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EARTHQUAKE RESISTANCE OF A TERRACED HOUSE - VERIFICATION OF A NEW DESIGN APPROACH VIA COMPARISON WITH TEST RESULTS

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ABSTRACT
The paper deals with the verification of a new design approach for shear stressed masonry structures. Therefore the results of pseudo-dynamic test on large-scale unreinforced masonry terraced houses have been compared with the respective results of the new approach and the already existing approach of DIN 1053. Both, the large-scale test and the development of the new design approach were part of the European research project ESECMaSE (Enhanced Safety and Efficient Construction of Masonry Structures in Europe).
On basis of the comparison, the deficits of the traditional way of designing shear stressed masonry walls have been discussed. It could be shown that the results of the new approach matches very good with the test results, if one considers a realistic structural system. The commonly used cantilever as structural system for unreinforced masonry shear walls does not seem to be appropriate. This applies for the new design approach as well as for the already existing approach of DIN 1053.

KEYWORDS: clay masonry design, earthquake resistance, pseudo-dynamic testing, shear resistance, ESECMaSE

INTRODUCTION
The design of masonry structures subjected to horizontal loads causes a lot of problems in practice, because the commonly structural design methods for load bearing masonry, particularly with regard to the in-plane lateral-load capacity of URM shear bearing walls are not longer sufficient. Reasons therefore are the increase of the design loads for wind and earthquake in the European standards and the fact, that the calculated in-plane lateral-load capacity of masonry structures on basis of the current standards seems to underestimate the ones observed in practice.

Within the European research project ESECMaSE comprehensive investigations were carried out on the shear load bearing capacity and earthquake resistance of masonry structures. The results of the ESECMaSE project have already been presented at IBMaC in 2008 [1 to 8]. A large-scale pseudo-dynamic test on two terraced house halves with a typical Central European ground plan was scheduled as the final experiment in the project. Thereby one specimen was built with clay

unit masonry, the other one with calcium silicate masonry [9]. This paper focuses on the clay unit masonry structure. The main purpose of this large-scale test was the examination of the transferability of the findings and conclusions of the previous investigations [1 to 8]. In particular it served to verify the developed design model [4] on a structure subjected to loading under near-practice conditions. The results of this verification will be presented in the following.
To enable a comparison of the test results with the calculated results, the calculation will be done without the consideration of any safety factors and with the mean values of the material properties.

On basis of the comparison, the deficits of the traditional way of designing shear stressed masonry walls will be discussed.

LARGE-SCALE PSEUDO-DYNAMIC TEST

Figure 1 shows the ground plan and elevation of the clay unit masonry specimen. Both ground plan and elevation of the house halve equate to common terraced houses.

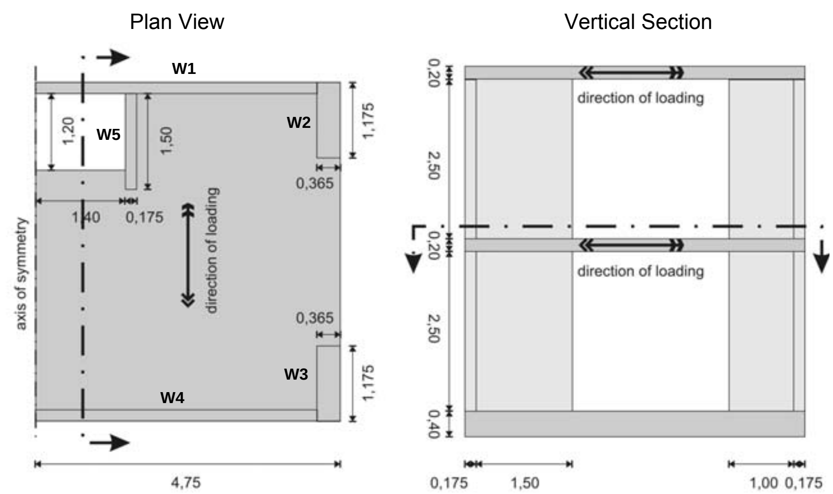


Figure 1: Ground plan and elevation of the terraced house tested in Ispra [9]

