ESECMASE – SHAKING TABLE TESTS AT THE NATIONAL TECHNICAL UNIVERSITY IN ATHENS

UDO MEYER

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SUMMARY

This paper presents preliminary results of shaking table tests on two-storey high specimens representing cut-outs of typical terraced houses in low seismicity regions in Central Europe.

INTRODUCTION

As one part of the European research project ESECMaSE, a series of shaking table tests have been carried out at NTU Athens (Carydis et al. 2007). The tests were foreseen as preliminary studies for the pseudo-dynamic test of a whole masonry structure to be carried out at the European Joint Research Centre in Ispra/Italy (Anthoine 2008). The investigations at NTU Athens were started with three tests on calcium silicate masonry and continued with three tests on clay unit masonry. The test series will be concluded with one test on a concrete masonry unit structure. This paper presents a brief overview of the tests on calcium silicate and clay unit masonry. The final report will be available in the beginning of 2008.

TEST SET-UP AND DESCRIPTION OF THE SPECIMENS

The shaking table at NTU Athens has a ground plan dimension of 4 m x 4 m and is suitable for specimens with a maximum weight of 100 kN. The maximum displacement of the platform is ± 100 mm.

The investigations were carried out using two storey high specimens representing a cut-out of a terraced house adopted to the dimensions of the available shaking table (see figure 1). Figure 2 shows two of the tested specimens.

