LATERAL STABILITY OF URM BUILDINGS ACCORDING TO DIN EN 1996/NA

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ABSTRACT
This paper deals with the carrying capacity of shear walls subjected to in-plane loads according to the design regulations of European Standard DIN EN 1996/NA. A new approach to determine the load effects more realistically is described. Furthermore, by using a dimensionless notation a full set of design equations is presented, which allows for the calculatation and comparison of the in-plane resistance of shear walls in an easy an understandable manner. In conclusion, the distinctive failure modes are derived.

KEYWORDS: lateral stability, shear resistance, bending, friction, sliding, diagonal tension, diagonal compression, failure modes

INTRODUCTION
In every building, the horizontal loads on the structure have to be transferred into the soil by bracing elements, which also carry vertical loads. Using unreinforced masonry for such elements, compression is always needed to fulfil static equilibrium of the cross section due to the bending moment over the strong axis of the wall which is caused by the horizontal load. This is the reason why it is not sufficient to check the shear-carrying capacity under maximum horizontal loads like wind and earthquake (e.g. sliding shear failure), but in addition, an assessment of the bending capacity under maximum and minimum vertical loads is necessary (e.g. compression failure, overturning). Also, one has to keep in mind that according to the semi-probabilistic safety format lateral loads (e.g. wind, earth pressure) and the vertical actions (dead and live load) are independent basic variables, which most likely do not occur with their maximum design values at the same time and same design situation. So called combination values ψ_0 take into account the probability of occurrence in the ultimate limit state, whereas different partial safety factors have to be imposed on load effects and cross-sectional capacity to ensure adequate reliability in ultimate limit state.

Due to a strong increase in the design loads for wind and earthquakes in the European standards, it is evident that the structural design methods for load-bearing masonry, particularly with regard to the in-plane lateral-load capacity of URM shear walls, will not be satisfying in the future. Furthermore, the implementation of the semi-probabilistic safety concept using partial safety factors leads to more unfavourable action effects for the design of unreinforced masonry in terms of shear failure. On the other hand, the calculated in-plane shear capacity of masonry structures on basis of out-dated standards does not model the lateral stability realistically. This especially applies to the structural model, particularly the estimated degree of restraint at the top and the bottom of the panel (see [1] to [6]). Therefore, it is obvious that an integral approach for the calculation of the in-plane shear capacity of URM (shear) bearing walls is strongly required.

This paper deals with the calculation of load effects in URM shear walls and the design method concerning the shear capacity of such panels. A new proposal [7], to describe the load effects in shear walls more realistically is presented, which is implemented in the German National Annex to Eurocode 6 (DIN EN 1996/NA). Various parameters on the relevant failure mechanisms and the main influencing parameters have been studied. The results illustrate the load-carrying behaviour of horizontal bracing elements in multi-storey office buildings and residential houses.

REALISTIC LOAD EFFECTS IN SHEAR WALLS

In general, the load effects on horizontal bracing members are calculated using a cantilever model of height H with rigid restraint at the bottom (see fig 1a). The horizontal loads on the façade are modelled as concentrated single loads at the height of the slabs. No restraint of the shear wall in the slabs is taken into account. Therefore, this model leads to very conservative values of the load effects concerning the bending moment at the distinctive cross section for the design and results in inefficient solutions concerning necessary wall lengths. Having in mind that for unreinforced masonry an eccentricity of the normal force (e = M/N) causes a significant reduction of the cross section under compression, it is clear that not only the axial capacity is strongly influenced but also the shear capacity is reduced. This is the reason why a new approach is needed to calculate the bending moment distribution in shear walls with regard to a certain restraint of the bracing member in the adjacent slabs. A proposal is presented in annex K of DIN EN 1996/NA (see fig. 1b) which originally has been suggested by [7]. The main enhancement is the use of a shear slenderness factor λ_v which is based on the relation of the wall dimensions h and l of a single storey instead the relation H/l of the whole wall.

$$\lambda_v = \psi \cdot h / l \tag{1}$$

In the following, only one storey of the whole shear wall with height h and length l is taken into account (see fig. 2). At the top, a normal force N and a horizontal force V are acting. Knowing the load eccentricity e_o at the top from equation 2, the eccentricity at the bottom e_u can be calculated and the factor ψ may be determined by equation (3). N_{upp} and e_{upp} are the normal force and the corresponding eccentricity at the bottom of the storey above (see fig. 1b). N_{sl} and e_{sl} describe normal force and eccentricity due to the loads on the slab and N_w is the dead load of the masonry wall. Fig. 2 shows some examples for load eccentricities and corresponding values of the factor ψ. One may notice that assuming an almost constant wind load over the height of the building the results of the cantilever-model are obtained, if one uses ψ=H/(2·h).

$$e_o = \frac{N_{upp} \cdot e_{upp} - N_{sl} \cdot e_{sl}}{N_{upp} + N_{sl}} \qquad e_u = e_o + \frac{V \cdot h}{N_o + N_w} \tag{2}$$