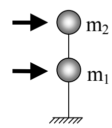
For the external walls W2 and W3 at the front side of the building, 36.5 cm thick lightweight vertically perforated precision units according to the German technical approval Z-17.1-490 were used. The interior wall (shear wall) W5 was constructed with 17.5 cm thick infill units according to the German technical approval Z-17.1-537, which were filled with unreinforced concrete (C 20/25). The long partition walls W1 and W4 were built of 17.5 cm thick optimized vertically perforated precision units according to the German technical approval Z-17.1-993, which were also a result of the project. Figure 4 of [9] shows these three different types of clay units. Table 1 gives the relevant material properties of the respective masonry walls for the use in the calculation as mean values. They were carried out mostly by tests with small specimens. For more detailed information see [9].

Table 1: Material properties of the clay masonry for the use in the calculation

Parameter	W1, W4	W2, W3	W5
Compressive strength of the masonry f [N/mm^2]	6.7	6.0	16.5
Tensile strength of the unit f_{bt} [N/mm^2]	0.83	0.50	0.80
Initial shear strength f_{v0} [N/mm^2]	0.24	0.30	0.35
Friction coefficient μ	0.68	0.60	0.60
Density γ [kN/m^3]	9	8	17

Additionally to the own weight of the construction, the terraced house halve was loaded with extension and life load. The resulting masses of the single storeys are given in table 2.

Table 2: Resulting storey masses of the terraced house halve

m_2 [t]	23.7	
m_1 [t]	23.9	

The pseudo-dynamic tests themselves were conducted according to the method developed at the ELSA [10]. The specimens were loaded uniaxially in the direction of the shear walls via the hydraulic pistons. Two degrees of freedom were taken into account in the pseudo-dynamic algorithm. These were the displacements at the height of the floors above the ground floor and above the upper storey respectively. The same synthetically generated earthquake, based on the same elastic response spectrum Type 1 according to Eurocode 8 (EN 1998-1 [11]) for the ground type B, such as was used for the shaking table tests at the NTU Athens [6], was taken as a basis for the algorithm. The strength, i.e. the maximum ground acceleration of the earthquake used, was increased step by step. For this purpose, with the same time course of the earthquake, the accelerations were scaled with factors. The step-wise increase of the action took place until the respective specimen showed a clear drop of horizontal resistance. Up to this point in time, each specimen had gone through several load stages (earthquakes of lower strength) without a failure of the building occurring.

Figure 2 shows the resulting horizontal force-displacement curve of the ground floor. One can see the strong influence of the load-direction on the maximum horizontal force due to redistribution effects. For the comparison with the two approaches only the minimum value of the load-direction will be taken into account. Furthermore does Figure 2 show that the specimens reacts strongly non-linear with large displacements in case of higher horizontal load-levels.

Out of Figure 2 one gets the total horizontal force in the basement of the terraced house. The values given in table 3 refer to a whole terraced house (test results multiplied by 2). It must be pointed out, that already an acceleration $a_{gR} = 0.12g$ corresponds to the most unfavourable combination of seismic zone 3 in regions with the ground conditions C-R according to the German standard for seismic design DIN 4149, which bases on Eurocode 8.

