}}{F} \left(-\frac{h_b}{2 \cdot F \cdot l_{ol}} + \sqrt{\left(\frac{h_b}{2 \cdot F \cdot l_{ol}}\right)^2 + 1 + \frac{\sigma_d}{f_{bt,cal}}} \right)} \tag{27}$$

Where:

$f_{bt,cal}$ is the design value of the characteristic tensile strength of the unit parallel to the bed joint. Using the following:

$f_{bt,cal} = 0{,}025 \cdot f_{bk}$ for hollow bricks;

$f_{bt,cal} = 0{,}033 \cdot f_{bk}$ for perforated bricks and units with grip holes or grip pockets;

$f_{bt,cal} = 0{,}040 \cdot f_{bk}$ for bricks without grip holes o grip pockets;

f_{bk} is the characteristic compressive strength of the units (classification of the unit's strength according to DIN 1053-1);

F is the shear stress factor for the masonry unit. For masonry with a bed joint thickness of $12\ mm$ at least, $F = 1.7$, and in all other cases $F = 2.3$ should be used.

The theories provide also a combined failure for compression in combination with shear stress. But this failure mode is covered by other effects, like buckling. Therefore it isn't needed for des design.

Through the standardisation process the decision was made to use only the red framed equations and neglect the ductile effect in the standard. This restriction leads to assumption of a total collapse due to the first local failure.

3. Proposed Equation from Deliverable 4.3

Beside numerous tests in the context of the research project ESECMaSE some analytical considerations about the shear load capacity were made in a subproject. A s a result of this investigations *Graubner/Kranzler* proposed some equation in Deliverable 4.3 [19] and in [21]. The proposed equations are recapitulated in the following.

For failure due to bending it is proposed:

$$v_{bending} = \frac{1}{2 \cdot \lambda_v} \cdot \left(n - n^2\right) \tag{28}$$

With:

$n = \frac{N}{t \cdot l \cdot f}$ is the standardized vertical force;

$v = \frac{V}{t \cdot l \cdot f}$ is the standardized horizontal capacity;

$\lambda_v = \psi \cdot \frac{h}{l}$ is the shear slenderness;

ψ is a coefficient to describe the static system of the wall;

- $\psi = 1.0$ for cantilever systems;
- $\psi = 0.5$ for full fixed walls with full restraint at the top.

For failure due to gaping it is proposed:

$$v_{gaping} = \frac{l_b \cdot n}{2}\left(\frac{1}{h_b} + \frac{1}{h}\right) \tag{29}$$

Where:

h_b is the height of the masonry unit;

l_b is the length of the masonry unit;

h is the height of the wall.

For sliding:

$$v_{friction} = \mu \cdot n \tag{30}$$

Where:

μ is the friction coefficient.

For tensile failure of the unit it is proposed:

$$v_{J/S,unit} = \frac{1}{c} \cdot \overline{f}_{bt} \cdot \frac{1}{F^*} \left(\sqrt{C^2 + 1 + \frac{k \cdot n}{\overline{f_{bt}}}} - C \right) \tag{31}$$

Where:

$$C = \frac{h_b}{2 \cdot F^* \cdot l_{ol}}$$

$$F^* = F \cdot \left(0.6325 + f_{bt} \cdot 0.46 \frac{mm^2}{N} \right)$$

F is the shear stress factor for the masonry unit; for thin layer mortar F = 2.0; for general purpose mortar F = 1.7;

c is the factor to consider the shear stress distribution over the cross section c ≤0.5+λ_v≤1.5;

k is a factor to take the distribution of the vertical stress in the cross-section into account k = 1.05;

$\overline{f_{bt}}$ is the standardized tensile strength of the unit $\overline{f_{bt}} = \frac{f_{bt}}{f}$;

l_{ol} is the overlapping length.

After the verification with test results in [20] some verification were made.

Latest version from [21]:

$$v_{tensile} = \frac{1}{c} \cdot \overline{f_{bt,cal}} \cdot \left(F^*\right)^{-2} \left(\sqrt{1 + \left(F^*\right)^2 \left(1 + \frac{n}{\overline{\overline{f_{bt,cal}}}}\right)} - 1 \right) \tag{32}$$

Where:

$f_{bt,cal}$ is the splitting tensile strength for perforated units with more the 25% of holes; in other cases the horizontal tensile strength should be used.

$$F^* = 1.2 + 0.85 f_{bt}$$

For AAC:

$$v_{tensile} = \frac{1}{c} \cdot \overline{f_{bt,cal}} \cdot 2 \cdot \left(F^*\right)^{-2} \left(\sqrt{1 + \frac{\left(F^*\right)^2}{4} \left(1 + \frac{n}{\overline{\overline{f_{bt,cal}}}}\right)} - 1 \right) \tag{33}$$

For the code a non-standardized notation should be used.

$$V_{bending} = \frac{1}{2 \cdot \lambda_v} \cdot \left(N - \frac{N^2}{t \cdot l \cdot f} \right) \tag{34}$$

$$V_{gaping} = \frac{l_b \cdot N}{2} \left(\frac{1}{h_b} + \frac{1}{h} \right) \tag{35}$$

$$V_{friction} = \mu \cdot N \tag{36}$$

$$V_{unit} = \frac{1}{c} \cdot f_{bt,cal} \cdot l \cdot t \cdot \left(F^*\right)^{-2} \left(\sqrt{1 + \left(F^*\right)^2 \left(1 + \frac{\sigma_d}{f_{bt,cal}}\right)} - 1 \right) \tag{37}$$

$$V_{unit,AAC} = \frac{1}{c} \cdot f_{bt,cal} \cdot l \cdot t \cdot 2 \cdot \left(F^*\right)^{-2} \left(\sqrt{1 + \frac{\left(F^*\right)^2}{4} \left(1 + \frac{\sigma_d}{f_{bt,cal}}\right)} - 1 \right) \tag{38}$$

The main difference to existing standards is an added failure criterion. It described a overturning of a unit strut. This one is still knows for many years and investigated in some research projects. But for old masonry it was assumed as negligible due to the aspect ration.

Within the new proposal no bonding strength is needed for calculation. This main characteristic of these proposals makes the calculation for the load bearing capacities much easier, because the loos of bonding strength due to opening of joints has no effect on the calculated shear load capacity. But the shear load capacity will be underestimated for low loaded walls.

4. Comparison and discussion of the proposed equations

4.1. Bending

The verification of bending is included in all masonry standards. However, it normally relates only to bending out of plane or for systems like beams. Therefore the stress block is the basis for the design in EC 6 (see chapter 2.2). In DIN 1053-100 which also has the stress block as basis, wind panels are clearly listed as an application.

The use of the stress block with masonry is not without controversy. The several stone-mortar-combinations show a different behaviour in compressive experiments. A variety, from ductile to very brittle, can be noticed there, which also presents the determined strength-strain- relations in [12] and [13]. But the stress block shows an adequate approximation for a failure in small ranges like the support of a slab. There is also no differentiation between stress block and brittle material for the load capacity in case of a centric loading of a cross section without influence of slenderness. The influence only results from eccentric loading and larger compressed length. This applies to stiffening panels. Is the compressive strength due to in-plane-bending reached in the whole compressed part, the strain state at the external edges will be much higher than the failure strain of the masonry.

Another special characteristic of stiffening walls is the superposition of compressive with shear stress and respectively the resulting principal tensile stress. This can lead to an early failure in the limiting part regarding bending compression in comparison to a pure bending load.

As the bending failure mostly appear at the bottom or the upper wall edge in the region of the load introduction from the slab, the differences of the stiffness between masonry and reinforced concrete also have an influence. The reinforced concrete slab leads to a restriction of transverse strain in the masonry. Through this a biaxial compressive stress state in the masonry is built up, which leads to an increase of the load capacity. The slabs also cause a special load distribution, when e. g. small parts fail due to local material inhomogeneity or rotation of units.

The above-mentioned effects partially cancel out each other. Figures 3-3 to 3-7 in [19] show that with large shear slenderness. The shear load capacity of the numerical calculation lies between the calculated under the assumption of a stress block and the one under the assumption of a linear stress distribution. There the geometry of the unit has a minor influence as the case may be (see Figure 3-3 wall D_6_K). It could actually lie below the linear elastic approach using smaller shear slenderness (= longer walls).

D 4.4. shows the comparative calculations with the stress block.

$$V_{bending,sb} = \frac{1}{2\lambda_v} \cdot \left(N - \frac{N^2}{t \cdot l \cdot f} \right) \tag{39}$$

The shear load capacity without using the tensile strength and a linear stress distribution of the vertical stress results:

$$V_{bending,le} = \frac{1}{2\lambda_v} \left(N - \frac{4}{3} \frac{N^2}{t \cdot l \cdot f_k} \right) \tag{40}$$

The ratio of both is:

$$r = \frac{3(1-n)}{3-4n} \tag{41}$$

This ratio is shown in the following graph over the relevant load range.

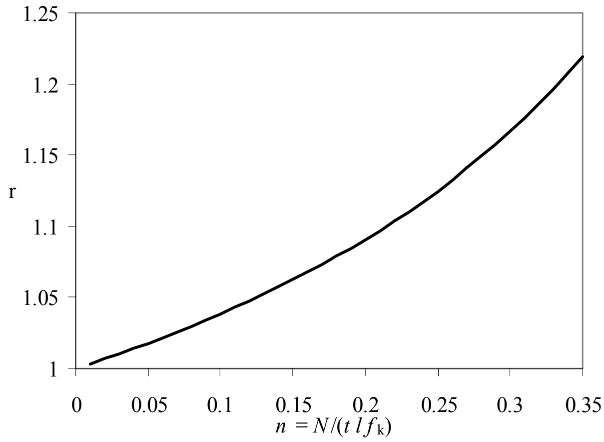


Figure 2 *Ratio between the shear design loads due to bending calculated from the stress block and linear elastic behaviour*

For small loads up to n=0.2 the difference between stress block and linear material behaviour is less then 10%. The use of stress blocks is here adequate.

As far as the design for bending under higher vertical loads only becomes important at large slenderness (=short walls) compared to other types of failure, and as with these walls there appears a ductile bending failure, the application of a stress blocks is an acceptable compromise.

The 10 % reduction of the stress block in Eq. 3-17 which is proposed alternatively in [19] would lead to a better correlation with the numerical results with smaller slenderness.

Another possibility to verify shear capacity of walls with low loads could be the use of the tensile strength. This was already done in [23] to recalculate experiments. In this approach a moment can be calculated using the following known equation (Navier) for the uncracked cross section together with the maximal stress at the edge, with is equal to the tensile strength.

$$\sigma = \frac{M}{I} \cdot z - \frac{N}{A} \qquad (42)$$

Were:

σ is the stress at the by z defined point of the cross section;

M is the moment;

I is the moment of inertia of the cross section;

z is the distance of a defined point from the centre of mass of the contributed cross section where the stress should be calculated;

N is the normal load;

A is the contributed area of the cross section.

Due to the fact that the edge stress of the cross section is given, the equation (42) can be converted to:

$$M = \left(\sigma + \frac{N}{A}\right) \cdot \frac{I}{z} \tag{43}$$

$$M = f_{x1} \cdot \frac{t \cdot l^2}{6} + N \cdot \frac{l}{6} \tag{44}$$

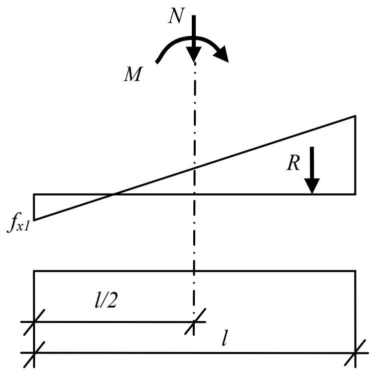


Figure 3 Schematic print of a cross-section with tensile strength

At the same time the second edge stress must be checked if the compressive strength is reached.

The shear load resistance can be calculated by the height of the wall and an existing restraint if applicable.

$$V_{bending,3} = \frac{M}{h \cdot \psi} = \frac{f_{x1} \cdot t \cdot l + N}{6 \cdot \lambda_v} \tag{45}$$

As from a defined load the bending load capacity of a cracked cross section (l_x) is higher. For this purpose the derivation of the required equations should be neglected here (see [19]).

The following graph shows an example of possible eccentricities depending on the uncracked length and the load.

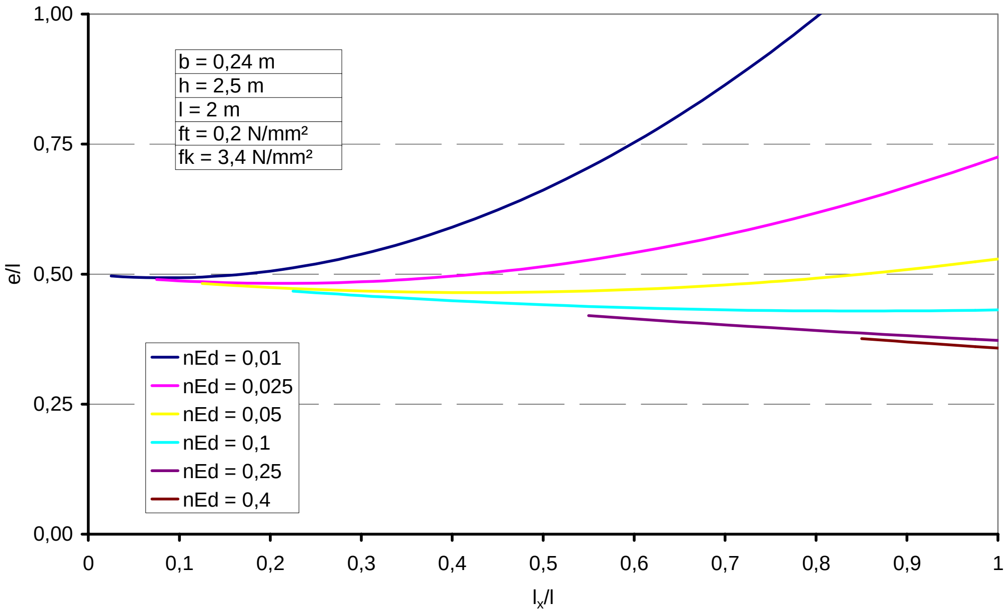


Figure 4 Eccentricity depending on the load and the uncracked length

It can clearly be seen that using tensile strength at the uncracked cross section up to a load factor of n = 0.05 result to a maximum-eccentricity and with it to a maximum moment.

Using a tensile strength for bending design can only lead to a larger shear load capacity when also bond strength is used for the other kinds of shear failure such as gaping and friction. Besides this, a layer like damp course is to be considered and defined within the design.

If not considering tensile strength, the self-weight of a wall can already give the required load for the shear load capacity.

For some of the tests which are summarised in [20] the measured shear load capacity is larger than the overturning load. Recalculating is only possible when using tensile strength.

Table 8 Test with an eccentricity greaten then l/2 from [20]

reference	unit	mortar	l	h	λ_v	ψ	SF	N_{obs}	V_{RTest}	max e
	Type		m	m	[-]	[-]		kN	kN	m
KS.D7.1a.13	HLz-opti2	tlm	1.10	2.50	1.14	0.50	SU	95	43	0.566
KS.D7.1a.14	HLz-opti2	tlm	1.10	2.50	1.14	0.50	SU	48	25	0.651
ZAG . BTW	HLz	LM5	2.4	1.75	0.73	1.001	SV	510	359	1.233
ZAG . 30.6	HLz	M 5	1.03	1.5	1.46	1.003	SV	183	65.92	0.542
60.1	KS	NM III	1.24	2.50	2.02	1.00	SV	109	40	0.920
ETH. ZW 3	V	NM	3.60	2.10	0.58	0.99	t- SV	417	380	1.903

4.2. Gaping

The type of failure gaping is already part of the theory of *Mann/Müller* [27]. Because of the fact that gaping becomes decisive only at a defined height of the unit and with unfilled joints, this type of failure could be left out in the design until now. As modern masonry increasingly uses large-sized units, it is necessary to consider this type of failure in the future standards.

The submitted proposal in [19] is based on a calculation of the load capacity by using a strut of masonry units. With the known vertical load the load capacity of the wall can directly be calculated. In doing so the bond strength between the unit at the mortar is neglected. In addition to the ratio of unit height to unit length (gaping I), the ratio of unit size to wall size (gaping II) should be considered. The dashed line in Figure 5 on the left corresponds to the case gaping I and the line drawn through to gaping II.

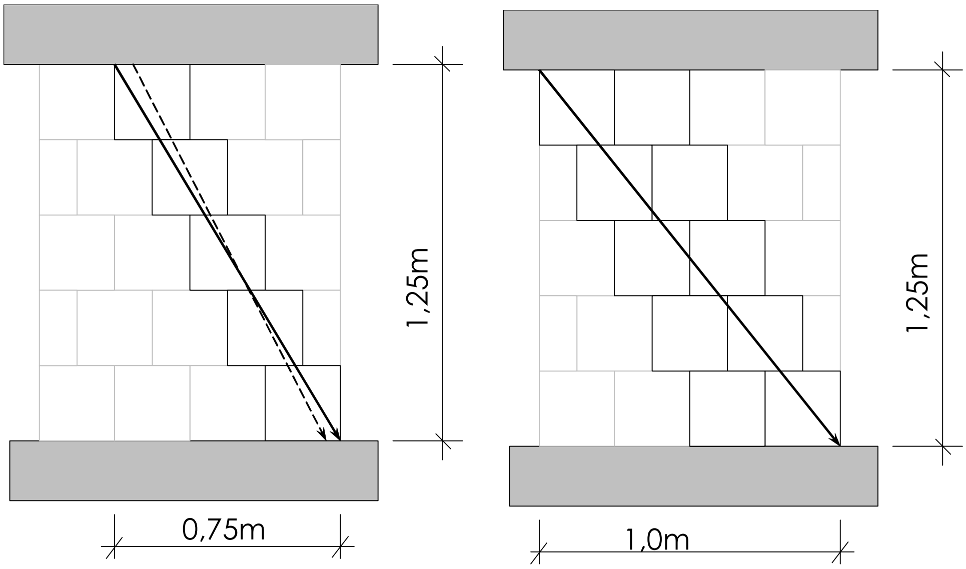


Figure 5 Critical inclination for gapping I and II and overturning

The equation of the load for gaping II is:

$$V_{gaping} = \frac{l_b \cdot N}{2}\left(\frac{1}{h_b} + \frac{1}{h}\right) \tag{46}$$

The image of the failure when reaching the shear load capacity is marked by the overturning of every unit strut. At this point the compressive stress at the overlapping area increases while the overlapping length decreases. A schematic diagram of this could be seen in Figure 6.

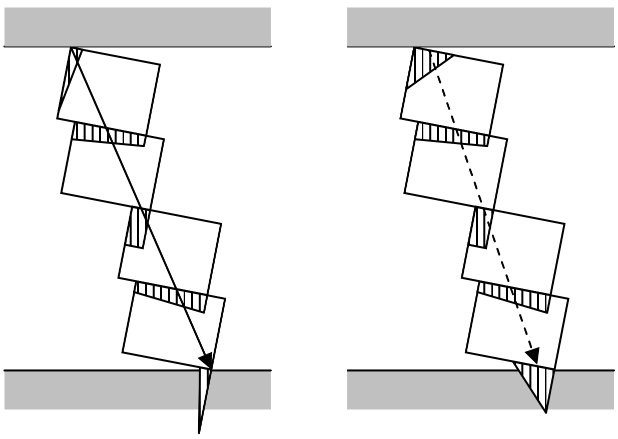


Figure 6 Schematic diagram of the compressive stress at the joints in one strut

At the first and at the last unit the stress peak becomes infinite for gapping II. This leads to a local compressive failure with plasticity like a stress block. The real strut behaviour will be somewhere between gapping I and II depending on the vertical load.

The proposed equations for gaping neglect the compression failure of each strut, but also the bond strength.

For regular bond and an overlapping length less then the half length of the unit, the shear capacity for gaping II can deviate from eq. (46) especially for very large units.

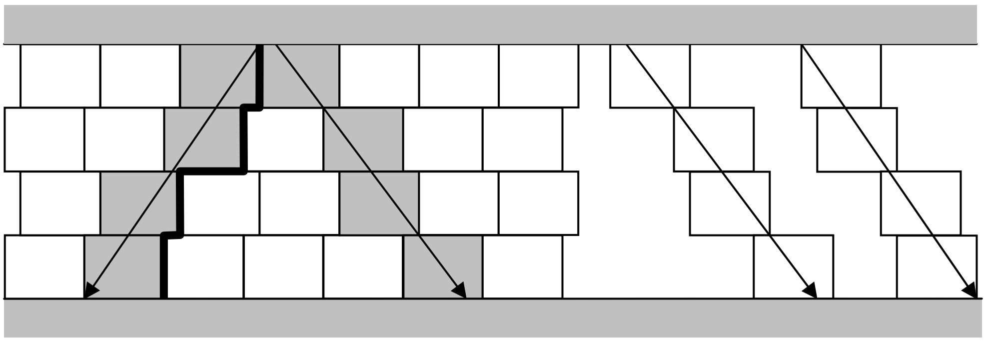


Figure 7 Inclination of the compressive struts for very large masonry units

In the example in Figure 7 a larger load capacity for one direction results while the load capacity in the other direction is minor. For walls with an even number of unit layers the unfavourable load capacity result to:

$$V_{gaping} = N\left(\frac{l_b}{2h_b} + \frac{l_{ol}}{h}\right) \qquad \text{for } \frac{h}{h_b} = 2,4,6... \tag{47}$$

The rule for an odd number of layers is eq. (46). The following graph shows the ratio of load capacity according to eq. (46) and (47).