$$\psi = 1 + \frac{e_o}{h_u} \cdot \frac{N_o + N_w}{V} \tag{3}$$

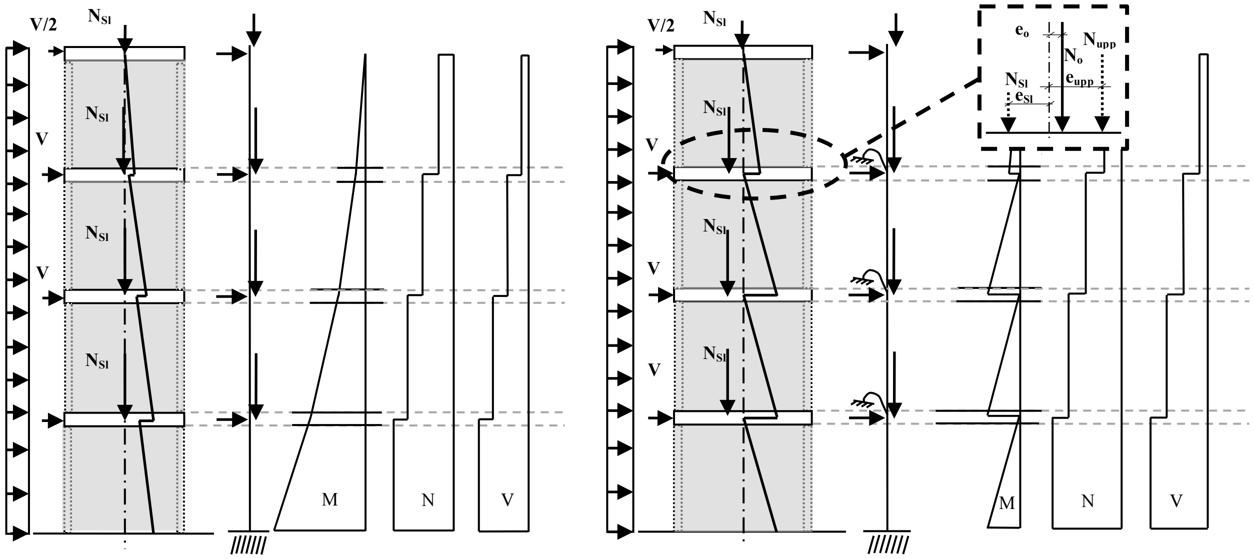

Figure 1: a) Cantilever Model; b) Model of Annex K DIN EN 1996-1-1/NA

When using equation (3) to calculate, ψ the algebraic sign of e_o has to be considered (see fig. 2). It is evident that by using this this proposal the designer can influence the shear slenderness λ_v by choosing an adequate value for the eccentricity e_{sl} of the vertical load N_{sl} coming from the slab.

If the slab is able to transfer the backturning moment $N_{sl} \cdot e_{sl,}$ a recentering of the vertical load in every storey is possible, which reduces the acting bending moment at the bottom of the shear panel significantly (see fig 1b).

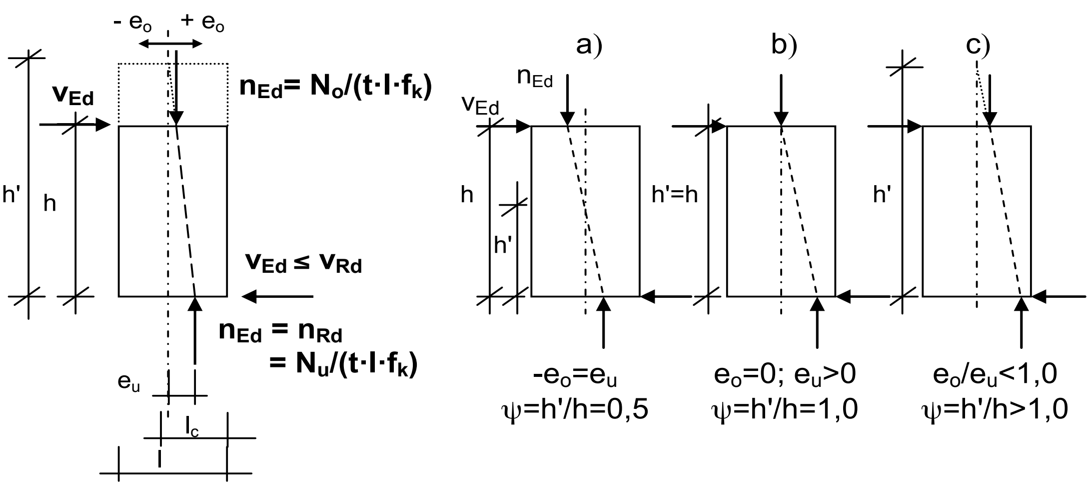

Figure 2: Definition of load eccentricities at the top and bottom of a wall and examples for the determination of the shear slenderness

The design of shear panels (see chapter 4) has to take into account various design situations and failure modes. The shear strength directly depends on the acting normal force and the length under compression l_c'. One has to keep in mind, that if minimum vertical load is relevant for the calculation of the shear capacity, the length under compression should be determined considering a linear stress distribution over the cross section. In each situation a different load combination may be relevant for the determination of the in-plane shear capacity. Three different load cases have to be checked under consideration of different partial safety and load combination factors:

a) Maximum horizontal force $V_{Ed,max}$ in combination with the minimum design value for the normal force $N_{Ed,min}$:

$$\text{LC 1:} \quad V_{Ed,\max} = \gamma_Q \cdot V_{Ek} \;;\quad N_{Ed,\min} = \gamma_{G,\inf} \cdot N_{Gk} = 1,0 \cdot N_{Gk} \tag{4}$$

b) Maximum horizontal force max V_{Ed} in combination with the associated design value for normal force:

$$\text{LC 2:} \quad V_{Ed,\max} = \gamma_Q \cdot V_{Ek} \;;\quad N_{Ed} = \gamma_{G,\sup} \cdot N_{Gk} + \gamma_Q \cdot \psi_0 \cdot N_{Qk} \tag{5}$$

c) Maximum normal force max $N_{Ed,max}$ with the associated design value for the horizontal force:

$$\text{LC 3:} \quad N_{Ed,\max} = \gamma_{G,\sup} \cdot N_{Gk} + \gamma_Q \cdot N_{Qk} \;;\quad V_{Ed} = \gamma_Q \cdot \psi_0 \cdot V_{Ek} \tag{6}$$

The values $N_{Ed,min}$ (LC 1) and $N_{Ed,max}$ (LC 2 and LC3) always differ by a value, which only depends on the ratio of live load to dead load q/g, the safety factors γ_G and γ_Q and the load combination factor ψ_0, which again is different for vertical live load and wind.

$$N_{Ed,\max} = \delta \cdot N_{Ed,\min} = \left(\gamma_Q - \frac{\gamma_Q - \gamma_G}{1 + q/g} \right) \cdot \left(1 + \frac{q}{g} \cdot \psi_0 \right) \cdot N_{Gk} \tag{7}$$

Taking into account that in common masonry buildings the relation between live load and dead load q/g ≈ 1/2 is realistic and using partial safety factors γ_G=1.35 and γ_Q=1.5 from EC 6 leads to:

LC 2: $N_{Ed} = 1.9 \cdot N_{Gk}$ (with $\psi_0 = 0.7$ $\delta = 1.9$)

LC 3: $N_{Ed} = 2.1 \cdot N_{Gk}$ (with $\psi_0 = 1.0$ $\delta = 2.1$)

This simplification allows an easy comparison of the shear capacity under the various load combinations (see chapters 4 and 5).

