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D9.2 Proposals for simplified rules for masonry buildings subjected to lateral loads

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Table of contents

1. Introduction

Based on the proposal of Deliverable 9.1 [12] simplified rules for the verification of masonry buildings subjected to lateral loads in plane will be given in this report. In the first part the limits of application are investigated and the proposed equations are simplified for standard applications. In a second version a relationship between the wind loaded area and the necessary total length of the shear walls are derived by assuming minimum material properties of the masonry. When it is obvious that the building is sufficiently braced and stabilised by shear walls it can be verified for wind loads more easily or the verification can even become obsolete.

2. State of standardisation

2.1. Eurocode 6 (EN 1996-1-3)

Simplified rules for design verifications are given in Part 3 of the Eurocode 6 (EN 1996-3 [8]), just as rules for shear are included there. Within chapter 4.4 a different notation of the equations of the first part of the EC 6 [7] is given. The major difference is the limitation of vertical loads to:

$$N_{Sd} \leq 0.5 \cdot l \cdot t \cdot f_d \quad (1)$$

This regulation shall prevent a bending failure of the shear walls. But even under less vertical loading a bending failure could occur.

An alternative way to neglect a detailed verification of the shear walls subjected to wind actions is given in Annex A.3 of the EC 6 Part 3[8].

The terms for the arrangement of shear walls are complied with, if:

- the characteristic wind load does not exceed 1.3 kN/m²;
- there are at least two walls or more in both perpendicular directions;
- the shear walls are load bearing and the load resistance of the shear walls excluding wind loads is verified in accordance with 4.2 in [8] assuming a reduced compressive strength of masonry of 0.8 f_k;
- the layout of the shear walls in plan is approximately symmetrical in both directions or at least in one direction if the ratio l_{bx}/l_{by} is limited to 3;

- in the plan the centre lines of the shear walls do not meet in one point;
- the sum of the web areas of shear walls in each perpendicular direction, considering only webs with a length of more than 0,2 h_{tot} and excluding flanges, complies with the following conditions:

$$\sum t \cdot l_{sx}^{2} \geq c_s \cdot l_{by} \cdot h_{tot}^{2} \text{ and } \sum t \cdot l_{sy}^{2} \geq c_s \cdot l_{bx} \cdot h_{tot}^{2} \qquad (2)$$

Where:

l_{bx}, l_{by}	are the plan dimensions of the considered building, where $l_{bx} \geq l_{by}$;
l_{sx},l_{sy}	are the shear wall lengths;
h_{tot}	is the height of the building;
c_s	$= c_t\ c_i\ w_{Sk}$
c_t	is a constant depending on α, obtained from Table 1;
c_i	= 1.0 for rectangular shear walls;
	= 0.67 for I-profiled shear walls with flange areas greater than 0.4 $t\ l$;
α	is the average of the ratio $\frac{N_{Sd}}{A \cdot f_d}$ of the shear walls being considered;
N_{Sd}	is the design value of the vertical load in a shear wall;
A	is the cross-sectional area of a shear wall;
f_d	is the design compressive strength of the masonry;
w_{Sk}	is the characteristic wind load, in kN/m².

Table 1 Values of c_t in [m²/kN] from Table A1 of [8]

α	f_k [N/mm²]			
	2	4	6	≥ 8
0,2	0,0192	0,0095	0,0064	0,0048
0,3	0,0128	0,0064	0,0042	0,0032
0,4	0,0095	0,0048	0,0032	0,0024
0,5	0,0075	0,0038	0,0025	0,0019
0,6	0,0095	0,0048	0,0032	0,0024
0,7	0,0128	0,0064	0,0042	0,0032

This simplified rule was developed by *Reeh/Schlund*. The derivation is described in [14] and bases on the verification of bending only. Shear failures like friction failure or tensile failure of the units are not included.

2.2. German standard (DIN 1053-100)

The German standard DIN 1053-100 [2] also bases on two steps of simplification. The first one is only a simplification of the equations for shear strength. The tensile failure of the units is reflected by one limiting value depending only on the characteristic compressive strength of the unit. This approach is similar to the EC 6 Part 1.

Table 2 Highest value of the shear strength from [2]

Kind of masonry unit	max. f_{vk}
hollow bricks	$0.012 \cdot f_{bk}$
bricks and units with grip holes or grip pockets	$0.016 \cdot f_{bk}$
bricks without grip holes or grip pockets	$0.020 \cdot f_{bk}$

The other often used simplification is given by a clause.
„A verification of the shear capacity of a building can be neglected, if the slabs are working as stiff panels or there are verified ring beams with a sufficient stiffness. And in both direction have to be obviously sufficiently bracing walls with also an adequate length. The bracing walls have to be without a major weakening and without an offset from the foundations to the top. " (from [2]).
Based on his experiences the designing engineer has to decide whether the above clause condition is fulfilled. But in general this can only result in a subjective decision.

2.3. ÖNORM B 1996-3

The national annex of the Austrian Republic gives an alternative method for a simplified verification of the shear resistance of a building. In [9] in table 2 minimal areas of the cross section for the shear walls are defined. In addition to this, some general design rules are listed.

