Engineering Structures 161 (2018) 231–249

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Experimental seismic performance of a full-scale unreinforced clay-masonry building with flexible timber diaphragms

Stylianos Kallioras[a], Gabriele Guerrini[b,c], Umberto Tomassetti[b,c], Beatrice Marchesi[c], Andrea Penna[b,c], Francesco Graziotti[b,c,*], Guido Magenes[b,c]

[a] UME Graduate School, IUSS Pavia, Piazza della Vittoria 15, 27100 Pavia, Italy
[b] Department of Civil Engineering and Architecture (DICAr), University of Pavia, via Ferrata 3, 27100 Pavia, Italy
[c] European Centre for Training and Research in Earthquake Engineering (EUCENTRE), via Ferrata 1, 27100 Pavia, Italy

ARTICLE INFO

Keywords:
Unreinforced masonry (URM) building
Full-scale shake-table test
Seismic performance
Solid clay brick
Flexible diaphragm
Induced seismicity

ABSTRACT

This paper presents the results of a unidirectional shake-table test performed on a full-scale, single-storey unreinforced masonry building. The specimen represented a typical detached house of the Groningen region of the Netherlands, consisting of double-wythe clay-brick unreinforced masonry walls, without any specific seismic detailing. The building prototype included large openings and a reentrant corner, causing a discontinuity in one of the perimeter walls. The 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 to represent two typical gable geometries. An incremental dynamic test was conducted up to the near-collapse state of the specimen, using input ground motions compatible with induced-seismicity scenarios for the examined region. This paper summarizes the main characteristics of the specimen and the shake-table experimental results, illustrating the dynamic response of the structure, the evolution of the damage mechanisms, and the attainment of significant limit states.

1. Introduction

The Groningen region of the northern Netherlands, historically not prone to tectonic earthquakes, in recent years has been subjected to seismic events induced by gas extraction and consequent reservoir depletion [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 [2].

Because of the limited available information on the seismic performance of Dutch building typologies, an experimental campaign was launched in 2015, aimed at investigating the performance of structural components, assemblies, and systems in pursuance of improving analytical prediction of URM damage for the vulnerability assessment of URM buildings in the Groningen region. The experimental program includes in-situ mechanical characterization tests [3] and laboratory tests, such as: (i) characterization tests on bricks, mortar and small masonry assemblies; (ii) in-plane cyclic shear-compression tests [4] and dynamic out-of-plane tests on full-scale masonry piers [5]; and (iii) full-scale unidirectional and bidirectional shake table tests on different URM building typologies [6–9].

With the aim of investigating the seismic behaviour of clay-brick URM detached houses dating back to before World War II, an incremental dynamic test was carried out on a prototype building at the EUCENTRE laboratory in Pavia, Italy, up to near collapse conditions. This typology constitutes a significant portion of the URM building stock of the Groningen region and comprises commonly one- or two-storey buildings with solid clay-brick walls, 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 [10,11].

The building specimen was dynamically excited in the direction perpendicular to roof trusses and floor beams, which is considered more vulnerable. The opening sizes and locations resulted in different shear stiffness and strength for the two parallel shear walls. Ground shaking of increasing intensity was applied to the building base, while random vibration tests were performed to monitor the evolution of the system dynamic properties at each testing step. The specimen was densely instrumented with sensors that recorded the response of various structural elements. Among other aspects, the experiments allowed to

* Corresponding author at: Department of Civil Engineering and Architecture (DICAr), University of Pavia, via Ferrata 3, 27100 Pavia, Italy.
E-mail address: francesco.graziotti@unipv.it (F. Graziotti).

https://doi.org/10.1016/j.engstruct.2018.02.016
Received 21 November 2017; Received in revised form 31 January 2018; Accepted 2 February 2018

investigate: (i) the load sharing and possible torsional effects, if any, between the two longitudinal walls, (ii) the out-of-plane behaviour of the gables, (iii) the degree of connection between walls and roof or floor framing systems, and (iv) the in-plane flexibility of roof and floor diaphragms, which mostly affect the seismic vulnerability of these structures.

This paper presents the geometric and mechanical characteristics of the building specimen, the construction details, the instrumentation, the seismic input and the testing protocol, and the major observations from the tests including damage evolution and hysteretic response. The general behaviour of the prototype building is discussed and its performance is assessed by linking engineering demand parameters, performance limits states, and points of the global force-displacement response.

2. Specimen overview

2.1. Specimen geometry

The full-scale single-storey prototype was characterized by a 2.9-m floor height (measured to the top of the attic floorboards) and a 3.3-m-high pitched roof. The overall footprint dimensions were 5.8 m in the shaking direction and 5.3 m in the transverse one. The load-bearing structural system consisted of 208-mm-thick, double-wythe clay URM walls, supported by a composite steel-concrete foundation rigidly fixed to the shake table. The specimen included large asymmetrical openings on all sides and a reentrant corner, causing a discontinuity in one of the perimeter walls (Figs. 1 and 2), with the intent to magnify differential wall displacements under uniaxial seismic excitation compared to a more regular rectangular layout.

