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ROBUSTNESS OF MASONRY UNITS AND SEISMIC BEHAVIOUR OF MASONRY WALLS

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Abstract

By testing reinforced and unreinforced hollow clay unit masonry walls it has been found that the robustness of masonry units is one of the decisive parameters which define the behaviour of masonry walls when subjected to seismic loads. If local brittle failure of units occurs, the mechanism of behaviour and known relationships between the strength and ductility properties of masonry walls change. This is especially the case of reinforced masonry, where brittle local failure prevents the transfer of predicted forces from reinforcing steel to masonry units. In order to provide limitations for the use of hollow units with large hole volume ratio and thin shells and webs in seismic areas, a measure to define the qualitative term "sufficient robustness", specified in Eurocode 8, should be found.

Key Words

Masonry units, robustness, seismic behaviour.

1 Introduction

Innovation and optimisation of technology of masonry construction introduced new types of masonry units and construction technologies, which have so far not been regulated by the codes. Masonry industry improved the thermal properties of masonry units and developed new, faster and cheaper technologies of construction. As a result of such development, hollow masonry units with very thin shells and webs are produced, and construction technologies are introduced where traditional head joints, fully filled with mortar, are replaced by either ungrouted or partly grouted head joints or mechanical interlocking between masonry units. The innovative proposals have been developed in the countries not prone to seismic hazard. However, though not significantly influencing the collapse mechanisms when subjected to gravity loads, these innovations significantly influence the behaviour of masonry structures of all systems in seismic conditions since they reduce the robustness of masonry units (thin

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shells and webs) and homogeneity of masonry walls (masonry bond) as structural elements.
According to Eurocode 8 (EC 8), masonry units should have sufficient robustness in order to avoid local brittle failure when subjected to seismic loads. The requirement is obvious, however, the definition and criteria for robustness are lacking. The selection of suitable units according to grouping specified in Eurocode 6 (EC 6) is up to the National Annexes. Group 1 units are represented by solid or almost solid units, whereas in the case of clay units belonging to Group 2, the amount of vertical holes is limited to 55 % of the volume and the minimum thickness of shells and webs to 8 mm and 5 mm, respectively. This means that the shape of Group 2 clay units varies in a wide range from almost solid to highly perforated hollow units. Without adequate quantification and testing it will be impossible to propose a proper selection.
So far, the influence of robustness of units on the seismic behaviour of masonry walls has not been systematically investigated. However, on the basis of some recent experimental research it can be concluded that the robustness of hollow units is one of the decisive parameters which govern the seismic behaviour of all systems of masonry construction where such units are used.

2 Robustness of units and reinforcement

In the first case, the effect of reinforcement on the seismic behaviour of masonry walls, designed to fail in bending, has been investigated (Tomaževič and Lutman 1997). For the construction of the walls, hollow clay masonry blocks, with holes to accommodate vertical steel, have been used. The shape and dimensions (175x290x190 mm - length x width x height) of the units were in conformity with the requirements of EC 6 for Group 2 clay masonry units. The volume of the holes was 44 % of the gross volume of the unit, whereas the thickness of shells and webs was 12 mm and 8 mm, respectively.
Two groups of four walls with different unit strengths have been tested. The amount of vertical reinforcement, placed in the holes at the ends of walls and grouted with concrete, varied as indicated in Table 1. Horizontal bed joint reinforcement was the same in all cases. In addition to reinforced walls, two unreinforced walls of each unit strength class have been made and tested as referential specimens.
The walls have been 96 cm long, 140 cm high and 29 cm thick. General purpose mortar of the strength class M2.5 has been used for the construction. The specimens have been built on r.c. foundation blocks, fixed to the strong floor during the testing. In the case of reinforced walls, deformed steel reinforcing bars, 14, 20, or 28 mm in diameter of the strength class M400, have been placed vertically at the ends of the walls. The reinforcing bars have been anchored into the foundation block at the bottom and into the bond-beam at the top of the wall. Stirrups, placed in each horizontal mortar bed joint, have been bent around the vertical reinforcement at the ends to improve the anchoring. Stirrups have been made of smooth reinforcing steel, 6 mm in diameter, of the strength class M250. Two specimens of each type have been tested.

