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# Shake-table test on a four-storey structure with reinforced concrete and unreinforced masonry walls

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**Abstract.** With the introduction of higher seismic design forces in the Swiss loading standard of 2003 most unreinforced masonry (URM) buildings failed to satisfy the seismic design check. For this reason, in new construction projects, a number of URM walls are nowadays replaced by reinforced concrete (RC) walls. The lateral bracing system of the resulting structure consists therefore of URM walls and some RC walls which are coupled by RC slabs and masonry spandrels. Within the scope of a FP7-Series project a four-storey RC-URM wall structure is tested on the shake-table at the TREES laboratory of the EUCENTRE in Pavia (Italy). The test is conducted at half-scale and is part of a larger research initiative on mixed RC-URM wall systems initiated at EPFL. The key objective of the test is to gain insights into the dynamic behaviour of RC-URM wall structures and to provide input for the definition of a performance-based design approach of such mixed structural system. This paper presents details on the structural system, the instrumentation and the selected ground motion and discusses preliminary results of the shake-table test.

*Keywords: Shake-table test; Unreinforced masonry wall; Reinforced concrete wall*

## 1 INTRODUCTION

In many countries, existing URM structures are reinforced by adding RC walls or by replacing selected URM walls by RC walls. In Switzerland, new residential buildings are often directly constructed as mixed RC-URM wall structures (Figure 1) with RC slabs. Although such structures are very common in design and retrofit practice their seismic behaviour is, at present, not well understood and they have never been tested before under seismic loading.

**Figure 1.** 3- and 6-storey residential buildings with RC wall and URM walls (Photos: Thomas Wenk).

Design codes themselves present lack of guidance for such structures; neither Eurocode 8 nor the Swiss design codes provide indications on the choice of the force reduction factor q or on the definition of appropriate deformation limits. As a consequence, the current design practice in Switzerland is to neglect the effect of the URM walls on the seismic behaviour of the structure, to assign all forces to the RC walls and adopt q-factors applicable to RC walls while limiting the average interstorey drift to 0.5% in order to control the damage on the URM walls. Numerical studies on mixed RC-URM wall structures have also shown that the numerical results concerning, for example, the base shear distribution between the walls are very sensitive to the modelling assumptions and experimental results for the validation of such models are therefore needed (Paparo and Beyer, 2013). In addition, the out-of-plane vulnerability of URM piers could be the Achilles's heel of typical Swiss URM structures. Recent studies on this topic (Dazio, 2008) showed that most Swiss URM walls do not fulfil the out-of-plane slenderness criteria in Eurocode 8. It was also showed that the slenderness criteria in all international building codes fail to capture the influence of the boundary conditions on the out-of-plane seismic behaviour of URM walls.

The objective of the shake-table test is to study the seismic behaviour of mixed RC-URM buildings and particularly address several issues such as: definition and associated drift limits of different performance states, information on the stiffness degradation and damage evolution, understanding of lateral force distribution and redistribution of forces among the different structural elements, insights into displacement and acceleration profile, influence of boundary conditions on the out-of-plane behaviour of URM walls. This paper presents the characteristics of the test unit and of the applied ground motion as well as the dynamic response of the structure observed during the shake-table test.

## 2 TEST SET-UP AND DETAILS

### 2.1 Test unit

The test unit was a four-storey structure built at half-scale representing a modern residential building where most walls are constructed as URM walls and some as RC walls and where the two systems are coupled by RC slabs. The test specimen, shown in Figure 2, was composed by two RC walls and six URM walls; four of which were loaded in-plane and two out-of-plane.

