Faculty of Architecture Chair of Structural Design

Summary of the Final Report

Title: **Using Confined Masonry to Increase the Load-Bearing Capacity of Stiffening Walls**

Introduction of confined masonry to increase the load-bearing capacity of shear panels with the aim of compensating extra costs caused by higher horizontal loads

file number: Z6-10.07.03-05.11

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with the kind support of:
Bundesverband Porenbeton und
Xella Technologie- und Forschungsgesellschaft mbH

„This research report was funded by the Federal Office for Building and Regional Planning [Bundesamt für Bauwesen und Raumordnung].
(reference number: Z6-10.07.03-05.11 / II 13 – 80 01 05 – 11)
The author is responsible for the content of the report.“

1 Aim of the Research

As the transition to the semi-probabilistic safety concept continues, the horizontal load acting on buildings will increase owing to the theoretical probabilistic assessment of the forces. Particularly in areas of high wind loads and increased seismic activity this will result in problematic analysis and certification as regards the stiffening of masonry buildings. The thus necessary strengthening will increase costs, on the one hand, and result in the loss of market shares of a construction method that has been profitable in the past – the masonry type.

Confined masonry is a viable option to improve the horizontal load-bearing capacity of masonry structures. The columns and beams embracing the masonry walls improve their behaviour under horizontal loads.

However, the research carried out to date in Germany is limited to only a few experimental and theoretical tests of the seismic behaviour of these structures. As a consequence of this, the significant improvement of the horizontal and also vertical load-bearing capacity through the use of confining elements in masonry walls is accounted for in the building code DIN 4149 only by an empirical behaviour factor q = 2.0.

It was the aim of this research to present the structural benefits on the one hand, and to examine the particular issues arising from the production process. To stimulate the propagation of confined masonry in essential stiffening structural members in Germany, a design algorithm has been developed which the engineer can use to identify all benefits in case of static and dynamic loads.

2 Execution of the Research Project

2.1 General facts

Confined masonry is distinguished both from reinforced masonry and infill walls. The major difference between confined masonry and infill wall structures is the fact that the infill walls bear part of the vertical load. Therefore, a decisive aspect of confined masonry is the order in which structural members are made. While in skeleton construction the RC frame is built first and then the infill, in confined masonry it is the other way round.

In reinforced masonry, the vertical reinforcement is located in the masonry units with holes or openings that are filled with concrete after the wall has been erected. Then, after their erection, the walls are framed with reinforced concrete as well. However, this frame does not contain shear reinforcement and does not constitute an independent frame. Yet improved bending resistance can be stated for confined masonry as well.

Hardly any of the German and European building codes deal with confined masonry. As a rule, it is up to the design engineer to take into account the various types of construction. While in Europe the favourite construction type in areas with high seismic activity is reinforced concrete, outside Europe masonry is also quite common in earthquake zones. For example, the Peruvian masonry code SENCICO 2006 distinguishes between confined masonry and infill walls for the stiffening of buildings. A behaviour factor of 3 is determined for confined masonry. The structural design equations are empirically derived from a large number of cyclic and vibration tests.

For a long time, the ‘standardised shear test’ has been performed to determine the shear strength. For this purpose, a section of the wall that is subjected to shear stress is considered. The shear stress is applied all around a square shear panel. This test arrangement is adopted from the analytical approach developed by *Mann/Müller*.

Over the last ten years, researchers of the shear resistance of masonry, in particular in earthquake zones, increasingly used test arrangements relating to shear walls in buildings.

For example, the European research project ESECMaSE conducted shear tests such that the vertical load can be applied via two vertical presses at the top of the wall and a moment load can be applied to the wall. The tests conducted in this project are also based on this procedure.

2.2 Experimental tests

Four tests have been performed with masonry made of autoclaved aerated concrete (AAC) to assess the shear resistance of confined masonry. These tests were designed to observe the particular behaviour of this structure compared with commonly used stiffening walls. It was the aim of the experimentation to document not only the shear resistance of a masonry wall with reinforced concrete frames but also the prestresses caused by different shrinkage processes. The test included two variants. In the first, a constant vertical load was applied to the test wall. Thus the external point of zero moment was located at the top of the wall. In the last variant, the two vertical cylinders were controlled thus that the point of zero moment was located at half the height of the wall with the total load remaining constant.

