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SHAKING TABLE TEST ON A FULL-SCALE UNREINFORCED CLAY MASONRY BUILDING WITH FLEXIBLE DIAPHRAGMS

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ABSTRACT

This paper presents the results of an experimental campaign which is part of a wider research project, aimed at assessing the vulnerability of buildings in the Groningen region of the Netherlands. This area, historically not prone to tectonic ground motions, has been subjected to seismic events induced by gas extraction during the last two decades. As part of this project, a unidirectional shaking table test was performed on a single-story, full-scale, unreinforced masonry building. The specimen represented a detached pre-1940's house, consisting of clay unreinforced masonry walls, without any specific seismic detailing. The building was designed to include large openings and a reentrant corner, causing a discontinuity in one of the perimeter walls. The first floor was made of timber beams and planks, resulting in a flexible diaphragm. The roof, characterized by a very steep pitch, consisted of a series of timber trusses connected by wood purlins and boards. The two façades perpendicular to the shaking direction were designed in order to represent two typical gable geometries. An incremental dynamic test was performed up to the near-collapse condition of the specimen, using input ground motions selected to reproduce a realistic scenario of seismic events in the examined region. This paper summarizes the main characteristics of the specimen and the shaking table experimental results, illustrating the dynamic response of the structure and the evolution of the damage mechanisms.

KEYWORDS: *clay URM, flexible diaphragms, full-scale building, shaking table test*

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INTRODUCTION

The Groningen region of the northern Netherlands, historically not prone to tectonic earthquakes, during the last two decades has been interested by seismic events induced by gas extraction and reservoir depletion. The most severe event was a M_L 3.6 earthquake that occurred on August 16th, 2012 near Huizinge, above the central area of the Groningen gas field [1]. Local structures, not specifically designed for seismic actions, have been exposed to low-intensity shakings during this period, with unreinforced masonry (URM) buildings representing almost 90% of the building stock.

Because of the limited available information on the seismic performance of Dutch building typologies, an experimental campaign has been launched in 2015, aimed at investigating the performance of structural components, assemblies, and systems. The experimental program includes in-situ mechanical characterization tests [2] and laboratory tests, such as: (i) characterization tests on bricks, mortar and small masonry assemblies; (ii) in-plane cyclic shear-compression tests [3] and dynamic out-of-plane tests on full-scale masonry piers [4]; and (iii) full-scale unidirectional shaking table tests on different URM building typologies.

With the aim of investigating the seismic behaviour of pre-1940's clay URM detached houses up to near collapse conditions, an incremental dynamic test was carried out on a prototype building at the EUCENTRE laboratory in Pavia, Italy in 2016 [5]. This typology represents a large portion of the URM building stock of the Groningen region and comprises commonly one-or two-story buildings with irregular plan configurations, wide openings, and flexible floor and roof diaphragms. Most detached houses are characterized by steep pitched roofs, with several combinations of external roof shapes and gable geometries.

This paper describes the geometric and mechanical characteristics of the specimen, the testing protocol, and the experimental results in terms of damage evolution and hysteretic response.

SPECIMEN OVERVIEW

Specimen geometry

The building specimen was designed to represent a pre-1940's clay URM detached house of the Groningen region (Figure 1a and b) and was built at full scale on the shaking table of the EUCENTRE laboratory. Even though it was not expected to be exhaustive of all possible geometric variations of the local building stock, the specimen was designed to include large asymmetrical openings on all sides and a reentrant corner, causing a discontinuity in one of the perimeter walls (Figure 1 and Figure 2). The load-bearing structural system consisted of 208-mm-thick, solid URM walls, supported by a composite steel-concrete foundation.

The floor and roof diaphragms were flexible, as timber floors and roofs are mostly found in this building typology. The roof external shape was designed to combine two different end geometries: a half-hipped roof with clipped gable at the North façade and a full-height gable at the South façade (Figure 1 and Figure 2). The perimeter walls extended above the first floor to form the 208-mm-thick gables. These elements are generally more vulnerable when subjected to out-of-plane excitation because of weak connections to the roof framing along this direction: for this reason, the unidirectional shaking table test was performed perpendicularly to the gables, as shown by the arrows on Figure 1.

