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Influence of brick–mortar interface on the mechanical behaviour of low bond strength masonry brickwork lintels

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ABSTRACT

A study of the influence of the brick–mortar interface on the pre- and post-cracking behaviour of low bond strength masonry wall panels subjected to vertical in plane load is presented. Using software based on the Distinct Element Method (DEM), a series of computational models have been developed to represent low bond strength masonry wall panels containing an opening. Bricks were represented as an assemblage of distinct blocks separated by zero thickness interfaces at each mortar joint. A series of sensitivity studies were performed supported with regression analysis to investigate the significance of the brick–mortar interface properties (normal and shear stiffnesses, tensile strength, cohesive strength and frictional resistance) on the load at first cracking and ultimate load that the panel can carry. Computational results were also compared against full scale experimental tests carried out in the laboratory. From the sensitivity analyses it was found that the joint tensile strength is the predominant factor that influences the occurrence of first cracking in the panel, while the cohesive strength and friction angle of the interface influence the behaviour of the panel from the onset of cracking up to collapse.

1. Introduction

Masonry is a combination of units such as stones, bricks or blocks usually laid in a cementitious or lime mortar. It is probably the oldest material used in construction and has proven to be both simple to build and durable. In spite of its simplicity of construction, the analysis of the mechanical behaviour of masonry structures remains a challenge. Masonry is a heterogeneous, anisotropic material whose mechanical response is non-linear even under low stress levels. It is the presence of mortar joints in a masonry structure, that create a resistance to tensile stress, that is markedly less than the material's resistance to compression. When subjected to very low levels of stress, masonry behaves approximately in a linear elastic manner. This becomes increasingly non-linear after the formation of cracks and the subsequent redistribution of stresses through the uncracked material as the structure approaches collapse. The behaviour of masonry is complicated even further by the inherent variations in the natural materials used, variations in workmanship, the effects of deterioration caused by weathering processes and the development of other defects during the life of the masonry structure [10]. Also, depending on the magnitude and direction of shear and normal stresses applied on masonry, different failure modes occur (Fig. 1).

This paper focuses on low bond strength masonry. Experimental evidence [2,3,9] has shown that cracking of such masonry is as a result of the de-bonding of the masonry units from the mortar joints and the post-cracking response up to collapse is influenced by the characteristics of the unit-mortar interface. Low bond strength masonry can be encountered in historic constructions where mortar has been deteriorated over time, as well as brick and stone arch bridges, tunnels linings and earth retaining walls where the unit-mortar bond has been disrupted by the action of water leeching through the masonry over a long time. Low bond strength masonry can also be found in more recent examples of construction. Experience shows that many low to medium rise domestic masonry buildings in UK contain low bond strength mortar. This is often caused by the use of low cement content mortar due to lack of quality control on site. A typical example of cracked brickwork above a window opening is shown in Fig. 2. The need to better understand the pre- and post-cracking behaviour of low bond strength masonry in order to inform decisions concerning repair and strengthening has led to the development of several numerical methods of analysis [18,15]. These tend to be focusing on either the micro-modelling approach where individual masonry

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units (i.e. bricks, stones) and the mortar are modelled separately, or on the macro-modelling approach in which masonry is considered as composite material.

The aim of this study is to investigate the influence of the brick–mortar interface on the pre- and post-cracking behaviour of low bond strength masonry wall panels with openings subjected to vertical in plane load. Given the importance of the masonry unit-mortar interface on the structural behaviour of low bond strength masonry, the micro-modelling approach based on the Discrete Element Method (DEM) of analysis has been adopted in this study. The software used was the Universal Distinct Element Code (UDEC) which was developed initially to model sliding rock masses in which failure occurs along the joints [7]. This has similarities with the behaviour of low bond strength masonry. An additional feature of UDEC is the capability to predict the onset of cracking; this is important when considering in-service as well as near-collapse behaviour of masonry structures. Typical examples of masonry structures modelled using UDEC are described by Schlegel and Rautenstrauch [23], Zhuge [25], Lemos [13], Sarhosis et al. [22].

The UDEC model developed by Sarhosis and Sheng [20] to simulate the pre-and post cracking of low bond strength masonry wall panels with openings has been adopted in this study. Bricks were represented as an assemblage of distinct blocks while the mortar joints were modelled as zero thickness interfaces which can open and close depending on the magnitude and direction of the stresses applied to them. A series of sensitivity studies were performed supported with regression analysis to investigate the significance of the brick–mortar interface properties on the load at first cracking and ultimate load that the panel can carry. Results from the developed numerical models are also compared against a series of full scale low bond strength masonry wall panels with openings tested in the laboratory.

2. Numerical modelling of masonry structures with UDEC

2.1. Overview

UDEC is a numerical program based on the distinct element method for discontinuous modelling [12]. When used to model brickwork structures, the bricks are represented as an assemblage of rigid or deformable distinct blocks which may take any arbitrary geometry. Rigid blocks do not change their geometry as a result of the applied loading and are mainly used when the behaviour of the system is dominated by the mortar joints. Deformable blocks are internally discretised into finite difference triangular zones and each element responds according to a prescribed linear or non-linear stress–strain law. Mortar joints are represented by zero thickness interfaces between adjacent blocks. In DEM, the unknowns are solved explicitly by the differential equations of Newton's Second law of motion at all bricks and the force–displacement law at all contacts. The force–displacement law is used to find the contact forces from known displacements while Newton's second law gives the motion of the blocks resulting from the known forces acting on them [7]. Convergence to static solutions is obtained by means of adaptive damping, as in the classical dynamic relaxation methods [13,17]. Large displacements and rotations between the blocks, including their complete detachment, are also allowed with the sequential contact detection and update as the calculation progresses.

2.2. Joint interface model

At the interfaces, the bricks are connected kinematically to each other by set of point contacts. These contact points are located at

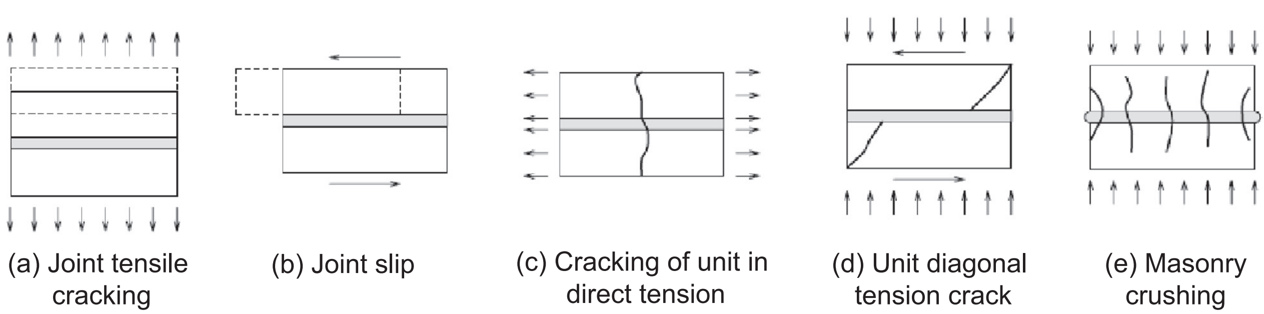


Fig. 1. Failure modes in masonry [14].

Fig. 2. Damage in a masonry wall above a window opening (Courtesy of Bersche Rolt Ltd).

