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SHEAR DESIGN OF UNREINFORCED MASONRY PANELS
- BASIC ASSUMPTIONS AND COMPARISON OF DIFFERENT STANDARDS -

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ABSTRACT

The basics and the procedures of shear design according to several international design codes are significantly different. Reasons are different assumptions for the material behaviour, the safety concepts or the considered type of masonry. This results in different values for the maximum bearable horizontal force depending on the chosen design standard. To advance the state-of-the-art in masonry design a big European research program (ESECMaSE) dealing with the shear capacity of masonry panels has been established. The paper presents first results of the research activities.

KEYWORDS: shear design, shear resistance, shear capacity, DIN 1053, Eurocode 6, SIA 266

INTRODUCTION

In many cases horizontal forces due to wind or earthquakes are transmitted to the foundation through shear walls. Therefore shear is an important mechanism of structural resistance in masonry. The adopted design methods for lateral loading (shear) of masonry panels in Europe (e.g. European standard EC6 [1], German standards DIN 1053-1 [2], DIN 1053-100 [3] or Swiss code SIA 266 [4]) are based on different failure criteria with the boundary conditions valid in the mid 1970s. Ordinary they are simplified to enable a user-friendly design in practice.

Due to significant increases in the design loads for wind and earthquakes in the European standards it is feared that the methods for the structural design of load bearing masonry will no longer be sufficient in the future. In other words: the calculated shear resistance of masonry structures seems to be significantly lower than the one observed in practice. Nowadays the boundary conditions have changed significantly, also because units, mortars and the way of constructing masonry have been developed further. These (new) developments have so far not been considered in existing design methods.

This paper gives a short review of the above-quoted design codes and their basic failure criteria from *Mann/Müller* [5, 6, 7], *Simon/Graubner* [8, 9, 10], *Ganz/Thürlimann* [11, 12] and *Mojsilovic/Marti* [13, 14]. The transfers (simplifications and assumptions) to the above quoted design codes are also shown. The results of several standards are analyzed and compared.

BASICS OF SHEAR DESIGN IN GERMANY

The shear design concept of the new DIN 1053-100 (design on the basis of the semi-probabilistic safety concept) corresponds to a large extent to the concept of DIN 1053-1 (design on the basis of the global safety concept). Both German standards are based on the failure criteria from *Mann/Mueller*.

In 1973 *Mann/Mueller* [5, 6, 7] enhanced the already known *Coulomb*-friction concept which was the only possibility to describe the behaviour of shear stressed masonry walls at this time. They developed a failure hypothesis for shear stressed masonry walls which allowed the consideration of different material properties of the units, the mortar and the form of the joints. The basis of this hypothesis is a shear wall, stressed by horizontal and vertical forces which undergoes normal stresses s_x and shear stresses $t = t_{xy} = t_{yx}$. Horizontal compressive stresses were neglected.

Mann/Müller based their failure criteria on the following assumptions:

- common sized units with $l_{st} = 2 \cdot h_{st}$ (l_{st}, h_{st} = length, height of single unit (brick, stone))
- regular stretcher course with an overlap $u = h_{st}$
- bed joints do not transfer stresses, especially not shear stresses t_{yx}
- horizontal shear stresses t_{xy} are distributed uniformly along the bed joint
- compressive stresses (compression = negative) are distributed linearly along the bed joint, their mean value is equal to s_x
- head joints are not able to transfer shear stresses.

Due to this theory shear stresses exist only in the bed joints. Based on the equilibrium at a single unit *Mann/Mueller* developed 4 failure criteria for shear stressed masonry:

(0) **Opening of the bed joint** (In general not decisive, can be neglected)
(1) **Failure of bed joints due to friction** (modified *Coulomb*-friction concept with f_{vo} = initial shear strength (cohesion))
(2a) **Failure of tensile strength in the center of the units** (Ripping of the units due to exceeding the tensile strength of unit f_{bz})
(3) **Failure of masonry under compression** (Exceeding the compressive strength of masonry f)

Mann/Mueller developed their criteria for shear walls made of common sized units (h_{st} < 250 mm, l_{st} < 500 mm). Large (250 mm < h_{st} < 625 mm, 500 mm < l_{st} <1000 mm) masonry units have many advantages and are therefore used nowadays more and more on construction sites. The industry reacted to this new situation and developed special formats and big blocks. In 2001 *Simon/Graubner* [8, 9, 10] optimized the failure criteria of *Mann/Mueller* for masonry units with variable dimensions and overlapping lengths and added another criterion for failure at the edge of the unit which has been observed in many shear-tests of big size units:

(2b) **Failure of tensile strength at the edge of the units**

Figure 1 shows, the failure criteria of *Mann/Mueller* and *Simon/Graubner*. It can be seen that the u/h_{st} – ratio has an important influence and should not be neglected, when calculating the shear strength of masonry structures.