Table 3 Minimal cross section area of shear walls per orthogonal direction from [9]

Number of storeys	$f_b \geq 10$ N/mm²; $f_m \geq 5$ N/mm²	$f_b \geq 5$ N/mm²; $f_m \geq 2.5$ N/mm²
1	2.0%	2.0%
2	2.0%	3.0%
3	2.0%	4.5%
4	3.0%	not allowed
5	4.5%	not allowed
6	6.0%	not allowed

The average length of the shear walls per direction has to be at least 100% of the storey height if 35% of the vertical loads are carried by the shear walls or at least 50% if 65% of the vertical loads are carried by the shear walls. The slabs have to work as stiff panels to distribute the wind load.

This arrangement refers to the simplified method as laid down in the Eurocode 8. This leads to direct conjunction of the building with the area of the shear walls. If the depth of the building is doubled the cross section is also doubled. But the influence of the depth for the wind load is not that high.

2.4. Proposed equations

In Deliverable 4.3 [11] some equations for the calculation of the shear load capacity were proposed. The major difference to existing standards is an added failure criterion. Another difference is that any bond strength is neglected. In [12] the proposal was enhanced for the use of the bond strength to get better results for minor vertical loads. For the simplified model equations are used that do not consider the bond strength. This results in a simplification of the calculation for the load bearing capacities. The equations are:

<u>Gaping</u>

$$V_{Rd} = \frac{N_{Ed}}{\gamma_M}\left(\frac{l_{ol}}{h_b} + \frac{l_b - l_{ol}}{h}\right) \quad (3)$$

Friction

$$V_{Rd} = \frac{\mu \cdot N_{Ed}}{\gamma_M} \tag{4}$$

Tensile failure of the unit

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

For masonry made of Autoclaved Aerated Concrete (AAC):

$$V_{Rd} = \frac{t \cdot l_{cal}}{c \cdot \gamma_M} 0.1 \cdot f_{bt} \sqrt{1 + \frac{16 \cdot N_{Ed}}{f_{bt} \cdot t \cdot l_{cal}}} \tag{6}$$

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;

γ_M for eq. (3) a partial safety factor of 1.35 could 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;

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.

The calculated length of the wall becomes:

$$l_{cal} = \frac{3}{2} l - 3e_{ini} - 3\frac{V_{Ed} h}{N_{Ed}} \left(\psi - \frac{1}{2} \right) \leq l - 2e_{ini} \qquad (7)$$

Where:

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

l is the length of the wall.

3. Simplification of the design procedure - Version1

A general way to develop simplified rules is to reduce the complexity of the design procedure by means of limiting the variation of input parameters. There are three possible simplifications for the proposed equation (see Deliverable 9.1 [12] or chapter 2.4). The first one is to limit the size of the unit in order to neglect the gaping failure. The second is to simplify the equation for the tensile failure of the unit by determining a limiting value in analogy to the existing EC 6.
For short and slightly loaded walls only the failure due to bending becomes the decisive factor. This criterion could be used as third simplification.

3.1. Limitation of the unit size in order to neglect the gaping failure

Gapping becomes crucial for rather high ratios of height to length of the masonry unit. This is known for many years and investigated in some research projects. But for elder masonry units it was assumed as negligible due to the lower aspect ratio. Through the comparison of equations (3) and (4) a limit for the use of eq. (3) can be found.

$$\left(\frac{l_{ol}}{h_b} + \frac{l_b - l_{ol}}{h}\right) \leq \mu \frac{\gamma_{M,gaping}}{\gamma_{M,friction}} \qquad (8)$$

The relation of the size of the unit to the height of the wall has a positive influence. It means that the shear load capacity due to gaping increases for large units. Neglecting of this term gives a limiting value on the safety side. Through this eq. (8) results in:

$$\frac{l_{ol}}{h_b} \geq \mu \frac{\gamma_{M,gaping}}{\gamma_{M,friction}} \qquad (9)$$

For the proposed values for the partial safety factors and a friction coefficient of 0.6 the limiting value becomes $\frac{l_{ol}}{h_b} \geq 0.6 \frac{1.35}{1.5} = 0.54$.

For regular bond (overlapping length could be assumed to half of the unit length) the height of the unit should be less than 0.93 of the unit length. The following diagram shows the relation of the shear load capacity due to gaping and due to friction.