SHEAR STRENGTH OF URM SHEAR WALLS

To determine the in-plane shear strength of masonry the model of Mann/Müller [1] is most widely used. This approach takes into account that due to rotation of a single unit the in-plane shear stresses increase compared to an equal shear stress distribution over the length of the unit. EC 6 takes this effect into account by reducing the design values of the material properties (friction coefficient μ=0.6 and initial shear strength f_{vk}) by a factor $1/(1+\mu)$. This leads to a characteristic value of the shear strength f_{vk1}, necessary for the calculation of the horizontal carrying capacity of a shear wall in case of sliding shear of

$$f_{vk1} = \frac{f_{vk}}{1+\mu} + \frac{\mu}{1+\mu} \cdot \sigma_d = f_{vk0} + 0,4 \cdot \sigma_d \tag{8}$$

with

$$\sigma_d = \frac{N_{Ed}}{l_c \cdot t} = \frac{N_{Ed}}{1,5 \cdot (1 - 2 \cdot \lambda_v \cdot V_{Ed} / N_{Ed}) \cdot l \cdot t} \geq \frac{N_{Ed}}{l \cdot t} \tag{9}$$

In equation (9) the factor 1.5 in the denominator represents a linear distribution of normal stresses. This assumption is valid because always the minimum value of the normal force is decisive for the design in case of sliding shear failure.

Another possible failure mode according to the model of Mann-Müller is collapse due to exceedance of the main tensile strength in middle wall-height. In this case, a characteristic shear strength f_{vk2} has to be used to calculate the shear capacity.

$$f_{vk2} = 0,45 \cdot f_{bt,cal} \cdot \sqrt{1 + \sigma_d / f_{bt,cal}} \tag{10}$$

The material parameter $f_{bt,cal}$ represents the tensile strength of the units and depends on the compressive strength of the units and the percentage of vertical holes. For the calculation of vertical stresses σ_d equation (9) is valid again.

FAILURE MODES AND CARRYING CAPACITY OF URM SHEAR WALLS

Concerning the failure of URM shear walls different failure modes (as shown in fig. 3) have to be analysed which result in different load-carrying capacities. To simplify the handling of the equations and to make them comparable, a dimensionless notation [5] is used with the following abbreviations:

$$n_{Ed} = \frac{N_{Ed}}{t \cdot l \cdot f_k} \qquad n_{Gk} = \frac{N_{Gk}}{t \cdot l \cdot f_k} \qquad v_{Ed} = \frac{V_{Ed}}{t \cdot l \cdot f_k} \qquad v_{Ek} = \frac{V_{Ek}}{t \cdot l \cdot f_k}$$

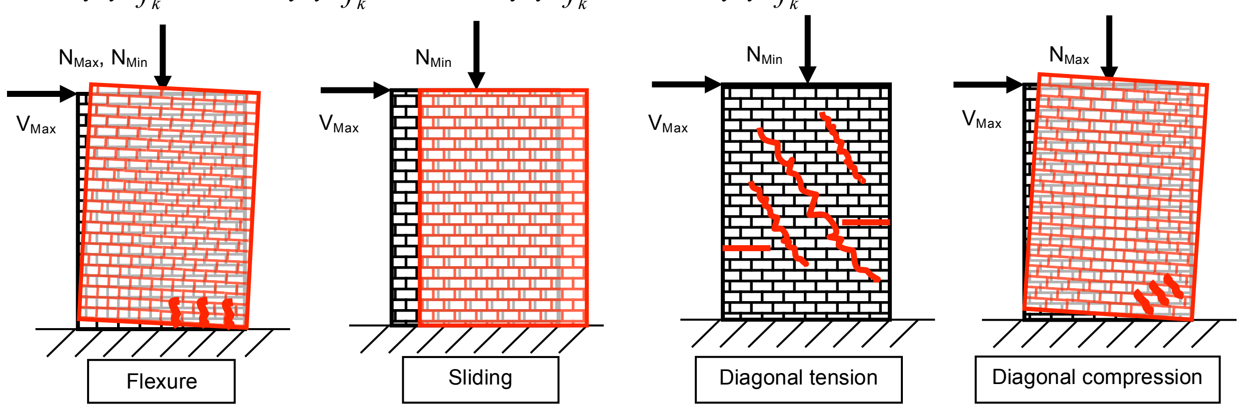

Figure 3: Failure modes of shear walls under in-plane shear loading

Flexural failure

A shear panel may fail due to flexure induced by a horizontal which can be defined as a shear capacity of the cross-section related to flexure. DIN EN 1996/NA always assumes rigid-plastic material behaviour when calculating the load-carrying capacity. A reduction factor $\varsigma = 0.85$ for long-term loads has to be taken into account under maximum vertical loads (LC 2 and LC 3), whereas under minimum load $\varsigma = 1.0$ is sufficient.

$$v_{Rd,B} = \frac{1}{2 \cdot \lambda_v} \cdot \left(n_{Ed} - \frac{\gamma_M}{\varsigma} \cdot n_{Ed}^2 \right) \qquad \max v_{Ek,B} \le v_{Rd,B} \cdot \frac{1}{\gamma_Q \cdot \psi_0} \tag{11}$$

Taking LC 1 (minimum normal force) from equation (4) leads to:

$$v_{Rd,B1} = \frac{1}{2 \cdot \lambda_v} \cdot \left(n_{Gk} - 1{,}5 \cdot n_{Gk}^2 \right) \qquad \max v_{Ek,B1} \le v_{Rd,B1} \cdot \frac{1}{1{,}5} \tag{12}$$

Load combination 2 from equation (5) under consideration of equation (7) leads to:

$$v_{Rd,B2} = \frac{1}{2 \cdot \lambda_v} \cdot \left(1{,}9 \cdot n_{Gk} - \frac{1{,}5}{0{,}85} \cdot 1{,}9^2 \cdot n_{Gk}^2 \right) \qquad \max v_{Ek,B2} \le v_{Rd,B2} \cdot \frac{1}{1{,}5} \tag{13}$$