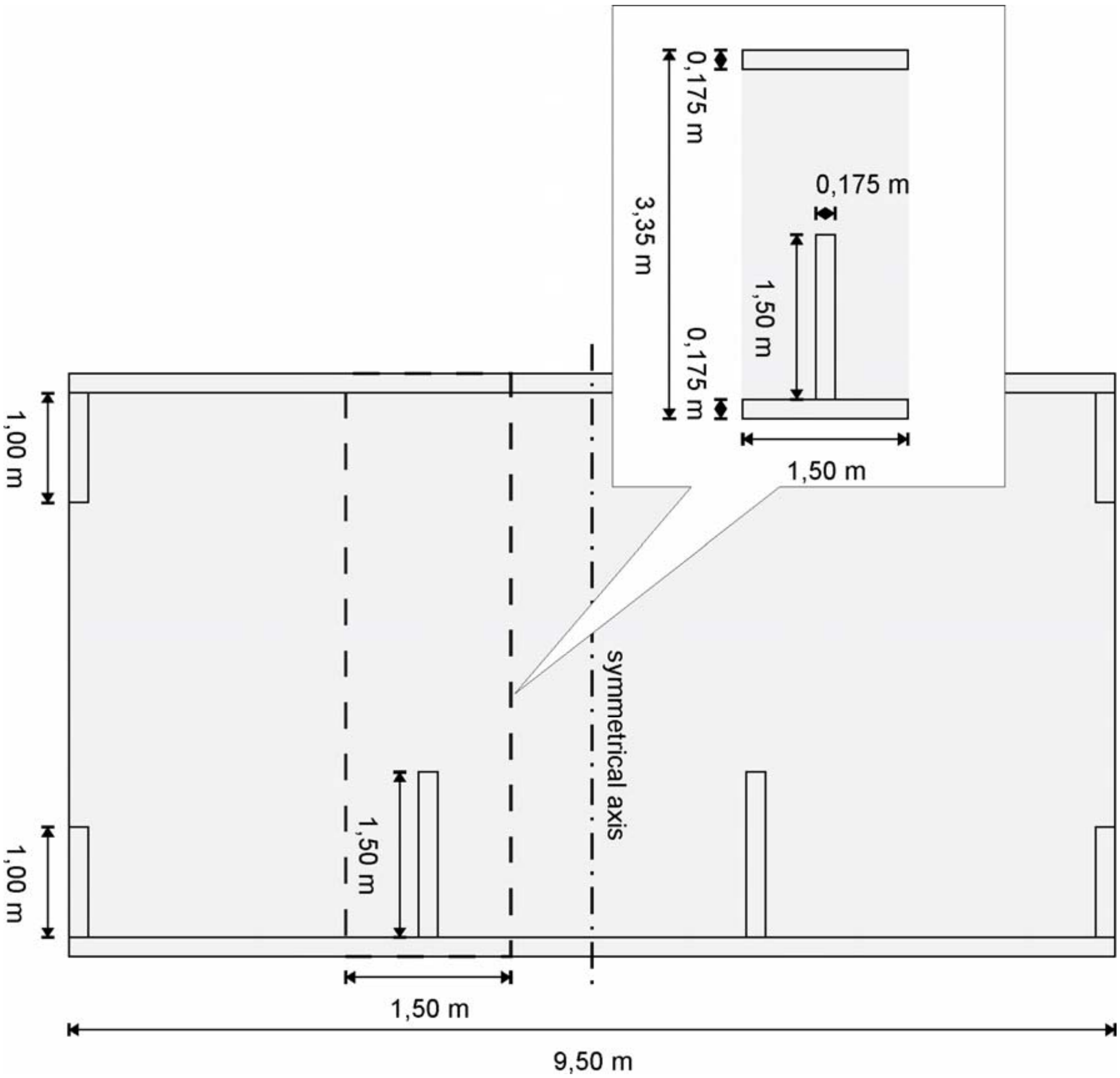

Figure 1. Ground plan of a typical two storey terraced house in Central Europe and cut-out of the tested specimen

The thickness of all walls, shear walls and transverse walls was 175 mm. The cross section of the shear wall in earthquake direction was 5,2% of the total floor area. The tests on calcium silicate masonry were carried out with KS-R P 20-1,8 6 DF[1] (Series A1) and KS XL-RE 20-1,8[2] (Series A2 and A3) with one type of thin layer mortar. The shear walls of the clay specimens were constructed with precision units (optimised HLz 12[3] (Series B1) and hollow clay units for concrete infill (Series B2 and B3)) with thin layer mortar. The infill units were filled with laboratory mixed concrete C20/25. In all Series mortar was only applied to the bed joints. The head joints of the units remained unfilled, something which usually is not

[1] denomination according to German DIN V 106 for a calcium silicate with tongues and grooves for use with thin layer mortar, complying with strength class 20 (mean compressive strength ≥ 25 N/mm²) and gross dry density class 1,8 (1610 kg/m³ ≤ ρ_d ≤ 1800 kg/m³). The standard size 6 DF corresponds to a unit size of length x width x height = 248 mm x 175 mm x 248 mm.

[2] denomination according to German DIN V 106 for a calcium silicate unit (element) with tongues and grooves for use with thin layer mortar, complying with strength class 20 and gross dry density class 1,8 (see above for class definitions). The unit (element) size is length x width x height = 498 mm x 175 mm x 498 mm.

[3] denomination for low density, vertically perforated clay unit complying with strength class 12 according to German DIN V 105-100 (mean compressive strength ≥ 15 N/mm²).

being regarded as appropriate for masonry structures in earth quake zones, but represents today's way of building masonry in central Europe, also in earthquake prone zones.

Figure 2. View of two specimens, clay units type HLz 12 (left) and calcium silicate units type KS-R P 20-1,8 6 DF (right)

Some relevant properties of the units are given in table 1.

Table 1. Type of unit, compressive strength perpendicular to the bed joint f_b, compressive strength parallel to the bed joint f_{bl}, splitting tensile strength $f_{t,sp}$

Type of unit	dry density kg/m³	percentage of voids	f_b	f_{bl}	$f_{t,sp}$
			N/mm²		
KS-R P 20-1,8 6 DF	1760	≈ 0	21,5	Not tested	Not tested
KS XL-RE 20-1,8	1770	6,7	20,5	Not tested	Not tested
PFz 10 (hollow clay unit for concrete infill)	750*)	46,1	13,6*)	Not tested	0,70*)
Optimised HLz 12	850	43	16,9	4,8	0,83

*) without concrete infill

Only the shear walls of Series A3 and B3 were vertically reinforced with one rebar ∅ 16 mm at both ends, which were connected end-to-end with the bottom slab (represented by the steel frame) and the concrete slab on top of the first floor and therefore acting as a frame (see

figure 3). All other walls and in particular the specimen of Series A1, A2, B1 and B2, were built of unreinforced masonry.

