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CONFINED MASONRY – A CHANCE TO IMPROVE THE LOAD BEARING CAPACITY

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ABSTRACT: *With the introduction of the semiprobabilistic safety concept with partial factors for masonry design in Germany an increase of the horizontal action due to wind- and seismic loads is connected (s. 1). This fact was overlaid by the reduction of the number and area of stiffening walls from the point of view of economy as well as by lowering the bulk density to achieve a higher thermal insulation. The latter leads to a reduction of the available strength of the units also. Therefore considerable research efforts have been made in Germany to compensate the losses of bearing capacity and to guarantee the competitiveness of masonry with other building materials (s. 2 - 6). Confined masonry is an adequate alternative – besides the execution as reinforced masonry – in order to increase the design resistance. It has until now found only a low application in Germany. That's why it was taken into account in the efforts to increase the load bearing capacity of stiffening walls by the research project presented in the following.*

KEYWORDS: *confined masonry, cyclic shear tests, effect of shrinkage, numerical simulation*

I. INTRODUCTION

Confined masonry was in the past and is in the present a continuous matter of research interest to understand the structural behaviors as well as to simplify the design methods for the practices without any complicated numerical tools (s. 3). In the following some new aspects were introduced in the research focus like the shrinkage of the RC-frame and the location of the zero point of moment.

Confined masonry differs both from reinforced masonry and from infill masonry. The most essential difference in comparison with infill masonry consists in the fact that the masonry carries a portion of the vertical load. The sequence in the erection of the structural members is therefore an important factor for confined masonry. In skeleton structures with infill masonry the RC-frame is fabricated first and afterwards comes the infill. For confined masonry the order is reversed. This has a direct effect on the bond strength between frame and masonry as well as on the vertical load upheld by the masonry.

Masonry units with vertical holes or openings are used in the case of reinforced masonry with vertical rebars. The openings or holes foreseen for the rebars are filled with concrete during or after erection of the masonry wall. At the end of the construction process the masonry is confined by reinforced concrete. This confinement however does not possess shear reinforcement and is not a real frame able to carry horizontal loads without the masonry infill. However, the increased load bearing capacity for bending due to inplane loads of the whole wall is also an effect of confinement.

Confined masonry combines the positive properties of both construction types and can achieve higher load-carrying capacities under static as well as under seismic actions.

EXPERIMENTAL TESTS

For the assessment of the shear load capacity of confined masonry four test were carried out with masonry made of autoclaved aerated concrete (AAC). In this connection the special behaviour of this construction type was examined as compared to common stiffening walls. The aim was to assess the shear load behaviour of a masonry wall confined by reinforced concrete along with the pre-stressing effects caused by different processes of shrinkage.

The reinforcement of the RC-frame followed the least requirements of the German earthquake code DIN 4149. The thickness of the frame corresponded to the thickness of the masonry of 24 cm. For the masonry were used units of the strength category 4 (fbk = 4 N/mm²), dimensions 50 cm × 24 cm × 25 cm and overlapping length of a half unit. The bed joints were made with thin layer mortar and the head joints were unfilled.

The deformation of the test specimens were measured after the erection and prior to the testing, as well as during the test process. In the period after the production the deformations due to shrinkage were measured with mechanical extensometers on the surface of the test walls and with digital meters at the end of steel bars, which were placed in cladding tubes inside of the RCframe. The arrangement of the measuring points is shown in Figure

2: Plan of measurement points for shrinkage of the wall (left: mechanical extensometer; right: steel bars in cladding tubes with digital meter)

The concrete of the upper beams was cast several days after the columns. The measurement of the length changes could only start on day 21 after the casting of the columns because of the formwork. An essential part of the shrinkage had already happened at that time. The length changes in the columns were therefore smaller than in the upper beam. This is also to be recognised by the lower increase rate of the curves at the beginning. For the walls 1 and 2 approximately -0.2 mm and for the walls 3 and 4 approximately -0.3 mm are to be added for a proper comparison. The shortening observed at all places results from the shrinkage of the concrete respectively from the resulting compression strain in the masonry part. The precompression of the masonry could be reproduced numerically (see below).

