Quasi-static cyclic tests of two prefabricated, reinforced masonry walls

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Ohrid
30.08-03.09
14 ECEE 2010

ABSTRACT:
In the second half of the 20th century, the majority of residential buildings in Switzerland have been built with unreinforced brick masonry walls and reinforced concrete floors. Following a re-evaluation of the seismic hazard in Switzerland, a country of moderate seismicity, the seismic design spectra have increased in the last revision (2003) of the Swiss building code. As a consequence, it has become very difficult to justify the use of unreinforced masonry walls as sufficient seismic resistance elements. In many new residential buildings, structural walls are constructed as reinforced concrete walls instead of unreinforced masonry walls although the latter provide better thermal and insulation properties at a lower cost. Reinforced masonry systems, which could constitute a viable alternative to reinforced concrete walls, are rarely used within Switzerland. The objective of this research project was therefore to develop a reinforced masonry system unlike most reinforced masonry systems that are available on the international market today. It was decided to develop a prefabricated wall system that would require less construction time and allow an easier introduction to the Swiss market. The most critical construction detail is the connection between the prefabricated wall and the foundation. In a preliminary study two different connection details were developed and their functionality tested by means of quasi-static cyclic tests of full-scale wall elements. This paper summarises the details of the system, the test setup, the instrumentation and the test results. The tests were able to show that the proposed systems achieve similar displacement ductilities as ductile reinforced concrete walls. Finally, recommendations for further improvement of the suggested prefabricated reinforced masonry wall system are made.

Keywords: Reinforced masonry, structural wall, quasi-static cyclic test

1. INTRODUCTION

In the past the majority of residential buildings in Switzerland have been constructed as unreinforced masonry (URM) buildings with reinforced concrete (RC) slabs. With the last revision of the Swiss building codes in 2003 the seismic design spectra have increased. Despite the moderate seismicity in Switzerland, it became very difficult with today's design and modelling approaches to satisfy seismic design requirements for URM buildings with 3 to 6 stories. In the current design practice it is therefore customary to replace some URM walls with RC walls. Only those URM walls that are required to fulfil the seismic design requirements are replaced by RC walls in a typical design. The seismic design is then based on the assumption that all lateral loads are carried by the RC walls alone and that the URM walls only carry vertical loads. This design trend was established despite the fact that masonry offers better thermal and insulation properties at a lower cost for residential buildings. The objective of this work was therefore to develop a reinforced masonry (RM) system, which could be a valuable alternative to common RC walls.

RM walls are scarcely used within Switzerland. Contrarily to most RM systems that are available on the international market, it was decided to develop a prefabricated system. Such a system has the

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advantage of ensuring easy quality control and faster construction. The planned production chain is as follows: The walls are manufactured in a precast company in one-story pieces, transported to the construction site, placed by a crane in the right position and finally connected to the foundation or to the wall of the adjacent floor. This construction method allows the construction time to be minimized, and in addition, eliminates the need for RC wall construction tools such as formwork panels. It is planned that within a building, only those walls that are necessary to carry the seismically induced horizontal loads shall be constructed as reinforced masonry walls. All other walls will be constructed as URM walls. The resulting structure is therefore a mixed system of reinforced and unreinforced masonry walls. The objective of two master's theses carried out at the Institute of Structural Engineering at the ETH Zürich was to investigate different design approaches for the reinforced masonry walls and to carry out a first study on the seismic behaviour of such mixed building systems. This paper, however, addresses only the quasi-static cyclic tests of two ductile prefabricated reinforced masonry walls that were tested as part of the this thesis work. They were developed in collaboration with Keller Ziegeleien AG, a brickwork company in Switzerland.

2. REINFORCED MASONRY WALL SYSTEMS

The reinforced masonry walls were constructed with hollow bricks from the UNIPRETON masonry system. The hollow brick type used and the construction of such a wall are shown in Figs. 1a and b respectively. The wall was constructed as a dry wall, i.e. without mortar. Each brick consists of two cells which form a continuous cavity over the entire height of the wall. Each cell accommodates (i) a reinforcing cage spanning the entire height of the wall, (ii) a duct placed in the bottom part of the wall, and (iii) a starter bar sticking out from the top of the wall. Horizontal reinforcement is inserted in each bed joint. As a final step the wall is filled with self-compacting concrete. When placing the wall on construction site, the starter bars of the foundation or of the wall of the adjacent floor are inserted into the ducts. The latter are then filled with mortar ensuring continuity of the flexural reinforcement.