During the erection of the test walls, additional specimens made of the same materials were manufactured to determine the necessary input parameters for the numerical models.

The deformations of the test walls were measured after manufacturing and before the actual tests and also during experimentation. After the manufacturing process, shrinkage deformation was measured with mechanical extensometer on the test walls and via steel rods that are sheathed and cast into the RC frame. These measurements confirmed the expected shrinkage which leads to the prestress in the masonry wall.

In the shear tests, a vertical load was applied before a cyclic horizontal deformation was generated. The deformation was intensified after each third cycle. Figure 1 shows the typical first cracks. In Figure 2, the limit loads obtained are compared with current AAC tests without confinement.

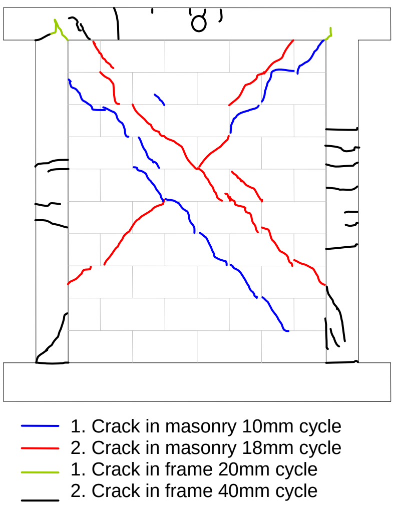

Figure 1 crack pattern wall 2 (load 132 kN)

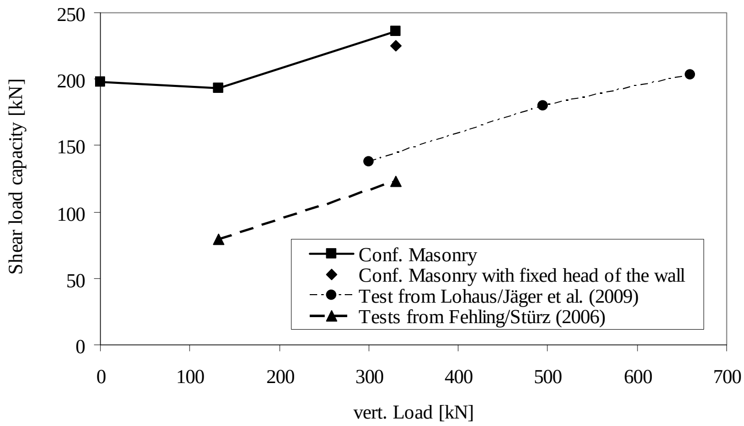

Figure 1 Comparison with current AAC tests reported in the literature

Bilinear simplification is applied to the envelope curve obtained from the hysteresis of the load displacement diagram to assess ductility. The values obtained in the traditional manner are given in the table below as variant 1 (V1).

