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Identification of material parameters for modeling of masonry structures

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Abstract

This paper deals with the problem of identifying material parameters for modelling the mechanical behaviour of masonry. The conventional method of obtaining material parameters, for masonry constitutive models, from the results of small sample tests was thought to be problematic and may not produce material parameters that are representative for masonry structures under complex loading conditions and stress states. This paper proposes a novel approach by which the material parameters were obtained from an optimization process and validated by the large scale tests carried out in the laboratory on single leaf wall panels. The wall panel tests were modelled using a Discrete Element Code UDEC. An optimization procedure was then used to tune the masonry material parameters in order to better simulate the pre- and post-cracking behaviour and the behaviour close to collapse as observed in the laboratory tests. The obtained material parameters were then validated by comparing the UDEC-predicted behaviour of a new set of wall panel, against the results obtained from the laboratory test. In spite the great variability of masonry properties, good correlation was obtained between the results from the computational model and those obtained from tests in the laboratory. The developed method provides an effective way to reduce uncertainties associated with the parameter identification of constitutive models for masonry, and can be further applied to other numerical methods and the studies of many other type of masonry structures including the reinforcement.

***Keywords**:* Parameter identification, brickwork masonry, Discrete element modeling, optimization.

1.0 Introduction

Many existing masonry structures are of complex construction and are subjected to complex loadings. Engineers often need the use of relatively sophisticated numerical or analytical methods in order to obtain a realistic assessment of the in-service behaviour or strength of such structures. However, any numerical or analytical model of analysis requires some forms of constitutive model to simulate the mechanical response of structures under various loading conditions. Constitutive models require a number of input material parameters to be identified in order to characterize the behaviour of the masonry. In the last few years, with the development of the sophisticated numerical models, the number of parameters for material models has increased significantly (Roca, 2010). Conventionally, material parameters for masonry constitutive models are determined directly from the results of tests on small masonry prisms or material samples. However, it is often the case that representative values of material parameters cannot be obtained accurately from the small scale tests due to the intrinsic variety of the masonry materials. In other cases, material parameters are difficult to be measured directly from physical tests. This paper investigates the parameter identification problem for modelling masonry structures and proposes an alternative identification procedure in which the unknown parameters are optimized against a series of full scale laboratory tests on wall panels. To demonstrate the effectiveness of the method, the material parameters for a constitutive model used in numerical modeling to represent the mechanical behaviour of low bond strength masonry construction are identified. Computational models have been developed using Discrete Element Method (DEM) to recreate full scale experiments. The load to cause first visible cracking, the ultimate load and the load versus mid-span displacement relationship obtained from the laboratory tests are compared with the computational predictions obtained from DEM modelling. An optimization method is adopted so that the material parameters used in DEM models can be tuned to minimize the difference between responses measured from the large scale tests and those obtained by the computational simulation. The optimised material parameters are then used in a new DEM model to predict the behaviour of a different lab test of masonry wall panel to validate the developed process for the identification of the masonry material parameters..

2.0 Discrete element modelling of masonry

Experimental evidence (Abdou et al., 2006; Adami et al., 2008; Garrity et al., 2010) has shown that at low values of normal stress, the principal failure mode of masonry with low strength mortar is either in the brick/mortar interface or in the mortar itself. This result in joint opening due to tensile cracking or sliding along a bed or head joint with friction. The Discrete Element Method (DEM) was originally developed by Cundall for rock engineering. More recently, it has been used to model masonry structures (Lemos, 2007; Schlegel et al., 2004; Toth, 2009; Zhuge, 2002) due to its intrinsic advantages over other numerical methods on modelling the interfaces and discontinuous media subjected to either static or dynamic loading. When used to model brickwork structures, the bricks can be represented in a DEM model 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 any 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. These zones are continuum elements as they occur in the finite element method (FEM). Mortar joints are represented as zero thickness interfaces between the blocks. The interactions between the blocks at interfaces are governed by interfacial constitutive laws, for example, the mechanical interaction between the blocks could be simulated at the contacts by spring like joints with normal (J_{kn}) and shear stiffness (J_{ks}) as well as frictional (J_{fric}), cohesive (J_{coh}) and tensile strengths (J_{ten}), see Figure 1.