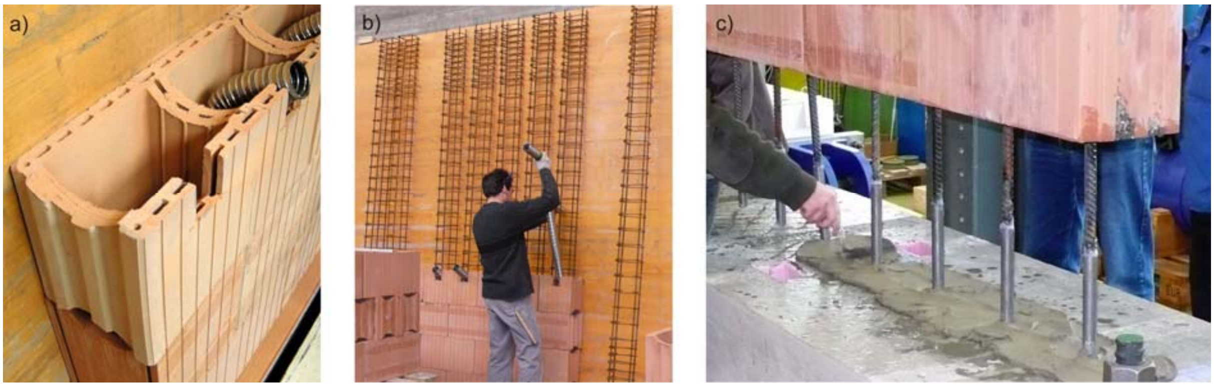

Figure 1. Hollow brick from the UNIPRETON system (a), construction of test units in the precast shop (b) and connection detail when assembling Test Unit BMW1 in the laboratory (c).

Since the bricks are dry stacked, the horizontal reinforcement is not included in the bed joints but is inserted in recesses in the bricks (see Figs. 1a and b), which can accommodate continuous horizontal reinforcement over the entire wall length. At each row of bricks 4 D8 mm bars are inserted into the recesses (see Fig. 2). The concrete columns resulting from the filling of the vertical cells are continuous over the entire wall height and are interconnected by the concrete filled recesses. The resulting reinforced concrete structure is then a grid with strong vertical elements and weaker horizontal elements. Due to the continuity of the concrete in the vertical and horizontal directions in the wall, and its relatively large contribution to the gross sectional area (see Fig. 2), the behaviour of the structural element is dominated by the reinforced concrete rather than the masonry. This has been accounted for when predicting the force-deformation behaviour of the tested walls (see Section 2.4). Concerning the vertical reinforcement in the wall and the starter bars, two different systems are possible: If the vertical wall reinforcement is given, stronger or weaker starter bars can be chosen. The following paragraphs describe the reinforcing layout of the two test units. The starter bars of the first test unit (BMW1) were weaker than the longitudinal reinforcement of the wall, and vice-versa for the second test unit (BMW2).

2.1. Test Unit BMW1: Weak connection

For the first test unit BMW1 the starter bars consisted of a single D18 mm bar per cell and had therefore a smaller cross section than the longitudinal wall reinforcement containing 4 D10 mm bars per cell (Fig. 2a). Figure 2a also shows a qualitative sketch of the bending moment diagram due to a lateral force acting at the top of the wall compared to the flexural strength available along the height of the wall. Based on this comparison, the critical section, i.e. the section where the onset of yielding was expected, was located at the base of the wall. The wall was placed onto a mortar bed on the foundation. The starter bars of the foundation block protruded 750 mm into the wall. Tubular metal sleeves of 200 mm length were slid over each starter bar to prevent bond between the starter bar and the surrounding mortar. This measure elongated artificially the plastic hinge length and therefore increased the displacement capacity of the wall. Fig. 1c shows this connection detail when assembling the test unit in the laboratory.