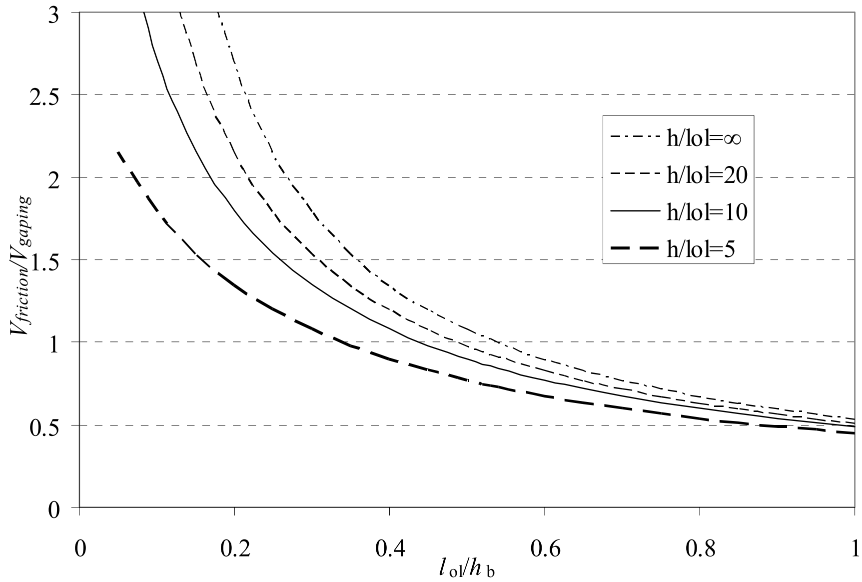

Figure 1 Relation of the shear load capacity due to gaping and due to friction

From Figure 1 it can be assumed, that for typical modern masonry with an aspect ratio of unit height to length of one and regular bond, the verification of gaping can be neglected.

3.2. Simplified rule for the verification of tensile failure of the unit

The simplest way to reduce the complexity of the equations (5) and (6) is to evaluate the equations for a zero vertical load. But assuming this the failure criterion of the unit does not become decisive. The friction or the bending failure leads to a smaller shear load capacity. That is why both the equation for friction (4) and the equations for tensile failure of the unit (5) and (6) where used for the simplification. The following figure illustrates this for AAC.

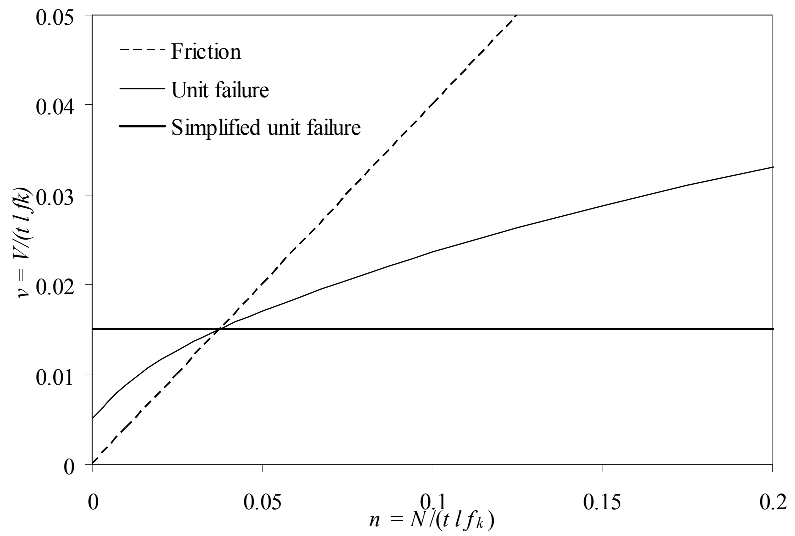

Figure 2 Simplification for the failure due to tension in the masonry unit for AAC (f_{bt}/f_k=0.1, c=1.0)

For higher loads this approach will reduce the shear load capacity in relation to eq. (5) and (6), while it is always on the safe side in comparison to the full design method.

The intersection of equation (4) and the equation for the tensile failure of the unit can be calculated by:

$$n = \frac{l_{cal}}{l} \frac{f_{bt}}{f_k} \frac{\alpha^2 + \alpha\sqrt{\alpha^2 + 4\mu^2 c^2 \beta^2}}{\mu^2 c^2 \beta^2} \tag{10}$$

The parameters α and β are the numbers in eq. (5) and (6) (e.g. β = 1/5 for common masonry).

The substitution of the calculated vertical load in equation (4) or eq. (5) and (6) gives the values for the simplified design equation.

$$V_{Rd} = \delta \cdot t \cdot l_{cal} \cdot \frac{f_{bt}}{\gamma_M} \tag{11}$$

With the proposed equation from [12] the parameter δ becomes:

Table 4 Value δ for simplified equation for the tensile failure of the unit (eq. (11); μ = 0.6)

	c = 1.0		c = 1.5	
	Common	AAC	Common	AAC
δ	0.5	0.3	0.262	0.148

If the characteristic compressive strength of the unit is used instead of the tensile strength eq. (11) becomes:

$$V_{Rd} = \delta \cdot t \cdot l_{cal} \cdot \frac{f_{bk}}{\gamma_M} \tag{12}$$

The following table gives the parameter δ according to table 21 in [12].

Table 5 Value δ for simplified equation for the tensile failure of the unit (eq. (12); μ = 0.6)

	Clay	CS	LC	AAC	
				$f_{bk} \leq$ 2 N/mm²	$f_{bk} >$ 2 N/mm²
δ (c = 1.0)	0.0175	0.025	0.035	0.036	0.024
δ (c = 1.5)	0.0092	0.0131	0.0183	0.0178	0.0118

The values are partially higher than those in Table 2 (DIN 1053-100).
For a further simplification the minimum calculated length of the wall l_{cal} could be assumed to ¾ l. For smaller calculated wall lengths, bending or overturning become decisive (see figure 45 in [12]). For a fully restrained wall l_{cal} is equal to l.