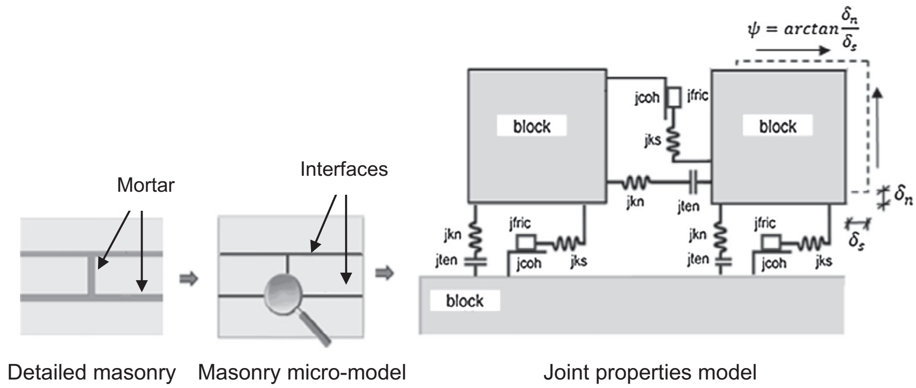


Fig. 3. Interface model [21].

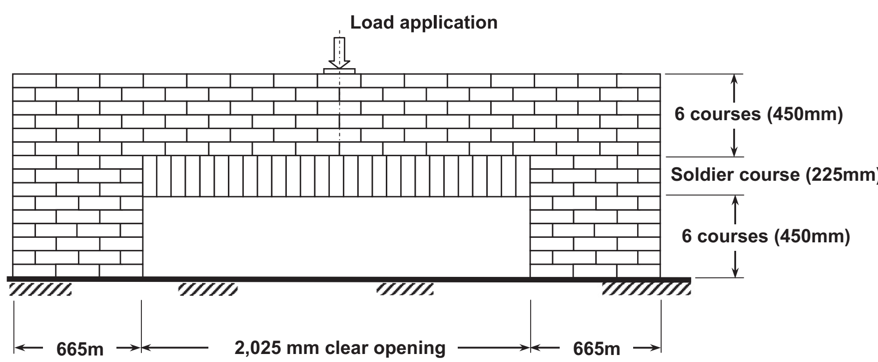


Fig. 4. Typical masonry wall panel with opening tested in the laboratory.

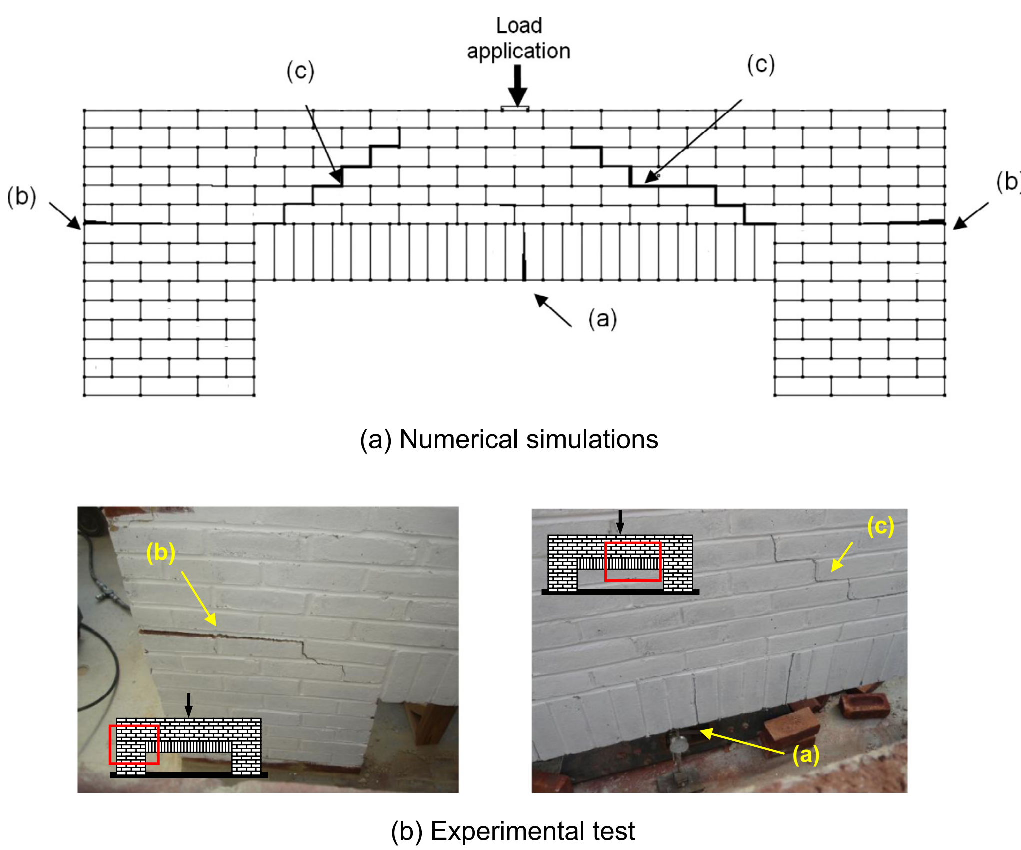


Fig. 5. Crack locations identified in the test panel: (a) from the computational analysis and (b) from the experimental tests (to be read in conjunction with Fig. 6).

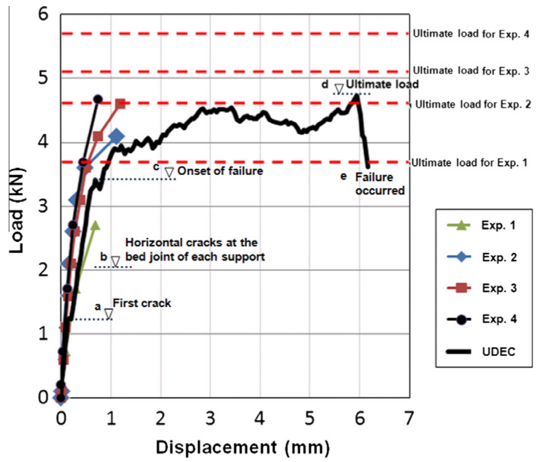


Fig. 6. Comparison of experimental against numerical results.

the outside perimeter of the bricks and are created at the corners of the bricks and the zones [13]. In the normal direction, the mechanical behaviour of mortar joints is governed by Eq. (1):

$$\Delta\sigma_n = -\mathrm{JKn} \cdot \Delta u_n, \tag{1}$$

where JKn is the normal stiffness of the contact, $\Delta\sigma_n$ is the change in normal stress and Δu_n is the change in normal displacement. Similarly, in the shear direction the mechanical behaviour of mortar joints is controlled by a constant shear stiffness JKs using the following expression:

$$\Delta\tau_s = -\mathrm{JKs} \cdot \Delta u_s, \tag{2}$$

where $\Delta\tau_s$ is the change in shear stress and Δu_s is the change in shear displacement. Stresses calculated at grid points along contacts are submitted to the Mohr–Coulomb failure criterion which limits shear stresses along joints. The following parameters are used to define the mechanical behaviour of the contacts (Fig. 3): the normal stiffness (JKn), the shear stiffness (JKs), the friction angle (Jfric), the cohesion (Jcoh), the tensile strength (Jten) and the dilation angle