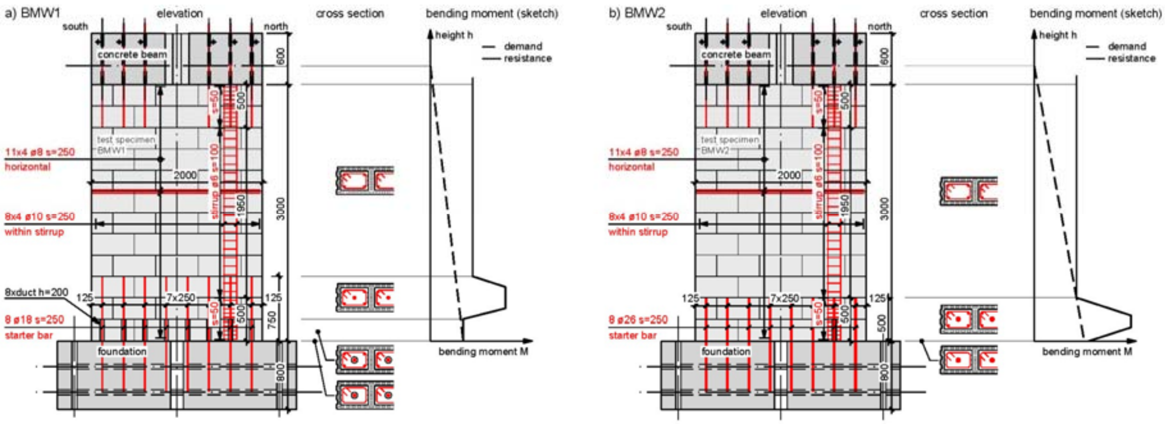

Figure 2. Elevation, cross section and sketch of bending moment demand and resistance for Test Units BMW1 (a) and BMW2 (b).

2.2. Test Unit BMW2: Strong connection

The only difference between BMW1 and BMW2 concerned the starter bars. The starter bars of BMW2 consisted of one D26 mm bar per cell and had hence a larger cross section than the longitudinal wall reinforcement (4 D10 mm bars per cell as for BMW1). The wall was again placed onto a mortar bed on the foundation. The length of the starter bars outside the foundation was 500 mm. Metal sleeves were not mounted since yielding of the starter bars was not expected. The reinforcement layout and the qualitative bending moment diagrams over the height of the wall are shown in Fig. 2b.

2.3. Material properties

Material tests on concrete, mortar and reinforcing bar samples were carried out according to the Swiss design code SIA 162/2 (1989). The most important results for the reinforcing bars are summarised in the left part of Table 1. These were: the E-Modulus E_s, the yield strength f_y, the ultimate strength f_t and the strain capacity A_{gt} at f_t. Both the dynamic and static values are given for the yield strength and the ultimate strength. The dynamic strength corresponds to the value measured during loading. To obtain the corresponding static values the loading was stopped for two minutes each at strains of 0.5% and 2.0% and the drop in strength of the bar was measured.

The reinforced masonry walls were cast with a self-compacting concrete (SCC) with a maximum aggregate size of 8 mm. The small aggregate size was chosen due to the close mesh of the reinforcing

Table 1. Mechanical properties of the reinforcing bars and mechanical properties of the employed concretes and mortars (mean values and standard deviations).

Reinforcing bars				
$Ø_{nom}$ [mm]	Ø8 8 bars	Ø10 6 bars	Ø18 6 bars	Ø26 5 bars
E_s [GPa]	215 ± 14.2	205 ± 1.2	203 ± 3.6	208 ± 12.4
$f_{y,dyn}$ [MPa]	510 ± 30.1	492 ± 5.1	536 ± 7.4	559 ± 6.5
$f_{t,dyn}$ [MPa]	599 ± 35.5	565 ± 6.3	636 ± 5.5	656 ± 5.8
$f_{y,stat}$ [MPa]	471 ± 32.6	457 ± 8.4	504 ± 12.2	528 ± 6.0
$f_{t,stat}$ [MPa]	548 ± 34.9	520 ± 10.7	584 ± 10.7	599 ± 5.9
A_{gt} [%]	7.99 ± 1.05	6.97 ± 0.76	9.27 ± 0.45	9.53 ± 0.10