3.3. Limits for verification due to bending

Short walls or walls with low vertical loads typically fail due to bending. For the proposed equation the limiting shear slenderness will be given below. To assure a better handling the standardised notation will be used.

3.3.1. Limit between bending and gaping

If the tensile strength is neglected for the verification of bending and a stress block is assumed the verification of bending becomes decisive if:

$$\frac{1}{2\cdot\lambda_v}\cdot\left(n-n^2\cdot\gamma_{M,B}\right)\leq\frac{n}{\gamma_{M,G}}\left(\frac{l_{ol}}{h_b}+\frac{l_b-l_{ol}}{h}\right) \quad (13)$$

$$\lambda_v\geq\frac{\gamma_{M,G}\left(1-n\cdot\gamma_{M,B}\right)}{2\cdot\left(\frac{l_{ol}}{h_b}+\frac{l_b-l_{ol}}{h}\right)} \quad (14)$$

For eq. (14) without safety factor and for regular bond see figure 3-20 in [11]. For l_{ol}/h_b =0.5 and $\gamma_{M,G}$ =1.35 the gaping has to be verified for a shear slenderness less or equal to 1.35.

3.3.2. Limit between bending and sliding

The initial shear strength for the verification of friction is neglected in the following.

The verification of bending becomes decisive if:

$$\frac{1}{2\cdot\lambda_v}\cdot\left(n-n^2\cdot\gamma_M\right)\leq\frac{\mu\cdot n}{\gamma_M} \quad (15)$$

$$\lambda_V\geq\frac{\gamma_M\left(1-n\cdot\gamma_M\right)}{2\mu} \quad (16)$$

$$l\leq\frac{2\mu\cdot h\cdot\psi}{\gamma_M\left(1-n\cdot\gamma_M\right)} \quad (17)$$

$$n\geq\frac{\gamma_M-2\mu\cdot\lambda_V}{\gamma_M} \quad (18)$$

For eq. (16) without safety factor see figure 3-23 in [11]. For μ = 0.6 and γ_M =1.5 the friction has to be verified for a shear slenderness less or equal to 1.25.

If the initial shear strength is taken into account, the shear capacity has to be calculated according to eq. (60) in [12]:

$$V_{Rd}=\frac{\frac{3}{2}t\cdot l\cdot f_{vk0}+\mu\cdot N_{Ed}}{\gamma_M\cdot c+\frac{3\cdot t\cdot h\cdot\psi\cdot f_{vk0}}{N_{Ed}}}\leq\frac{1}{\gamma_M\cdot c}\left(t\cdot l\cdot f_{vk0}+\mu\cdot N_{Ed}\right) \quad (19)$$

The limit for the shear slenderness for low vertical loads becomes:

$$\lambda_v \geq \frac{\gamma_M \cdot c \cdot (1 - n \cdot \gamma_M)}{3 \cdot \overline{f_{vk0}} \cdot \gamma_M + 2 \cdot \mu} \tag{20}$$

For a reduced friction coefficient of 0.4 the limit of the shear slenderness is shown in the following diagram.

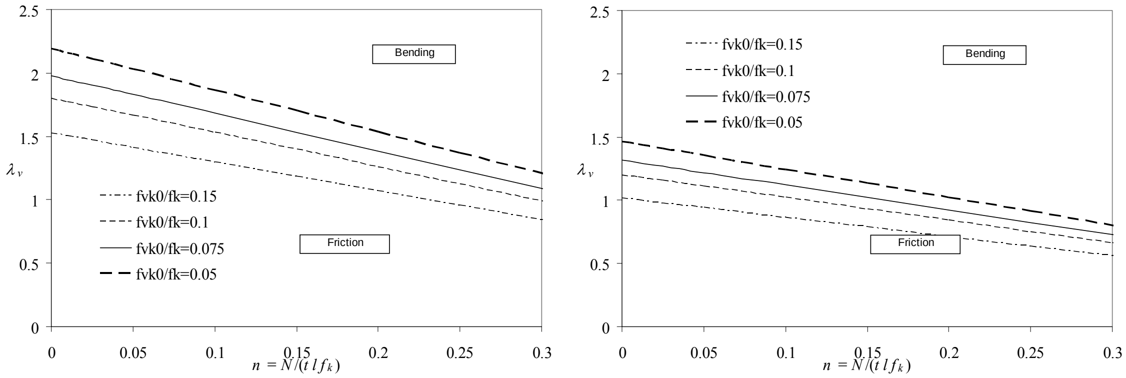

Figure 3 Limit of shear slenderness for failure due to friction and bending (μ=0.4, γ_M=1.5, left: c=1.5; right: c=1.0)

3.3.3. Limit between bending and tensile-failure of the unit

The comparison of the failure due to bending and due to the tensile failure of the unit leads to the following equation for the limiting shear slenderness.

$$\frac{1}{2 \cdot \lambda_v} \cdot (n - n^2 \cdot \gamma_M) \leq \frac{l_{cal}}{l} \frac{\alpha \cdot \overline{f_{bt}}}{c \cdot \gamma_M} \sqrt{1 + \frac{n \cdot l}{\beta \cdot \overline{f_{bt}} \cdot l_{cal}}} \tag{21}$$

The calculated length of the wall has to be determined by eq. (7). For a static system of cantilever the notation for λ_v becomes fairly complex, thus it is not shown here. For ψ=0.5 (fully restrained on top of the wall) the limiting shear slenderness can be calculated by:

$$\lambda_v \geq \frac{c \cdot \gamma_M \cdot (n - n^2 \cdot \gamma_M)}{2 \cdot \alpha \cdot \overline{f_{bt}} \sqrt{1 + \frac{n}{\beta \cdot \overline{f_{bt}}}}} \tag{22}$$

The following diagrams show the influence of the vertical load and the tensile strength of the unit.

