Structural Engineering and Mechanics, Vol. 44, No. 2 (2012) 257-266
DOI: http://dx.doi.org/10.12989/sem.2012.44.2.257

Crack development depending on bond design for masonry walls under shear

A. Ural*[1] and A. Doğangün[2a]

[1]*Faculty of Engineering (Civil), Aksaray University, Aksaray, Turkey*
[2]*Faculty of Engineering (Civil), Uludag University, Bursa, Turkey*

(Received July 13, 2011, Revised August 17, 2012, Accepted September 28, 2012)

Abstract. Walls are the most important vertical load-carrying elements of masonry structures. Their bond designs are different from one country to another. This paper presents the shear effects of some structural bond designs commonly used for masonry walls. Six different bond designs are considered and modeled using finite element procedures under lateral loading to examine the shear behavior of masonry walls. To obtain accurate results, finite element models are assumed in the inelastic region. Crack development patterns for each wall are illustrated on deformed meshes, and the numerical results are compared.

Keywords: masonry; bonding design; inelastic behavior; crack development

1. Introduction

Masonry structures are built using earthen materials such as mud, stone, and clay brick units with mortar and can be found all over the world. Many masonry structures have been severely damaged during the destructive earthquakes, and this has caused a great number of human deaths and a large amount of economic loss in some countries, including Turkey. Even in countries with expert earthquake-engineering, most of the research is focused on the study of complex structures, such as relatively high-rise buildings, while little attention is given to masonry buildings. Masonry walls are the vertical load-carrying elements of the structure. These walls are generally intended to carry vertical loads, such as self weight and live loads. However, they are sometimes subjected to earthquakes with in-plane and out-of-plane directions. The behaviors of masonry walls subjected to shear have been studied experimentally and analytically by Lourenço *et al.* (2004) on stack bonded masonry prisms. In a similar study the influence of hollow brick masonry panels on the shear capacity of masonry walls was investigated by Gabor *et al.* (2006). Texture effects on the shear behavior of masonry walls were investigated by Berto *et al.* (2004) on running and stack bonded masonry wall models by using micro and macro modeling approaches. Crack development on masonry walls have been studied recent years by Reyes *et al.* (2009), Fathy *et al.* (2009), Fouchal *et al*. (2009).

*Corresponding author, Assistant Professor, E-mail: uralali@gmail.com
[a]Professor, E-mail: ademdogangun@gmail.com

Masonry walls have been frequently damaged due to earthquakes under shear effects. Damages have generally occurred at the interface between units and mortar because these interfaces are the weakest links in masonry walls. Masonry structures cannot be evaluated using numerical approaches like those for reinforced concrete structures. Although reinforced concrete is a heterogeneous material, it is possible to model it with the same types of finite elements. However, masonry exhibits distinct directional properties due to mortar joints. For this reason, a different modeling approach should be utilized for masonry structures as at the macro- and meso-levels. At the macro-level, masonry is assumed to be a composite element composed of units and mortar after some homogenization processes. This unit generally behaves as an orthotropic material, but it can be modeled with the anisotropic Rankine-Hill plasticity model. At the meso-level, masonry units are assumed to be continuous elements with elastic properties, and the joints are assumed to be interface elements with various inelastic properties. Two main approaches have been used at the meso-level: detailed and simplified approaches. In the detailed approach, the mechanical properties of units and mortar are taken into account separately. The probable cracks are assumed to be on the interface line between units and mortar. In the simplified approach, each joint, consisting of mortar and the unit-mortar interfaces, is lumped into an average interface, while units are expanded in order to keep the geometry unchanged. Masonry is thus considered to be a set of elastic blocks bonded by potential fracture/slip lines at the joints. A detailed description of these modeling approaches is provided by Zucchini *et al.* (2009), Chaimoon and Attard (2007), Zucchini and Lourenço (2002).