3: Deformation due to shrinkage for wall 4 (left: mechanical extensometer; right: steel bars in cladding tubes with digital meter With the test arrangement for the shear tests two variants were examined. With the first variant only a constant vertical load was applied on the test walls. Therefore, the external point of zero moment lies at the top of the wall. For the last test both vertical cylinders were computer controlled to keep the external point of zero moment at the half wall height. So the load of both cylinders is partially different, but the total load has a constant value. In the shear test the vertical load were applied during the first load step. In the second step a cyclic horizontal displacement was imposed at the top. This was increased after every third cycle. Figure 4: shows the typical first cracks. .

4: Typical crack pattern (wall 2) On the right side of Figure 4: is shown the crack pattern at the end of the test. The shear resistance in the last cycle is still higher than that of a frame without masonry. Partially the displacement capacity of the test equipment was reached. A further increasing of the horizontal displacement was possible in all tests.

For the evaluation of the ductility the envelope of the hysteresis of the load-displacement diagram has to be simplified to a bilinear curve. This is shown for all four hysteresis in the following picture. The shown displacements are the relative ones between the upper and the lower beams. These were measured with a separate decoupled measuring frame.

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5: Force-displacement -hysterese - envelopes und the bilinear simplification (top left: wall 1, right: wall 2, down left: wall 3, right: wall 4, dashed line: bilinear curve) The initial stiffness is in the common approach determined at 70% of the maximum shear load. For the maximum usable displacement, the point of 80% of the maximum shear load on the declined part of the envelope has to be used. The maximum load of the bilinear simplification is given by the equality of the enclosed area of both curves. In Table 1 are listed the essential results of the test.

By the distinctive non-linear characteristics of the load-deformation-curves in the rising part a larger plastic displacement is reached at 70% of the maximum shear load. This leads formally to a lower initial stiffness and much lower ductility. The values of dcr are between 4.3 mm and 13.1 mm and would already equal the maximum displacement for the case of normal masonry.

Hence the observed first crack load is used in Table 2 for the calculation of the ductility and not 70% of the maximum load. So the value for the ductility becomes much higher.

An essential potential of confined masonry remains unused by the restriction of the usable area to the load-carrying capacity of 80%, because an increasing of the displacement at the top of the wall is possible and the load still lies clearly above that of conventional walls..

Wall	vert. Load	H_{max}-	H_{max}+	d_{Hmax}-	d_{Hmax}+	70% H_{max}-	70% H_{max}+	dcr-	dcr+	Ke-	Ke+
	kN	kN	kN	mm	mm	kN	kN	Mm	mm	kN/mm	kN/mm
1	330	250	236	24.3	24.5	175	165	10.7	6.5	16.4	25.3
2	132	217	193	22.5	20.9	152	135	9.1	8.7	16.7	15.6
3	-	198	198	25.1	26.0	138	138	13.1	10.0	10.6	13.9
4	330	242	225	15.0	22.1	170	158	6.8	4.3	24.9	36.6

Wall	80% Hmax-	80% Hmax+	du-	du+	Hu-	Hu+	de-	de+	μ^-	μ^+
	kN	kN	mm	mm	kN	kN	mm	mm		
1	200	189	51.4	52.0	235	213	14.3	8.4	3.58	6.18
2	173	154	30.8	40.2	206	189	12.3	12.1	2.51	3.32
3	158	158	32.5	56.3	210	181	19.8	13.1	1.64	4.30
4	194	180	50.6	48.3	222	207	8.9	5.6	5.68	8.55

Wall	Hcr-	Hcr+	dcr-	dcr+	Ke-	Ke+	Hu-	Hu+	de-	de+	μ^-	μ^+
	kN	kN	mm	mm	kN/mm	kN/mm	kN	kN	mm	mm		
1	100	97	2.4	2.2	41.3	44.9	213	204	5.2	4.6	9.95	11.42
2	107	116	4	5.1	26.7	22.8	185	177	6.9	7.8	4.43	5.18
3	86	87	2.7	2.1	31.9	41.8	158	166	4.9	4	6.56	14.18
4	125	124	3.8	2.6	32.5	47.6	217	204	6.7	4.3	7.59	11.28