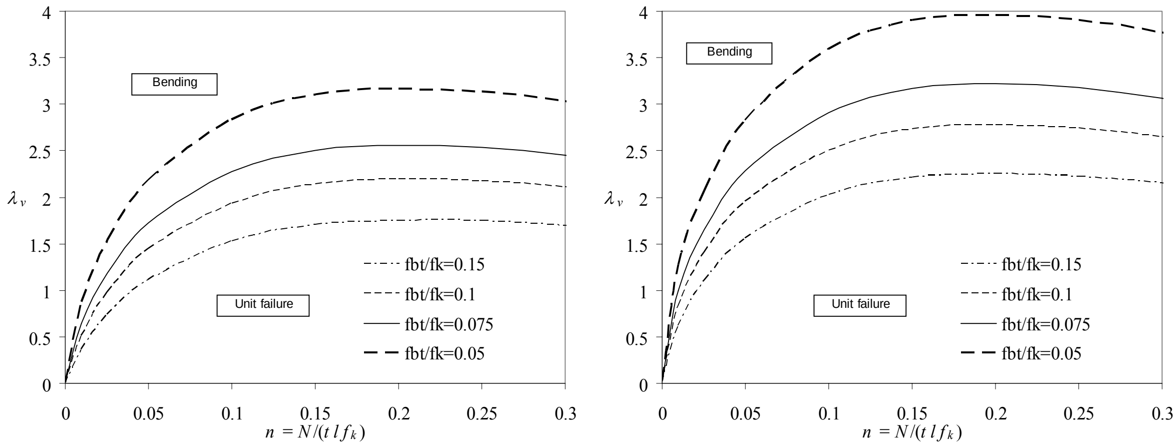

Figure 4 Limit of shear slenderness for failure due to tension in the unit and bending (γ_M=1.5, c=1.5, ψ=1.0, left: common masonry eq. (5); right: AAC eq. (6))

4. Simplification of the design procedure - Version 2

The aim is to provide a simplified method for typical housing geometries. Normally it is not a problem to provide enough shear walls within a masonry building for the required resistance against wind loads. But the simplified rule in the German standard (see chapter 2.2) requires structural engineering experiences and is still not satisfying. In the following a solution will be derived by making some basic assumptions.

4.1. Calculating procedure

The total characteristic wind load for the whole building can be determined by eq. (23). A more detailed explanation is given within the next chapter.

$$V_{Ed} = \gamma_E \cdot c_f \cdot q(z_e) \cdot h_t \cdot l_t \quad (23)$$

With

γ_E is the partial safety factor for design load;

h_t is the effective height of the building;

l_t is the total length of the building.

It is assumed, that half of the wind load affecting the ground floor is directly transferred to the foundation. So the height of the ground floor is to be cut in to half.

$$h_t = (n_{st} - 0.5) \cdot h_{st} \quad (24)$$

With

h_{st} is the height of a storey;

n_{st} is the number of storeys.

For the verification the design load has to be less or equal to the design resistance.

$$V_{Ed} \leq V_{Rd} \quad (25)$$

The design resistance is the sum of the resistance of all shear walls.

$$V_{Rd} = \sum V_{Rd1} = n_w \cdot V_{Rd1} \quad (26)$$

With

n_w is the number of stiffening walls of the same resistance (in minimum two).

The following calculation is made with a minimal wall length, which is equal for all walls.

Applying eq. (23) till (26) it could be written:

$$\gamma_E \cdot c_f \cdot q(z_e) \cdot (n_{st} - 0.5) \cdot h_{st} \cdot l_t = n_w \cdot V_{Rd1}(n_{st}) \tag{27}$$

The wind load as well as the shear resistance depends on the number of storeys.

The necessary number of shear walls with a minimum length becomes:

$$n_w = \frac{l_s}{l_w} = \frac{\gamma_E \cdot c_f \cdot q(z_e) \cdot (n_l - 0.5) \cdot h_{st} \cdot l_t}{V_{Rd1}(n_{st})} \tag{28}$$

Where:

l_s is the total length of all shear walls in one direction greater or equal to the minimal shear wall length;

l_w is the minimal shear wall length.

The required total length of all shear walls in direction results in:

$$l_s = \frac{\gamma_E \cdot c_f \cdot q(z_e) \cdot (n_l - 0.5) \cdot h_{st} \cdot l_t}{V_{Rd1}(n_{st})} l_w \tag{29}$$

To consider the shear resistance of a masonry wall a minimal vertical load is required. The assumption for this is described in the next chapter.

$$N_{Ed} = l_w \cdot q(n_{st}) \tag{30}$$

With

q is the load for the wall per metre length.

