Seismic performance of thin-bed layered masonry Walls made of clay blocks and comprising a door Opening

H. Degée
University of Liège (Belgium)

L. Lascar & L. Vasseur
Wienerberger n.v. (Belgium)

SUMMARY
The recent transposition of Eurocode 8 for low-to-moderate seismicity areas implies that some seismic guarantees need to be given. The use of conservative models considering masonry structures as a set of cantilever walls does often not allow justifying the seismic resistance of usual buildings even with design accelerations only up to 1 m/s². It is therefore necessary to consider the coupling effects of lintels and spandrels and to consider the structure as a frame. According to design codes, this is however only allowed if the lintels and spandrels are "regularly bonded and correctly connected". In this perspective, this contribution presents in a first part results of cyclic tests carried out on 5 walls with dimensions 3,0 x 2,8 m (one full wall and 4 walls including a door opening with different configurations of lintel). It presents then a comparison of experimental and numerical results both in terms of stiffness and resistance.

Keywords: Masonry structures, frame effect, lintels, spandrels, experimental study

1. INTRODUCTION AND CONTEXT

Seismicity level in North-European countries is obviously lower than in other well identified seismic countries like Greece or Italy. Significant earthquakes can however occur, even if they are more spaced in time. On the other hand, traditional masonry construction in countries like Belgium, Netherlands or UK exhibits some aspects that are not particularly suitable for earthquake resistance purposes. One issue is that no confining elements are used. Another issue is that the wall is structured as follows: a facing wall with no structural role, a gap for thermal insulation purposes and a bearing wall with rather limited thickness (between 10 and 20 cm), see Fig. 1.1.

The use of such thin structural walls implies that they must be realised with material exhibiting a relatively high resistance, in particular when used for building made of bearing unreinforced masonry up to 4 to 5 levels, which are nowadays common. To this purpose, clay block manufacturers are producing units made of pure clay that can exhibit nominal resistance up to more than 20 MPa but that have as a counterpart a relatively brittle behaviour, thus a priori less suitable for earthquake conditions.

In the mean time, the recent approval of Eurocode 8 for low-to-moderate seismicity areas implies that some seismic guarantees need to be given. The use of conservative models considering masonry structures as a set of cantilever walls (and in particular the "rules for simple masonry buildings" proposed by Eurocode 8, section 9.7) does often not allow justifying the seismic resistance of usual buildings, even for design accelerations as low as 1 m/s². It is therefore useful, and even necessary, to consider the coupling effects of lintels and spandrels and to consider the structure as a frame. According to design codes, this is however only allowed if the lintels and spandrels are "regularly bonded and correctly connected".

This contribution presents results of cyclic tests carried out on 5 thin masonry walls made of high strength clay and including a door opening, for different configurations of lintel and different types of masonry reinforcements. These tests have the multiple aims of (i) characterizing the ductility of the masonry panels for the different types of reinforcement system (including of course the purely unreinforced situation), (ii) quantifying the frame effect and (iii) assessing the connection of the RC lintel to the masonry panels.

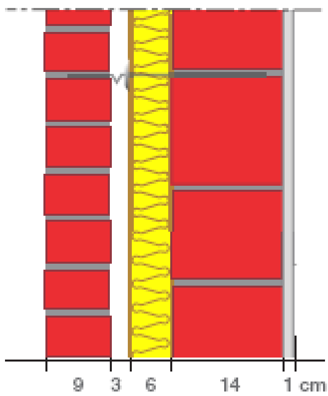


Figure 1.1. Typical structure of Belgian masonry walls.

2. MATERIAL CHARACTERISTICS

Tests are carried out on walls built with thin bed mortar horizontal layer and empty ("tongue and groove") vertical joints. Units are Wienerberger Zonnebeke Porotherm "Système-Collage" units (see Fig. 2.1). Horizontal joint are executed with Poro+ glue mortar.