In Figure 6: the observed loads are compared with actual results for conventional AAC-walls. The wall dimensions are identical for all tests resp. were projected on a uniform wall thickness. The conventional walls, which were used for the comparison, were fully fixed at the top of the wall. With the attempt of a cantilever-system the load-bearing capacity would clearly decrease in contrast to confined masonry.
In all four tests no joint failure could be observed. Also the bond between masonry and RCframe remained intact through the whole test procedure. The failure always happened in the stone units. So the tensile strength of the unit gives the limit for the shear load capacity.

NUMERICAL INVESTIGATIONS
The appropriate parameters for the numerical calculation of the test were determined in small "material"-tests parallel to the wall tests. In order to obtain the values of the material point some of the small tests have also been recalculated with a numerical model. The relevant material characteristics, which were used for the numerical simulation of the cyclic shear test, are summarised in Table 3.

Table 3: Input values for numerical analyses

	Parameter		Value
Concrete	Density	γwc	22.9 kN/m^3
	Young's Modulus	Ec	27187 N/mm^2
	Compressive Strength	fc	36.8 N/mm^2
	Tensile Strength	fct	2.51 N/mm^2
	Poisson's ratio	vc	0.2
	Fracture Energy	GIc	274 N/m
Reinf.	Yield Stress	fy	590 N/mm^2
	Tensile Strength	fstt	640 N/mm^2
	Young's Modulus	Est	200000 N/mm^2
	Parameter		Value
Masonry Unit	Density	γ_{wb}	5.4 kN/m^3
	Compressive Strength	f_b	4.59 N/mm^2
	Tensile Strength	fbt	0.87 N/mm^2
	Poisson's ratio	ν_b	0.15
	Young's Modulus	Eb	1654 N/mm^2
	Fracture Energy	GIb	10 N/m
Mortar	Tensile Strength	fmt	2.59 N/mm^2
	Young's Modulus	Em	6809 N/mm^2
	Poisson's ratio	ν_m	0.17
Bond	Shear strength	$fvko$	1.1 N/mm^2
	Tensile Bond Strength	f_t	0.84 N/mm^2
	Friction Coefficient	μ	0,84 N/mm^2

The program system ANSYS® 7 used for the numerical investigations allows to integrate user developed elements or material routines by means of a program interface. Because the typical behaviour of masonry under shear is essentially characterized by joint failure and a softening unit failure, different interface elements were implemented within the scope of the research project. In addition a material routine was developed for the available two-dimensional elements; this however shall not be further discussed in this paper.
The deformations due to shrinkage as well as the external vertical loads were applied within the first numerical load step. A preliminary investigation proved that up to the time of the test procedure approximately 20-25% of the final shrinkage of an unloaded concrete were to be registered. In this connection plastic strains of the fresh concrete are included.

The resulting stresses in the masonry remained clearly below its strengths. But in the concrete, it almost reached the tensile strength. As expected, the intensity of the shrinkage influences the prestressing of the masonry and with it the shear load capacity. The

horizontal shrinkage of the upper beam also induces extra shear stresses in the masonry. This leads to a reduction of the load-carrying capacity. The additional vertical load as a result of the shrinkage amounted for the masonry approximately 86 kN.
The following picture shows a typical numerical crack pattern. Compared with unconfined masonry the cracks are fanned out clearly wide and also the stress distribution in the masonry is more homogeneous. In Figure 4: is to be seen that the first cracks goes diagonally through the masonry and both resulting wall halves are held together by the frame. With the most unfavorable estimation that both wall halves take the same portion of the shear load a shear action arises for the frame by the half height of the external shear load.
The joint failure could be examined only numerically, because the bond strength of the used AAC-unit is greater than the tensile strength of the units. The failure type varies depending on the unit geometry, the vertical load and the relation of tensile bond strength to the initial shear strength. The gaping shown on the right in Figure 8: must not lead to failure in the case of confined masonry, but rather to an additional load of the frame.

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