THE EFFECT OF THE BOUNDARY CONDITIONS ON THE OUT-OF-PLANE BEHAVIOUR OF UNREINFORCED MASONRY WALLS

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ABSTRACT:

To prevent the out-of-plane failure of unreinforced masonry (URM) walls during earthquakes, most design codes prescribe a minimum wall thickness and a maximum slenderness ratio. These requirements are typically independent of the boundary conditions of the wall, i.e. the design guidelines do not take into account important parameters affecting the lateral stability of the walls like the level of applied axial load or the restraints of the wall. To investigate the effect of different top and bottom boundary conditions on the out-of-plane behaviour of URM walls, a series of slender wall units were tested dynamically. The test units were 2.4 m high, 1.2 m wide and had thicknesses between 12.5 and 20 cm yielding wall slenderness ratios between 19.2 and 12, respectively. Different top boundary conditions ranging from fully fixed to simply supported, as well as different levels of initial axial load were applied. The walls were tested on the shaking table of the ETH Zurich using a ground motion representing the level of shaking that can be expected on the fourth floor of a URM building located in a region of moderate seismicity. For each wall, the intensity of the ground motion was stepwise increased until the test unit collapsed. The paper presents selected test results showing that the boundary conditions can have a larger effect on the lateral stability of a URM wall than the slenderness of the wall.

KEYWORDS: Unreinforced masonry, out-of-plane behaviour, dynamic tests, boundary conditions

1. INTRODUCTION

The seismic design action defined by the Swiss design code has increased steadily over the past decades. For this reason and despite the moderate seismicity of the country, current requirements render the design of unreinforced masonry (URM) structures almost impossible. This applies even to the lower seismic zones with design ground accelerations of 10% g and less. It is widely accepted that force-based design methodologies for URM structures implemented in most current codes have the tendency to be rather conservative while displacement-based methodologies show the potential of leading to more rational and economic structures. However, before they can be applied in practice, important data e.g. on stiffness, frame-action and deformation capacity of typical Swiss URM structures need to be provided. To do so, the research project “Deformation Behaviour of Unreinforced Masonry Structures” was initiated at the ETH Zurich.

The main goal of the research project is the investigation of the in-plane behaviour of URM structural elements and structural element assemblies. However, it was soon recognized that typical Swiss URM structures are potentially very sensitive to out-of-plane actions. Statistical data provided by the masonry industry shows that about 70 to 80% of the exterior walls in single-family houses and apartment buildings are single leaf; 60% thereof are built with 15cm thick bricks while the thickness of the remaining 40% is almost always 17.5cm. Interior walls are nearly all single leaf with following thickness distributions: 12.5cm: 33%, 15cm: 43% and 17.5cm: 17%. For a typical storey height of 2.4 m, the slenderness ratios corresponding to these three wall thicknesses are $h/t = 19.2$, 16 and 13.7, respectively. Both the thickness and the slenderness of most Swiss walls do not fulfil the Eurocode 8 (CEN, 2004) requirements. This holds for both the regions of low seismicity (Swiss seismic Zone Z1; SIA, 2003) – for which EC8 prescribes a minimum wall thickness of $t = 17$cm and a maximum slenderness of $h/t = 15$ – and for all other regions where the EC8 requirements are: $t \geq 24$cm and $h/t \leq 12$. Based on these figures, typical Swiss walls might be too slender.

To gain experimental evidence on the out-of-plane behaviour of slender URM walls typical for Switzerland and to complement the results obtained by other researchers (e.g. ABK, 1981; Doherty, 2000; Meisl, 2006), an experimental program was set up. In the following, the tests are described while selected results are presented and briefly discussed. A comprehensive report on the test results is in preparation (Dazio, 2009).

2. TEST UNITS, SETUP AND PROTOCOL

2.1. Test Units

The test units represent full scale models of slender URM walls that can be found in modern Swiss residential buildings. All units were 2.4 m high and 1.2 m wide corresponding to a wall located between adjacent openings and spanning vertically between two floor slabs. In total six test units were manufactured using standard hollow clay bricks 29 cm long, 19 cm high and with thicknesses varying between 12.5 and 20 cm depending on the test unit yielding slenderness ratios of 19.2 to 12. The units were laid in running bond with partially filled head joints ("Doppelspatz"). All head and bed joints were 1 cm thick. Dry factory-made cement mortar "M15F" according to CEN (2003) was used. Every wall featured 12 rows of bricks and 13 bed joints which were numbered from bottom to top for easier reference. The geometry of the test units as well as important material properties are summarized in Table 2.1. Noteworthy is the low flexural tensile strength of the masonry which is typical fur current masonry practice in Switzerland. To allow safe transportation and easy mounting on the ETH shaking table, each test units was built on a steel channel filled with concrete.