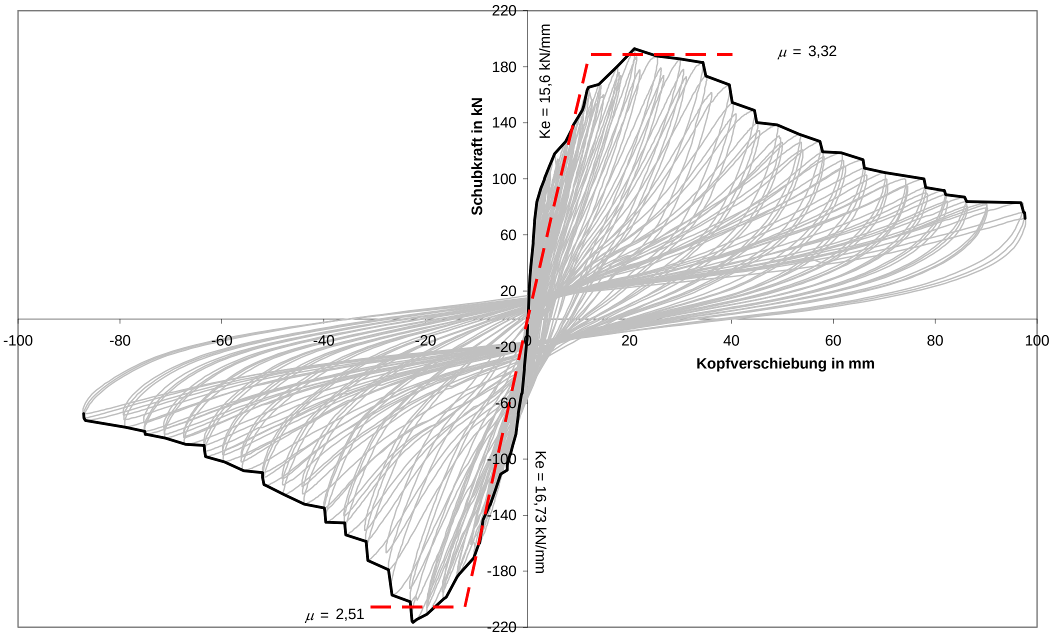

Figure 1 Hysteresis and envelope curve for wall 2 (vertical load 132 kN)

Table 1 Overview of the wall properties

	wall		1	2	3	4
	vertical load	kN	330	132	-	330
V1	H_{max}^{-}	kN	250	217	198	242
	H_{max}^{+}	kN	236	193	198	225
	μ^{-}		3,58	2,51	1,64	5,68
	μ^{+}		6,18	3,32	4,30	8,55
V2	H_{cr}^{-}	kN	100	107	86	125
	H_{cr}^{+}	kN	97	116	87	124
	μ^{-}		9,95	4,43	6,56	7,59
	μ^{+}		11,42	5,18	14,18	11,28
V3	d_u^{-}	mm	94,3	87,2	104,3	75,7
	d_u^{+}	mm	97,6	97,7	112,1	66,6
	μ^{-}		21,13	17,72	26,13	12,08
	μ^{+}		24,87	16,55	32,57	16,69

Owing to the marked non-linear course of the load displacement diagram in the rise region, the plastic deformation is already greater for 70% of the maximum shear load and thus the formal initial stiffness and ductility are smaller. The calculated values ranging between 4.3 mm and 13.1 mm for d_{cr} represent maximum deformation for normal masonry panels. For this reason, we did not use 70% of the maximum shear load for variant V2 for H_{cr} but the observed first crack load to calculate ductility. In the third variant V3, we additionally included the maximum deformation obtained in the test to determine ductility.

2.3 Numerical and analytical considerations

The program system ANSYS, which we used in the analytical studies of this research, allows to change parameters in the numerical models easily through input that is controlled by scripts. Moreover, a programming interface allows users to integrate their own elements and material routines. As already ascertained in the studies of other research projects, the projection of the bond behaviour is decisive when considering the shear resistance behaviour. Therefore, various interface elements were implemented in the program system ANSYS within this project. For AAC, the compression-tension failure within the masonry units is also essential. An additional material routine has been developed for the existing two-dimensional elements. This routine can be used to model the failure process in the test wall numerically and to perform a parameter study. The figure below shows a typical numerical crack pattern and the flow of forces within the wall after the first cracks appear.

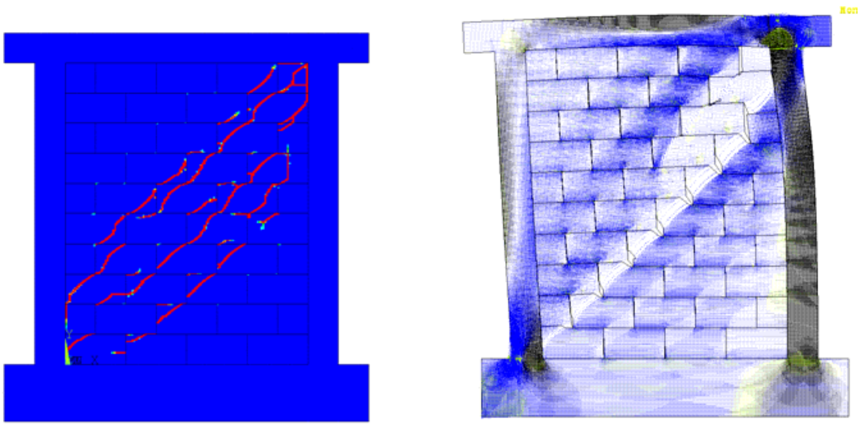

Figure 1 Typical numerical crack pattern and flow of forces of a test wall

The two halves produced by the cracks form partial panels and continue to transfer the shear load. At the end of the crack, the frame is subjected to shear loading.