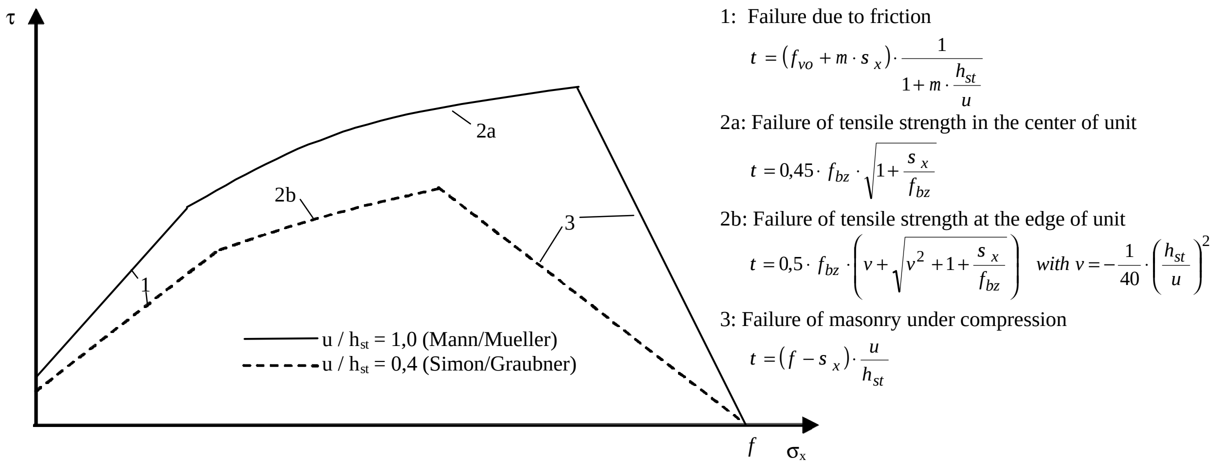

Figure 1 – Failure criteria of *Mann/Mueller* and *Simon/Graubner* (envelope)

The enhancements of *Simon/Graubner* has not yet been considered in DIN 1053-100. Instead, an additional standard for big sized units (DIN 1053-5) is planned, which will take the new developments into account.

SHEAR DESIGN ACCORDING TO DIN 1053-100

The safety concept in DIN 1053-100 follows the semi-probabilistic safety concept with a partial factor for materials g_M (in general g_M = 1.5) in connection with partial factors g_F depending on the kind of load. In general g_F = 1.5 for the horizontal loads, g_F = 1.0 for the vertical dead load and g_F = 0 for vertical live loads are used in the Ultimate Limit State.

$$g_F \cdot V_{Ek} = V_{Ed} \le V_{Rd} = \frac{V_{Rk}}{g_M} \qquad \text{Equation 4}$$

The design concept for shear walls in DIN 1053-100 is based on the theory of *Mann/Mueller* with the failure criteria (1), (2a) and (3) and the boundary conditions already mentioned. The characteristic value of the friction coefficient is set to $m_k = 0.6$ with leads to $\overline{m_k} = 0.4$ (c.f. equation 5).

$$\overline{m_k} = m_k \cdot \frac{1}{1 + m_k \cdot \frac{2 \cdot u}{l_{st}}} \cong 0.4 \qquad \text{Equation 5}$$

Considering these assumptions and simplifications the values for the characteristic shear strength f_{vk} of masonry structures are given depending on the different failure criteria (compression stresses positive):

Failure of bed joints due to friction

$$f_{vk} = k \cdot f_{vko} + \overline{m_k} \cdot s_{Dd} \qquad \text{Equation 6}$$

Failure of tensile strength of units (Ripping of the units)

$$f_{vk} = \frac{f_{bz}}{2.3} \cdot \sqrt{1 + \frac{s_{Dd}}{f_{bz}}} \qquad \text{Equation 7}$$

Failure of masonry under compression

$$f_{vk} = \left(f_d - s_{Dd} \right) \cdot g_M \qquad \text{Equation 8}$$

k	factor to consider the type of construction of the head joints: k = 0.5 for unfilled head joints k = 1.0 for filled head joints
f_{vko}	characteristic initial shear strength (cohesion)
s_{Dd}	compressive stress perpendicular to the bed joint, based on the average vertical stress over the compressed part of the wall, that is providing the shear strength, ignoring any part that is in tension
f_{bz}	tensile strength of the units
f_d	design value for the compressive strength of masonry in vertical direction (= $0.85 \cdot f_{xk} / g_M$)
g_M	partial factor on material side

Similarly to the regulations in the European Standards the shear resistance according to DIN 1053-100 will not be verified by comparing local shear stresses, but will be calculated by comparing the highest horizontal action with the shear resistance of the wall due to the vertical action.

$$g_F \cdot V_{Ek} = V_{Ed} \leq V_{Rd} = \frac{V_{Rk}}{g_M} = \frac{1}{g_M \cdot c} \cdot f_{vk} \cdot t \cdot l' \qquad \text{Equation 9}$$

c	factor to consider the distribution of the shear stresses in the compressed section of the wall: c = 1.5 if $h/l \geq 2$ c = 1.0 if $h/l \leq 1$ (interpolation is possible)
l, h, t	length, height, thickness of the wall
l'	compressed length of the wall