The calculation of the shear load capacity of one shear wall is made with the following equations.

$$V_{Rd1} = \min \begin{cases} \dfrac{1}{2 \cdot \lambda_v} \cdot \left(N_{Ed} - \dfrac{{N_{Ed}}^2 \cdot \gamma_M}{t \cdot l \cdot f_k} \right) \\ \dfrac{\mu}{\gamma_M} \cdot N_{Ed} \\ \dfrac{t \cdot l_{cal}}{c \cdot \gamma_M} \alpha \cdot f_{bt} \sqrt{1 + \dfrac{N_{Ed}}{\beta \cdot f_{bt} \cdot t \cdot l_{cal}}} \end{cases} \tag{31}$$

For masonry, built with regular bond and a maximum aspect ratio of one for the unit height to unit length, the gaping failure can be neglected. The second simplification is to neglect the bond strength.

For the assumption of a linear stress distribution in the centre of the wall the calculated length becomes:

$$l_{cal} = \frac{3}{2} l_w - 3 \frac{V_{Ed} h}{N_{Ed}} \left(\psi - \frac{1}{2} \right) \leq l_w \qquad (32)$$

Due to the assumed direct dependency of vertical load to wall length (eq. (30)) an increase of the shear capacity is mainly influenced by the wall length.

While the equation for the friction failure gives a proportional increase for the shear load capacity depending on the wall length, the equations for the bending failure and for the failure due to tension in the unit provide a disproportionately high increase of the capacity. For this reason, a minimum length of the shear walls has to be defined. Choosing bigger wall lengths for single walls, while retaining the same overall wall length, results in a higher shear load capacity of the building. In this case the simplification is on the safe side.
The minimum wall length is set to 1.5 metres. All subsequent calculations are made with this wall length.
As it can be seen from the equations, the wall thickness has only an influence on the equation for the unit failure and the bending failure. With an increasing wall thickness the utilisation of the vertical load capacity is decreasing. This means a shorter stress block for the bending, and thus a possibly higher eccentricity, which also leads to a higher shear load capacity. For the failure criterion of the unit an increasing shear wall thickness also means an increase of the load bearing capacity.
The minimum wall thickness is set to 0.175 m.

Bending of the building

The bending of the whole building was neglected in this report till now. If the bending of the building is only prevented by a column of shear walls, the shear slenderness is rather high. In this case only bending becomes decisive. The bending capacity results from:

$$V_{Rd} = \frac{l_w}{2 \cdot h_R} \cdot \left(N_{Ed} - \frac{{N_{Ed}}^2 \cdot \gamma_M}{t \cdot l_w \cdot f_k} \right) \qquad (33)$$

The vertical load can be increased by the weight of the shear wall in the ground floor. The lever arm of the resulting wind force is:

$$h_R = \left(\frac{n_{st}}{2} + 0.25 \right) h_{st} \quad (34)$$

For the necessary number of shear walls or the total wall length eq. (28) or (29) can be used.

4.2. General vertical load

The vertical load has a major influence on the shear capacity of a wall, but it depends on the floor plan of a building, the building height, the used material and the applied dead and live loads. For a general simplification of the shear design some assumptions have to be made, which should be on the safe side for the majority of cases.

Roof loads are normally carried by the external walls. That is why the shear walls are only loaded by slabs. For the verification of wind loads the minimal vertical load has to be taken into account. In this case only the dead loads are applied. The minimal thickness of the concrete slabs is assumed with 16cm.

- self weight of the slab 25kN/m³ · 0.16m = 4kN/m²
- floor screed 25kN/m³ · 0.05m = 1.25 kN/m²

Consequently the dead load per square metre results in g_{sl} = 5.25 kN/m². The density of masonry spreads from 5 to18 kN/m³. The self weight for a 17.5 cm thick and 2.75m high wall is:

$$\min g_w = 5\ kN/m^3 \cdot 0.175m \cdot 2.75m = 2.4kN/m \quad (35)$$

$$\max g_w = 18\ kN/m^3 \cdot 0.175m \cdot 2.75m = 8.7kN/m$$

For the shear wall at the ground floor the load per metre wall length is calculated by:

$$q = n_{st} \cdot g_{sl} + (n_{st} - 1) \cdot g_w \cdot A_{ref} \quad (36)$$

For an assumed contributively area of A_{ref} = 2.5m² per metre shear wall length the dead load is listed in the following table.

Table 6 Assumed dead load per metre length of a shear wall

n_f	min q	max q
	kN/m	kN/m
1	13.1	13.1
2	28.7	35.0
3	44.2	56.8
4	59.7	78.6
5	75.2	100.4

The numerical simulation in [15] indicates a minimal vertical load for shear walls of 60.10 kN/m, such as calculated for wall no. 1 at the top of a ground floor wall in a three storey building.
At 30% live loads the assumption of Table 6 for n_f = 3 leads to 48.7 kN/m for the minimum dead weight of the masonry and 61.3 kN/m for the maximum dead weight. In comparison with the numerical calculation in [15] the assumption with the minimal self weight of the wall is clearly on the safe side. An increase of the vertical loads, while a horizontal load is applied, is not taken into account.