Concrete and mortar					
	Wall BMW1	Wall BMW2	Ducts BMW1	Ducts BMW2	Mortar bed BMW2
Type	SCC 0-8 mm		SikaGrout-314		KELIT110
Age [d]	26	68	19	12	12
ρ [kg/m³]	2'370 ± 7.0	2'400 ± 4.6	2'200 ± 13.6	2'230 ± 7.8	1'690 ± 35.5
$f_{cm,cube}$ [MPa]	61.3 ± 1.4	79.0 ± 1.6	67.2 ± 1.6	77.5 ± 1.1	15.7 ± 1.2
f_{cm} [MPa]	52.0 ± 0.1	69.9 ± 0.8	-	-	-
f_{ctm} [MPa]	3.80 ± 0.25	5.81 ± 0.14	9.8 ± 1.18	6.1 ± 0.4	3.9 ± 0.5
E_{cm} [GPa]	33.3 ± 0.09	37.8 ± 0.34	-	-	-
ε_{c2m} [‰]	2.0 ± 0.1	2.3 ± 0.1	-	-	-

cage. The ducts for the starter bars were filled with the high-strength grout SikaGrout-314. For the mortar bed between foundation and wall a normal masonry mortar KELIT 110 was used. For each wall, concrete samples in the form of three cubes and five cylinders were taken. Two of the cylinders were halved to perform double-punch tests (Chen, 1970) to determine the tensile strength of the concrete. Three prisms were cast for each batch of mortar. Table 1 lists the material properties of the different types of concrete and mortar used. For all types the density ρ, the cube strength $f_{cm,cube}$ and the tensile strength f_{ctm} were determined. The latter was determined by means of the double-punch tests for the concrete samples and by 3-point-bending tests for the mortar samples. For the concrete also the cylinder strength f_{cm}, the E-modulus E_{cm} and the strain e_{c2m} at f_{cm} were determined.

2.4. Prediction of the force-deformation characteristic

For each of the two test units, the prediction of the force-deformation characteristic was performed in two steps: In a first step the failure mechanism was determined, and in a second step the force-deformation characteristic was computed for this failure mechanism.

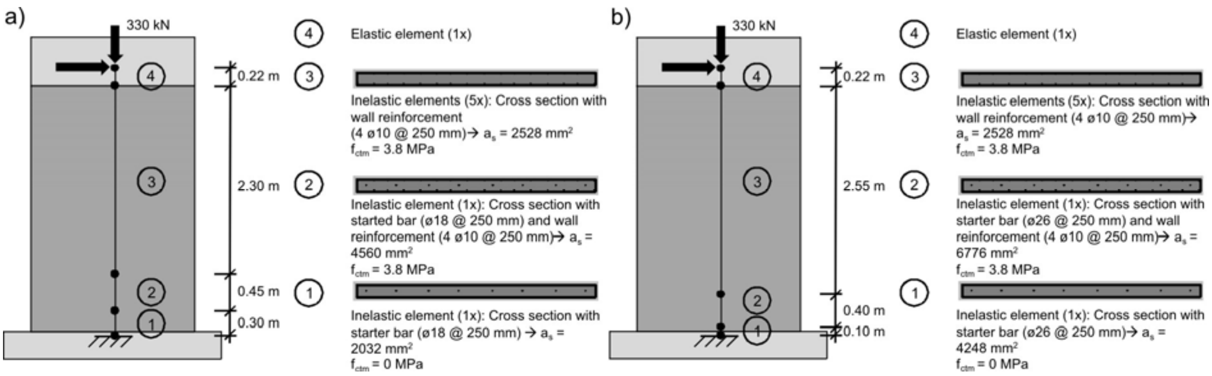

Figure 3. Numerical model for computing the force-deformation relationship of BMW1 (a) and BMW2 (b).

The design objective was to exclude shear failure by providing sufficient horizontal reinforcement. The shear capacity was determined according to the Swiss concrete design code SIA 262 (2003). The sliding shear capacity was also checked according to SIA 262 (2003). The formation of a flexural hinge at the base of the wall was anticipated for the first test unit BMW1 because the starter bars were weaker than the longitudinal reinforcement of the wall (see Fig. 2a). A flexural failure within the wall itself and above the starter bars was expected for the second test unit BMW2 (Fig. 2b).

The force-displacement relationship was computed by means of the finite element program Seismostruct (Seismosoft, 2009). The walls were modelled as beams whose cross sections varied over the height of the wall according to the variation of the longitudinal reinforcement (Fig. 3). Concrete cross-sections were defined with dimensions equal to the effective dimensions of the reinforced masonry wall specimen's sections. The outer masonry shell was neglected and the inner masonry separating the cells of concrete was treated as concrete. The load distribution beam at the top of the wall was modelled as an elastic element. The computations are based on the assumption that plane sections remain plane. Mean values from the material tests were used for the material properties (see Section 2.3). The force-deformation relationships of the two cantilever systems as well as the bilinear approximation of the two predictions are plotted in Fig. 4a. The horizontal force F_y' corresponds to the onset of yielding of the first longitudinal reinforcing bar.