The timber floor and roof diaphragms were flexible, as 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 (Figs. 1 and 2). The perimeter walls extended above the first floor to form 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 shake table test was performed perpendicularly to the gables, as shown by the arrows on Fig. 1.

Even though not expected to be exhaustive of all possible geometric variations of the local building stock, the building prototype was deemed representative of a pre-1940s clay-brick URM detached house of the Groningen region (Fig. 1a and b). Contractors from the Groningen area built the specimen, using materials shipped from the Netherlands. The specimen was built at full scale directly on the shake-table of the EUCENTRE laboratory, to avoid possible damage during transportation.

2.2. Construction details

Construction details of the Dutch practice preceding the 1940s were reproduced in the specimen. The *Dutch cross* brickwork bond (Fig. 3a) was adopted for the masonry bearing walls, with 208 × 100 × 50 mm solid clay bricks and 10-mm-thick, fully mortared head and bed joints. Lintels were built above all openings (Fig. 3b, and c): they consisted of a 100-mm-wide × 50-mm-deep timber beam below the interior masonry wythe, extending into the masonry 100 mm on each side of the opening for support. A 300-mm-deep brick flat arch was built below the exterior masonry wythe, with the brick stretchers facing outwards.

The floor system consisted of 200-mm-wide × 24-mm-thick spruce timber floorboards, nailed perpendicularly to ten 80-mm-wide × 180-mm-deep timber joists spanning continuously between the East and West URM walls (Fig. 4). The joist ends were cut at an 80° angle (Fig. 5) and were supported on the interior wythe of the longitudinal walls at a height of 2.7 m above ground level.

Connection between the floor diaphragm and the East and West walls was provided by 14-mm-diameter L-shaped steel anchors (labelled X1 on Figs. 4a and 5), screwed to the timber joists and embedded into the masonry between the two wythes (Fig. 5a, b, d, and e). Flat S-shaped steel connectors (labelled Y1 on Figs. 4a and 5) provided wall-to-diaphragm connection for the North and South façades, restraining these walls against out-of-plane overturning. These anchors, located at mid-span of each façade (Fig. 4a), ran below the timber floorboards and were screwed to the first two floor joists from the restrained walls (Fig. 5c, and f). Additional S-shaped and L-shaped anchors were installed at the gable-floor and gable-roof joints, respectively: these anchors were not connected to the timber framing initially, but were placed with the possibility to be nailed subsequently in case of premature development of local out-of-plane mechanisms of the gables. Since they were never activated, the additional anchors are not represented on Fig. 4c.

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

A truss was placed back-to-back with the South gable; however, the roof purlins extended through and protruded 100 mm beyond the masonry gable (Fig. 7d). This resulted in a very small fraction of gravity load being transmitted to the South gable under static conditions. Two planks were nailed to the purlins outside the gable (Fig. 7e), forming an L-shaped end-block which restrained the relative displacement between

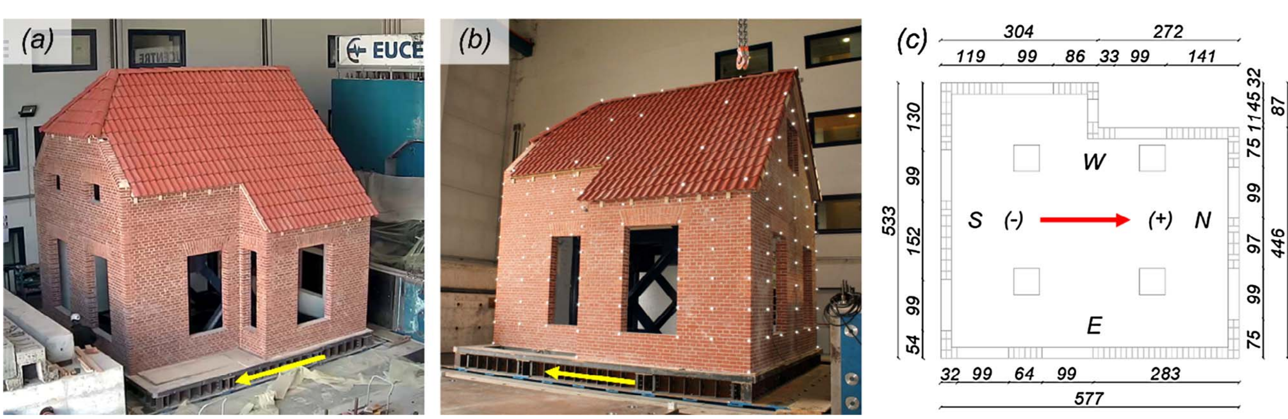

Fig. 1. Full-scale specimen: (a) North-West view; (b) South-West view; (c) first-floor plan (units of cm).