2. Inelastic behavior of masonry walls

Masonry walls are modeled with different inelastic properties depending on the modeling approach. Although Drucker-Prager (1952), Hill (1967) criteria are used under compression, the Rankine (1857) criterion is used under tension at the macro-modeling level. Furthermore, at the micro-level, the criterion must include all the basic types of failure mechanisms: cracking of the joints, sliding along the bed or head joints, cracking of the units in direct tension, diagonal tensile cracking of the units and masonry crushing. The first composite interface model to satisfy all these phenomena for the model was introduced by Lourenço (1996). Besides, many scientific researches and also experimental programs were focused and validated on this material model, i.e., Milani *et al.* (2005), van Zijl (2004), van Zijl *et al.* (2001), Pina and Lourenço (2004). In this study, because the masonry walls were modeled with the simplified approach, the composite interface criterion is easily explained in detail.

Each joint, consisting of mortar and the two unit-mortar interfaces, is lumped into an average interface while the sizes of the units are increased in order to keep the wall geometry unchanged in the simplified approach. Masonry is thus considered as a set of elastic blocks bonded by potential fracture/slip lines along the joints (Fig. 1).

Due to the zero thickness assumption of brick-mortar interfaces, the elastic properties of the enlarged units and interface joints must be adjusted to yield correct results. It is assumed that the elastic properties of the units remain unchanged because of the relative dimensions of mortar and units. Then, under the assumption of stack bond and uniform stress distributions both in the units and mortar, the components of the elastic stiffness are calculated as follows (Lourenço 1996)

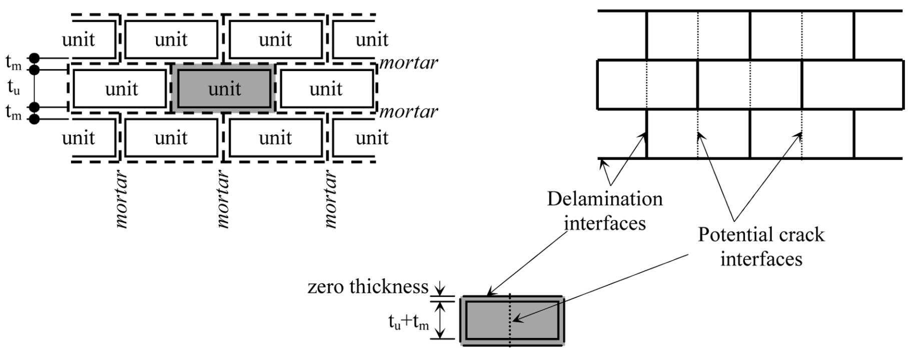


Fig. 1 Simplified micro modeling approach for masonry walls

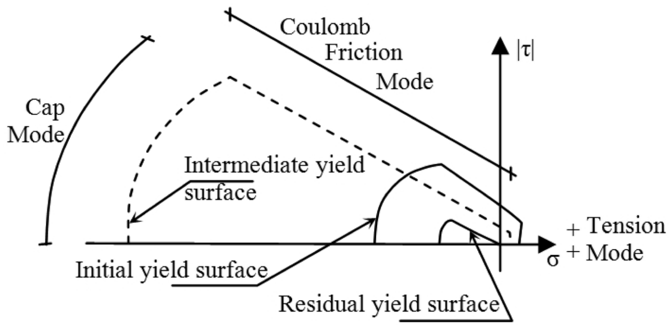


Fig. 2 Composite interface model (Lourenço 1996)

$$k_n = \frac{E_u \cdot E_m}{t_m(E_u - E_m)} \tag{1}$$

$$k_s = \frac{G_u \cdot G_m}{t_m(G_u - G_m)} \tag{2}$$

where E_u and E_m are the Young's modules, G_u and G_m are the shear modules of units and mortar, respectively, and t_m is the actual thickness of the mortar. More information on the micro modeling approach can be found in Lourenço (1996).

This inelastic constitutive model includes a tension cut-off for mode I failure, a Coulomb friction envelope for mode II failure, and a cap mode for compressive failure (Fig. 2)

For the tension mode, the yield function is

$$f_1(\sigma, \kappa_1) = \sigma - \sigma_1(\kappa_1) \tag{3}$$

where the principle stress or yield value is

$$\sigma_1 = f_t \exp\left(-\frac{f_t}{G_f^I}\kappa_1\right) \tag{4}$$

f_t is the tensile strength of the joint interfaces, G_f^I is the mode I fracture energy and κ_1 is the plastic relative displacement.