Table 1 Designation and reinforcement of tested walls

Wall designation	Strength of masonry units	Vertical steel	Vertical reinforc. ratio	Horizontal steel	Horizontal reinforc. ratio
H1-0	6.2 MPa	-	-	-	-
H1-14	6.2 MPa	2 ϕ 14	2x0.056 %	2 ϕ6/20 cm	0.099 %
H1-20	6.2 MPa	2 ϕ 20	2x0.115%	2 ϕ6/20 cm	0.099 %
H2-0	9 MPa	-	-	-	-
H2-20	9 MPa	2 ϕ 20	2x0.115%	2 ϕ6/20 cm	0.099 %
H2-28	9 MPa	2 ϕ 28	2x0.225%	2 ϕ6/20 cm	0.099 %

The walls have been tested as vertical cantilevers, with foundation block fixed onto the strong floor by means of steel bolts (Figure 1). They have been subjected to constant vertical load and cyclic lateral displacements, repeated three times at each displacement amplitude. Constant compressive stress (working stress/compressive strength ratio 20 %) has been induced in the walls' horizontal section during the lateral resistance tests.

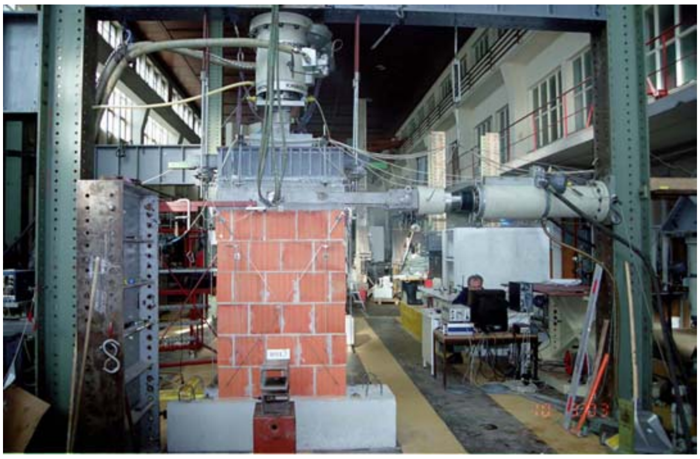

Figure 1 Typical testing arrangement for lateral resistance test of masonry walls

Although the specimens have been designed to fail in bending, brittle local failure of units, i.e. buckling and crushing of thin shells and webs (Figures 2 and 3), resulted into predominant ultimate shear behaviour and collapse of the walls. Consequently, only a small part of the available tension capacity of vertical reinforcing bars has been utilized. As can be seen in Table 2, where the experimental values of lateral resistance H_{exp} are correlated with predicted values of flexural capacity $H_{u,cal}$, the predicted values overestimated the actual resistance of the walls.

Figure 4 Shear failure of a vertically reinforced wall due to local brittle failure of units

Figure 5 Local buckling and crushing of shells and webs of masonry units

Table 2 Comparison between experimentally obtained H_{exp} and calculated values of flexural capacity $H_{u,cal}$ of tested walls

Wall designation	h (m)	Experimental H_{exp} (kN)	Experimental M_{exp} (kNm)	Calculated $H_{u,cal}$ (kN)	Calculated $M_{u,cal}$ (kNm)	exp/cal
H1-14	1.60	80.9	129.5	87.6	140.2	0.92
H1-20	1.60	78.2	125.2	118.3	189.3	0.66
H2-20	1.58	109.1	172.4	136.6	215.9	0.80
H2-28	1.58	103.2	163.0	173.3	273.8	0.60

This can be also seen in Figures 4 and 5 where experimentally obtained lateral resistance - displacement hysteresis envelopes for the walls type H1 and H2, respectively, are presented. No difference in lateral resistance can be observed as a result of different amounts of vertical reinforcement. As the measurements of strain in horizontal and vertical steel indicate, the improvement in lateral resistance and ductility with regard to referential unreinforced walls of the same quality can be attributed to the effect of horizontal bed joint reinforcement.