Considering the mechanical system for the wall shown inset in figure 6 and linear elastic material behaviour for calculating the compressed length l' of the wall the standardized shear resistance (bearable horizontal force) $v_{Rd} = V_{Rd} / (t \cdot l \cdot f_d)$ in relation to the standardized vertical force $n_{Ed} = N_{Ed} / (t \cdot l \cdot f_d)$ and the different failure criteria are given by the following equations [16, 17]:

Failure by friction

$$v_{Rd1,cr} = \frac{1.5 \cdot k_1 + \overline{m}_k \cdot n_{Ed}}{g_M \cdot c + \frac{3 \cdot k_1}{n_{Ed}} \cdot \frac{h}{l}} \qquad \text{if:} \quad \frac{h}{l} \geq \frac{g_M \cdot c}{6 \cdot \overline{m}_k} \quad \text{(cracked cross section)} \qquad \text{Equation 10a}$$

$$v_{Rd1,un} = \frac{1}{g_M \cdot c} \cdot \left[k_1 + \overline{m_k} \cdot n_{Ed}\right] \quad \text{if: } \frac{h}{l} < \frac{g_M \cdot c}{6 \cdot \overline{m_k}} \text{ and } n_{Ed} \geq \frac{k_1 \cdot h/l}{g_m \cdot c/6 - \overline{m} \cdot h/l} \qquad \text{Equation. 10b}$$

$k_1 = k \cdot f_{vko} / f_d$ standardized initial shear strength

Failure of unit by tension

$$v_{Rd2,cr} = \frac{1}{2} \cdot \frac{A \cdot B}{1 - A^2} \cdot \left(3 + \frac{n_{Ed}}{k_2}\right) \cdot \left[1 - \sqrt{1 + \frac{3 \cdot (3 + 2 \cdot n_{Ed} / k_2)}{(3 + n_{ED} / k_2)^2} \cdot \frac{1 - A^2}{A^2}}\right] \qquad \text{Equation 11a}$$

$$v_{Rd2,un} = B \cdot \sqrt{1 + \frac{n_{Ed}}{k_2}} \qquad \text{(uncracked cross section)} \quad \text{Equation 11b}$$

$k_2 = f_{bz} / f_d$ standardized tensile strength of the units

$$A = \frac{3}{2.3 \cdot g_M \cdot c} \cdot \frac{k_2}{n_{Ed}} \cdot \frac{h}{l}; \qquad B = \frac{k_2}{2.3 \cdot g_M \cdot c}$$

Failure due to compression caused by shear

$$v_{Rd3,cr} = \frac{1}{g_M \cdot c} \cdot \left[\frac{\frac{3}{2} - n_{Ed}}{\frac{1}{g_M} + \frac{3}{g_M \cdot c \cdot n_{Ed}} \cdot \frac{h}{l}}\right] \quad \text{if: } n_{Ed} \leq \frac{h/l}{c/6 + h/l} \qquad \text{Equation 12a}$$

$$v_{Rd3,un} = \frac{1}{c} \cdot \left[1 - n_{Ed}\right] \quad \text{if: } n_{Ed} > \frac{h/l}{c/6 + h/l} \qquad \text{Equation 12b}$$

In context with the calculation of a shear wall an additional verification under eccentric compression governs in the Ultimate limit state. In contrast to the verification of the shear force, the maximum bearable normal force N_{Rd} in this case will not be calculated on the assumption of a linear, but rather a rectangular stress-strain-relation within the compressed cross sectional area (c.f. equation 13).

Failure due to compression caused by bending

$$v_{Rd4} = \frac{1}{2} \cdot \left[n_{Ed} - n_{Ed}^2\right] \cdot \frac{1}{h/l} \qquad \text{Equation 13}$$

Figure 2 shows the standardized shear resistance for several brick-mortar-combinations and wall dimensions *h/l*. It is obvious that the shear resistance for tall panels (*h/l* >1) is less than for longer (*h/ l* < 1) ones. The positive effects of greater values for the initial shear strength of the mortar and the tensile strength can be shown as well.