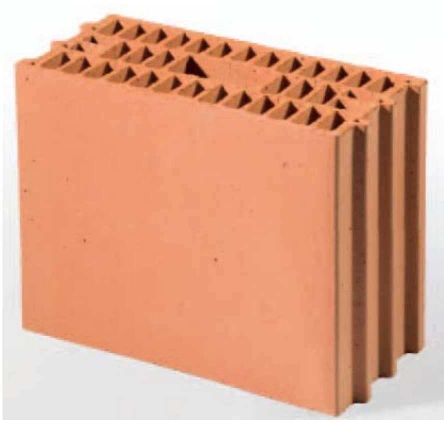


Figure 2.1. Porotherm système-collage units.

A specific material characterization has been realized for the batch of blocks used for the tests. The main characteristics are the following:

- Size of the unit:
  - Length: 301.0 mm
  - Width: 139.6 mm
  - Height: 176.7 mm
- Resistance:
  - Normalized compressive stress of the units, according to EN 772-1 annex A
    $f_b = 13.0$ N/mm²
  - Measured characteristic masonry compressive strength of wallets according to EN 1052-1

$f_k = 5.6$ N/mm²

- Characteristic compressive strength of the masonry obtained from unit resistance according to EN 1996-1-1

  $f_k = 4.2$ N/mm²
- Characteristic compressive strength of the masonry obtained from unit resistance according to NBN-EN 1996-1-1 (Belgian National Annex to Eurocode 6)

  $f_k = 3.9$ N/mm²

No specific characterization has been carried out for shear behavior. Usual standard values are considered for further assessment:

- Initial shear strength according to NBN-EN 1996-1-1

  $f_{vk0} = 0.3$ N/mm²
- Characteristic shear strength according to NBN-EN 1996-1-1 for masonry with empty vertical joints

  $f_{vk} = 0.5\ f_{vk0} + 0.4\ \sigma_d = 0.15 + 0.4\ \sigma_d \leq 0.045\ f_b$ (= 0.585 N/mm²)

3. TEST SET-UP

3.1. General set-up

The test rig is presented in Fig. 3.1. The masonry specimen is built between two reinforced concrete beams representative of the floors. Vertical load is introduced in the wall through two couples of Dywidag bars put into tension by hydraulic jacks placed under the testing slab and taking their reaction on a steel beam lying on the top of the upper RC beam. This setup results in a fairly homogeneous compression thank to the load diffusion through the steel and top RC beams, without constraining the wall since then tension bars are free to follow the possible horizontal motion. Cyclic horizontal loading (displacement controlled) is then applied to the upper RC beam by a hydraulic jack transferring its horizontal reaction to the reaction slab through a truss system.

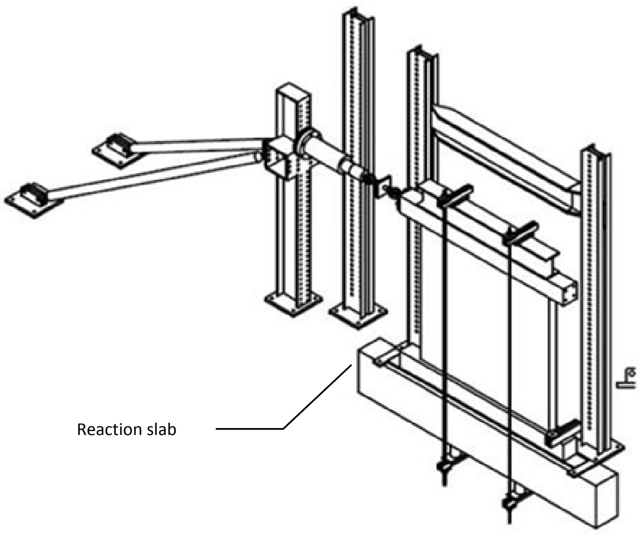


Figure 3.1. Test set-up.

The test procedure consists in two phases:

- Compression up to a level of compressive stress equal to 1 N/mm². This load level is typical of a ground level of regular buildings with 4 to 5 levels;
- Cyclic imposed horizontal displacements. The level of imposed displacement is increased every 3 cycles. The increment of displacement is different for each test and depends on each

particular situation. This is detailed in the appropriate sections.