Load combination 3 from equation (6) in combination with equation (7) and regarding a combination factor $\psi_0 = 0.6$ for horizontal load results in:

$$v_{Rd,B3} = \frac{1}{2 \cdot \lambda_v} \cdot \left(2{,}1 \cdot n_{Gk} - \frac{1{,}5}{0{,}85} \cdot 2{,}1^2 \cdot n_{Gk}^2 \right) \qquad \max v_{Ek,B3} \le v_{Rd,B3} \cdot \frac{1}{0{,}9} \tag{14}$$

Figure 4a demonstrates the shear carrying capacity v_{Rd} in the case of flexural failure, but one cannot determine directly which load combination (LC 2 or LC 3) will be relevant, because in LC 3 the load combination coefficient $\psi_0 = 0.6$ on horizontal load is not included in v_{RD}. Figure 4b shows the advantages of using equation (7) to identify the relevant load combination (taking n_{Gk} instead of n_{Ed}) and using v_{Ek} instead of v_{Rd}. It is evident that LC 1 is relevant if $n_{Gk} < 0.185$ whereas LC 3 governs for $0.242 < n_{GK} < 0.27$. In cases $0.185 \le n_{Gk} \le 0.242$, LC 2 gives the lowest load-carrying capacity v_{Ek}. If one assumes $\varsigma = 1.0$ in LC 2, because short- term wind is the dominant (leading) action in this load combination, equation (13) never becomes decisive and equation (14) is relevant in case $n_{Gk} \ge 0.218$. One may notice that equation (11) (LC 1) also covers a possible tip over of the entire wall with sufficient reliability. Because of the ratio of live load to dead load q/g = 0.5, the safety factors $\gamma_G = 1.35$ and $\gamma_Q = 1.5$ and a maximum value of $n_{Gk} = 0.27$ is acceptable in LC 3.

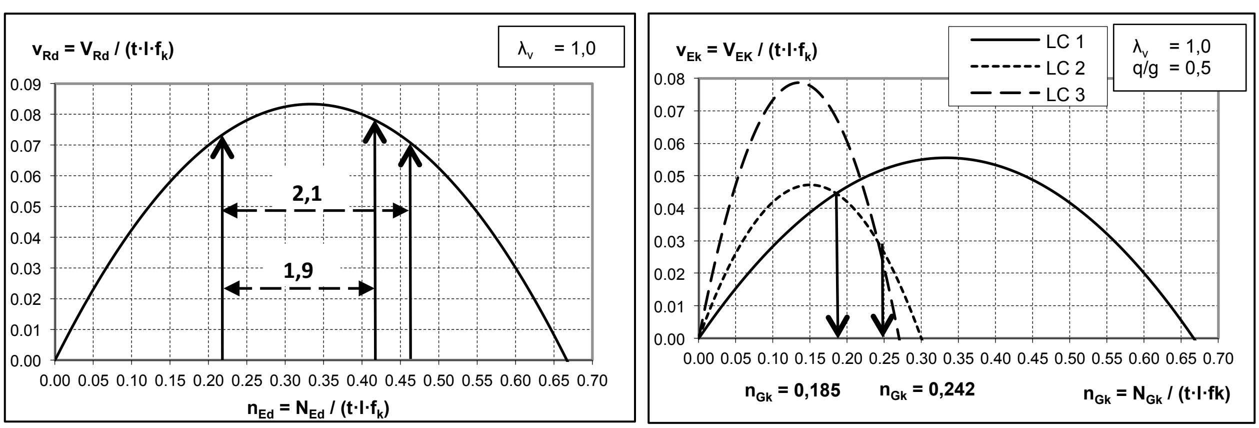

Figure 4: a) Design value of shear carrying capacity in case of bending failure
b) Allowable characteristic value of horizontal force under LC 1 to 3

Shear failure

Calculating the shear resistance in the case of sliding and diagonal tension failure the minimal normal force ($n_{min} = n_{Gk}$) has to be always considered. Due to this reason DIN EN 1996/NA allows for the determination of the length under compression taking into account a linear stress distribution. This assumption leads to a length under compression of l_c:

$$l_c = l_{c,lin} = 1,5 \cdot (1 - 2 \cdot \lambda_v \cdot \frac{v_{Ed}}{n_{Gk}}) \cdot l \le l \tag{15}$$

In general, the load-carrying capacity of shear walls can be calculated from:

$$v_{Rd,Shear} = \frac{1}{c \cdot \gamma_M} \cdot \frac{f_{vk}}{f_k} \cdot \frac{l_c}{l} \qquad \max v_{Ek,S} \le v_{Rd,shear} \cdot \frac{1}{\gamma_Q} \tag{16}$$

In equation (16), the factor c takes into account that shear stresses are not distributed constant over the length of a cross section (average) according to theory of linear elasticity, but have a maximum value of 1.5 times the average. Therefore, Mann/Müller [1] propose to use c = 1.5, if $h \ge 2 \cdot l$ and to take c = 1.0 for h = l.

Calculating the shear resistance, one has to differentiate between a fully compressed cross-section ($l_c = l$) and a cracked cross section ($l_c < l$). Using equation (15) and equation (16) results in two shear capacities from which the minimum has to be taken.

a) Sliding shear

The horizontal carrying capacity of shear walls in the case of sliding shear has to be determined from equation (16) taking $f_{vk} = f_{vk1}$ from equation (8):

Cracked cross-section ($l_c < l$):

$$v_{Rd,S1} = \frac{1,5 \cdot \frac{f_{vk0}}{f_k} + 0,4 \cdot n_{Gk}}{c \cdot \gamma_M + 3 \cdot \lambda_v \cdot \frac{f_{vk0}}{f_k} \cdot \frac{1}{n_{Gk}}} \qquad \max v_{Ek,S1} \le v_{Rd,S1} \cdot \frac{1}{\gamma_Q} \tag{17}$$