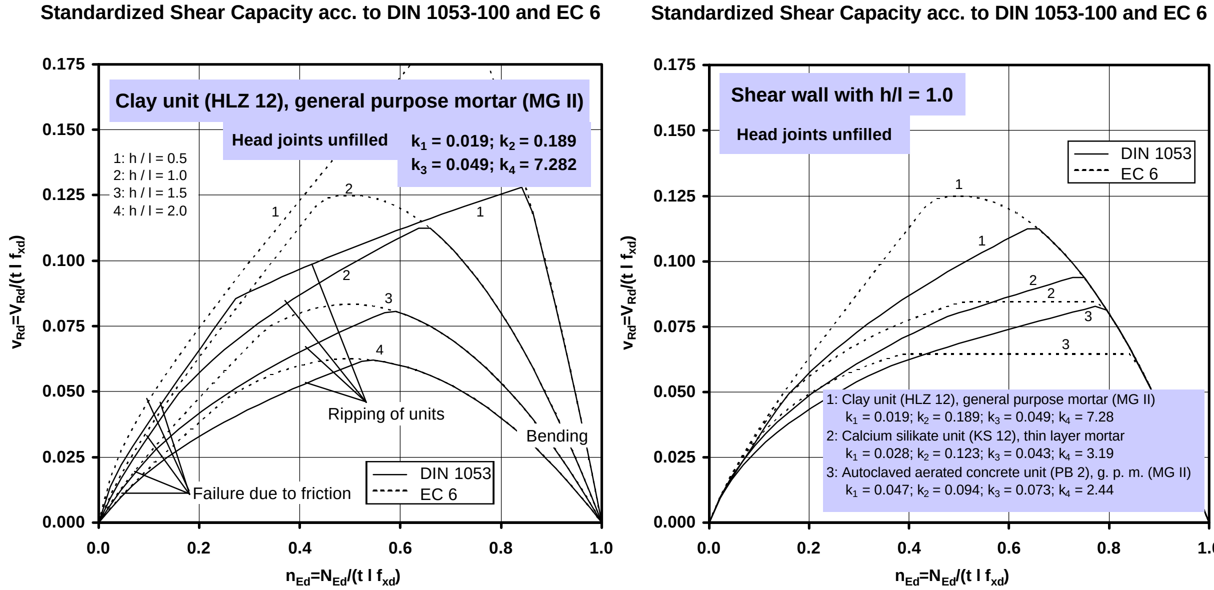

Figure 2 – Standardized Shear Capacity acc. to DIN 1053-100 and EC 6 for several brick-mortar-combinations (left) and several *h/l*-ratios (right)

SHEAR DESIGN ACCORDING TO EUROCODE 6

The shear design concept of Eurocode 6 is equal to the concept of DIN 1053-100, with the following exceptions:

- the parabolic distribution of the shear stresses for walls with $h > l$ is neglected (factor c = 1.0)
- for failure by friction the values of the characteristic initial shear strength are significantly greater than the values in DIN 1053-100

$$v_{Rd1,cr} = \frac{1.5 \cdot k_3 + \overline{m}_k \cdot n_{Ed}}{g_M \cdot c + \frac{3 \cdot k_3}{n_{Ed}} \cdot \frac{h}{l}} \qquad \text{if:} \quad \frac{h}{l} \geq \frac{g_M \cdot c}{6 \cdot \overline{m}_k} \quad \text{(cracked cross section)} \qquad \text{Equation 14a}$$

$$v_{Rd1,un} = \frac{1}{g_M \cdot c} \cdot \left[k_3 + \overline{m}_k \cdot n_{Ed} \right] \qquad \text{if:} \quad \frac{h}{l} < \frac{g_M \cdot c}{6 \cdot \overline{m}_k} \text{ and } n_{Ed} \geq \frac{k_3 \cdot h/l}{g_m \cdot c/6 - \overline{m} \cdot h/l} \qquad \text{Equation 14b}$$

$k_3 = k \cdot f_{vko,EC} / f_{d,EC}$ standardized initial shear strength due to EC 6

- the failure criterion for the tensile strength in EC 6 is restricted to a constant value of the shear strength, $f_{vk,max}$ = 0.065·f_b for filled head joints, and 0.045·f_b for unfilled head joints (f_b = normalized compressive strength of unit) and does not increase with greater values of n_{Ed}. Thus for regular masonry with unfilled head joints equation 11 can be simplified to

$$v_{Rd2,cr} = \frac{0.045 \cdot k_4}{\frac{2}{3} \cdot g_M + 2 \cdot \frac{0.045 \cdot k_4}{n_{Ed}} \cdot \frac{h}{l}} \qquad \text{if:} \quad n_{Ed} \leq \frac{0.045 \cdot k_4 \cdot h/l}{g_m / 6} \quad \text{(cracked)} \qquad \text{Equation 15a}$$

$$v_{Rd2,un} = \frac{0.045 \cdot k_4}{\gamma_M} \qquad \text{if:} \qquad n_{Ed} > \frac{0.045 \cdot k_4 \cdot h/l}{\gamma_m / 6} \qquad \text{(uncracked)} \qquad \text{Equation 15b}$$

$k_4 = f_b / f_{d,EC}$ characteristic compressive strength of the units related to the compressive strength of the masonry

- Failure by compression (equation 12) does not need to be verified, because for c = 1.0 equation 12 will not govern in practice compared to equation 13

Figure 2 shows clearly the effect of the greater values for the shear strength according to EC 6 in comparison with DIN 1053-100. This is founded both in the greater initial shear strength and in the higher maximum value due to failure of the unit (ripping). For units with lower values of standardized tensile strength the shear resistance according to EC 6 reaches a horizontal plateau before bending governs, whereas for the DIN 1053-100 the bearable capacity still increases with larger vertical forces (e.g. autoclaved aerated concrete). The factor *c* to consider the shear stress distribution also results in a smaller shear resistance according to DIN 1053-100 for wall dimensions *h/l* > 1.0. In the course of calculating the shear resistance using the given graphs, it must be pointed out, that two loading cases $n_{Ed,min}$ and $n_{Ed,max} \approx 1.4 \cdot n_{Ed,min}$ have to be considered.