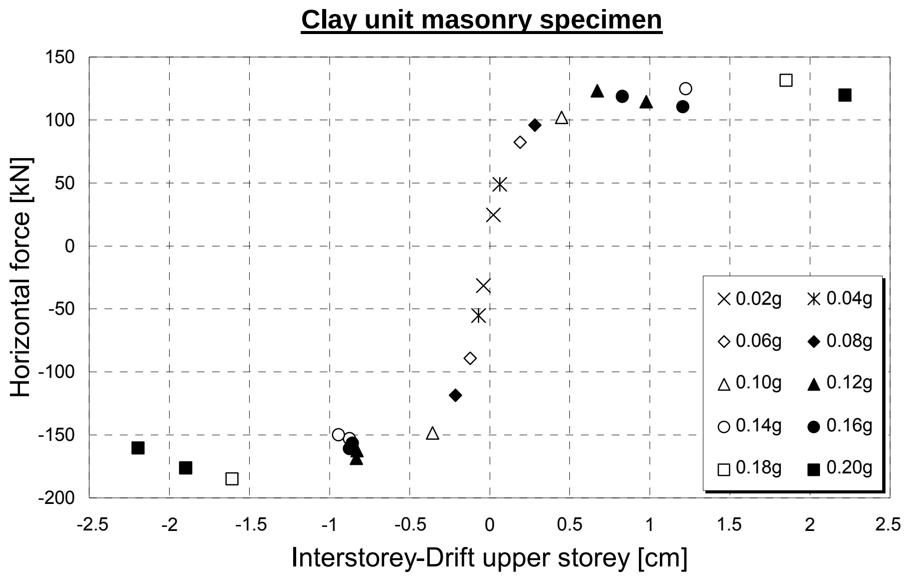


Figure 2: Horizontal force-displacement curve for the terraced house halve [12]

Table 3: Total horizontal force in the basement of the terraced house

a_{gR} [g]	Total horizontal force in the basement [kN]
0.04	96
0.08	190
0.12	250
0.14	255
0.16	250
0.18	260
0.20	250

CALCULATION OF THE TOTAL HORIZONTAL FORCE

In the following the total horizontal force of the terraced house will be calculated in dependence of two different design approaches.

Commonly for the design of a building, the most decisive single wall is taken into account and interlocking effects between the different walls are neglected. It will be assumed that the total horizontal force due to earthquake is distributed proportional to the mass and the height along the height of the building. The distribution of the total horizontal force on the single walls W2, W3 and W5 will be chosen according to their bending stiffness. Table 4 gives the parts of the horizontal force of the single walls as percentages of the total horizontal force. The respective vertical loads of the single walls determined with an FE-calculation are shown there as well.

Table 4: Horizontal- and vertical load parts of the single walls

	W2	W3	W5
Horizontal load	10.8%	10.8%	28.5%
Vertical load	6.6%	7.1%	14.7%

As already mentioned the calculation will be carried out by neglecting any safety factors on the action side and on the material side to enable an direct comparison with the test results. It has to keep in mind that the calculated results would be significant lower if one would consider the safety factors and the characteristic values of the material parameters.
The determination of action effects will be done under consideration of three different structural systems for the single walls regarding the distribution of the internal moments. The different structural systems are shown in figure 3 and differ as follows:

System A: flexible floors (cantilever)
System B: partially restraint of the walls in the floors, which leads to a centering of the vertical load under the respective floor (storey high cantilever)
System C: flexural rigid floors (rigid restraint of the walls in the floors)

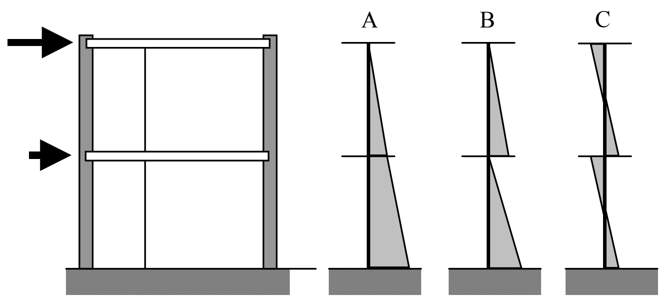


Figure 3: Different assumptions regarding the moment distribution in the single walls

According to DIN 1053 [13, 14] the design has to be done at the most decisive cross section of a wall. Commonly this is at the bottom of the wall between the support and the first course of the masonry. Considering only the sections remaining uncracked one has to check the resistance due to bending (flexural) failure and the shear resistance as well. Bending (flexural) failure has to be checked by assuming a stress block for the strain-stress-relationship. Proofing the shear resistance bases on the theoretical model of Mann/Mueller which is explained in [15] in detail.