3.2. Test specimens

Five specimens are tested. Their description is given in table 1 and drawings are given in Fig. 3.2. All specimens exhibit the same overall dimensions (Length x Thickness x Height = 3000 x 140 x 2800 mm).

Table 3.1. Description of test specimens

Specimen	***Description***
A	Full wall without any specific additional elements. This specimen is considered as a reference result.
B	Wall with a door opening and a regular lintel (support of the lintel on 150 mm on each side of the opening). The specimen is unsymmetrical (length of the left wall is 900 mm and length of the right wall is 1200 mm. The door opening is 900 x 2000 mm.
C	Same as B with a long lintel (support of the lintel of 450 mm on each side of the opening).
D	Same as B with vertical confining elements (4 x 8 mm re-bars for each confining element) located on the external sides of the specimen and on each side of the opening).
E	Same as B with Murfor (light horizontal reinforcement, see Fig. 3.3) located in every two horizontal mortar layer. Flat Murfor are used (type 2 in Fig. 3.3).

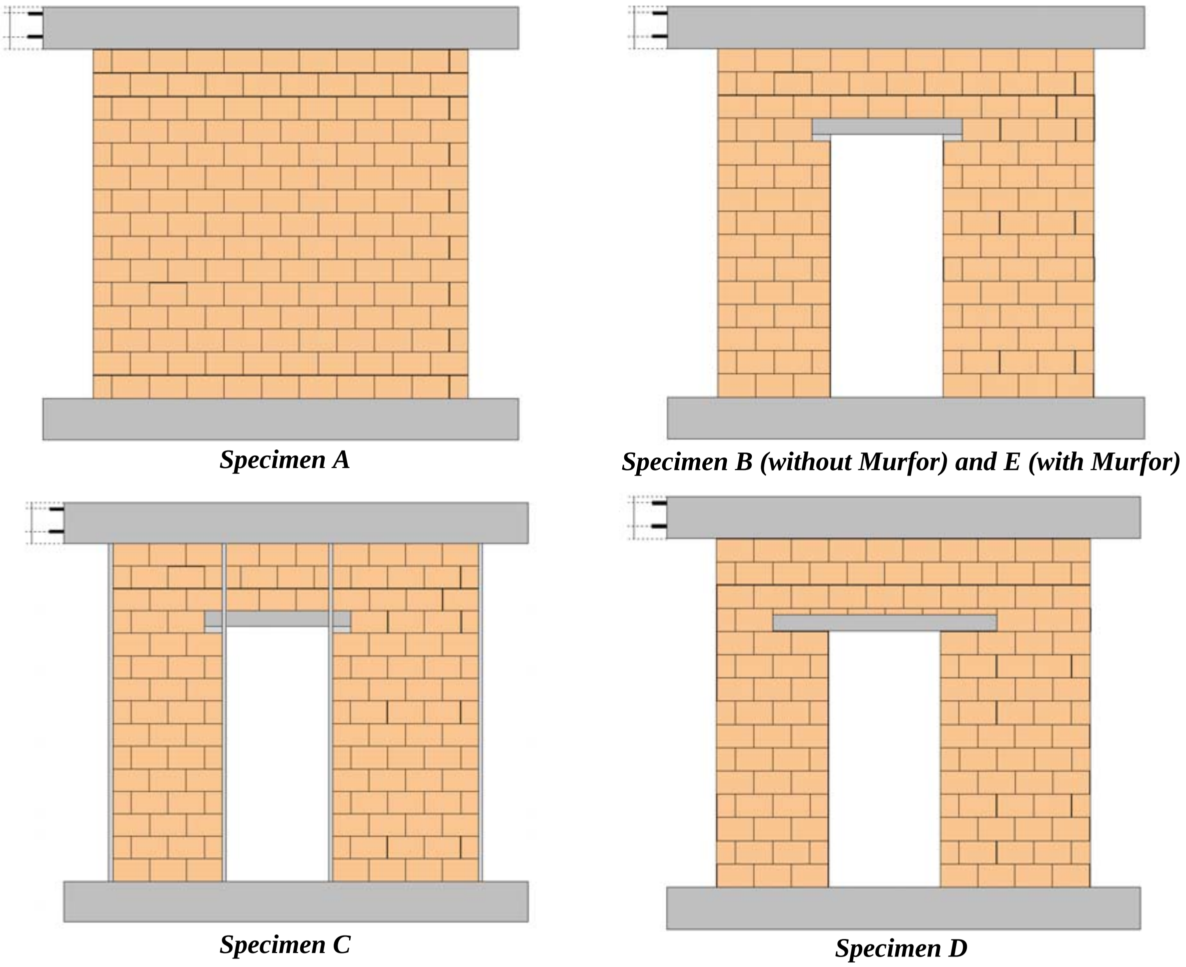


Figure 3.2. Test specimens.

