Seismic response of a 4 storey building with reinforced concrete and unreinforced masonry walls

BEYER, KATRIN[1]; TONDELLI, MARCO[2]; PETRY, SARAH[3], PELOSO, SIMONE[4]

In Switzerland many new residential buildings are designed as mixed structures with unreinforced masonry (URM) walls and reinforced concrete (RC) walls while existing URM structures are often retrofitted by replacing selected URM walls by RC walls. The lateral bracing system of the resulting structure consists therefore of URM walls and some RC walls. All walls are coupled by RC slabs and/or masonry spandrels. Since the seismic performance of such mixed structures– despite being very common in design and retrofit– is still not well understood, a shake-table test on a four-storey RC-URM wall structure was performed at the TREES laboratory of the EUCENTRE in Pavia (Italy). In this paper we present the details of the shake-table test and discuss its results. We compare the observed behaviour to that generally observed for URM structures and conclude on the effect of adding slender RC walls to URM buildings in terms of damage evolution. The force-displacement response of the structure is presented, its response for the two loading directions discussed and different methods for computing the base shear force compared.

Keywords: Shake-table test, mixed structure, unreinforced masonry wall, reinforced concrete wall, seismic behaviour.

1 INTRODUCTION

Existing URM structures are often reinforced by adding RC walls or by replacing selected URM walls by RC walls. In Switzerland, many new residential buildings are directly designed as mixed RC-URM wall structures with RC slabs (Figure 1). Although such structures are very common in design and retrofit practice, their seismic behaviour is not well understood and the design often based on very simplifying assumptions. Existing numerical studies addressed the seismic performance of mixed RC-URM structures where the RC walls were not capacity designed and failed before the URM walls [1], while previous shake table tests studied either mixed systems where the RC element was so flexible that it had no influence on the overall performance of the structure [2] or the RC wall was pinned at the base [3].

The objective of the shake table test on a half-scale four-storey structure with RC and URM walls coupled by RC slabs was to observe the behaviour of such mixed structures when subjected to seismic loading and to collect high-quality data that can be used for the validation of numerical models. The shake table test was conducted at the TREES laboratory of the European Centre for

[1)] Assistant Professor, Earthquake Engineering and Structural Dynamics Laboratory (EESD), School of Architecture, Civil and Environmental Engineering (ENAC), École Polytechnique Fédérale de Lausanne (EPFL), Switzerland, katrin.beyer@epfl.ch
[2)] Scientific Assistant, Earthquake Engineering and Structural Dynamics Laboratory (EESD), School of Architecture, Civil and Environmental Engineering (ENAC), École Polytechnique Fédérale de Lausanne (EPFL), Switzerland, marco.tondelli@epfl.ch
[3)] PhD student, Earthquake Engineering and Structural Dynamics Laboratory (EESD), School of Architecture, Civil and Environmental Engineering (ENAC), École Polytechnique Fédérale de Lausanne (EPFL), Switzerland, sarah.petry@epfl.ch
[4)] Researcher, EUCENTRE - European Centre for Training and Research in Earthquake Engineering, Pavia, Italy. E-mail: simone.peloso@eucentre.it

Training and Research in Earthquake Engineering in Pavia, Italy. It complements a quasi-static test on a structure with one RC and one URM wall conducted at EPFL, which allowed to study in detail the force distribution between the two structural elements [4].

Since RC walls are typically not distributed evenly in the plan of such buildings, the buildings respond differently for loading in the positive or negative direction along one axis. This was replicated in the test unit by placing the RC walls at one end of the structure. The objective of this paper is to discuss the global response of the structure and focus in particular on the differences in the behaviour for the two loading directions. The paper commences with a brief presentation of the test unit (Section 2) and then discusses the response of the structure based on the observed damage (Section 3) and the force-displacement response (Section 4). A more extensive presentation of the results will be available in [5]; the raw and processed data collected during the test is publically available and documented in [6].

Figure 1. 3- and 4-storey residential buildings with RC wall and URM walls (Photos: Thomas Wenk).

2 TEST UNIT, TEST PROGRAMME AND INSTRUMENTATION

2.1. Test unit

The test unit was a four storey structure built at half-scale. Since the structure comprised two different types of materials (masonry and concrete) and the interaction of the structural components was of prime interest, it was important that the scaling did not introduce a distortion in the relative strengths and stiffnesses of the two materials. The reinforced concrete was scaled by using reduced size bars and limiting the size of maximum aggregates to 8 mm. The RC element's deformation capacity was less critical, since neither RC slabs nor walls were expected to undergo large inelastic deformations. The URM walls, on the contrary, were expected to reach deformations close to their capacity and therefore it was important that the behaviour of the scaled masonry resembled that of the prototype masonry for all limit states. For the construction of the test unit, special fabricated half-scale bricks were used. A study comparing prototype and half-scale masonry by means of quasi-static cyclic tests on piers revealed that the two types of masonry had very similar stiffness and strength properties but that the half-scale masonry lead to somewhat larger drift capacities [7].

The test unit was tested on a uni-directional shake table, -whose direction of motion was aligned with the longitudinal axis of the structure (North-South axis). The test unit was symmetric along this axis, avoiding hence all but accidental torsion in the global behaviour. This was done to facilitate the interpretation of the measurement results and concentrate on the in-plane response of mixed URM-RC wall structures.