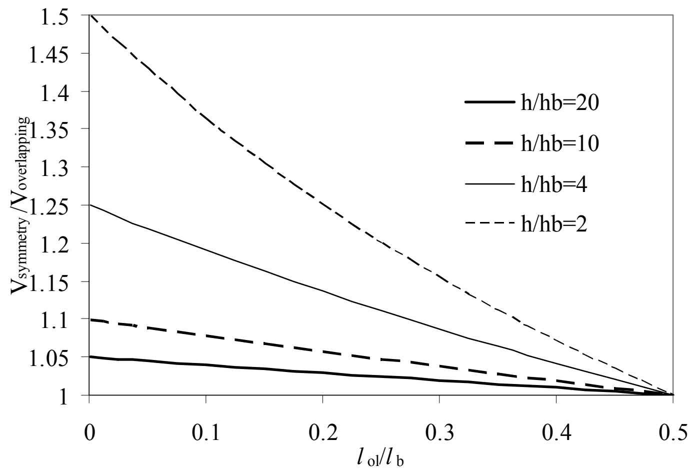


Figure 8 Comparison of eq. (46) and (47)

For common ranges of overlapping lengths and heights of units the overestimation of load capacity with eq. (46) for regular bond is lower than 10%

In [19] it is shown, that in case of gaping I and regular bond, no influence of the size of the overlapping area exists.

For cases of staircase bond with an overlapping length less then the half length of the unit *Graubner/Kranzler* propose eq. 3-36 in [19].

$$V_{ga,I} = N \cdot \frac{l_{ol}}{h_b} \tag{48}$$

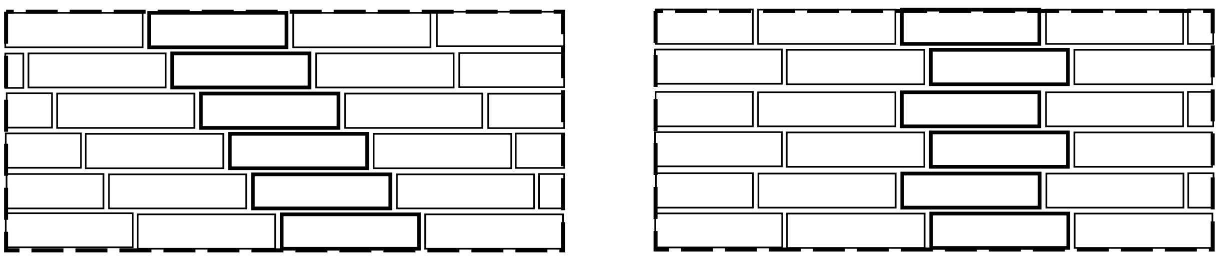


Figure 9 Staircase (left) and regular bond (right)

In addition to (48) an equation for gapping II can be developed for the staircase bonds.

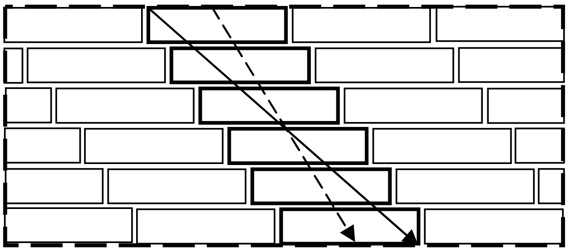


Figure 10 Gaping I and Gaping II in case of staircase bond

$$\frac{N}{V} = \frac{h}{\left(\frac{h}{h_b} - 1\right) \cdot l_{ol} + l_b} \tag{49}$$

$$V_{staircase,galI} = N\left(\frac{l_{ol}}{h_b} + \frac{l_b - l_{ol}}{h}\right) \tag{50}$$

For an overlapping length of the half length of the unit eq. (46) and (50) are identical. In the theoretical example the overlapping length can become zero.

As shown in Figure 11 the shear load capacity will decrease to:

$$V = N\frac{l_b}{h} \tag{51}$$

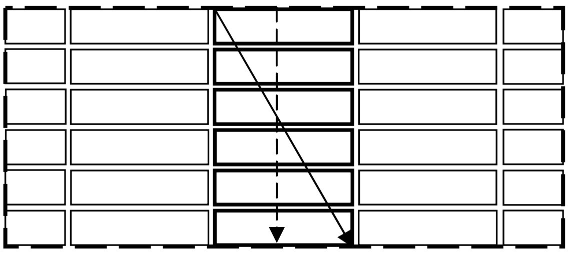


Figure 11 Regular and staircase bond for a theoretical overlapping length of zero

For this case the equation for the regular bond is not valid.

The following graph clarifies the difference of load capacity between regular bond and staircase bond depending on the overlapping length and the height of the stone.

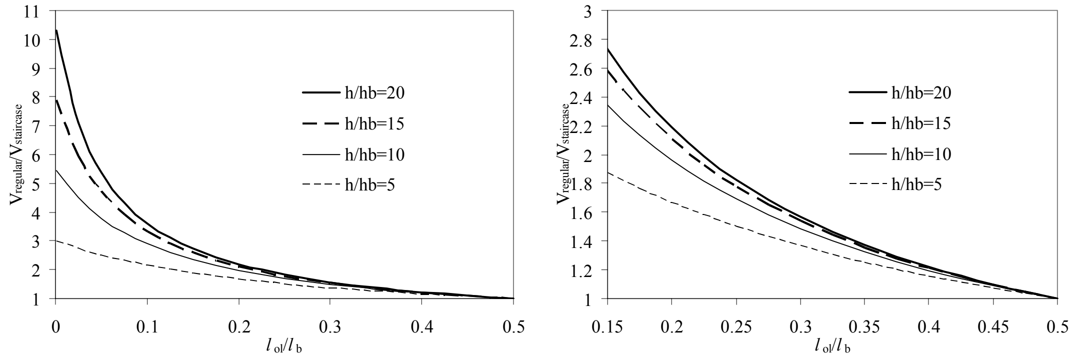


Figure 12 Comparison of load bearing capacity (right: the relevant range)

For staircase bond Eq. (50) includes eq Gl. (47) for regular bond with variable overlapping length.

The proposed equation also applies to dry masonry when the bond strength is neglected.

The numerical results in [19] show a good match to the proposed method. The calculation was mainly done with an overlapping of the half of a unit length. The sporadic calculations with a variable overlapping length were probably made with regular bond. So the proposed equation is still valid. But only one combination of materials were analysed.

Experiments within [22] correspond with this method at h/h_b=10 very well. However, at h/h_b=5 a shear load capacity of more than 12 % was analysed numerically and experimentally according to the proposed method.

Herein the resultant force lays between the load capacity due to overturning of the unit strut and overturning of the wall (see Figure 5). This is caused by the effect of tensile bond strength.

<u>Full restrained at the top of the wall</u>

According to Figure 2-14 in [19] it could be seen that the static system also influences the load capacity in the case of gaping failure. For a wall of 4 metres length the numerical calculation was made for the case of a fully restrained top of the wall and also a for a cantilever wall. This can also be observed in the numerical example further in this report (chapter 6.1).

So far it is not possible to take this into consideration with the proposed method.

Verification on the basis of shear strength

Considering the tensile bond strength for the case of gaping failure is only possibly by using the equation for shear strength instead equations for shear forces.

The equation by *Mann/Müller* [27] at gaping only considers masonry with an overlapping length of the half length of a unit.

$$f_{vk} = \left(f_t + \sigma_d\right)\frac{l_b}{2h_b} \tag{52}$$

By *Simon* [33] it was extended by a variable overlapping length.

$$f_{vk} = \left(f_t + \sigma_d\right)\frac{l_{ol}}{h_b} \tag{53}$$

The ratio of eq. (52) and (53) results in:

$$r = \frac{l_b}{2l_{ol}} \tag{54}$$

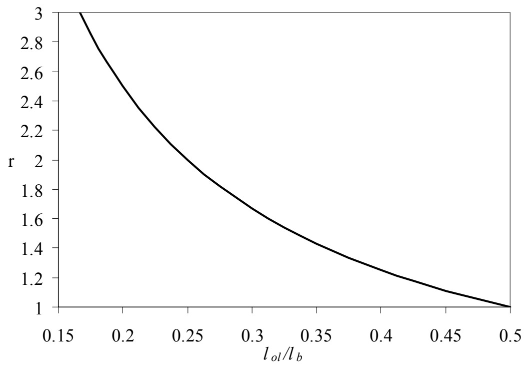


Figure 13 Relation of shear strength calculated by Mann/Müller and Simon

Equation (53) applies for the staircase bond. For regular bond a further development was carried out by *Jäger/Schöps* [22] (see the standard proposal in chapter 2.4 eq. (25).

$$r = \min\left(\frac{l_b}{2l_{ol}}; \frac{\overline{f_t} + n}{\overline{f_t} - \overline{f_t}\frac{l_{ol}}{l_b} + n}\right) \tag{55}$$

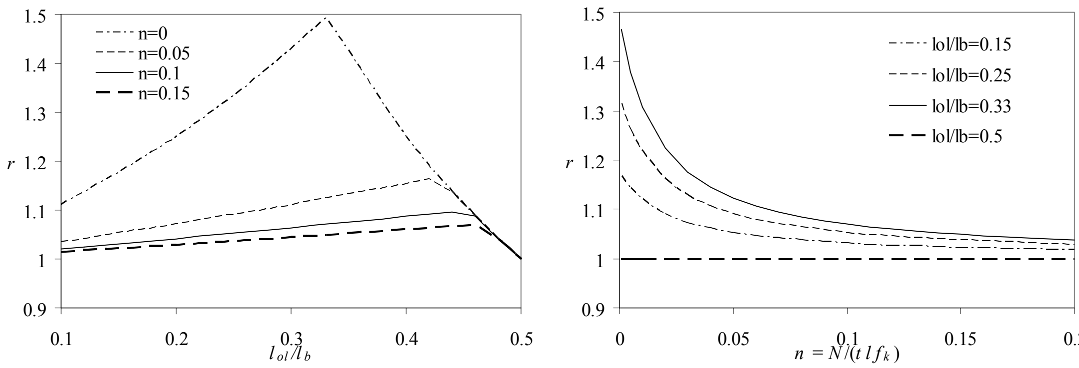


Figure 14 Relation of shear strength due to gaping of the units calculated by Mann/Müller and Jäger/Schöps (f_{x1}/f_k=0.025)

The largest overestimation of the shear strength due to the method of *Mann/Mü*ller results at a overlapping length of 1/3 of the unit size and zero load to 1.5. The overestimation already decreases significantly with low loads. The model of *Mann/Müller* is an acceptable simplification with corresponding safety factor.

In [19] figure 3-12 the shear load capacities according to *Mann/Müller* were already considered. Here, an overestimation of the load capacity when calculating in the centre of the wall with the common calculation of c, the factor for the shear stress distribution at the cross-section, can be seen. According to chapter 5 of this report a factor of c =1.5 for the centre of the wall would be more convenient. The following graph shows the load capacity in relation to the proposal in [19].

$$r = \frac{\overline{f_t} + n}{c \cdot n \left(1 + \frac{h_b}{h}\right)} \tag{56}$$

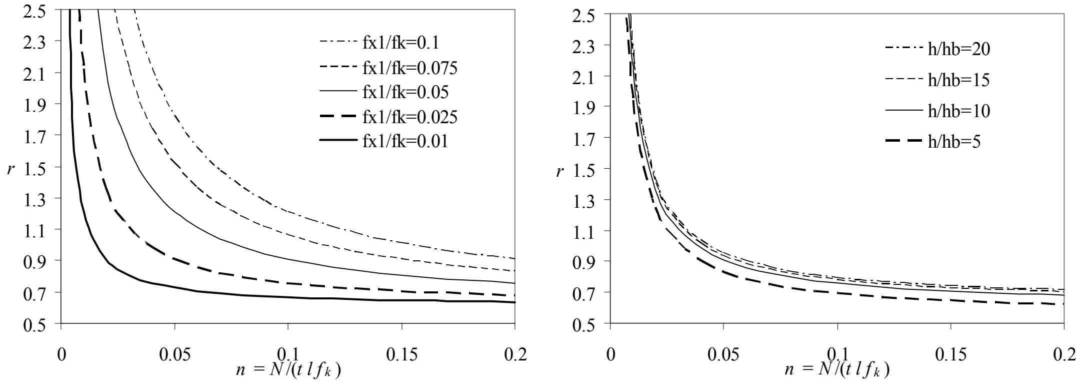


Figure 15 Relation between shear load capacity calculated by theory of Mann/Müller in the middle of the wall and the proposal [19] (c=1.5; left: h/h_b=10; right f_{x1}/f_k=0.025)

If the verification is done in the range of the minimal compressed length, the ratio will decrease. However, it should be checked first if c=1.5 is applicable. The following graph comes from c=1.25.

$$r = \frac{3\left(\overline{f_t} + n\right) \cdot h_b \cdot h}{\left(2h_b \cdot c \cdot n + 3\overline{f_t} \cdot l_b \cdot \lambda_v\right) \cdot \left(h + h_b\right)} \tag{57}$$

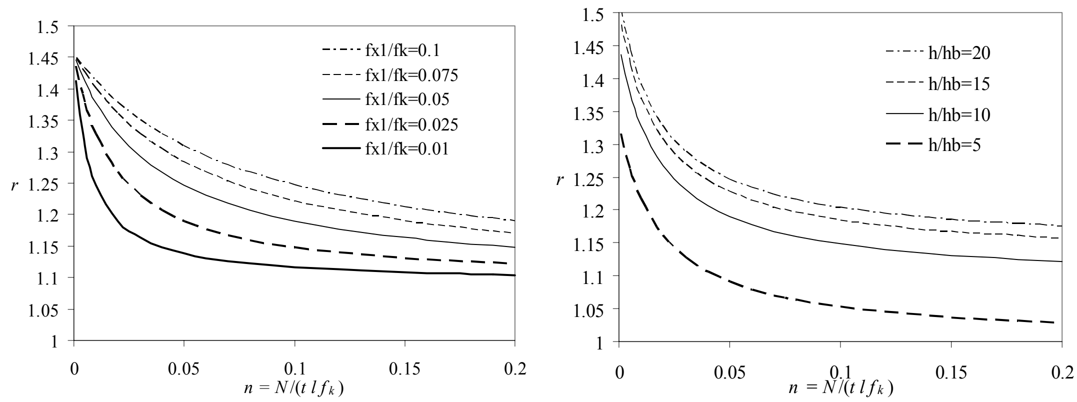


Figure 16 Relation between shear load capacity calculated by theory of Mann/Müller at the bottom of the wall and the proposal of [19] (c=1.25, λ_v=0.625; left: h/h_b=10; right f_{x1}/f_k=0.025)

The related tensile strength is 0.29/7.3 =0.04 in the numerical analysis in [18]. For masonry made of autoclave-aerated-concrete-units (AAC) with thin layer mortar a ratio of f_{x1}/f_k = 0.1 can be reached.

The reduction of the tensile strength in shear by stress peaks in the range of the head joints must be considered with a corresponding decrease (if applicable due to partial safety factor). But specific analyses are still necessary to define the variables.

The approach of *Mann/Müller* does not completely apply to the range of the slabs. The parabolic shear stress distribution according to the bending theory can not completely arise in this range. For this, further theoretical analysis for the failure model and the factor c need to be done.

Considering tensile bond strength for the gaping failure is a necessary base for considering tensile strength for the bending verification. Otherwise the bending verification for walls without load gives a certain shear load capacity but due to the verification of gaping, a load cannot be applied.

4.3. Friction

In the case of friction failure two versions have to be investigated. For the first version a sliding joint arises along a continuous bed joint. This can be caused by lower bond strength, eg. by a damp proof course. The outcome of the second version is a staircase displacement and the splitting of the wall in two sections (for one loading direction).

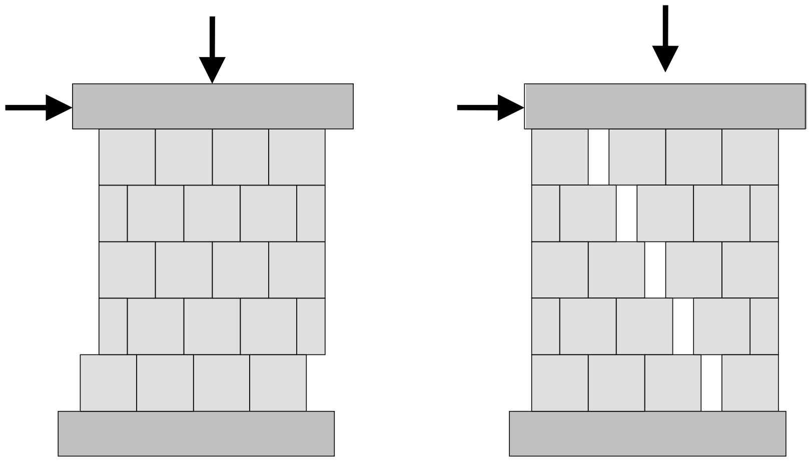


Figure 17 Variations of sliding failures

4.3.1. Version 1

The shear strength due to friction failure is fundamentally determined by the friction coefficient and the vertical loading in the first version. If the initial shear strength is taken into account, this can only be considered for the uncracked areas. Therefore the compressed length has to be calculated at first for the calculation of the load capacity.

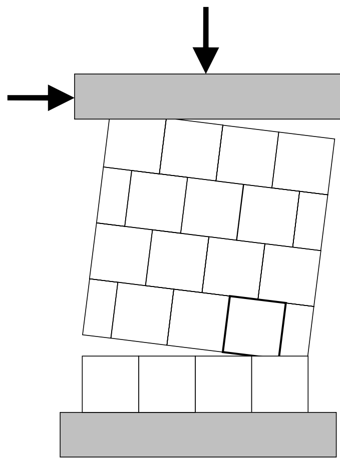


Figure 18 Reduction of the initial shear strength caused by opening of the bed joints

$$V_{Rd} = \frac{1}{\gamma_M \cdot c}\left(l_c \cdot t \cdot f_{vk0} + \mu \cdot N\right) \tag{58}$$

For the approach of the stress block the vertical stress is:

$$V_{Rd} = \frac{t \cdot l \cdot f_{vk0} + \mu \cdot N_{Ed}}{\gamma_M \cdot c + \dfrac{2 \cdot t \cdot h \cdot \psi \cdot f_{vk0}}{N_{Ed}}} \tag{59}$$

For a linear stress distribution the vertical stress is:

$$V_{Rd} = \frac{\frac{3}{2} t \cdot l \cdot f_{vk0} + \mu \cdot N_{Ed}}{\gamma_M \cdot c + \dfrac{3 \cdot t \cdot h \cdot \psi \cdot f_{vk0}}{N_{Ed}}} \leq \frac{1}{\gamma_M \cdot c}\left(t \cdot l \cdot f_{vk0} + \mu \cdot N_{Ed}\right) \tag{60}$$

In case of unfilled head joints the single rotation of the units lead to an asymmetric distribution of the vertical stress and the shear stress. Furthermore the shear stress distribution of the wall is not equal to the distribution of the shear strength. This fact has to be considered with the shear stress factor c.

The following figure shows the vertical stress at the bed joints of a wall under shear loading.

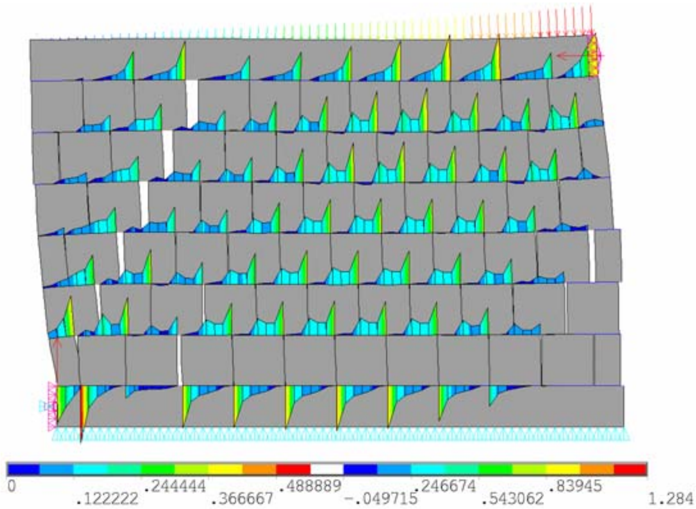


Figure 19 Vertical stress distribution at the bed joint under shear load (eccentricity load 2 in Figure 42)

Any change of load direction and opening of the cross section leads to the loss of the initial shear strength in the decisive area of the other load direction. In this case, only the friction part can be considered for the verification. This corresponds to the proposal in [19] (cp. Gl. (36)).