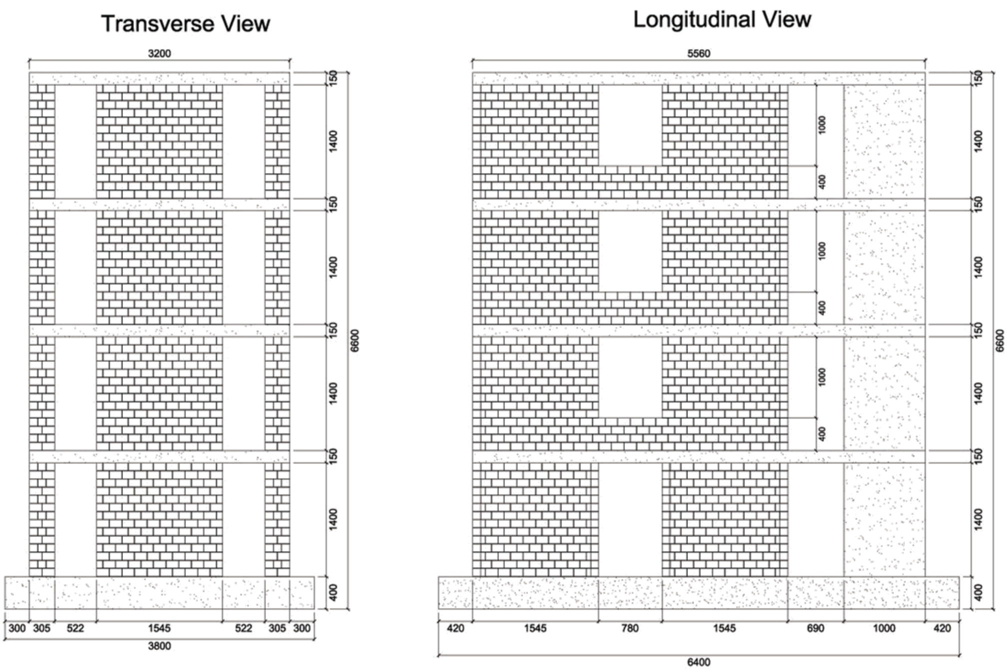


**Figure 2.** Transverse (North) and longitudinal (West) views of the test unit, dimensions are in mm.

URM walls were built using half-scale clay brick units (width 9.5 cm, length 15 cm, height 9.5 cm) produced by the Swiss brick manufacturer "Morandi Frère SA" and using a standard mortar M15. The masonry unit that were used to build the URM piers were selected after a large experimental campaign performed before the shake-table test intended to investigate the effect of scaling on the properties of the masonry. In fact while it could be shown that concrete elements, when scaled properly, show similar behaviour at reduced scale, this is often not the case for masonry elements. The experimental campaign included material tests as well as quasi-static cyclic tests on half- and full-scale masonry piers to compare their behaviour (Petry and Beyer, 2013a,b). The tests could show that a very satisfactory correspondence between half- and full-scale masonry was obtained in terms of force and deformation capacity. RC walls (thickness 10 cm) were built with a concrete of class C28/35 and the reinforcement consisted of a double layer of longitudinal steel bars of class B450C. The RC slabs (thickness 15 cm) presented a double layer of steel net reinforcement; both concrete and steel of the same class of the RC walls.

In order to enforce the scaling laws it was necessary to load the structure with additional masses, in the form of unreinforced concrete blocks, which were added directly on top of each slab. The structural masses are listed in Table 1.

**Table 1.** Summary of structural masses

| | |
|---|---|
| Foundation | 13.1 t |
| Mass of the structure | 34.9 t |
| Additional masses | 34.9 t |
| Total mass | 82.9 t |
| Total mass without foundation | 69.8 t |
| Total mass during transportation | 48.0 t |

The test unit was built outside the laboratory and was then lifted and pulled inside using hydraulic jacks and a slider system. For the transportation phase a prestress system was installed in order to assure the integrity of the structure during the transportation (Figure 3).

**Figure 3.** Test specimen during transportation.

## 2.2 Instrumentation

The motion of the shaking table and the seismic response of the test unit were monitored through a dense network of instrumentation including 20 accelerometers, 49 displacement transducers, 24 omega gages and an optical measurement system.

Four accelerometers, two recording acceleration in the longitudinal direction and two in the transverse direction, were installed on each slab. Additional four accelerometers where used to control the acceleration of the building foundation and of the shake-table.

Since the highest level of deformation was expected to concentrate at the first-storey a dense network of displacement transducers was installed there. The URM piers of the East side of the building were instrumented with wire potentiometers along the two diagonals, the potentiometers were installed in such a way to record both shear and rocking deformation of the piers. The RC pier of the same side was instrumented with wire potentiometers along the diagonal and additionally the two sides of the wall where instrumented with eight potentiometers covering the whole height of the pier.