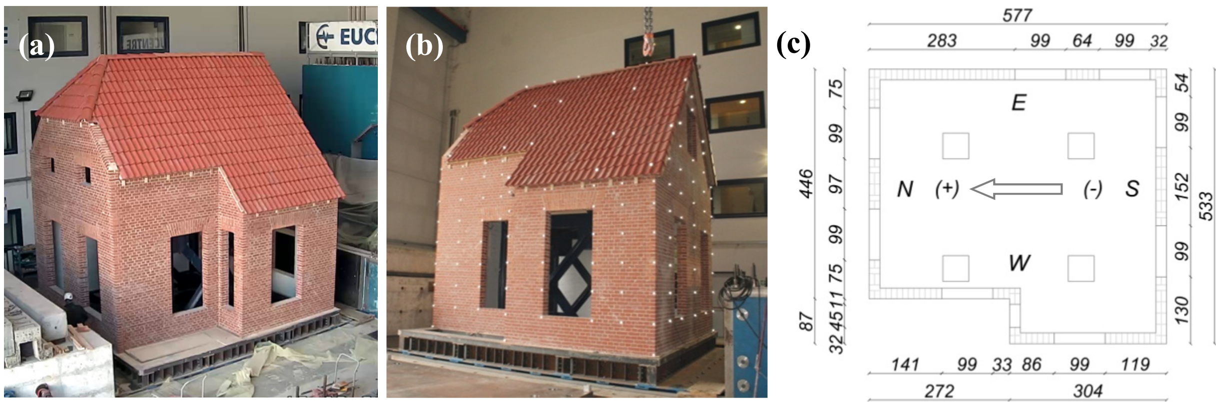

Figure 1: Full-scale specimen: (a) N-W view; (b) S-W view; (c) 1st-floor plan (units of cm)

Figure 2: Elevation views of the specimen (units of cm)

Construction details

Construction details of the specimen were developed to represent the Dutch construction practice common before the 1940s. The Dutch cross brickwork bond (Figure 3a) was adopted for the masonry bearing walls.

Lintels were built above all openings (Figure 3b and c). They consisted of a 100×50 mm timber beam below the interior masonry wythe, extending into the masonry 100 mm on each side of the opening for support. A 300-mm-high brick flat arch was built below the exterior masonry wythe with the brick stretchers facing outwards.

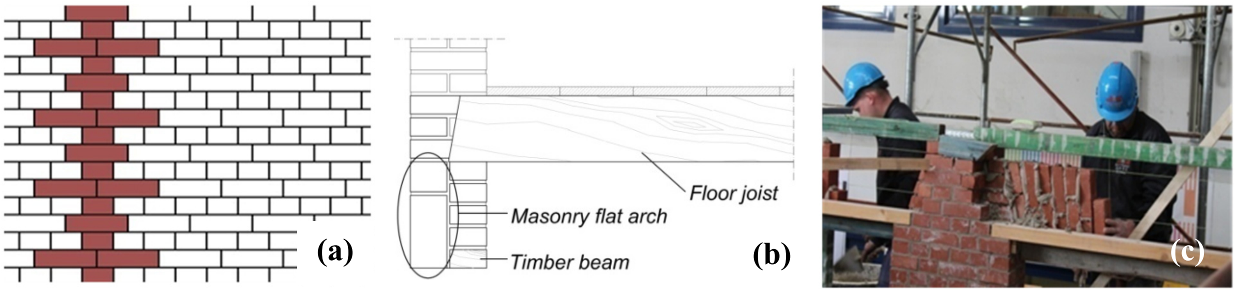

Figure 3: Construction details of the test-building: (a) Dutch cross bond scheme; (b) section of the lintel; (c) construction of the lintels

The floor system consisted of 200×24 mm timber (spruce) floorboards, nailed perpendicularly to ten 80×180 mm timber joists spanning continuously between the East and West URM walls (Figure 4). The joist ends were cut at an 80° angle (Figure 5) and were supported on the interior wythe of the longitudinal walls at a height of 2700 mm above ground.

