Experimental simulation of seismic response of masonry walls

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SUMMARY:
In this paper, the results of an experimental programme aimed at analyzing different types of testing in plane seismic response of masonry walls, are presented. A series of six identical masonry walls with dimensions 100/100/30 cm (length/height/thickness), built from modern hollow clay masonry units and thin layer mortar in bed joints was tested by subjecting the walls to cyclic lateral load. Walls were tested using three different boundary conditions. The first one is cantilever type with only prescribed lateral displacements at the free end. The second one has fixed rotations and vertical displacements at both ends, while the third one has fixed rotations at both ends and fixed (constant) vertical compressive load. The response and failure mechanism as well as limit states, deformation and resistance capacities are presented and compared.

Keywords: cyclic shear test, boundary conditions, test set-up, masonry, seismic resistance

1. INTRODUCTION

There are a number of differences between laboratory tests of seismic resistance of masonry and earthquake damage of masonry buildings. Similar differences can also be observed, when comparing laboratory tests with in-situ shear tests of masonry walls. The most obvious difference is almost complete lack of rocking and rocking related damage in structures and in-situ tested walls, whereas considerable rocking and toe crushing is sometimes observed in laboratory tests (Figure 1). When designing the test setup, two parameters are crucial for representative simulation of seismic response: boundary conditions and compressive force.

In terms of boundary conditions simulated in the laboratory, two types are most commonly used: the cantilever type and the fixed-fixed (or symmetrically fixed) type. The term symmetrically-fixed applies to rotations, because one end of the wall must be free to move in horizontal and vertical directions. The other end is completely fixed, usually to the laboratory floor. Cyclic shear test using cantilever type of boundary conditions is easier to perform then the fixed-fixed type and because almost identical results are obtained for many parameters in many cases (Bernardini et al. 1980) this type of testing is often used. In some cases, however, depending on the masonry type, quality of the mortar, dimensions of the wall and also the level of compressive stress, there are significant differences, and fixed-fixed boundary conditions are preferred (compare Figs. 1 and 2).

Figure 1. Typical earthquake damage of masonry building and in-situ shear test

Many different test setups, which enable testing walls under symmetrically fixed conditions, can be found in literature (van Vilet 2004, Frumento *et al*. 2009), and the earliest such testing was performed already in the seventies (Turnšek and Sheppard, 1980). They can be divided into two groups, based on the way rotations are constrained. Test setups of the first group use strong steel mechanical mechanism to prevent rotations at the free end. The other group relies on hydraulic actuators with regulation algorithm to achieve the same. In most cases walls are tested at constant level of vertical force, but there are also cases with fixed vertical displacement (Vermeltfoort and Raijmakers, 1993).

Figure 2. Laboratory tests

The second parameter of interest in this research is the compressive force in the wall during testing. Even though this force is not constant in a real building in an event of earthquake, the actual change is unknown. Large majority of tests are therefore performed at constant vertical force, but the level of this force is somewhat different between laboratories. Due to the fact, that usually only a few walls are tested, the test is performed at the highest level of compressive stress allowed in the building wall, which is usually between 20 % and 30 % of average compressive strength (Tomaževič and Gams, 2009).

In this paper we attempt to analyze the effect of different boundary conditions on “modern” masonry. By modern we are referring to masonry constructed of modern porous clay masonry units with grind surfaces with thin layer mortar in bed joints and with unfilled head joints. Since such products and building techniques have not been around long enough to experience major earthquakes, the research provides insight into response of such walls as well as into the main research focus of this paper, which is the analysis of different test setups.

2. MATERIALS AND METHODS

2.1. Walls

Six masonry walls with dimensions 100/100/30 cm (length/height/thickness) were built on for the purpose prepared r.c. foundation blocks. On top of the walls, r.c. bond beam was constructed for application of compressive (vertical) and horizontal loads. Clay masonry units with ground bed joint surface and thin layer mortar were used in the construction. Walls were built with mortar in bed joints, while the head joints were left un-filled.

Masonry units used in the construction had dimensions 25/25/30 cm (length/height/thickness). They have vertical holes in the amount of about 48 % of gross volume, and classify as Group 2 masonry units according to EC 6-1 (>25 % and ≤55 % of holes). Compressive strength of masonry units (12.1 MPa) was determined by testing. As already mentioned, clay units have ground head and bed surfaces specifically for the use with thin layer mortar.

The thickness of thin layer mortar was about 1mm. Compressive strength of the mortar, determined by testing samples (7.07/7.07/7.07 cm^3 cubes) taken during construction was 9.9 MPa.