Compressed cross-section ($l_c = l$):

$$v_{Rd,S2} = \frac{1}{c \cdot \gamma_M} \cdot \left(\frac{f_{vk0}}{f_k} + 0,4 \cdot n_{Gk} \right) \qquad \max v_{Ek,S2} \le v_{Rd,S2} \cdot \frac{1}{\gamma_Q} \tag{18}$$

Equation (18) for fully compressed cross-sections is relevant, when $\lambda_v < 5/8$ and $n_{Gk} \ge n_{Gk}^*$:

$$n_{Gk}^* = \frac{6 \cdot \lambda_v \cdot f_{vko}/f_k}{\gamma_M - 6 \cdot \lambda_v \cdot \mu} = \frac{6 \cdot \lambda_v \cdot f_{vko}/f_k}{1,5 - 2,4 \cdot \lambda_v} \tag{19}$$

b) Diagonal tension

The shear resistance in the case of diagonal tension can be determined from equations (15) and (16). The only difference is that the shear strength f_{vk2} has to be taken from equation (10). Furthermore, one can take into account that the maximum tensile stresses always occur in the middle of the storey height instead of the bottom of the wall [4]. Therefore $\lambda_v = \overline{0,5 \cdot} \lambda_v$ instead of λ_v can be used in the design equations and the shear capacity can be calculated from equations (20) and (22) using the variables A, B and C:

$$A = 0,45 \cdot \frac{f_{bt,cal} / f_k}{\gamma_M \cdot c} \qquad B = \frac{3 \cdot A}{n_{Gk}} \cdot \bar{\lambda}_v \qquad C = 3 + 2 \cdot \frac{n_{Gk}}{f_{bt,cal} / f_k} \tag{20}$$

Cracked cross section ($l_c < l$):

$$v_{Rd,T1} = \frac{1}{2} \cdot \left(3 + \frac{n_{Gk}}{f_{bt,cal} / f_k}\right) \cdot \frac{A \cdot B}{1 - B^2} \cdot \left[-1 + \sqrt{1 + \frac{1 - B^2}{B^2} \cdot \frac{3}{C}}\right] \qquad \max v_{Ek,T1} \le v_{Rd,T1} \cdot \frac{1}{\gamma_Q} \tag{21}$$

Compressed cross section ($l_c = l$):

$$v_{Rd,T2} = A \cdot \sqrt{1 + n_{Gk} \cdot \frac{f_k}{f_{bt,cal}}} \qquad \max v_{Ek,T2} \le A \cdot \sqrt{1 + n_{Gk} \cdot \frac{f_k}{f_{bt,cal}}} \cdot \frac{1}{\gamma_Q} \tag{22}$$

As for sliding shear the minimum value from equations (21) and (22) has to be taken also for diagonal tension.

Figure 5 shows a comparison of the shear capacities in the failure modes sliding shear and diagonal tension for various values of the shear slenderness.

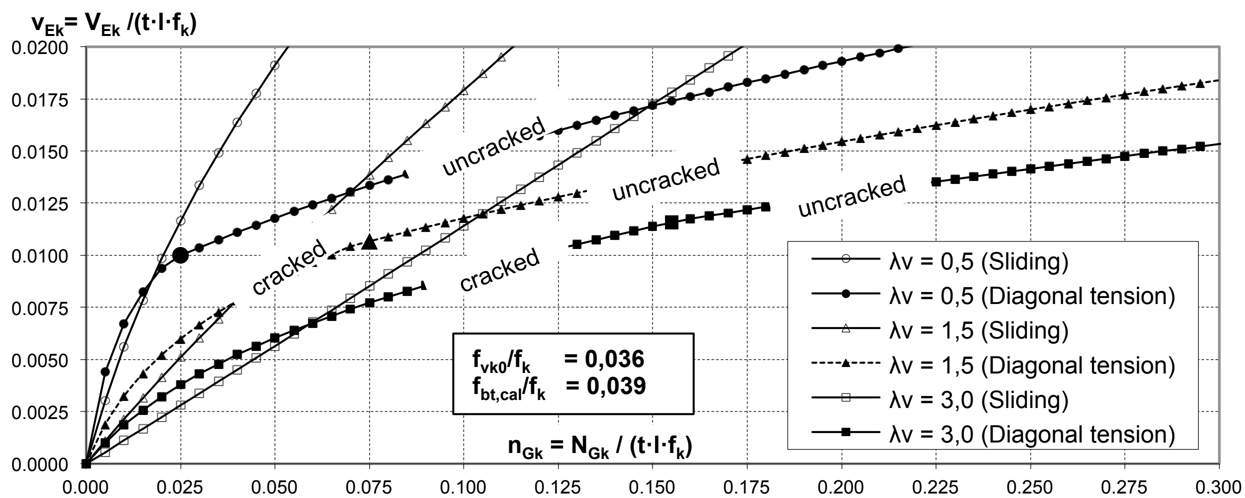

Figure 5: Sliding and diagonal tension for various shear slenderness

Diagonal Compression

Under maximum normal force $n_{Ed,max,}$ besides flexural failure, also the failure of a diagonal compression strut at the bottom of the wall is possible (see fig. 6). Compared to flexural failure three main differences exist:

1. The shear capacity in this failure mode depends on the overlap of the units.
2. Concerning the allowable compression strength, no long-term load factor $\varsigma = 0.85$ is necessary because wind loads always act for a short time.
3. According to the model of Mann/Müller [1], the shear distribution factor c has to be used.