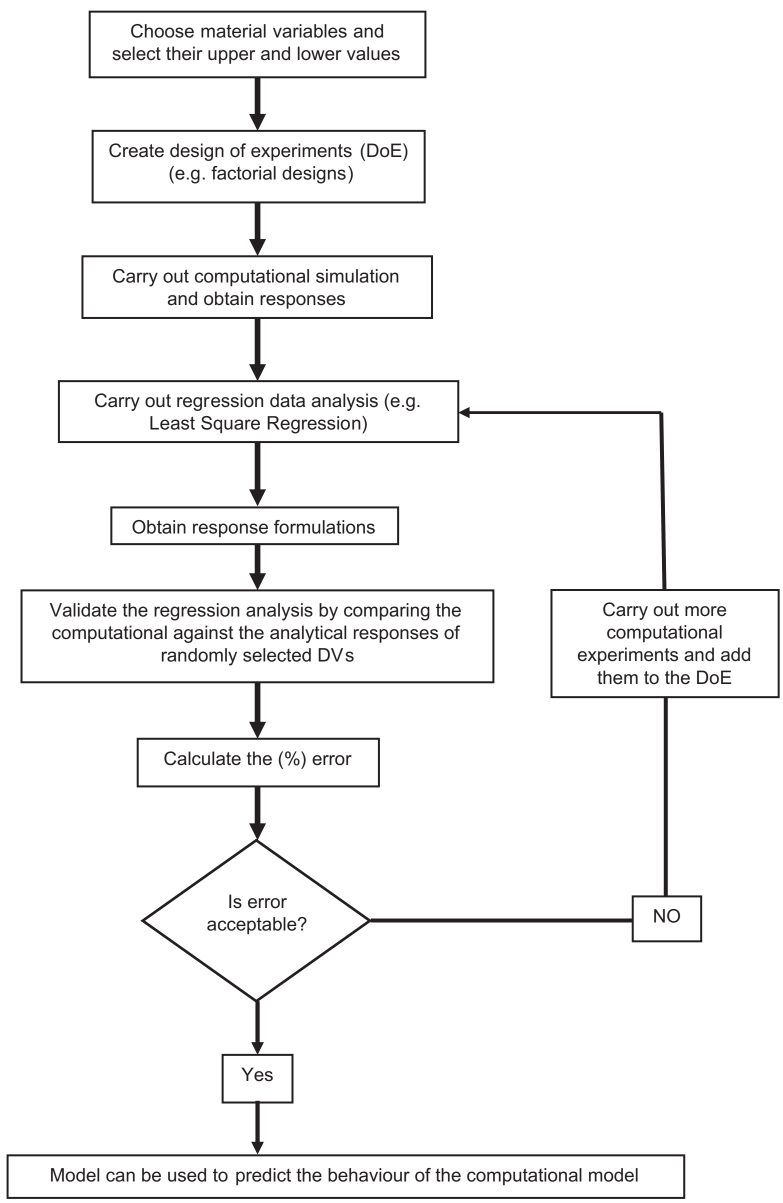


Fig. 7. Methodology for determination of response surfaces of model behaviour.

(Jdil). For the proposed model, there is a limiting tensile strength, Jten, for the mortar. If the normal stress (σ_n) reaches the tensile strength, i.e. $\sigma_n >$ Jten, then $\sigma_n = 0$. For shear, the model uses the explicit incorporation of Coulomb's frictional behaviour at contacts between the brick units. Thus, slippage between bricks will occur when the tangential or shear force (τ_s) at a contact reaches a critical value $\tau_{\max}$ defined by:

$$|\tau_s| \geqslant \mathrm{Jcoh} + \sigma_n \cdot \tan(\mathrm{Jfric}) = \tau_{\max}. \quad (3)$$

Table 1
Range of material parameters.

Unit parameters	Symbol	Value	Units
Elastic parameters			
Density	*d*	2000	kg/m^3
Elastic modulus	*E*	6050	MPa
Poisson's ratio	*v*	0.14	–
Interface joint parameters			
Joint normal stiffness	JKn	50–90	GPa/m
Joint shear stiffness	JKs	30–85	GPa/m
Inelastic parameters			
Joint friction angle	Φ	20–40	Degrees
Joint cohesion	Jcoh	0.05–0.15	MPa
Joint tensile strength	Jten	0.05–0.15	MPa
Joint dilation angle	Ψ	0	Degrees

3. Computational model of brickwall panels with opening

The computational model developed by Sarhosis and Sheng [20] to study the mechanical behaviour of single leaf thick brickwork

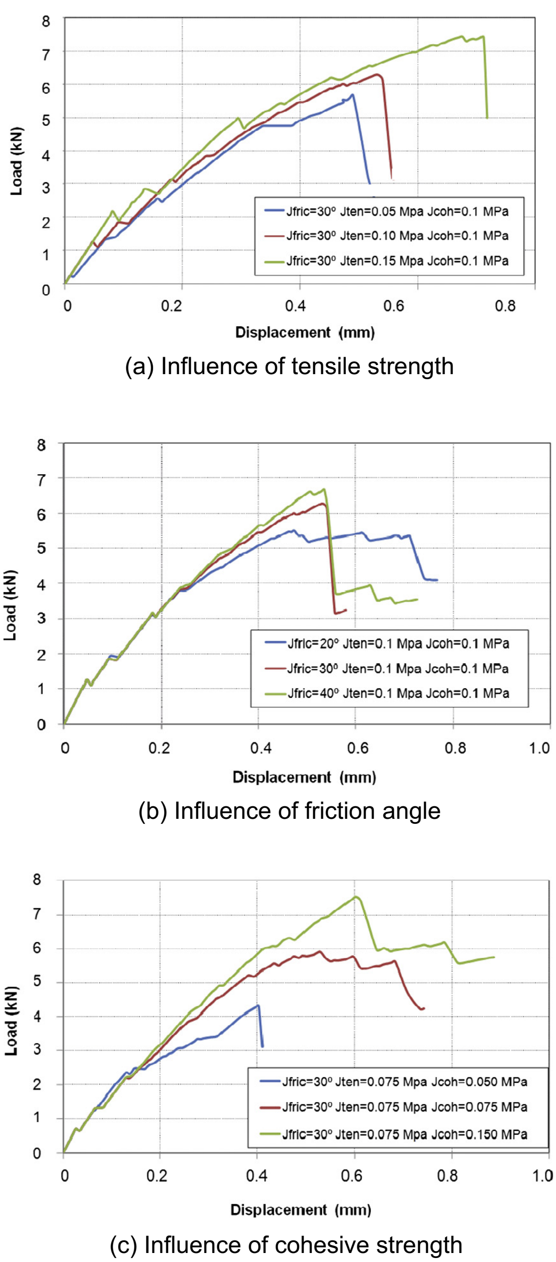


Fig. 8. Influence of inelastic interface parameters.