Live loads on the floors were simulated using steel-plates tightly connect to the floors to avoid displacement and vibration of the live loads during the shaking table test. For test Nr. A3, the first one to be carried out, a load of 7,0 t has been placed on the ceiling of the ground floor as there were fears that an equal distribution of the “live loads” on both ceilings might cause an overturning moment that might damage the vertical actuators of the shaking table.

These fears turned out not to be reasonable so that the live loads for the next tests were distributed, placing 3,5 tons on the first floor and 4 t above the top slab. Table 2 gives an overview about the tested variations.

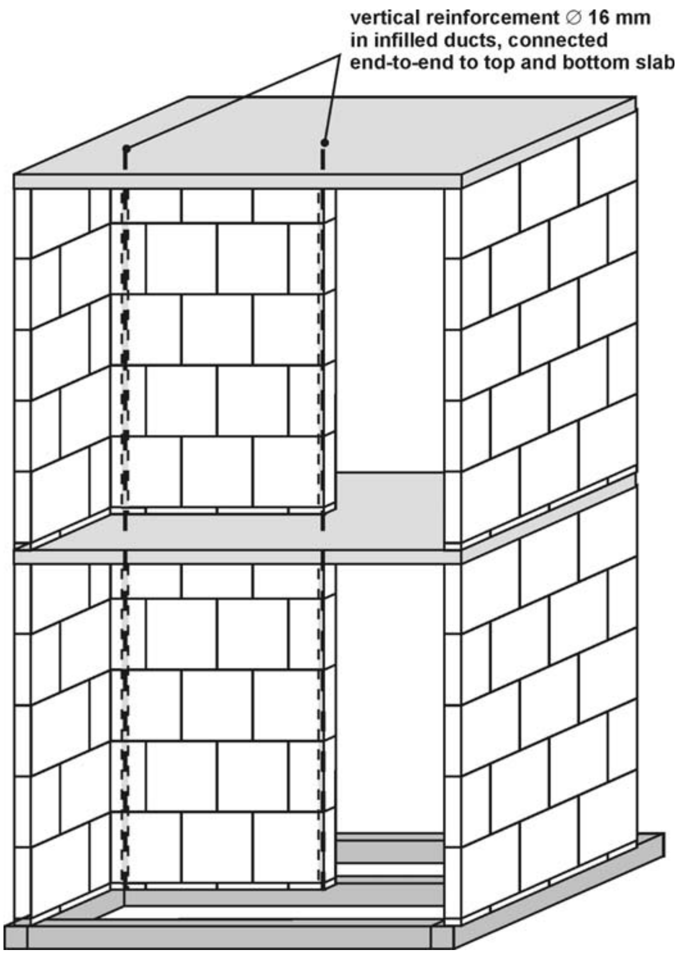

Figure 3. Configuration of vertical reinforcement exemplary shown for Series A3 (calcium silicate units KS XL-RE 20-1,8

Table 2. Shaking table tests at NTU Athens, tested variations, additional loads on first floor P1 and top slab P2, maximum nominal base acceleration max a_g