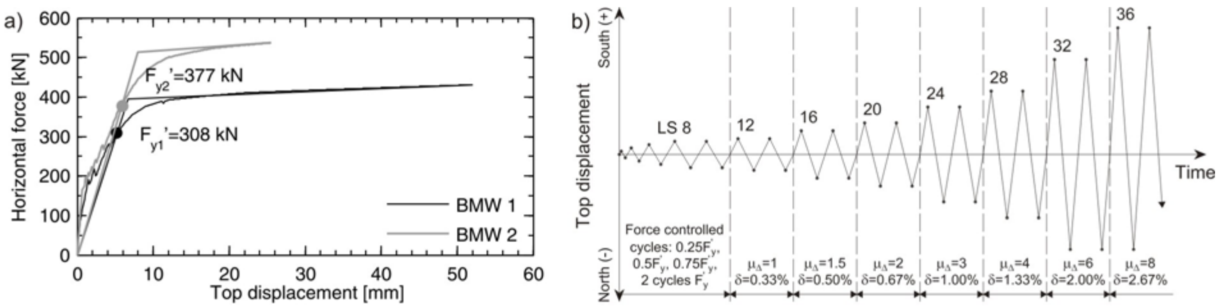

Figure 4. Predicted force-displacement relationships (a) and applied loading history (b).

3. TEST SETUP AND LOADING HISTORY

It was concluded from the analysis of typical Swiss residential buildings with 3-stories that a wall slenderness ratio of 1.6 should be chosen for the test units. The size of the test units was limited by the boundary conditions of the laboratory setup and the need to use the full-size hollow bricks. The wall length was hence chosen as 2.00 m, the wall height as 3.00 m and the wall width as 0.20 m. The horizontal force was applied to the load distribution beam at a height of 3.22 m above the base of the wall. An axial force corresponding to a normal stress of 0.83 MPa at the base of the wall was reached by means of the self-weight of the test unit and by an externally applied prestressing force.

3.1. Test setup

The test setup is shown in Fig. 5a. Its basic components are (i) the reaction wall, (ii) the actuator and (iii) the test unit whose foundation was prestressed onto the strong floor. For applying the vertical and horizontal loads to the wall, a load distribution beam was mounted on top of the wall. At the front and back side of the beam two steel plates were mounted and tensioned together with a total force of 900 kN by means of ten BBRV threaded steel bars. The hydraulic actuator was connected at the North face of the beam. The axial force was applied by means of two BBRV prestressing cables running along the East and West faces of the test unit. The cables were prestressed by means of hollow core jacks, which were placed on top of the test unit. Shear keys at the North and South ends of the foundation prevented sliding of the latter on the strong floor. A frictionless lateral support system was mounted at the height of the load distribution beam to avoid out-of-plane movements of the wall.

3.2. Instrumentation

Thirteen linear variable differential transformers (LVDTs) were installed as shown in Fig. 5b (labels "u" and "v") in order to record the wall deformations Additionally, an optical measurement system

(NDI Optotrak Certus HD) was used to measure the deformation of the wall in more detail. To do so, 10 LEDs were glued onto each row of bricks. Each brick was fit with two LEDs and one additional LED was fixed at each end of the wall. Ten additional LEDs were glued onto the foundation in order to measure the rocking and sliding of the wall on the foundation. In total the x-, y-, and z-coordinates of 120 LEDs were recorded. The applied horizontal force and the axial forces in the BBRV prestressing cables were measured by means of three load cells (label "F" in Fig. 5b). The measurement frequencies of the hard-wired instruments (LVDTs and load cells) and of the optical system were 2 Hz and 10 Hz, respectively.