With regard to the East-West axis, the structure was, however, asymmetric since the URM walls that were loaded in-plane were grouped at the North end of the structure and the RC walls on the South end. This plan layout reflected our observation for real buildings where RC walls are typically not evenly distributed over the floor plan. For example, often the walls surrounding stair cases and lift shafts are constructed using RC walls and are therefore located at the extremity of a building.

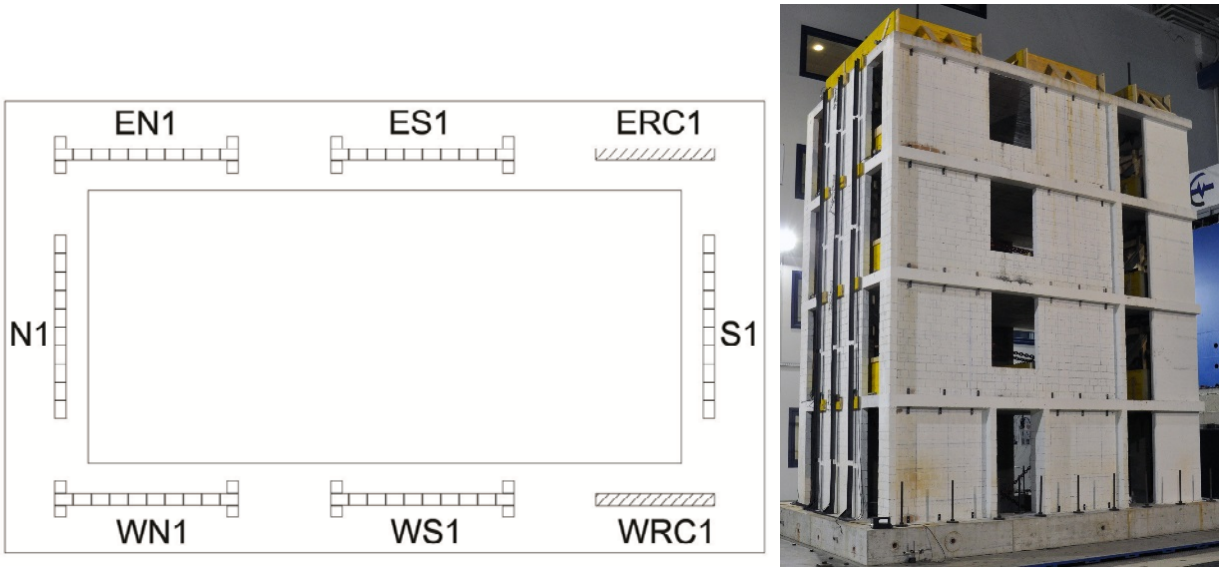

Figure 2. Plan view with cardinal directions (left) and South/West view of the test unit over the shake table (right)

2.2. Test programme

The shake table test was conducted using the record of the Ulcinj station (E-W-component) during Montenegro 1979 earthquake as input ground motion. To account for the reduced scale, the time axis of the record was divided by $\sqrt{2}$ and additional masses were added at the storeys by means of large unreinforced concrete blocks. The entity of these blocks had the same mass as the test unit without its foundation. The test unit was subjected to nine runs of increasing peak ground acceleration (PGA). After the 6th run with a nominal PGA of 0.6g, a run with a smaller PGA of 0.4g was inserted in order to simulate the effect of an aftershock or a more distant epicentre in a sequence of events. The sequence of nominal PGAs was: 0.05, 0.1, 0.2, 0.3, 0.4, 0.6, 0.4, 0.7, 0.9g.

2.3. Instrumentation

The behaviour of the test unit as well as the motion of the shake table was recorded by 20 accelerometers, 49 displacement transducers, 24 omega gages and an optical measurement system. The latter monitored approximately 600 points on the URM walls of the West façade.

3 OBSERVATIONS

The experimental campaign required four days of testing. After each shaking new cracks were marked and the damage evolution documented. The effect of the damage on the dynamic characteristics of the structure, i.e. the natural periods of vibration, was estimated by performing a structural identification. The first five runs had PGAs smaller than 0.4g and induced only minor damage in the form of hairline cracks in the URM piers and RC slabs. Test 6 (0.6g) increased the damage to the structure significantly. At this point all URM piers that were loaded in-plane developed diagonal cracks at all storeys. After test 8, the structure displayed a severe level of damage. At this point the damage in the URM panels started concentrating in one diagonal crack. From quasi-static

cyclic tests on URM piers it is known that this indicates that the post peak branch has been reached and failure is rather imminent [10]. In addition, diagonal cracks passed now for the first time through bricks. The following sections outline the effect of the loading direction (Section 3.1) and discuss the particularities of the observed damage pattern (Section 3.2) on the basis of the crack pattern observed after the last run (Run 9).

3.1. Effect of loading direction

The uni-directional shake table excited the structure along its longitudinal axis in North-South direction (Figure 2). Since the structure was not symmetric about its East-West axis, the behaviour of the structure was different for the two loading directions. This section discusses the damage pattern of the first storey after the final run with a nominal PGA of 0.9g. This run brought the structure near to collapse (Figures 3 and 4).