Again, it is necessary to take into account the two possible load combination LC 2 and LC 3. Because the failure occurs under maximum normal force $n_{ED,max,}$ theory of plasticity is used to calculate the compressed length of the wall.

$$l_c = l_{c,pl} = (1 - 2 \cdot \lambda_v \cdot \frac{v_{Ed}}{n_{Gk}}) \cdot l \ \le \ l \tag{23}$$

This gives the design equations (24) for diagonal compression failure.

$$v_{Rd,C} = \frac{1 - \gamma_M \cdot n_{Ed}}{c \cdot \gamma_M + \frac{2 \cdot \lambda_v}{n_{Ed}} \cdot \frac{l_{ol}}{h_u}} \cdot \frac{l_{ol}}{h_u} \qquad \max v_{Ek,C} \le v_{Rd,C} \cdot \frac{1}{\gamma_Q \cdot \psi_0} \tag{24}$$

Load combination 2 from equation (5) under consideration of equation (7) gives:

$$v_{Rd,C2} = \frac{1 - 1{,}5 \cdot 1{,}9 \cdot n_{Gk}}{1{,}5 \cdot c + \frac{2 \cdot \lambda_v}{1{,}9 \cdot n_{Gk}} \cdot \frac{l_{ol}}{h_u}} \cdot \frac{l_{ol}}{h_u} \qquad \max v_{Ek,C2} \le v_{Rd,C2} \cdot \frac{1}{1{,}5} \tag{25}$$

Load combination 3 from equation (6) in combination with equation (7) and applying a combination factor $\psi_0 = 0.6$ for horizontal load results in:

$$v_{Rd,C3} = \frac{1 - 1{,}5 \cdot 2{,}1 \cdot n_{Gk}}{1{,}5 \cdot c + \frac{2 \cdot \lambda_v}{2{,}1 \cdot n_{Gk}} \cdot \frac{l_{ol}}{h_u}} \cdot \frac{l_{ol}}{h_u} \qquad \max v_{Ek,C3} \le v_{Rd,C3} \cdot \frac{1}{0{,}9} \tag{26}$$

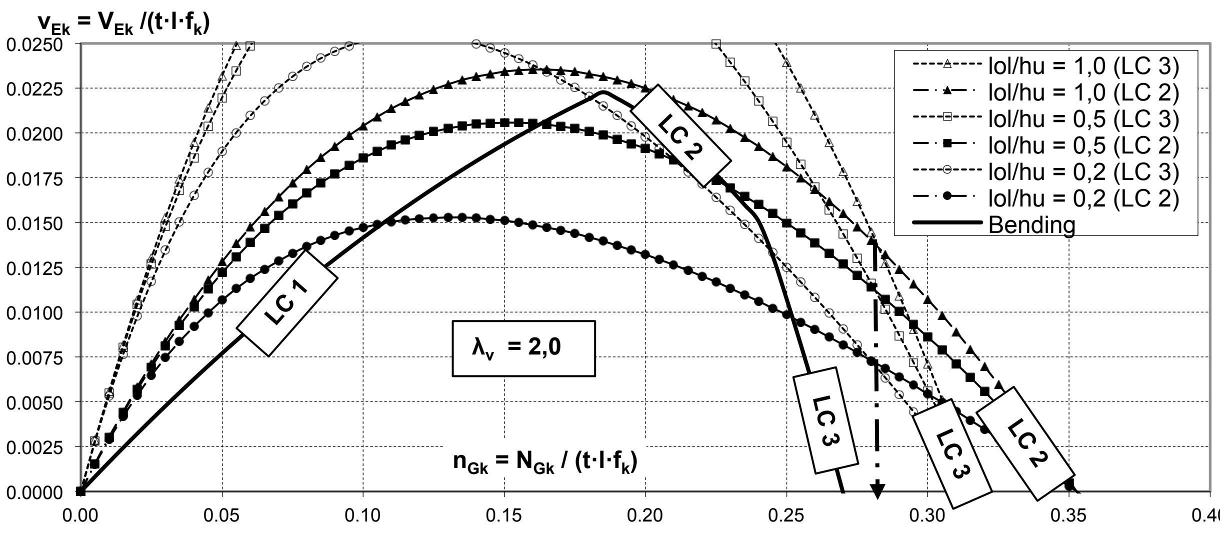

Figure 6: Diagonal compression failure compared to flexural failure

It can be seen from fig. 6, that – nearly independent from overlapping ratio l_{ol}/h_u - LC 2 is always decisive, if $n_{Gk} \le 0{,}28$ (l_{ol} = overlap). This is valid for all values of shear slenderness λ_v. Knowing from flexural failure that the normal force always has to be $n_{Gk} < 0.27$, one can recognize that LC 3 is never distinctive for diagonal compression failure. Finally, fig. 6 shows that for a ratio $l_{ol}/h_u > 0.5$ diagonal compression failure leads to larger shear capacities than flexural failure. Furthermore, diagonal compression failure is only relevant compared to flexural failure (LC 1 and LC 2), if the ratio $\lambda_v \cdot l_{ol}/h_u$ is smaller than 1.25.

COMPARISON OF DIFFERENT FAILURE MODES OF URM WALLS

With the equations (11) to (26), a full set of design equations is available to describe the load-carrying capacity of URM shear walls under in-plane lateral loading. As already mentioned, a dimensionless notation of the acting forces N and V and drawing the largest allowable characteristic value for the acting horizontal load (max v_{Ek}) over the minimum value of the dead load (n_{Gk}) is appropriate to compare the various failure modes in one graph. The figures (7) to (9) show the results of a parameter study for the different masonry materials like autoclaved aerated concrete (AAC), calcium-silicate (CS) and clay (C).