<table>
<tr><th rowspan="2">Specimen Nr.</th><th rowspan="2">Masonry units in the shear wall</th><th rowspan="2">Reinforcement</th><th>G</th><th>P1</th><th>P2</th><th>max a_g</th></tr>
<tr><th colspan="3">T</th><th>m/s²</th></tr>
<tr><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td><td>6</td><td>7</td></tr>
<tr><td>A 1</td><td>KS-R P 20-1,8 6 DF</td><td rowspan="2">none</td><td rowspan="3">10,1</td><td rowspan="2">3,5</td><td rowspan="2">4,0</td><td>1,6</td></tr>
<tr><td>A 2</td><td rowspan="2">KS XL-RE 20-1,8</td><td>1,6</td></tr>
<tr><td>A 3</td><td>2 x 1 ∅ = 16 mm in the external infill ducts</td><td>7,0</td><td>-</td><td>3,0</td></tr>
<tr><td>B 1</td><td>Optimised HLz12</td><td rowspan="2">none</td><td>6,3</td><td rowspan="3">3,5</td><td rowspan="3">4,0</td><td>1,8</td></tr>
<tr><td>B 2</td><td rowspan="2">PFz 10
Hollow clay units with concrete infill, filled with concrete C 20/25</td><td rowspan="2">10,8</td><td>1,6</td></tr>
<tr><td>B 3</td><td>2 x 1 ∅ = 16 mm in the external infill ducts</td><td>2,6</td></tr>
</table>

TEST PROCEDURE

The tests were carried out using an elastic response spectrum type 1 according to EN 1998-1 for ground type B (Figure 4). The structures were subjected to simulated earthquakes (Figure 5). The nominal base acceleration of these earthquakes was increased in steps of 0,02 g, starting with an acceleration of 0,02 g. The acceleration was increased up to a failure of the structure respectively up to the last cycle to be regarded safe by the laboratory staff with regards to laboratory equipment.

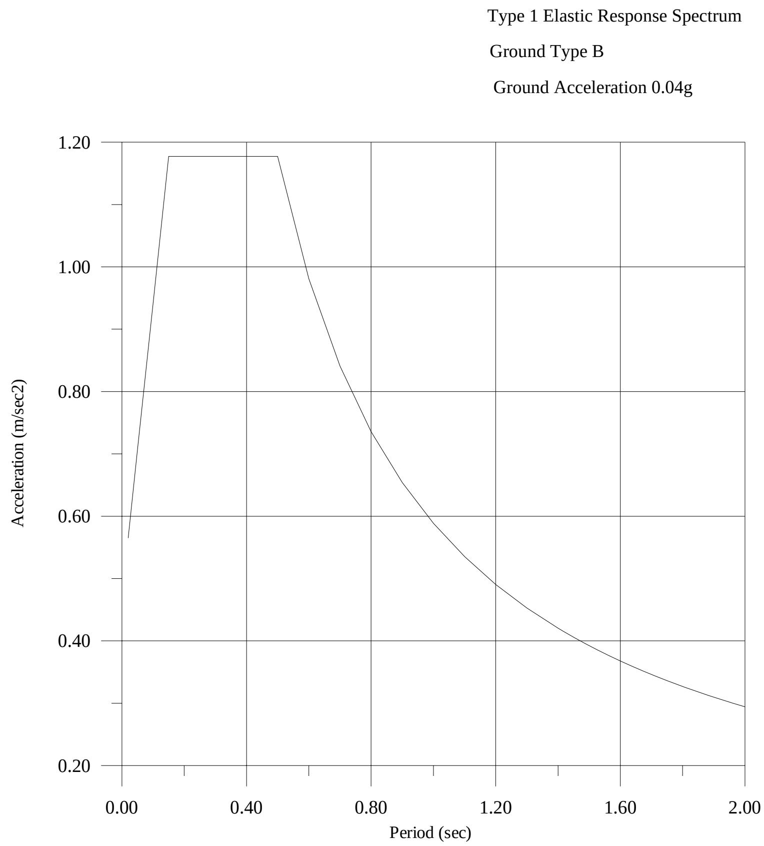

Figure 4.Elastic Response Spectrum Type 1 according to EC8.