The yield function of the Coulomb friction criterion reads

$$f_2(\sigma, \kappa_2) = |\tau| + \sigma\tan\phi(\kappa_2) - \sigma_2(\kappa_2) \tag{5}$$

where the yield value σ_2 and friction angle $\tan\phi$ are

$$\sigma_2 = c\exp\left(-\frac{c}{G_f^{II}}\kappa_2\right) \tag{6}$$

$$\tan\phi = \tan\phi_0 + (\tan\phi_r - \tan\phi_0)\frac{c - \sigma_2}{c} \tag{7}$$

and c is the cohesion of the joint interfaces, ϕ_0 is the initial friction angle, ϕ_r is the residual friction angle and G_f^{II} is the mode II fracture energy.

The two-dimensional configuration of the compressive cap mode was first introduced by Lourenço (1996). The yield function for the cap mode is

$$f_3(\sigma, \kappa_3) = C_{nn}\sigma^2 + C_s\tau^2 + C_n\sigma - (\sigma_3(\kappa_3))^2 \tag{8}$$

where C_{nn}, C_s and C_n are the material parameters and σ_3 is the yield value.

3. Masonry bonds

Following the assumptions of the simplified micro-modeling approach, six different commonly used structural bond designs are modeled for determining their behaviors under shear loading. The considered masonry walls with different structural bonds are illustrated in Fig. 3.

The running bond is one of the simplest bond designs. Masonry units build crosswise up the wall. Thus, mortar joints cannot be aligned in the vertical direction. The stack bond is another very simple bond design, in which all vertical and horizontal joints are aligned. The Dutch bond is a variation of the English bond. It differs only in that the joints between the stretchers in the stretcher courses align vertically. These joints center on the headers in the courses above and below. The common, or American, bond is a variation of the running bond with a course of full-length headers at regular intervals that provide the structural bond as well as the pattern. Header courses usually appear at every fifth, sixth, or seventh course, depending on the structural bonding requirements. The English bond consists of alternating courses of headers and stretchers. The headers center over and under the stretchers. However, the joints between stretchers in all stretcher courses do not align vertically. In the Flemish bond, each course consists of alternating headers and stretchers. The headers in every other course center over and under the stretchers in the courses in between. The joints between stretchers in all stretcher courses align vertically (URL-1 2009).

The shear walls have a width/height ratio of one with dimensions 1800 × 1800 mm^2, built up with 12 courses, from which 11 courses are active and 1 course is clamped in a reinforced concrete beam. Vertical precompression uniformly distributed force $p = 0.30$ N/mm^2 is applied to the walls, before a horizontal load is monotonically increased under top displacement control d for keeping the bottom and top boundaries horizontal and precluding any vertical movement, see Fig. 4.