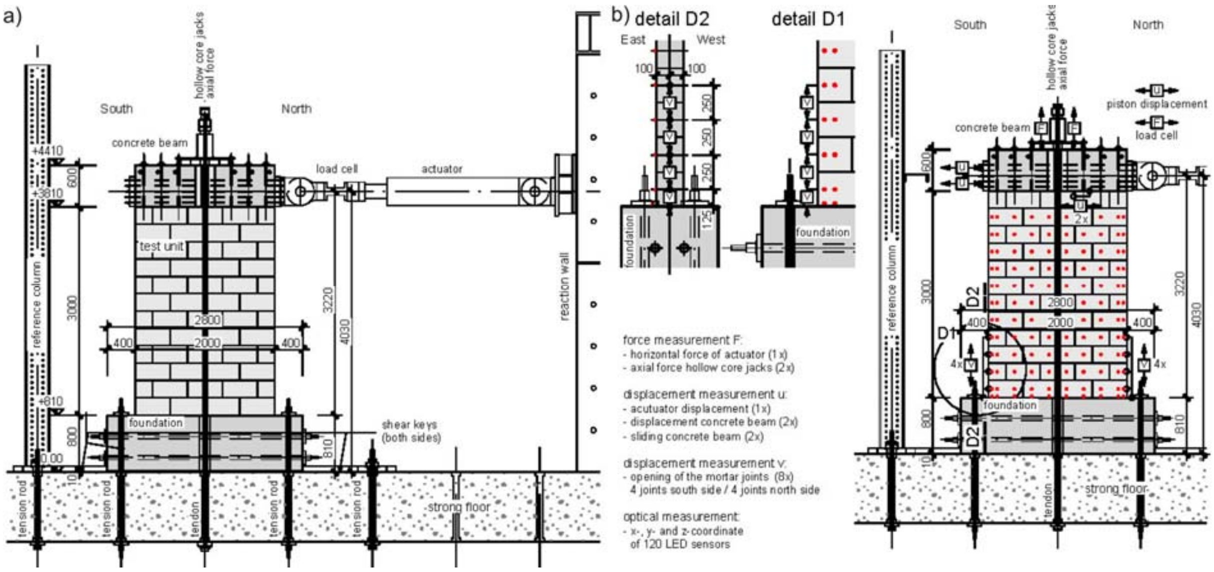

Figure 5. Test setup (a, Note: To improve the readability of the drawing the lateral support system to restrain the test unit from excessive out-of-plane movement is not shown) and instrumentation (b).

3.3. Loading history

The applied loading history is shown in Fig. 4b and corresponds to the standard loading history for reinforced concrete walls. The test duration was two days for BMW1 and one day for BMW2. Zero measurements were taken at LS00 before the axial load was applied (LS01). In a next step the hydraulic actuator was connected to the wall. The following load cycles were separated in force and displacement cycles. In the first three cycles, the wall was displaced until a horizontal force of $\pm^1/_4\ F_y$‘ , $\pm^2/_4\ F_y$‘ and $\pm^3/_4\ F_y$‘, respectively, was reached. At $^4/_4\ F_y$‘ two cycles were carried out, the measured wall top peak displacement Δ_y’ averaged over the four peaks and the nominal yield displacement Δ_y determined as Δ_y’F_y/F_y‘. The values for F_y‘ and F_y were taken from the prediction of the force-deformation relationship (Fig. 4a). The rest of the test was conducted in displacement control, starting at displacement ductility μ_Δ=1. The target displacement ductility was increased stepwise according to Fig 4b. For each ductility step, two loading cycles were carried out. The velocity of the actuator was increased continuously during the test from 1.2 mm/min to 12 mm/min.

For BMW2, the first yield force F_y‘ was initially computed incorrectly and was larger than the actual force F_y‘. For this reason, during the cycles with $^4/_4\ F_y$‘ the actual yield force as well as the actual yield displacement had already been significantly exceeded. As soon as the mistake was noticed, the computation of the yield force was corrected, and to compensate for the larger cycles already carried out it was decided to skip the cycles with $\mu_\Delta = 1.0$ and 1.5 and to continue directly with the cycles corresponding to $\mu_\Delta = 2.0$. The yield displacement of the two test units was of similar magnitude. For this reason for both test units a top displacement of 10.7 mm was defined as $\mu_\Delta = 1.0$, which corresponds to an average drift of $\delta = 0.33\%$.

4. EXPERIMENTAL RESULTS

4.1. Failure mechanism

For both walls the failure mechanism was predicted correctly. BMW1 developed a flexural mechanism where the starter bars yielded. BMW2 developed also a flexural mechanism but the hinge was located above the strong starter bars. At the South end the flexural deformations were concentrated in the third bed joint and at the North end in the fourth bed joint above the foundation (Fig. 6d). The crack patterns at failure are shown in Fig. 6a and Fig. 6b. The ultimate drift ratio (BMW1: $\delta = 2.66\%$, BMW2: $\delta = 2.00\%$) was similar to that of reinforced concrete walls of similar slenderness ratios.