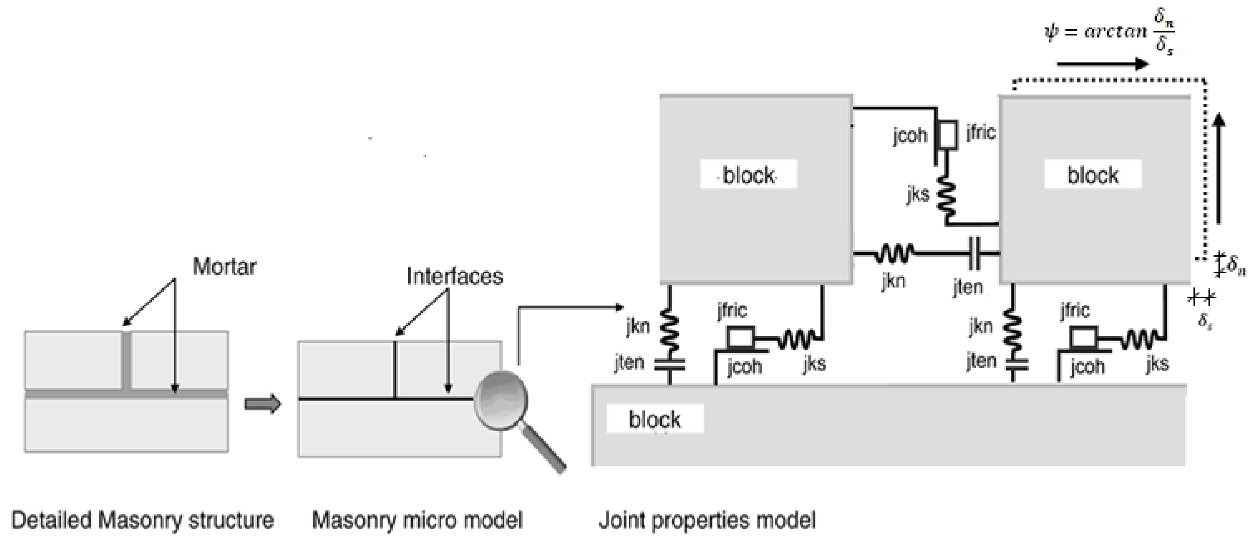


Fig. 1. Interface model in UDEC (after Idris et al., 2009).

As with the Finite Element Method, the unknown variables in DEM equations are the nodal displacements and rotations of the blocks. However, unlike FEM, in DEM the unknowns are acquired by solving explicitly the differential equations of Newton's Second law of motion at all blocks 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. In this way, large displacements along the mortar joints and the rotations of the bricks are allowed with the sequential contact detection and update (ITASCA, 2004). In this study, a commercial DEM code Universal Distinct Element Code (UDEC) developed by ITASCA is adopted to model the masonry wall panels.

3.0 Methods for masonry material parameter identification

Conventionally, material parameters for masonry constitutive models are determined directly from the results of compressive, tensile and shear strength tests on small masonry prisms (Lourenco, 1996) as shown in Figure 2. These usually consist of a small number of bricks and mortar joints, usually assumed that the stress and strain fields in the specimen are uniform.

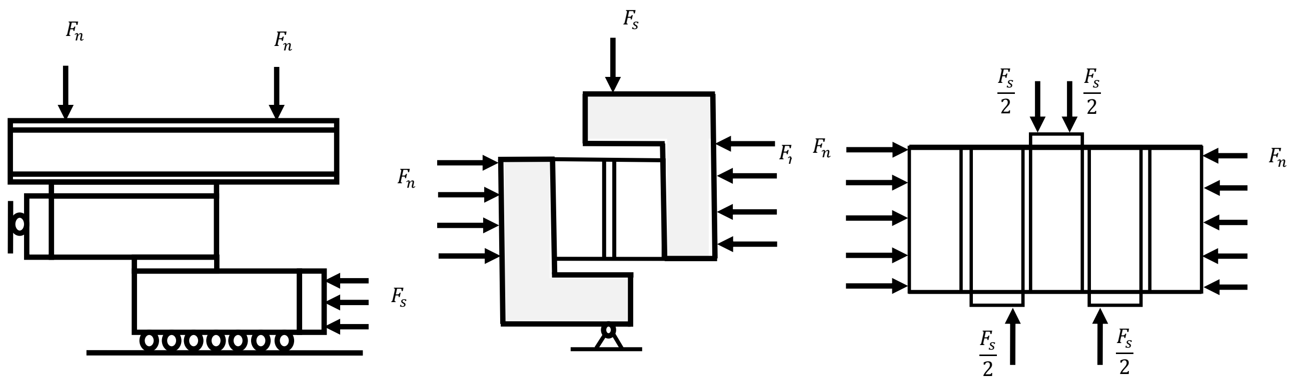
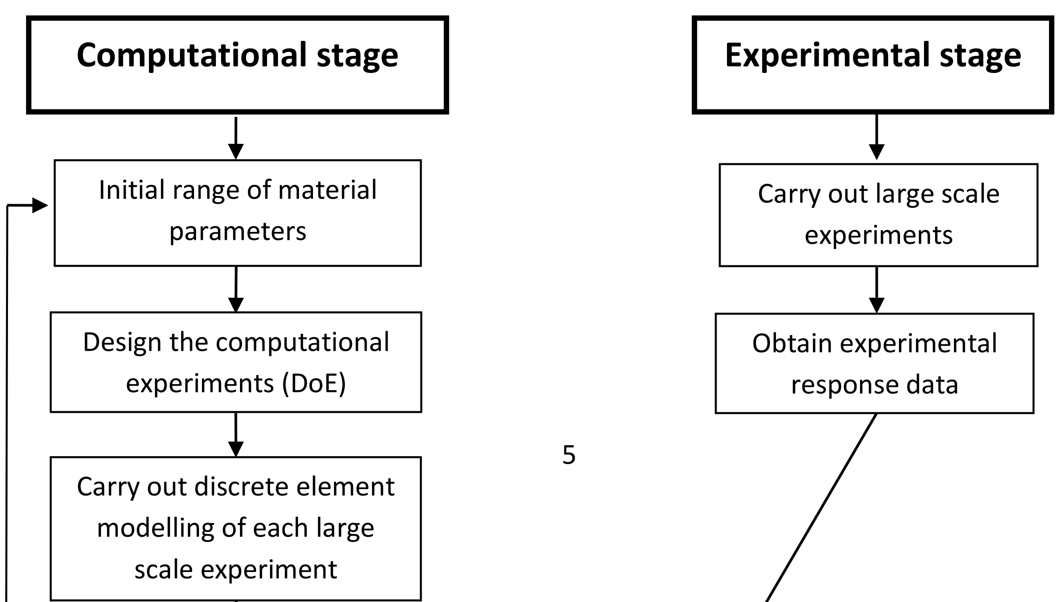


Fig. 2. Different types of shear tests: a) couplet test; b) Van der Pluijim test [1993] and c) triplet test (Lourenco, 1996).