When the structure was pushed towards North, the axial force in the URM piers at the North end increased while it decreased in the RC walls and the South URM piers, which were loaded out-of-plane. For the out-of-plane loaded URM piers, an increase in axial force led to reduced out-of-plane movements. For a detailed discussion on the behaviour of the out-of-plane loaded piers during the test see [1]. If only the in-plane loaded walls are considered, loading towards North hence increased the axial force in the outer URM piers while it decreased it in the RC walls. As a result, the RC wall developed shear-flexure cracks over the entire height of the first storey. The axial force in the centre remained approximately equal, though small variations might have been caused by the different types of horizontal elements (RC slab only, RC slab plus masonry spandrel) framing into these URM piers. Since the deformation capacity of URM piers decreases with increasing axial load [9][10], the outer URM piers failed first and developed failure patterns indicating that the failure had occurred for loading towards the North. Pier NW failed due to crushing of the toe while pier NE failed along a diagonal crack and crushing of the web. Hence, although the structure was symmetric along its NS-axis, the behaviour of the East and West face was not exactly identical; most likely due to slight variations in the material properties.

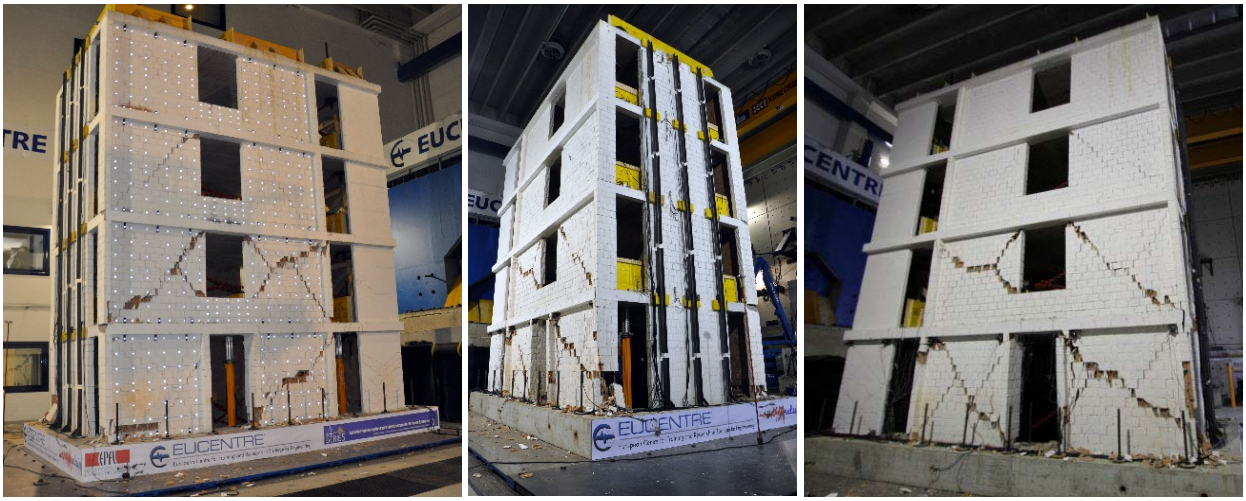

Figure 3. Test unit after final run.

When the structure was pushed towards the South, the axial force in the outer URM piers decreased and led hence to a behaviour which was dominated by a rocking motion (see the video on http://eesd.epfl.ch/). The centre piers were subjected to similar demands for the two loading directions and developed therefore the typical X-type crack pattern, which is well known from quasi-static cyclic tests with symmetric cycles. For loading towards South, the axial force in the RC wall increased and – according to numerical investigations – the two RC walls carried at this point approximately two thirds

of the total base shear. Since the wall was in compression, the residual crack widths remained close to zero indicating in fact that for this loading direction the RC wall had not yet yielded.

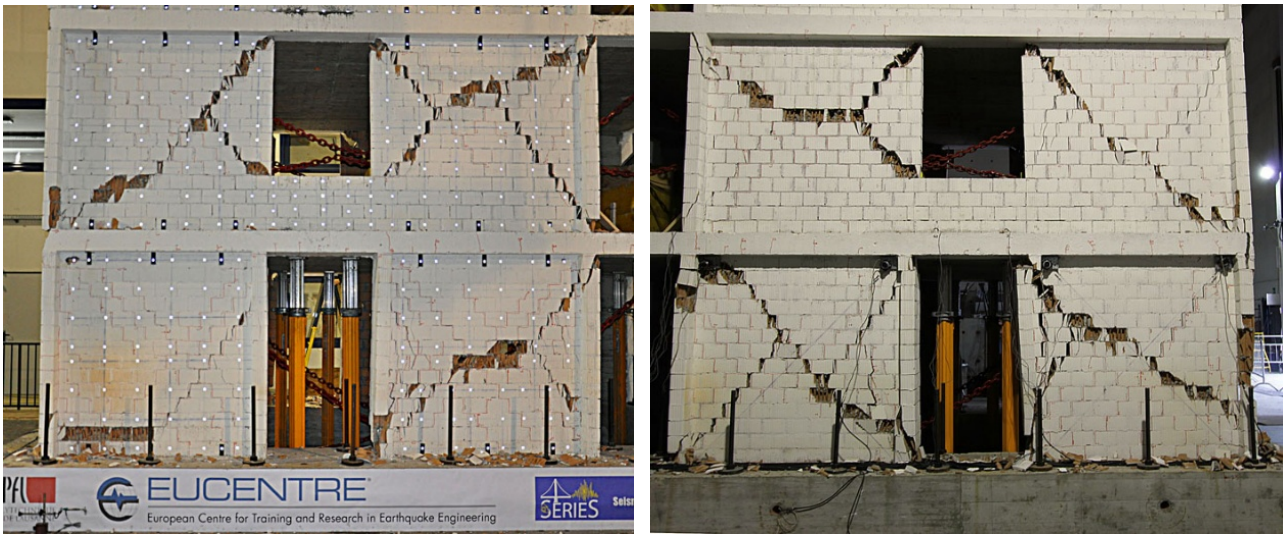

Figure 4. URM piers of first and second storey after final run (Left: West face, right: East face).

