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Experimental and numerical investigation on collar-jointed masonry walls

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ABSTRACT: This paper presents experimental and numerical studies on pre-damaged and post-damaged collar-jointed masonry walls subjected to external loading. The collar-jointed masonry walls were created by constructing a wall parallel to an existing single-leaf wall and “tied” to it using collar joint technique. For the pre-damaged case, both walls were constructed and bonded together at the same time. For the post-damaged case, the single leaf masonry wall was tested initially until partial damage occurred and then a second leaf wall was constructed and bonded to it, making it essentially a means of retrofitting. From the results analysis, it was found that the pre-damaged wall can carry 50% more load than the post-damaged one. Also, a three dimensional numerical model has been developed using the Finite Element (FE) software MIDAS FEA. Bricks were modeled as rigid bodies connected together with zero thickness interfaces representing the mortar joints. The numerical results were compared against the experimental findings and good agreement obtained.

1 INTRODUCTION

Masonry is a composite material made of masonry units bonded together with or without mortar. In the past decades, several researchers have conducted studies on the mechanical behavior of single leaf masonry wall panels subjected to external loading (Lourenco 1996; Chaïmoon and Attard 2007, Sarhosis and Sheng 2014) with the aim to understand their mechanical behavior and inform repair and/or strengthening measures. However, the behavior of double-leaf masonry walls as a means of a retrofitting technique has not been studied in detail as yet.

According to BS 5628-1:(2005), a double-leaf (collar jointed) wall is defined as *“two parallel single-leaf walls, with a space between not exceeding 25 mm, filled solidly with mortar and so tied together as to result in common action under load”*. Double-leaf masonry walls are common in modern constructions as they are characterized by soundproof, fireproof, and waterproof characteristics. Furthermore, double-leaf (collar jointed) walls can also be used to improve a structure’s lateral stability

(e.g. against wind or blast loading) through adding stiffness..

Anand and Yalamanchili (1996) investigated both experimentally and numerically the mechanical behavior of a composite walls made of brick and hollow concrete blocks connected together by collar joint. From the results analysis, it was found out that collar joint failed in brittle manner. Pian-Henriques et al. (2004) conducted experimental tests on three-leaf masonry walls under shear and compressive load. The two leaves were connected with two different types of collar joints including: a) straight collar joint; and b) keyed collar joint. It was found out that structures made with different types of collar joints will behave differently under the application of external load. For the wall panels constructed with a straight collar joint, vertical shear failure occurred. However, for the wall panels constructed with keyed collar joints, failure was mainly due to the diagonal cracks in the inner leaf. Ramalho et al. (2008) undertook numerical investigations with the aim to simulate the aforementioned experimental tests (Pian-Henriques et al. 2004) by applying a non-local damage constitutive law.

The constitutive law allowed the damage-induced anisotropy accompanying the cracking process to be described. However, as perfect bonding was assumed between the adjacent layers during the modeling, both the overall stiffness and peak-load were overestimated.

This paper investigates experimentally and numerically the mechanical behavior of double-leaf collar jointed brickwork masonry walls as a means of a retrofitting technique. Different configurations of wall panels tested in the laboratory. These are: a) two pre-damaged walls; and b) one post-damaged wall. For the construction of the pre-damaged walls, the second wall was built parallel to the existing one and bonded with a 10 mm thick collar joint before testing. For the post-damaged wall, the second wall was attached to the existing one after it had been tested (and as such partially damaged).

Also, a three dimensional finite element numerical model based on the simplified micro-modeling approach (Lourenco 1996) has been developed. The commercial software MIDAS FEA (2008) has been used. Bricks were divided into elastic constitutive elements while the mortar joints were smeared into zero-thickness interfaces where shear sliding and de-bonding is allowed to occur. In order to keep the geometry unchanged, the brick units were expanded by taking the smear out of mortar into consideration. Moreover, predefined vertical zero thickness interfaces were constructed through the masonry units to allow failure of the bricks (Dolatshahi 2011), making these regions similarly quite prone to forming separations (Lourenco and Rots 1997). The suitability of the numerical model to predict the mechanical behavior of collar jointed masonry walls were verified against the experimental tests carried out in the laboratory.

2 EXPERIMENTAL WORK

2.1 Experimental tests' arrangements and procedure

Two pre-damaged (Pre-W1 and Pre-W2) and one post-damaged (Post-W3) full scale double leaf brickwork masonry walls were tested in the laboratory. For the pre-damaged case, both walls were constructed and bonded together at the same time. For the post-damaged case, a single leaf masonry wall ("reference" wall) was initially constructed and tested until partial damage occurred. Then a second leaf wall ("secondary" wall) was constructed and "tied" to it, making it essentially a means of retrofitting.