Table 2.1 Geometry of the test units and mechanical properties of the masonry.

Test unit	W1	W2	W3	W4	W5	W6
Height [cm]	240	240	240	240	240	240
Width [cm]	120	120	120	120	120	120
Thickness [cm]	15	15	12.5	17.5	20	15
Slenderness [-]	16	16	19.2	13.7	12	16
Weight [kN]	4.66	4.66	3.88	5.44	6.22	4.66
Compressive strength [MPa]	7.3	7.3	8.3	9.8	8.0	7.3
Flexural tensile strength [MPa]	0.05	0.05	0.03	0.03	0.04	0.05

2.1. Test Setup

The test setup is shown in Figure 1 where its most important parts are identified by numbers. The core part of the setup is the ETH shaking table (1). A stiff steel frame was built around the table to support two steel beams (2) whose bottom flanges served as rails for the carriage (3) used as a top support of the test units. A guying system (4) connected the carriage and the table ensuring that both elements moved solidly in the direction of motion. Hence, during the tests the same motion was imposed to the top and to the bottom support of the test units. Two steel angles were mounted on the carriage to provide restraint to the test unit (5) in the direction of motion of the table. The two angles had slotted holes, so that they were adjustable to the different thicknesses of the test units. The actual bearing surface consisted of a 1 x 1 cm square-shaped timber strip glued on each angle. The strip had barely contact to the surface of the wall allowing the rotation and vertical displacement of the top row of bricks. The bottom support of the test unit consisted of the steel channel on which the unit was built (6). The channel was rigidly connected to the platen of the shaking table. On both sides of the test unit, three timber studs (7) were mounted one wall thickness away from the face of the test unit. The studs were meant primarily as a protection for the testing equipment; however, they were also able to catch the failing test unit at a stage which prevented its total collapse and allowing therefore multiple testing of the same wall.

Within the testing program, five different top boundary conditions were considered; these are shown in Figure 2: The "simply supported (SS)" (a) condition allowed the top of the test unit to rotate and to elongate freely providing only lateral support by means of the timber strips mentioned earlier. The "Fixed (FX)" (b) condition was used only for Test Unit W1 where it fully restrained the rotation and the elongation at the top of the unit. Since it was recognized that i) in a real structure the walls often support an overburden weight N, that ii) this weight does not always act centrically on the wall, and that iii) the floor can constitute a partial axial (k_N) and rotational (k_M) restraint to the top of the wall, three additional boundary conditions were introduced which