SHEAR DESIGN ACCORDING TO SIA 266

In 2003 the Swiss Society of Engineers and Architects (SIA) introduced the new masonry code SIA 266. The basis of this standard are the investigations of *Ganz* and *Thuerlimann* [11, 12]. In 1985 they carried out tests on masonry panels with inclined joints under normal and shear stress and developed a new failure theory based on the theory of plasticity. For their failure criteria for unreinforced masonry without tensile strength they based their work on the following considerations and assumptions:

- Acceptance of the theory of plasticity with rigid-perfectly plastic material behaviour
- Punched units have an anisotropic strength distribution. Therefore, the unit can be divided in biaxial and uniaxial stressed parts.
- Unequal *Poissons*'s ratios of unit and mortar result in lateral tension stresses. This leads – strongly simplified – to a reduced masonry compressive strength, compared with the unit strength
- Only shear and tension failure are analyzed in the joints. The uniaxial compressive strength of mortar does not become critical because of the triaxial state of stress.
- Shear failure through the head joints and adjoining units are excluded

With four independent material parameters f_x, f_y, f_{vo} and (φ = arctan μ) the following failure criteria (called *regimes*) were derived (compression = negative):

(1) Tensile failure in the unit

$$\tau \leq \sqrt{\sigma_x \cdot \sigma_y} \qquad \text{Equation 16}$$

(2) Compressive failure in the unit

$$\tau \leq \sqrt{(\sigma_x + f_x) \cdot (\sigma_y + f_y)} \qquad \text{Equation 17}$$

(3) Shear failure in the unit

$$t \leq \sqrt{-s_y \cdot (s_y + f_y)} \qquad \text{Equation 18}$$

(4) Tensile failure in the bed joints

$$t \leq \sqrt{s_x \cdot \left(s_x + 2 \cdot f_{vo} \cdot \tan\left(\frac{p}{4} + \frac{\arctan m}{2} \right) \right)} \qquad \text{Equation 19}$$

(5) Sliding along the bed joints

$$t \leq f_{vo} - s_x \cdot m \qquad \text{Equation 20}$$

Combining the Equations 16 to 20, we get the complete failure surface for unreinforced masonry without tensile strength. All stress points inside this envelope are not in danger of failure (cf. figure 3, left). Figure 3 (right) shows the compressive stress subjected to inclined bed joints. The smaller compressive strength f_y can not be exceeded under uniaxial stress, except for $\alpha = 0$. Without taking into consideration sliding along the bed joints a quasi-isotropic behaviour of strength arises. For joint inclinations larger than the limit angle ($\alpha > j$) the uniaxial compressive strength resistance decreases significantly as a result of sliding in the bed joints and remains on the minimum level as long as $\alpha < \pi/2$.

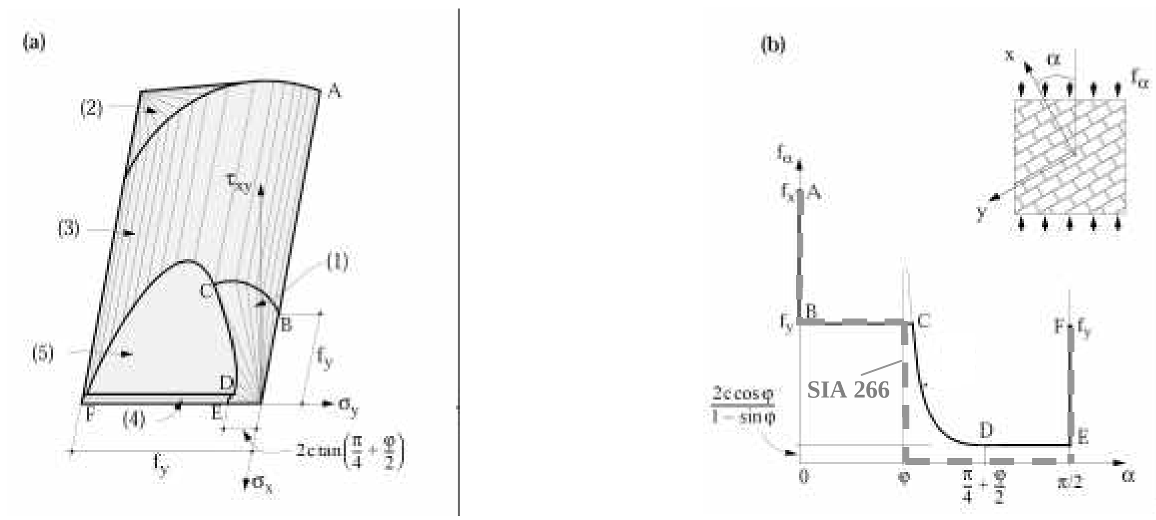

Figure 3 – Axonometric view of the failure criteria of *Ganz/Thuerlimann* for unreinforced masonry structures without tensile strength (left) and compressive stress subjected to inclined bed joints for $f_{vo} \neq 0$ (right)

In 1999 *Mojsilovic* and *Marti* [13, 14] extended the failure criteria of *Ganz* by one additional regime. Similar to the failure criterion of *Ganz* for bed joints, a failure condition using the cohesion $f_{vo,b}$ and the friction angle j_b of the unit material was used for the head joints.