All walls were 975 mm × 900 mm x 102.5 mm (height x span x breadth) and had thirteen courses of stretcher bonded brickwork. Walls were constructed with UK standard size bricks 225 x 102.5 x 65 mm. The bricks were perforated Engineering Class B (BS 6100-1:2004 and BS 3921:1985). The mortar joints were all nominally 10 mm thick with composition 1 : 1 : 6 (Cement: Lime: Sand); this is designation (N) as defined by (BS 6100-1:2004). Also, the brickwork masonry walls were "tied" together with a 10 mm thick mortar joint. A 20 mm gap allowed between the left hand side of the "reference" wall and the steel column of the portal frame. The 20 mm gap filled with mortar up to a height of 200 mm from the floor.

Collar joint masonry walls tested by applying a horizontal and vertical point load at the top left hand side of the "reference" wall. The load applied to the wall using two hydraulic rams and was distributed through two thick steel spreader plates which were embedded in mortar on the top left hand corner of the brickwork panel. External load applied to the "reference" wall only. The load applied incrementally until the panel could no longer carry the applied load. At each load increment, deflections at the top left hand corner of the double leaf panels recorded with the use of a LVDT. Also signs of crack development monitored visually. Typical details of the double-leaf wall panels are shown in Figure 1.

Figure 1. Experimental set up of a typical collar jointed masonry wall

2.2 Experimental results and discussion

a) Pre-damaged collar jointed masonry walls

The experimental tests showed that the pre-damaged Pre-W1 and Pre-W2 collar joint brickwork masonry walls failed by diagonal cracking. Figure 2 shows the failure pattern of the Pre-W2. There were three notable features of behavior of the pre-damaged collar joint masonry wall panels namely: i) initial flexural cracking in the bed joints of the walls; followed by, ii) propagation of stepped shear cracks; with increasing load leading to, iii) complete collapse. The collar joint has not failed (Figure 4a). Also, more cracks observed in the “reference” wall rather than the “secondary” wall. This is due to the fact that the load applied at the “reference” wall panel only. The load was transferred to the second leaf via the collar joint. Also, the “secondary” wall weren’t restrained and was therefore less stiff compared to the “reference” wall panel.

b) *Post-damaged collar jointed masonry walls*

For the post-damaged double leaf collar joint masonry wall panel Post-W3, there were four notable features of behavior namely: i) initial flexural crack; followed by ii) formation of diagonal stepped cracks from the top right hand side of the panel to the bottom left hand side; with increasing load leading to iii) detachment of the collar joint from the wall; and iv) collapse as a result of shear failure. Figure 3 shows the failure mode of the Post-W2 panel while Figure 4b shows the de-bonding of the collar joint for the “reference” wall. During testing, once de-bonding of the collar joints occurred, the “reference” and the “secondary” walls behaved individually.

(a) Front side (“reference” wall)

(b) Back side (“secondary” wall)

Figure 2. Failure pattern of the pre-damaged collar jointed masonry wall Pre-W2

(a) Front side ("reference" wall)

(b) Back side ("secondary" wall)

Figure 3. Failure pattern of the post-damaged collar jointed masonry wall Post-W3

(a) Top side of Pre-W2

(b) Top side of Post-W3

Figure 4. View of the collar jointed masonry walls after failure

2.3 Load against deflection curves

Figure 5 shows the load-deflection curves for the double leaf collar-joint masonry wall panels tested in the laboratory. All panels behaved in an elastic-plastic behavior before the peak load. The Pre-W1 and Pre-W2 collar jointed walls failed at 91 kN and 93 kN respectively. In spite the large variation of masonry, the experimentally observed ultimate load from both wall panels is similar. However, the deflection of Pre-W1 and Pre-W2 looks different. This is because the LVDT measurement of Pre-W2 slipped during testing. For the post-damaged collar jointed masonry wall Post-W3, the ultimate load obtained experimentally was 73 kN.