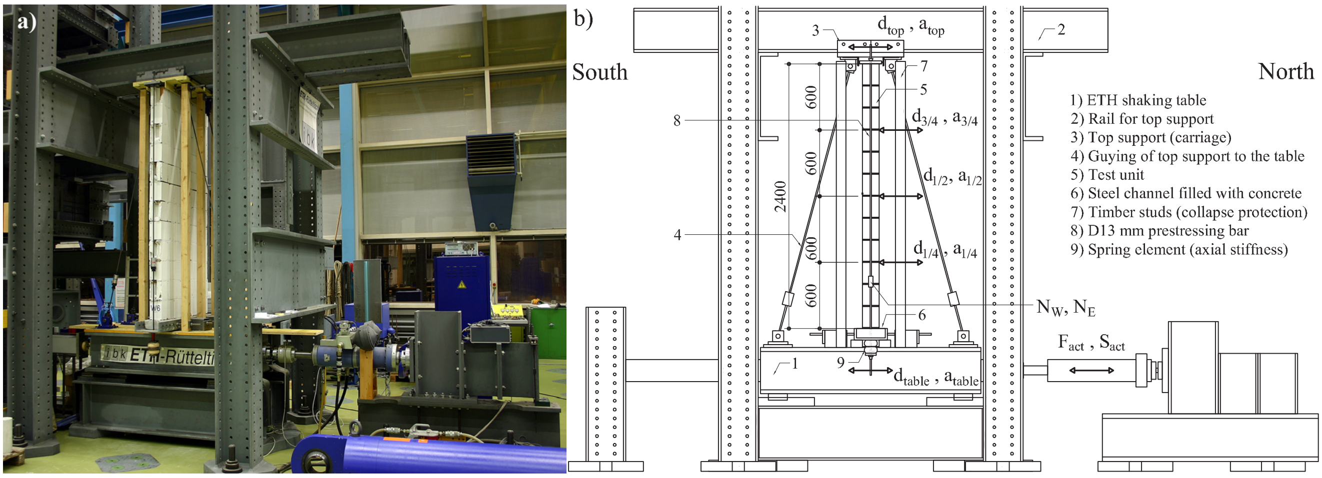

Figure 1 Picture (a) and schematic representation (b) of the test setup including the main instrumentation (a=acceleration, d=displacement, F=force, S=stroke, N=axial load in the prestressing bars).

aim at reflecting these real support conditions: "Centrically Prestressed (CP)" (c), "Crosswise Prestressed (XP)" (d) and "Eccentrically Prestressed (EP)" (e). The initial overburden weight N was applied in proportion to the weight G of the wall as suggested by Doherty et al. (2000). During the test, three axial load levels characterized by the ratios $\psi = 2N/G = 1$, 2 and 3 were considered. The overburden weight was applied through prestressing. To this purpose a 140 cm long, 20 cm wide and 2.5 cm thick steel plate was placed on top of the test unit. To anchor the D13 mm prestressing bars running along the two edges of the test unit (8) the steel plate had five holes at each end: one hole at the centre line, the others at ±40 and ±75 mm, respectively. The same hole pattern was drilled into the steel channel (6) of the foundation where the lower end of the bars was anchored. To simulate the different bearing conditions of a floor at the top and the bottom of the wall (Fig. 2), the prestressing bars were run through different holes in the top plate and in the bottom channel. In this way the prestressing force could be applied centrically, crosswise or eccentrically – hence the naming of the boundary conditions. To simulate the axial restraint provided by a reinforced concrete deck typical for a residential building in Switzerland, special spring elements (9) were added to the anchorage of the prestressing bars which adjusted the total axial stiffness of the latter to about k_N=1.4 kN/mm. The rotational stiffness k_M was zero for all test units but W6.

The main instrumentation of the test units is shown in Figure 1b. In addition to the force F_{act} and the stroke S_{act} of the actuator, also the absolute acceleration and the relative displacement of the table (d_{table}, a_{table}), of the carriage (d_{top}, a_{top}) and of the quarter points over the wall height ($d_{1/4}$, $d_{1/2}$, $d_{3/4}$, $a_{1/4}$, $a_{1/2}$, $a_{3/4}$) were recorded at a 100 Hz sampling rate. Relative displacements of the test units were obtained by subtracting the table displace-