(6) Failure along the alignment of the head joints

$$t \leq \frac{f_{vo,b}}{2} - s_x \cdot \tan j_b \qquad \text{Equation 21}$$

The enhancements of *Mojsilovic* have not been considered in SIA 266 up to now. The design concept for shear walls in SIA is only based on the theory of *Ganz/Thuerlimann* with the failure criteria (1) to (5). Simlilarly to DIN 1053-100 and EC 6 the partial safety concept is used to design masonry structures. The values of the partial factors on the action side are equal to DIN 1053-100 and EC 6 depending on the type of the load, whereas the partial factor for materials γ_M is set to 2.0. The uniaxial compressive strength $f_{\alpha d}$, depends on the angle of inclination of the assumed inclined compressive stress field and is shown in figure 3 (right). For $\alpha \geq \varphi$ the compressive strength $f_{\alpha d}$ is set to 0, what means that cohesion is neglected.

SIA 266 gives design charts (provided for different ratios f_{yd} / f_{xd}) for shear walls, which are based on the lower-bound theorem of the theory of plasticity. To develop these design charts *Schwartz* [15] combined the inclined stress fields with vertical stress fields subjected to f_{xd} - $f_{\alpha d}$ (figure 4). The best lower bound for the ultimate shear resistance can be determined by varying α. It must be pointed out, that the design value for the friction coefficient μ_d is set to 0.6.

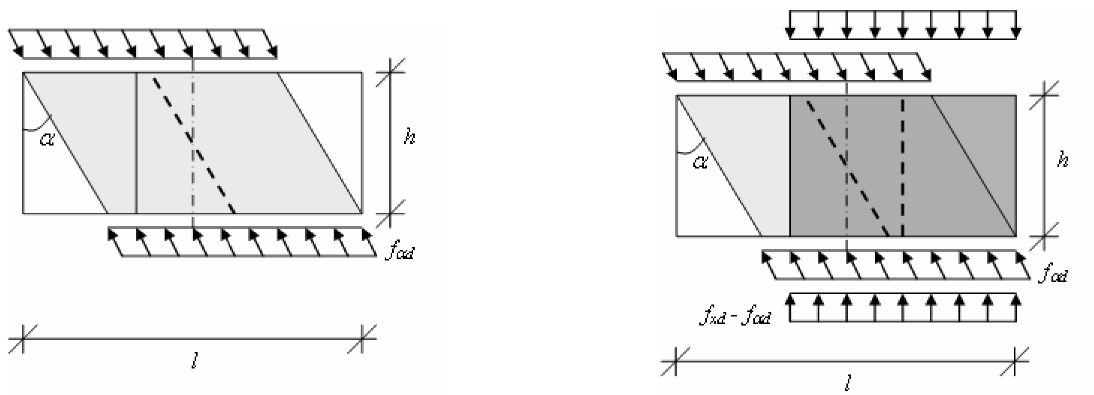

Figure 4 – Simple (left) and enhanced (right) models for allowed stress fields [4]

Figure 5 shows the shear capacity for different ratios f_{yd} / f_{xd}. It is obvious, that the shear resistance increases significantly with increased values for the masonry compressive strength parallel to the bed joints f_y.

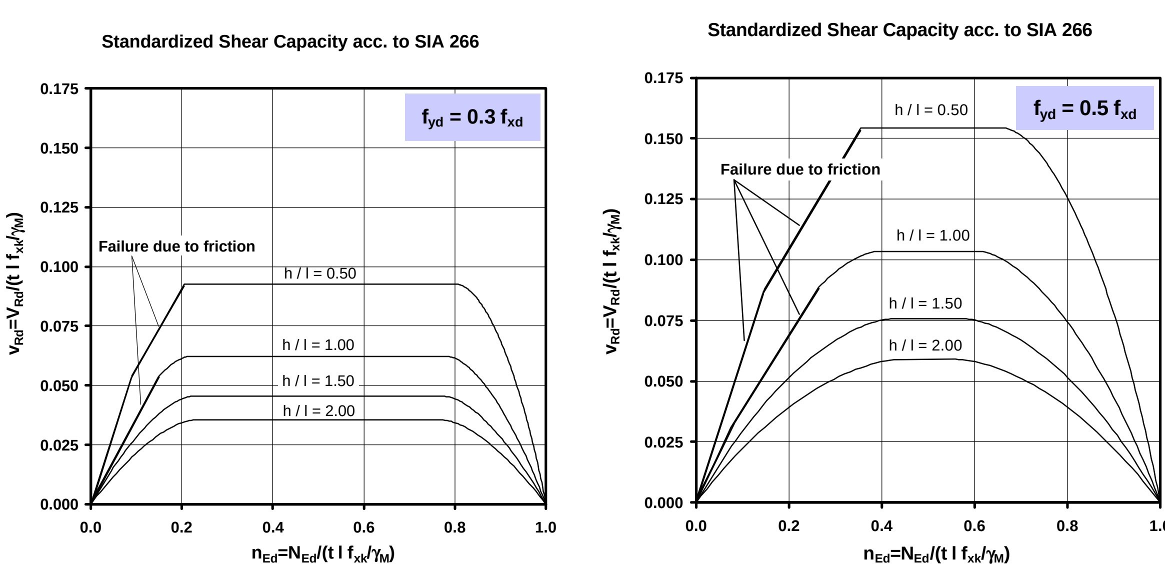

Figure 5 – Shear capacity acc. to SIA 266 for several *h/l*-ratios and f_{yd} / f_{xd} = 0.3 resp. 0.5

SHEAR DESIGN ACCORDING TO CSA S304.1-04

To compare the above quoted design codes with the Canadian Standard S304.1-04 [18] the criteria used in [18] will be stated shortly. The notations have been modified from the original and several assumptions will be made, because of the totally different type of construction and design of Canadian Masonry compared to the European one.