Each URM pier loaded out-of-plane was instrumented with five potentiometers, one measuring the out-of-plane deformation at the mid-height of the panel and two measuring the internal and external vertical displacement of the top and bottom row of bricks. The out-of-plane displacement of the first storey RC slab was monitored by means of 24 omega gages installed in two different locations both on the top and bottom face of the slab.

An optical measurement system was used to measure the displacements of the West façade of the building. The motion of the structure was detected through reflecting markers glued onto the specimen whose movement is acquired by a high definition camera system.

### 2.3 Input ground motion

The source ground motion input selected for the shake-table test was the record of the Montenegro 1979 earthquake (Herceg-Novi station). The accelerogram was applied in the longitudinal direction of the structure and to account that the test specimen was built at half-scale the record was scaled in time reducing the duration by a factor of $\sqrt{2}$. The record was then scaled to match different levels of peak ground acceleration (PGA) that were used as input signal for the shake-table test (ranging from 0.05 g to 0.9 g). Figure 4 shows the acceleration history for the record scaled to a maximum PGA of 1 m/s$^2$.

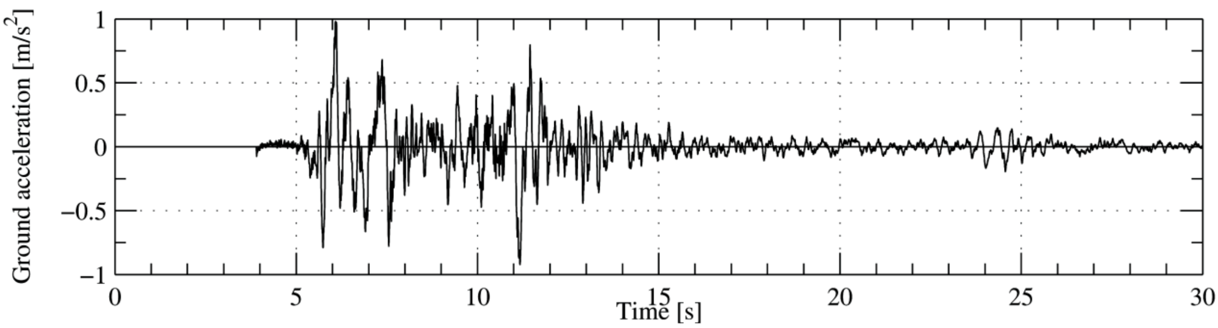


**Figure 4.** Input signal for shake-table test for a PGA of 1 m/s$^2$.

## 3 SHAKE-TABLE TEST

The shake-table test was performed at the TREES laboratory of the European Centre for Training and Research in Earthquake Engineering (Pavia, Italy). The experimental program of testing subjected the specimen to nine shakings with different levels of intensity of the excitation (PGA); Table 2 summarizes the testing chronology indicating for each test nominal values of PGA and testing dates.

**Table 2.** Summary of the nominal PGAs and testing dates

| Test Number | Nominal PGA | Testing Date |
|---|---|---|
| 1 | 0.05 g | 18.12.2012 |
| 2 | 0.1 g | 18.12.2012 |
| 3 | 0.2 g | 19.12.2012 |
| 4 | 0.3 g | 19.12.2012 |
| 5 | 0.4 g | 19.12.2012 |
| 6 | 0.6 g | 20.12.2012 |
| 7 | 0.4 g | 20.12.2012 |
| 8 | 0.7 g | 20.12.2012 |
| 9 | 0.9 g | 21.12.2012 |

## 3.1 Test results

The experimental campaign required four days of testing; after each shake-table test a survey of the specimen was performed in order to report the evolution of the structural damage. Moreover before each test a structural identification was performed to estimate how the dynamic characteristics (fundamental period of vibration) of the structure changed after every single shaking.

The first three tests, characterized by a low intensity of shaking, induced a very limited level of damage to the structure. The formation of residual hairline cracks in the URM piers of the first storey was observed; the latter were mainly concentrated at the corners of the panels. After test number 4 and 5 the formation of first hairline cracks in the URM piers of the second storey and in the RC walls and slab of the first storey was observed. The URM panels of the first storey showed diagonal cracks along the whole height; however the level of residual deformation was still limited.