3.2. Damage pattern

In URM buildings the piers of the first storey are typically subjected to the largest damage while piers of the storeys above are often only subjected to minor damage. This results from the global behaviour as “shear beam” which leads to the largest interstorey drifts at the first storey. In a structure with URM and RC walls like the one tested here, the URM walls are coupled to slender RC walls, which deform chiefly in a flexural mode. As a result, the displacement profile is almost linear over the height of the building and the damage spreads over the first two storeys almost equally. The same behaviour was observed in quasi-static cyclic tests on two mixed structures [4][11].

Figure 5 shows the drift envelopes for all nine runs. Section 3.1 explained the difference in behaviour of the structure for loading in the negative and positive direction. For this reason, the envelopes are constructed for the two loading directions separately. The plots show that up to the second last run, the drift profile is almost constant over the first three storeys while the drift of the top storey is typically slightly smaller. For the last run, for loading in the negative direction (i.e. towards North leading to an increased axial load in the outer URM walls), the interstorey drifts of the bottom two storeys were considerably larger than for the top two storeys reflecting the damage shown in Figures 3 and 4. For loading in the positive direction, the drift profile reflects for this run clearly the displaced shape of a structure dominated by flexural deformations indicating that for this loading direction the behaviour was dominated by rocking URM piers and the slender RC walls, which were for the positive loading direction still largely elastic.

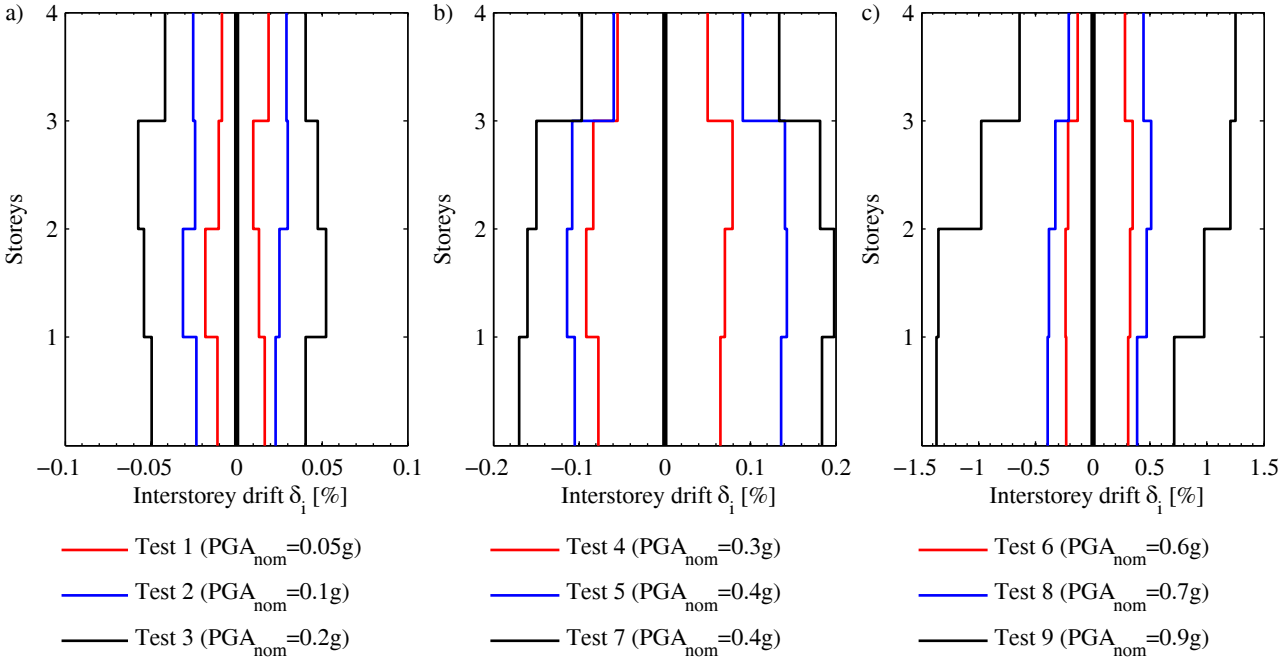

Figure 5. Drift envelopes for the nine runs: Run 1-3 (a), runs 4, 5 and 7 (b), and runs 6,8 and 9 (c)

4 FORCE-DISPLACEMENT RESPONSE

4.1. Computing the base shear

Unlike in quasi-static cyclic tests, where the applied forces can be measured directly through load cells or pressure gauges, in shake table tests the forces need to be derived from other measurement quantities. In the test discussed here, the base shear force can be obtained in three different ways:

1. Force of the actuator of the shake table minus the inertia force of table and foundation. The latter can be estimated from the accelerometer mounted on the table.
2. Sum of the inertia forces of the four storeys where the inertia forces are computed from the accelerometers placed on each floor times the storey mass.
3. As point 2 but the floor accelerations are derived from the optical measurements of the markers glued onto the NW and SW piers (Figure 2).