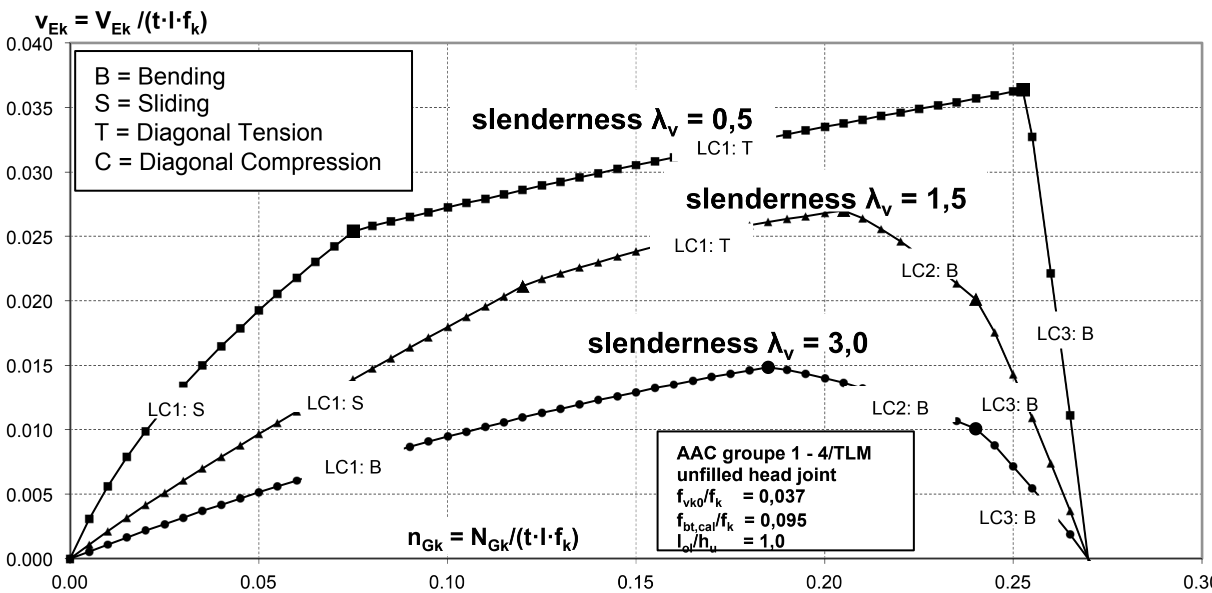

Figure 7: Shear capacity and failure modes for AAC masonry

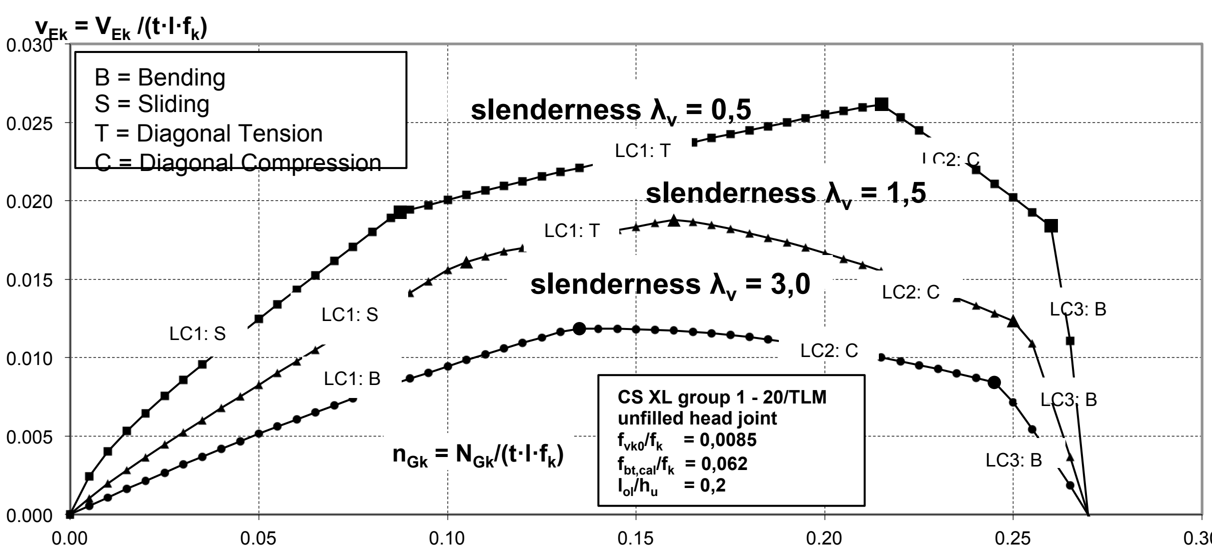

Figure 8: Shear capacity and failure modes for CS masonry

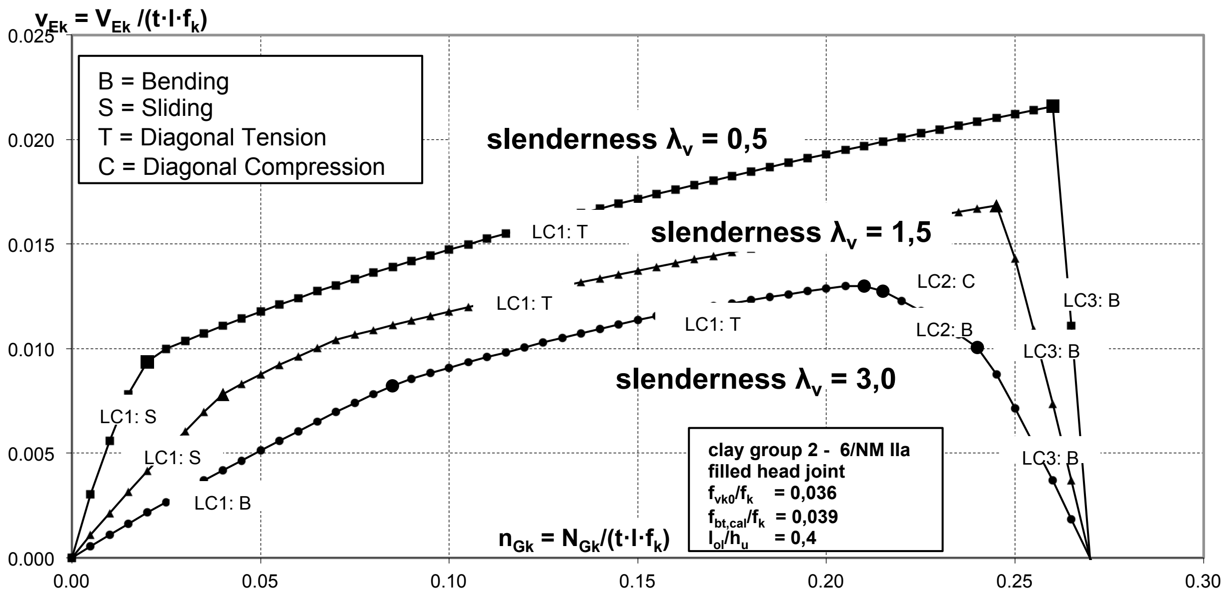

Figure 9: Shear capacity and failure modes for masonry made of clay units

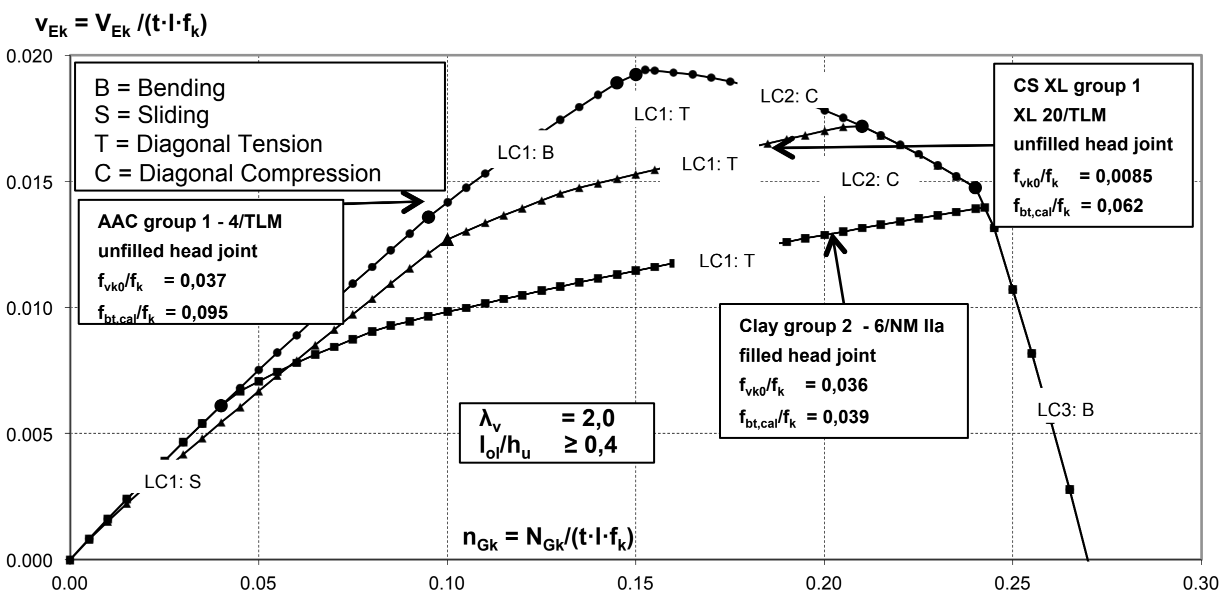

Figure 10: Comparison of Shear capacity and failure modes for different masonry types

The exemplary comparison of the shear capacity for different masonry types in fig. 10 with $\lambda_v = 1.0$ and dimensionless notation reveals that for low normal forces (sliding or flexural failure) the resistance is very similar. For higher dead load n_{Gk} it becomes obvious that the ratio tensile strength/compressive strength ($f_{bt,cal}/f_k$) of a material has a significant influence on the shear capacity. In case of flexural failure under very high vertical loads (LC 2 and LC 3) the load-carrying capacity is again independent. Diagonal compression failure only becomes relevant in the case of $l_{ol}/h_u \leq 0.5$ and $\lambda_v \leq 3$.

CONCLUSIONS

This paper deals with modelling the horizontal load-carrying capacity of shear walls made of URM. A new approach to include a possible restraint of the wall due to the adjacent slabs is presented. With this approach and using a dimensionless notation of acting forces, it is possible to calculate and compare the resistance of shear panels under consideration of various failure modes.

From the parameter study the following conclusions can be derived:

1) For shear panels with shear slenderness $\lambda_v > 3.5$ flexural failure is always relevant. Under low normal forces $n_{Gk} < 0.185$ the lowest design value of the vertical force is decisive.

2) Sliding shear only occurs under minimum vertical force n_{Gk}, if

   a) the shear slenderness is $\lambda_v < 2$ and the normal force is very low

   b) the tensile strength of units ($f_{bt,cal}$) is large compared to the initial shear strength (f_{vk0}).