Table 5 shows the results of the calculations. Thereby the total horizontal force was increased till the resistance due to bending or the shear resistance of the decisive wall (always wall W5) was reached. Considering system A (cantilever) the total horizontal force reaches 69 kN. If one considers the reduction of the internal moments in the walls due to an interaction with the floors (system B), the total horizontal force reaches 104 kN (+50%). As expected, the maximum total horizontal force (166 kN (+240%)) can be reached, if one assumes a rigid restraint of the walls in the floors (system C). Table 5 shows that the shear failure criterion “diagonal tensile failure of the units” is always the decisive one.

Table 5: Total horizontal force according to DIN 1053

System		A			B	C
Total horizontal force (terraced house)	[kN]	**69**			**104**	**166**
Force upper storey	[kN]	45			68	109
Force ground floor	[kN]	24			36	57
Total internal moment at the base	[kNm]	287			261	207
Single wall		**W2**	**W3**	**W5**	**W5**	**W5**
Horizontal load parts of the single walls of a house halve (hh)	[-]	0.108	0.108	0.285	0,285	0,285
Vertical load parts of the single walls of a house halve (hh)	[-]	0.066	0.071	0.147	0,147	0,147
Internal forces and moments at the bottom of the single wall (hh)						
Internal moment $M_{E,bottom}$	[kNm]	31.0	31.0	81.9	74.3	59.1
Shear force $V_{E,bottom}$	[kN]	7.5	7.5	19.8	29.7	47.3
Vertical loads resulting of the floors	[kN]	49.3	53.0	109.8	109.8	109,8
Own weight	[kN]	17.2	17.2	10.5	10.5	10.5
Normal force $N_{E,bottom}$	[kN]	66.5	70.2	120.3	120.3	120.3
Eccentricity related to the length of the single wall	[-]	0.397	0.376	0.454	0.412	0.328
Resistance against bending (flexural) failure $N_{R,bottom}$						
$N_{R,bottom}$	[kN]	528.4	637.2	145.9	278.6	543.2
$N_{E,bottom}$ / $N_{R,bottom}$	[-]	**0.13**	**0.11**	**0.82**	**0.43**	**0.22**
Resistance against shear failure						
Shear distribution parameter c	[-]	1.50	1.50	1.50	1.50	1.50
Uncracked cross sectional area A_c	[m²]	0.13	0.16	0.04	0.07	0.14
Mean normal stress in the uncracked cross section σ_x	[N/mm²]	0.503	0.441	3.298	1.728	0.886
Shear strength, in-plane shear failure	[N/mm²]	0.376	0.353	1.456	0.867	0.551
Shear strength, diagonal tensile failure of the unit	[N/mm²]	*0.319*	*0.309*	*0.815*	*0.640*	*0.523*
Shear strength, diagonal compression failure of units	[N/mm²]	5.497	5.559	2.702	4.272	5.114
Decisive (minimum) shear strength f_{vk}	[N/mm²]	0.319	0.309	0.815	0.640	0.523
$V_{R,bottom}$	[kN]	28.1	32.8	*19.8*	29.7	47.3
$V_{E,bottom}$ / $V_{R,bottom}$	[-]	**0.27**	**0.23**	***1.00***	***1.00***	***1.00***

Analogous to the calculation on basis of the model of DIN 1053, in the following the total horizontal force of the terraced house will be calculated with the new approach.
The approach consists of the bending failure criterion (assuming a stress block for the strain-stress-relationship) and three criteria for shear failure (Gaping of the single units, Friction failure/Sliding and Diagonal tensile failure). With the equations 1 to 4 the standardized horizontal bearing capacity $v=V/(t\cdot l\cdot f)$ can be calculated in dependence of the standardized vertical force $n=N/(t\cdot l\cdot f)$ and other material and geometric parameters of the respective single wall. More details regarding the new approach and the equations can be found in [4] and [16].