None of the three methods is completely non-ambiguous and therefore correct but all three suffer from certain shortcomings:

- Next to the inertia force of the table and foundation, the actuator force comprises also the friction force between table and rails and any friction force in the actuator and the hydraulic system. This is particularly evident for the runs with small PGAs.
- Method 2 and 3 assume both that the system can be approximated as a lumped mass system. This approximation seems, however, reasonable given that the largest part of the mass is concentrated in the slabs and the additional masses placed on the slabs and that out-of-plane deformations of the N and S URM walls are small (with the exception of N4 during run 9).
- The accelerometers of the third and fourth floor saturated during run 8 and 9 and therefore base shear can only be computed from accelerometer measurements up to run 7.
- Computing accelerations from the optical measurements, which were recorded with 10 cameras, requires two steps: First, the measurements from the cameras need to be

synchronised. Second, the displacement histories need to be derived twice in order to obtain accelerations. Both steps introduce errors into the base shear estimation. In addition, when compared to the accelerometer readings, the measurements frequency is considerably lower (60 Hz instead of 1024 Hz).

As an example Figure 6 shows the base shear force-histories obtained for Test 6. Test 6 was the test with the largest nominal PGA for which none of the accelerometers saturated (PGA_{nom}=0.6g). Run 7 simulated an aftershock and applied a nominal PGA of 0.4g only. For Run 8 and 9 the accelerometers of the third and fourth storey saturated. One can see that the three force measurements lead to rather similar results when the forces are large. However, at the beginning of the test, the base shear computed from the load cell in the actuator (Method 1) deviates significantly from the base shear computed according to Method 2 and 3. This could be possibly linked to friction forces in the actuator, in the hydraulic system or between rails and table. For tests with small nominal PGAs the base shear computed from the load cell measurement is therefore not reliable (Figure 7). This underlines that for shake table tests the force measurements are somewhat ambiguous and not as clearly defined as for quasi-static cyclic tests.

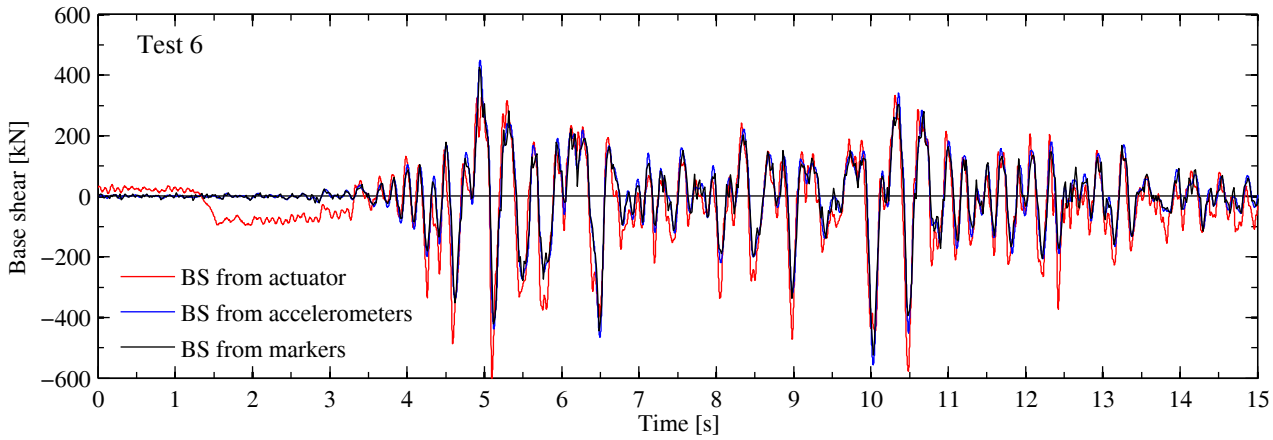

Figure 6. Run 6: Force time-history with the base shear (BS) obtained from load cell in actuator (red line), from accelerometers (blue line) and from markers (black line).

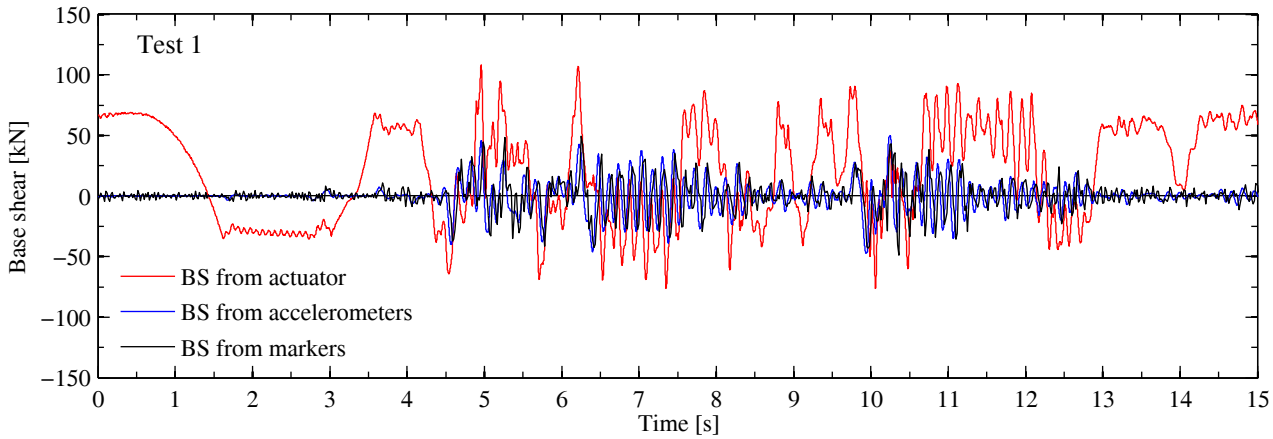

Figure 7. Run 1: Force time-history with the base shear (BS) obtained from load cell in actuator (red line), from accelerometers (blue line) and from markers (black line).