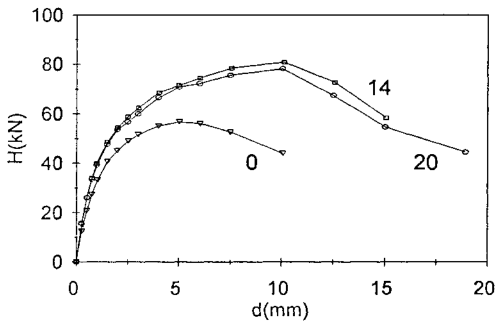

Figure 4 Lateral load - displacement envelopes for walls H1: 0 - referential unreinforced wall

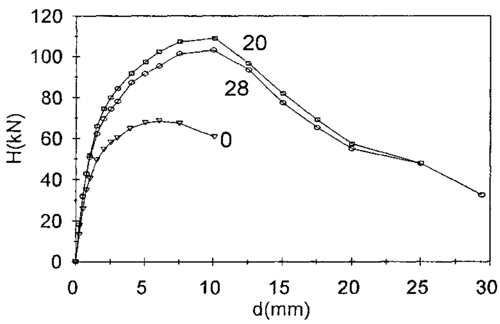

Figure 5 Lateral load - displacement envelopes for walls H2: 0 - referential unreinforced wall

Yielding of steel at ultimate state is assumed in the calculations of sectional capacity. However, if local brittle failure of masonry units takes place and bond is degraded under cyclic loading before the tension capacity of reinforcing steel is attained, the design values of resistance capacity of a reinforced masonry wall's section will overestimate the actual resistance capacity. Although designed for earthquake loads "by code", there is a risk that the actual degree of seismic safety of a masonry structure under consideration is be lower than required and/or verified by calculation.

3 Robustness of units and masonry bond

Traditionally, only the construction of masonry walls with fully grouted (filled) head joints has been allowed in seismic zones. By definition given in EC 6, head joints can be considered to be filled if mortar is provided to the full height of the joint over a minimum of 40 % of the width of the unit. In the new draft of EC 8, however, a note states that the National Annex will select which of the three classes of head joints (fully grouted, ungrouted, and ungrouted with mechanical interlocking between the units) will be allowed to be used in a country.

Since not much reliable experimental evidence to prove that these technologies are suitable and acceptable exists, a research project has been recently carried out where

the influence of different systems of filling the head joints on the seismic behaviour of the walls has been investigated. Four different types of filling the head joints have been studied (Figure 6):

- Fully filled head joint (walls series BN - referential walls),
- Dry, unfilled head joint (walls series BG),
- Partly filled head joints with mortar in the pockets (walls series BP),
- Dry, grove and tongue head joint (walls series BZ).