Compressive strength of walls was determined by testing three samples according to the European standard EN 1052-1. Obtained average compressive strength and elastic modulus were 6.7 MPa and 5600 MPa, respectively. The layout of the walls and the test setup are shown in Figure 3.

2.2. Test setup

Walls in the test setup were fully fixed to the strong laboratory floor. A strong steel girder was placed on top of the bond beam, which was connected to two servo hydraulic actuators at both ends. The steel beam and the r.c. bond beam were strongly connected by bolts, preventing relative rotations between them. By using the different regulation for the actuators, one of three boundary conditions could be simulated: a) cantilever type boundary condition, b) fixed rotations and constant vertical displacements and c) fixed rotations and constant vertical force. The test setup is shown in Figure 3.

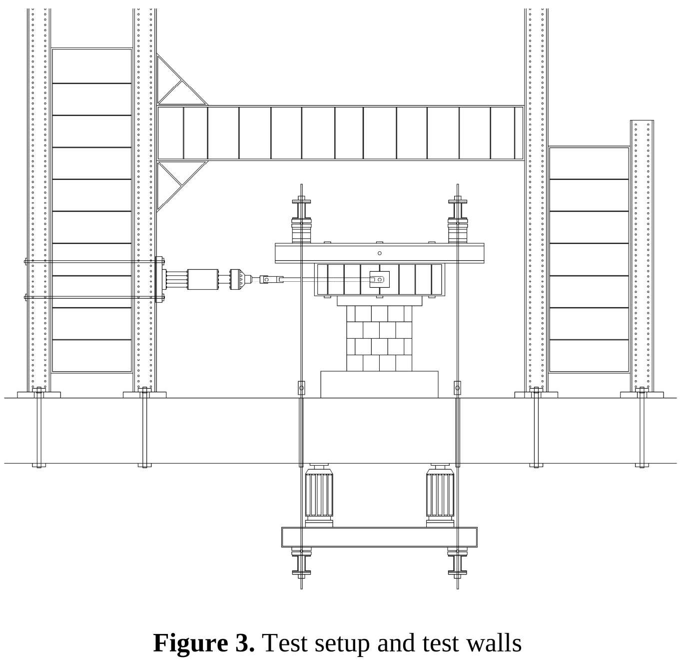

Figure 3. Test setup and test walls

2.3. Program of testing

To investigate the effect of different boundary conditions, six identical walls were tested. The first two walls were tested as cantilevers. The compressive load levels used in tests were 10 % and 20 % of compressive strength, respectively. The next two walls were tested by preventing rotations at the free end, and also by preventing vertical displacements. Hence the bond beam could only move horizontally in a straight line. The last two walls were tested by fixing the rotations and maintaining constant level of vertical force. Test matrix is presented in Table 2.1

In all cases, the desired vertical load was applied first, divided equally between the hydraulic actuators. Once this was completed, the regulation loop was turned on and the rotation was fixed at the current value. The absolute rotation of the top of the wall was not zero at this stage, but this rotation was small.

Once the regulation loop was active, cyclic lateral displacements with step-wise increased amplitudes, repeated three times at each displacement peak, have been used to simulate the in-plane lateral seismic loads.

Table 2.1. Test program

Wall	Vertical stress level [%]	Fixed rotations	Fixed vertical displacements	Fixed vertical force
Wall 1	20	-	-	-
Wall 2	10	-	-	-
Wall 3	20	Yes	Yes	-
Wall 4	10	Yes	Yes	-
Wall 5	20	Yes	-	Yes
Wall 6	10	Yes	-	Yes

3. RESULTS AND DISCUSSION

3.1. Observed response

Wall 2, which was tested as cantilever at low level of precompression exhibited extensive rocking, which is evident from the “S” shaped hysteresis in Figure 4. Under the same boundary conditions, but with higher level of precompression (wall 1), rocking was much less evident, but still the response shows a tendency towards the “S”, indicating some rocking. The effect of increased vertical force F on lateral resistance H is very pronounced. The lateral resistance of more compressed wall 1 was 122 kN, compared to 79 kN of wall 2.

In case of walls 3 and 4, which fix the vertical displacement at the top of the wall, rocking is naturally prevented by the boundary conditions. This is confirmed by the response curves, which do not show even the slightest tendency towards the “S” shaped response curve. The drawback of such testing is that vertical force is not constant. In fact the vertical force increased high above the initial value, which was 406 kN for wall 3 and 203 kN for wall 4. The maximum attained vertical force was 480 and 450 kN for walls 3 and 4, respectively. This in turn influenced maximum lateral resistance, which was almost identical in both cases. By the end of testing, vertical force dropped to practically zero.