Figure 5. Load-Deflection relationship of the pre-damaged and post-damaged collar jointed walls

a) Pre-damaged collar jointed walls

Variable	Brick-mortar interface	Collar joint	Since the collar jointed walls have been constructed at the same time and were cured under identical
k_n (N/mm ³)	13	9	
k_s (N/mm ³)	5.3	3.6	
f_t (N/mm ²)	0.4	0.22	
G_f^I (N/mm)	0.022	0.018	
C (N/mm ²)	0.56	0.30	
$\tan \theta$	0.75	0.75	
$\tan \varphi$	0.56	0.56	
G_f^{II} (N/mm)	0.175	0.2	

3 COMPUTATIONAL MODELLING OF DOUBLE LEAF MASONRY WALL PANELS

Three dimensional geometric models representing the masonry wall panels tested in the laboratory have been developed in MIDAS (2013). Masonry bricks represented using 8-noded hexahedron solid elements separated by zero thickness interfaces at each mortar bed and perpendicular joint. To allow for the 10 mm thick mortar joints in the real wall panels, each masonry unit was based on the nominal brick size used in the laboratory built panels increased by 5 mm in each dimension to give a block size of 225 mm x 102.5 mm x 75 mm. It was assumed that the bricks would exhibit linear elastic behavior and that slip along the mortar joints would be the predominant failure mechanism. Mortar joints were represented using a Mohr-Coulomb failure surface combined with a tension cut-off. Also, the base of the wall fixed in all directions. Load applied incrementally until collapse occurred. The model implemented in MIDAS FEA is shown in Figure 6.

Figure 6. Micro-modelling strategy for modeling masonry (Lourenco 1996)

conditions, their mechanical properties assumed the same. The material parameters used for the development of the computational models are shown in Table 1.

Table 1. Properties of different interfaces (Wang et al. 2015)

k_n = normal stiffness, k_s =shear stiffness, f_t = tensile strength, G_f^I = tensile fracture energy adhesion, C = cohesion, $\tan \theta$ = friction coefficient, $\tan \varphi$ = dilatancy coefficient and G_f^{II} = shear fracture energy.

Figure 7 compares the experimental results against the predicted using the numerical model load-deflection curves for the Pre-W1 collar jointed masonry walls. From

Figure 7, the numerical model can capture with sufficient accuracy of the ultimate load and stiffness of the panel obtained from the experiment. Figures 8 and 9 compare the experimental against the numerical failure pattern of the Pre-W1 panel. The development of the cracks at different stages of applied loading and failure mode were very similar to the behavior observed in the laboratory.

Figure 7. Load-deflection curve of Pre-W1

(a) Experimental results

(b) Numerical results

Figure 8. Failure pattern of the Pre-W1 (back side)

(a) Experimental

(b) Numerical

Figure 9. Failure pattern of Pre-W1 (front side)

b) Post-damaged collar jointed masonry wall

For the simulation of the mechanical behavior of the post-damaged wall (Post-W3), the presence of initial cracking in the “primary” wall has been taken into consideration in the numerical analysis. So, a grid of pre-defined cracks has been constructed; this is based on the

experimental observations (Figure 10a). is the grid is represented by dashed lines in Figure 10 (b). During the development of the numerical model, the zero thickness interfaces representing the crack formations were assumed open. The boundary conditions used for the development of Post-W1 were identical as in the case of the Pre-damaged double-leaf walls (Pre-W1 & Pre-W2).

(a) Cracking observed in the experiment

(b) Pre-defined cracks in the numerical model

Figure 10. Cracking in masonry wall panel

The “preliminary” and the “secondary” masonry walls were cured at different times. Thus, their strength properties will be different. As the “preliminary” wall has been cured for a longer time than the “secondary” one, the strength properties of the “preliminary” wall are higher than the properties of the “secondary” one. The collar joint in post-damaged wall is modeled differently with the one in pre-damaged walls. In this model, the collar joint hasn’t been smeared out. This is because the interface 1 (interface between first leaf and collar joint) is different with the interface 2 (interface between the second leaf and collar joint). The bond strength of interface 2 is stronger because the collar joint and second leaf were cured together, which can provide a better bond effect. This can be proved from the experimental results (Figure 4b) that the collar joint was still connected with the “secondary” wall. Based on the above findings and assumptions, the extended table of material parameters is given in Table 2.

Figure 11 shows the experimental against the numerical load-deflection curve of Post-W3. From Figure 11, the numerical model can predict with sufficient accuracy the ultimate load and stiffness of the collar jointed wall obtained from the experiment. Figure 12 and 13 compares the experimental against the numerical failure patterns. Good agreement has been observed. The numerical model can simulate the crack development and failure mode with sufficient accuracy.