Figure 3.3. Horizontal reinforcement of specimen E.

4. RESULTS

4.1. Reference results on wall without opening

First, the elastic modulus can be estimated from the compression stage. Based on the ratio between average vertical displacement and average vertical stresses, the measured modulus is equal to 2310 N/mm². This corresponds to 412 f_k of the masonry, to be compared with the suggestion of Eurocode 6 (1000 f_k). As a matter of comparison, the elastic modulus obtained for specimens B to E ranges between 448 f_k and 728 f_k.
Cyclic behaviour of the full wall is given on Fig. 4.1. The figure also shows the envelope of the cyclic curves (in red) as well as a bilinear curve ("elastic-perfectly-plastic") obtained from the envelope by a least square fitting procedure (in green). The behaviour shows a limited hysteretic behaviour most likely due to the opening/closing of the vertical joints.

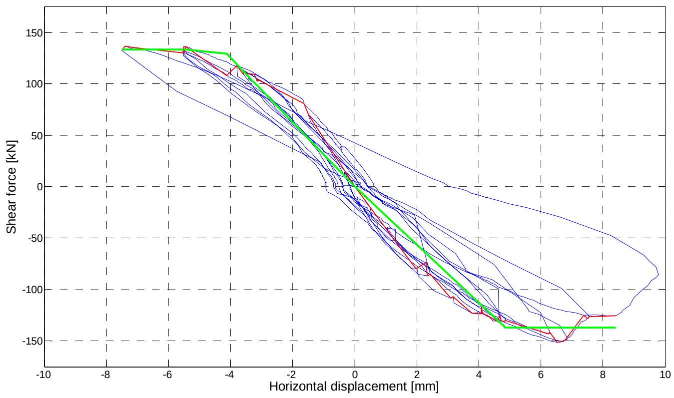


Figure 4.1. Horizontal cyclic behaviour of the full wall.

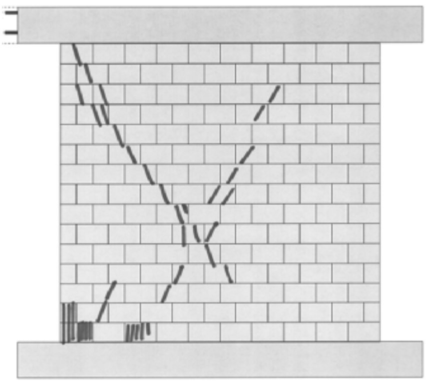


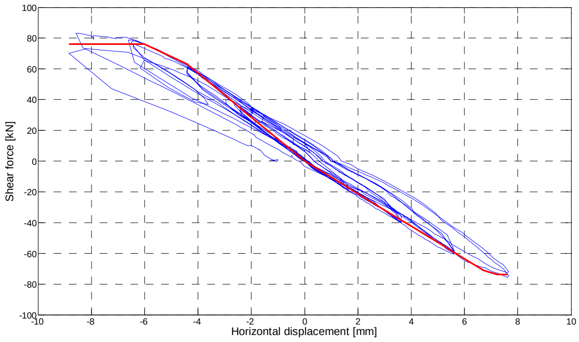
Figure 4.2. Final crack pattern of the full wall.