However, should an initial shear strength be taken into account the calculating engineer has to carry out the verification of the bond. This is conform with the verification of the strain in a corner of the wall according to DIN 1053-100. The excess of the tensile strength should be excluded with the limitation of the tensile strain in masonry to 0.1‰. Therefore the bonding remains.

The following charts show the ratio of the shear load capacities according to equation (60) and the proposal in [19].

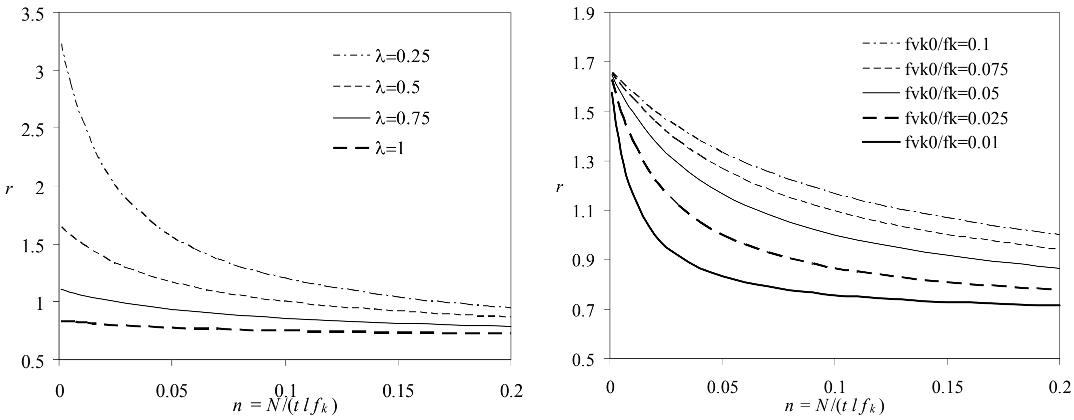


Figure 20 Relation between shear load capacity calculated by eq. (60) and eq. (36) (c=1.5, μ=0.6, γ_M=1.0, left: f_{vk0}/f_k=0.05; rigth: λ_v=0.5)

In the following charts a partial safety factor of 1.5 has been used. The parameters are the same as before.

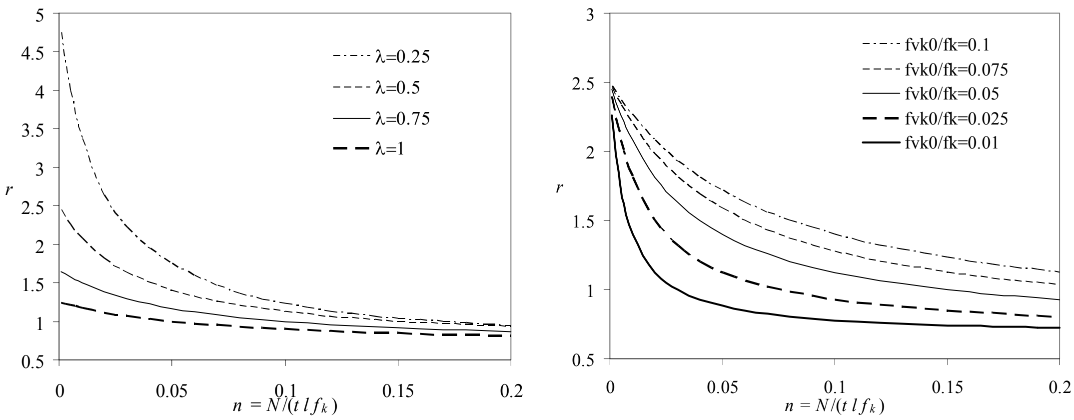


*Figure 21 Relation between shear load capacity calculated by eq. (60) and eq. (36) (c=1.5, μ=0.6, **γ_M=1.5**, left: f_{vk0}/f_k=0.05; rigth: λ_v=0.5)*

It can be seen that the approach of the initial shear strength leads to a larger load capacity in particular for small vertical loads and compact walls (small λ_v). By the inclusion of a parabolic shear stress distribution ratios smaller than 1.0 could arise for higher loads. Without the approach of the initial shear strength an ideal-plastic behaviour is expected according to the considerations in [19] (eq. (36)). Therefore the factor is set to 1.0. The increase of the vertical load leads to a decrease of the influence of the initial shear strength and the reducing of $\mu \cdot N_{\mathrm{Ed}}$ by the factor c leads to a ratio smaller than 1.0.

Because only few experimental results are available for this failure variant, no empirical equation can be derived. There remains just the analytical approach

4.3.2. Version 2

The theories of *Mann/Müller*, and for a smaller overlap the amplification of *Simon*, provide an approach for the failure beginning. At this the shear strength is exceeded at the lower compressed section of the unit and the wall splits itself in two parts. The masonry units at the top and the bottom of the resulting crack do not fail due to friction (cp. Figure 3-47 in [19]). Here it comes to unit failure because the vertical load is higher in these sections than in the sub areas with cracks. For a complete description of a failure there are still not enough research results available. That is why the failure is described in the following the beginning. Figure 6 shows schematically the influence of the bond.

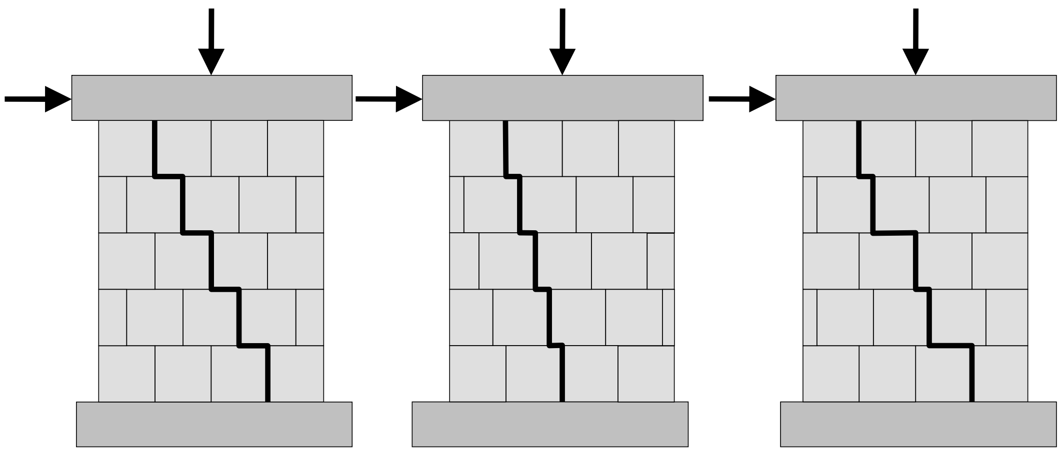


Figure 22 Theoretical approach (left: Mann/Müller; middle: Simon; right: Jäger/Schöps eq. (26))

According to *Mann/Müller* [27] the shear strength for the first crack is:

$$f_{vk} = \frac{(f_{vk0} + \mu \cdot \sigma_d)}{1 + \mu \cdot 2 \cdot \frac{h_b}{l_b}} \tag{61}$$

For the staircase bond (Figure 22 middle) the shear strength according to *Simon* [33] is:

$$f_{vk} = \frac{\left(f_{vk0} + \mu \cdot \sigma_d\right)}{1 + \mu \cdot \frac{h_b}{l_{ol}}} \tag{62}$$

In the case of a regular bond (Figure 22 right), this approach does only apply for every second layer. The overlapping length for the remaining layers is l_b-l_{ol} (see proposal *Jäger/Schöps* eq. (26)). After failure of the shorter overlapping length it comes to a rearrangement to the larger. The precise connection has yet to be investigated.

The approach of Simon will be used for the further considerations. For the regular bond, the half unit length comes up to the overlapping length. This corresponds to the approach based on the theory of *Mann* and *Müller*.

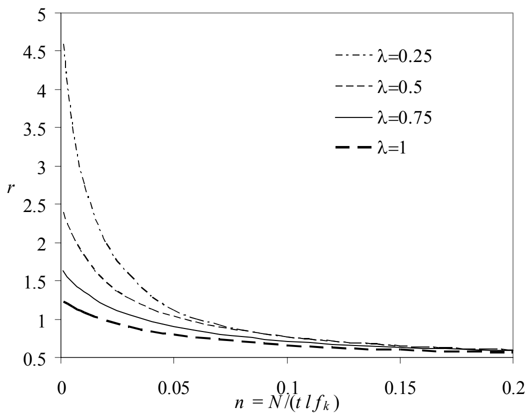
Using the approach above the shear load capacity for a linear-elastic stress distribution is:

$$V_{Rd} = \frac{t \cdot l_c \cdot f_{vk}}{\gamma_M \cdot c} \tag{63}$$

$$V_{Rd} = \frac{1}{\gamma_M \cdot c} \frac{l_c \cdot t \cdot f_{vk0} + \mu \cdot N}{1 + \mu \cdot \frac{h_b}{l_{ol}}} \tag{64}$$

$$V_{Rd} = \frac{\frac{3}{2} t \cdot l \cdot f_{vk0} + \mu \cdot N_{Ed}}{\gamma_M \cdot c + \gamma_M \cdot c \cdot \mu \cdot \frac{h_b}{l_{ol}} + \frac{3 \cdot t \cdot h \cdot \psi \cdot f_{vk0}}{N_{Ed}}} \leq \frac{1}{\gamma_M \cdot c} \frac{l \cdot t \cdot f_{vk0} + \mu \cdot N}{1 + \mu \cdot \frac{h_b}{l_{ol}}} \tag{65}$$

In the following charts the shear load capacity obtained in this way is compared to the approach without initial shear strength.



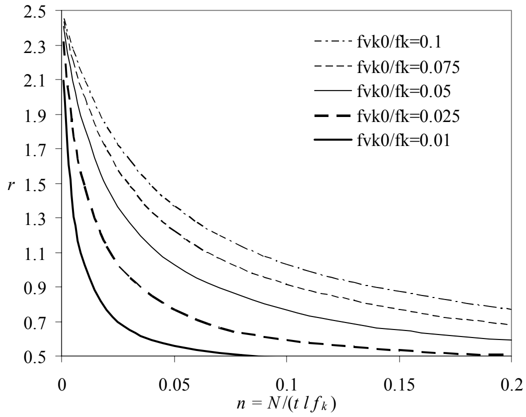


Figure 23 Relation between shear load capacity calculated by eq. (65) and eq. (36) (c=1.5, μ=0.6, γ_M=1.5, h_b/l_{ol}=1.0 left: f_{vk0}/f_k=0.05; rigth: λ_v=0.5)

In analogy to version 1, there is a higher shear capacity for small vertical loads and small shear slenderness. However, the advantage decreases rapidly with increasing load.

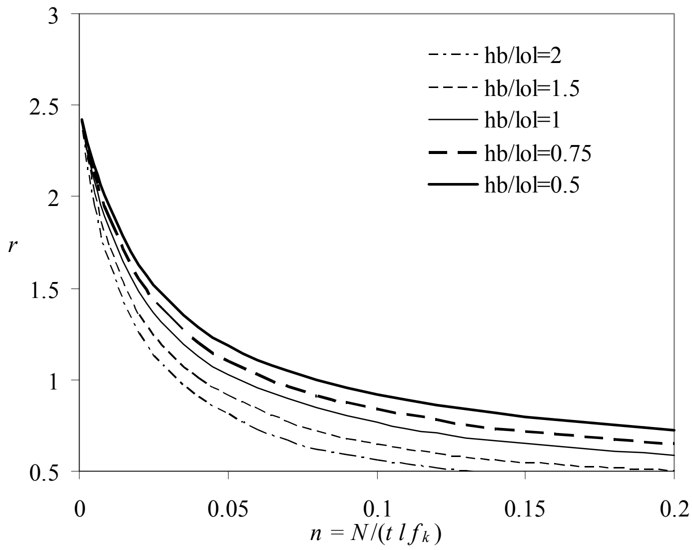


Figure 24 Relation between shear load capacity calculated by eq. (65) and eq. (36) (c=1.5, μ=0.6, γ_M=1.5, f_{vk0}/f_k=0.05, λ_v=0.5)

Partially, there exist considerably smaller ratios than 1.0 if the parabolic shear stress distribution and the influence of the unit geometry are regarded.

For a calculation in the middle of the wall, approaching the whole length of the wall, the ratio is:

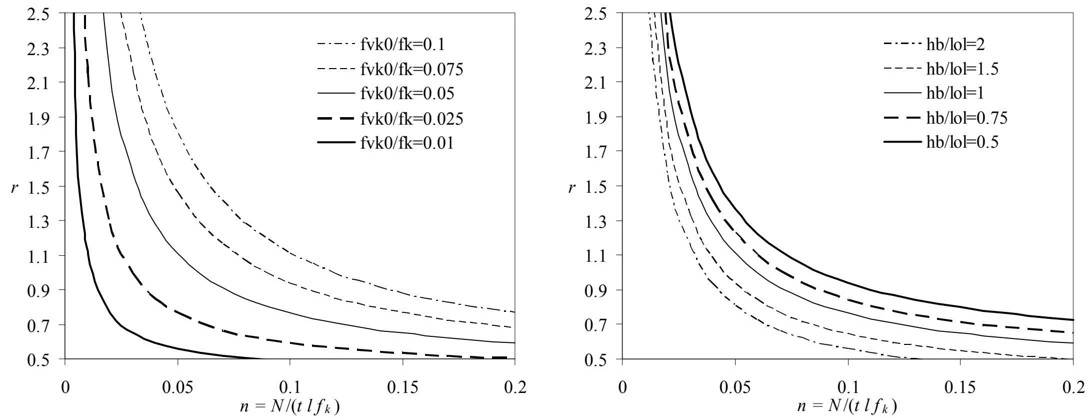


Figure 25 Relation between shear load capacity calculated by eq. (65) and eq. (36) (c=1.5, μ=0.6, γ_M=1.5, λ_v=0.5, left: h_b/l_{ol}=1.0; right: f_{vk0}/f_k=0.05)

The larger effective length of the cross section mainly affects the quota of the initial shear strength and therewith leads to a higher ratio for smaller vertical loads

Because mainly tests with small vertical loads show a failure of the joints, the approach of the initial shear strength can yield a higher load capacity.

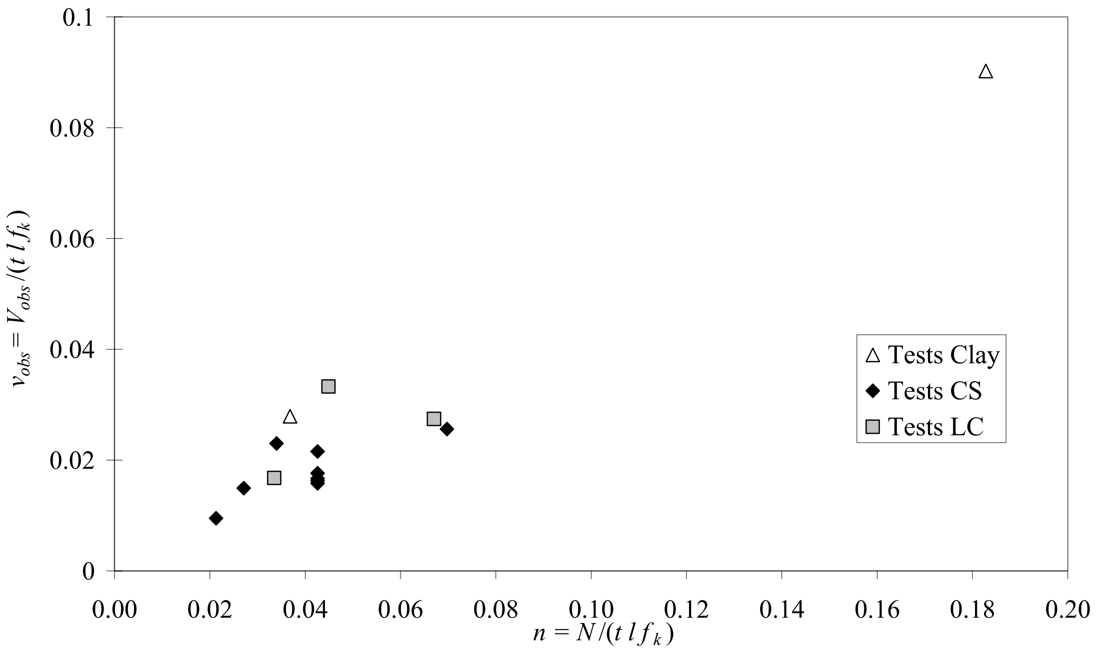


Figure 26 Test with documented joint failure

In fact, in most tests mixed pictures of failure occur.

Because the considered fact indicates only the first crack, but the final load capacity is determined by the sliding friction along the crack and by the load capacity of the external bricks, the entire load bearing capacity of the shear wall can thoroughly be higher. This failure requires a further analysis, as it was also discussed and established in chapter 3.5 in [19].

For longer walls the shear strength can be higher due to friction, when the staircase crack occurring by friction failure misses the corner unit. Therefore the partial friction failure was considered by *Jäger/Schöps.* The second equation (26) has to be considered.

The recommendation in [19] delivers a simple proposal for the shear verification without initial shear strength. If necessary, the friction coefficient has to be reduced as a result of the shear stress distribution, for masonry without an effective bond (like dry masonry) because the additional safety of the bond is missing.

For a verification based on the strength the initial shear strength can be used. There, however, the values for the initial shear strength and the friction coefficient have to be reduced in order to consider the failure of version 2. The reduction ratio can be calculated in the following way:

$$F = \frac{1}{1 + \mu \cdot \frac{h_b}{l_{ol}}} \tag{66}$$

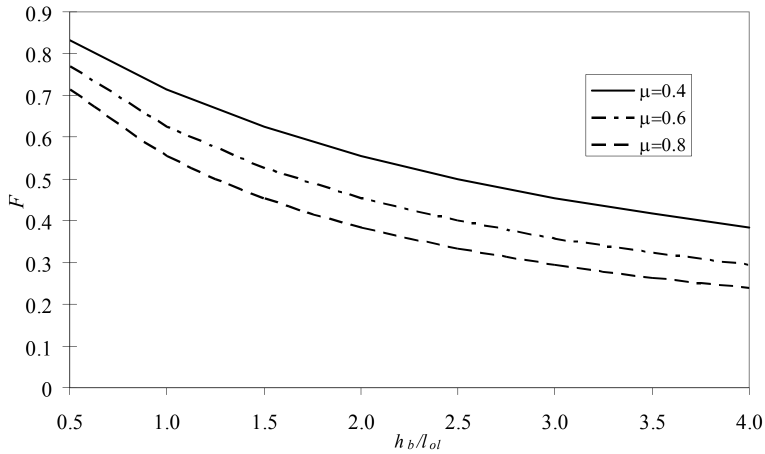


Figure 27 Reduction factor depending on the relation of unit height to overlapping length

The equation of the EC 6 for masonry with unfilled head joints results for the case of μ=0.8 and h_b/l_{ol} =1.25 (cp. eq. (13)). The simplification according to DIN 1053-100 is based on μ=0.65 and h_b/l_{ol} =1.0. Masonry made of large-sized units and with staircase bond requires an additional reduction. However, it is waived, because the influence of the corner unit is unexplained and the final load capacity can be higher, as explained above. Moreover, the ratio h_b/l_{ol} should be restrained.

4.4. Tensile failure of the unit

While the previously discussed failure modes bending, gaping and friction – mainly analytical failure models for the description of the shear failure – are forming the base of the standards, an empirical model and an analytical model (DIN 1053) follows for the failure due to principal tensile stresses in the masonry units. According to DIN 1053-1 the first principal stress are used as criterions (eq. (21)) while the EC 6 considers the failure of tensile strength of the unit only with a fixed boundary value, which depends on the compressive strength of the unit (see chapter 2.2). Besides to the dependence on the tensile strength of the unit the shear strength is also a function of the vertical load. For the definition of the tensile strength of the unit, the pattern of the holes has to be considered in addition to the compressive strength.

The proceeding of *Graubner/Kranzler* in [19] and [20] are equal to a combination of an analytical model and an empiric adjustment. The proposed equations (see eq. (31) till (33)) are based on an analytical model (*Jäger/Schöps* [22]) in which the parameters first were adjusted to numerical results and afterwards to test results. As an advantage of the numerical proceeding, different influencing factors can be investigated under controlled boundary conditions.

The equations of this method are more complex compared to the approach of the EC 6. A simplification is urgently necessary for a normative suggestion. Hereinafter some possible influencing factors should be considered.

4.4.1. Influence of the vertical load

During most experimental shear tests only two ore three load steps are investigated. The failure cases of it are mostly different. It may be that the one is bending and the other one a tensile failure of the unit. Against it, any number of load steps can be investigated with numerical considerations. The numerical results from [19] show the nonlinear increase of the shear capacity with increase of the vertical load and failure of tensile strength of the units. Concerning this matter, the criterion of the principal tensile stress is therefore applicable for the failure in the stone, as based on the analytical derivations of *Mann/Müller* in [22] and [27].