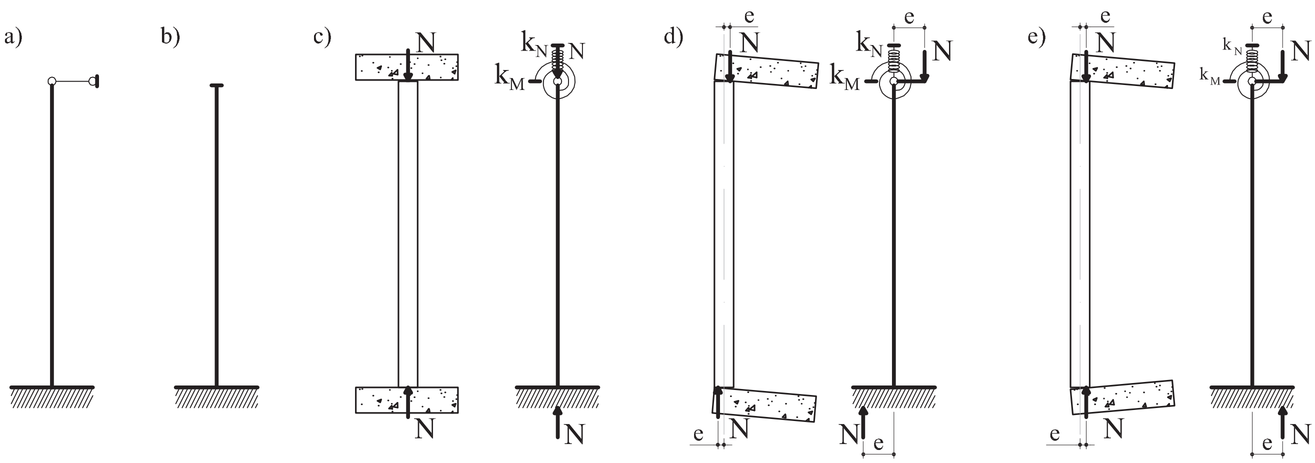

Figure 2 Boundary conditions for URM walls loaded out-of-plane modelled during the shaking table tests.

ments from the displacements measured along the wall; the relative displacements are defined as positive when the test units are deformed to the North. Two load cells measured the forces N_W and N_E in the two D13 mm prestressing bars, i.e. the axial load acting on the test unit.

2.1. Loading History

The selection of the input motion for the shaking table tests was considered an important issue. URM walls located on the top floor of a building are the most vulnerable to out-of-plane deformations because i) the overburden weight acting on the walls is lowest on the top floor and therefore also the strength of the wall, and because ii) for the top floor the amplification of the ground motion is largest. For this reason, the selection of the input motion occurred in two steps: First a ground motion representative of the Swiss seismic Zone Z2 (peak ground acceleration of 0.1 g) and soft soil (soil class E) was chosen (SIA, 2003). In a second step, the floor response of a typical Swiss multi-storey URM building subjected to this ground motion was computed and the floor acceleration at the 4th floor extracted so it could be used as an input motion for the shaking table.

In total, 30 spectrum compatible ground motions were generated and the top floor response of the 4-storey shear frame depicted in Figure 3a was computed by means of transient analysis assuming a linear elastic behaviour with 5% Rayleigh damping for the first and third mode. For the 4-DoF system a fundamental period of 0.5 s was chosen as estimate of the fundamental period of a real URM building in Switzerland. A period of 0.5 s is also close to the expected fundamental period of the test units; hence unfavourable resonance effects were expected. Of the 30 computed top floor responses, the one best approximating the average shaking was finally retained as the input motion for the shaking table. The time history of the shaking table input motion and the corresponding response spectra are depicted in Figure 3. In the acceleration and displacement response spectra of the 4th floor motion, which corresponds to the motion of the shaking table, resonant effects at periods close to 0.5 s are clearly noticeable. For all tests the same shaking table motion was used, merely varying its amplitude by means of a scaling factor (SF).

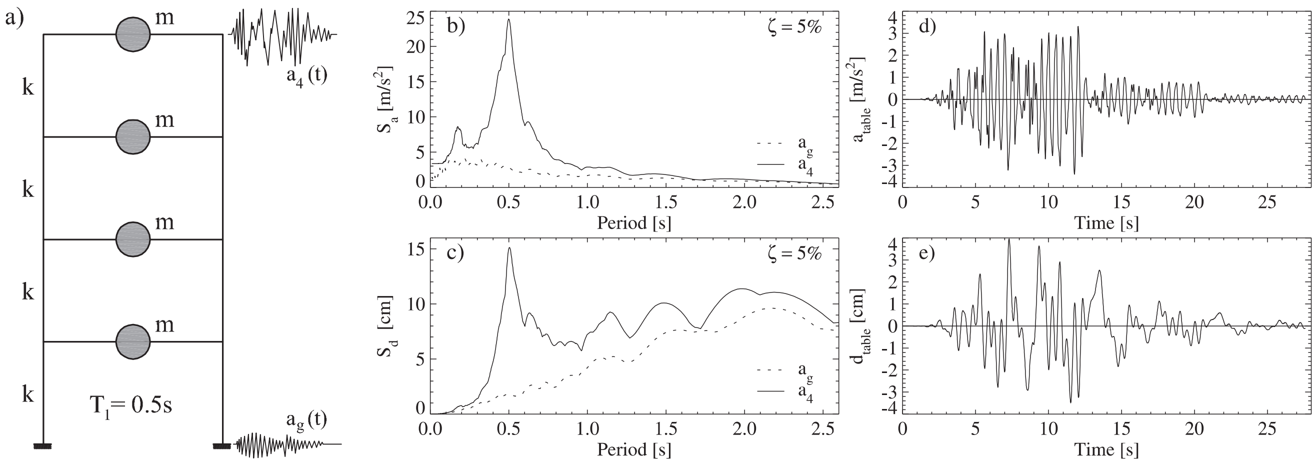

Figure 3 Shear frame with fundamental period of 0.5 s (a). Spectral acceleration (b) and displacement (c) of the ground and 4th floor motions. Shaking table input motion: acceleration (d) and displacement (e) time histories.