In some other cases, separate tests are carried out on material samples, such as masonry units and/or mortar specimens (Rots, 1997; Van der Pluijm, 1999). The testing of small specimens is simple, relatively inexpensive and involves little specialist equipment. However, the conventional approach is considered to be problematic and may not produce material parameters that are representative for masonry under complex loading and stress conditions. This is a particular problem for DEM modelling of masonry, which uses parameters that represent the behaviour of the interfaces between the masonry units and the mortar joints, as brick and mortar properties are highly variable and depend primarily on the local supply of raw materials and manufacturing methods (Hendry, 1998). Also, the assumption that the stress and strain in the specimen are uniform is not applicable for masonry which is an intrinsically inhomogeneous material. Moreover, the simple conditions under which the small specimens are tested in the laboratory do not usually reflect the more complex boundary conditions, the combinations of stress-state types and load spreading effects that exist in a large scale masonry structure. In addition, some of the parameters obtained from small scale tests are variable and sensitive to the method of testing. This is likely to be due to the combined effects of eccentric loading, stress concentrations and variations in the resistance to applied stress that are likely to exist in the test specimens (Hendry, 1998). According to Vermeltfoort (1997), the effects of boundary conditions such as platen restraint and the shape and size of the test specimen can have a significant influence on the magnitude of the measured parameter. Also, the restraint conditions on the mortar in the cube test will be different to those existing in the mortar joint between masonry units. The situation is made more complex when workmanship is considered. The variations in workmanship will not be captured if the material parameters are based on the results from the testing of the limit number of small scale specimens. As a result of these difficulties it is often necessary to adjust the material parameter values obtained from small scale experiments before they can be used in the numerical model. The authors have found a further complication when using the Distinct Element Method (DEM) to model masonry. As the material parameters define the characteristics of the zero thickness interfaces between the mortar joints and the blocks, they can be difficult to measure directly from physical tests. The UDEC user manual (ITASCA, 2004a) states that "*it is important to recognize that joint properties measured in the lab typically are not representative of those for real joints in the field*". It further states that "*scale dependence of joint properties is a major question and the only way to guide the choice of appropriate parameters is by comparison to similar joint properties derived from field tests*". The use of field test results presents another set of difficulties. The stress and strain levels that are found in structures in the field are likely to be very low and affected by effects such as moisture movements, shrinkage and creep. Any material parameters determined from field measurements are unlikely to represent the behaviour of masonry in the post-cracking and near-collapse conditions. Other factors such as load spreading effects, residual thermal stresses in bricks and large inclusions sometimes found in bricks are all contribute to the uncertainty of material parameters obtained from small scale experiments. Therefore, it is essential to develop a more reliable method to identify the parameters for modelling the masonry.

4.0 Proposed methodology for material parameter identification

An alternative method of identifying material parameter that better reflects the complex nature of masonry is adopted based on the optimization of the responses of larger scale masonry structures (Toropov and Garrity, 1998). The identification procedure of the material parameters is based not on the conventional try-and-error approach but on an optimization procedure. According to the proposed method, numerical analysis for each large scale experiment is carried out and values of material parameters are tuned so that the difference between responses measured from a series of large scale laboratory experiments and those obtained from the numerical simulation can be minimized. Initially, a range of material parameters based on results of conventional small-scale experiments or on the codes of practice or on engineering judgment are used in the model for the simulation of the large experiments. . These material parameters can then be modified and fine tuned through an optimization process in which the function to be minimized is an error function that expresses the difference between the responses measured from the large scale experiments and those obtained from the numerical analysis. Such responses can include: failure load, load at initial visual cracks, load-deflection characteristics etc.. An optimization software, Altair HyperStudy 10 (Altair, 2010) is used for the implementation of the optimization process. The proposed method for material parameter identification is illustrated diagrammatically in Figure 3 and described as following.



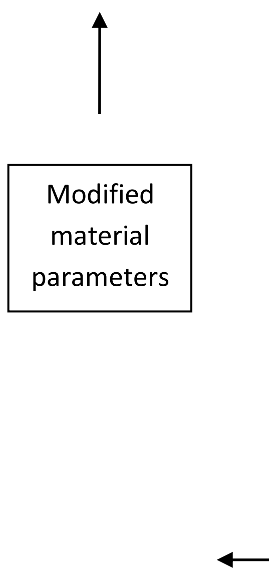


Fig. 3. Schematic chart for the proposed material parameters identification method.