Since the reinforced concrete frame used has not been tested individually, its shear resistance is numerically and analytically estimated. Without vertical load, the shear load is in the range 59 to 65 kN, with vertical load it is slightly higher and is reached only when the deformation is approximately 50 mm. Hence the direct share of the frame in the shear resistance is small.

Using an FE model is the only method to study exactly the interaction between frame and masonry during shrinkage. Since even the early shrinkage of the concrete is prevented by the masonry, first plastic elongations occur already in the concrete which reduce the final shrinkage value. A second influencing factor is the comparatively short time of approximately 2 months between production and testing. Therefore, only 20 - 25% of the final shrinkage value were reached during the time period reported. The resulting tensions in the masonry wall remained clearly below the strength values, but in the concrete they almost reached the tensile strength values. However, local relaxation in the frame corners, for example as a result of crack formation, does not lead to a complete loss of the prestress, which was shown in another numerical study.

As expected, the degree of shrinkage has an effect on the prestress of the masonry and thus on its shear resistance. The horizontal shrinkage of the upper beam itself causes shear stress in the masonry wall, which in turn reduces the load-bearing capacity. A further increase in the shrinkage value can therefore increase the load-bearing capacity only slightly or even not at all.

The numerical model has a greater initial stiffness and load reduction is greater in case of crack formation. The inhomogeneities of the masonry are the cause of the divergent behaviour. The simulation is based on the assumption that the material is homogenous and its strength and stiffness are the same for each element. Additionally, all head joints are mapped thereby neglecting the joint thickness. A comparative calculation assuming no contact in the head joints provided, however, the same crack load, but stiffness was lower. In case of a dispersive tensile strength of the masonry units, it is expected that crack formation is less brittle and may start earlier. Despite the limitations mentioned, agreement between the experimental and numerical results is good.

Both the length of the wall and the format of the masonry units were modified in the parameter study. The unit length, which was halved, resulted in a smaller initial stiffness when compared with the units used in the tests, and also in a smaller load-bearing capacity for the smaller vertical loads. Lengthening the wall proportionally increases the shear resistance.

In further variants, the thickness of the surrounding RC frame and the reinforcement degree were varied. Joint failure could be analysed only numerically because the bond strength values in the AAC units used are greater than the tensile strength of the masonry units. Failure varies depending on the unit geometry, the vertical load and the ratio of tensile bond strength and initial shear strength. The gapping effect illustrated in Figure 5 will not necessarily lead to failure in confined masonry.

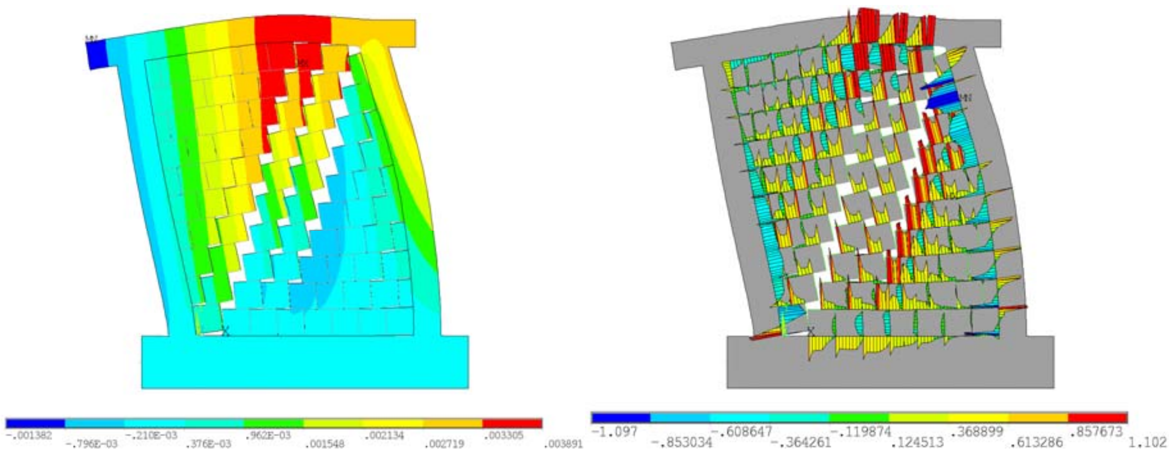

Figure 1 Vertical deformations and normal stresses in joints without vertical loads

The increased vertical deformation activates an additional vertical load via the frame, which increases shear resistance.