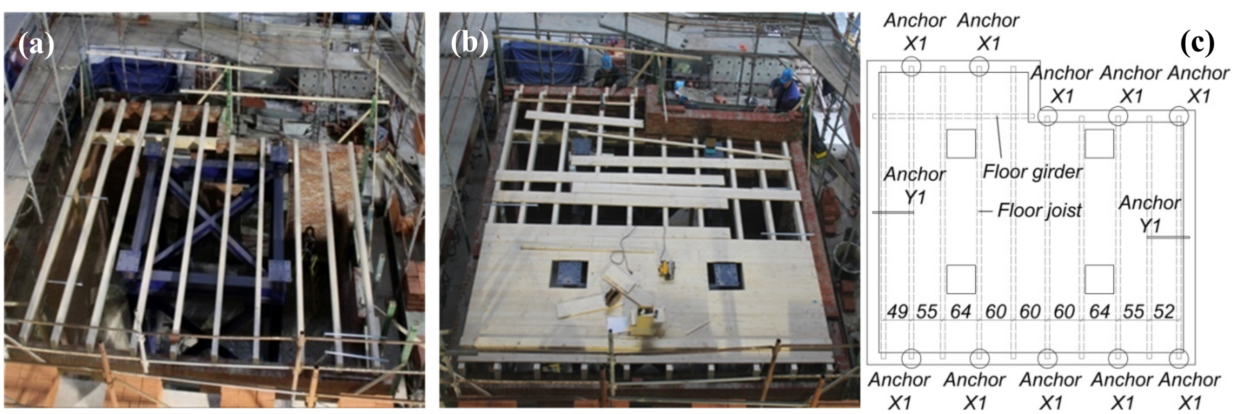

Figure 4: First floor framing: (a) floor joists during construction; (b) floorboards during construction; (c) floor framing plan (units of cm)

Connection between the floor diaphragm and the East and West walls was provided by 14-mm-diameter L-shaped steel anchors (labeled X1 on Figure 4c and Figure 5), screwed to the timber joists and embedded into the masonry between the two wythes (Figure 5a, b, d, and e). Flat S-shaped steel connectors (labeled Y1 on Figure 4c and Figure 5) provided wall-to-diaphragm connection for the North and South façades, restraining these walls against out-of-plane overturning mechanisms. These anchors, located at mid-span of each façade (Figure 4c), ran below the timber floorboards and were screwed to the first two floor joists from the restrained walls (Figure 5c and f). One laid-across brick per side was modified in order to fit the anchor in the brickwork.

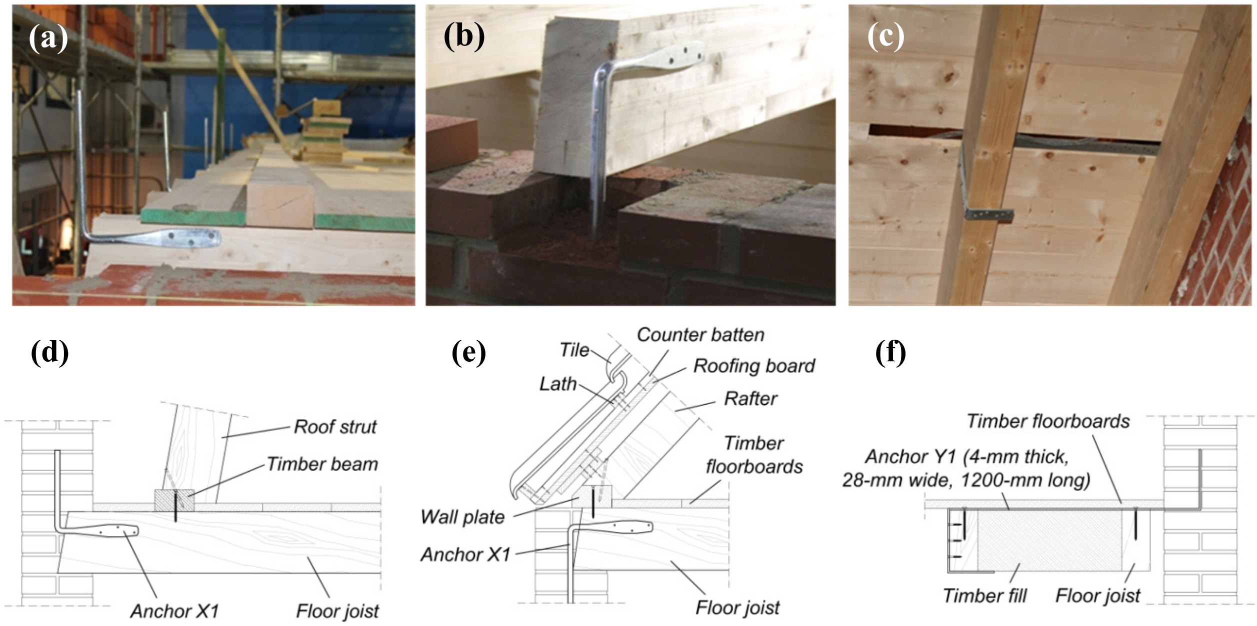

Figure 5: Wall-to-diaphragm connections: (a, d) East wall; (b, e) West wall; (c, f) North and South walls