The structure started showing a more severe level of damage after test number 6. At this point URM piers at all the storeys were showing diagonal cracks. At the first storey the pre-existing cracks started widening up to a crack width of 0.8 mm, nevertheless in the majority of cases crack widths were still less than 0.2 mm (Figure 5).

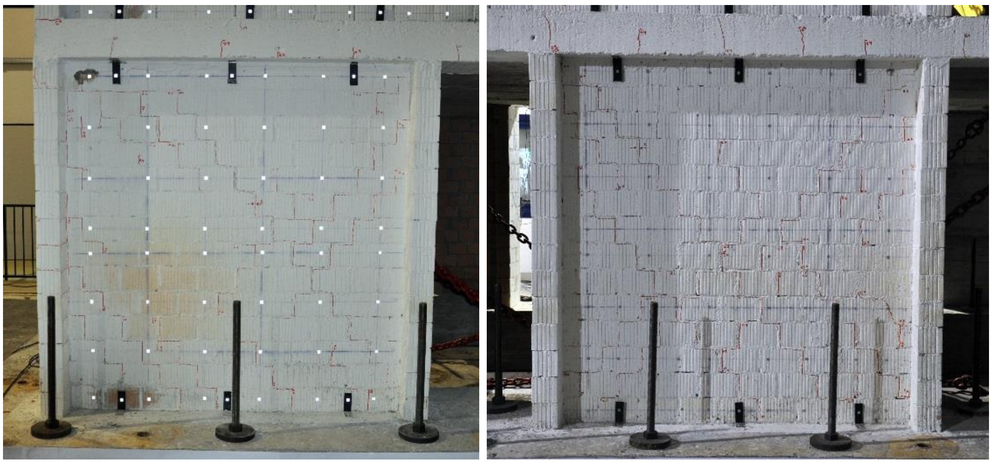

**Figure 5.** Crack propagation on the URM piers of the first storey after test number 6.

Test number 7, performed with a lower level of intensity with respect to the previous, was meant to simulate a possible aftershock during an earthquake event. The survey of pre-existing cracks and their width indicated that the aftershock provoked a very little additional damage to the specimen.

The structure showed a severe level of damage after test number 8 as the deformations in the URM piers of the first and second storey started concentrating in a single diagonal with an average crack residual deformation around 0.8 mm. Additionally for the first time cracks passing through bricks and not only in the mortar joints were observed.

The final test at 0.9 g brought the structure rather close to its collapse limit state as the four in-plane loaded URM piers of the first and second storey lost their axial load bearing capacity. The axial load was finally carried by the URM piers in the out-of-plane direction and by the two RC walls. Immediately after the test it was necessary to install steel props to support the first storey slab to prevent the collapse of the structure (Figure 6).

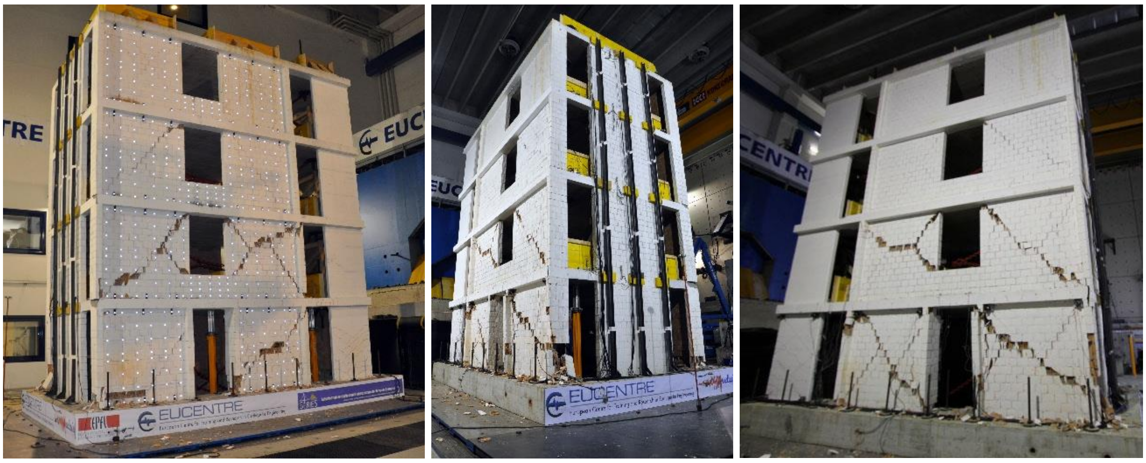


**Figure 6.** Different views of the structure after test number 9.

An interesting feature showed by the specimen was that the two lower storeys experienced the same level of damage (Figure 7), differently from what is expectable for a URM structure were the damage is mainly concentrated at the first storey. This showed how the addition of RC walls to the URM structure influenced the seismic behaviour as similar levels of drift were attained for the two lower storeys.