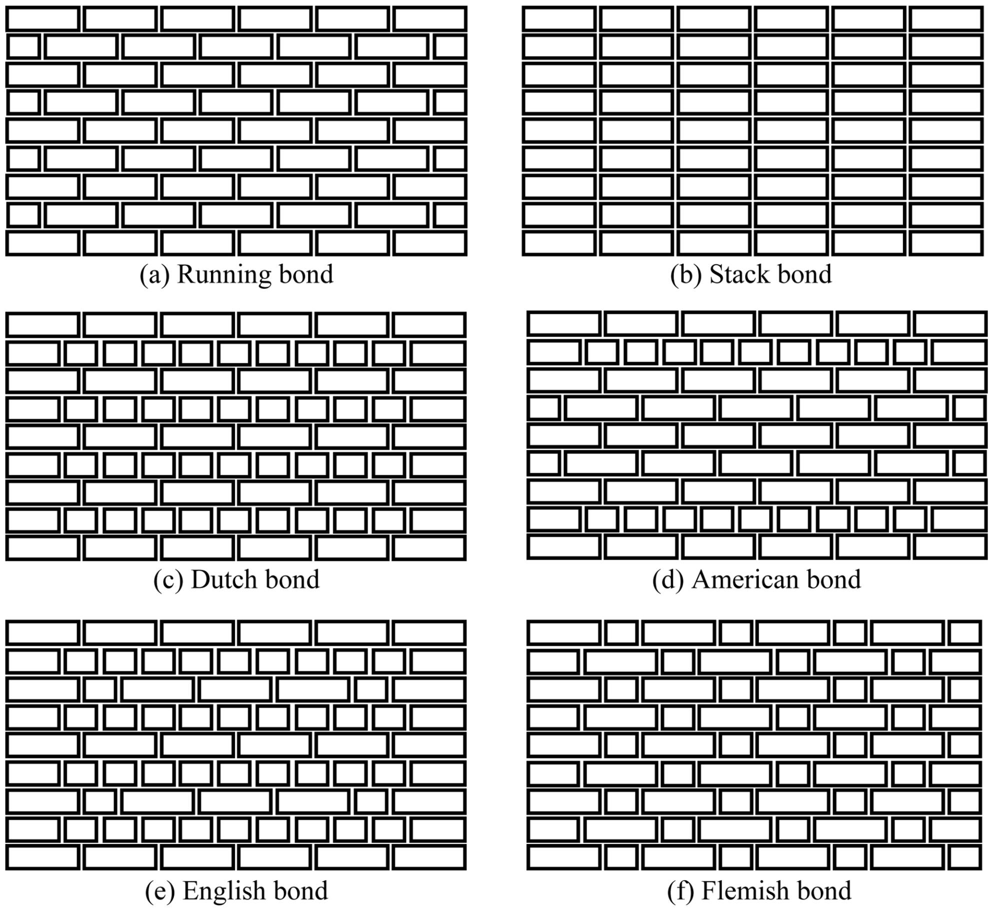


Fig. 3 Commonly used bond designs for masonry walls

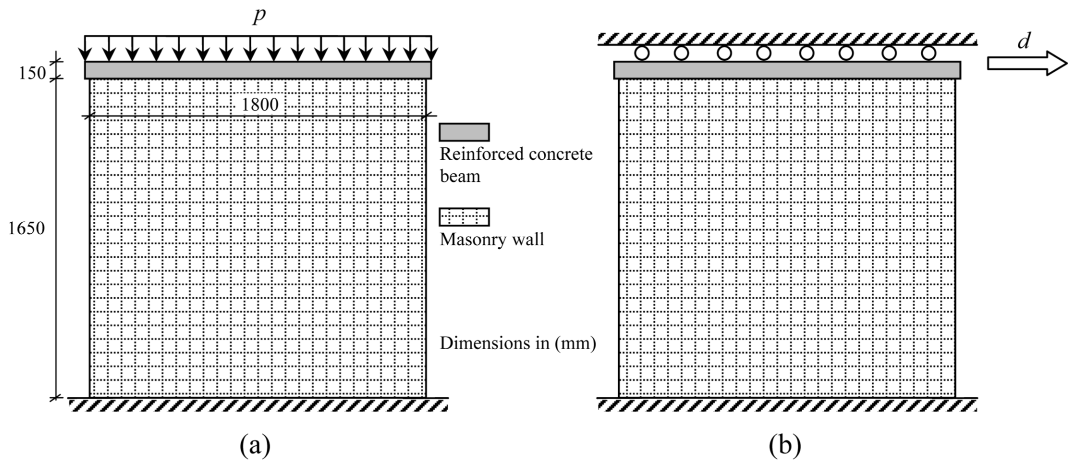


Fig. 4 Loads for different bond design masonry walls: (a) phase 1 – vertical loading, (b) phase 2 – horizontal loading under displacement control

Table 1 Elastic properties of materials

Masonry units		Top bond-beam		Potential crack interface		Delamination interfaces	
E	v	E	v	k_n	k_s	k_n	k_s
17500 (N/mm^2)	0,2	28000 (N/mm^2)	0,2	1×10^4 (N/mm^3)	1×10^3 (N/mm^3)	83,0 (N/mm^3)	36,0 (N/mm^3)