4.2. Force-displacement hysteresis

To obtain the hysteretic behaviour of the structure, the force is plotted against the average drift. The latter is computed as:

$$\delta_{avg}(t) = \frac{\Delta_{top}(t) - \Delta_f(t)}{H}$$

where Δ_{top} is the displacement of the top slab, Δ_f the instantaneous displacement of the foundation and H the total height of the structure (H=6.2 m). To plot the force-displacement response using the Method 1 and 2 for the base shear, the average drift computed from the markers needs to be resampled since the sampling rate of the conventional instruments such as load cell and accelerometer was 17 times larger than the sampling rate of the optical measurement system (f=1024 Hz vs. f=60 Hz). The optical and conventional measurements need also to be synchronised. The latter is done by synchronising the measurements of the LVDT measuring the table displacement and a marker on the foundation of the structure.

Figure 8 shows the force-drift hysteresis of Test 6 using two different force-measurements, Method 3 where the base shear is computed from the displacement of the markers (Figure 8a) and Method 2 where the base shear is computed from the accelerometers placed at every storey (Figure 8b). Due to the larger measurement rate of the accelerometers, the hysteresis in Figure 8b is smoother than the hysteresis in Figure 8a.

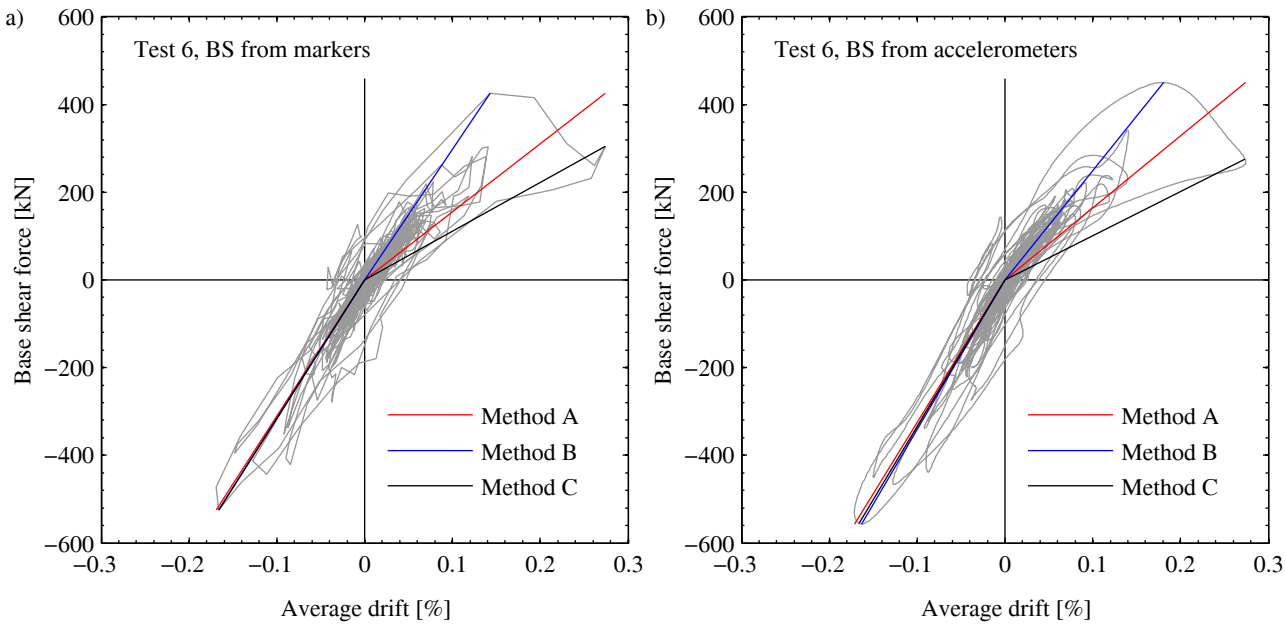

Figure 8. Force-displacement response: Hysteretic response of Run 6 where base shear is obtained from displacement of markers (a), and from accelerometer measurements (b).

4.3. Pushover curve

When comparing the experimental results from the shake table test to results of static analyses, it is helpful to construct a “pushover” curve. This curve can be defined in several different ways. In the following three ways are explored which use for all runs one pair of force and displacement:

a) Method A: Peak force/ peak displacement (not simultaneous)
b) Method B: Peak force and simultaneous displacement

c) Method C: Simultaneous normalised force/ displacement couple with largest distance to origin

Figure 8 visualises these three ways of defining the force/ displacement couple for run 6. For this run, the behaviour in the negative loading direction was largely elastic and hence the three methods lead to similar force/ displacement couples. For the positive loading direction, the behaviour was already largely inelastic and as a result the three force/ displacement couples are rather different. Figure 9 shows the pushover curves according to the three methods. The plots confirm that the methods lead to rather similar results when the structure behaves elastically but differ considerably in the inelastic range. For comparison with static pushover analyses Method A seems most appropriate since it envelopes forces and displacements and it is not affected by damping forces or the shape of the hysteresis, which cannot be reproduced in static analysis.