3) Diagonal tension becomes decisive for a value of the shear slenderness of $\lambda_v < 3$ and low to average values of $n_{Gk.}$

4) Under large normal forces $n_{GK} > 0.185$, two load combinations (LC 2 and LC 3) have to be checked. It is possible to decide in advance whether normal force or horizontal force have to be taken as leading action.

5) Diagonal compression is only relevant for higher values of n_{Gk}, if an overlap ratio $l_{ol}/h_u < 0.5$ is given. For usual overlapping lengths $l_{ol}/h_u \geq 0.5$, flexure always governs under maximum normal force.

Further investigations should concentrate on the realistic determination of the relevant material parameters and the effect of filled and unfilled head joints.

REFERENCES

1. Mann, W.; Müller, H.: Schubtragfähigkeit von Mauerwerk. In: Mauerwerk-Kalender 3 (1978), S. 35-65. Berlin: Ernst & Sohn
2. Simon, E.: Schubtragverhalten von Mauerwerk aus großformatigen Steinen, Dissertation, Technische Universität Darmstadt, 2002
3. Graubner, C.-A.; Kranzler, T.; Schubert, P.; Simon, E.: Schubfestigkeit von Mauerwerkscheiben, in: Mauerwerk-Kalender 30 (2005), S. 7-88. Berlin: Ernst & Sohn
4. Graubner, C.-A. & Kranzler, T.: "Report on the integral shear design model for structural masonry members", Technical reports D4.3 and D4.4 of the collective research project ESECMaSE, Institut für Massivbau, Technische Universität Darmstadt, 2007
5. Graubner, C.-A. & Kranzler, T., "In-Plane Bearing Capacity of Unreinforced Masonry Panels", Proceedings of the 7th International Masonry Conference, London, 2006, paper 70
6. Brehm, E.: Reliability of Unreinforced Masonry Bracing Walls – Probabilistic Approach and Optimized Target Values, Dissertation, Technische Universität Darmstadt, 2011
7. Kranzler, T.: Tragfähigkeit überwiegend horizontal beanspruchter Aussteifungsscheiben aus unbewehrtem Mauerwerk, Dissertation, Technische Universität Darmstadt, 2008
8. Graubner, C.-A.; Brehm, E.: Bemessung von unbewehrtem Mauerwerk nach neuer Normengeneration, in: Mauerwerksbau aktuell 2010, S. E.66 – E.96. Berlin: Bauwerk Verlag