Table 2 Inelastic interface model parameters

Tensile strength, f_t (N/mm^2)	0.25
Tensile fracture energy, G_f (N/mm)	0.018
Cohesion, c (N/mm^2)	0.35
Friction angle, $\tan\phi$	0.75
Dilatancy angle, $\tan\psi$	0.60
Stress at which dilatancy is zero, σ_u (N/mm^2)	-1.3
Dilatancy softening gradient, δ	5
Compressive strength, f_c (N/mm^2)	8.5
Shear traction control factor, C_s	9.0
Compressive fracture energy, G_{fc} (N/mm)	5.0
Compressive plastic strain at f_c, κ_p	0.093
Fracture energy factor, b	0.05

The full brick dimensions are assumed to be $300 \times 150 \times 100$ mm^3 and the half brick dimensions are assumed to be $150 \times 150 \times 100$ mm^3 for all masonry models. Simplified micro modeling approach is used for the modeling and DIANA (2008) finite element software is used to perform the analyses. While one vertical potential crack interface is defined on the half masonry unit, four vertical potential crack interfaces are defined on the full length masonry units. In addition, delimitation interfaces are also defined on each side of all units/mortar intersections and have inelastic behaviors. Some elastic and inelastic material parameters used in the models are given below (Table 1 and Table 2).

All the nodes at the bottom of the models are fixed, and all nodes at the top of the models are fixed only in the y direction. The models are displaced at the top bond-beam level in the lateral or $+x$ direction in the analyses under monotonically increasing displacement. Inelastic nonlinear analyses are carried out for all masonry models, and the crack development and progress inside the walls are shown in Figs. 5-10.

The first cracks in all models developed near the top and bottom of the walls in the two opposite corners. Under increasing lateral loads, some diagonal cracks start to develop near opposite corners of the walls, except for the stacked bond model. The English bond design has two diagonal cracks under progressive load levels. Furthermore, the Flemish bond has irregular crack development due to the arrangement of masonry units. However, the running bond and American bond designs are similar with the development of diagonal cracks.

Load-displacement curves are collectively given in Fig. 11. In addition, the maximum values of loading and matched displacements to maximum loadings are summarized in Table 3.