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

$$v_{friction} = \mu\cdot n \tag{2}$$

$$v_{gaping} = \frac{1}{2}\cdot\left(\frac{1}{h_{st}}+\frac{1}{h}\right)\cdot l_{st}\cdot n \tag{3}$$

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

only if $\lambda_v \leq 1.5$ otherwise not needed

With:	v	$= V/(t \cdot l \cdot f)$	standardized horizontal bearing capacity
	n	$= N/(t \cdot l \cdot f)$	standardized vertical force
	h,l		storey height, length of the single wall
	t		thickness of unit /wall
	f		compressive strength of the masonry
	λ_v	$= \psi \cdot h/l$	shear slenderness
	μ		friction coefficient bed joint
	h_{st}, l_{st}		height, length of the single units
	c	$= 0{,}5+\lambda_v$	parameter to consider the shear slenderness
	$\overline{f_{bt,cal}}$	$= f_{bt,cal}/f$	standardized tensile strength of the unit
	F^*	$= 1{,}2+0.85 \cdot f_{bt,cal}$	calibration parameter

A very important item within the calculation with the new approach is the determination of the parameter ψ, which is needed to calculate the shear slenderness λ_v of a single wall. The parameter ψ depends on the load eccentricity at the top and at the bottom of the respective wall when the calculated load capacity of the wall is reached. In dependence of the assumed system A, B or C one gets the following values for the wall W5 in the ground floor, when the maximum horizontal force is reached:

$$\text{System A:}\quad \psi_{gf,W5,A} = \frac{e_{bottom}}{e_{bottom} - e_{top}} = \frac{0.69}{0.69-0.29} = 1.72$$

$$\text{System B:}\quad \psi_{gf,W5,B} = \frac{e_{bottom}}{e_{bottom} - e_{top}} = \frac{0.69}{0.69-0} = 1.0$$

$$\text{System C:}\quad \psi_{gf,W5,C} = \frac{e_{bottom}}{e_{bottom} - e_{top}} = \frac{0.35}{0.35-(-0.35)} = 0.5$$

Table 6 shows that the out of ψ resulting shear slenderness λ_v, with the exception of wall W5 under consideration of system C, is always greater than 1.5. Due to this reason only in this case the failure criterion “diagonal tensile failure” in the middle of the wall has to be considered in addition to the criteria “gaping of the single units” and “friction failure” and becomes decisive. The background of the limit value 1.5 is that it was found out in [4, 16] that for common clay unit masonry walls with $\lambda_v > 1.5$ the failure criterion for bending (flexural) failure by assuming a stress block for the strain-stress-relationship gives already adequate horizontal forces if one compares them with the ones observed in tests. The criterion “diagonal tensile failure” only needs to be taken into account if the shear slenderness of the wall is smaller than $\lambda_v \leq 1.5$.

Table 7 shows that again the wall W5 is the decisive one. One can see that its shear resistance is larger than the shear resistance according to the model of DIN 1053. However, hence bending (flexural) failure becomes decisive, the new approach only leads to a marginal increase of the total horizontal load (71 kN (+3% related to the result of DIN 1053, system A)) if one assumes system A. Stronger differences between the total horizontal force according on the new approach compared with the total horizontal force on basis of the model of DIN 1053 one gets if one assumes system B (117 kN (+170% related to the result of DIN 1053, system A)) or system C (234 kN (+340% related to the result of DIN 1053, system A)).