The following chart shows the tests arranges in [20] depending on the vertical stress level with tensile failure of the units and full restraint support at the top.

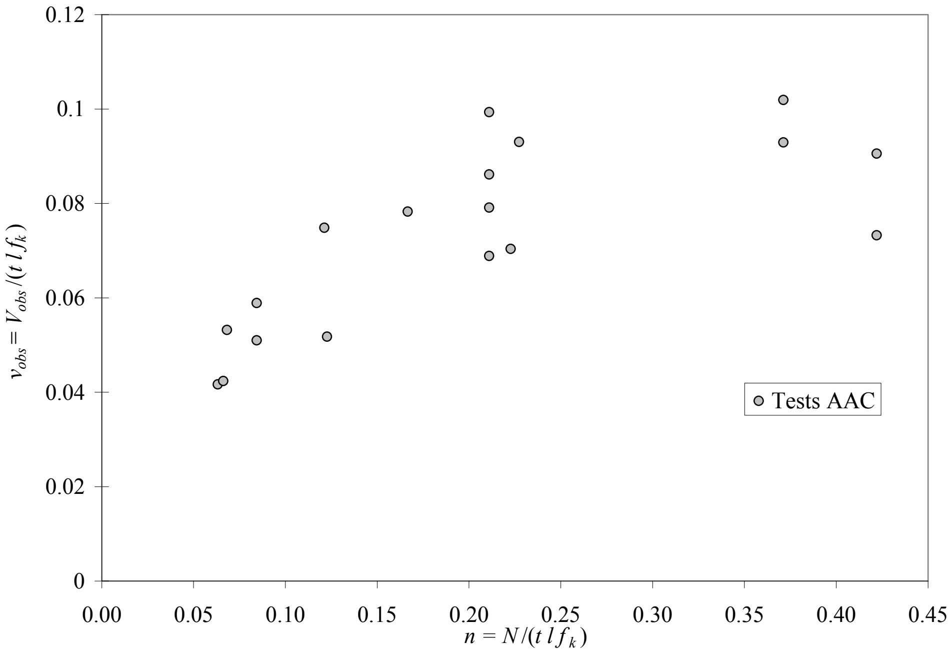


Figure 28 Relative shear load capacity depending on the vertical stress level for AAC units and tensile failure of the units (full restraint support at the top)

Analogue to the numerical investigation there is also a dependence on the vertical load. There is a non-linear coherence for the example of AAC.

4.4.2. Influence of tensile strength of the unit

The numerical comparative analysis in [19] shows for a tensile strength of the unit of 1.12 N/mm² an acceptable coincidence with the existing theoretical models. For the calculations with the reduced tensile strength of the unit, 0.2 N/mm ² instead of 1.12 N/mm², partially significantly higher capacities were revealed, than the theoretical models of *Mann/Mueller* [27] and *Jäger/Schöps* [22] have shown (see pictures 3-29 to 3-32 and 3-37 to 3-40 in [19]).

Therefore it could be noted, that at the level of input parameters for numerical calculation only the tensile strength of the unit changed. The other parameters, like modulus of elasticity, compressive strength and the defined crack energy, are equal in both calculations. Therefore the masonry with the lower tensile strength of the unit shows a significantly ductile behaviour. Not until the quintuple crack width is reached does the tensile strength at the crack growth become zero. With more ductile behaviour, similar to the theoretical plastic hinge, larger forces can be transferred. This is known from the investigations on the bending tensile strength. This fact has maybe also an influence on the failure of tensile strength of the units. Therefore comparative analyses with reduced cracking energy have to be carried out.

In [19], only two values were available for the adjustment of the equations for the unit failure with the tensile strength of the unit (f_{bt} = 0.2 N/mm² and 1.12 N/mm²). Because of the higher shear capacity at a smaller tensile strength of the unit an increase of the equation in the lower range of f_{bt} was carried out. Compared with the experimental results in [20] for masonry with a small tensile strength of the unit (AAC) a reduction was implemented again.

The following chart shows the test results depending on the standardised tensile strength of the unit.

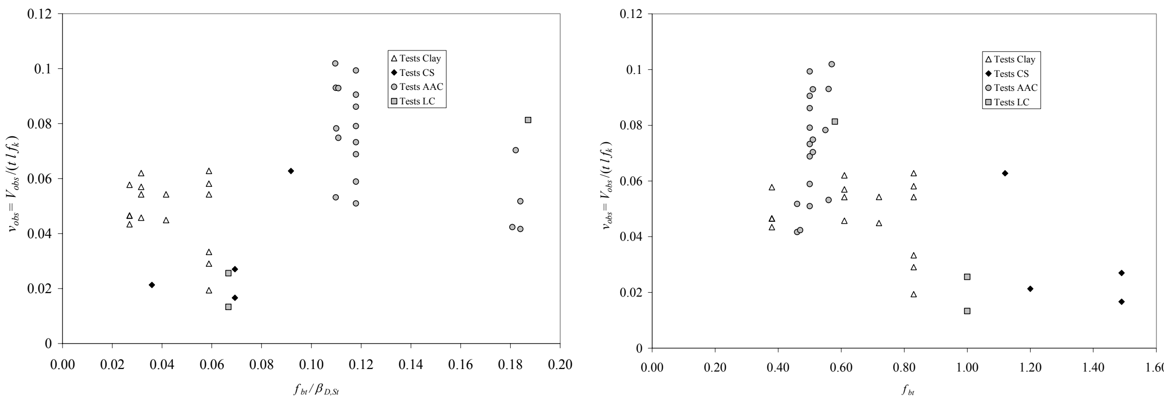


Figure 29 Relative shear load capacity depending on the tensile strength of the masonry units (left: relative to the compressive strength of the unit)

In the following chart the ratio of the standardised tensile strength of the unit to the standardised vertical load is shown, because the influence of the tensile strength of the unit is superimposed with the influence of the vertical loads.

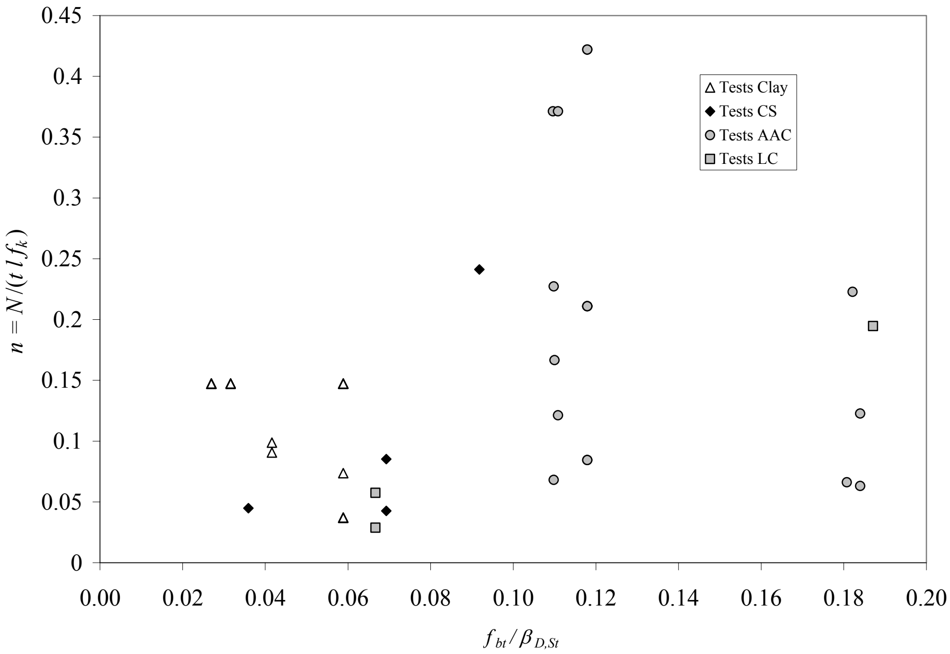


Figure 30 *Relation of the relative tensile strength of the unit and the used vertical stress level in the tests*

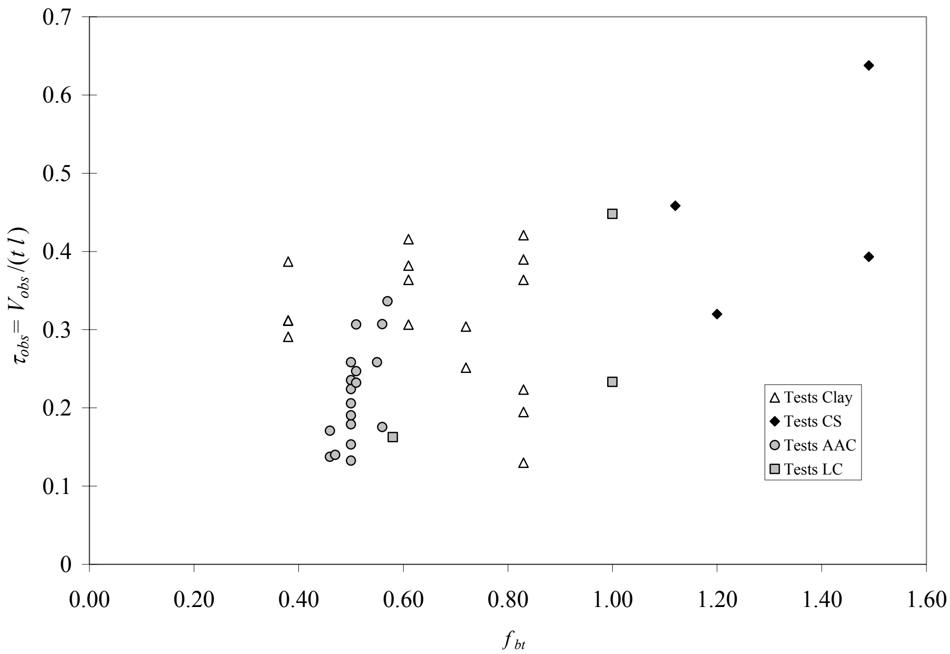


Figure 31 *Shear load capacity depending on the tensile strength of the masonry units*

In Figure 31 a basic dependence of the shear capacity from the tensile strength of the unit can be recognized. A more precise relationship to the unit- or masonry compressive strength (see Figure 29) can not be established with the present experimental results. Therefore further investigations are necessary.

4.4.3. Influence of filled head joints

For the failure due to exceeding of the tensile strength of the unit within the numerical comparative analysis a significant increasing of the load-bearing capacity can be recognized for masonry with filled head joints (cp. [19]). The difference between filled and unfilled joints exists only in the contact. This means that the face sides of the bricks do not contact each other in case of unfilled head joints. The initial shear strength was not defined in case of filled head joints.

Within the ESECMaSE-project mainly masonry with unfilled head joints was investigated. The influence of a contact of the face sides of the units on the shear capacity can not be fixed by experimental data. For one test with filled head joints and AAC in [16] a significant increasing of the load-bearing behaviour could be recognized (ca. 20%).

While within DIN 1053 filled head joints are not considered for the tension failure of the unit, the boundary value for filled head joints in EC 6 can be raised up to 44% ($0.065f_b/0.0.45f_b$).

The number of tests is not sufficient for a separate evaluation. Already ongoing research projects will deliver more technical expertise in the next time.

4.4.4. Influence of overlapping length / geometry of the units

According to the results in [19], the size of the unit or the overlapping length have only a marginal influence on the shear capacity due to tension failure of the unit. Two test with a reduced overlapping length made by *Fehling/Stürz* [15] showed contrary to expectations an increasing of the load up to 25%. For these experiments the regular bond was used. The transferability to the staircase bond still has to be investigated if necessary.

Based on the results of ESECMaSE, currently the consideration of the geometry of the stone or the overlapping length is not necessary for the shear verification due to tension failure of the unit.

4.4.5. Influence of the ratio between unit and wall

In the investigations in [22] of about the influence of the ratio between unit and wall geometry greater capacities for large stones were measured.

The results of the numerical simulation of the walls D_1 and D_11 in [19] gave slightly smaller capacities on the other hand, when using large stones (see Figure 3-25 and 3-28). For large sized units it seems that the aspect ratio of the unit have now an impact on the results (see D_8 and D_11). Within ESECMaSE in Pavia also tests with KS Quadro-elements were made. The pictures show a joint failure in combination with a bending failure.
A clear statement could not yet be given. Therefore a consideration for the standard will be omitted in this report.

4.4.6. Influence of a full restrained support at the top of the wall

The determination of the shear load capacity as a result of unit failure by half a wall height after eq. (37) and (38) leads to the same shear loads capacities for a full restrained top of the wall and for the static system of a cantilever. Only a few comparative tests results are available for that.

Thus different kinds of failure were decisive at the experiments 7 and 8 from Pavia (cp. Figure 3-11 in [20]). The same applies to the tests 1, 2 and 6 from Pavia (cp. Figure 3-12 in [20]). With the tests 19 and 22 of the University of Kassel tensile failure of the units or bending failure appeared with the same geometry and load. The load of the wall with full restraint support was twice as big, on this occasion (cp. Figure 3-8 right and Figure 3-9 in [20]). The test values lie both just under the curve for the bending failure.

As a clue for the influence can be used the numerical investigations in [19] on the wall D_3. In Figure 2-14 both static systems are confronted with each other. The difference amounts to 20%-25%.

For the consideration of this influence the calculated wall length can be used (see chapter 6.2). The factor c for the shear stress distribution defined by the shear slenderness is not suited (see chapter 5).

4.4.7. Alternative equations

Alternatively to the proposed equations the equation given in DIN 1053-100 can also be modified.

$$f_{vk} = \frac{f_{bz}}{F}\sqrt{1+\frac{\sigma_{Dd}}{f_{bz}}} \tag{67}$$

The factor *F* reflects the shear stress distribution in the masonry unit. According to *Mann/Müller* *F* =2.3 was defined. Therefore the factor in eq. (21) is 1/ *F* =0.45.

If the statement in [19] is taken into consideration that the failure begins in middle of the wall the equation for the load-bearing capacity results to:

$$V = l \cdot t\,\frac{f_{bz}}{c \cdot F}\sqrt{1+\frac{\sigma_{Dd}}{f_{bz}}} \tag{68}$$

If the factor *F* is used as a free parameter the equation can be adapted about this parameter.

$$V = l \cdot t \cdot \frac{1}{c} \cdot \alpha \cdot f_{bz}\sqrt{1+\frac{\sigma_{Dd}}{f_{bz}}} \tag{69}$$

For the proposed equations in [21] (cp. eq. (32) und (33))) the parameter becomes to α = 0.394 (R^2=0.77) for the general masonry and α = 0.292 (R^2=0.73) for AAC.

By an evaluation of the equation for the test carried out up to now and collected in [19] the parameter results for the general purpose mortar to α/c = 0.28 (R^2=0.29) and for AAC to α/c = 0.294 (R^2=0.65). On this occasion, only test with a tensile unit failure, unfilled head joints and a zero moment near the wall middle were used. The determination of the kind of failure, tensile failure of the unit or compressive failure due to bending is not clear for test without moment at the wall head (cantilever). Hence, in case of the general consideration carried out here these tests are not taken into consideration.

For the estimation of the adaptation quality the coefficient of determination is given.

A little better adaptation could be made by using a second free parameter.

$$V = l \cdot t \cdot \frac{1}{c} \cdot \alpha \cdot f_{bz} \sqrt{1 + \frac{\sigma_{Dd}}{\beta \cdot f_{bz}}} \tag{70}$$

Table 9 Parameter curve fitting eq. (70)

	by eq. (32) und (33)		by tests (incl. 1/c)	
	Common	**AAC**	**Common**	**AAC**
α	0.219	0.103	0.00705	0,247
β	0.220	0.064	0.00031	0,563
R^2	0,88	0,95	0.67	0,67

Further simple equations for a statistical adaptation are:

$$V = l \cdot t \cdot \frac{1}{c} \cdot \alpha \cdot \sigma_{Dd}{}^{\beta} \cdot f_{bt}{}^{\delta} \tag{71}$$

Table 10 Parameter curve fitting eq. (71)

	by eq. (32) und (33)		by tests (incl. 1/c)	
	Common	**AAC**	**Common**	**AAC**
α	0.505	0.353	0,383	1,032
β	0.365	0.501	0,590	0,282
δ	0.323	0.324	0,143	2,065
R^2	0,96	0,98	0.88	0,79

and

$$V = l \cdot t \cdot \frac{1}{c} \cdot \left(\alpha + \beta \cdot \sigma_{Dd} + \delta \cdot f_{bt}\right) \tag{72}$$

Table 11 Parameter curve fitting eq. (72)

	by eq. (32) und (33)		**by tests (incl. 1/c)**	
	Common	**AAC**	**Common**	**AAC**
α	0.069	0.011	0,084	-0,285
β	0.148	0.142	0,236	0,101
δ	0.270	0.233	0,051	0,866
R^2	0,98	0,99	0.85	0,73

Table 12 Parameter curve fitting eq. (72) without α

	by eq. (32) und (33)		**by tests (incl. 1/c)**	
	Common	**AAC**	**Common**	**AAC**
α	-	-	-	-
β	0.164	0.144	0.270	0.115
δ	0.327	0.253	0.107	0.212
R^2	0.96	0.99	0.80	0.65

The best result by the adaptation to the test results arose for an equation with free exponents (eq. (71)). This equation is the same function as for the calculation of the masonry compressive strength. Indeed, the coefficient of determination of the AAC masonry is clearly lower as for the general masonry. This could also be seen at the parameter δ, which corresponds to a nearly quadratic dependence of the shear load capacity on the tensile strength of the unit. An adaptation of the equations for AAC masonry to the tensile strength of the unit is not possible with the available data, as could already be seen in the diagram in Figure 29 right.

Variant EC 6

According to the EC 6 the existing regulations, with one limiting value, should be used for a approach in the following.

Related to the tensile strength of the unit

$$V = l \cdot t \cdot \alpha \cdot f_{bt} \tag{73}$$

Table 13 Parameter curve fitting eq. (72)

	by tests (incl. 1/c)	
	Common	**AAC**
α	0.384	0.428
R^2	0,0	0,24

Related to the mean compressive strength of the unit

$$V = l \cdot t \cdot \alpha \cdot f_b \tag{74}$$

Table 14 Parameter curve fitting eq. (72)

	by tests (incl. 1/c)	
	Common	**AAC**
α	0.019	0.052
R^2	0,0	0,31

Addition related to the compressed length

In [5] the load capacity relates to the compressed length. Thus, the smallest compressed length without any tensile strength would be used for the fitting in the following:

$$V = l_c \cdot t \cdot \alpha \cdot f_b \tag{75}$$

Table 15 Parameter curve fitting eq. (75)

	by tests (incl. 1/c)	
	Common	**AAC**
α	0.043	0.09
R^2	0,0	0,22

As far as using the mean compressive strength of units in the verification procedure would also result a mean value for the resulting shear strength, the conversion to a characteristic strength has to be realised by with the functional parameters. This approach is equal to the definition of the compressive strength of masonry in EC 6. In the following the parameter α was decreased and consequently a certain number of experiments lie below the theoretical load capacity.

Table 16 Parameter curve fitting eq. (75) for 5% Quartile (hand fitted)

	by tests (incl. 1/c)	
	Common	**AAC**
α	0.032	0.063

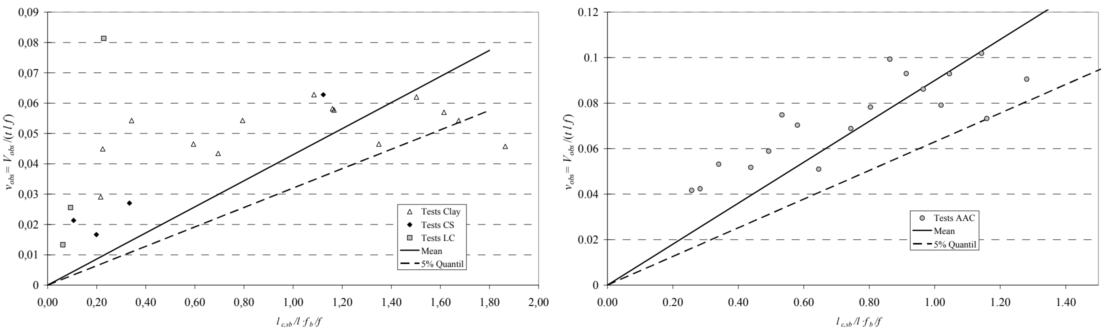


Figure 32 Equation (75) with parameters from Table 15 and (17) (left: common; right: AAC)

The approach of the EC 6 underestimated the load capacity for low relations of $\frac{l_{c,sb}}{l} \cdot \frac{f_b}{f}$.

Some walls may fail due to a bending in this range. Especially the test with light weight concrete differs upwards.

But as there are only 18 experimental values for AAC, none of the values should lie under the curve. Thus, the curve for the characteristic values results from the lowest test results. The common masonry includes 23 tests. Here, one value can theoretically lie under the curve. According to this the second lowest value was used for adaptation.

4.4.8. Summary for tensile failure of the unit

It can be certainly expected that besides length and width of a wall, both tensile strength of the unit and vertical load influence significantly shear load capacity due to tensile failure of the unit.