Interestingly, there is a difference in response of walls 5 and 6. Response of wall 6, which was tested at the lower compressive level, shows some rocking behaviour, while wall 5 does not. This clearly shows that fixing the rotations alone is not enough to prevent rocking.

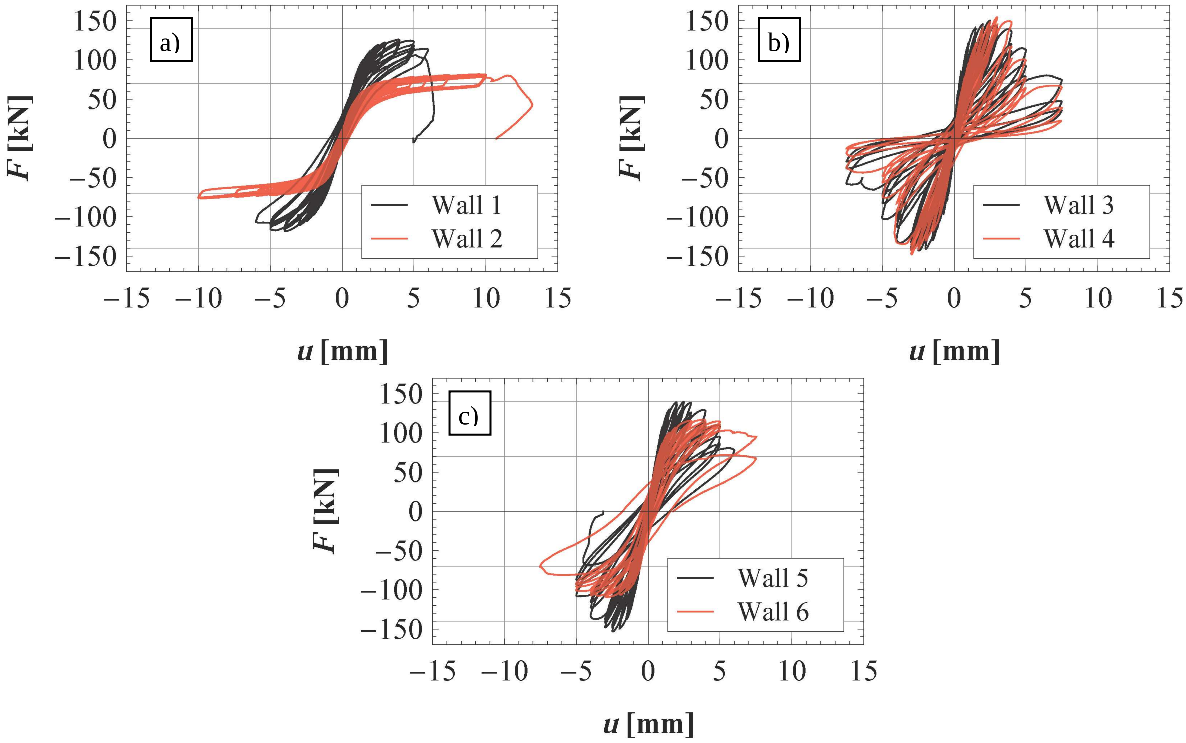

Figure 4. The effect of the level of precompression for a) cantilever, b) fixed rotations and vertical displacements and c) fixed rotations and vertical load. Hysteretic response of walls at 20 % precompression is drawn in black, of walls at 10 % in red

To objectively compare the three approaches, resistance and displacement capacities at damage, maximum resistance and collapse limit states are compared in Table 3.1. Deformation and resistance capacities are presented in Table 3.2. The data in these tables along with plots of resistance envelopes in Figure 5 show, that the type of test setup has a significant effect on results if walls are tested at low levels of precompression and rocking of the wall develops. In case rocking does not develop then the differences between different test-setups is quite small. As this research clearly shows, the conditions of testing (boundary and precompression level) have a significant effect on the response, but it must be noted, that the type of masonry should also be considered. In case of walls with normal thickness mortar and weak mortar, rocking is much less likely to occur than if mortar is very strong and thin.