The aim of the identification problem is to obtain the optimum estimate of the unknown model parameters taking into account uncertainties which may exist in the problem, such as the inherent variation of material properties, experimental errors and errors in the model estimation method. The estimates of the material parameters obtained from this approach could be referred to as the *maximum likelihood estimates* and can be used to *"inform"* the computational model. Examples validating the use of the proposed material parameter identification technique for large deformation plasticity models include: a) test data of a solid bar in torsion (Toropov et al., 1993) and b) test data for the cyclic bending of thin sheets (Yoshida et al., 1998). Later, Morbiducci (2003) applied the method to two different masonry problems in order to: a) identify the parameters of a non-linear interface model (Gambarotta et al., 1997a) to describe the shear behaviour of masonry joints under monotonic loading, where shear tests were chosen as the experimental tests; and b) to evaluate the parameters of a continuum model for brick masonry walls under cyclic loading (Gambarotta et al., 1997b). Based on the above mentioned studies, the following points need to be considered in the optimization process:

a) When modelling masonry, different material parameters influence different stages of mechanical behaviour;
b) large number of full scale experiments may be required; and
c) a significant amount of computational time is required to carry out parameter sensitivity studies.

4.1 Formulation of the alternative material parameter identification problem as an optimization problem

Consider an experimental test performed on $\mathcal{M} = 1,2,\ldots,m$ specimens in order to estimate the design variables or unknown parameters $\mathcal{P} = 1,2,..,\mathrm{p}$ of the constitutive model. Let $\mathcal{N} = 1,2,\ldots n$ be the number of responses of interest recorded from the experimental data. Also, consider $\mathrm{R}_n^{\mathrm{exp}}$ as the value of the nth measured response quantity corresponding to the large scale experiment carried out in the laboratory. Consider $\mathrm{R}_n^{\mathrm{comp}}$ as the value of the nth measured response quantity corresponding to the computational simulation. Then, the responses $\mathcal{R}$ are functions of the design variables of the model $\mathcal{P}$. The model takes the general function form $x = \mathcal{R}(\mathcal{P})$. To calculate values of this specific function for the specific set of parameters, x, requires the use of a non-linear numerical simulation (e.g. discrete/finite element) of the experimental test under consideration. The difference between the experimental and the numerical response is an error function that can be expressed by the difference D=$\mathcal{R}_{\mathrm{M,N}}^{\mathrm{exp}} - \mathcal{R}_{\mathrm{M,N}}^{\mathrm{comp}}$. The optimization problem can then be formulated as follows:

$$\mathrm{F}_{(x)}^{1} = \sum\left[\left(\mathcal{R}_{1,1}^{\mathrm{exp}} - \mathcal{R}_{1,1}^{\mathrm{comp}}\right)^2 + \left(\mathcal{R}_{1,2}^{\mathrm{exp}} - \mathcal{R}_{1,2}^{\mathrm{comp}}\right)^2 \ldots\ldots. + \left(\mathcal{R}_{1,\mathrm{n}}^{\mathrm{exp}} - \mathcal{R}_{1,\mathrm{n}}^{\mathrm{comp}}\right)^2\right] \quad (1)$$

$$\mathrm{F}_{(x)}^{2} = \sum\left[\left(\mathcal{R}_{2,1}^{\mathrm{exp}} - \mathcal{R}_{2,1}^{\mathrm{comp}}\right)^2 + \left(\mathcal{R}_{2,2}^{\mathrm{exp}} - \mathcal{R}_{2,2}^{\mathrm{comp}}\right)^2 \ldots\ldots. + \left(\mathcal{R}_{2,\mathrm{n}}^{\mathrm{exp}} - \mathcal{R}_{2,\mathrm{n}}^{\mathrm{comp}}\right)^2\right] \quad (2)$$

$$\vdots$$

$$\mathrm{F}_{(x)}^{m} = \sum\left[\left(\mathcal{R}_{m,1}^{\mathrm{exp}} - \mathcal{R}_{m,1}^{\mathrm{comp}}\right)^2 + \left(\mathcal{R}_{m,2}^{\mathrm{exp}} - \mathcal{R}_{m,2}^{\mathrm{comp}}\right)^2 \ldots\ldots. + \left(\mathcal{R}_{m,\mathrm{n}}^{\mathrm{exp}} - \mathcal{R}_{m,\mathrm{n}}^{\mathrm{comp}}\right)^2\right] \quad (3)$$

$F^M(\mathbf{x}) = \mathrm{F}_{(x)}^{1} + \mathrm{F}_{(x)}^{2} + \cdots + \mathrm{F}_{(x)}^{m}$ is a dimensionless function. The problem is then to find the vector $\boldsymbol{x} = [x_1, x_2, x_3 \ldots x_p]$ that minimizes the objective function:

$$\mathrm{F}_{(x)}^{\mathrm{total}} = \sum \theta^{\mathcal{M}}\left(F^M(\mathbf{x})\right), \qquad A_i \le X_i \le B_i \qquad (i = 1 \ldots\ldots.. N) \quad (4)$$

where $\mathrm{F}_{(x)}^{\mathrm{total}}$ is a function of the unknown parameters $(x_1, x_2, x_3 \ldots x_p)$, $\theta^{\mathcal{M}}$ is the weight coefficient which determines the relative contribution of information yielded by the M-th set of experimental data, and A_i , B_i are the lower and upper limits on the values of material parameters. The optimization problem equation (4) has the following characteristic features:

- The objective function is an implicit function of parameters x, wher $x \in \mathbb{R}$;
- to calculate the values of this function for the specific set of parameters x requires the use of a non-linear numerical (e.g. discrete element) simulation of the process under consideration, which is usually involves a considerable amount of computational time;
- function values may present some level of numerical noise.