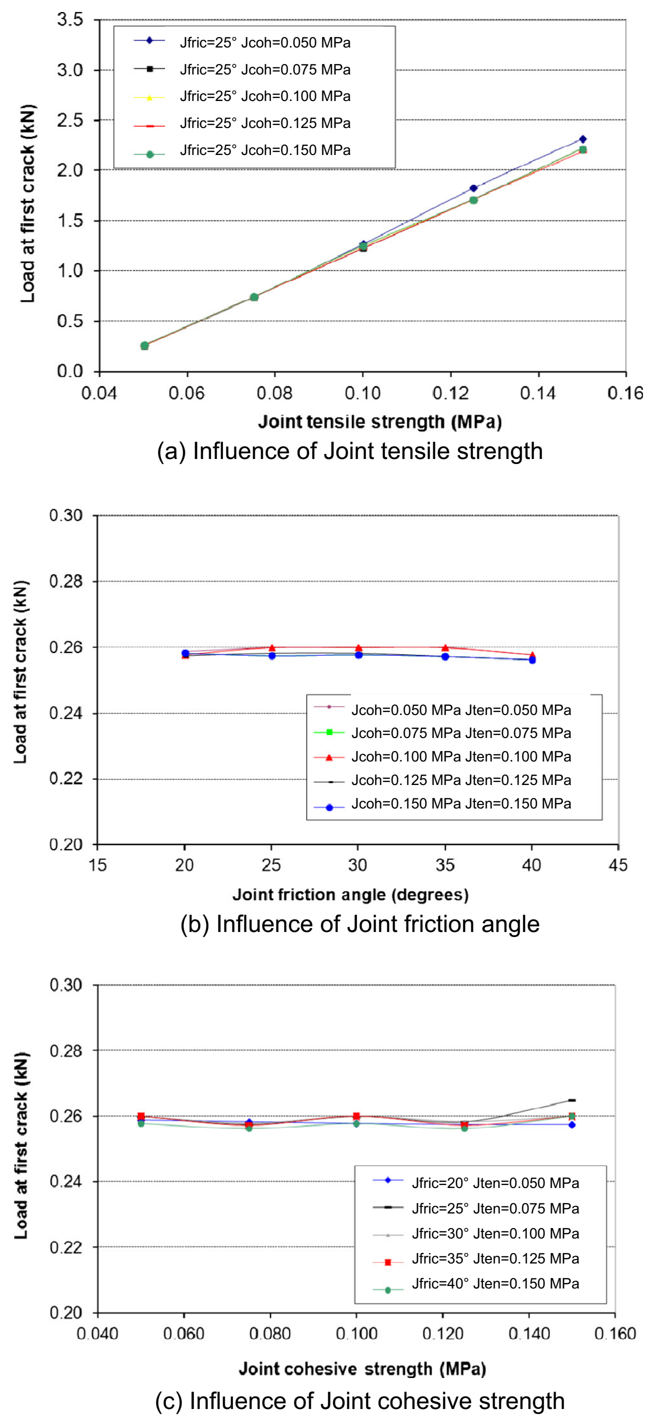


Fig. 9. Influence of the inelastic interface parameters on the load at first cracking.

wall panels with openings under vertical in plane load has been adopted for the parametric analysis in this study. The model was validated against a series of different in geometry large-scale masonry wall panels tested in the laboratory. The geometry of the masonry wall panel used in this study is shown in Fig. 4. Each brick of the panel was represented by a deformable block separated by zero thickness interfaces at each mortar joint. The zero thickness interfaces between adjacent blocks were modelled using the elastic-perfectly plastic Coulomb slip failure criterion with tension cut-off. UDEC also provides a residual strength option to simulate tension softening effects. However, this was not selected as the bond strength of the masonry used in the research was much lower than that exhibited by modern masonry materials. As a result, any tension softening effects were likely to be an order of magnitude smaller than the bond strength and so were considered to be insignificant.

Fig. 5 compares the experimental against the computational development of cracks at different stages of loading for a typical masonry wall panel with opening. Both experimental and numerical results showed that there were four notable features of the behaviour of the wall panels namely: (a) initial flexural cracking in the soffit of the panel; followed by (b) the development of flexural cracks in the bed joint of each support; with increasing load leading to (c) propagation of diagonal stepped cracks at mid depth; and (d) collapse as a result of shear failure.

Fig. 6 compares the experimental against the computational load versus mid-span displacement responses. The experimental curves stop at a point before ultimate load has been reached. Deflections at ultimate load were not taken for safety reasons and to avoid damage of the dial gauge. Only the value of the ultimate load has been recorded. From Fig. 6, the predicted ultimate load (4.6 kN) of the masonry brickwork wall panel compares quite well with that obtained experimentally (average load 4.7 kN). Also from Fig. 6, the load versus mid-span displacement curve starts at zero load with zero displacement. With the application of the external load, an initial elastic range can be observed up to point (a). At point (a), an initial flexural tensile crack occurs in the soffit of the panel, at or close to the point of maximum bending moment, accompanied in the test by a relaxation of the applied load at constant transverse deflection. As the applied load increased, a non-linear response (a–b) is produced. This is accompanied physically by the formation of two horizontal cracks at the bed joint of each support, point (b). These cracks propagate from the each end of the panel, which suggests that they occurred as a result of the in-plane rotation/overturning of the brickwork due to flexure. As the applied load is increased further, diagonal shear stepped cracks propagate, point (c), until the masonry wall panel reaches its ultimate load, point (d). Further attempts made to apply the external loading show a reduction in the load that can be sustained by the panel, points (d–e). This is accompanied by further opening of the cracks until the panel suddenly collapses, point (e). It is important to note that once first cracking has occurred, the sequence of events to collapse happens very quickly with little warning of impending collapse. Such non-linear load deflection behaviour appears to result from the ability of masonry, once a joint has cracked, to maintain a substantial proportion of its moment of resistance over a considerable change in strain because of the resistance of the mortar/unit interface where the masonry units overlap. The little peaks in the curve shown in Fig. 6 represent relaxation of the loading and moment re-distribution in the panel due to the formation of a new crack. When a crack propagates, there is an abrupt loss of stiffness in the panel.

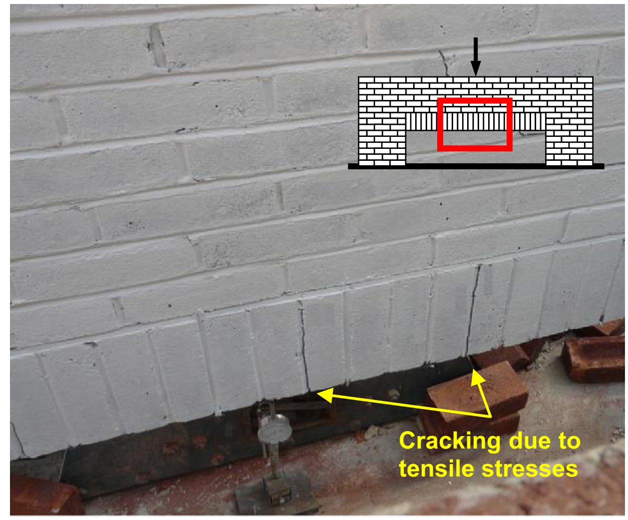


Fig. 11. Cracking at the soffit of the panel.