Table 3.1. Limit states

Wall	Crack/damage			Max resistance			Collapse		
	u [mm]	*H* [kN]	*Φ* [%]	*u* [mm]	*H* [kN]	*Φ* [%]	*u* [mm]	*H* [kN]	*Φ* [%]
Wall 1	1.50	92.5	0.14	3.93	122.2	0.37	5.00	106.2	0.47
Wall 2	1.00	50.5	0.09	9.62	79.1	0.89	9.62	79.1	0.89
Wall 3	0.75	98.8	0.07	2.45	146.5	0.23	7.49	29.2	0.70
Wall 4	1.00	96.6	0.09	2.96	151.1	0.28	7.47	12.8	0.71
Wall 5	2.00	144.2	0.19	2.45	146.4	0.23	4.51	63.2	0.42
Wall 6	1.00	89.2	0.09	3.35	113.1	0.31	7.50	67.1	0.70

Table 3.2. Deformation capacity

	Crack/damage		Collapse	
	H/H_{max}	$\Phi\ /\ \Phi_{max}$	H/H_{max}	$\Phi\ /\ \Phi_{max}$
Wall 1	0.76	0.4	0.87	1.27
Wall 2	0.64	0.10	1.00	1.00
Wall 3	0.67	0.31	0.20	3.05
Wall 4	0.64	0.34	0.08	2.52
Wall 5	0.98	0.8	0.43	1.84
Wall 6	0.79	0.3	0.59	2.24

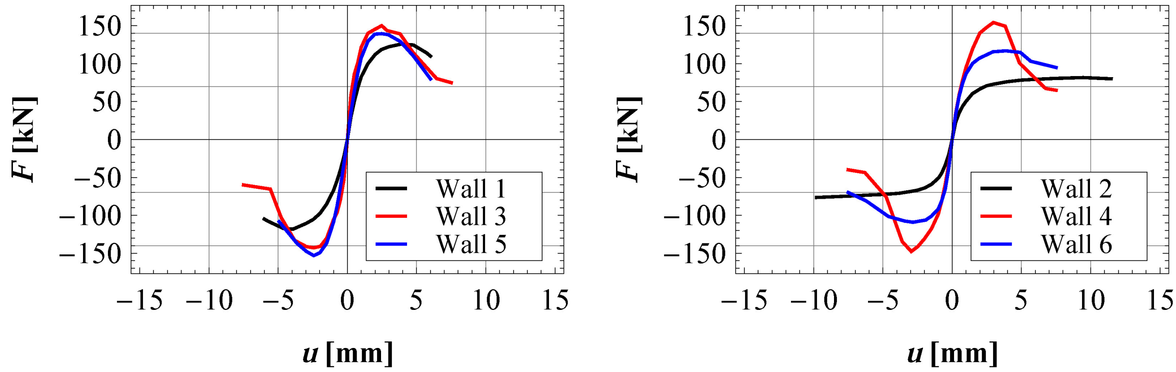

Figure 5. Resistance envelopes

3.2. Failure mechanisms

An interesting observation can be made, if damage patterns of wall 2 and wall 4 are compared. Wall 4 is clearly a diagonal shear type of collapse, but this is not so clear for wall 2, as Figure 6 demonstrates. Despite the fact, that the damage pattern is not so clear for wall 2, it is still a shear type of failure. The crack pattern follows head and bed joints in addition to some inclined cracking of the units. Significant damage to the toes has a negative influence on lateral resistance, but is not the overall reason for collapse. All of the tested walls ultimately failed in shear.

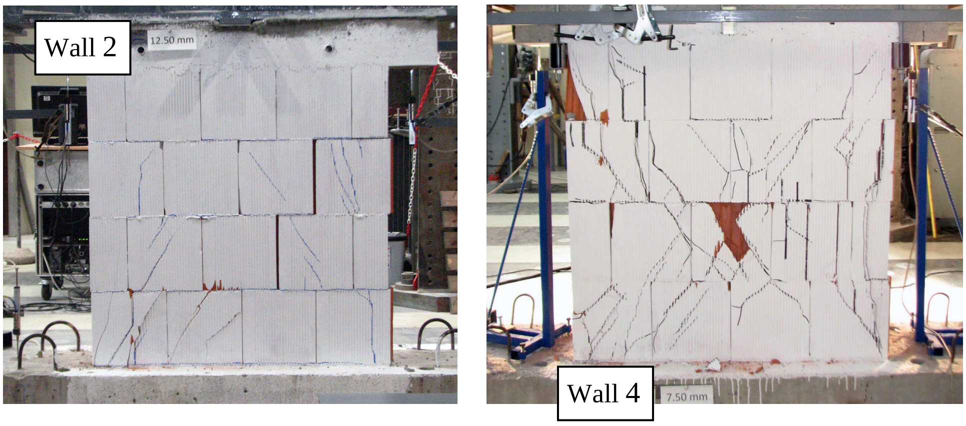

Figure 6. Collapse of wall 2 (left) and wall 4 (right)