The factors c and F can be summarized when considering that tensile strength of the unit begins in the middle of the wall and that the shear stress distribution in the middle of the wall is also parabolic to the time of failure. These both factors are part of the parameter when adapting the equations to values of the tests. In the simplification of the equation proposed in [21] the factor c was left as a separate factor.

It turned out that the previous regulation of EC 6 with only one limiting value depending on compressive strength of the unit gives no satisfying accordance to the test results after the adjustment. A considerable better adaptation is possible due to the current solution in the

German standard. When using two free parameters a coefficient of determination of 0.67 compared to the values of the experiments and of 0.91 to the equations in [21] will result.

Still higher coefficient of determination can be reached with the exponential and the linear approach. For both 3 free parameters for adapting were used. But the linear approach gives for a theoretical tensile strength of the unit of zero and without load a low but not irrelevant shear load capacity. The solution with the exponential approach results for a load close to zero a shear load capacity also close to zero independent of the tensile strength of the unit.

But it can be seen when adapting the values of the experiments that in face of the high coefficient of determination wrong values could be result. A considerable disadvantage of the statistic approach is due to the still minor database. Compared to the definition of compressive strength of masonry the size is less than 1/10 at a greater number of possible influencing factors. There are still analytical/numerical considerations necessary to define accurate equations. That is why the proposal for the standard is based on a simplification of the equation in [21]. For this, equation (70) is applied based on DIN 1053-100.

5. Shear Stress Distribution

In [24] the shear stress distribution was investigated. The classic bending theory gives a shear factor c = 1.5 for the maximum shear stress for a rectangular cross section. The maximum shear stress is calculated by:

$$\max \tau = c \cdot \frac{V}{A} \tag{76}$$

V shear force

A area of the cross section

In some codes the shear factor is explicitly prescribed. In the German masonry code DIN 1053 the factor depends on the wall geometry. In the Eurocode 6 (EN1996-1-1 [5]) the factor is neglected or should be implicit included in the equations for the shear strength.

In [24] the classic bending theory was combined with a nonlinear material behaviour to calculate the shear stress distribution in case of plasticity. The maximum shear stress reaches a value of 1,78 · V/A for a example beam. The maximum shear stress further increases for a material law with a softening if the maximum strain is reached. The reason for this is the smaller area of the cross section which can take even further load increases. The example shows that the plastic deformation due to bending does not lead to a reduction of the maximum shear stress.

Another way to obtain information about the shear stress distribution is to measure the strain at a wall during shear test. For this, two walls with different vertical loads were tested (see also [24]). For the first one the load was chosen to get a tension failure in the units. The second one should have the failures in the joints. Strain gauges were placed on the walls in several places.

In the first load step the vertical load was imposed on the wall. Logically in this step no shear stress were established in the middle of the wall. At the bottom the R/C beam restricted by his higher stiffness the horizontal expansion of the masonry, which leads to an additional shear stress at the corners. This behaviour could also be observed in FEM calculations.

After the horizontal force was applied on wall one, in the middle of the wall a distribution factor of 1.5 for a quadratic function was reached. After the first cracking no changes happened. Some peaks could be seen just before the final crack. But the factor is still higher than 1.5.

For the second wall the vertical load was much lower, the additional shear stress induced from the R/C beam is also lower. In the middle of the wall the relationship is till the first crack nearly the same like in the first wall. After the crack it is more increasing.

A plastic behaviour was registered in both tests.

Since neither in the first test nor in the second a relationship of the shear stresses between the wall middle and wall edge near to 1 was reached, a unification of the shear stress distribution cannot be assumed for a cross section in a wall due to plasticity prior to failure.

The third way to investigate the shear stress distribution is a numerical simulation. In [24] this part was restricted to a sliding failure in the middle of a homogenous wall. This represents a point of interest, because the sliding has an ideal plastic behaviour after the loss of the initial shear strength.

Figure 33 shows the shear stress at the maximum shear load level.

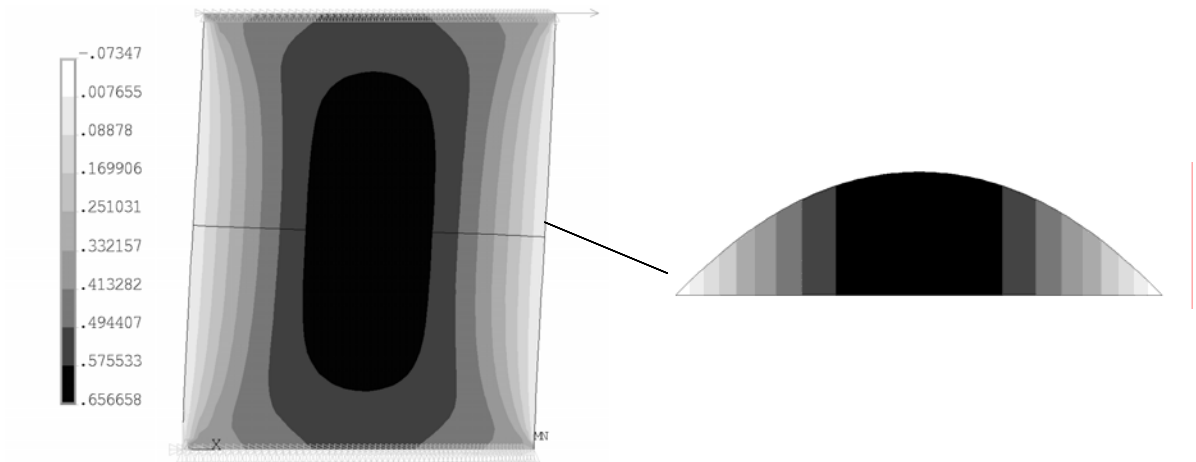


Figure 33 Shear stress in a wall at the maximum and along the interface (l_w = 2m) (from [24])

At the maximum shear load a nonlinear shear stress distribution could be seen. If the horizontal load or displacement at the top of the wall increases more, sudden changes happen. The connections at the interface snap off. The next figure shows the shear stress and the deformation of the wall after the wall starts to slide. On the right the average shear stress is plotted against the horizontal displacement at the top of the wall. Due to sliding a constant shear stress state in the interface region is established. The resistance force remains also constant for further horizontal displacement.

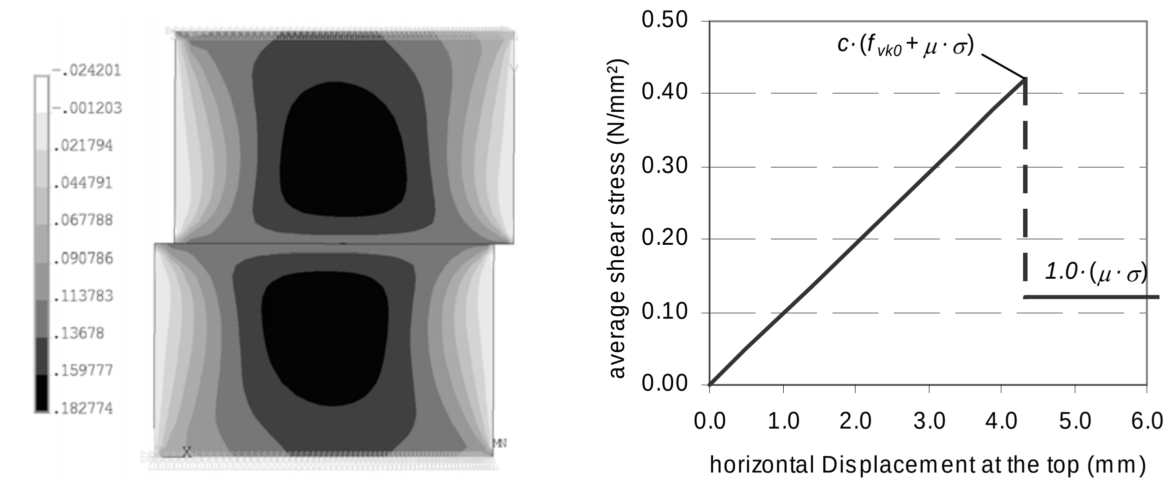


Figure 34 Shear stress in a wall after sliding and load-displacement-curve (l_w = 2m) (from [24])

The average shear strength for the investigated area is given by:

$$f_v = \max \begin{cases} \dfrac{f_{vk0} + \mu \cdot \sigma}{c} \\ \mu \cdot \sigma \end{cases} \tag{77}$$

In the following diagrams some dependencies for the factor c could be seen.

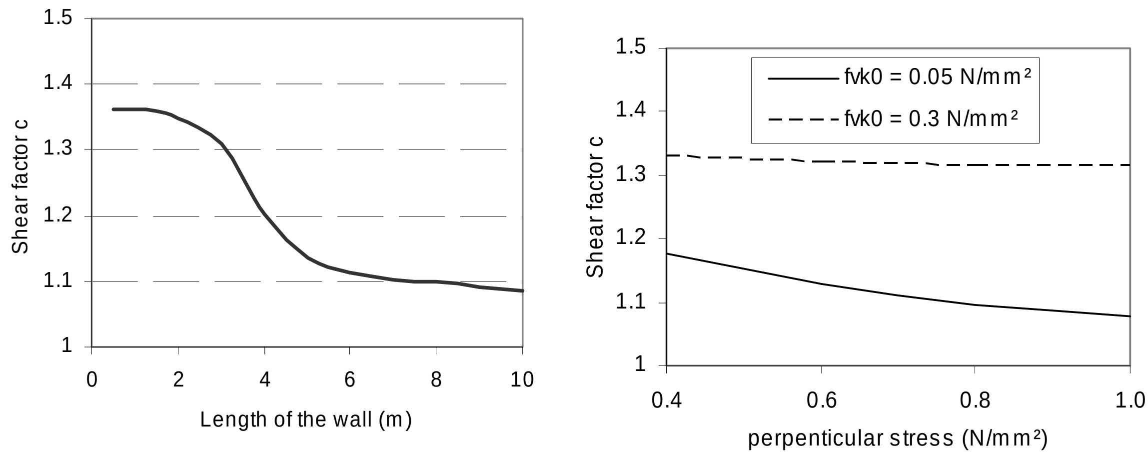


Figure 35 Shear factor for sliding in the middle of the wall depending on the length of the wall, the vertical stress and initial shear strength (G_{fII} = 0.02 N/mm; left: f_{vko} = 0.5 N/mm²; right: l_w = 2 m) (from [24])

With the done numerical parametric study a dependencies of the factor c on the crack energy, the wall length, and the relation between the initial shear strength and the stress perpendicular to the joint could be detected.

The principle conclusion from [24] is that the shear stress distribution can not be neglected. A shear stress distribution factor greater than 1.0 arises from the theoretical, the experimental and numeric analysis in any investigated cases. To be on the safe side 1.5 should be used as the factor for the shear stress distribution. In the case of high crack energy or very large walls a smaller value could be used.

The numerical investigations of the shear stress distribution in [19] also confirmed this fact and result a shear stress distribution factor between 1.4 and 1.5. Alternatively to the regulation of DIN 1053-100 it is proposed in [19] to use shear slenderness to calculate the shear stress distribution factor c.

$$c = 1.0 \leq 0.5+\lambda_v \leq 1.5 \tag{78}$$

Where:

$\lambda_v = \psi \cdot \frac{h}{l}$ shear slenderness

$\psi = 1.0$ for cantilever systems

$\psi = 0.5$ for full fixed walls with full restraint at the top.

In DIN 1053-100 [3] the calculation rule is basically:

$$c = 1.0 \le 0,5 + \frac{h}{2l} \le 1.5 \tag{79}$$

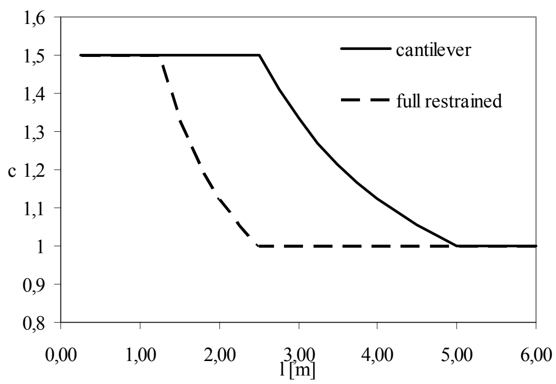


Figure 36 Comparison of the factor for the shear stress distribution for a wall high of 2.5 m

The curve for the full restrained wall is equal to the definition of DIN 1053-100 [3]. But it is clearly under the one in Figure 35 at the same statistic system.

In identical design equations, e. g. for tensile failure of the unit in the middle of the wall, a load capacity difference between the walls of the same size but with a different restraint grade is only shown with factor c. The biggest difference between the cantilever and a full restraint results at a quadratic wall. In this case the load capacity would be 50 % higher when appearing shear failure with full restraint like the following graph shows:

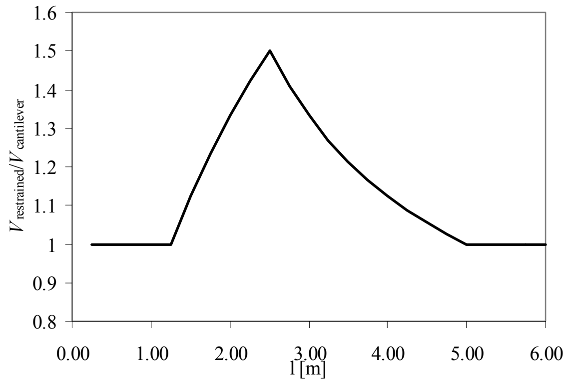


Figure 37 Relation of shear load capacity due to shear stress distribution factor c for a full restrained system and a cantilever

The load difference due to the restraint grade of a wall can not be showed with the factor *c*. In this case a calculative wall length would be better. For the normative consideration of shear stress distribution in the wall a fixed value should be used to simplify. It should be 1.5 for the middle of the wall.

Because of the basis of the proposed equations for the standard on Deliverable 4.4 and [21] respectively for the tensile failure of the unit and the calibration with eq. (78) and a variable calculated shear stress distribution factor c, this solution will initially be used in the standardisation proposal. At a magnification of the data base and an ensuing calibration of the equations on experimental values, the influence of shear stress distribution in the wall can incorporate into the equation parameters.

6. Consideration of Wall Geometries

Normally the design engineer has to decide or to investigate the structural system for calculating the decisive cross section. From the design of the concrete slabs he will get a vertical load and a distribution of it or an eccentricity. The eccentricity maybe leads to a partial loaded shear wall. This has to be taken in to account because of the influence on shear load capacity. The next influencing factor is the support at the top. The shear wall could work as a cantilever arm or with a restraining at the top.

The proposal of Deliverable 4.4 takes only full loaded walls into account. This means the vertical load is distributed over the whole length of the wall. So the wall geometry has only an influence to the verification of bending and indirect on the other failure cases.

For failure due to overturning/bending of an axial loaded wall the shear load capacity depends on the static system. The value lies between the capacity of a cantilever and a at the top full restraint wall. And the cantilever has the half capacity of the full restraint system. The full restraint could also be seen as a cantilever of the half height. This effect is covered by the proposed equation (28).

But the vertical load on a stiffening wall may also be eccentric due to an unsymmetrical floor plan. This will have an influence on the load bearing capacity. Therefore the next chapter should give a clue.

6.1. Numerical Example

The following numerical Example should point the basic problem. A simple example should show in the following the influence of different kinds of applied vertical loads.

The aspect ratio of the wall is 6 to 11 (high to length). The material behaviour is linear elastic. The modelled contact between the units is the only nonlinearity in this simulation. The average vertical load is 0.275 N/mm². The initial bond strength is f_t=0.36 N/mm² and f_{vk0}=0.78 N/mm² with a sliding coefficient of μ =0.6. The module of elasticity for the unit is E_b=1850 N/mm².

Four cases will be studied. The first wall has a full restraint support at the top. The second works like a cantilever with a constant load distribution. The third and the fourth wall are eccentric loaded by a eccentricity of ± *l*/6. The horizontal load is applied from the right side.

Shear stresses are showed:

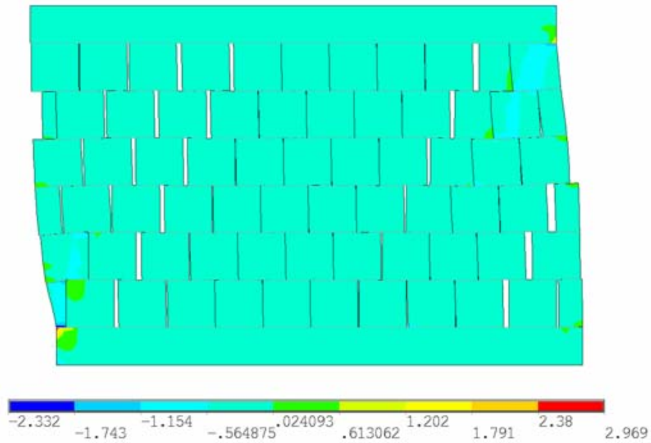


Figure 38 Wall with fixed support at the top

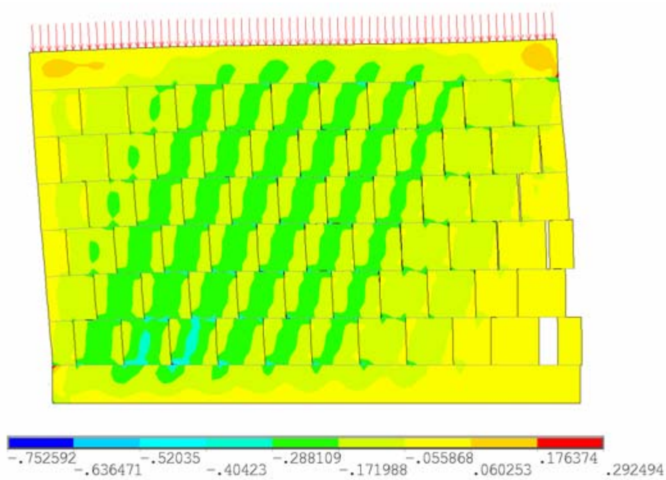


Figure 39 Wall with an constant vertical load (cantilever) at the ultimate limit state

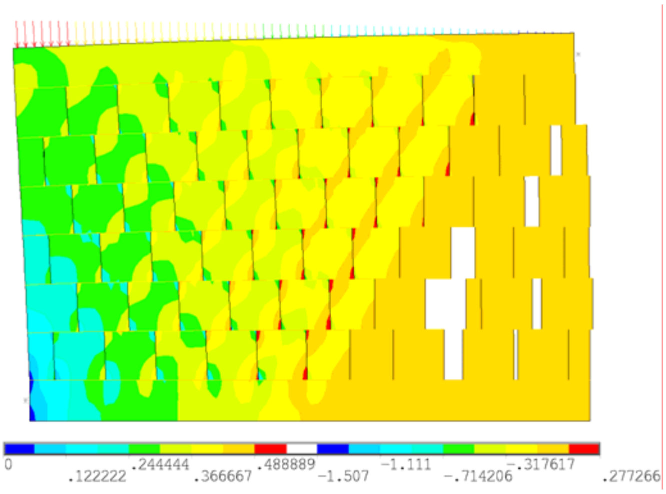


Figure 40   Wall with an eccentric vertical load on the opposite side of the load introduction

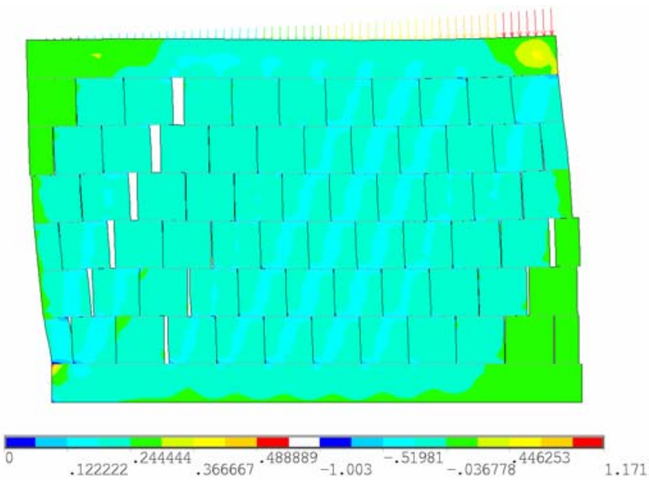


Figure 41   Wall with an eccentric vertical load on the same side of the load introduction

The following diagram shows the load displacement behaviour.