Figure 4.3 (a) Diagonal cracking - **(b)** Crushing of the toe

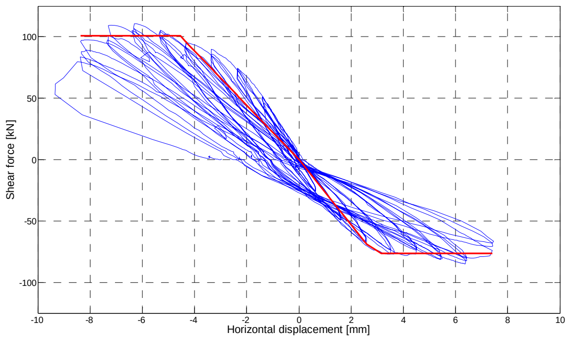
The collapse mode is given on Figures 4.2 (crack pattern at the maximum displacement) and 4.3 (pictures of the wall). It can be observed from the progression of cracking that the resistance is governed by the development of one unique crack in each direction, going throughout the blocks (and thus not following the vertical open joints). This can be explained by the brittle behaviour of the material and by the rather high pre-compression level. The collapse (i.e. the end of the test) is reached when the diagonal cracking reaches the toe of the wall, leading to the crushing of the side block, as seen on Figure 4.3.(b).

4.2. Comparative results for walls with door opening

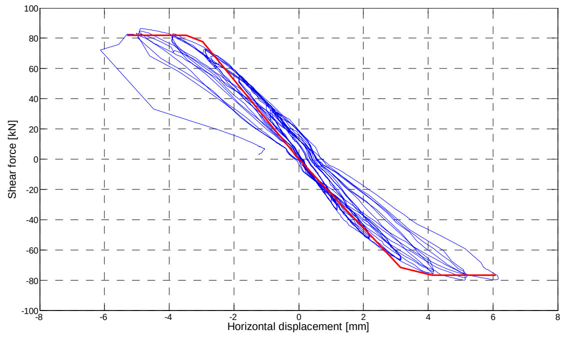
Horizontal cyclic curves, as well as derived bilinear curves, are given in Figure 4.4 with corresponding final crack patterns in Figure 4.5. The main numerical values obtained from the bilinear curve are also given in table 4.1.