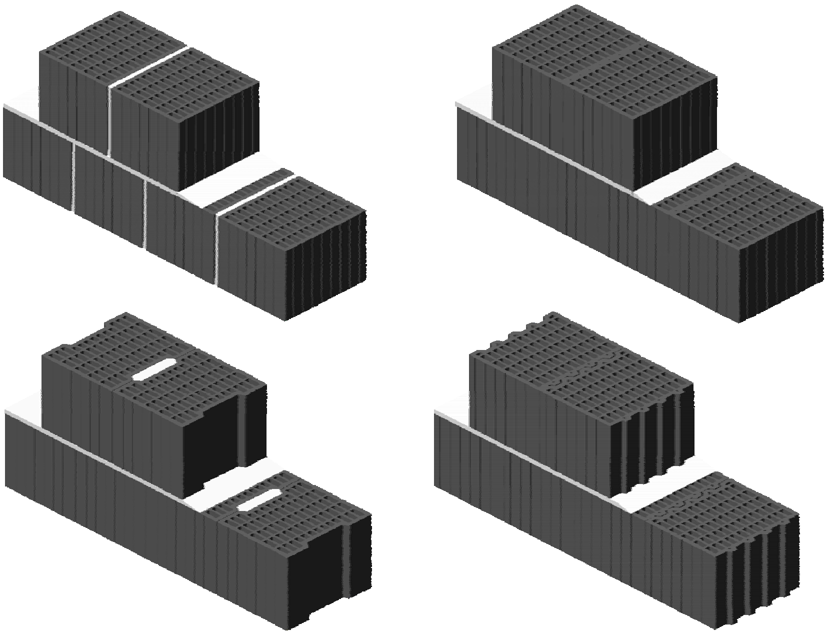

Figure 6 Schematic presentation of the investigated types of head joints

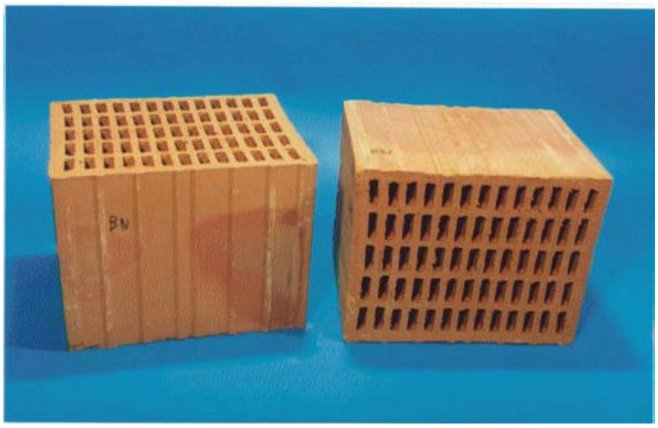

Figure 7 Masonry units for the construction of walls of series BN and BG

The walls to be tested have been designed to fail in shear. They have been 100 cm long, 150 cm high and 30 cm thick. Grade M10 clay hollow units with dimensions 245x300x240 mm (length x width x height), with 12 mm thick shells, 8 mm thick webs and 50 % of holes per volume of the unit (Group 2 units according to EC 6) have been used, same in all cases except for the head joint face (Figure 7). Grade M5 general purpose mortar has been used for the construction of specimens, built on r.c. foundation blocks and tested in the same way as shown in Figure 1. Three specimens

of each type have been tested at working stress/compressive strength ratio which varied from 20% to 33 % (Bosiljkov et al 2004).
All walls failed in shear, as expected. However, brittle local failure of units, i.e. buckling and crushing of thin shells and webs was the predominant phenomenon which determined the failure mode (Figures 8 and 9). It can be seen that, against expectations, the main diagonal cracks passed the joints in the case where the head joints were fully filled (referential walls type BN), whereas they passed the units in the case where the head joints were dry, not filled at all (walls type BG).

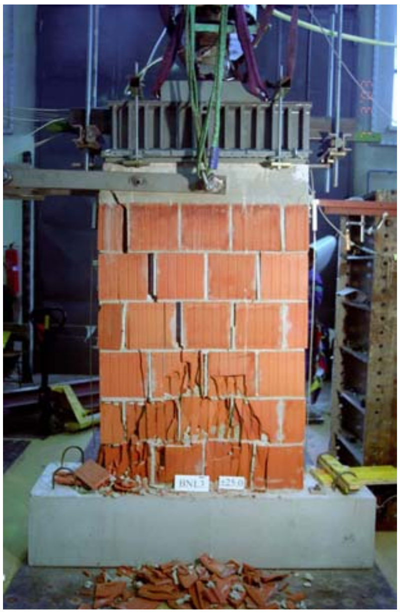

Figure 8 Brittle local failure of units at shear failure of wall type BN

Figure 9 Brittle local failure of units at shear failure of wall type BG

As can be seen in Figures 10 and 11, which show the hysteretic relationships between the lateral load and displacements for typical referential wall with fully filled head joints and a wall with dry head joints, all walls failed in a non-ductile, brittle mode, soon after the maximum resistance has been attained.

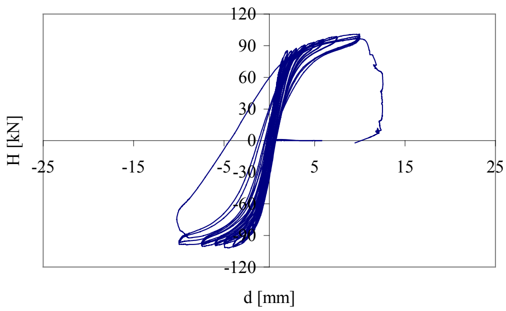

Figure 10 Lateral load - displacement envelopes for wall type BN

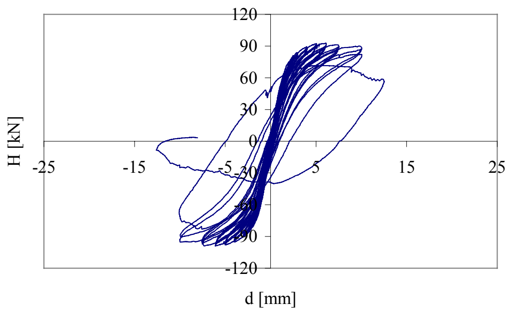

Figure 11 Lateral load - displacement envelopes for wall type BG

The test results are summarized in Tables 3 and 4. In Table 3, mechanical characteristics of the tested types of walls, such as compressive f and tensile strength f_t, as well as modulus of elasticity E and shear modulus G, are evaluated.