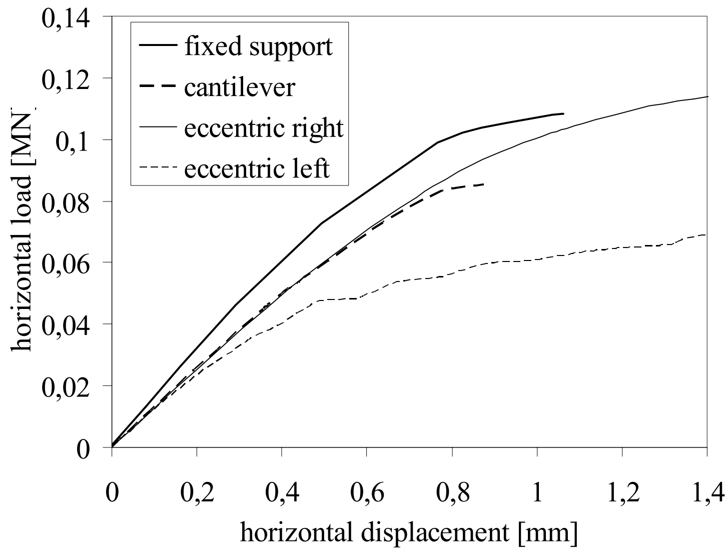


Figure 42 Comparison of the load-displacement-curves with different load-systems

It could be seen, that the factor between the fixed support and the cantilever are less then two. The fixed support leads also to a higher stiffness. The lowest shear load capacity was reached with an eccentricity at the left side as expected.

6.2. Theoretical Approach

In the following an effective wall length for a partial loaded wall is developed for the verification in the middle of the wall.

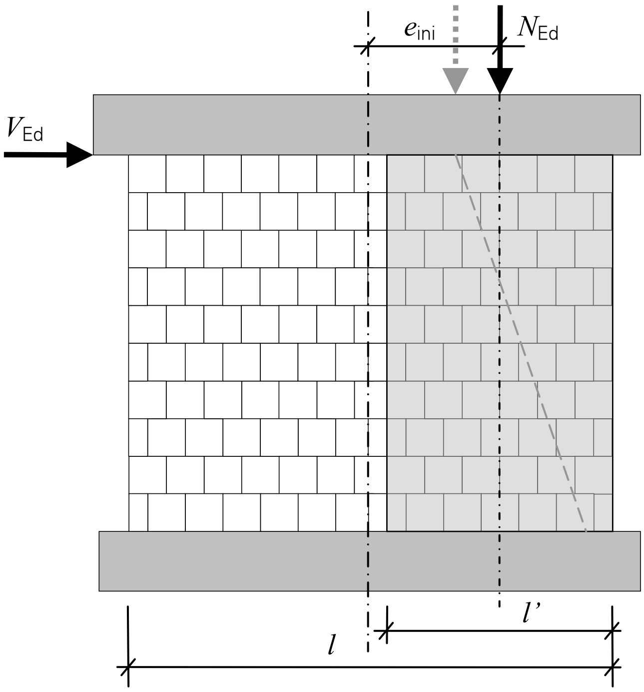


Figure 43 Schematic print of the reduced wall length

$$l' = l - 2e_{ini} \leq l \tag{80}$$

As explained in the chapters before the load capacity depends also on the restraint grade in cases of friction failure and tensile failure of the unit. To calculate the shear load at half of the wall height, the influencing wall length must be considered.

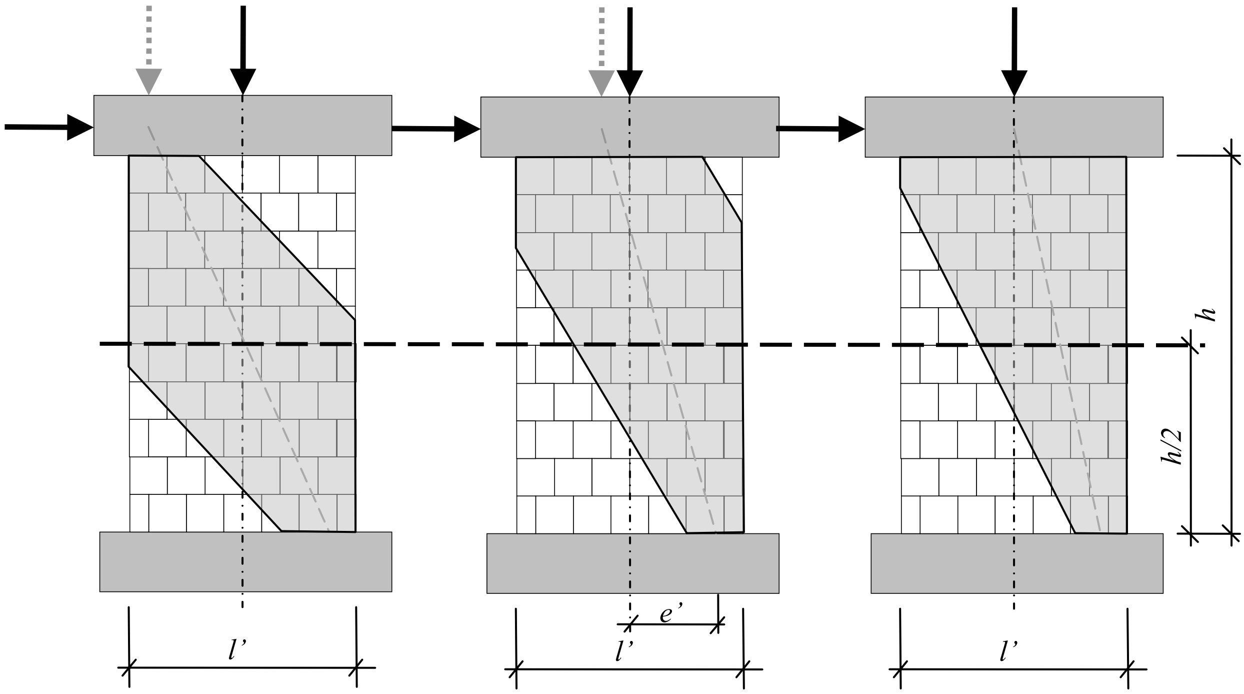


Figure 44 Shear walls with different levels of restraining at the top

$$e' = \frac{V}{N} \cdot \psi \cdot h \tag{81}$$

$$e'_{\frac{h}{2}} = \frac{V \cdot h}{N} \cdot \left(\psi - \frac{1}{2} \right) \tag{82}$$

For the assumption of a stress block the calculative length results to:

$$l_{cal} = l - 2e_{ini} - 2\frac{Vh}{N}\left(\psi - \frac{1}{2} \right) \leq l \tag{83}$$

The result for using a linear stress-strain-relation is:

$$l_{cal} = \frac{3}{2}\left(l - 2e_{ini} - 2\frac{Vh}{N}\left(\psi - \frac{1}{2} \right) \right) \leq l - 2e_{ini} \leq l \tag{84}$$

There is a l_{cal}/l' of 0.5 in the limit sate of overturning when using the stress block and 0.75 with linear elastic stress distribution.

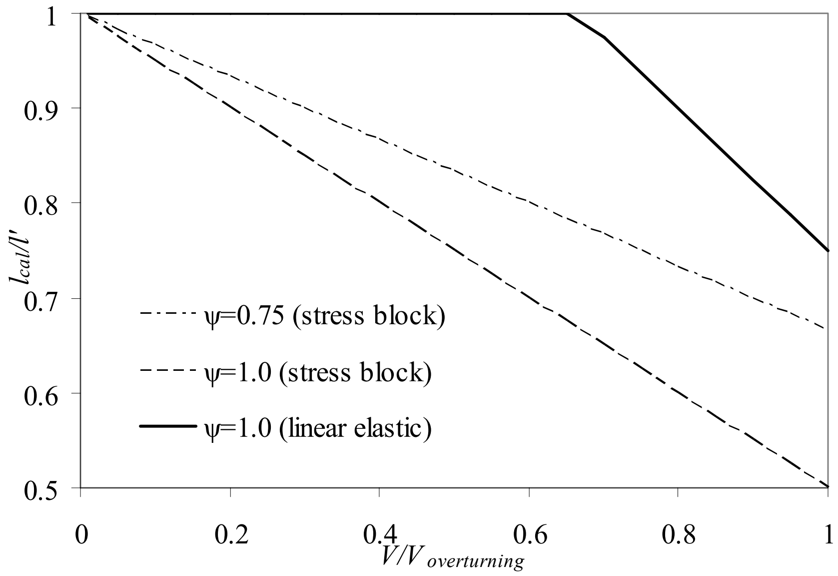


Figure 45 Relation of calculated wall length in the middle of the wall to the full used length (or reduction factor for the static system)

Due to the calculation of an effective wall length different restraint situations in the building can be considered. Besides this the calculation of the compressed length can be left out. For every direction a usable length has to be calculated.

Alternatively the compressed length can be used as well because the systems restraint and cantilever also differ for it. But the tensile failure of the unit does not start in this area. The arising failure in the model is to be corrected due to an adaptation of the parameter.

7. Other effects

7.1. Combined action

The vertical load of a stiffening wall comes manly from the support of slabs amongst others. Due to the supported slabs and the bending of the slabs, an eccentric load introduction in out of plane direction will occur. But also wind or impact loads cause an irregular stress distribution over the wall thickness. The cross section even cracks at larger eccentricities. Investigations and approaches respectively about the load capacity under combined shear and bending load out of plane are not yet sufficiently available. Mojsilović [28] developed a proposal where the shear load capacity decrease especially for tensile failure of the unit and compressive failure by a increasing eccentricity. One test was carried out within ESECMaSE [16] as well. Here, a significant influence of the load eccentricity was found, too.

The influence must be quantified by further investigations. But the influence can be estimated for the verification of friction failure. If the initial shear strength is considered in the verification, the used part must be minimised according to the eccentricity and the resulting compressed cross section area. For members which could be collapse through buckling under shear load, the buckling length as well as the compressive load must be increased if necessary. For this, the length of the compressive strut which arises in the shear wall can be used.

7.2. Determination of internal forces of a building

In analyses about the load bearing behaviour in building in [14] and [30] a restraint effect as well as an increase in load at a certain load direction was found. Especially short walls in an apartment house reached quickly their shear load capacity. When increasing the shear load on the building, the percentage part of some walls decrease while at others an increase was seen. Large scale tests in Athens [10] as well as in Ispra [8] shows that it came to a load transfer of the vertical loads from the external wall which border to the shear walls to the shear walls while horizontal load applies. This would lead to, e. g. at shaking table experiments in Athens, partial collapse of the external walls due to plane bending.

The following graph shows the relation of the shear load to the horizontal deformation for the large scale test in Ispra.

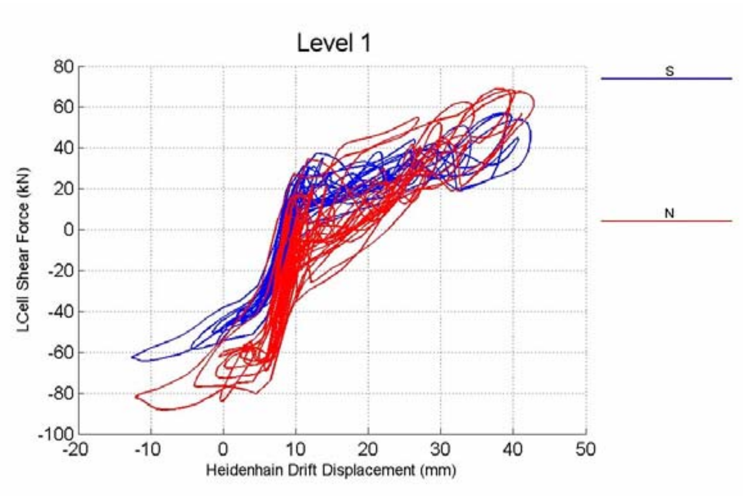


Figure 46 Shear load capacity of the main shear wall at the large scale test in Ispra (blue main shear wall)

It can be seen that the shear load is higher in the one direction than in the other. The ductile behaviour is vice versa. If the noticed effects in the large scale tests and in the numerical calculations should be taken into account for the verification, an individual consideration of the particular building will be necessary. A simple numerical calculation of the verified building is advisable to take advantage of the load reserves especially in case of earthquake. For this, the expected horizontal shear load should be included to measure the vertical load of the stiffening walls which changes due to that.

The resultant force due to an earthquake load increase from the bottom of a building to the top of it. The horizontal loads to wind are the same for every floor excepting the top one. In the top floor only half of the load becomes effective. Furthermore the verification for the horizontal stiffening under wind load is to be done with a minimal load.

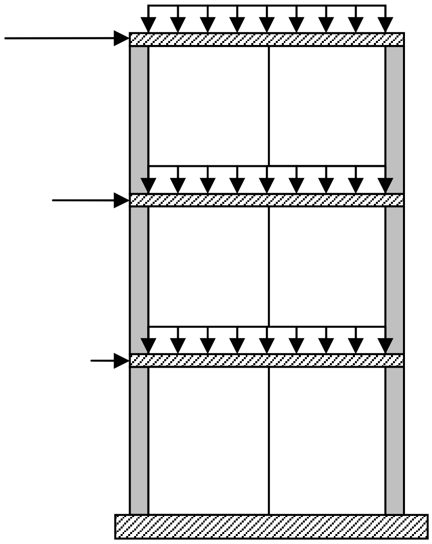


Figure 47 Schematic comparison of the resultant force at the earthquake design and under wind load

But there is a significant difference for the verification. While the stiffening wall at the top is less risky at wind load, the top floor can be more loaded under earthquake load due to the lower vertical load.

8. Material properties

Only two different material properties are needed for the proposed shear model in [19]: the sliding coefficient and the tensile strength of the units.

The properties still used in some standardisation codes are listed in chapter 2. However, the initial shear strength and the tensile strength are also required for the extended proposal in this report. The compressive strength of masonry is already regulated in all masonry standards and therefore should not be considered separately.

8.1. Tensile bond strength

The tensile bond strength is a strong scattering material parameter with only few statistic evaluable test data.

In German standard the approach of a tensile strength perpendicular to the bed joints is generally not permitted. This means that a verification of non-loadbearing walls without vertical load, infill walls for example, is not possible directly. However, the regulation for the yet allowed infill sections implies a certain tensile strength.

EC-6 includes a proposal for the bending tensile strength (see Table 5), but in the respective National Annex it can be defined differently.

A compilation of test results shows Table 6 c in [32]. The range of values fluctuate for a small number of tests per material combination between 0.07 N/mm² for perforated clay units with light weight mortar and 0.67 N/mm² for CS-units with thin-layer mortar. Masonry with light weight mortar generally achieves the lowest values.

For the optimised CS-units developed within the ESECMaSE-project the tensile bond strength was determined in [9]. Mean values between 0.26 and 0.35 N/mm² were indicated for the different test series. The bending tensile strength of the bond was also determined. This was higher by a factor of 1.77 within the test series A, but is only valid for a specimen height of 40mm. On a larger lever arm as in shear walls, the tensile strength and the bending tensile strength are nearly equal.

As can be seen in Deliverable 5.5 [34], the bending tensile strength of the bond determined with the 4-point-bending test and with the bond wrench test is larger than the tensile bond strength which is determined with a axial test. The identified tensile bond strength of CS (0.20 N/mm² for d=115mm and 0.24N/mm² for d=175mm) is located somewhat below the level determined in [9]. For clay bricks arose values of 0.23 N/mm² (d=115mm) and 0.16 N/mm² (d=175mm). However, the values of at least 3 of 12 test lays under the level of the characteristic values of the EC 6 (see Table 5). Both investigations ([9] and [34]) dealt only with thin layer mortar.

Whether the specified values for the bending tensile strength in EC6 are applied also for the tensile bond strength, then a lower value must be defined in the standard for hollow bricks. For this, however, further investigations with different units are necessary.

It is suggested to differentiate the index of the variables too, because the bending tensile strength perpendicular to the bed joints can reach a much larger value than the tensile bond strength. For the bending tensile strength it is still f_{x1} and for tensile bond strength f_t.

8.2. Sliding coefficient and initial shear strength

From the standards presented at the beginning of this report several classifications are conducted during the normative regulation of the initial shear strength. Whereas the Australian Standard (AS) defines the initial shear strength with the bending tensile strength and two further limiting values, according to the German standard the initial shear strength is addicted to the type of mortar. The EN 1996-1-1 differentiates between unit type as well as mortar type.

According to the German standard the friction coefficient is 0.6. This applies, however, only for loading out of plane. Corresponding to the theory of *Mann/Müller* a reduced friction coefficient is specified with a value of 0.4 for shear walls. The values for the initial shear strength were reduced with the same reduction ratio (see chapter 2.3). The reduction ratio is 1.65. In EC6 the friction coefficient for the initial shear strength is also 0.4. On the other hand, the AS 3700 differentiates between general masonry and masonry made up of aerated concrete (AAC) as well as different separation layers (cp. Table 2).

Table 17 shows a comparison between the shear strength defined in standards and in values from the literature.

Table 17 Initial shear strength f_{vko} (N/mm²) for different kinds of mortar according to DIN 1053-1 (reduced initial shear strength) and EN 1996-1-1 as well as according to Schubert [32]

<table>
<tr><th colspan="2">kind of mortar</th><th>NM I</th><th>NM II</th><th>NM IIa</th><th>LM 21
LM 36</th><th>NM III</th><th>DM/TL</th><th>NM IIIa</th></tr>
<tr><th colspan="2">equal to</th><th>M1.0</th><th>M2.5</th><th colspan="2">M5</th><th colspan="2">M10</th><th>M20</th></tr>
<tr><td colspan="2">DIN 1053-100 b)
Table 6</td><td>0.03</td><td>0.13</td><td colspan="2">0.30</td><td colspan="2">0.36</td><td>0.43</td></tr>
<tr><td colspan="2">minimum value
DIN 1053-1
suitability test</td><td>-</td><td>0.10</td><td colspan="2">0.20</td><td>0.25</td><td>0.50</td><td>0.30</td></tr>
<tr><td rowspan="5">EN 1996-1-1</td><td>Clay</td><td>-</td><td>0.20</td><td>0.20</td><td rowspan="5">0.15</td><td>0.30</td><td>0.30</td><td>0.30</td></tr>
<tr><td>Calcium silicate</td><td>-</td><td>0.15</td><td>0.15</td><td rowspan="2">0.20</td><td>0.40</td><td rowspan="2">0.20</td></tr>
<tr><td>Aggregate concrete</td><td>-</td><td>-</td><td>-</td><td rowspan="3">0.30</td></tr>
<tr><td>Autoclaved Aerated Concrete</td><td>-</td><td>0.15</td><td>0.15</td><td rowspan="2">-</td><td rowspan="2">-</td></tr>
<tr><td>Manufactured stone and Dimensioned natural stone</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td rowspan="4">Schubert [32]</td><td>Clay</td><td>-</td><td>0.40</td><td>0.50</td><td rowspan="4">-</td><td>0.70</td><td>0.63</td><td>1.00</td></tr>
<tr><td>Calcium silicate</td><td>-</td><td>0.15</td><td>0.20</td><td>0.30</td><td>0.66 a)</td><td>0.40</td></tr>
<tr><td>lightweight concrete (Hbl, V, Vbl)</td><td>-</td><td>0.40</td><td>0.60</td><td>0.70</td><td>0.68 a)</td><td>0.90</td></tr>
<tr><td>Autoclaved Aerated Concrete</td><td>-</td><td>0.10</td><td>0.15</td><td>0.20</td><td>0.68 a)</td><td>0.25</td></tr>
<tr><td colspan="9">a) The minimum value was used (smallest declared value).
b) Values were increased by the reduction factor (1.65)
ADVICE:
The bold values indicate a greater initial shear strength according to EN 1996-1-1 than the minimum values according to DIN 1053-1.</td></tr>
</table>

The values in Table 17 apply for filled head joints. In comparison with values from the literature, the values of DIN 1053-100 and EC6 are on the safe side.

In [9], *Brahmeshuber* and *Schmidt* investigate for the optimized CS-units the initial shear strength between 0.24 and 0.34 N/mm² with a thin layer mortar. However, they achieve 0.65 N/mm² with the verification according to DIN 18555-5. The static friction coefficient was 0.54 while the dynamic friction coefficient was 0.73. The carried out torsion shear tests also revealed an initial shear strength of 0.49 N/mm² and a static friction coefficient of 0.46.

In Deliverable 5.5 [34], values for the initial shear strength (between 0.16 and 0.24 N/mm²) and the friction coefficient of CS-units (between 0.65 and 0.89) were indicated. For clay-masonry the band width is between 0.08 and 0.32 N/mm² for initial shear strength and 0.58 and 0.96 for the friction coefficient.

Kirtschig and *Anstötz* published a proposal for the various separation layers [25]. Table 18 shows the proposed characteristical values.

An indication of the initial shear strength separated in kind of stone and mortar is necessary for the differentiate determination of the shear strength. Therefore the appropriation of the characteristic initial shear strength according to EN 1996-1-1 is intended for the German National Annex. The values determined within the ESECMaSE-project are partly under the level of EC6. Therefore a further consideration is necessary. The actual values of the EC 6 have been maintained for the shear-proposal at first.