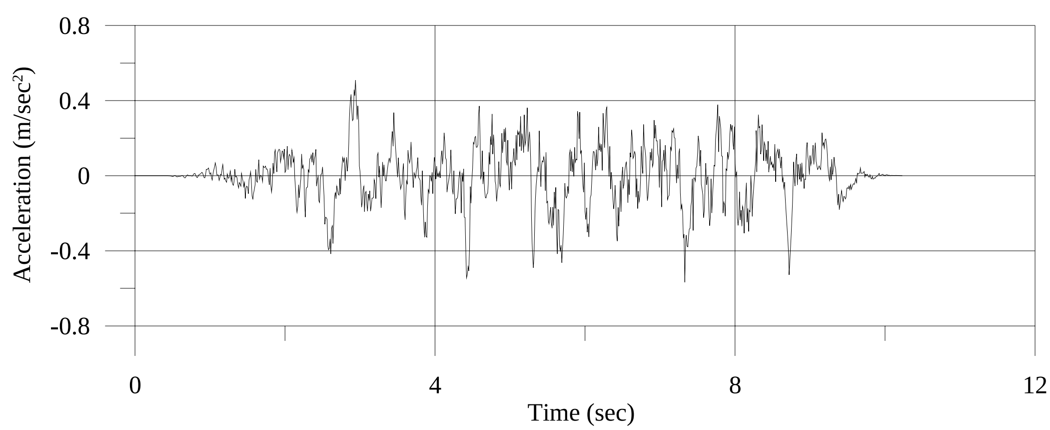

Figure 5. Artificial time history generated to match Elastic response Spectrum Type 1.

In a first step, the natural frequencies and the damping ratio of the specimens were determined, using a random acceleration between DC and 50 Hz. The amplitude of vibration was 0,02 g.

Table 3 gives the determined natural frequencies and damping ratios of the specimen.

Table 3. Natural frequency and damping ratio of the specimens

Specimen Nr.	Natural frequency	Damping ratio
	Hz	%
A1	3,71	4,37
A2	3,91	3,96
A3	4,98	3,46
B1	4,10	2,43
B2	4,39	2,17
B3	4,20	5,19

TEST RESULTS

All tested specimens withstood the nominal acceleration a_g = 0,8 m/s² (≅ 0,08 g), the limit value of the design ground acceleration for low seismicity regions from EN 1998-1, without cracks. Cracking and failure occurred only after the application of much higher ground accelerations.

Table 4 gives the base shear forces and the relative displacements of the ceilings above the storeys for a nominal ground acceleration of 0,08 g. The values for test A3 are still not available and therefore missing in table 2. The base shear and the relative displacements for all specimens vary only to a small extent. This small variation can be assumed to be in the range of scatter of test results.

Table 4. Base Shear and relative ceiling displacements D_{GF} (ceiling above ground floor) and DG_{1FL} (ceiling above first floor) respectively for a nominal ground acceleration of nom a_g = 0,08 g

<table>
<tr><th rowspan="2">Specimen Nr.</th><th rowspan="2">Masonry units in the shear walls</th><th>nom a_g</th><th>Base Shear</th><th>D_{GF}</th><th>D_{1FL}</th></tr>
<tr><th>m/s²</th><th>KN</th><th colspan="2">mm</th></tr>
<tr><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td><td>6</td></tr>
<tr><td>A1</td><td>KS-R P 20-1,8 6 DF</td><td rowspan="5">0,8</td><td>27,7</td><td>3,5</td><td>8,1</td></tr>
<tr><td>A2</td><td>KS XL-RE 20-1,8</td><td>28,2</td><td>8,1</td><td>7,5</td></tr>
<tr><td>B1</td><td>Optimised HLz 12</td><td>25,2</td><td>5,0</td><td>7,6</td></tr>
<tr><td>B2</td><td>PFz 10
Hollow Clay Units for concrete infill</td><td>28,3</td><td>6,0</td><td>8,0</td></tr>
<tr><td>B3</td><td>PFz 10
Hollow clay units for concrete infill with reinforcement of the external ducts</td><td>26,9</td><td>6,1</td><td>8,0</td></tr>
</table>

The failure of the specimens was not in all cases caused by a shear wall failure but, especially in the case of the calcium silicate specimens by a failure of the transverse walls on first floor.