Table 6: Total horizontal force according to the new approach

System		A			B	C
Total horizontal force (terraced house)	[kN]	**71**			**117**	**234**
Force upper storey	[kN]	46			76	153
Force ground floor	[kN]	25			41	81
Total internal moment at the base	[kNm]	292			292	292
Single wall		**W2**	**W3**	**W5**	**W5**	**W5**
Horizontal load parts of the single walls of a house halve (hh)	[-]	0.108	0.108	0.285	0.285	0.285
Vertical load parts of the single walls of a house halve (hh)	[-]	0.066	0.071	0.147	0.147	0.147
Internal forces and moments at the bottom of the single wall (hh)						
Internal moment $M_{E,bottom}$	[kNm]	31.6	31.6	83.3	83.3	83.3
Shear force $V_{E,bottom}$	[kN]	7.6	7.6	20.2	33.3	66.7
Vertical loads resulting of the floors	[kN]	49.3	53.0	109.8	109.8	109.8
Own weight	[kN]	17.2	17.2	10.5	10.5	10.5
Normal force $N_{E,bottom}$	[kN]	66.5	70.2	120.3	120.3	120.3
Eccentricity related to the length of the single wall	[-]	0.404	0.383	0.462	0.462	0.462
Resistance against bending (flexural) failure $N_{R,bottom}$						
$N_{R,bottom}$	[kN]	491.8	602.5	120.3	120.3	120.3
$N_{E,bottom}$ / $N_{R,bottom}$	[-]	**0.14**	**0.12**	***1.00***	***1.00***	***1.00***
Resistance against shear failure						
Sliding shear $V_{R,bottom,friction}$	[kN]	39.9	42.1	*72.2*	72.2	72.2
Height of the unit	[m]	0.25	0.25	0.25	0.25	0.25
Length of the unit	[m]	0.25	0.25	0.375	0.375	0.375
Gaping of the single units $V_{R,bottom,gaping}$	[kN]	*36.6*	*38.6*	99.3	99.3	99.3
eccentricity upper edge of the wall in the groundfloor e_o	[m]	0.22	0.20	0.29	0	-0.35
eccentricity lower edge of the wall in the groundfloor e_u	[m]	0.48	0.45	0.69	0.69	0.35
Parameter ψ	[-]	1.8	1.8	1.7	1.0	0.5
Shear slenderness λ_v	[-]	3.9	3.9	2.8	1.7	0.8
Parameter c	[-]	(1.5)	(1.5)	(1.5)	(1.5)	1.33
Parameter F*	[-]	(1.62)	(1.62)	(1.88)	(1.88)	1.88
Diagonal Tensile failure $V_{R,tensile}$ in the middle of the wall height	[kN]	(59.5)	(60.1)	(61.5)	(61.5)	*69.2*
Decisive (minimum) shear resistance V_R	[kN]	36.6	38.6	72.2	72.2	69.2
$V_{E,bottom}$ / V_R	[-]	**0.21**	**0.20**	**0.28**	**0.46**	**0.96**

COMPARISON OF THE CALCULATED RESULTS WITH THE TEST RESULTS

In a first step, the results of the large-scale test have been analysed with regard to the relevance for earthquake design in Germany. Table 7 shows the total horizontal forces (seismic base shear forces F_b) for different ground conditions and seismic zones according to DIN 4149 [17]. The lateral force method of analysis has been used for the calculation of F_b. The seismic base shear force F_b, is the outcome of the total mass of the building M multiplied with the ordinate of the design spectrum S_d. The behaviour factor was chosen as q = 1.8 according to table 17 in [17]

Table 7: Total horizontal force (seismic base shear force) F_b according to DIN 4149 [17]

Seismic zone	Ground conditions					
	C-S	B-T	C-T	A-R	B-R	C-R
1	46	62	77	62	77	92
2	69	92	115	92	115	138
3	92	123	153	123	153	183

The maximum total horizontal force observed in the pseudo-dynamic large-scale test (≈250 kN) are significantly higher than the seismic base shear forces in table 7 which cover all seismic zones and ground conditions in Germany. Even the most unfavourable combination 3/C-R is covered abundantly clear (183/250 = 0.73). Furthermore one has to keep in mind, that this maximum seismic base shear force according to DIN 4149 is even less than total horizontal load of 190 kN, where still no cracks did occur in the terraced house.

Figure 2 in combination with table 3 shows that in case of accelerations larger than 1 m/s² the terraced house did not react with an increase of the horizontal load, but with large deformations. Obviously the behaviour factor 1.8 is completely not able to describe this fact.