The factored shear resistance according to CSA S304.1-04 $v_{Rd} = V_{Rd} / \left(t \cdot l \cdot f_m^{'} / g_M \right)$ is taken in general as dependent on the vertical load $n_{Ed} = N_{Ed} / \left(t \cdot l \cdot f_m^{'} / g_M \right)$ as:

$$v_{Rd} = \frac{1}{g_M} \cdot \left(0.16 \cdot \left(2 - \frac{h}{l} \right) \cdot \frac{g_M}{\sqrt{f_m^{'}}} \cdot 0.8 + 0.25 \cdot 0.9 \cdot n_{Ed} \right) \cdot g \qquad \text{Equation 22a}$$

but not greater than

$$v_{Rd} = 0.4 \cdot \frac{1}{\sqrt{f_m^{'}}} \cdot 0.8 \cdot g \qquad \text{Equation 22b}$$

f'_m specified masonry compressive strength normal to the bed joint

g_M $= 1/f_m$

f_m = resistance factor for masonry according to CSA S304.1-04

g factor to account for partially grouted or ungrouted walls that are constructed of hollow or semi-solid units

g = 1.0 for fully grouted masonry, fully solid concrete block masonry, or solid brick masonry; otherwise

g = A_e/A_g, but not greater than 0.5

A_e effective cross-sectional area

A_g gross cross-sectional area

COMPARISION

Figure 6 show the calculated shear capacity for the above quoted design codes. In the left figure all carrying capacities are related to the compressive strength of the masonry using the respective codes. This kind of presentation allows the identification of the carrying capacity according to the mechanical model used in the particular code, but still includes the different safety concepts of the various countries. For a better comparison of the allowable shear capacity of unreinforced masonry in the different countries the results in the right figure are normalized to the design value $f_{xd,DIN}$ according to DIN 1053-100.

Neglecting the different safety factors on the material side (figure 6, left), the European standards show a good match in general. The calculated curves of DIN 1053-100 and Eurocode 6 are running in the range between $0.3 \cdot f_{xd} \leq f_{yd} \leq= 0.5 \cdot f_{xd}$ used in SIA 266. For lower values of n_{Ed} the influence of the greater friction coefficient according to SIA 266 yields higher shear capacities. However, due to the different basic theoretical models the results are significantly different depending on the respective type of unit (A or B). Using quite high values of the tensile strength of clay units (curve 1), the shear resistance according to DIN 1053-100 and Eurocode 6 are higher than in SIA 266, whereas considering the lower compressive strength parallel to the bed joints f_{yd} according to SIA 266 yields lower shear resistances. For homogenous materials (e.g.

autoclaved aerated concrete) the effects are opposite. Lower tensile strength results in less shear resistance taking into account DIN 1053-100 or EC 6, whereas according to SIA 266 greater values for the horizontal compressive strength can be taken resulting in higher shear resistances.