The roof structure consisted of four East-West timber trusses, supporting longitudinal North-South purlins and a ridge beam (Figure 6). The truss rafters were connected to wall plates above the longitudinal East and West walls and above the North clipped gable. The longitudinal wall plates were screwed to a series of gutter beams (recessed into the masonry) and placed above a mortar layer (Figure 7a). At the North roof-gable interface the wall plate was nailed to three gutter beams, without mortar above the bricks (Figure 7b); this configuration was expected to accommodate relative displacements between the roof and the top of the clipped gable.

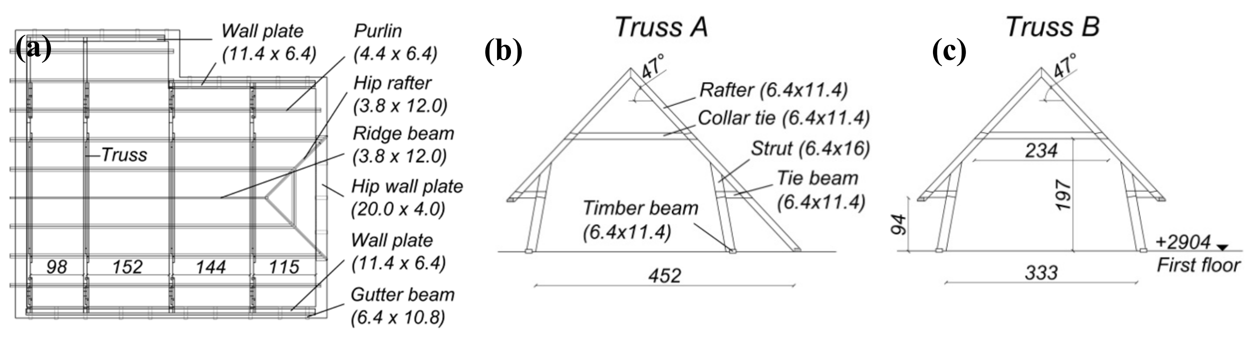

Figure 6: Roof framing: (a) roof framing plan; (b) roof truss spanning between the East and the outermost West walls; (c) roof truss spanning between the East and the reentrant West walls (units of cm)

A truss was placed back-to-back with the South gable, where the roof purlins extended through and protruded 100 mm beyond the masonry gable (Figure 7c). This resulted in a very small fraction of gravity load transmitted to the South gable under static conditions. Two planks were nailed to the purlins outside the gable (Figure 7d), forming an L-shaped end-block which restrained the relative displacement between gable and roof due to gable out-of-plane mechanisms. 18-mm-thick×200-mm-wide timber boards were mounted perpendicularly to the purlins above the roof framing. The roof was completed with clay tiles, supported by a mesh of laths and counter battens nailed above the timber boards (Figure 7e).