An optical system was used to measure displacements over the entire surface of the wall, and these results can be used to obtain major strain fields over the wall. Major strain fields at maximum resistance limit state for walls 2, 4 and 6 are presented in Figure 7. Exclusive cracking of bed joints and opening of head joints indicates almost perfect rocking of units in wall 2. Wall 4, on the other hand, shows several parallel shear cracks, which are surely influenced by unfilled head joints, but cracking of the units is extensive. Situation in wall 4 is between the extremes – there is some rocking, but also some shear damage.

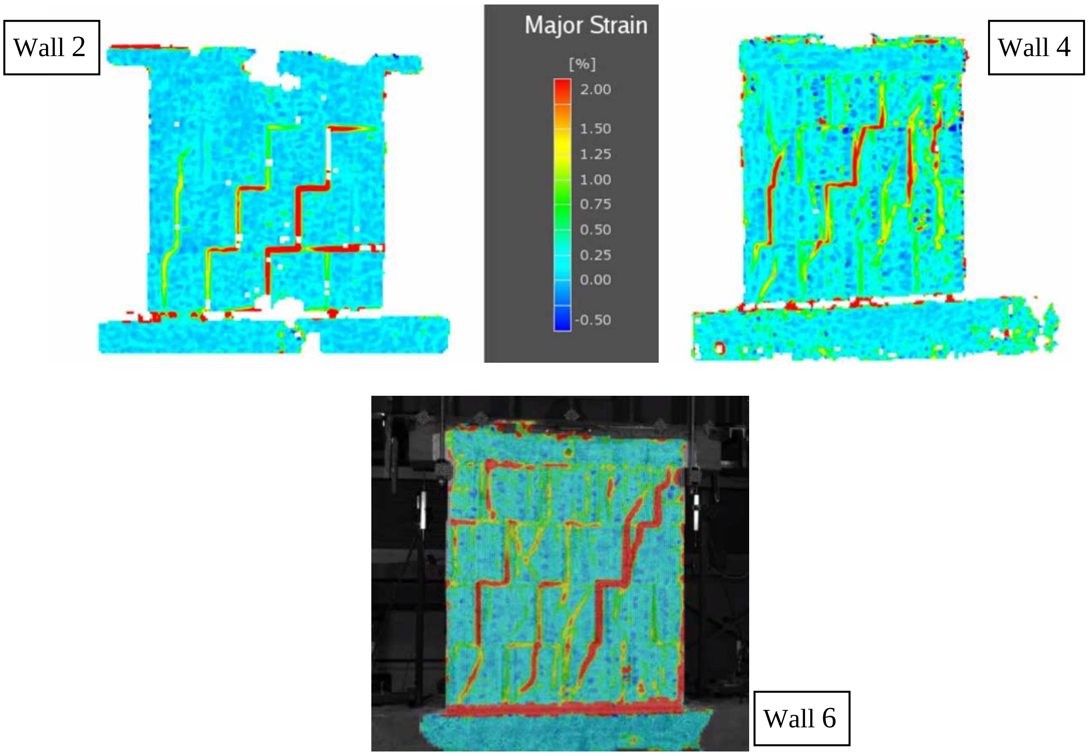

Figure 7. Major strain fields for walls 2 (top left), 4 (top right), and 6 (bottom center)

4. CONCLUSIONS

Three types of boundary conditions for testing masonry walls in cyclic shear have been compared by experiments. The first type is the cantilever type with one free end, the second type is with fixed rotations, prescribed vertical displacement and variable vertical force, and the last one is with fixed rotations and prescribed (constant) vertical force. Two walls were tested in each test setup at different levels of precompression.

The results show, that under certain conditions, the walls exhibit rocking behaviour. Two types of rocking were observed: rocking of entire wall as a rigid body and rocking of individual units within the masonry wall. The biggest difference in response between walls with rocking and without is the shape of the lateral displacement – resistance curve. If rocking is present, the curve starts to form an "S" shape, whereas nothing like it can be observed if there is no rocking.

Results conclusively show that the higher the precompression, the less likely rocking will develop and that rocking can develop even with symmetrically fixed boundary conditions at constant vertical force. Collapse mechanism of walls with rocking is often due to shear and not necessarily due to toe crushing.

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