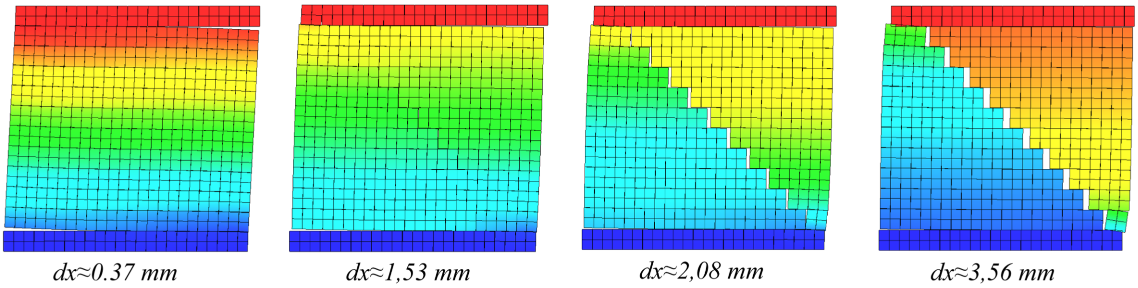


Fig. 5 Crack development of running bond design

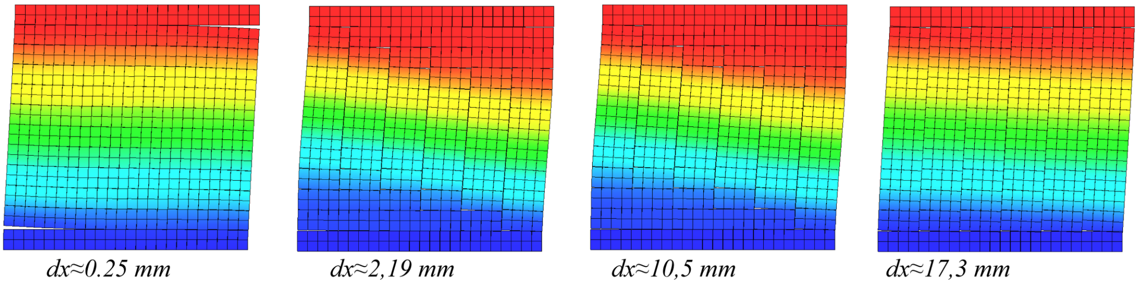


Fig. 6 Crack development of stack bond design

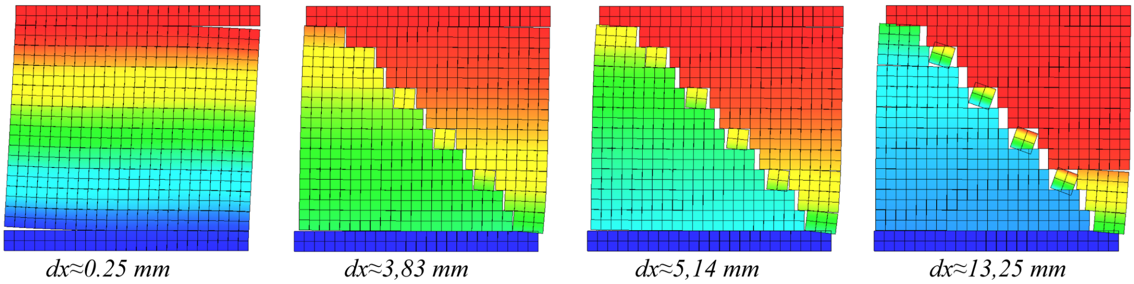


Fig. 7 Crack development of Holland bond design

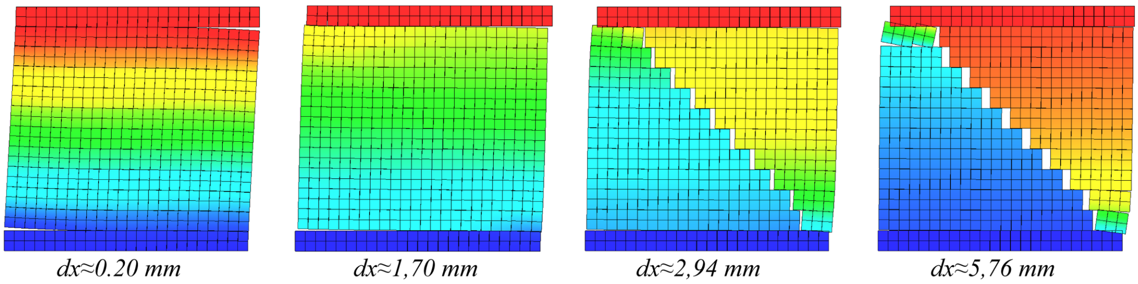
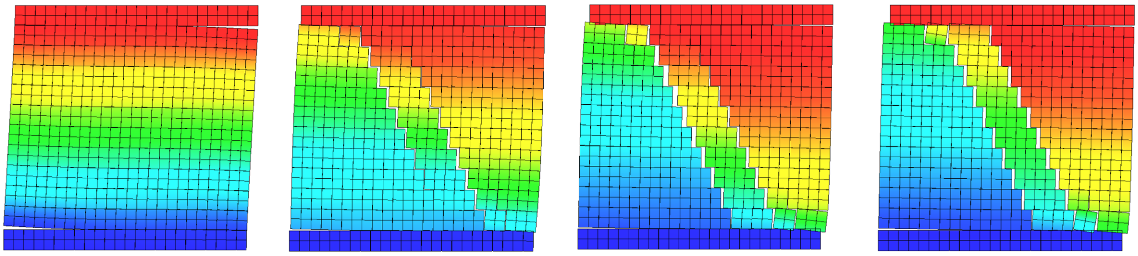
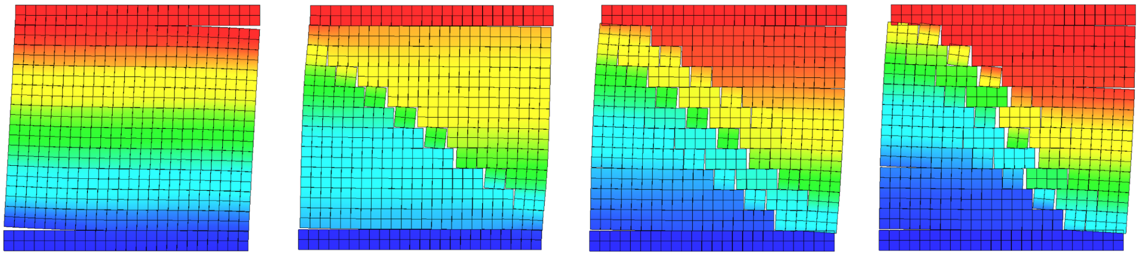