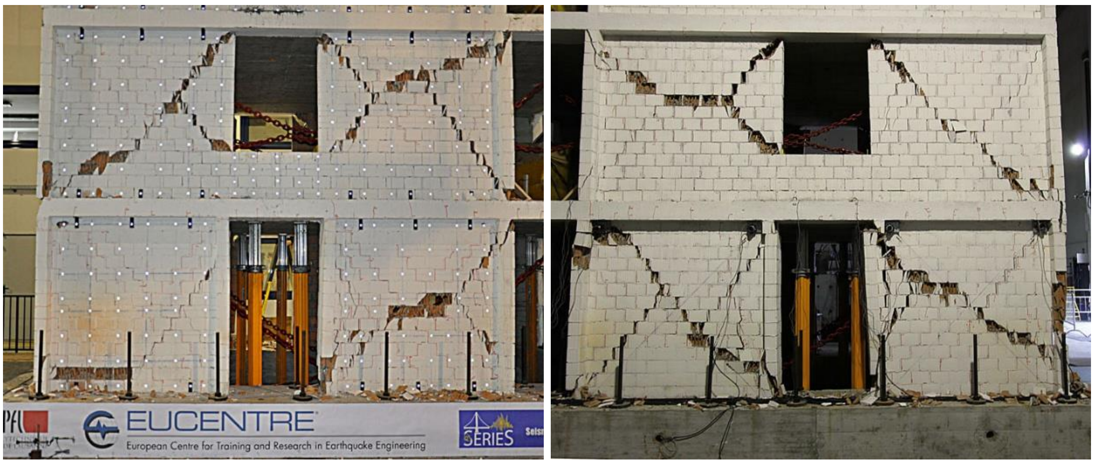


**Figure 7.** URM piers of the first and second storey after test number 9.

The failure mechanism of the URM piers was characterized by the crushing of the compressed diagonal at the corners and at the mid height where the two diagonals intersected. Large residual deformations were observed which were associated with the sliding down of the top part of the piers along the diagonal cracks (Figure 8). Differently from the URM piers the spandrels at all storeys remained uncracked.

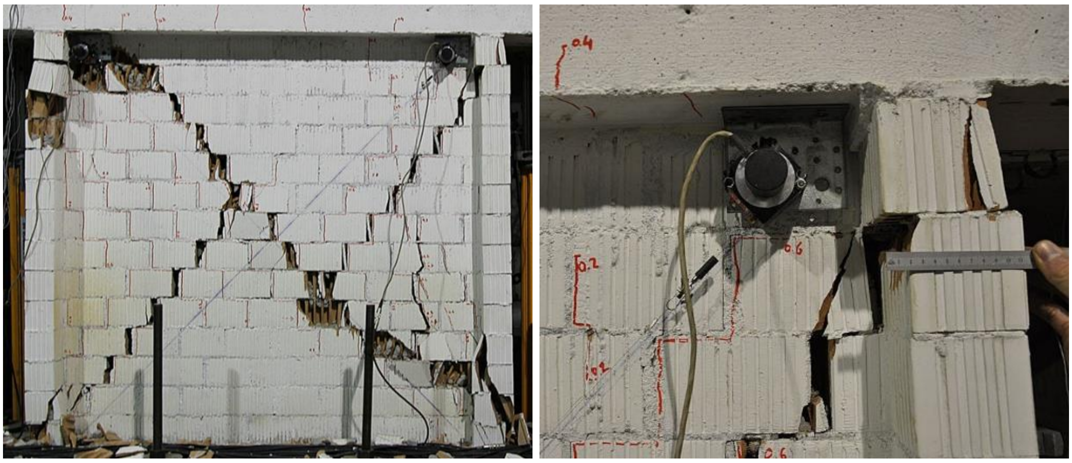

**Figure 8.** Residual deformations of one URM piers after test number 9.

RC walls had a different behaviour depending on the loading direction, when the structure was pushed toward North the walls were in tension and therefore the South side of the walls developed horizontal flexural cracks along all the height. The latter showed a residual width of approximately 1 mm indicating that the longitudinal reinforcement had yielded (Figure 9). When the structure was instead pushed in the South direction the RC walls were in compression and remained largely uncracked; the few cracks which developed had a residual width close to zero indicating that in this direction the longitudinal reinforcement did not yield.