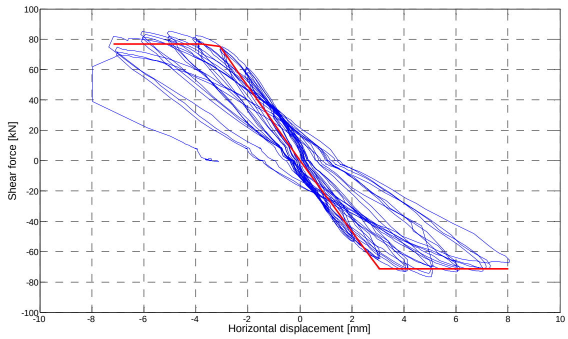
Specimen B



Specimen C



Specimen D



Specimen E

Figure 4.4. Horizontal cyclic behaviour

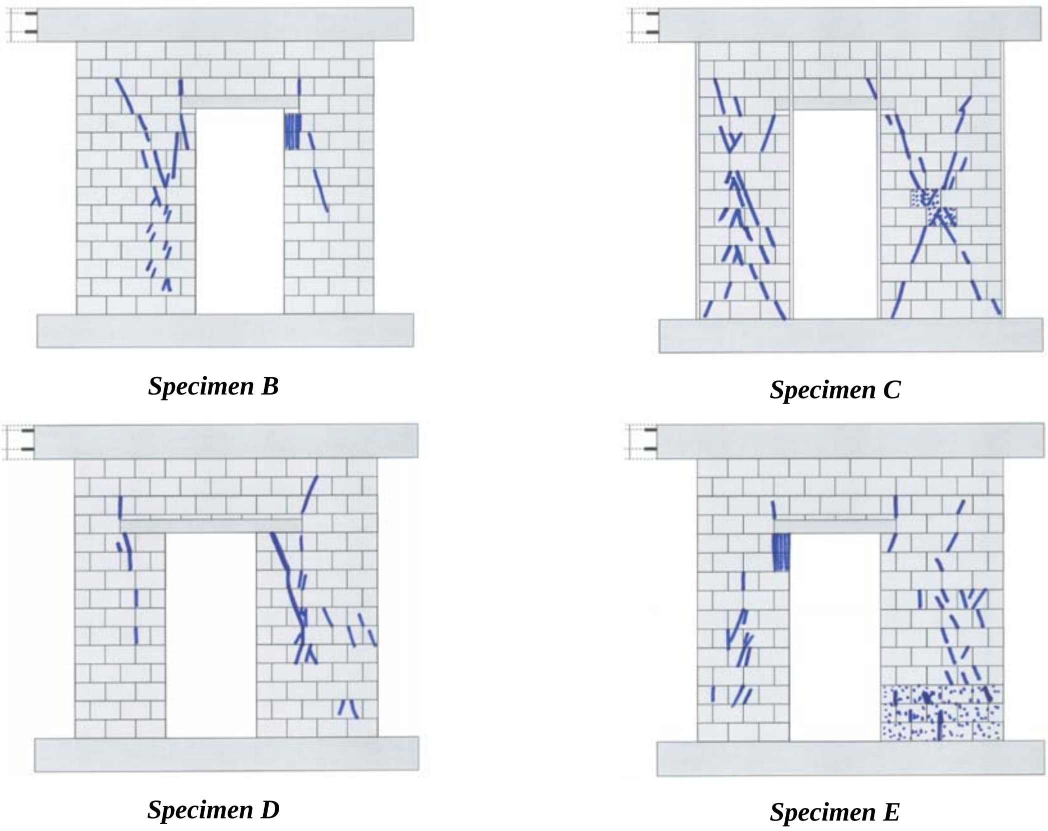


Figure 4.5. Crack patterns.

Table 4.1. Test results

Test	*Ultimate load + (kN)*	*Ultimate load – (kN)*	*Ultimate drift + (mm / %)*	*Ultimate drift - (mm / %)*	*Ductility +*	*Ductility -*
A	133.0	137.1	7.5 / 0.27	8.4 / 0.30	1.8	1.7
B	76.1	73.8	8.8 / 0.32	7.6 / 0.27	1.7	1.1
C	100.8	76.3	8.4 / 0.30	7.4 / 0.26	1.8	2.6
D	82.0	76.6	5.3 / 0.19	6.1 / 0.22	1.7	1.8
E	76.9	71.2	7.2 / 0.26	8.0 / 0.29	2.3	2.6

The collapse mechanism of the system is significantly changing according to the configuration.

- ***Specimen B – frame with regular lintel configuration***
  Cracking starts in the larger wall for positive loading. This corresponds to a compression in this large wall induced by the global overturning of the frame system. For negative loading, the localized compression induced at the lintel support results in the brittle crushing of the blocks at the support (see. Fig. 4.6.a). As a consequence of this brittle failure, the ductility of the system is very limited.
- ***Specimen C – frame with vertical confinement***
  Vertical confinement allows a better distribution of the shear cracking (see. Fig. 4.6.b). On the other hand, compared to the reference test, it provides a reaction to the compression struts induced in the wall after cracking and avoids the risk of a crushing of the toe. The consequent ductility is therefore rather high, as well as the energy dissipation capacity. The ultimate limit state is reached by the crushing of the block located as the crossing of the two diagonal cracks networks in the large wall. It must however be noticed that the execution of such a reinforcement scheme, and in particular the anchoring of the vertical rebars in the top beam, is quite difficult to

implement in practice. The highly varying efficiency of this anchoring, obviously more efficient in the positive direction than in the negative one, leads to a largely unsymmetrical behaviour of the system.

- ***Specimen D – frame with increased lintel supports***

  Compared to specimen B, the longer lintel reduces the risk of brittle failure of the supporting blocks. Except for that specific issue point, the behaviour is very similar to the one observed for the short lintel (i.e. vertical cracking related to the compression in the walls induced by the overturning of the system). The ultimate state is reached by excessive diagonal cracking at the support, leading to the ejection of the part of the wall located below this crack (see Fig. 4.6.c). The resulting ductility is increased compared to situation with regular lintel, but remains limited compared to reinforced solutions.