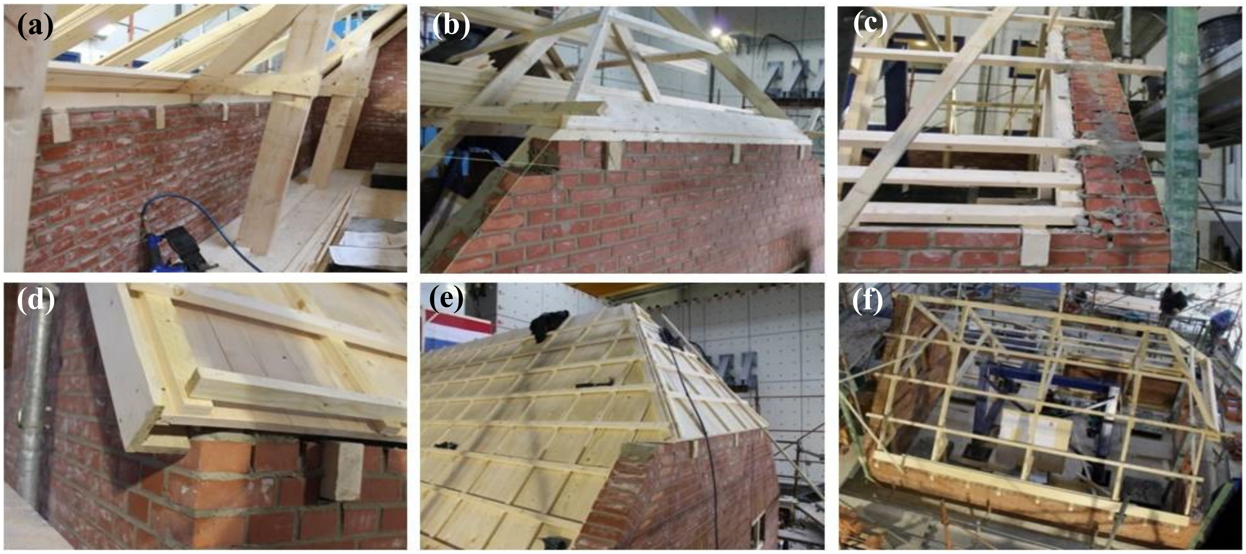

Figure 7: Details of the roof: (a) roof support at East wall; (b) roof support at North gable; (c) purlins and truss at South gable; (d) planks blocking the purlins outside the South gable; (e) laths and counter battens mesh; (f) roof framing

A rigid steel frame was installed inside the building specimen. This structure served as a safety system, providing support in case of partial or global collapse of the specimen, and constituted a rigid reference system for direct measurement of floor, walls and roof displacements. The frame was not in contact with the building, since its columns ran through four holes in the floor diaphragm, oversized to accommodate the specimen lateral displacements (visible in Figure 4 and Figure 7f). A video showing the construction phases of the specimen is available on line [6].

Masses

The masonry had a mean density of 1984 kg/m^3. Masonry walls, floor diaphragm, and finished roof provided masses of 28.96 t, 0.47 t and 1.87 t, respectively. An additional mass of 1.31 t was applied to the first floor by means of laminated rubber blocks, evenly distributed over the diaphragm. The total mass of the building specimen resulted in 32.61 t.

Mechanical properties of materials and components

The 208×100×50-mm clay bricks had compressive strength f_b = 47 MPa and flexural-tensile strength f_{bt} = 8.5 MPa. The compressive and flexural-tensile strength of the mortar were f_c = 4.12 MPa and f_t = 1.2 MPa, respectively [7][8]. Six masonry wallettes were tested in compression perpendicularly to the horizontal bed-joints [8], allowing an estimation of the masonry compressive strength (f_m = 9.23 MPa) and elastic modulus secant at 33% of the compressive strength (E_{m1} = 8123 MPa). Four-points in-plane and out-of-plane bending tests were performed on six and five masonry wallettes, respectively, in order to evaluate the in-plane (f_{x3} = 0.44 MPa) and out-of-plane (f_{x2} = 0.64 MPa) masonry strengths [8]. Bond wrench tests were performed on eighteen specimens in order to determine the bond strength of masonry (f_w = 0.23 MPa) [8]. Masonry triplets were subjected to shear tests to determine cohesion (f_{v0} = 0.15 MPa) and shear friction coefficient (μ = 0.55) [8]. These mechanical characteristics are in line with the one observed in situ on pre-1945 Dutch clay URM [2].

INSTRUMENTATION AND TESTING PROTOCOL

Several sensors were installed on the building, in order to monitor its structural response. The instrumentation consisted of 37 accelerometers, 21 wire potentiometers, 37 linear variable displacement transducers (LVDTs), and a 3D optical acquisition system. Accelerometers were installed on the walls, on the floor diaphragm, and on the roof. Wire potentiometers recorded the in-plane response of the floor diaphragm, the in-plane response of the East squat wall, and the out-of-plane displacement of the North and South façades. LVDTs were installed to monitor the longitudinal and transverse displacements of the floor diaphragm and of the top of the East and West walls with respect to the steel structure; the relative displacements between the floor and the North and South walls; and the relative displacements between the roof and the North and South gables.