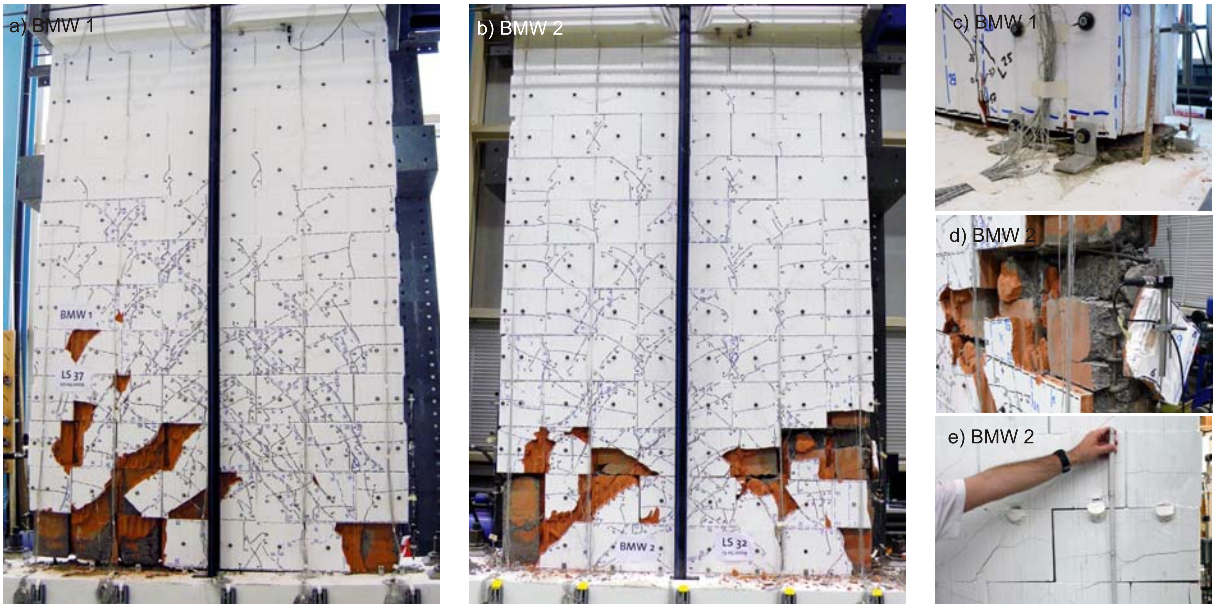

Figure 6. Photos of the test units: Final crack pattern of BMW1 (a, $\delta = 2.66\%$) and BMW2 (b, $\delta = 2.00\%$), joint between foundation and BMW1 (c, $\delta = 2.66\%$), flexural failure of BMW2 due to fracture of the longitudinal reinforcement in the fourth joint above the foundation (d, $\delta = 2.00\%$), rotation of the bricks of rows 3 and 4 of BMW2 (e, $\delta = 1.00\%$).

4.2. Force-deformation relationships

The hysteretic behaviour of the two test units is shown in Fig. 7; also included in the plots are the predicted force-displacement relationships (see Fig. 4a). The plotted top displacement was computed as the average measurement obtained from the two LVDTs that recorded the horizontal displacement of the load distribution beam. The horizontal force was measured by the load cell in the hydraulic actuator. For both walls the strength was overestimated. Analyses carried out after testing showed that a likely reason in the case of BMW1 was the weak mortar bed (Fig. 6c), which led to a larger compression zone at the wall base and therefore to a reduced lever arm of the internal forces. The crack pattern of BMW2 revealed that a diagonal compression crack had formed, which aligned with the base corner of the wall. Hence, at the third bed joint above the foundation the lever arm of the internal forces was also reduced.