3. TEST RESULTS

The results of the tests carried out on all six test units are reported in detail in Dazio (2009). Here, only selected results relevant to Test Unit W2 are presented in Section 3.1 and important findings regarding the influence of wall slenderness, applied axial load and top restraint on the out-of-plane behaviour of the test units are discussed in Section 3.2.

3.1. Test Unit W2

Test Unit W2 was tested for different top boundary conditions and different overburden weights. Furthermore, each configuration was subjected to different intensities of the input ground motion, starting with a low scaling factor and increasing it successively until failure of the wall occurred (failure of the wall was defined as the event when the wall touched the timber studs). In total, 50 different tests were conducted on Test Unit W2 and 8 of them are summarized in Figure 4. Of course, only the first test was carried out on a fully undamaged unit. However, because of the low flexural tensile strength of the masonry, the dynamic behaviour of the cracked wall was not very different than the behaviour of the uncracked wall. Additionally, due to the relatively small axial load only very limited damage to the mortar joints and basically no damage to the bricks occurred. Moreover, the primary goal of the test program was to compare the effects of different boundary conditions on the out-of-plane stability of the wall, and hence the relative behaviour of the different configurations was at least as important as the absolute one.

In Figure 4, the results of four configurations are displayed: 1) “Simply supported” (SS), 2) “Centrically Prestressed” with an overburden weight ψ=2 (CP2), 3) “Crosswise Prestressed” with ψ=2 (XP2) and 4) “Eccentrically Prestressed” with ψ=2 (EP2). For every configuration the test with the largest scaling factor (SF) of the input motion that did not cause failure of the unit as well as the next following test for which failure occurred are displayed. For all tests the same results are plotted: The diagrams on the left display the time histories of the relative displacements at mid- ($d_{1/2}$) and ¾-height ($d_{3/4}$) of the wall. The diagrams in the middle display the hysteresis curves of the absolute acceleration as a function of the relative displacements. The curve for both the mid- and ¾-height movement are given as an indicator for the nonlinear behaviour of the test units. The diagrams on the right display the axial load variation during the test. For ψ=2 the initial overburden weight was about 4.66 kN corresponding to an average axial stress of 0.026 MPa.

The behaviour of the SS-test units was very flexible and hence the horizontal displacements shown in Figure $4a_1$ are significantly larger compared to the other configurations. Figure $4a_1$ shows that the acceleration and the displacement at ¾-height are noticeably larger than at mid-height. This is due to the complex rocking behaviour displayed by SS-test units under intense shaking. During a typical strong rocking phase, cracks opened first at joints 1 (foundation) and 9 (1.6 m above the foundation, =2/3 of the wall height). During subsequent cycles it was observed that the location of the main crack in the top third of the wall changed continuously moving from joint 9 up to joint 12 and back again. This rocking behaviour is due to the low masonry tensile strength which is not able to keep the lower and upper rocking bodies intact after formation of the first crack. This behaviour differs significantly from the rigid-body rocking observed by other researchers in the past (see e.g. Doherty et al., 2002). However, because of this intense rocking the SS-test units were able to withstand severe input motions without collapsing. Simply supported walls with no overburden weight are often considered as the most critical in terms of out-of plane stability; hence the behaviour of this configuration is used as a benchmark which the other configurations are compared to.