4.3. Wind load

The principles of the German and the European code are the same. The wind load is assumed on the safe side. From the German code DIN 1055-4 [1] the acting wind forces on a building can be calculated from:

$$F_w = c_f \cdot q(z_e) \cdot A_{ref} \quad (37)$$

With:

c_f is the aerodynamic coefficient;

q is the dynamic pressure in kN/m²;

A_{ref} is the reference surface for the aerodynamic coefficient.

The aerodynamic coefficient can be calculated from

$$c_f = c_{f,0} \cdot \psi_r \cdot \psi_\lambda \tag{38}$$

With

$c_{f,0}$ is the basic value of the aerodynamic coefficient;

ψ_r is the reduction factor for quadratic cross sections with rounded corners;

ψ_λ is the reduction factor to consider the slenderness.

For the simplification the assumptions were made unfavourable. The basic value of the aerodynamic coefficient is set to 2.383.

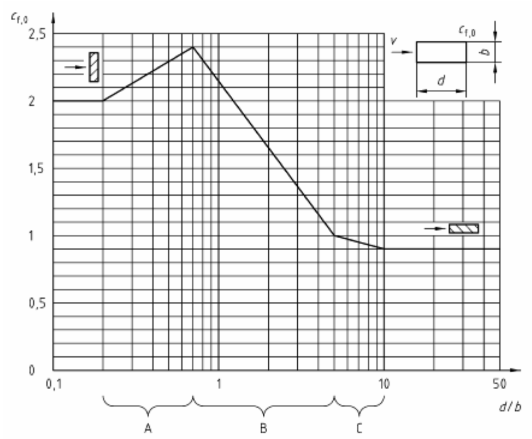

Figure 5 Basic value of the aerodynamic coefficient $c_{f,0}$ for a sharp-edged quadratic-section from [4]

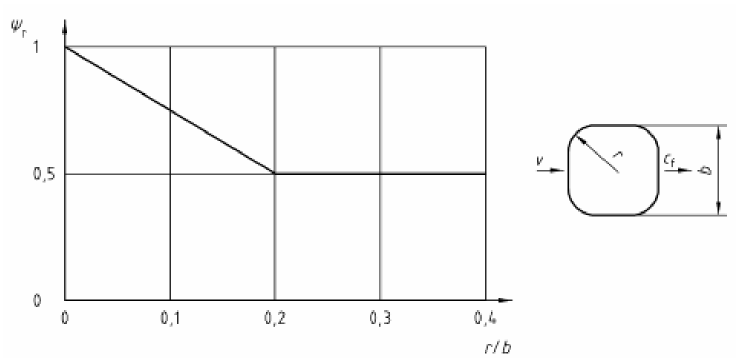

Figure 6 Reduction factor for quadratic cross sections with rounded corners ψ_r from [4]

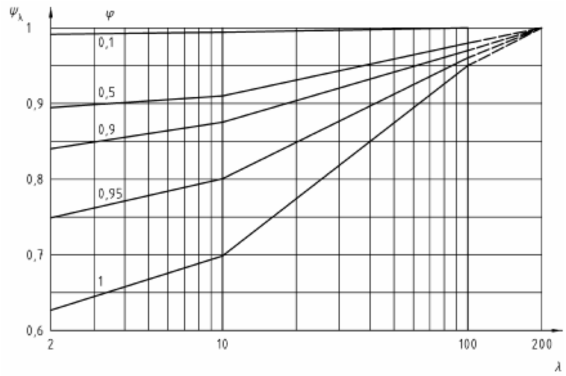

Figure 7 Reduction factor ψ_λ to consider the slenderness from [4]

Both reduction factors were assessed to 1.0; hence the aerodynamic coefficient c_f becomes 2.383.

For buildings with less than 25 m total height the code [4] gives simplified values for the blast dynamic pressure.

Wind zone		Blast dynamic pressure q in kN/m² for a building height h in the range of		
		$h \leq 10$ m	10 m < $h \leq 18$ m	18 m < $h \leq 25$ m
1	Inland	0.50	0.65	0.75
2	Inland	0.65	0.8	0.90
	Coast and islands of the Baltic Sea	0.85	1.00	1.10
3	Inland	0.80	0.95	1.10
	Coast and islands of the Baltic Sea	1.05	1.20	1.30
4	Inland	0.95	1.15	1.30
	Coast of the North and Baltic Sea and islands of the Baltic Sea	1.25	1.40	1.55
	Islands of the North Sea	1.40	-	-

Figure 8 Simplified blast dynamic pressure for buildings up to 25 m height from [4]

Figure 9 Map of the wind zones of Germany from [4]

Tensional effects as a result of eccentric wind loading are neglected in this report. In case of eccentric wind loads a detailed verification has to be executed.

4.4. Proposal for a minimum wall length

Beside the explained assumptions in the chapters before, some additional limits have to be defined for the following tables.