Fig. 8 Crack development of American bond design



dx≈0.25 mm dx≈2,93 mm dx≈5,05 mm dx≈8,07 mm

Fig. 9 Crack development of English bond design



dx≈0.25 mm dx≈2,22 mm dx≈5,58 mm dx≈9,58 mm

Fig. 10 Crack development of Flemish bond design

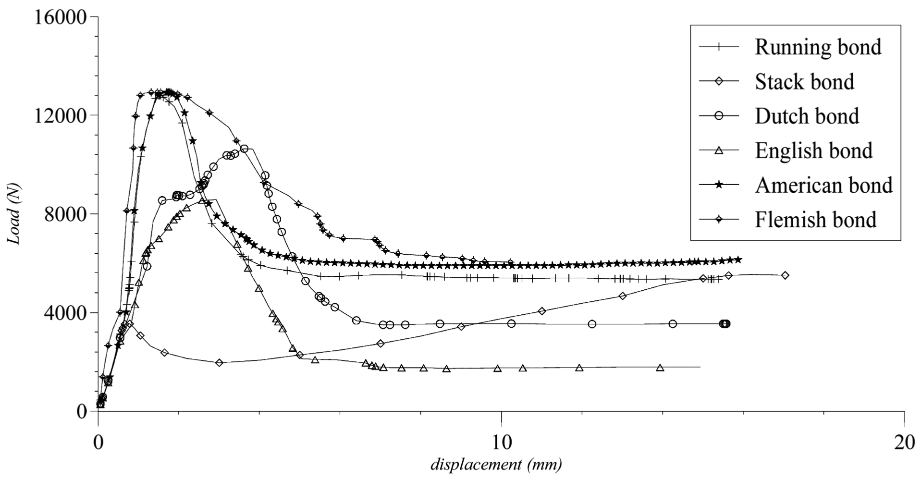


Fig. 11 Load-displacement curves belonging to different bond designs

Along with the results presented in Figs. 5-10 and Table 3, some observations are made on the crack development and progress of the wall models. Running, American and Flemish bond designs achieved a higher load-carrying level than the other bond designs. These bond designs reached a maximum load-carrying capacity of approximately 12 kN and reached an ultimate load-carrying capacity of approximately 6 kN. The Dutch and English bonds behave similarly, but the Dutch bond has a greater load-carrying capacity then the English bond. The Stack bond has the lowest load-carrying capacity of all the models studied with a capacity of approximately 3.5-4 kN.

Table 3 Structural analysis results of different bond designs

Model	Max. load (N)	Matched displacement to maximum load (mm)
Running bond	12808	1.53
Stack bond	5509	17.03
Dutch bond	10626	3.83
American bond	12937	1.77
English bond	8581	2.93
Flemish bond	12560	2.22

4. Conclusions

Due to their distinctive directional properties, masonry structures should be modeled with special techniques or approaches in numerical analyses. Three approaches are explained in this study to model masonry: detailed and simplified micro modeling and macro modeling.