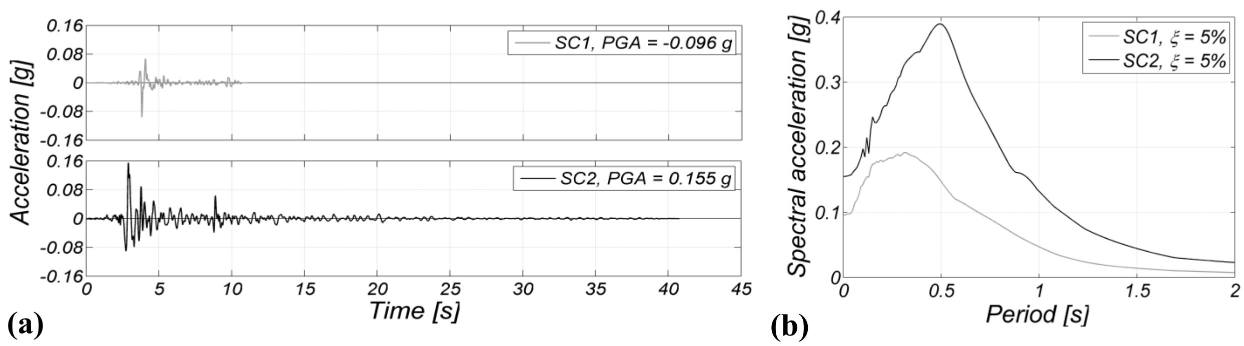

Figure 8: SC1 and SC2 signals: (a) acceleration time histories; (b) acceleration elastic response spectra (5% viscous damping ratio)

The specimen was subjected to an incremental dynamic test, applying a series of shake-table motions of increasing intensity to assess the ultimate capacity and failure modes of the building. The selected input motions represented realistic ground motions for the Groningen region. A detailed study on the seismic hazard characteristics [9] identified two main scenarios, with return periods of 50 and 500 years; accordingly, two smooth-response-spectra records SC1 and SC2 were generated, with 5-75% significant durations of 0.39 s and 1.73 s, and peak ground acceleration (PGA) of 0.096 g and 0.155 g. Figure 8 shows the theoretical acceleration time-histories of the experimental inputs and their acceleration response spectra at 5% viscous damping ratio. These two records were then scaled in order to obtain the desired incremental test protocol.

Table 1 illustrates the applied testing sequence specifying the input record, the acceleration scale factor (SF), nominal and recorded peak ground accelerations (PGA), and recorded peak ground velocities (PGV). The building was subjected to random noise tests between those listed in Table 1, to monitor dynamic properties evolution and stiffness degradation of the specimen at each testing step.

Table 1: Summary Testing Sequence

Test Input	SF [%]	Nominal PGA [g]	Recorded PGA [g]	Recorded PGV [m/s]
SC1	25%	0.024	0.026	0.022
SC1	50%	0.048	0.050	0.035
SC1	100%	0.096	0.098	0.058
SC1	150%	0.144	0.149	0.086
SC2	50%	0.077	0.080	0.073
SC2	100%	0.155	0.140	0.122
SC2	150%	0.232	0.227	0.186
SC2	200%	0.310	0.293	0.241
SC2	250%	0.387	0.392	0.308
SC2	300%	0.465	0.500	0.365
SC2	400%	0.620	0.679	0.467

TEST RESULTS

The building did not suffer any visible damage up to the SC1 - 150% test (PGA = 0.149 g), began showing minor cracks under the SC2 - 150% motion (PGA = 0.227 g), and was considered at near-collapse state after the SC2 - 400% test (PGA = 0.679 g). Videos of the testing sequence are available on the EUCENTRE Youtube channel [10]. The following sections illustrate the performance of the specimen reporting qualitative damage observations and significant hysteretic response plots.

Damage evolution

At the end of every shaking test, structural damage was surveyed in detail. During testing under SC1 input motions, scaled from 25% to 150% (PGA from 0.026 g to 0.149 g), the building did not experience any visible damage. Minor damage became visible on the North clipped gable during testing under SC2 - 150% (PGA = 0.227 g): few horizontal cracks were found just above the openings, at the interface between the timber lintels and the masonry. The observed damage did not change significantly after testing under SC2 - 200% (PGA = 0.293 g).