4. Methodology

A series of computational experiments (or parameter sensitivity studies) were developed to study the influence of the brick–mortar interface on the mechanical response of the masonry wall panels subjected to vertical in-plane loading. The computational experiments devised to assess parameter sensitivity were based on a factorial approach; that is one in which several parameters were considered together rather than separately. According to Barrentine [5], factorial designed experiments are more efficient than studying one factor at a time and can generate a distribution of possible collections of parameter values. This is because factorial designed experiments allow a study of the effect of each material

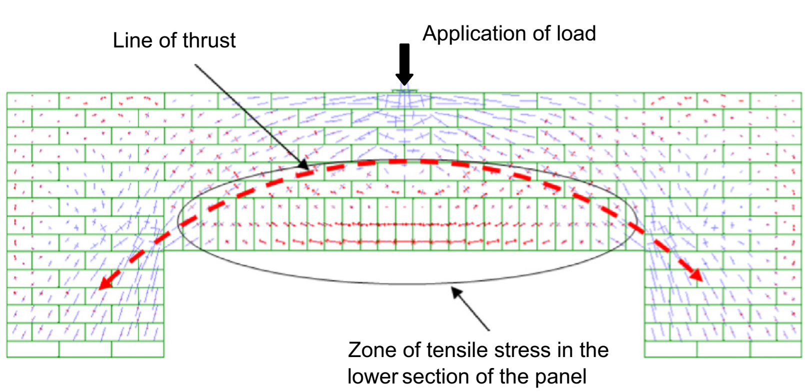


Fig. 10. Distribution of the principal stresses just before the occurrence of the first crack.

parameter on the response variable as well as the effects of interactions between the different material parameter on the response variable. Such a strategy follows the methodology developed by Abdallah et al. [1] to study the vulnerability of masonry buildings to mining subsidence and Idris et al. [11] to study the complex ageing phenomena of ancient tunnel masonry structures. The methodology used to determine the influence of the interface material properties on the mechanical response of low bond strength masonry is presented at Fig. 7.

Two parameter sensitivity studies were carried out. In each case, studies were carried out to investigate the influence of the interface properties on the mechanical behaviour of masonry wall panels with respect to:

(a) the load at first cracking; and
(b) the ultimate load.

The first parameter sensitivity study concerned the influence of the inelastic parameters of the mortar joint (i.e. Jfric, Jcoh and Jten), while the second one the elastic parameters (i.e. JKn and JKs). The range of material parameters used for the parametric study is shown in Table 1. Such range has been adopted from [15,19,24]. An analysis of variances (ANOVA) has also been carried out using the Altair Hyperstudy 10 software [4]. The aim was to study the relative importance of each joint interface parameter and their interactions to the outputs [6].

In addition, a preliminary parameter sensitivity study has been undertaken to assess the effect of varying the brick parameters. As expected, since failure of low bond strength masonry is as a result of the dominant brick–mortar interface and/or the mortar strength, the analysis showed that failure was largely independent of the brick density, elastic modulus and Poisson's ratio, provided that approximately realistic values are selected to ensure that the self-weight load effects and overall stiffness are of the correct order of magnitude.

5. Results and discussion

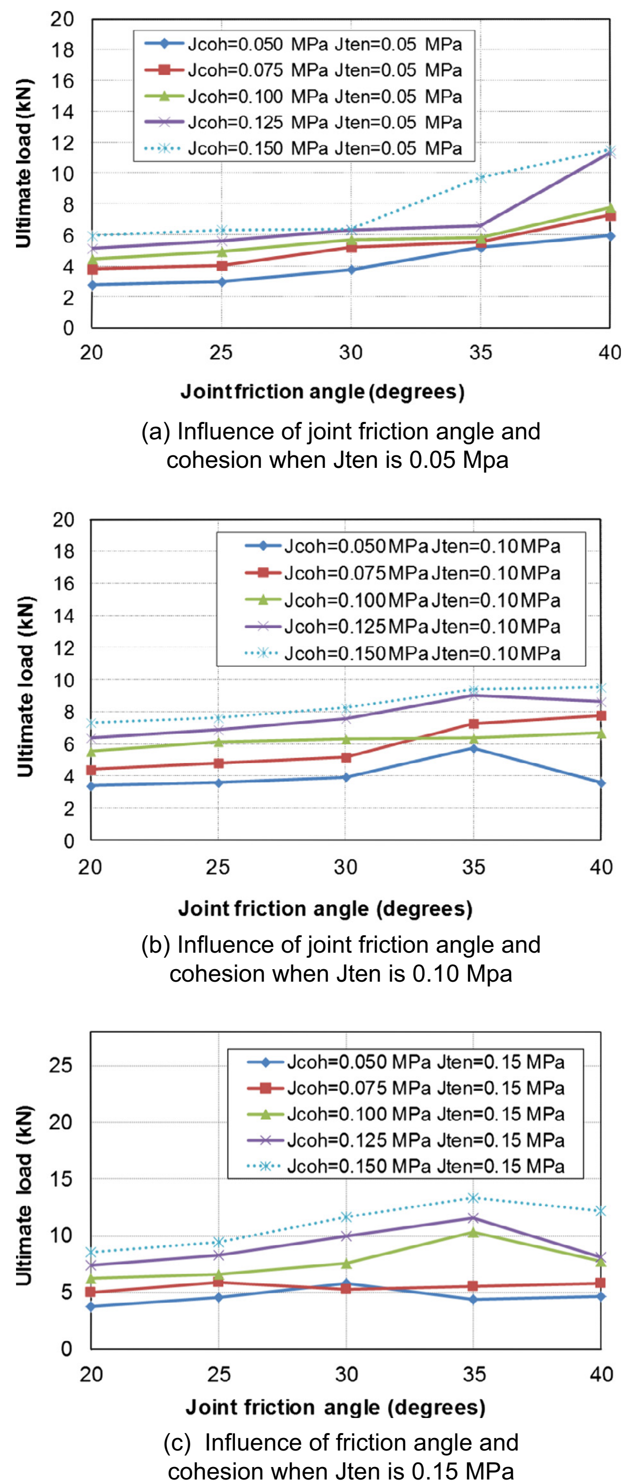
5.1. Influence of the inelastic parameters of the interface

The first parameter sensitivity study deals with the influence of the three inelastic interface parameters (Jfric, Jcoh, Jten) on the load at first cracking and the ultimate load that the panel can carry. A factorial design with five levels of magnitude for each parameter studied (i.e. 5^3) was created. This means that the three factors were considered, each one at five levels of magnitude. Consequently, a complete factorial design was used with 125 experiments. Values of Jfric were varied by 5° while that of Jcoh and Jten were varied by 0.025 MPa for the range of values shown in Table 1. Normal and shear stiffness were kept constant and equal to 82 GPa/m and 36 GPa/m, respectively [15]. Each experiment was numerically simulated and responses obtained. Fig. 8 shows the evolution of tensile, cohesive and frictional resistance of the interface with respect to the load against the mid-span displacement. The little peaks in the curves show the formation of new cracks. From Fig. 8c, there is a limiting cohesive strength for the mortar. If the cohesive strength reaches its ultimate strength, sliding occurs and the frictional resistance of the interface contributes towards the behaviour of the mortar joint.

5.1.1. Influence of the inelastic interface parameters on the occurrence of first cracking

Fig. 9 shows the influence of each of the inelastic material parameters (Jfric, Jcoh, Jten) on the load at which first cracking occurs in the masonry wall panel under investigation. From Fig. 9a, the joint tensile strength influences the initiation of first cracking in the panel. The lower the value of the joint tensile strength is, the earlier the occurrence of first cracking in the panel. This reduction is linear. Also, the frictional resistance of the joint and the cohesive strength has no influence on the occurrence of first cracking in the panel (Fig. 9b and c).