Walls are the most important vertical load carrying elements of masonry structures. Thus, bond designs of these elements affect their structural stability. The lateral behaviors of masonry walls with six different bond designs are investigated in this study. The deformation and lateral load carrying capacity of these bonds are clearly explained by crack development illustrations. According to the results of the nonlinear inelastic analyses, running bond, American bond and Flemish bond designs are the optimum bond designs among these six models. Besides, the Stack bond design is not suggested for the construction of masonry walls because it has the lowest load carrying capacity of all the considered bond designs.

Acknowledgements

This study constitutes a portion of the first author's Doctoral research.

References

Berto, L., Saetta, A., Scotta, R. and Vitaliani, R. (2004), "Shear behaviour of masonry panel: parametric FE analyses", *Int. J. Solid Struct.*, **41**, 4383-4405.

Chaimoon, K. and Attard, M.M. (2007), "Modeling of unreinforced masonry walls under shear and compression", *Eng. Struct.*, **29**, 2056-2068.

DIANA (2008), *Finite Element Analysis Software,* Delft University, Delft, Netherlands.

Drucker, D.C. and Prager, W. (1952), "Soil mechanics and plastic analysis or limit design", *Q. Appl. Math.,* **10**(2), 157-175.

Fathy, A.M., Planas, J. and Sancho, J.M. (2009), "A numerical study of masonry cracks", *Eng. Fail. Anal.*, **16**, 675-689.

Fouchal, F., Lebon, F. and Titeux, I. (2009), "Contribution to the modeling of interfaces in masonry construction", *Constr. Build. Mater.*, **23**, 2428-2441.

Gabor, A., Ferrier, E., Jacquelin, E. and Hamelin, P. (2006), "Analysis and modeling of the in-plane shear

behavior of hollow brick masonry panels", *Constr. Build. Mater.,* **20**, 308-321.
Hill, R. (1967), *The Mathematical Theory of Plasticity*, Oxford University Press, London, UK.
Lourenço, P.B. (1996), "Computational strategies for masonry structures", Ph.D. Thesis, Delft University of Technology, Delft, Netherlands.
Lourenço, P.B., Barros, J.O. and Oliveira, J.T. (2004), "Shear testing of stack bonded masonry", *Constr. Build. Mater.*, **18**, 125-132.
Milani, G., Lourenço, P.B. and Tralli, A. (2005), "A simple homogenized micro mechanical model for the analysis at the collapse of out-of-plane loaded masonry walls", *XVII Congresso AIMETA di Meccacica Teorica e Applicata*, Frenze, Italia.
Pina-Henriques, J. and Lourenço, P.B. (2004), "Masonry micro modeling adopted a discontinuous framework", *Proceedings of Seventh International Conference on Computational Structures Technology*, Lisbon, Portugal.
Rankine, W. (1857), *On the Stability of Loose Earth,* Philosophical Transactions of the Royal Society of London, Vol.147.
Reyes, E., Galvez, J.C., Casati, M.J., Cendon, D.A., Sancho, J.M. and Planas, J. (2009), "An embedded cohesive crack model for finite element analysis of brickwork masonry fracture", *Eng. Fract. Mech.,* **76**(12), 1930-1944.
URL-1 (2009), Masonry, http://en.wikibooks.org/wiki/Adventist_Youth_Honors_Answer_Book/Vocational/Masonry.
Van Zijl, G.P.A.G. (2004), "Modeling masonry shear-compression: role of dilatancy highlighted", *J. Eng. Mech.*, **130**(11), 1289-1296.
Van Zijl, G.P.A.G., Rots, J.G. and Vermeltfoort, A.T. (2001), "Modelling shear-compression in masonry", *Proceedings of 9th Canadian Masonry Symposium*, Fredericton, Canada.
Zucchini, A. and Lourenço, P.B. (2009), "A micro-mechanical homogenisation model for masonry: application to shear walls", *Int. J. Solids Struct.*, **46**, 871-886.
Zucchini, A. and Lourenço, P.B. (2002), "A micro mechanical model for the homogenization of masonry", *Int. J. Solids Struct.*, **39**, 3233-3255.