5. SUMMARY AND OUTLOOK

Similar to capacity-design reinforced concrete walls, both test units developed ductile flexural hinge mechanisms and failed eventually after undergoing several cycles with large inelastic deformations ue to fracture of the starter bar or the longitudinal reinforcement. Visible damage such as spalling of the masonry shell only occurred at relatively large drifts. The experimental results hence suggest that the

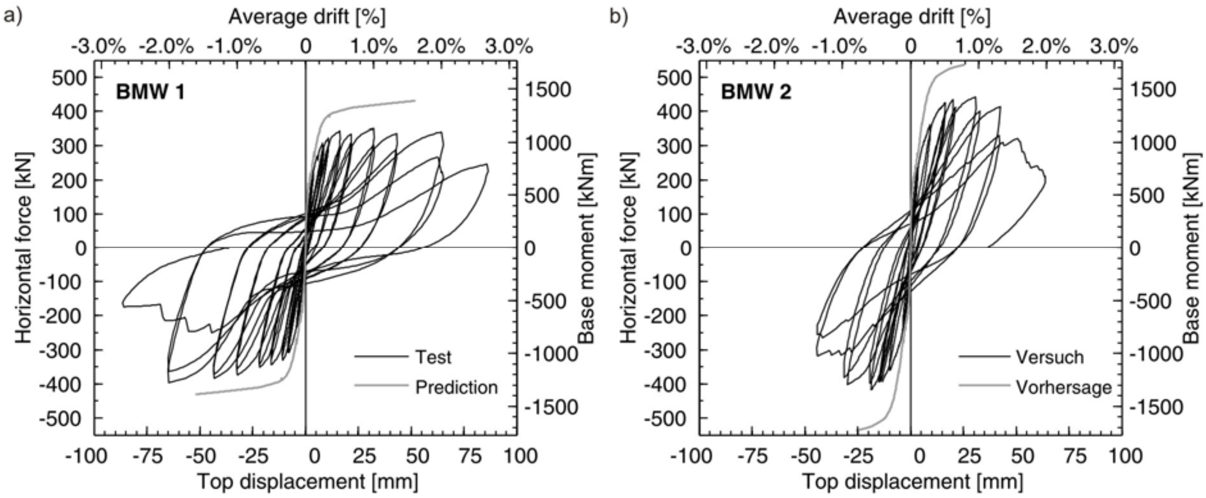

Figure 7. Hysteretic behaviour of BMW1 (a) and BMW2 (b). The drift is defined as the top displacement divided by the effective height (h = 3.22 m).

tested reinforced UNIPRETON masonry system can be designed to develop a robust seismic behaviour. Moreover, the comparison of the predicted and observed force-displacement relationships showed that both force and displacement capacity can be estimated based on approaches similar to those developed for ductile reinforced concrete walls. Test unit BMW1 had a slightly larger deformation capacity than BMW2. This is partly due to the failure of the mortar bed in the case of BMW1 which generated an additional deformation capacity. Since the inelastic deformations were concentrated in the starter bars, the wall itself suffered only small damage mainly to the masonry shell. The concrete and reinforcement of the wall was subjected almost exclusively to elastic deformations.

Based on the observed behaviour of both test units, it is recommended that this prefabricated reinforced wall system be further developed based on the connection detail of BMW1. Improvements should concern the optimisation of the design to reduce fabrication costs. Additionally, the choice of the material for the mortar bed should be revisited. Furthermore, in both tests shear failure was prevented by using a large horizontal reinforcement ratio. Hence, the results allow no inference on the shear resistance of the reinforced masonry system other than it was sufficiently large to develop the flexural mechanisms. Additional tests with smaller horizontal reinforcement leading to shear failure are therefore recommended.

ACKNOWLEDGEMENTS

The prefabricated reinforced UNIPRETON masonry system was developed in collaboration with Max Wassmer from Keller Ziegeleien AG in Pfungen (Switzerland), who also undertook the construction, transport and mounting of the walls. The tests were carried out in the laboratory of the Institute of Structural Engineering at the ETH Zürich (Switzerland) where Dominik Werne, Thomas Jaggi and Christoph Gisler supported the authors during testing. All contributions are gratefully acknowledged.

REFERENCES

Chen, W. F. (1970), Double Punch Test for Tensile Strength of Concrete, ACI Journal, 67(12), 993-995.
Seismosoft (2009), SeismoStruct – A computer program for static and dynamic nonlinear analysis of framed structures, Version 4.0.3, Seismosoft, Messina, Italy, available online at http://www.seismosoft.com.
SIA 162/1 (1989), Betonbauten: Materialprüfungen (Concrete structures: material testing), Building code, Swiss Society of Engineers and Architects (SIA), Zürich, Switzerland.
SIA 262 (2003), Betonbau (Concrete structures), Building code, Swiss Society of Engineers and Architects (SIA), Zürich, Switzerland.