- ***Specimen E – frame with horizontal reinforcements***

  The main positive effect of the bed joint reinforcements is to limit the consequences of the lintel support crushing. Indeed, it is observed that the blocks are crushed similarly to what is observed for specimen B for a same load level, but that the presence of the joint reinforcements are holding the inner walls of the blocks together, allowing them to sustain the maximum load for increasing displacements.

  The second effect is to increase the shear capacity of the walls. As a consequence, it is observed from the crack pattern, and in particular from the pealing of the outer walls of the blocks, that the ultimate limit state is governed by the bending/overturning behaviour of the wall.

a - Specimen B

b - Specimen C

c - Specimen D

d - Specimen E

Figure 4.6. Collapse modes

5. COMPARISON WITH THEORETICAL MODELS

Regarding the full wall, a comparison is done with the resistance model proposed by Eurocode 6 for in plane loading of a shear wall. The maximum shear resistance obtained accordingly after an iterative procedure is equal to 137.7 kN, which is in good agreement with the measured values (133 kN, resp. 137 kN, from the bilinear curves respectively for positive and negative loading, and 137 kN, resp. 151 kN, from the envelope curves respectively for positive and negative bending). The theoretical assessment also predicts that the ultimate limit state is governed by the shear resistance, which is again in agreement with the experimental observations.

In order to estimate the resistance of the frame system of specimens B to E, three different assumptions are considered for performing the structural analysis (see Fig. 5.1). In the first and second assumptions, the system is assumed to behave as two cantilevers, considering that both sides of the opening are coupled in terms of horizontal displacements, but not in terms of rotation (i.e. no frame effect). Assumption 1 is considering a wall height equal to the opening height, while assumption 2 considers the full height of the wall. In the third assumption, a frame effect is taken into account considering the full stiffness of the spandrel. The internal forces (compression, shear and bending) obtained from these structural analyses are then used as input for a classical EC6 verification. Moreover, for each assumption, it is also considered that the ULS is reached as soon as one of both walls reaches it maximum capacity or that a full load redistribution is possible. Results are presented in Figure 5.2