The fact that joint tensile strength is the predominant factor that influences the occurrence of first cracking in the panel, agrees with the observations made during the full scale tests in the labo-



(a) Influence of joint friction angle and cohesion when Jten is 0.05 Mpa

(b) Influence of joint friction angle and cohesion when Jten is 0.10 Mpa

(c) Influence of friction angle and cohesion when Jten is 0.15 MPa

Fig. 12. Influence of the inelastic interface parameters on the ultimate load.

ratory. The application of the external vertical load at mid span of the panel induces high compressive stresses at the corners of the opening and horizontal tensile stresses at the top of the soffit of the panel (Fig. 10) which results in the formation of vertical cracks in the soffit of the opening due to tensile failure (Fig. 11).

5.1.2. Influence of the inelastic interface parameters on the ultimate load

Fig. 12 shows the influence of the inelastic parameters of the interface on the ultimate load that the masonry wall panel can carry, as obtained from the results of the computational experiments. The influence of each inelastic material parameter on the failure load can be summarised as follows:

(a) When the value of the angle of internal friction of the interface ranges from 20° to 30°, then:
- The ultimate load that the wall panel can carry increases with an increase of frictional strength of the interface.
- The ultimate load that the wall panel can carry increases with an increase of interface cohesive strength.
- The ultimate load that the wall panel can carry increases with an increase of interface tensile strength.

(b) When the value of the angle of internal friction of the interface ranges from 30° to 40°, then:
- The tensile, friction and cohesive strength is not proportional to the ultimate load that the wall panel can carry.

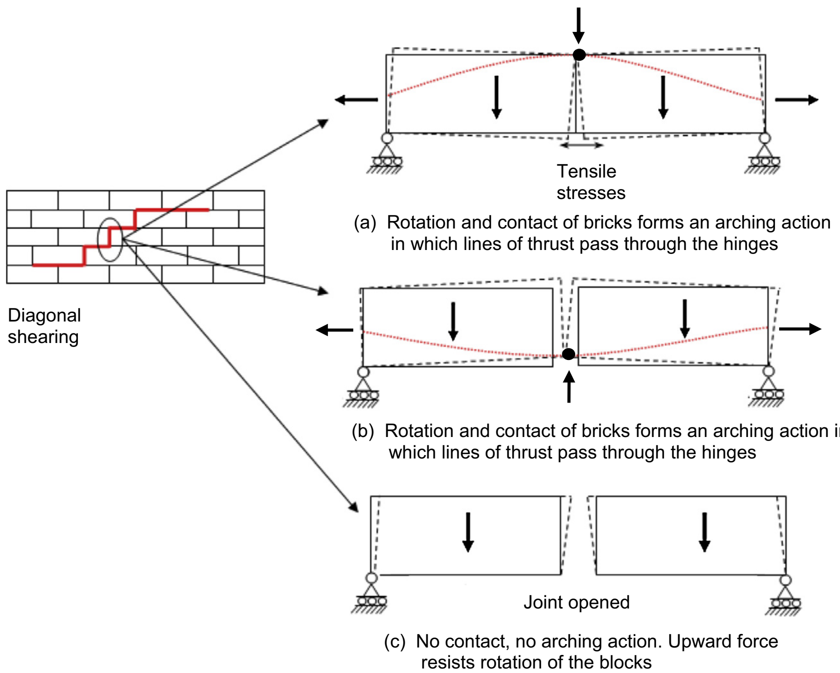


Fig. 13. Mechanical behaviour of bricks under shearing. Arrows indicate forces while the red dotted lines indicate the line of thrust. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

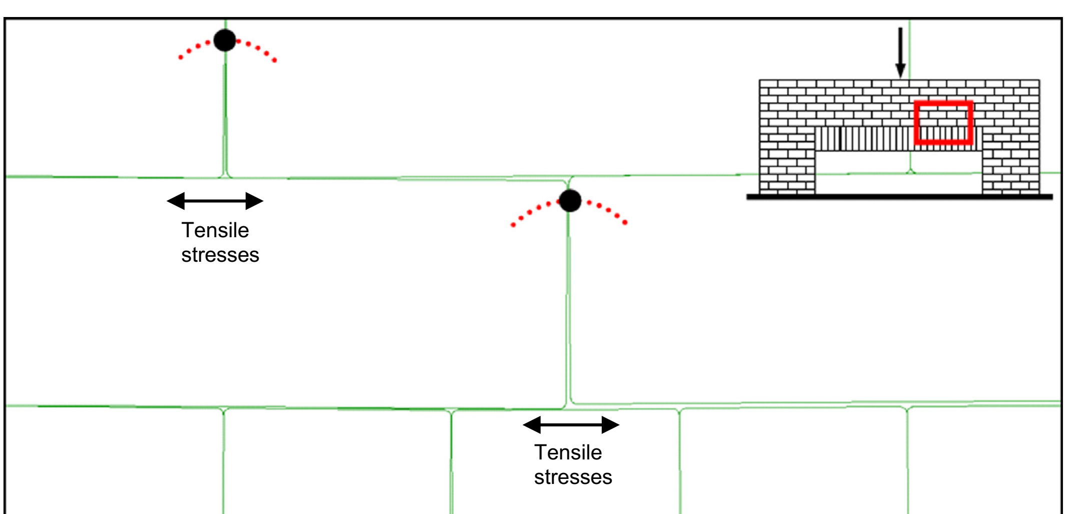


Fig. 14. Joints opened during shearing as observed from the numerical analysis. Black dots indicate hinge locations while the red dotted lines indicate the lines of thrust. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

- The influence of the friction and cohesive strength on the ultimate load that the panel can carry are significant.
- The influence of the interface friction and cohesive strength on the ultimate load are also significant but less than that of the tensile strength.

Also, from Fig. 12 can be observed that there is a highly non-linear and coupled interaction between the inelastic material parameters (joint friction, joint cohesion and joint tension) which exerts a significant influence on the load carrying capacity of the masonry wall panel. Displacements due to external load applied to the low bond strength masonry wall panel are accompanied by the opening of cracks at the brick–mortar interface and shear sliding of the bricks. As the load increases in the panel, hinges are formed (see Fig. 15) as bricks slide and rotate against each other. According to Mifsud [16], the development of hinge formation depends on the material properties of masonry, the geometric characteristics, the confinement of masonry as well as the load distribution in a structure. From the sensitivity study of the inelastic parameters of the brick–mortar interface it was observed that there is a threshold value for the joint friction angle (i.e. when Jfric = 30°) where the monotonic behaviour breaks down and affects the ultimate load carrying capacity of the panel. For values of joint friction angle ranging from 20° to 30°, the ultimate load carrying capacity of the panel increases as the joint friction angle increases. However, when the value of joint friction angle ranges from 30° to 40°, an increase in the joint friction angle will not necessarily result in an increase in the ultimate load carrying capacity of the panel. With the increase of the joint friction angle in the panel (i.e. when Jfric > 30°), deformations at the brick–mortar interfaces were not "elastic" in any sense. Large displacement discontinuity between the bricks occurred without much loss in strength of the panel. It seems that the panel sustained cracks which held in place by the confining action of surrounding bricks due to hinge development [8]. However, the aforementioned displacement discontinuity was not evident when values of joint friction angle in the panel were below 30°. Further numerical and experimental studies are required to investigate the extent of displacement discontinuity and hinge formation, although these will not form part of this study.