Figure 4 shows that with the model of DIN 1053 in combination with the commonly used structural system A (cantilever), only the seismic base shear force for the combinations (1/C-S, 1/B-T, 1/C-T, 1/A-R, 1/B-R and 2/C-S can be covered. In case of system B, the combinations 1/C-R, 2/B-T, 2/A-R and 3/C-S are covered as well. If System C is assumed, all combinations with exception of the most unfavourable combination 3/C-R are covered. Nevertheless, it is remarkable that even under consideration of system C the calculated total horizontal force is far from the one observed in the large-scale test (166/250 = 0.66)!

Considering system A, the new approach allows the verification of the same combinations as the model of DIN 1053. With system B, the new approach can cover all seismic base shear forces of seismic zone 1 and 2 with exception of the combination 2/C-R. Considering a full restraint of the wall in the floor slabs, which leads to a point of zero moment in the middle of the wall (system C) enables to cover all seismic base shear forces listed in table 7. The respective total horizontal force of 234 kN matches the observed test result very well (234/250 = 0,94).

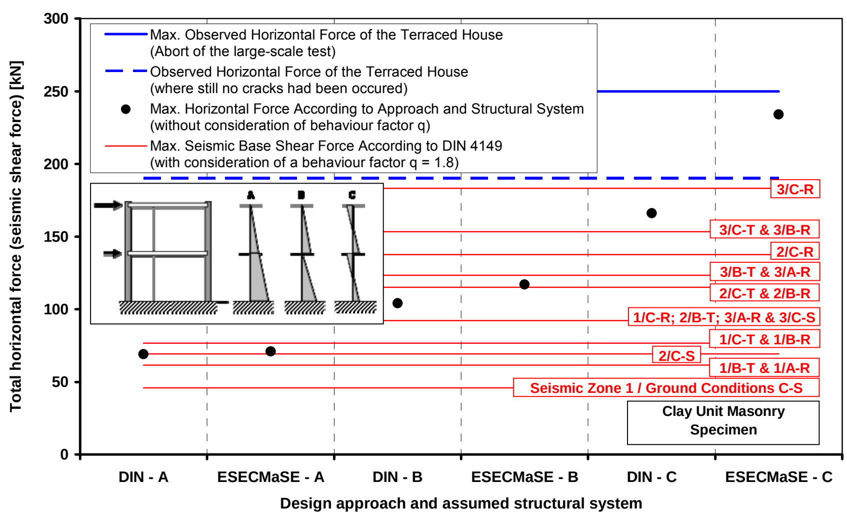


Figure 4: Comparison of the observed total horizontal forces of the large-scale test with the seismic base shear forces according to DIN 1449 and on basis of different approaches

CONCLUSIONS

The presented results show clearly that the resistance of masonry structures subjected to earthquake is extremely underestimated with the commonly assumed cantilever as structural model. This applies for the design concept of DIN 1053 as well as for the new and more realistic approach. The main reason is that assuming a cantilever system in combination with the other

simplifications which are necessary to enable a force-based design by hand (e.g. neglecting of the occurring load redistributions and of the deformation behaviour of the masonry building) does not allow a realistic description of the behaviour of the masonry structure. This can not be compensated with the currently valid behaviour factor for masonry structures.
Higher and more realistic design resistances can be reached taking the restraining effect between the walls and the floors into account. Because of the already mentioned simplifications even assuming a rigid restraint of the walls one is not able to reach the horizontal total force observed in the large-scale test with the design concept of DIN 1053 without the consideration of a behaviour factor. On the other hand, a calculation with the new approach under consideration of a rigid restraint of the walls in the floors and an additional consideration of a behaviour factor could lead to an overestimation.
Based on the presented results, for further design of masonry structures it will be suggested to take the new approach by assuming a storey high cantilever (system B) into account and to consider an additional behaviour factor. Many countries already use this storey model limiting it to maximum two floors for safety reasons. In this way one would get higher and more realistic design resistances and avoids that nonlinear effects will be considered twice. Nevertheless one has to keep in mind, that both model B and model C could have great implications on floors, slabs and lintels that should not be neglected.
It is foreseen to verify this suggestion procedure with further tests and with further calculations on basis of the deformation-based design method as shown in [12,18] and to implement it in the new version of DIN 1053.

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