The computational simulations of masonry wall panels with UDEC would require a large amount of computational time. Also, convergence of the above method cannot be guaranteed due to the presence of noise in the objective function values. Thus, routine task analysis such as design optimization, design space exploration and sensitivity analysis becomes impossible since a large amount of simulation evaluations is required. One way to mitigate against such a problem is by constructing surrogate models (*also referred to as response surface models or metamodels*) (Queipo et al., 2005). These models mimic the behaviour of the model as closely as possible while at the same time they are time effective to evaluate. Surrogate models are constructed based on modelling the response predicted from the computational model to a limited number of intelligently chosen data points. New combinations of parameter settings, not used in the original design, can be plugged into the approximate model to quickly estimate the response of that model without actually running it through the entire analysis. Different methods of regression analysis (i.e. Least Squares Regression and Moving Least Squares) can be used to construct the expression for the function $\mathrm{F}_{(\boldsymbol{x})}^{\mathrm{total}}$. This approach can result in less computational iterations and lead to substantial saving of computational resources and time. Using this approach, the initial optimization problem, eq. (4), is replaced with the succession of simpler mathematical programming sub-problems as: finding the vector $\boldsymbol{x}_k^*$ that minimizes the objective function:

$$\tilde{F}_k(x) = \sum \theta^{\mathcal{M}} \tilde{F}_k^M(x), \quad A_i^k \leq X_i \leq B_i^k,\ A_i^k \geq A_i, B_i^k \leq B_i \quad (i = 1 \ldots \ldots .. N) \tag{5}$$

where k is the iteration number. The limits A_i^k and B_i^k define a sub-region of the optimization parameter space where the simplified functions $\tilde{F}_k^M(x)$ are considered as current approximations of the original implicit functions $F^M(\mathrm{x})$. To estimate their accuracy, the error parameter $r_k = \left|[F(x_k^*) - \tilde{F}_k(x_k^*)]/F(x_k^*)\right|$ is evaluated. The value of the error parameter gives a measure of discrepancy between the values of the initial functions and the simplified ones. Any conventional optimization technique (Toropov and Yoshida, 2005) can be used to solve a sub-problem in equation (5), because the functions involved in its formulation are simple and noiseless.

5.0 Material parameter identification for low bond strength masonry

To demonstrate the general procedure of the proposed method of material parameter identification for masonry modeling, a simple but typical masonry structure was used for this purpose. A series of single leaf low bond strength masonry wall panels with opening were used in this study. For validation of the developed procedure, the parameters from the optimization method will then be used to predict the behavior of a wall panel in different geometry, but constructed from similar brick and mortar combination. The stages of the material parameter identification method are described in further details below.

5.1 Experimental stage

Seven single leaf brickwork masonry wall panels were tested in the laboratory. The wall panels were developed to represent the clay brickwork outer leaf of an external cavity wall containing openings for windows. Panels were built with a soldier course immediately above the opening with the remainder of the brickwork being constructed in stretcher bond. Four of the wall panels (S1 to S4), as listed in Table 1, had an opening of 2.025m and six courses of stretcher bonded brickwork above the opening; two of the wall panels (L1 & L2) had an opening of 2.925m and 6 courses of stretcher bonded brickwork above the opening and the final wall panel had an opening of 2.025m and nine courses of stretcher bonded brickwork above the opening (DS1). Typical details of the panels are shown in Figure 4. All panels have been constructed with UK standard size 215mm x 102.5mm x 65mm Ibstock Artbury Red Multi Stock bricks with water absorption of 14% and a sand faced finish. The joints were all 10mm thick with 1:12 (opc:sand) weigh-batched mortar. The bricks and mortar were selected deliberately to produce brickwork with low bond strength. The aim is to represent low quality, high volume wall construction which, in the authors' experience, is fairly typical of low rise domestic construction in the UK. Each wall panel was tested by applying a central point load to the top of the wall at midspan. The load was applied to each panel using a hydraulic ram and was distributed through a thick steel spreader plate which was embedded in mortar on the top of the brickwork. A structural steel frame bolted to the laboratory floor provided the support. The load was applied to each wall incrementally until the panel could no longer carry the applied load. At each load increment, vertical deflections were measured at midspan using a dial gauge supported on a magnetic stand and a steel base plate. The painted surfaces of the panels were inspected visually for signs of cracking at each load increment. Typically the first visible cracks were observed in the order of 0.2mm wide. In addition, cracking in the each panel under test was identified from the dial gauges readings. For example, sudden increases in the deflection measurements during testing indicated crack formation and other effects such as stress redistribution following cracking and very short-term creep effects. Deflections at ultimate load were not taken for safety reasons and to avoid damage to the dial gauge. The experimental test results are summarised in Table 1.