3 Summary of Results

With the kind support of the Federal Association of AAC Germany / Xella Technology- and Research Centre Emstal, four cyclic shear tests were performed with confined AAC walls within the research project ‘Using Confined Masonry to Increase the Load-Bearing Capacity of Stiffening Walls’. A parallel study examined all relevant properties of the construction materials used. Moreover, interface elements and a material routine were implemented into the program system ANSYS that is employed in the numerical simulation. Thus a numerical parameter study could be performed to examine further influencing factors.

The aim of this project is to verify a higher shear resistance and improved ductility for masonry walls. A test without vertical load was the lowest limit for confined masonry. The static system assumed for the first three tests was a cantilever. The shear resistance obtained here without vertical load very clearly exceeds that of conventional masonry with vertical load and fixed support at the top. Increasing the vertical load further increased the load-bearing capacity.

The fourth test wall was used to simulate a fixed support through the top of the building. This could not improve shear resistance as it does in conventional masonry. However, the initial stiffness was higher.

Another major benefit of confined masonry is its high ductility which is reflected above all in the maximum deformation. At some points of the tests, the maximum capacity (displacement) of the test equipment was reached. If ductility μ is calculated following the conventional procedure it does not adequately reflect this property. The areas under the envelope curve may improve comparability instead.

The high deformability of the RC frame ensures a high remaining load-bearing capacity which may save the building from total collapse.

The comparative numerical analysis showed the influence of the manufacturing process. Casting the concrete after the masonry wall is erected on the one hand improves the bonding quality between frame and infill and on the other hand the masonry is prestressed. This increases the load-bearing capacity in particular for small vertical loads.

Moreover, further influencing factors were numerically varied, which demonstrated a certain dependency of the load-bearing capacity on the format of the masonry units. Failure analysis of the masonry joints could be done only numerically. In case units with a high h/l ratio were used, additional loads can arise for the frame because of gaping and the resulting rotation of individual masonry struts.

The first structural design proposal is made. It rests on the design equations for unreinforced masonry and allows the transfer of the results to confined masonry. The testing and the numerical analysis showed a stress condition that is similar to the unified shear test due to shrinkage and the good bonding quality between masonry and frame. It is therefore suggested to start from the simplified assumption of a masonry segment with uniform shear load. Additional structural analyses should be done for the frame.

A simple simulation using FEM and linear-elastic material behaviour can be employed to determine the distribution of the vertical load between masonry and the RC frame. Further structural analytical calculations are done following DIN 1053 for simple shear wall panels made of unreinforced masonry. However, the factor of the shear stress distribution for the tensile failure of the units can be set at 1.0 or the load-bearing capacity of an unreinforced wall can be increased by 1.5. In case of friction failure, the load-bearing capacity is considerably higher since the total length of the wall infill is used as opposed to the compressed length of the normal wall panels. A reducing coefficient is introduced to take account of the effect of non-uniform stress distribution within the wall. This effect is vital particularly because of the resulting extensive loss of the bonding strength.

For the bending analysis of the wall panel, we assume reinforced masonry with vertical reinforcement as a first approximation. In a more detailed calculation, shrinkage and also – additionally for the compression area – the higher compression strength and Young's modulus of the concrete can be taken into account.

After crack formation in the infill, which is diagonal as a rule, the frame is also subjected to shear forces at the ends of the cracks since the two resulting halves of the wall are held together based on the dowel effect. The worst assumption would be that two halves of the wall take up the same share of the shear force, which results in a lateral force for the frame that is half the shear force. It is necessary to perform a lateral force analysis for the frame.

Since the external static system has no decisive effect on the shear resistance, the dimensioning can be simplified in comparison with buildings with masonry walls only. It is not necessary to exactly calculate the internal static forces as regards the moment distribution in the stiffening walls. Extensive building modelling is not obligatory.

The shear wall tests performed in this project are limited to AAC masonry. For more general statements, further experimentation with walls made of calcium silicate and clay units are particularly interesting and indispensable. It is expected that the bonding behaviour and also the behaviour in case of tensile failure of the bricks will be different.

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