Figure 10 shows the curves for Method A which are obtained for the three ways of computing the base shear (Section 4.1). The correspondence between the base shear computed from accelerometer measurements and marker displacements is very satisfactory for both loading directions. For run 8 and 9 the base shear computed from accelerometer measurements is not available. However, it seems reasonable to assume that for these runs the base shear derived from the marker displacements leads also to reliable estimates of the base shear. For the negative loading direction, the latter corresponds also well to the base shear obtained from the actuator force. For the positive loading direction, however, the actuator force does not agree with the force computed for any of the other two approaches. To conclude, it is suggested to compute the pushover curve from the marker displacements since these are available for all runs and seem to lead to reliable estimates of the base shear for both loading directions.

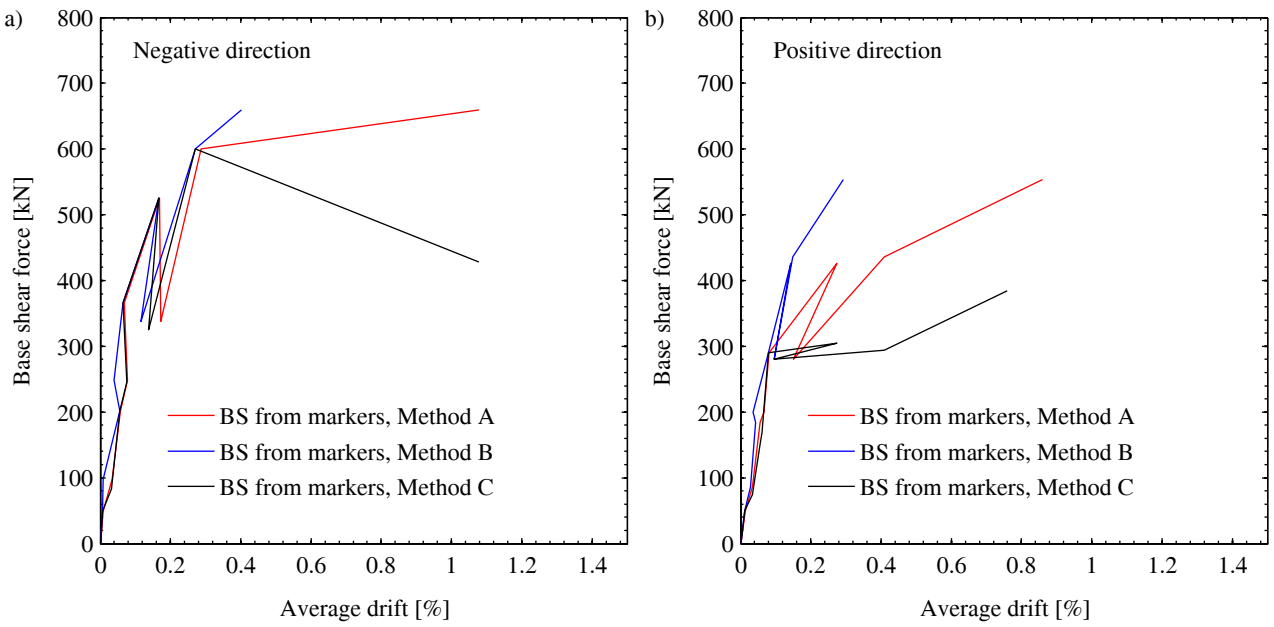

Figure 9. Pushover curve for negative (a) and positive (b) loading direction. Base shear was computed from marker displacements and pushover curve defined using three different methods.

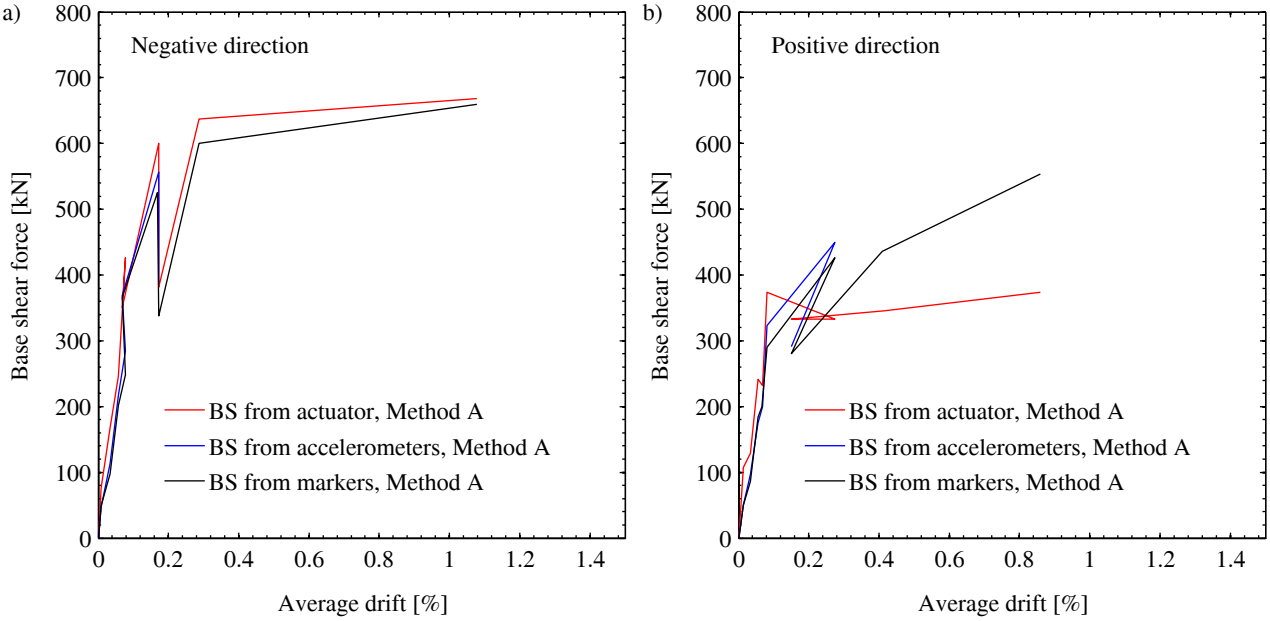

Figure 10. Pushover curve for negative (a) and positive (b) loading direction. Base shear was computed applying Method A to three different approaches for computing the base shear.