Table 7 Application limitations for the simplified method

	AAC	Common
$f_k \geq$	2.5 N/mm²	5.0 N/mm²
$f_{bt} \geq$	0.3 N/mm²	0.4 N/mm²
$t \geq$	17.5 cm	
$l_w \geq$ (of a single wall)	1.5 m	
$h_{st} \leq$	3.0 m	

The calculated values based also on safety factors of 1.5 for the action and the material. Likewise the aspect ratio of the units has to be smaller than one.

At first a general version should be shown. The needed minimum shear wall length can be calculated using the factor from the table, the length of the building facing the wind, the aerodynamic coefficient and the dynamic pressure.

$$l_s = \alpha \cdot l_t \cdot c_f \cdot q(z_e) \tag{39}$$

In the first table the values are given for non-restrained shear walls. In the second table full restraint on top of the wall was assumed.

Table 8 Factor α for calculating the necessary shear wall length for eq. (39) (ψ=1.0)

Number of floors	α [m²/kN] AAC	α [m²/kN] Common
1	0.72	0.70
2	1.05	0.99
3	1.32	1.10
4	1.55	1.18
5	1.77	1.27

Table 9 Factor α for calculating the necessary shear wall length for eq. (39) (ψ=0.5)

Number of floors	α [m²/kN] AAC	α [m²/kN] Common
1	0.43	0.43
2	0.93	0.59
3	1.27	0.81
4	1.54	1.00
5	1.77	1.17

The following tables give the necessary shear wall length per metre for wind zone 2 (inland) and the simplification of chapter 4.3.

$$l_s = \beta \cdot l_t \tag{40}$$

Table 10 Necessary shear wall length per metre building length that faces the wind for wind zone 2 and inland (ψ=1.0)

Number of floors	$q(z_e)$ [kN/m²]	β [-] AAC	Common
1	0.65	1.11	1.09
2	0.65	1.62	1.54
3	0.65	2.05	1.71
4	0.8	2.95	2.24
5	0.8	3.38	2.43

Table 11 Necessary shear wall length per metre building length that faces the wind for wind zone 2 and inland (ψ=0.5)

Number of floors	$q(z_e)$ [kN/m²]	β [-] AAC	Common
1	0.65	0.67	0.66
2	0.65	1.44	0.91
3	0.65	1.96	1.25
4	0.8	2.94	1.91
5	0.8	3.38	2.23

A building with a ground floor area of 10 m x 10 m and one storey needs 1.2% of this area for fully restrained shear walls and 1.9% in case of a cantilever system.
For more than four storeys the static system has only a minor influence, because the decisive failure for the shear walls in the ground floor is the tensile failure of the unit. The cross section at the half height of the wall is nearly fully under compression for both static systems. This leads to the same shear capacity.

Bending of the building

If the shear walls of the whole building act like a cantilever system over the total height, the necessary shear wall length significantly increases for buildings with more than three storeys.

Table 12 Necessary shear wall length per metre building length that faces the wind for wind zone 2 and inland due to bending of the building (l_w =1.5 m)

Number of floors	$q(z_e)$ [kN/m²]	β [-] AAC	Common
1	0.65	0.71	0.69
2	0.65	1.88	1.78
3	0.65	3.12	2.85
4	0.8	5.53	4.87
5	0.8	7.45	6.31

Due to the higher dead load the values for the first floor that are given here are smaller than those in Table 10.

If the minimum shear wall length is increased to 2 m, the total length can be reduced.

Table 13 Necessary shear wall length per metre building length that faces the wind for wind zone 2 and inland due to bending of the building (l_w =2.0 m)

Number of floors	$q(z_e)$ [kN/m²]	β [-] AAC	Common
1	0.65	0.53	0.52
2	0.65	1.41	1.33
3	0.65	2.34	2.13
4	0.8	4.15	3.65
5	0.8	5.59	4.73

The simplified rules (version 2) are valid only for:

- masonry built with a regular bond;
- a maximum aspect ratio of one for unit height to length;
- multi-storey buildings, if the floor plans are widely the same in every storey;
- a minimum shear wall length of 1.5 m and a minimal wall thickness of 0.175 m;
- symmetrically distributed shear walls;
- at least two shear walls per direction;
- bracing slabs, which are connecting all shear walls.

5. Summary

Verification with simplified design rules should be always on the safe side compared to the exact procedure. This means a verification, which could not be proven with the exact method, may not pass the simplified methods.
The proposal for a simplified procedure is divided into two parts, explaining two different methods. The first version used the proposed equations for the precise design method without any bond strength. The simplification was reached by reducing the allowable aspect ratio of the unit and a simplified equation for the tensile failure of the unit.
At a high level of shear slenderness always bending becomes decisive. But for medium vertical loads the limiting shear slenderness becomes higher. Particularly for AAC the tensile failure of the units plays a major role.
For the assessment of standard buildings, a second version of the simplified procedure was created. Now it is possible to omit the full design procedure by comparing the total length of all stiffening walls in a building with a simple reference value.

The proposals for simplified procedures are based on the proposals of Deliverable 9.1 and respectively Deliverable 4.3. These will be discussed in the further process of standardisation. Some receivable test results from the ESECMaSE project will also have an influence on the standardisation progress. That is why the developed simplifications in this report have to be adapted in this progress as well if the full design procedure will be updated.

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