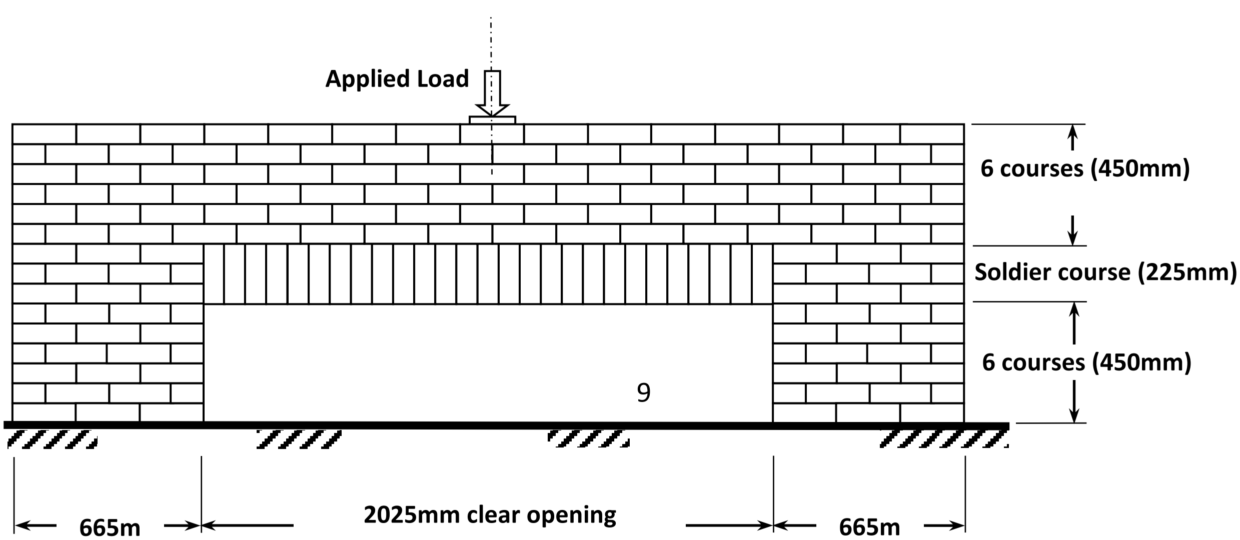


Fig. 4. Typical details of masonry wall panel tested in the laboratory.

Table 1. Masonry wall panel experimental test results.

Panel	Clear opening (mm)	Courses of stretcher bond above opening	Type of masonry	Mortar compressive strength (MPa)	Load at first visible crack (KN)	Failure load (KN)
S1	2025	Six	Unreinforced	0.72	0.72	3.69
S2	2025	Six	Unreinforced	0.79	1.6	4.6
S3	2025	Six	Unreinforced	0.86	1.6	5.1
S4	2025	Six	Unreinforced	1.18	1.71	5.67
L1	2925	Six	Unreinforced	0.64	0.1	1.6
L2	2925	Six	Unreinforced	0.71	0.6	2.6
DS1	2925	Nine	Unreinforced	0.96	0.72	10.6

Notations: (S) refers to short span panels, (L) refers to long span panels and (D) refers to deep panels.

5.2 Computational model for masonry wall panels

The brickwork panels tested in the laboratory were recreated in UDEC models. Bricks were represented by a deformable block separated by interfaces at each mortar bed and perpend joint. To represent for the 10mm thick mortar joints in the real wall panels, each deformable block was based on the nominal brick size increased by 5mm in each face direction to give a UDEC block size of 225mm x 102.5mm x 75mm. The expanded dimensions of the bricks have no significant influence on the accuracy of the model's mechanical behaviour predictions (Sarhosis, 2011). The UDEC model for a masonry wall panel with a 2.025m long opening is shown in Figure 5.

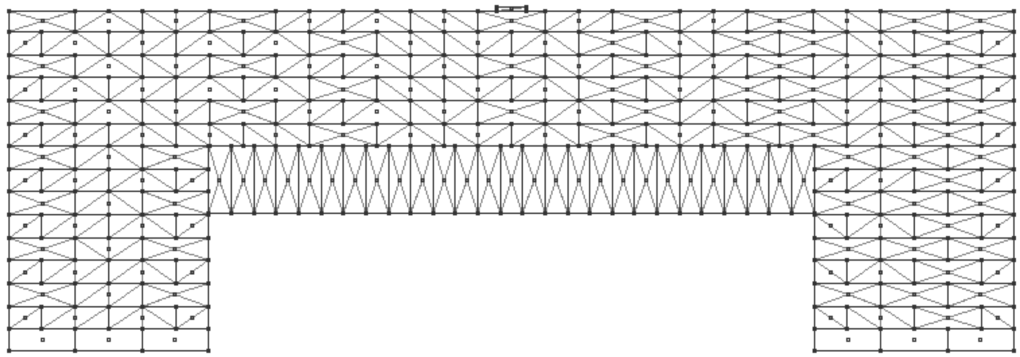


Fig. 5. Typical UDEC geometric model of a masonry wall panel with a 2.025m opening.

It was assumed that each brick in UDEC model would behave as a homogeneous, isotropic continuum which will exhibit linear stress-strain behaviour as they would be loaded well below their strength limit. Material properties have been selected such that the bricks would remain intact at all stages of applied loading and the predominant failure mode would be slips and openings along the brick/mortar interfaces. Similar failure modes have also been observed in the full scale lab experiments. The block parameters required by UDEC to represent the behaviour of the bricks are listed in Table 3. The mortar joints were represented by interfaces modelled using UDEC's *elastic-perfectly plastic coulomb slip-joint area contact* option (ITASCA 2004). This option is intended for closely packed blocks of any shape with area contact, such as masonry, and provides a linear representation of the mortar joint stiffness and yield limit. It is based on six input material parameters namely: the elastic normal (JK_n) and shear (JK_s) stiffnesses, frictional (J_{fric}), cohesive (J_{coh}) and tensile (J_{ten}) strengths, as well as the dilation (J_{dil}) characteristics of the mortar joints. According to the model, If, in the numerical calculation, the bond tensile strength or shear strength is exceeded, then the tensile strength and cohesion are reduced to zero in accordance with the Mohr-Coulomb relationship for low bond strength masonry, it has been assumed that tension softening will be insignificant. Finally, it was assumed that the mortar properties would be the same for the vertical and horizontal joints.