The hinge mechanisms observed in the low bond strength masonry wall panel under investigation are illustrated in Fig. 13. Such mechanisms have been identified both from the experimental tests carried out on full scale masonry wall panels (Fig. 15) as well as during the computational simulations (Fig. 14). The mechanism shown in Fig. 13a has been identified at the middle of the panel where diagonal shear occurred. The mechanism shown in Fig. 13b occurred both at the middle of the panel, where diagonal shear occurred and at the panel's supports as a result of a rocking motion. The third mechanism, Fig. 13c, was found in the middle of the panel after failure when detachment of the bricks has occurred and the panel has failed. The arrows at Fig. 13 represent the resultant forces due to the gravitational load, the external load applied in the masonry panel and the interaction between the neighbouring bricks.

A Least Squares Regression (LSR) analysis has been carried out and relationships between the inelastic parameters and the ultimate load that the under investigation low bond strength masonry wall panel can carry have been obtained. From the analysis of variance (ANOVA) it was found that:

(i) the contribution of the inelastic interface parameters on the ultimate load is ranked in the following order of importance: (a) Jcoh; (b) Jfric; and (c) Jten; and
(ii) Jcoh and Jten together exhibit a significant influence on the mechanical response of the wall panel close to failure. The aforementioned has been evident in the experiments where diagonal cracks propagated from the middle of the panel towards the top before failure occurred.

5.2. Influence of the elastic interface parameters

The second parameter sensitivity study deals with the influence of the elastic interface parameters (i.e. JKn, JKs) on the load at first cracking and the ultimate load that the masonry wall panel can carry. A series of 72 computational experiments were undertaken. Values of normal and shear stiffness were obtained from Table 1. In each of the computational experiments, the value of normal stiffness was kept constant while the value of shear stiffness was varied by increments of 5 GPa/m. Representative responses of the load

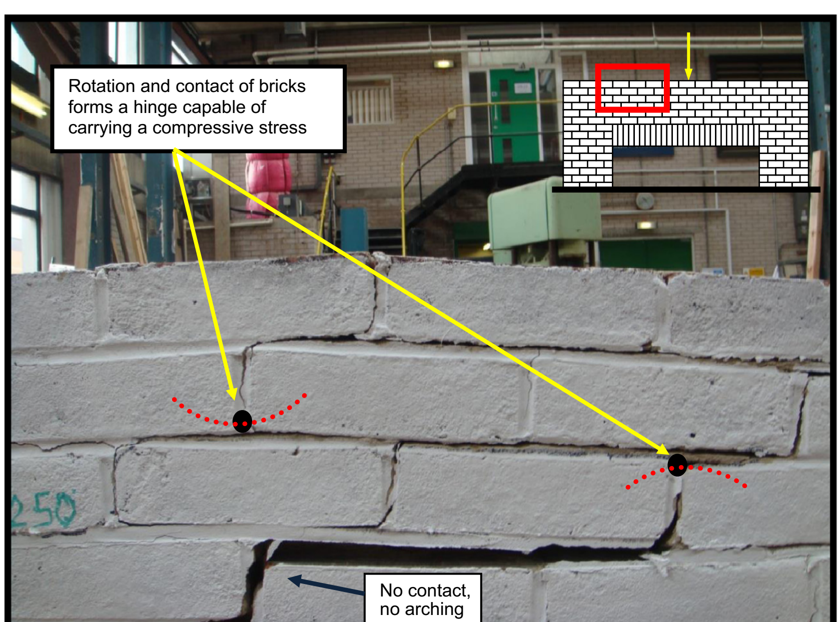


Fig. 15. Mechanical behaviour of bricks under shearing. Black dots indicate hinge locations and the red dotted lines indicate the lines of thrust. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

against mid-span displacement curves obtained are shown in Fig. 16. For all cases, the inelastic interface parameters kept constant and equal to: Jfric = 33°, Jcoh = 0.5 MPa and Jten = 0.1 MPa. The angle of dilation assumed zero [15].

5.2.1. Influence of the elastic interface parameters on the occurrence of first cracking

Fig. 17 shows the relation between the elastic interface parameters and the load at which first cracking occurs in the panel. From Fig. 17, when shear stiffness is kept constant, an increase in the normal stiffness results to a lower load at first cracking. Also, an analysis of variance (ANOVA) has been carried out and found that the contribution of the interface elastic parameters on the load at first cracking is ranked in the following order of importance: (a) JKn and (b) JKs.

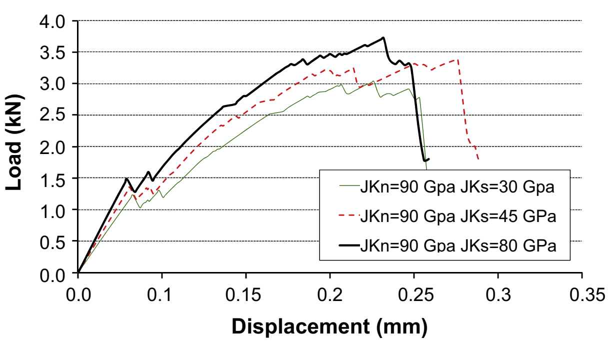


Fig. 16. Influence of shear stiffness of the interface.

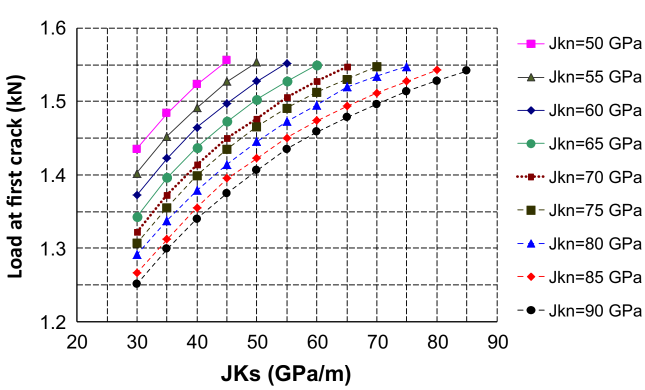


Fig. 17. Influence of normal and shear stiffness on load at first crack.

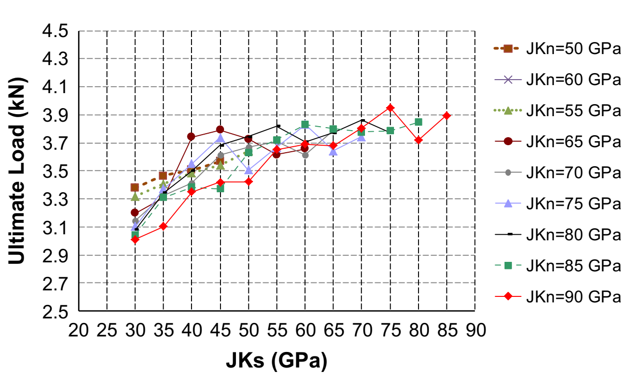


Fig. 18. Influence of normal and shear stiffness on the ultimate load capacity.