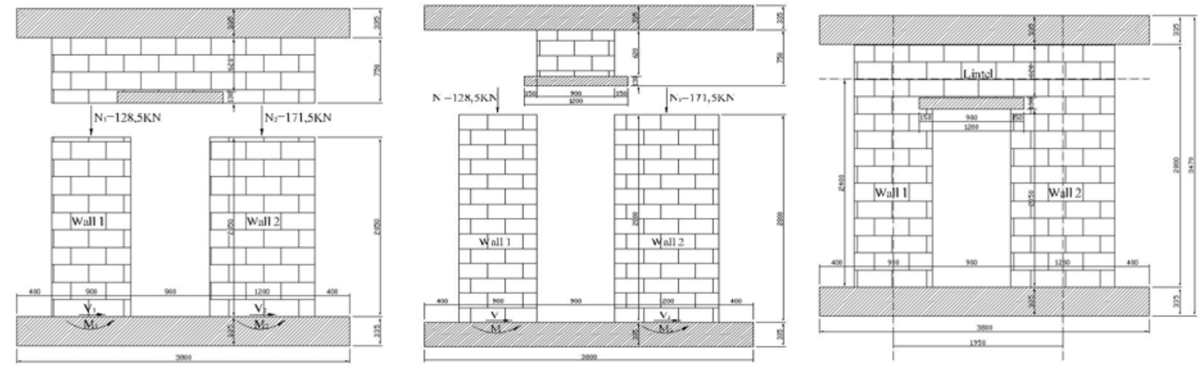


Figure 5.1 Models for the structural analysis

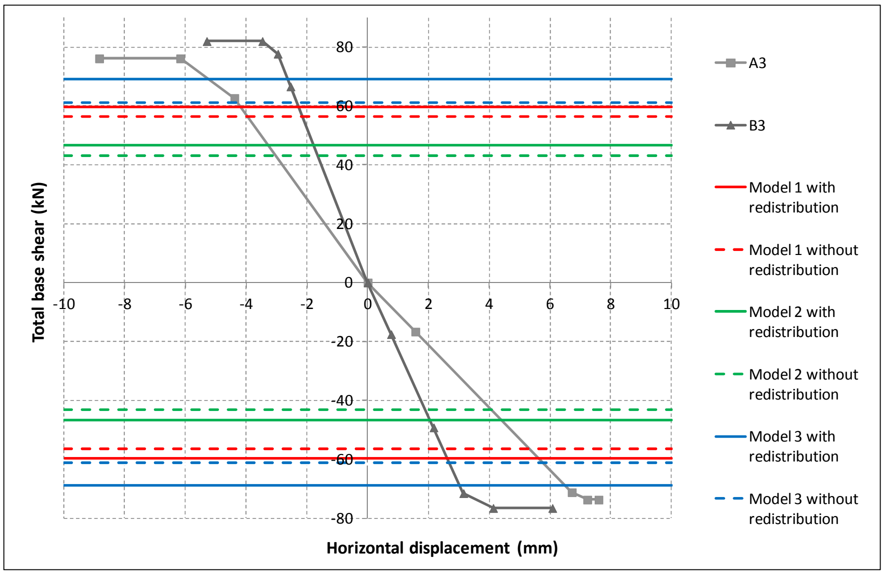


Figure 5.2 Comparison of theoretical models with test results

From this comparison, it can be seen that, regarding the resistance, test results are over the most favourable estimate and that a frame model considering load redistribution provide safe though reasonably accurate evaluation of the system strength.

6. CONCLUSIONS

From the experimental observations summarized in this paper, the following conclusions can be drawn on the behaviour of a wall including a regular door opening:

- The resistance of the system is not significantly depending on the configuration;
- If no specific measure is taken (i.e. in case of a regular length of lintel support without local reinforcement), collapse is triggered by the brittle crushing of the lintel support, leading to a system without ductility;
- The behaviour is slightly better with longer supports, increasing the ductility by more than 50%;
- The presence of vertical confining elements results in a better spreading of the shear cracking, leading to a increased deformation capacity;
- The presence of horizontal reinforcements shifts the collapse mode of the walls from shear to bending, leading to an increased deformation capacity. This increase is of the same order of magnitude than in case of vertical confinement and much easier to implement in practice.
- The system exhibits significant frame behaviour. This effect can be properly predicted by standard analysis tools combined with Eurocode 6 resistance formulations.

AKCNOWLEDGEMENT

These tests have been carried out within the research program FIRST-Enterprise 6231, partially founded by Wallonia. H. Degée acknowledges for the support received from F.R.S.-FNRS. The strong support of the company Wienerberger for providing the material and building the test specimens is also to be emphasized.

REFERENCES

M. Tomazevic. (1999). Earthquake-Resistant Design of masonry Buildings. *Imperial College Press*.

A. Plumier & H. Degée. (2003). Guide technique parasismique belge pour maisons individuelles. *Technical brochure (in French)*. **http://hdl.handle.net/2268/104393**, 121 pp.

H. Degée & A. Plumier. (2006). Experimental investigation on the seismic behaviour of masonry housing in low seismicity area. *Proceedings of the 1st European Conference on Earthquake Engineering and Seismology*. **http://hdl.handle.net/2268/32367**, paper n°220, 10 pp.

H. Degée, A. Plumier, L. Lascar & L. Vasseur. (2012). Contribution of the spandrels and perpendicular walls to the seismic performance of masonry walls. *Proceedings of the SERIES international workshop "Role of research infrastructures in seismic rehabilitation"*. **http://web.itu.edu.tr/series/files/ss7-3_plumier.pdf**, presentation slides.

EN 1996-1-1. (2005). Eurocode 6: Design of masonry structures. General rules for reinforced and unreinforced masonry structures. *European Committee for Standardization*, Brussels.

EN 1998-1. (2005). Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings. *European Committee for Standardization*, Brussels.