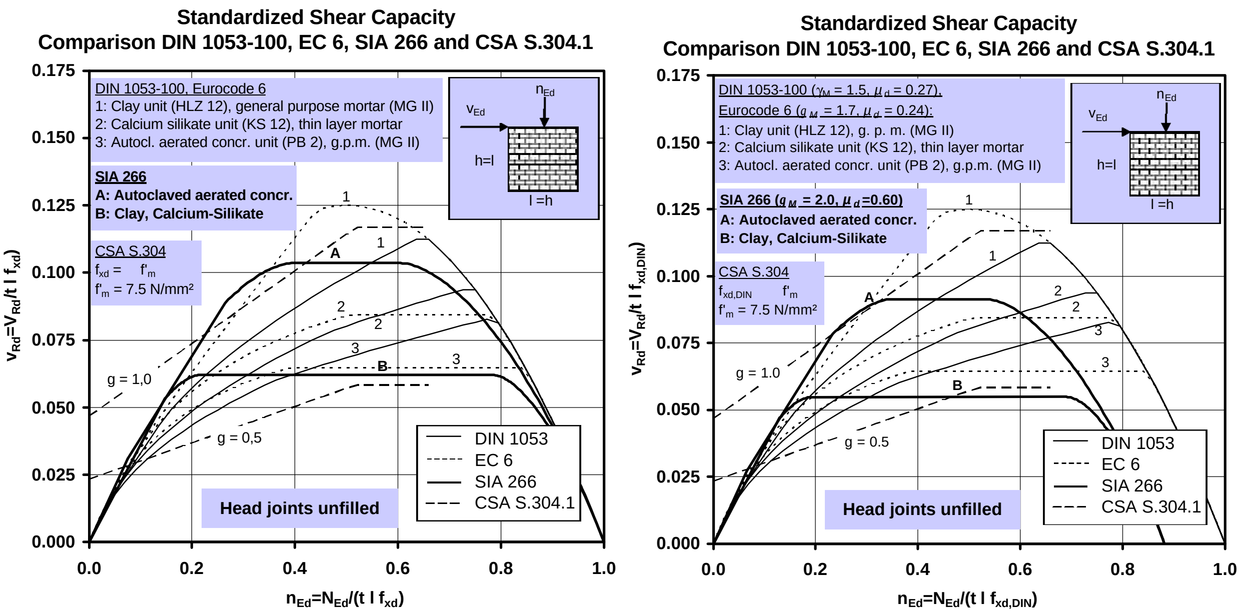

Figure 6 – Standardized Shear Capacity - Comparison between DIN 1053-100, EC 6, SIA 266 and CSA S304.1-04, for *h/l* = 1 (left: related to f_{xd}, right: related to $f_{xd,DIN}$)

Due to the different safety factors on the material side ($\gamma_{M,SIA} \approx 1.13 \cdot \gamma_{M,DIN}$), the shear resistance according to SIA 266 is significantly lower than according to DIN 1053-100 and Eurocode 6 (figure 6, right). Only for greater values of f_{yd} the graphs of the other two standards can be reached, but only up to a medium level of the vertical force. For greater values of n_{Ed} the shear resistance according to SIA 266 is significantly lower than the other two European Standards.

A calculation of the shear capacity using the Canadian code shows large differences in dependency of the factor g. For solid blocks quite high resistances are reached compared with the European Standards, whereas for hollow units only lower capacities are allowed. The very low friction coefficient used in CSA S304.1 is significant.

CONCLUSIONS

The basis of the two design standards (DIN 1053-100 and SIA 266) are significantly different. The method of *Mann/Mueller* leads to higher shear capacities for units with high tensile strengths and higher vertical loads. On the other hand for homogenous materials with lower compression forces the method of *Ganz/Thuerlimann* yields higher shear resistance.

More sophisticated calculation models for the design of unreinforced masonry panels are necessary to realistically take into account the different material behaviour. The utilization of unit-specified features would make the construction technically and economically more efficient.

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REFERENCES

1. prEN 1996-1-1 04-04. Eurocode 6: Design of Masonry Structures - Part 1-1: Common rules for reinforced and unreinforced masonry structures. Final draft. German Version. Unreleased. CEN/TC 250, SC6.
2. DIN 1053-1 11-96. Masonry – Part 1: Design and construction
3. DIN 1053-100 08-04. Masonry – Part 100: Design on the basis of semi-probabilistic safety concept. Unreleased. NABau 06.30.00.
4. SIA 266:2003. Masonry
5. Mann, W., Mueller, H. Bruchkriterien für querkraftbeanspruchtes Mauerwerk und ihre Anwendung auf gemauerte Windscheiben. Die Bautechnik, Heft 12. Berlin. 1973.
6. Mueller, H. Untersuchungen zum Tragverhalten von querkraftbeanspruchtem Mauerwerk. Dissertation TH-Darmstadt. 1974.
7. Mann, W., Mueller, H.: Schubtragfähigkeit von Mauerwerk. Mauerwerk-Kalender 3. Berlin. 1978.
8. Graubner, C.-A., Simon, E. Zur Schubtragfähigkeit von Mauerwerk aus großformatigen Steinen. Mauerwerk-Kalender 26. Berlin. 2001.
9. Simon, E. Schubtragverhalten von Mauerwerk aus großformatigen Steinen. Dissertation TU Darmstadt. 2002.
10. Simon, E. Schubtragverhalten von Mauerwerk aus großformatigen Steinen. Das Mauerwerk, Heft 1. Berlin. 2003.
11. Ganz, H. R. Mauerwerksscheiben unter Normalkraft und Schub. IBK Bericht 148. ETH Zürich. 1985.
12. Ganz, H., Thuerlimann, B. Versuche an Mauerwerkscheiben unter Normalkraft und Querkraft. Bericht Nr. 7502-4. ETH Zürich. 1984.
13. Mojsilovic, N. Zum Tragverhalten von kombiniert beanspruchtem Mauerwerk. Dissertation ETH Zürich. 1995.
14. Mojsilovic, N., Marti, P. Strength of Masonry Subjected to Combined Actions. ACI structural journal, V. 94, No. 6, Nov.-Dec. 1997.
15. Zimmerli, B., Schwartz, J., Schwelger, G. Mauerwerk: Bemessung und Konstruktion. Basel 1999.
16. Graubner, C.-A., Kranzler, T., Schubert, P., Simon, E. Schubfestigkeit von Mauerwerksscheiben. Mauerwerk-Kalender 30. Berlin. 2005.
17. Gaubner, C.-A., Kranzler, T. Shear Resistance of unreinforced masonry walls - A comparison between DIN 1053-1 and DIN 1053-100. Darmstadt Concrete 2004. Darmstadt. 2004 (www.darmstadt-concrete.de).
18. CSA S304.1-04. Design of Masonry Structures. Canadian Standards Association. 2004.