Table 3 Mechanical characteristics of the tested types of masonry walls

Type	f (MPa)	f_t (MPa)	E (MPa)	G (MPa)	f_t/f	G/E
BN	4.13	0.17	3088	330	0.041	0.107
BG	4.31	0.19	3302	354	0.044	0.107
BP	6.28	0.22	4815	320	0.035	0.066
BZ	6.24	0.20	5548	367	0.033	0.066

It can be noticed that the ratio between the tensile and compressive strength of masonry f_t/f of all tested types is low and did not exceed 4 % of the value of compressive strength (usually 6 - 8 %). The ratio between the values of shear modulus G, evaluated on the basis of the effective stiffness of the walls measured during lateral resistance tests, and modulus of elasticity E, evaluated on the basis of compression tests, did not exceed 0.11. The ratio decreased with the increased compressive strength of masonry.

In Table 4, the lateral resistance and displacement capacity indicators are given for the tested types of wall, defined as the ratios between the lateral resistance and displacement values at different limit states, such as crack (d_{cr}, H_{cr}), maximum resistance (d_{Hmax}, H_{max}), and ultimate limit (d_u, H_u).

Table 4. Lateral resistance and ductility capacity indicators of the tested types of walls

Series	H_{cr}/H_{max}	H_{du}/H_{max}	d_{cr}/d_{Hmax}	d_u/d_{Hmax}	d_u/d_{cr}
BN	0.98	0.44	1.00	2.15	2.15
BG	1.00	0.66	1.00	1.13	1.13
BP	0.97	0.45	0.84	1.73	2.06
BZ	0.93	0.34	0.68	1.75	2.57

No significant influence of different types of filling the head joints can be observed on the lateral resistance and displacement capacity of the tested walls. The occurrence of diagonal shear cracks in masonry characterises the attainment of lateral resistance of the walls (H_{cr}/H_{max} ratio close to 1.00). Relatively large resistance degradation has been observed in all cases, i.e. small values of H_{du}/H_{max} ratio have been obtained despite the relatively small ultimate displacements. The displacement capacity of the tested walls in terms of displacement capacity indicators is small, below the expected values for usual unreinforced masonry walls.

A conclusion can be therefore made that in all cases the behaviour of the tested walls under cyclic lateral loading was governed by the premature local brittle failure of units. The failure of the walls subjected to cyclic lateral loading occurred at a stage where the type of filling the vertical, head joints did not yet influence the behaviour of the walls. Therefore, no firm general conclusion can be made as regards the influence of types of masonry bond on the seismic behaviour of masonry walls.

4 Conclusions

As the recent experimental investigations indicated, the development and innovation in masonry brought to the front the robustness of masonry units as one of the basic parameters which determine the seismic behaviour of modern masonry walls and structures. In the case of the local brittle failure of masonry units, the failure mechanisms of the walls subjected to seismic loads change, so that, because of the changed basic relationships, the validity of most practically used calculation procedures for seismic resistance verification of unreinforced and reinforced masonry

structures becomes questionable. This is especially the case of hollow unit reinforced masonry, where sufficient anchorage and bond, as well as strength of the units should be provided to utilise the tension capacity of reinforcing steel.
This has been also recognized in the new draft of EC 8, where the requirement that masonry units, used for the construction of masonry structures in seismic zones, should have sufficient robustness to avoid local brittle failure, is given. The selection of suitable units is up to the National Annexes, whereas no quantitative criteria to fulfill such requirement are specified in the basic document. Therefore, additional experimental research is needed to provide the basis and criteria for the classification of hollow masonry units regarding the robustness. It is planned that, as a result of a recently initiated research project, such criteria will be determined and a testing method for simple evaluation of robustness will be developed. The influence of mortar strength and the way of laying the units will be also studied, along with the influence of different types (classes) of head joints (fully and partly grouted, ungrouted, mechanical interlocking) on the homogeneity of structural walls, built with masonry units which will comply with the criteria for sufficient robustness.

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