The bottom edges of the UDEC wall panels were modelled as rigid supports in the vertical and horizontal direction whilst the vertical edges of the wall panel were left free. Self weight effects have also been included in the model as a gravitational load. Local damping was assigned to the model to simulate quasi-static loading.

In order to determine the collapse load, displacement-controlled boundary conditions was used in the UDEC modelling (UTASCA, 2004). As a result, a constant vertical velocity was applied at the load spreader plate on the top of the wall panel. The velocity was converted to a vertical displacement and the force acting on the spreader plate for each load increment. Hence, a load versus mid-span displacement relationship could be determined for the panel. The modelling results are compared in the optimization process to the experimental results obtained in the laboratory.

5.3 Identification of masonry material parameters for UDEC models

5.3.1 Optimization aims

The material parameters used in UDEC were optimized to produce similar responses to the following aspects of behaviour observed from the large-scale tests in the laboratory:

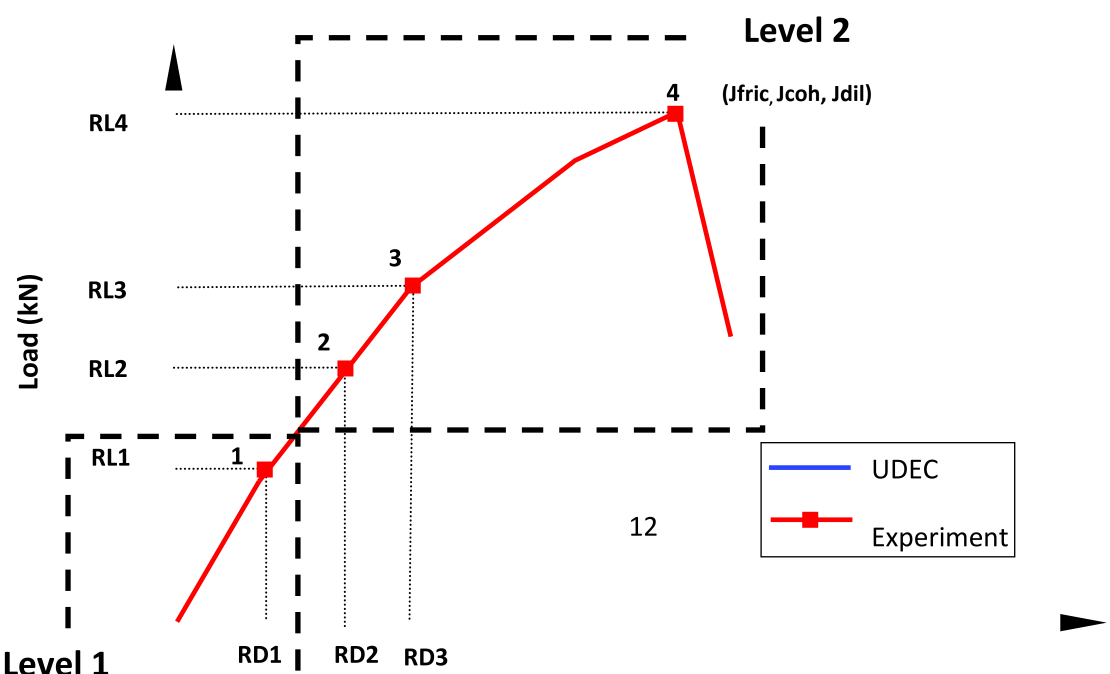
a). The applied load and deflection of the panel at the occurrence of first cracking (point 1 in Figure 6);

b). The maximum load supported by the wall/beam panels (point 4 in Figure 6);

c). The intermedian load-displacement relationships (points 2 and 3 in Figure 6).

d). The propagation of cracks in the wall/beam panels with increasing applied load;

e). The mode of failure.