The first significant cracks were identified after the SC2 - 250% test (PGA = 0.392 g): horizontal cracks developed on the South façade, a few centimetres above the floor level; other cracks were observed at the bottom of the wider piers of the West façade, indicative of their rocking response. No damage was detected on the East walls until this intensity level. During shaking under SC2 - 300% (PGA = 0.50 g) a global response of the structure was triggered, as evidenced by the formation of new cracks and the propagation of pre-existing ones. A diagonal crack was clearly observed on the South gable, starting from the lower West corner of the opening at an angle of 45° above the horizontal: this indicated the activation of an out-of-plane mechanism with unequal displacements at the intersecting East and West walls. In-plane mechanisms were also developing in the longitudinal piers of the West façade with prevailing flexural-rocking behaviour as suggested by the propagation of horizontal cracks at their top and bottom ends.

After the SC2 - 400% test (PGA = 0.679 g) extensive damage was observed throughout the building, which was deemed to have reached near-collapse conditions. The maximum recorded average first-floor drift ratio was 0.94%. An out-of-plane mechanism involved the upper portion of the North gable and resulted in rigid-body sliding and rocking: mortar joint sliding and block de-cohesion were observed along a major horizontal crack located just below the timber lintels of the openings (Figure 9a); this crack extended throughout the length of the gable with a maximum residual sliding of the order of 20 mm. The out-of-plane overturning mechanism fully activated on the South gable after formation of an X-shaped crack pattern below the opening, mainly attributed to the restraining effect of the steel anchor Y1 at the floor level (Figure 9b). Due to good interlocking between the intersecting walls, parts of the East and West façades participated in the out-of-plane response of the gables with the formation of stair-stepped cracks in the upper areas of the longitudinal walls.

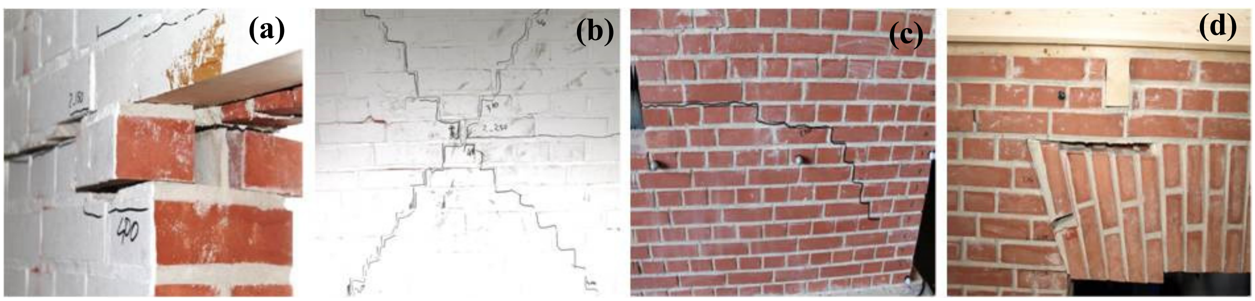

Figure 9: Failure modes: (a) crack and block de-cohesion on the North gable; (b) X-shaped cracks on the South gable; (c) crack at the base of a West pier; (d) near-collapse of a lintel

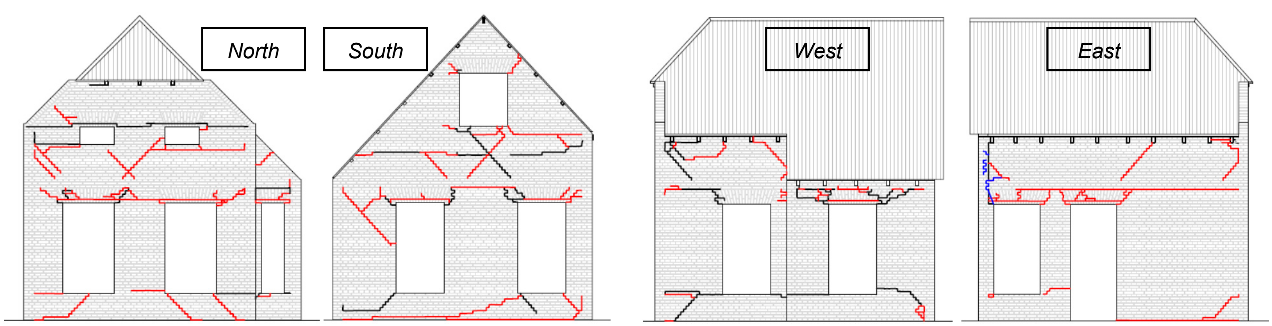

Figure 10: Crack pattern observed after the SC2 - 400% test (PGA = 0.679 g)