Table 2. Properties of different interfaces (Wang et al. 2015)

Variable	First leaf mortar	Second leaf mortar	Collar joint interface 1	Collar joint interface 2
k_n (N/mm ³)	15	13	10	12
k_s (N/mm ³)	6.3	5.3	4.2	5
f_t (N/mm ²)	0.25	0.18	0.14	0.18
G_f^I (N/mm)	0.018	0.014	0.011	0.014
C (N/mm ²)	0.35	0.25	0.2	0.25
$\tan \theta$	0.75	0.75	0.75	0.75
$\tan \varphi$	0.56	0.56	0.56	0.56
G_f^{II} (N/mm)	0.2	0.125	0.2	0.125

Figure 11. Load-deflection curve of Pre-W3

(a) Experimental results

(b) Numerical results

Figure 12. Failure pattern of Post-W3 (back side)

(a) Experimental results

(b) Numerical results

Figure 13. Failure pattern of Post-W3 (front side)

4 CONCLUSIONS

A series of experimental studies on pre-damaged and post-damaged double-leaf brickwork masonry wall panels have been conducted. The collar-jointed masonry walls were constructed by building a wall parallel to an existing single-leaf wall and “tied” to it using collar joint technique. For the pre-damaged case, both walls were constructed and bonded together at the same time. For the post-damaged case, the single leaf masonry wall was tested initially until partial damage occurred and then a second leaf wall was constructed and bonded to it, making it essentially a means of retrofitting. From the analysis of the experimental results, the following conclusions are made:

- a. There were three notable features of behavior of the pre-damaged collar jointed masonry wall panels namely: i) initial flexural cracking in the bed joints of the walls; followed by, ii) propagation of stepped shear cracks; with increasing load leading to, iii) complete collapse.

- b. For the post-damaged collar jointed masonry wall Post-W3, there were four notable features of behavior namely: i) initial flexural crack; followed by ii) diagonal stepped shear cracks from the top right hand side of the panel to the bottom left hand side; with increasing load leading to iii) detachment of the collar joint from the wall; and iv) collapse as a result of shear failure.
- c. The pre-damaged collar jointed wall can carry 50% more load than the post-damaged one.

Furthermore, a three dimensional model based on the simplified micro-modeling approach has been developed to simulate the mechanical behavior of double leaf collar joint brickwork masonry wall panels. The proposed model was used to predict the behaviour of the wall panels tested in the laboratory. Good correlation was achieved between the predicted behaviour of the collar joint masonry wall panels and that observed in the laboratory.

Reference

- Anand, S.C. & Yalamanchili, K.K. 1996. Three-dimensional failure analysis of composite masonry walls. *Journal of Structural Engineering*, 122(9): 1031-1039.
- British Standard Institution. (1985). BS 3921: 1985. *Specification for Clay bricks*. London: BSI
- British Standard Institution. (2004). BS 6100-1: 2004. *Building and civil engineering-Vocabulary- Part 1: General terms*. London: BSI
- British Standard Institution. (2005). BS 5628-1:2005. *Code of practice for the use of masonry-Par1: Structural use of unreinforced masonry*. London:BSI
- Chaimoon, K. & Attard, M.M. 2007. Modelling of unreinforced masonry walls under shear and compression. *Engineering Structures*, (29): 2056-2068.
- Dolatshahi, K.M. & Aref, A.J. 2011. Two-dimensional computational framework of meso-scale rigid and line interface elements for masonry structures. *Engineering Structures*, 33(12): 3657-3667.
- Lourenco, P.B. 1996. *Computational strategies for masonry structures*. PhD thesis. Netherlands: Delft University of Technology.
- Lourenco, P.B. & Rots, J.G. 1997. Multi-surface interface model for analysis of masonry structures. *Journal of Engineering Mechanics*, 123(7): 660-668.
- MIDAS FEA. 2013. MIDAS Information Technology Co.

- Pina-Henriques, J., Lourenco, P.B., Binda, L. & Anzani, A. 2004. Testing and modelling of multiple-leaf masonry walls under shear and compression. *In: Modena C, Lourenco PB, Roca P, editors. Proceedings of the fourth international seminar on structural analysis of historical constructions*, Balkema: Leiden (NL); 299-310.
- Ramalho, M.A., Taliencio, A., Anzani, A., Binda, L. & Papa, E. 2008. A numerical model for the description of the nonlinear behaviour of multi-leaf masonry walls. *Advances in Engineering Software*, (39); 249-257.
- Sarhosis, V. & Sheng, Y. 2014. Identification of material parameters for low bond strength masonry, *Engineering Structures*, (60):100-110.
- Wang, C., Forth, J.P. & Nikitas. N. 2015. Retrofitting of masonry walls by using a mortar joint technique; experiments and numerical validation. Manuscript submitted for publication