The results of the configuration CP2 are shown in Figure 4c and 4d. Under the input motion scaled at 140%, the test unit behaved almost linearly and only a very minor opening of the joints 1 and 9 could be observed. The linear behaviour is confirmed by the hysteresis curves shown in Figure $4c_2$ which deviate only slightly from a straight line. During the shaking, the axial load in the wall reached a maximum of 8.65 kN which nearly doubled the initial value. Increasing the ground motion to 160%, i.e. by just 14% compared to the previous test, led to a sudden failure of the test unit, which would have been catastrophic, had the timber studs not been in place. The failure mechanism was as follows: After about 7.5 seconds, a strong rocking cycle commenced. The test unit was first displaced 50 mm to the North with opening of the cracks at joints 1 and 9. In Figure $4d_2$ an almost horizontal plateau in the hysteresis curves is noticeable. Upon reversal of the deflected shape the wall failed in the South direction while rocking on the same joints 1 and 9. The axial load reached a maximum of 20.4 kN. The increase in axial load is caused by the tendency of the wall to elongate during the rocking motion which is partially restraint by the top boundary condition. A good estimate of the increase in axial force can be obtained from the relative displacement of the wall, simple geometrical considerations and the axial stiffness of the prestressing bars (Dazio, 2009). In contrast to the SS test configuration, rigid-body rocking with no noticeable cracks opening neither in the lower body nor in the upper one was observed and the same simple geometrical considerations show that the location of the upper crack in correspondence of joint 9 yield the smallest wall elongation; hence the smallest increase in axial load.