The West piers exhibited pure flexural-rocking failure mechanisms, with widening and propagation of pre-existing cracks at their top and bottom ends (Figure 9c), and severe damage to one of the flat-arch lintels (Figure 9d). Visible cracks formed on the East squat pier only at this final stage of the test: a flexural-rocking mechanism developed under southward displacement, with subsequent sliding all along its base and a residual of about 0.2 mm. No sliding or differential displacements were noted between roof and gables and between wall plates and longitudinal walls, despite weak connections. Figure 10 shows the overall damage pattern after the SC2 - 400% test (PGA = 0.679 g). Cracks marked in red and blue were observed at this stage on the internal and external side of the walls, respectively. Cracks shown in black had already been detected under previous shaking.

Hysteretic response

Figure 11 shows the hysteretic response of the specimen in terms of base shear and first-floor average displacement (relative to the base) from the last three tests. The base shear was computed as the sum of the products of each accelerometer reading times its tributary mass, lumped at the accelerometer location. Nonlinear response was initially observed in the SC2 - 250% test (PGA = 0.392 g) and became rapidly pronounced under the two following tests; it was associated with the formation of flexural-rocking cracks in West piers and out-of-plane behaviour of the gables. As the East squat pier was significantly stiffer and stronger than the West ones, the floor diaphragm was subjected to significant shear deformation, but its flexibility prevented overall torsional response of the building.

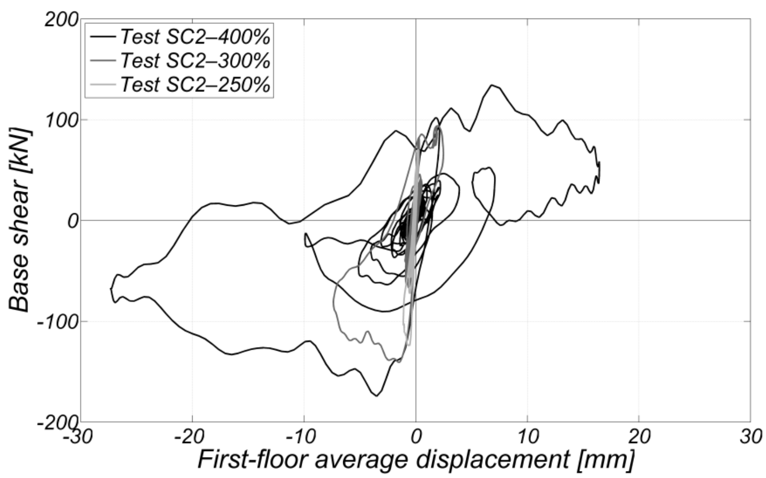

Figure 11: Specimen hysteretic response

CONCLUSIONS

This paper discussed the full-scale unidirectional shaking table test of a building specimen simulating a Dutch URM detached house. The specimen was subjected to incremental input motions representative of induced seismicity scenarios for the Groningen region in the Netherlands, characterized by smooth response spectra and short significant duration. The building suffered only minor damage under the input motion with PGA of 0.23 g and reached its near-collapse state at a PGA of 0.68 g.

At the end of the tests the building experienced damages to the longitudinal piers and to the gables (induced by an overturning mechanism of the roof). Good interlocking between intersecting walls resulted in the participation of longitudinal wall portions to the gable mechanisms. Significant damage occurred in the slender West masonry piers, which developed flexural-rocking in-plane mechanisms during final stages of the test. The East side resulted to be stiffer and stronger than the West one, due to a long squat pier.

Because of differential in-plane displacements between the two longitudinal walls, the flexible floor diaphragm underwent significant in-plane shear deformations. The roof-gables assembly exhibited very flexible response, resulting in displacement demands significantly larger than at lower levels. Despite weak connections between the roof structure and the gables or the longitudinal walls, significant differential displacements were not observed at their interfaces.

ACKNOWLEDGEMENTS

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