5.2.2. Influence of the elastic interface parameters on the ultimate load

Fig. 18 shows the influence of the interface elastic parameters (JKn and JKs) on the ultimate load carrying capacity of the masonry wall panel. From Fig. 18, the trend of the curves shows that an increase in shear stiffness will result in an increase in the load carrying capacity, irrespective to the normal stiffness. Also, for the range of values of the joint stiffness considered in this study (i.e. from 50 to 90 GPa), the load variation was found to vary by approximately 1kN.

6. Conclusions

Masonry is a heterogeneous, anisotropic composite material whose mechanical response characterised by high non-linearity. Mortar joints act as a plane of weakness in masonry and their mechanical properties influence the global behaviour of the masonry structure. This paper deals with the influence of the brick–mortar interface on the pre- and post-cracking behaviour of low bond strength masonry wall panels containing openings and subjected to vertical in plane loading. Experimental evidence has shown that cracking in low bond strength masonry occur along the brick–mortar interfaces and failure usually results from debonding of the bricks. The computational model based on the Distinct Element Method and developed by Sarhosis and Sheng [20] to study the mechanical behaviour of low bond strength masonry wall panels has been used in this study. Bricks were represented as an assemblage of distinct blocks while the mortar joints were modelled as zero thickness interfaces which can open and close depending on the magnitude and direction of the stresses applied to them. Both experimental and numerical results showed that there were four notable features of the behaviour of the panel namely: (a) initial flexural cracking in the soffit of the panel; followed by (b) the development of flexural cracks in the bed joint of each support; with increasing load leading to (c) propagation of diagonal stepped cracks at mid depth of the panel; leading to (d) collapse as a result of shear failure. A parametric study supported with regression data analysis was also carried out to evaluate the influence of the elastic properties (joint elastic stiffnesses) and inelastic properties (joint tensile strength, joint cohesive strength and frictional resistance of the brick–mortar interface) on the load at first cracking and the ultimate capacity of the wall panel. Results of the parametric analysis have compared against full scale experimental tests carried out in the laboratory. From the parametric study it was found that:

a. Joint tensile strength is the predominant factor that influences the occurrence of first cracking in the panel. An increase in the joint tensile strength will result in a higher load at which first crack occurs in the panel.
b. Joint normal and shear stiffness also influence the occurrence of first cracking. However, their contribution is lower compared to that of the joint tensile strength. An increase of normal stiffness under constant shear stiffness will contribute to a lower load to cause first cracking.
c. For the ultimate load that the masonry wall panel can carry, the contribution of the inelastic joint interface parameters are ranked in order of importance as: (i) joint cohesion; (ii) joint friction angle; and (iii) joint tension.
d. The influence of the cohesive strength and friction angle of the interface together exhibits a significant interaction capable of influencing the mechanical response of the wall panel at the near-failure condition.
e. With the application of the external load, hinges are formed as bricks slide and rotate against each other. Hinge development influence the ductility and the ultimate load carrying capacity of the low bond strength masonry wall panel.

In the current parametric study, the assessment of load at first crack and ultimate load carrying capacity is based on a specific in geometry and material properties low bond strength masonry wall panel. In addition, the masonry wall panel studied has been subjected only to vertical in-plane load. Therefore, conclusions obtained are not applicable to any wall panel containing an opening. Extrapolating the results to different geometries and different load configurations would require additional numerical and experimental investigations. The next phase of the research will focus on the experimental behaviour of different in geometry plain and reinforced masonry wall panels subjected to various types of loading.

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References

[1] Abdallah M, Verdel T, Deck O. Vulnerability of masonry buildings to mining subsidence studied through experimental design. In: Symp post-mining, Nancy, France; 2008.
[2] Abdou L, Ami SR, Meftah F, Mebarki A. Experimental investigations of the joint-mortar behaviour. Mech Res Commun 2006;33(3):370–84.
[3] Adami CE, Vintzileou E. Investigations of the bond mechanism between stones or bricks and grouts. Mater Struct 2008;41(2):255–67.
[4] Altair Engineering. Altair Hyperstudy 10 – user's manual, design of experiments, optimization and stochastic studies. Alter Engineering Inc.; 2010.
[5] Barrentine BL. An introduction to design of experiments: a simplified approach experiments with three factors. USA: The American Society for Quality (ASQ); 2003.
[6] Christensen R. Analysis of variance design and regression applied statistical methods: applied statistical methods. Florida: Chapman and Hall; 1996.
[7] Cundall PA. A computer model for simulating progressive large scale movements in blocky rock systems. In: Proc symp inter soc rock mech, vol. 2; 1971. p. 8.
[8] Elshafie H, Hamid A, Nasr E. Strength and stiffness of masonry shear walls with openings. Masonry Soc J 2002.
[9] Garrity SW, Ashour AF, Chen Y. An experimental investigation of retro-reinforced clay brick arches. Int Masonry Soc 2010;2010(1):733–42.
[10] Hendry AW. Structural masonry. 2nd ed. London: Palgrave Macmillan; 1998.
[11] Idris J, Al-Heib M, Verdel T. Numerical modelling of masonry joints degradation in built tunnels. Tunn Undergr Space Technol 2009;24(6):617–26.
[12] ITASCA. UDEC – universal distinct element code manual: theory and background. Minneapolis: Itasca Consulting Group; 2011.
[13] Lemos JV. Discrete element modelling of masonry structures. Int J Archit Herit 2007;1(2):190–213.
[14] Lourenço PB, Rots JG. A multi-surface interface model for the analysis of masonry structures. J Eng Mech 1997;123(7):660–8.
[15] Lourenço PB. Computational strategies for masonry structures. PhD thesis. Delft, Netherlands: Delft University of Technology; 1996.
[16] Mifsud J. Load paths in masonry construction. PhD thesis. University of Malta; 2003.
[17] Otter JRH, Cassell AC, Hobbs RE. Dynamic relaxation. Inst Civ Eng 1996;35(4):633–56.
[18] Roca P, Cervera M, Gariup G, Pela L. Structural analysis of masonry historical constructions. Classical and advanced approaches. Arch Comput Methods Eng 2010;17(3):299–325.
[19] Rots JG. Structural masonry: an experimental/numerical basis for practical design rules. Netherlands: Balkema; 1997.
[20] Sarhosis V, Sheng Y. Identification of material parameters for low bond strength masonry. J Struct Eng 2014;60(1):100–10.
[21] Sarhosis V. Computational modelling of low bond strength masonry. PhD thesis. Leeds: University of Leeds; 2012.
[22] Sarhosis V, Oliveira DV, Lemos JV, Lourenco PB. The effect of skew angle on the mechanical behaviour of masonry arches. Mech Res Commun 2014;61(1):53–9.
[23] Schlegel R, Rautenstrauch K. Failure analyses of masonry shear walls. In: Konietzky H, editor. Numerical modelling of discrete materials in geotechnical engineering, civil engineering and earth sciences. London, UK: Taylor and Francis Group; 2004. p. 15–20.
[24] van Der Pluijim R. Non-linear behaviour of masonry under tension. HERON 1997;42(1):25–54.
[25] Zhuge Y. Micro-modelling of masonry shear panels with distinct element approach. In: Chowdhury L, Fragomeni M, editors. Advances in mechanics of structures and materials. Swets & Zeitinger: Lisse; 2002. p. 131–6.