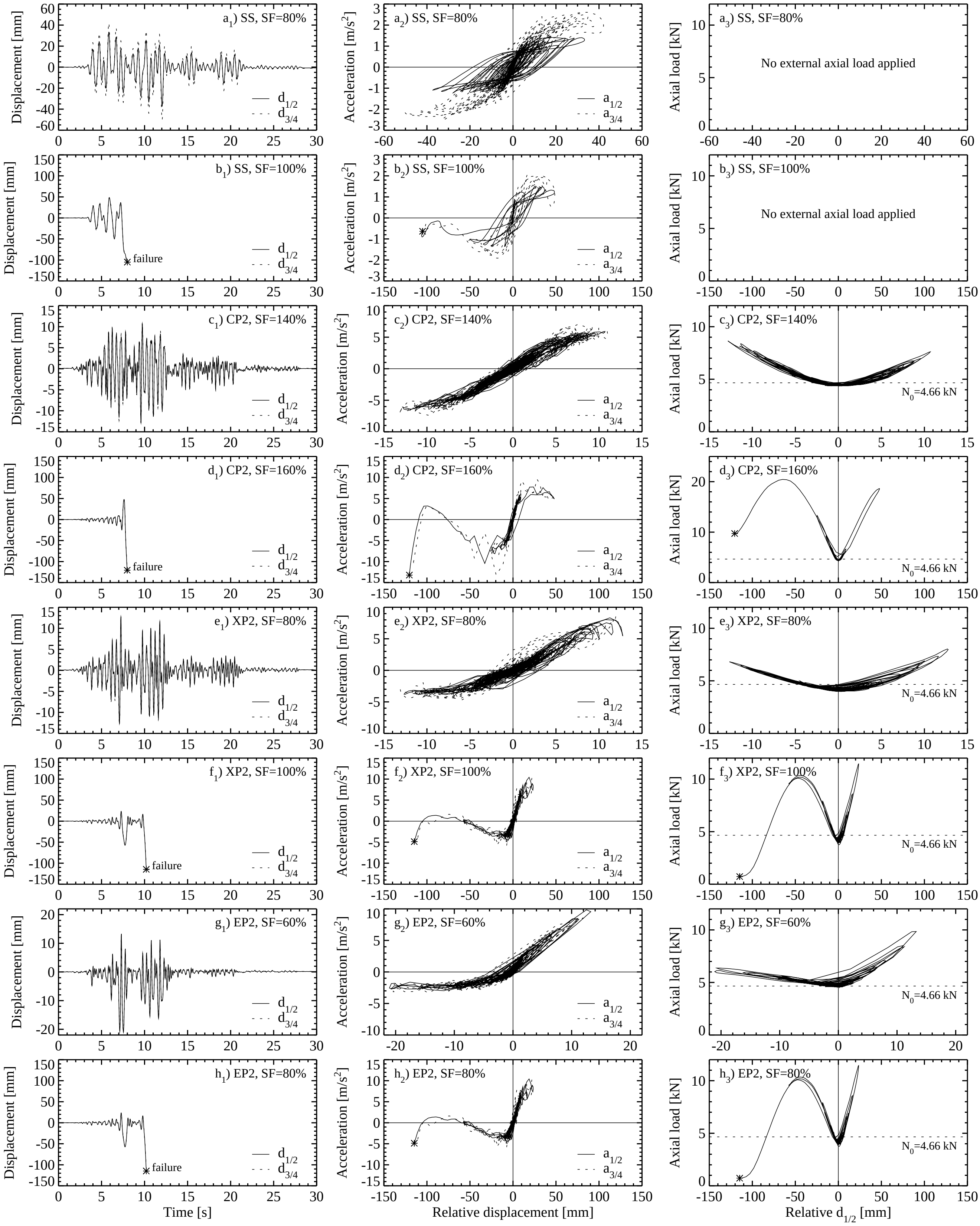

Figure 4 Test Unit W2: Displacement time-histories (a), hysteretic behaviour (b) and axial load variation (c) for selected tests. For each boundary condition the last test before failure and the test with failure are plotted. (Note that in the plots different scales are used for different tests).

To summarize regarding the behaviour of the configuration CP2, it is important to observe that a non-linear deformation regime due to significant rocking does exist; however, in this particular case it does not allow the wall to sustain excitations significantly larger than those where rocking is barely noticeable.

The behaviour of the XP2 configuration is shown in Figures 4e and 4f. In this case, the top eccentricity was 40 mm to the North while the bottom one was 75 mm to the South. Already under the input motion scaled at 80%, the test unit showed an asymmetric behaviour. The maximum relative displacements in the North and in the South direction were almost the same (Fig. $4e_1$), however, during deflections to the South, opening of the joints 1 and 9 could clearly be seen, while during deflections to the North the opening of the same joints was significantly smaller. This is consistent with the observation that the absolute maximum acceleration was significantly larger when the unit was deflected to the North by the Prestressed bars (Fig. $4e_2$), and can be explained by the fact that the rotation of the top of the unit was restraint for deflections to the North. Increasing the scaling factor to 100% led to the failure of the test unit. As in the case of CP2, significant rocking did only occur just prior to failure and did not contribute to the increase of the sustained excitation significantly. The results of configuration EP2 are shown in Figures 4g and 4h. The top eccentricity was 40 mm to the North while the bottom one was 75 mm also to the North. For this boundary condition the asymmetric behaviour is more pronounced, however the force-deformation characteristic of the test units features similar qualities as in the previous case.

3.2. Comparison Between Different Test Units

Test Units W3 to W5 were tested with the same boundary conditions as Test Unit W2 but they had smaller (W3) or larger (W4, W5) wall thicknesses than Test Unit W2. Part of the results are summarized in Figure 6 in terms of the maximum ground motion scaling factor that the test units were able to sustain in a particular configuration before failing. In Figure 6a the maximum scaling factor for the configurations SS, CP1, EP1 and XP1 are compared. The tests confirm that increasing the thickness of the walls, which in this case corresponded also to reducing the wall slenderness, increases the out-of-plane stability – this holds for all investigated boundary conditions. However, the maximum scaling factors for the boundary conditions SS and CP1 were for all wall slenderness ratios considerably larger than for the asymmetric boundary conditions EP1 and XP1.

Figure 6b shows that increasing the axial load ratio increases the level of shaking that the wall can sustain before failure. The plot shows the EP case but similar trends were also observed for the other configurations. However, the configurations differed regarding their sensitivity to the axial load level: The sensitivity was largest for the EP configuration and smallest for the CP configuration. Note also that the levels of shaking sustained by the EP walls were smaller than for the SS walls.