Table 18 Characteristic initial shear strength and friction coefficient according to [25], for damp proof course and sealing sludges

Stein-art	Mörtel-gruppe	Feuchte-sperrschicht	Grundwert β_{HSo}	Reibungsbeiwert μ
-	-	-	N/mm²	-
1	2	3	4	5
KSL	II	Folie	0,04	0,73
		Pappe	0,19	0,52
		Schlämme	0,26	1,10
	III	Folie	0,08	0,82
		Pappe	0,26	0,35
		Schlämme	0,55	1,36
Hlz	II	Folie	0,06	0,54
		Pappe	0,24	0,71
		Schlämme	0,16	1,04
	III	Folie	0,06	0,73
		Pappe	0,31	0,29
		Schlämme	0,31	0,88

8.3. Tensile strength of the unit

By means of the cracking, which is caused by the failure due to the excess of the tensile strength of the unit, it can be determined in which direction the tensile strength of the unit is decisive. The longitudinal tensile strength can be applied for the usage of masonry units with neglect of insignificant anisotropic material properties. Perforated units have a discontinuous crack growth with diagonal cracks, as explained in [22]. The cracking starts in the smallest cross-section (hole) and propagates along the principal stress direction. Due to this the decisive cross-section of the unit is changing. The crack reaches a web and for this the tensile strength of the units is increasing. This behaviour can not be adequately displayed with conventional testing methods.

Further specific problems of the perforated clay bricks are cracks as a result of the influence of high temperatures. Due to the firing of the bricks cracks occur, which prevent the transfer of the tensile stresses over a couple of webs. This leads to additional bending stresses in the rest of the webs within a centric tensile test and gives a smaller tensile strength.

The longitudinal tensile strength as an initial parameter for the calculation of the shear capacity are significantly underestimated the experimental shear load capacity.

In Deliverable 4.4 [20] the used tensile strength of the units was calculated for perforated clay units. For the first four tests made in Kassel the calculated tensile strength is a mixture of vertical and horizontal tensile strength. For the other units the horizontal tensile strength was doubled.

By the proposal of Deliverable [31] the splitting test should be used to get a practical estimation of the tensile strength of the unit. Therefore *Graubner/Kranzler* used these values for the comparison in [21].

From Deliverable 5.5 [31] the following values for the splitting tensile strength of the used material in ESECMaSE are given.

Table 19 Splitting tensile strength perpendicular to the longitudinal axis of masonry units from Deliverable 5.5 [31]

Kind of Unit	Tensile strength $f_{t,sp}$	Standard Deviation	Coefficient of Variation	Characteristic values (5% Quantile)		
				normal dist.	80%	70%
	[N/mm²]	[N/mm²]	[%]	[N/mm²]	[N/mm²]	[N/mm²]
KS R	2.09	0.13	6.25	1.88	1.67	1.46
KS XL-PE	1.06	0.07	6.48	0.95	0.85	0.74
HLz B12	0.61	0.05	7.67	0.53	0.49	0.43
T 16	0.34	0.04	11.00	0.28	0.27	0.24
Pexider Poroton HLZ 30/25/25	0.46	0.09	18.58	0.32	0.37	0.32
Alveolater 45	0.39	0.07	18.50	0.27	0.31	0.27
Alveolater Incastro	0.39	0.04	8.86	0.33	0.31	0.27
HBI 18	1.13	0.14	11.96	0.91	0.90	0.79
HBI 20	0.54	0.05	9.56	0.46	0.43	0.38
Liapor M	1.41	0.14	10.10	1.18	1.13	0.99

Assuming a normal distribution, the characteristic tensile strength can be calculated with the help of the coefficient of variation indicated in [31]. It turns out that in most cases with the 80% average a good assumption can be made “on the safe side”.

The following ratios can be investigated together with the compressive strength determined in [31]. Additionally, the values of the tensile strength parallel to the bed joints are given.

Table 20 Relation between splitting tensile and compressive strength from the test results in [31]

Kind of Unit	Tensile strength $f_{t,sp}$	Tensile strength $f_{t,II}$	Compressiv strength f_b	$f_{b,sp}/f_b$	$f_{bt,II}/f_b$
	[N/mm²]	[N/mm²]	[N/mm²]	[-]	[-]
KS R	2.09	1.67	26.54	0.079	0.063
KS XL-PE	1.06		14.4	0.074	
HLz B12	0.61	0.21	19.25	0.032	0.011
T 16	0.34	0.24	13.12	0.026	0.018
Pexider Poroton HLZ 30/25/25	0.46		10.11	0.045	
Alveolater 45	0.39	0.15	15.06	0.026	0.010
Alveolater Incastro	0.39		15.38	0.025	
HBl 18	1.13		15.33	0.074	
HBl 20	0.54		6.49	0.083	
Liapor M	1.41		19.8	0.071	

A longitudinal tensile strength of 1.49 N/mm² and a compressive strength of 21.5 N/mm² in direction of unit height were determined for the optimized CS-units investigated in [9]. The outcome of this is a ratio of $f_{bt,II}/f_b$ = 0.069. The parallel investigations on cylinders result in higher ratios: 2.15/20.6=0.104 (50mm cylinder) and 2.31/19.3=0.12 (80mm cylinder), which was due to the lack of sectional weakening by the attributed grip hole.

Along with the literature review in [26] for the normative definition of the tensile strength of the units the following table can be proposed.

Table 21 Proposal for standardized tensile strength of the units for the shear verification

	Clay	KS	LC	AAC	
				$f_{bk} \leq 2$ N/mm²	$f_{bk} > 2$ N/mm²
f_{bt}	$0.035\,f_{bk}$	$0.05\,f_{bk}$	$0.07\,f_{bk}$	$0.12\,f_{bk}$	$0.08\,f_{bk}$

With the proposed values was simply assumed that the influence of any hole pattern on the longitudinal tensile strength is already taken into account with the particular compressive strength, respectively the values were accordingly determined (e.g. clay).

If subsequent studies show a different dependence of the tensile strength and the compressive strength from the pattern of the holes, the proposed values in Table 21 may be further differentiated if necessary.

9. Safety Concept

In typical codes the calculated shear strength will be reduced by a partial safety factor for the material. The other part of the safety concept is realised on the action. In addition to the partial safety factors, the global safety is reached by the usage of characteristic values for action and resistance. Therewith the required failure probability according to DIN 1055-100 or EN 1990 can be ensured

Bending

For the failure due to bending two cases are possible. The first one is the standard way in which the compressive strength will be reduced by the partial safety factor. The total capacity comes to.

$$V_{Rd,bending} = \frac{1}{2 \cdot \lambda_v} \cdot \left(N_{Ed} - \frac{N_{Ed}^{\ 2} \cdot \gamma_M}{t \cdot l \cdot f_k} \right) \tag{85}$$

The other possibility is to reduce the whole capacity.

$$V_{Rd} = \frac{V_{bending}}{\gamma_M} = \frac{1}{2 \cdot \lambda_v \cdot \gamma_M} \cdot \left(N_{Ed} - \frac{N_{Ed}^{\ 2}}{t \cdot l \cdot f_k} \right) \tag{86}$$

This option conforms an increasing of the partial safety factor for the action, because γ_M can be also brought as a factor on the other side of the equation.

The ratio of the design resistance after eq. (85) and (86) is:

$$r = \gamma_M \cdot \frac{1 - n \cdot \gamma_M}{1 - n} \tag{87}$$

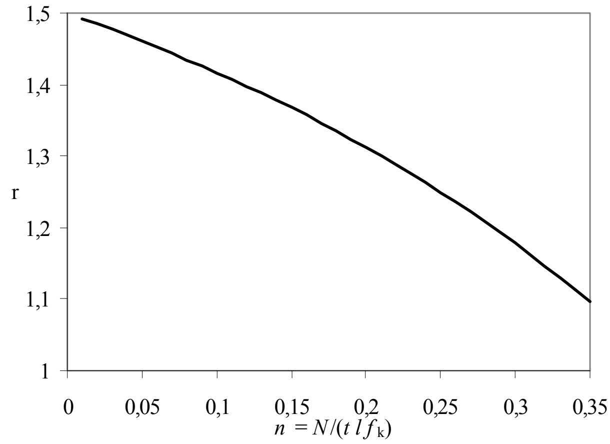


Figure 48 Relation between the shear design loads due to bending calculated from version one and two

The bending verification conforms to the first case and is already included in the EC 6. It is merely a matter of formal adjustments to include shear walls in the verification. Therefore it is necessary to adjust the determination of the eccentricities for shear walls (see chapter 10.3).

If the tensile strength is applied, the characteristic value of the strength has to be reduced with the partial safety factor. For every country the precise value can be found in the National Annex.

Gaping

For the proposed failure due to gaping without tensile bond strength no material parameter is involved. This failure is only a geometrical criterion. The variation because of the masonry unit size is very small. Therefore mainly the model error and the execution quality have to be covered by the partial safety factor of the material. This includes for example the variation of the masonry bond. As mentioned in chapter 4.2, a staircase bond can lead to a significantly reduced load capacity depending on the load direction. The model error can be reduced, if this bond is considered on the verification with the compressed length. A value of γ_M = 1.35 for the failure type gaping without any bond strength is suggested on the safe side.

By the consideration of the tensile bond strength by use of the tensile strength arise two possibilities for the use of the partial safety factor of the material. On the one hand, the partial safety factor is applied to the scattering material parameter, the initial shear strength.

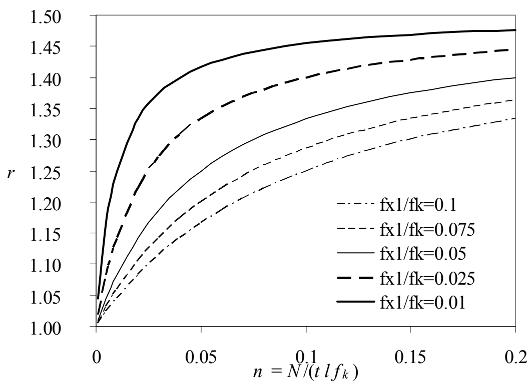
$$f_{vd} = \left(\frac{f_t}{\gamma_M} + \sigma_d \right) \frac{l_b}{2h_b} \tag{88}$$

The vertical stress is still taken into account as an opportunely acting load with γ_E=1.0 and with the according loading case. If the partial safety factor for the material will be applied to the whole equation, this will lead to an increasing of the safe distance. The second possibility for the application of the partial safety factor of the material corresponds to the current regulations in DIN 1053-100 and EC6.

$$f_{vd} = \frac{f_{vk}}{\gamma_M} = \left(f_t + \sigma_d \right) \frac{l_b}{2h_b \cdot \gamma_M} \tag{89}$$

The ratio of the shear strength according to the equations (88) and (89) adds up to:

$$r = \frac{\overline{f_t} + n \cdot \gamma_M}{\overline{f_t} + n} \tag{90}$$



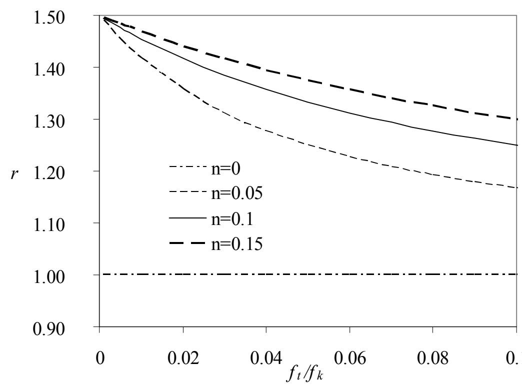


Figure 49 Relation between the shear strength due to gaping from equation (88) and (89) (γ_M=1.5)

The shear strength which was determined according to eq. (89) is lower than on the direct application of the material partial safety factor on the tensile bond strength. The magnitude depends on the tensile bond strength and the vertical loading. A part of the safety-deficit which occurs with an asymmetric overlapping length can thereby be compensated as can be seen in Figure 14 in chapter 4.2. However, with an asymmetric overlapping length and without vertical loads the shear strength still can be overestimated. Staircase bonds should either not be allowed, or be considered via eq. (53). Another security benefit can be achieved by the use of the tensile bond strength instead of the bending tensile strength of the bond. Because the decreasing factor for this value is 1.3 to 2.0.

Friction/sliding

Both terms of the design equation contains a material depending factor. The initial shear strength as well as the coefficient of friction are scattered material parameters. Due to this the safety factor for the material has to be applied to the whole equation. A variation, like the one for gaping, is not possible.

Tensile failure of the unit

Similar to the gaping the partial safety factor for the material will be related to the shear strength and alternatively to the tensile strength of the units in the following. If only the tensile strength of the unit should be reduced by the safety factor the shear load capacity becomes to:

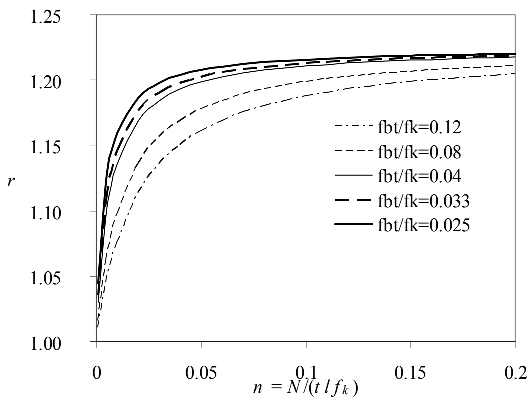
$$V_{Rd} = \frac{t \cdot l}{c \cdot} \alpha \cdot \frac{f_{bt}}{\gamma_M} \sqrt{1 + \frac{\gamma_M \cdot N_{Ed}}{\beta \cdot f_{bt} \cdot t \cdot l}} \tag{91}$$

If the partial safety factor is considered for total shear the equation result to:

$$V_{Rd} = \frac{t \cdot l}{c \cdot \gamma_M} \alpha \cdot f_{bt} \sqrt{1 + \frac{N_{Ed}}{\beta \cdot f_{bt} \cdot t \cdot l}} \tag{92}$$

The ratio of the shear capacity according to the equations (91) and (92) are:

$$r = \frac{\sqrt{1 + \frac{\gamma_M \cdot n}{\beta \cdot \overline{f_{bt}}}}}{\sqrt{1 + \frac{n}{\beta \cdot \overline{f_{bt}}}}} \tag{93}$$



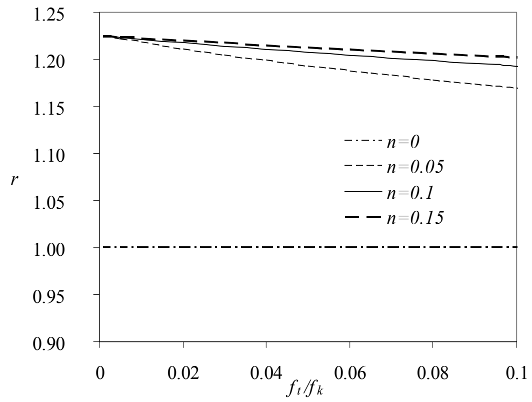


Figure 50 Relation between the shear strength due to tensile failure of the units from equation (91) and (92) (γ_M=1.5; β=0.18)

The shear strength which was determined according to eq. (92) is lower than by an application of the material partial safety factor only on the tensile bond strength. The magnitude depends on the tensile strength of the unit, the vertical loading and the material depending factor β. The ratio approaches to the limiting value $\sqrt{\gamma_M}$.

A higher design shear strength respectively shear load capacity could be seen for gaping with bond strength and the unit failure by an application of the partial safety factors for the material only on the basic material parameters. But there is no alternative approach when all summands including material parameters, such as the friction failure. As in actual normative regulations the shear strength obtained completely the material safety factors and no deeper investigation were made for the global safety with a relation of the material safety to the basic material properties, the proposal in this report will use the current approach.

A detailed theoretical analysis for the global failure probability is needed.

10. Proposal for the EC 6

The proposed indexing differs slightly from the current designations. A standardisation shall be carried out in the corresponding normative committees. For example the tensile strength of the unit is not yet regulated in the Eurocode. The bending tensile strength for masonry perpendicular to the bed joints, which is essentially corresponding to the bending tensile strength of the bond, is defined as f_{x1}. The tensile bond strength, required for the failure due to gaping, is defined by f_t.

10.1. Shear verification on the basis of shear load capacity

The formulation in chapter 6.2 should be changed to:

6.2 Unreinforced masonry walls subjected to shear loading

(1)P At the ultimate limit state the design value of the shear load applied to the masonry wall, V_{Ed}, shall be less than or equal to the design value of the shear resistance of the wall, V_{Rd}, such that :

$$V_{Ed} \leq V_{Rd} \tag{6.12}$$

(1) The design value of the shear resistance is given by the minimum of:

$$V_{Rd} = \frac{N_{Ed}}{\gamma_M}\left(\frac{l_{ol}}{h_b} + \frac{l_b - l_{ol}}{h}\right) \tag{6.13a}$$

$$V_{Rd} = \frac{\mu \cdot N_{Ed}}{\gamma_M} \tag{6.13b}$$

$$V_{Rd} = \frac{t \cdot l_{cal}}{c \cdot \gamma_M} 0.22 \cdot f_{bt} \sqrt{1 + \frac{5 \cdot N_{Ed}}{f_{bt} \cdot t \cdot l_{cal}}} \tag{6.13c}$$

$$V_{Rd} = \frac{t \cdot l_{cal}}{c \cdot \gamma_M} 0.1 \cdot f_{bt} \sqrt{1 + \frac{16 \cdot N_{Ed}}{f_{bt} \cdot t \cdot l_{cal}}} \quad \text{for AAC-masonry}$$

where:

t is the thickness of the wall resisting the shear;

h is the height of the wall;

h_b is the height of the masonry unit;

l_b is the length of the masonry unit;

l_{ol} is the overlapping length of the masonry bond, for a regular bond it could be taken half of the unit length;

l_{cal} is the calculated length of the wall see clause (6);

γ_M for eq. (6.13a) a partial safety factor of 1.35 could be used, if the size of the units and the overlapping length is fixed. In the other case 1.35 has to be used.

μ is the friction coefficient; For typical masonry a value of 0.6 could be used. For dry masonry the value should be 0.4. Damp-proof courses or other material affecting the sliding should be taken into account;

f_{bt} is the characteristic tensile strength of the unit;

f_{bk} is the characteristic compressive strength of the unit;

α is a factor to consider the shear stress distribution in the unit and the wall.

c is the factor to consider the shear distribution at the cross section; $c = 1.0 \le 0.5+\lambda_v \le 1.5$;

λ_v is the shear slenderness $\lambda_v = \psi \cdot \frac{h}{l}$, with ψ = 1.0 for cantilever systems and ψ = 0.5 for full restraint walls.

(2) For masonry with filled heat joint eq. (6.13a) don’t need to be used.

(3)The characteristic tensile strength of the unit, f_{bt}, may be determined from either:

the evaluation of a database on the results of splitting tests on the tensile strength of masonry units,

or

from the values given in ….

(4)P The connections between shear walls and flanges of intersecting walls shall be verified for vertical shear.

(6) The calculated length of a wall may be taken from equation (6.14)

$$l_{cal} = \frac{3}{2} l - 3 e_{ini} - 3 \frac{V_{Ed} h}{N_{Ed}} \left(\psi - \frac{1}{2} \right) \leq l - 2 e_{ini} \tag{6.14}$$

where:

e_{ini} is the initial eccentricity of the vertical loads at the top of the wall;

h is the height of the wall;

l is the length of the wall;

ψ is a factor for the consideration of restraining effects at the top of the wall, the value should be chosen between 0.5 for a full restrained system and 1.0 for a cantilever system.

(5) The length of the compressed part of the wall should be verified for the vertical loading applied to it and the vertical load effect of the shear loads.

10.2. Shear verification on the basis of shear strength

The formulation in chapter 6.2 should be changed to:

6.2 Unreinforced masonry walls subjected to shear loading

(1)P At the ultimate limit state the design value of the shear load applied to the masonry wall, V_{Ed}, shall be less than or equal to the design value of the shear resistance of the wall, V_{Rd}, such that :

$$V_{\mathrm{Ed}} \leq V_{\mathrm{Rd}} \tag{6.12}$$

(2) The design value of the shear resistance is given by the minimum of the shear resistance due to gaping, friction and unit failure :

$$V_{\mathrm{Rd}} = \frac{f_{\mathrm{vd}}}{c} \, t \, l_{\mathrm{cal}} \tag{6.13b}$$

where:

f_{vd} is the design value of the shear strength of masonry, obtained from 2.4.1 and 3.6.2, based on the average of the vertical stresses over the compressed part of the wall that is providing the shear resistance;

t is the thickness of the wall resisting the shear;

l_{cal} is the calculated length of the wall, in case of gaping of the unit and friction the calculated length is identical to the length of the compressed part of the wall l_c; For shear strength due to tensile failure of the units see clause (5).

c is the factor to consider the shear distribution at the cross section; $c = 1.0 \leq 0.5+\lambda_v \leq 1.5$;

λ_v is the shear slenderness $\lambda_v = \psi \cdot \frac{h}{l}$, with ψ = 1.0 for cantilever systems and ψ = 0.5 for full restraint walls.

(3) The length of the compressed part of the wall, l_c, should be calculated assuming a linear stress distribution of the compressive stresses, and taking into account any openings, chases or recesses; any portion of the wall subjected to vertical tensile stresses should not be used in calculating the area of the wall to resist shear.

(4)P The connections between shear walls and flanges of intersecting walls shall be verified for vertical shear.