The comparison of the pushover curves for the negative and positive loading direction shows that the stiffness of the structure in the elastic range is rather similar for the two loading directions. This is expected since the asymmetry only appears once significant cracking in the URM piers occurs. The force capacity of the structure is significantly larger for the negative loading direction that for the positive loading direction. For the negative loading direction, the axial force in the outer URM pier increases leading hence to an increase in force capacity while the opposite is the case for the positive loading direction. The capacity of the RC wall is not so sensitive to the change in axial force. The curves show further that the building had reached its maximum force capacity during Run 8 for the negative direction of loading. This agrees with the observation in Section 3 that Run 8 led to the concentration of damage in a single diagonal crack indicating that the peak strength had been reached or passed [10].

5 SUMMARY AND OUTLOOK

The paper evaluates the force-displacement response for the shake table test on a 4-storey structure with RC and URM walls and links the measured response to the observed damage pattern. A particular emphasis is placed on the different behaviour for the positive and negative loading direction, which results from the asymmetry of the plan layout. Several ways of computing the base shear are compared and found to agree in general rather well. Only the base shear computed from the actuator force is afflicted with a considerable error, which is most dominant for small forces and forces in the positive loading direction.

Ongoing works address the modelling of such mixed structures [11] and the comparison of the predicted to observed response [5], the comparison of URM pier deformation capacities under dynamic and static loading [12] and the link between peak and residual deformations [12]. The final objective is to formulate next to the modelling recommendations, force- and displacement-based design guidelines for such mixed structures.

ACKNOWLEDGEMENTS

The research leading to these results received funding from the European Community's Seventh Framework Programme [FP7/2007-2013] for access to TREES laboratory of EUCENTRE under the grant agreement nº 227887. Additional financial support was received from the Office Fédéral de l'environement (OFEV) in Switzerland. The authors appreciate and gratefully acknowledge both financial contributions. The authors would like to thank all members of the TREES laboratory and in particular the head of the laboratory, Prof. Alberto Pavese, for their invaluable support during the entire duration of the project. Additionally the authors would like to thank Morandi Frères SA, Switzerland, for the effort put into the production and donation of the small-scale bricks for the construction of the test specimen.

REFERENCES

[1] Cattari S, Lagomarsino S. Seismic assessment of mixed masonry-reinforced concrete buildings by non-linear static analyses. Earthquake and Structures 2013; 4 (3).

[2] Tomazevic M, Modena C, Velechovsky T. Seismic behaviour of mixed structural systems with peripheral masonry walls and internal RC columns: an earthquake simulator study. Proceedings of "Fifth North American Masonry Conference"; Urbana Champaign (USA); 1990.

[3] Jurukovski, D., Krstevska, L., Alessi, R., Diotallevi, P.P., Merli, M. and Zarri, F. [1992] "Shaking table tests of three four-storey brick masonry models: original and strengthened by RC core and by RC jackets", Proc. of 10th World Conference on earthquake Engineering, Madrid, Spain.

[4] Paparo, A. & Beyer, K.: Quasi-static cyclic tests of two mixed reinforced concrete - unreinforced masonry wall structures, submitted to the *Engineering Structures*, 2013.

[5] Beyer, K., Tondelli, M., Petry, S. & Peloso, S.: Dynamic testing of a 4-storey building with reinforced concrete and unreinforced masonry walls. To be submitted to *Bulletin of Earthquake Engineering*, 2014.

[6] Tondelli, M., Beyer, K., Petry, S., & Peloso, S.: Data set of dynamic tests of a 4-storey building with reinforced concrete and unreinforced masonry walls. To be submitted to the *Bulletin of Earthquake Engineering*, 2014.

[7] Petry, S. & Beyer, K.: Scaling unreinforced masonry for reduced-scale shake table testing, submitted to the *Bulletin of Earthquake Engineering*, 2013.

[8] Tondelli, M. & Beyer, K.: Observations on out-of-plane behaviour of URM walls in buildings with RC slabs. In: *Proc. of the 9th International Masonry Conference, Guimarães, Portugal.*

[9] Lang, K.: Seismic vulnerability of existing buildings. Doctoral Thesis. Zurich, Switzerland: ETH Zurich; 2002.

[10] Petry, S. & Beyer, K.: Influence of boundary conditions and size effect on the drift capacity of URM piers, accepted for publication in *Engineering Structures*, 2014.

[11] Paparo, A. & Beyer, K.: Modelling the seismic response of URM buildings retrofitted by adding RC walls, submitted to the *Journal of Earthquake Engineering*, 2014.

[12] Beyer, K., Petry, S., Tondelli, M. & Paparo, A.: Towards performance-based design of modern unreinforced masonry structures, submitted to the *2nd ECEES, Istanbul*, 2014.