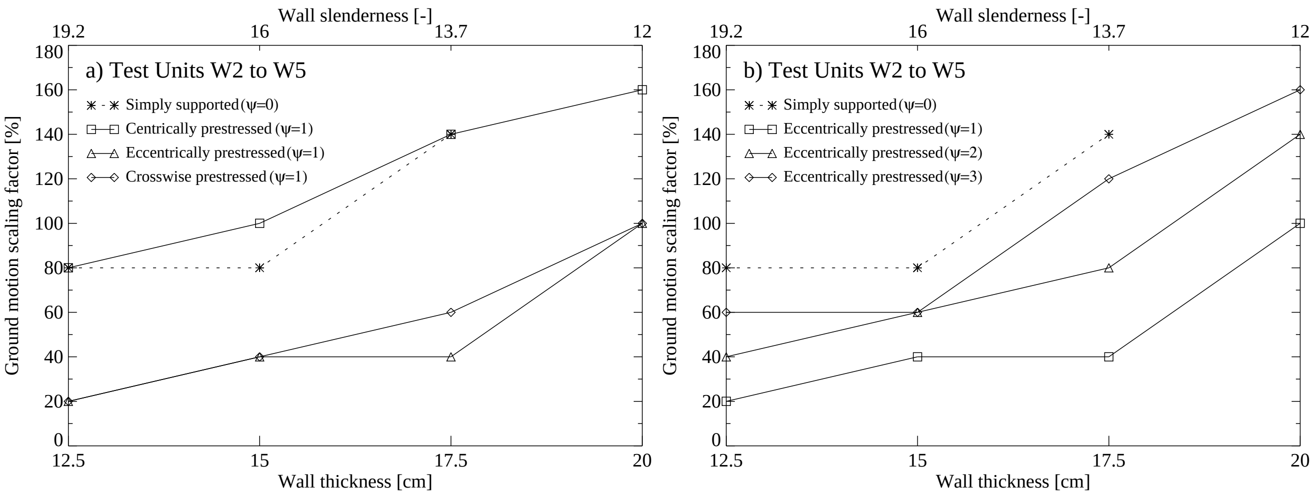

Figure 6 Influence of wall thickness, boundary conditions (a) and axial load (b)
on the out-of-plane resistance of URM walls.

4. CONCLUSIONS AND OUTLOOK

The paper gave a brief overview on the results of an experimental study on the out-of-plane behaviour of URM walls typical for Switzerland. The main aspects investigated were the slenderness ratio of the walls, the axial load ratio and most importantly the restraints of the wall. Most experimental studies carried out in the past assume that the boundary condition "Simply Supported" represents the most vulnerable state of the wall since the wall is not subjected to any axial load nor is its top restrained against rotation. The results of this study have, however, shown that this is not always the most critical boundary condition but that boundary conditions that introduce an eccentric axial force to the wall can cause failure of the wall at considerably smaller shaking levels. This raises the question whether the simply supported case can be looked upon as a worst case scenario. Moreover, it was observed that the walls that were simply supported had a very large displacement capacity which led for large shaking levels to extensive rocking motions but not necessarily to failure. On the contrary, the walls subjected to an axial load deformed only relatively little before a sudden failure of the wall occurred.

In a next step of this study simple numerical models will be used to analyse the rocking motion of the walls subjected to an axial load. The objective of these analyses is to check whether the numerical models confirm the trends observed in the experiments. Since the experiments have shown that the out-of-plane behaviour of URM walls is at least as sensitive to the boundary conditions as to the axial load and slenderness ratio, a future study will investigate in more detail the true boundary conditions of URM walls in real buildings and, if required, a further experimental study will be undertaken.

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REFERENCES

ABK (1981). Methodology for mitigation of seismic hazards in existing unreinforced masonry buildings – Wall testing, out-of-plane. Topical Report 04. ABK, A joint venture, El Segundo, California.

CEN (2003). Specification for mortar for masonry – Part 2: Masonry mortar. European Standard EN 998-2:2003. European Committee for Standardization, Brussels.

CEN (2004). Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic action and rules for buildings. EN 1998-1:2004, European Committee for Standardization, Brussels.

Dazio, A. (2009). Out-of-plane shaking table tests on unreinforced masonry walls. IBK research report in preparation. ETH Zurich.

Doherty, K. (2000). An investigation of the weak links in the seismic load path of unreinforced masonry buildings. Ph.D. Thesis. Faculty of Engineering, The University of Adelaide, Adelaide.

Doherty, K., Griffith, M.C., Lam, N., Wilson, J. (2002). Displacement-based seismic analysis for out-of-plane bending of unreinforced masonry walls. *Earthquake Engineering and Structural Dynamics* **31**, 833-850.

Meisl, S. (2006). Out-of-plane seismic performance of unreinforced clay brick masonry walls. Master thesis. The Faculty of Graduate Studies, The University of British Columbia, Vancouver.

SIA (2003). Actions on Structures. SIA 261:2003. Swiss design code. Swiss Society of Engineers and Architects, Zurich.