(5) The calculated length of a wall may be taken from equation (6.14)

$$l_{cal} = \frac{3}{2} l - 3e_{ini} - 3\frac{V_{Ed}h}{N_{Ed}}\left(\psi - \frac{1}{2}\right) \leq l - 2e_{ini} \qquad (6.14)$$

where:

e_{ini} is the initial eccentricity of the vertical loads at the top of the wall;

h is the height of the wall;

l is the length of the wall;

ψ is a factor for the consideration of restraining effects at the top of the wall, the value should be chosen between 0.5 for a full restrained system and 1.0 for a cantilever system.

(6) The length of the compressed part of the wall should be verified for the vertical loading applied to it and the vertical load effect of the shear loads.

The formulation in chapter 3.6.2 should be changed to:

3.6.2 Characteristic shear strength of masonry

(1)P The characteristic shear strength of masonry, f_{vk}, shall be determined from the results of tests on masonry.

(2) The characteristic shear strength of masonry, f_{vk}, due to gaping of single units or struts using general purpose mortar in accordance with 3.2.2(2), or thin layer mortar in beds of thickness 0,5 mm to 3,0 mm, in accordance with 3.2.2(3), or lightweight mortar in accordance with 3.2.2(4) with all joints satisfying the requirements of 8.1.5, may be taken from equation (3.5).

$$f_{vk} = \left(f_t + \sigma_d\right)\frac{l_{ol}}{h_b} \tag{3.5a}$$

where:

h_b is the height of the masonry unit;

l_b is the length of the masonry unit;

l_{ol} is the overlapping length of the masonry bond, for a regular bond it could be taken half of the unit length;

σ_d is the design compressive stress perpendicular to the shear in the member at the level under consideration, using the appropriate load combination based on the average vertical stress over the compressed part of the wall that is providing shear resistance;

f_t is the characteristic tensile bond strength.

(3)The tensile bond strength of the masonry, f_t, may be determined from either:

the evaluation of a database on the results of tests on the tensile bond strength of masonry,

or

by calculating from the values for bending strength perpendicular to the bed joints given in chapter 3.6.3 by $f_t = f_{x1}/\ 1.5$

(4) The characteristic shear strength of masonry, f_{vk}, due to friction using general purpose mortar in accordance with 3.2.2(2), or thin layer mortar in beds of thickness 0,5 mm to 3,0 mm, in accordance

with 3.2.2(3), or lightweight mortar in accordance with 3.2.2(4) with all joints satisfying the requirements of 8.1.5, may be taken from equation (3.5).

$$f_{\mathrm{vk}} = f_{\mathrm{vko}} + 0{,}4\,\sigma_{\mathrm{d}} \tag{3.5}$$

where:

f_{vko} is the characteristic initial shear strength, under zero compressive stress; the perpend joints unfilled, but with adjacent faces of the masonry units closely abutted together, f_{vko} have to divide in half;

(5) In shell bedded masonry, where the units are bedded on two or more equal strips of general purpose mortar, the initial shear strength f_{vko} has to be reduced by the relation of the width of the stripes to the width of the wall.

(6) The characteristic initial shear strength of masonry, f_{vko}, should be determined from tests in accordance with EN 1052-3 or EN 1052-4.

(7)The initial shear strength of the masonry, f_{vko}, may be determined from either:

the evaluation of a database on the results of tests on the initial shear strength of masonry,

or

from the values given in table 3.4, provided that general purpose mortars made in accordance with EN 1996-2 do not contain admixtures or additives.

(8) The characteristic shear strength of masonry, f_{vk}, due to tensile failure of the units, may be taken from equation (3.6)

$$f_{vk} = 0.22 \cdot f_{bt}\sqrt{1 + 5\frac{\sigma_d}{f_{bt}}} \tag{3.6a}$$

$$f_{vk} = 0.1 \cdot f_{bt}\sqrt{1 + 16\frac{\sigma_d}{f_{bt}}} \qquad \text{for AAC-masonry} \tag{3.6b}$$

where:

σ_{d} is the design compressive stress perpendicular to the shear in the member at the level under consideration, using the appropriate load combination based on the average vertical stress over the calculated length of the wall that is providing shear resistance;

f_{bt} is the characteristic tensile strength of the unit;

f_{bk} is the characteristic compressive strength of the unit;

α is a factor to consider the shear stress distribution in the unit and the wall.

(9) The characteristic tensile strength of the unit, f_{bt}, may be determined from either:

the evaluation of a database on the results of splitting tests on the tensile strength of masonry units,

or

from the values given in ….

10.3. Bending

The formulation in chapter 6.1.2.1 clause 2 should be changed to:

(2) The design value of the vertical resistance of a single leaf, N_{Rd}, is given by:

$$N_{Rd} = \Phi\, t\, f_d \tag{6.2}$$

where:

Φ is the capacity reduction factor, Φ_i, at the top or bottom of the wall, or Φ_m, in the middle of the wall, as appropriate, allowing for the effects of slenderness and eccentricity of loading in or out of plane, obtained from 6.1.2.2;

t is the thickness of the wall or in case of shear walls, the length of the wall;

f_d is the design compressive strength of the masonry, obtained from 2.4.1 and 3.6.1.

NOTE For the out of plane eccentricity the verification should be made per unit length.

6.1.2.2 Reduction factor for slenderness and eccentricity

(1) The value of the reduction factor for slenderness and eccentricity, □, may be based on a rectangular stress block as follows:

(i) At the top or bottom of the wall (Φ_i)

$$\Phi_i = 1 - 2\frac{e_i}{t} \tag{6.4}$$

where:

e_i is the eccentricity at the top or the bottom of the wall, as appropriate, calculated using the equation (6.5) or in case of shear walls with an eccentricity in plane (6.5b):

$$e_i = \frac{M_{id}}{N_{id}} + e_{he} + e_{init} \geq 0{,}05\, t \tag{6.5}$$

$$e_i = \frac{V_{Ed} \cdot h_{cal}}{N_{id}} + e_{init} \tag{6.5b}$$

M_{id} is the design value of the bending moment at the top or the bottom of the wall resulting from the eccentricity of the floor load at the support, analysed according to 5.5.1 (see figure 6.1);

V_{Ed} is the design value of the horizontal load at the top of the wall;

N_{id} is the design value of the vertical load at the top or bottom of the wall;

e_{he} is the eccentricity at the top or bottom of the wall, if any, resulting from horizontal loads (for example, wind);

e_{init} is the initial eccentricity (see 5.5.1.1);

h_{cal} calculated height of a wall;

t is the thickness of the wall.

In case of a restraint support at the top of the wall ;

$$h_{cal} = \psi \cdot h$$

ψ is a factor for the consideration of restraining effects at the top of the wall, the value should be chosen between 0.5 for a full restrained system and 1.0 for a cantilever system.

h clear height of a masonry wall to the level of the load;

(2) For a high initial eccentricity or a full restrained system, the vertical resistance for shear walls should also be verified at the top of the wall.

11. Comparison of the standards, the proposed Equations and test results

11.1. Comparison of Proposal, DIN 1053-100 and EN 1996-1-1

Based on a sample wall, the proposal should be compared to the current state of the German standard DIN 1053-100 [3] and the European standard EN 1996-1-1 [5] in the following charts. For this the ratios subjected to the vertical loading will be presented for modern masonry with thin layer mortar and with unfilled head joints.

Table 22 Geometric data for the comparison

l_s [m]	h_s [m]	t [m]	l [m]	h [m]
0.25	0.25	0.24	2.5	2.5

Table 23 General material data for the comparison

	f_t [N/mm²]	f_{vk0} [N/mm²]
EN 1996-1-1 [a]	0.15	0.3 (0,4 CS)
DIN 1053-100	-	0.22[b]

[a] with the proposed amplification
[b] reduced value

For the unit tensile strength of the proposed equations the values given in Table 21 were used. For both standards the same partial safety factors were used for a better comparability. These are γ_E = 1.5 for the actions and γ_E = 1.5 for the material. The wall is considered as fully fixed (ψ = 0.5).

To simplify the verification of the friction, the tensile strength was not taken into account in the calculation of the overlapping length even in the light of the bond strength.

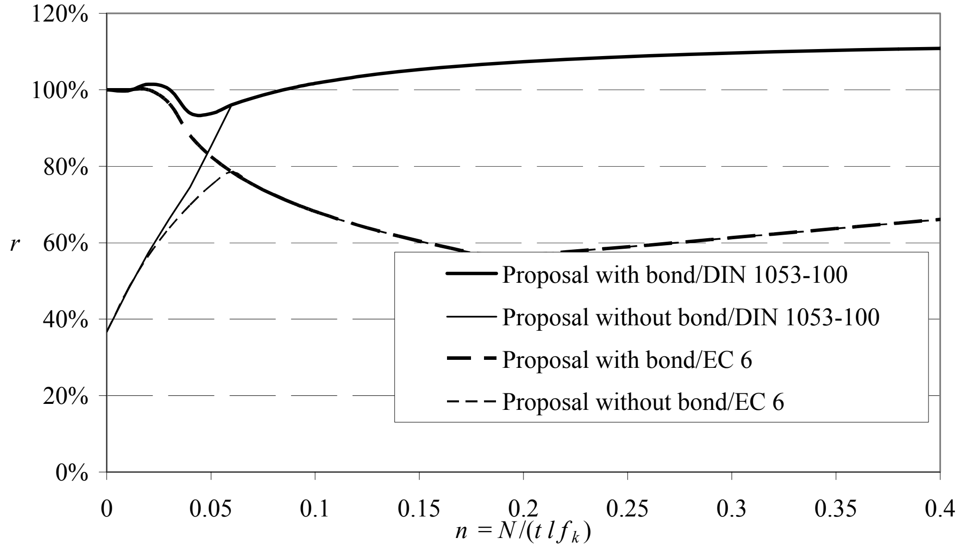


Figure 51 Comparison of proposal and standards for perforated Clay-bricks (f_{bk} = 12N/mm², with holes)

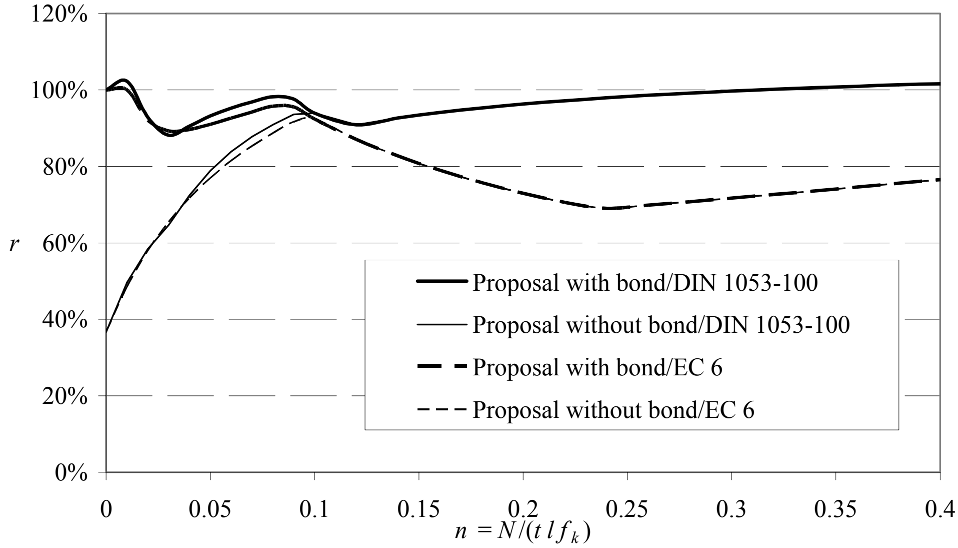


Figure 52 Comparison of proposal and standards for a CS-Wall (f_{bk} = 20N/mm², without any holes)

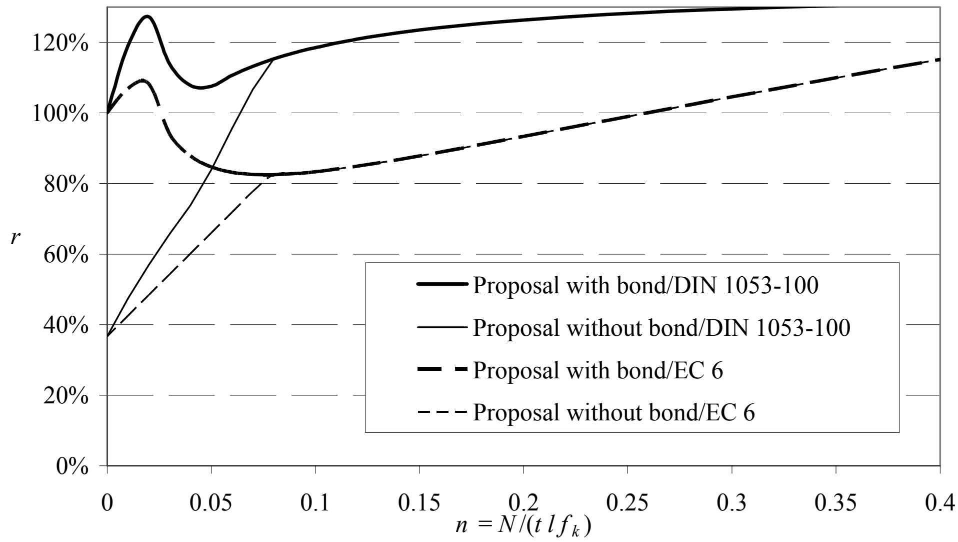


Figure 53 Comparison of proposal and standards for a AAC-Wall PP2 (f_{bk} = 2N/mm², without any holes)

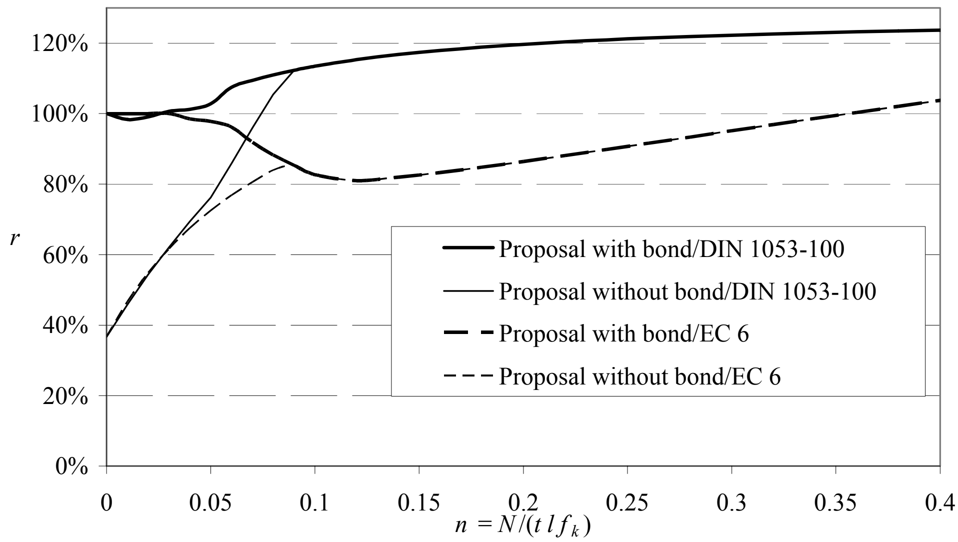


Figure 54 Comparison of proposal and standards for a LC-Wall (f_{bk} = 8N/mm², without any holes)

In the chosen example, there can be seen fractional major normative capacities compared to DIN 1053-100, while there is in relation to the EN 1996-1-1 a partial reduction. This is mainly due to the limit for the conditional failure of the unit tensile strength. Since in the current

EC 6 it is not depending on the vertical loads, it leads for smaller loads to an overestimation of the loading capacity.

With the chosen shear slenderness of 0.5, provided the equations a higher load capacity for all four combinations of materials with consideration of the bond strength. For shorter walls or larger shear slenderness even larger capacities can arise without consideration of the bond strength, especially at friction failure, than with consideration of the bond strength. Depending on the restraint level or length of the wall, the ratios can also vary due to the different definition of the factor *c* (see Figure 37).

11.2. Comparison with test results

In the following chart the ratios between the theoretical model (with bond strength) and experimental value (with and without safety factors) are described. The partial safety factors are for the action as well as for the material 1.5.

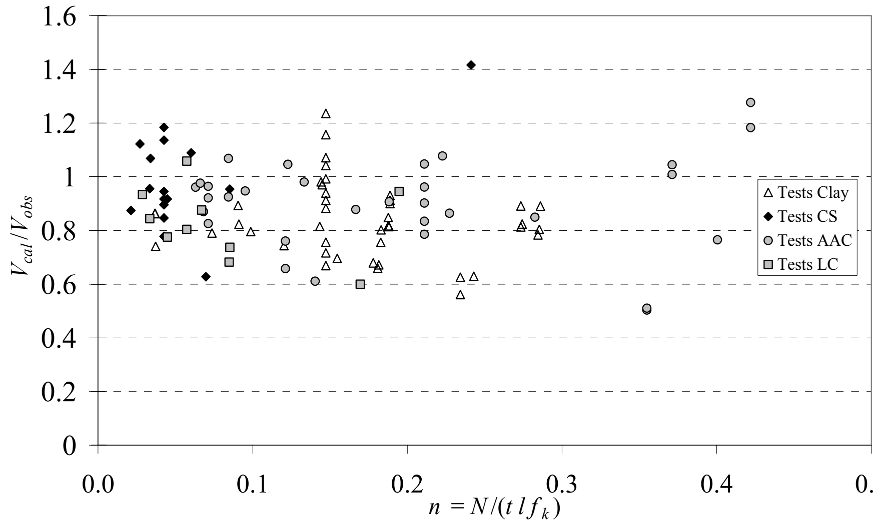


Figure 55 Comparison of the test result with the theoretical model without partial safety factors

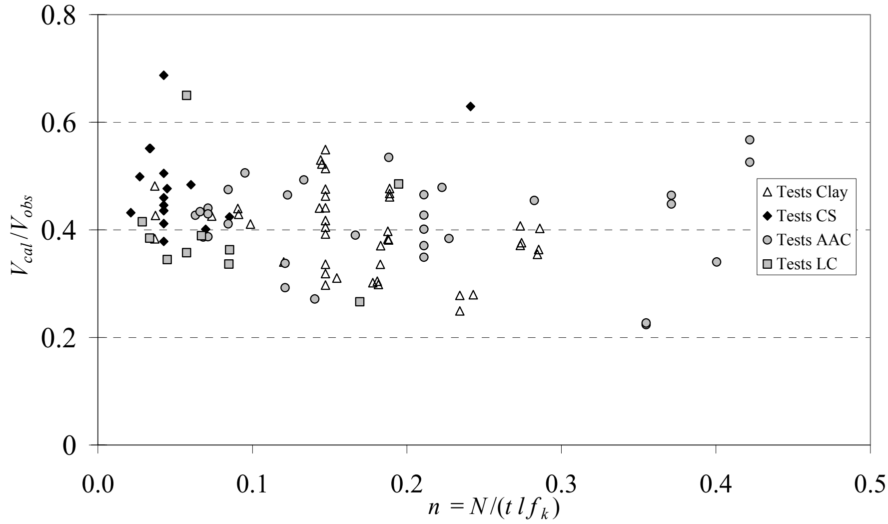


Figure 56 Comparison of the test result with the theoretical model with partial safety factors

In opposition to the proposed equations in the Deliverable 4.4 and [21], the consideration of the bond strength has no significant influence to the ratio values (cp. Figure 55 and Figure 10-4 in [20]). When using the partial safety factors it can be recognized, that none of the experimental values is above the allowed characteristic load. The minimum safety is 1/0.68 = 1.47.

12. Conclusion

The objective of this report is a proposal for an advanced design model of masonry under lateral loads for the purpose of implementation in Eurocode 6-1-1 (EC6).

Therefore, various analytical and empirical models as well as the existing standards have been discussed. An essential basis for this was the proposal from the Work Package 4 of the ESECMaSE project. For the analytically described failure modes at the joints (gaping and friction) an additional consideration of the bond strength was made. The equation for the tensile failure of the units was simplified.

A text proposal for the revision of the EC 6 was given after a brief reflection of the safety concept. The proposal was thereupon compared with the existing experimental results. It could be established, that within the terms of the tested range, an acceptable safety margin can be guaranteed.

For some test the relevant material data had to be taken from the literature. These values could in fact be accurately assessed in the meantime, but the material variety and the changing composition over the time, leads to some uncertainty.

An investigation of the overall failure probability is however needed.

A significant reserve in the design of masonry buildings against horizontal loads consists in determining the real forces for the member in shear. Through the rearrangement of the load distribution during the action of the wind or earthquake loads a significantly higher vertical load for the shear walls will be reached and thus a greater shear resistance will result. The numerical simulations and large scaled tests, which were performed in the framework of ESECMaSE showed that circumstance.

Since the test procedure will be continuing until the end of the project, the evaluation process will going on to the end. In particular, the interpretation of the failure shape in each test itself will give further evidence of the influencing factors and the proposed equations. With an increase of the data base also a distinction between Clay, CS and LC instead of a common rule is possible.

The confined masonry could not be discussed within this report, because no new theoretical investigations were made and the relevant tests have not been completed or released till now. But the results will incorporated into an ongoing project beside ESECMaSE and lead there to an advanced design model.

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