The failure was caused by an uplift of the ceiling due to the shear wall displacement and a resulting lack of vertical load on the transverse walls, especially on first floor. The transverse walls were consequently free cantilevers flattering and colliding with the shear walls. This failure mode is not likely to occur in real building situations as the uplift of the ceiling will be restricted to a limited area and the remaining length of the transverse walls will restrain the flattering tendencies in the area of the shear wall.

The maximum nominal ground accelerations and the head displacements for the two storeys in all tests are given in table 5. The maximum ground accelerations and the maximum base shear for the unreinforced specimens A1, A2, B1 and B2 are in the same range, regardless of the differences especially in the longitudinal compressive strength, so far being regarded as one of the decisive influencing parameters for the earthquake resistant layout of masonry units. The results of the specimens with vertical frame reinforcement show that this measure strengthens the shear walls considerably compared to unreinforced specimens.

Table 5. nominal maximum ground acceleration nom $a_{g,max}$, Base Shear and head displacement D_{GF} (ground floor) and DG_{1FL} (first floor) respectively

Specimen Nr.	Masonry units in the shear walls	nom $a_{g,\,max}$	Base Shear	D_{GF}	D_{1FL}
		m/s²	kN	Mm	
1	2	3	4	5	6
A1	KS-R P 20-1,8 6 DF	1,6	37,4	13,5	35,2
A2	KS XL-RE 20-1,8	1,6	39,2	35,3	35,1
A3	KS XL-RE 20-1,8 with reinforcement in the external ducts	3,0	74,4	15,4	28,9
B1	Optimised HLz 12	1,8	36,1	29,0	53,4
B2	PFz 10 Hollow clay units for concrete infill	1,4*)	38,3	87,9	18,3
B3	PFz 10 Hollow clay units for concrete infill with reinforcement in the external ducts	2,4**)	67,7	25,2	45,7

*) Specimen subjected to a maximum ground acceleration of 1,6 m/s², evaluation of this test cycle not yet completed

**) Specimen subjected to a maximum ground acceleration of 2,6 m/s², evaluation of this test cycle not yet completed

All tested specimens were capable to carry the applied vertical load after the last test cycle. Figure 6 shows an overview and the ground floor shear wall of specimen Nr. B3 (hollow clay units with frame reinforcement) after the test cycle with a nominal ground floor acceleration of nom a_g = 2,6 m/s². Significant cracking only occurred in the ground floor walls, the walls in the storey above are more or less uncracked.

Figure 6 Specimen B3 (shear wall hollow clay units with concrete infill and frame reinforcement) after the cycle with a nominal ground acceleration of 2,6 m/s²
Overview (left) and shear wall on ground floor (right)

Figure 7. Specimen A1 after a simulated earthquake with a nominal ground acceleration of 1,6 m/s²; Overview (left) and shear wall on ground floor (right)

Figures 7 and 8 show the specimens A1 and A2, both specimens without vertical reinforcement after the cycle with a nominal ground acceleration of $a_{g,max}$ = 1,6 m/s². Figure 7 shows the failure of the upper transverse wall described above, caused by a collision of this transverse wall with the shear wall in first floor. The cracks in the shear walls followed the joints, the calcium silicate units were more or less undamaged, see figures 7 and 8 (right).

Figure 8. Specimen A2 after a simulated earthquake with a nominal ground acceleration of 1,6 m/s²; Overview (left) and shear wall on ground floor (right)

CONCLUSIONS

All tested specimens withstood the maximum design accelerations for low seismicity regions from Eurocode 8-1 (EN 1998-1) without cracking. The recorded corresponding base shear varied only in the assumed range for the scatter of test results.

Significant differences between the maximum base shear of the unreinforced specimens were not observed at the moderate level of applied loads, although the tested longitudinal compressive strength of the masonry units differed significantly. Based on these test results, variations A1 and B2 have been chosen for the full scale pseudo-dynamic tests at Ispra.

REFERENCES

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