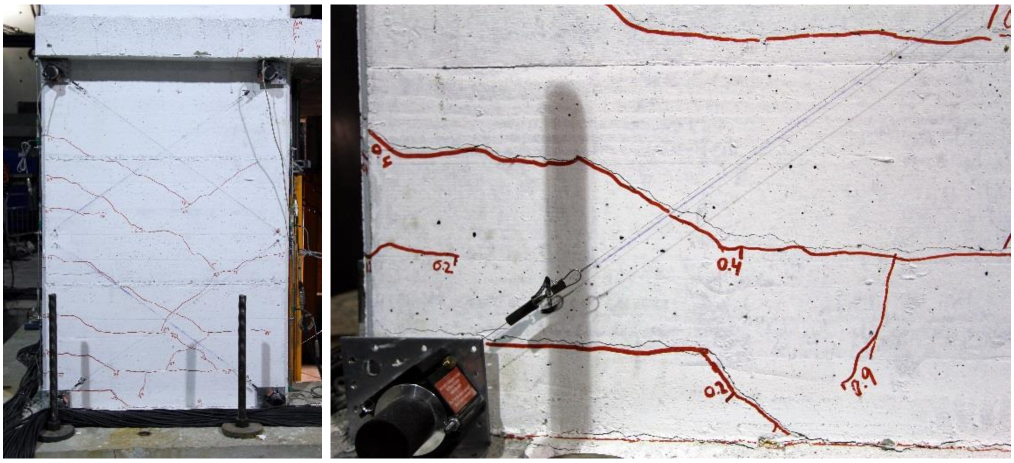

**Figure 9.** Cracking patter of the RC wall of the East façade after test number 9.

During the last test, for the first time, a clear out-of-plane movement of one of the URM piers in the transverse direction was observed. The fourth storey panel of the North face of the building showed a large deformation during the shaking and touched the steel beams which had been installed to prevent a possible out-of-plane collapse. After the test clear cracks over the entire pier length were observed at midheight and at the top and bottom of the panel (Figure 10). As expected the boundary conditions provided by the RC slab and lateral walls influenced the seismic behaviour of the out-of-plane wall since the transverse wall of the opposite façade did not show any type of out-of-plane deformation and no cracks in the horizontal joints after the test. For the future numerical investigations of the boundary conditions of out-of-plane loaded URM walls are planned.


**Figure 10.** Out-of-plane failure of URM wall: horizontal cracks at mid height and on top of the wall.

## 4 CONCLUSIONS

The shake-table test on the four storey mixed RC-URM wall structure constructed at half-scale allowed investigating the performances of such a structure for a large range of ground motion intensities. The first runs with small peak ground accelerations caused only a limited level of damage in the structure mainly concentrated in URM piers of the first storey. The final run on contrary, brought the structure very close to collapse as the in-plane loaded URM piers of the bottom two storeys lost their axial bearing capacity. During all the tests about 90 hard-wired instruments were recorded measuring local and global quantities. Additionally an optical measurement system was employed allowing the possibility of tracking the position of approximately 300 points on the URM piers of the West face of the structure. During the shaking the structure showed that the presence of RC walls highly influences the performances of URM walls. While in a masonry building the damage would be mainly concentrated in the first storey the mixed structure has shown a more evenly spread level of damage as the two lowest storeys were subjected to a similar drift demand. The drift demand in the two higher storeys was significantly lower but anyway sufficient to cause cracks. The last test also allowed to observe the out-of-plane failure of one URM pier in the transverse direction underlining the influence of boundary conditions on seismic behaviour of out-of-plane loaded piers. The data recorded during the test will help addressing different open issues related to the seismic behaviour of mixed RC-URM buildings.

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