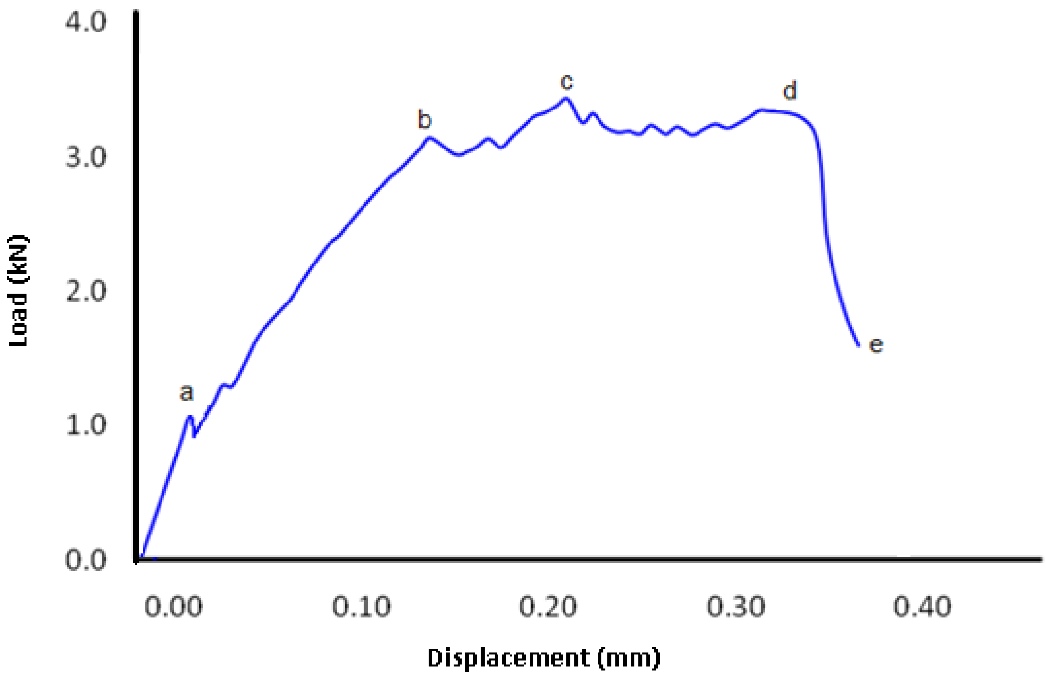


Fig. 6. Response evaluation points for Level 1 and Level 2 optimization (R, L and D denotes response, load and displacement, respectively).

5.3.2 Sensitivity analysis

Initially a sensitivity analysis was carried out to identify the effect of the brick and brick/mortar joint interface parameters used in UDEC on the pre- and post-cracking behaviour (Sarhosis et al., 2011). As expected, behaviour was found to be independent of the brick properties and was dominated by the brick/mortar joint interface properties. From the analysis it was found that:

a). The load versus displacement relationship for the masonry wall/beam panels was linear up to the occurrence of first cracking;

b). Of the six parameters used by UDEC to define the characteristics of the brick/mortar joint interface:

  i). The normal stiffness(JKn); the shear stiffness (JKs) and the tensile strength (Jten) of the interface, have dominate influences the behaviour of panels up to and including the occurrence of the first crack;

  ii). The cohesive strength (Jcoh); the angle of friction (Jfric) and angle of dilation (Jdil) influence more on the behaviour of the panels after first cracking up to collapse.

To reflect results from the sensitivity study and to minimise the computational time, the optimization of the material parameters was carried out in two different Levels as indicated in Figure 6.

- ***First Optimization (Level 1):*** Optimization of the joint interface material parameters JK_n, JK_s and J_{ten} up to the occurrence of the first crack; and
- ***Second Optimization(Level 2):*** Optimization of the joint interface material parameters J_{fric}, J_{coh} and J_{dil} just after the occurrence of first crack and up to the ultimate load in the panel.

The two levels of optimization are described in more detailsbelow.

5.3.3 Level 1 Optimization

For the Level 1 optimization, the material parameters to be optimized up to the occurrence of first crack in the panel are: JK_n, JK_s, and J_{ten}. After Lourenco (1996), the normal and shear stiffness of mortar joint can be estimated according to the brick and mortar properties as follows:

$$JK_n = \frac{E_b E_m}{h_m(E_b - E_m)} \tag{6}$$

$$JK_s = \frac{G_b G_m}{h_m(G_b - G_m)} \tag{7}$$

$$G = \frac{E}{2(1+v)} \tag{8}$$

where E_b and E_m are the Young's moduli and G_b and G_m are the shear moduli, respectively, for the blocks and mortar and h_m is the actual thickness of the mortar joint. These equations give the ration of normal to shear stiffness as:

$$JK_n/JK_s = 2\left[1 + \frac{E_b v_m - E_m v_b}{E_b - E_m}\right] \tag{9}$$

Table 2 demonstrates a range of material properties determined by using the data from the literature (Hendry, 1998; Rots, 1991 & 1997; Sarangapani et al., 2005 and Van der Pluijm, 1999) for brick and mortar combinations similar to these used for the construction of the large scale experiments described in Section 5.1. These properties have been obtained from the testing of small samples of material or small assemblages.

Table 2. Variation of brick and mortar properties as identified from the literature.

Interface parameter	Young's modulus of brick (N/m^2)	Young's modulus of mortar (N/m^2)	Poisson's ratio of brick	Poisson's ratio of mortar	Height of mortar joint (m)
Symbol	E_b	E_m	v_b	v_m	h_m
Range	(4 to 10) x10^9	(1 to 11) x10^8	0.1 to 0.2	0.1 to 0.2	0.01

Using the range of material parameters from Table 2, a variation analysis of the minimum and maximum values of the normal and shear stiffness has been carried out. The analysis showed that:

- Normal stiffness ranged from $10\ to\ 150\ GPa/m$;
- Shear stiffness ranged from $4.3\ to\ 65\ GPa/m$; and
- The ratio of normal to shear stiffness ranged from 2.18 to 2.5.

From the above findings it is reasonable to assume that the ratio of normal to shear stiffness of masonry with brick and mortar properties varying according to the values given in Table 8.1 can be taken as the average of 2.18 and 2.52, namely a value of 2.3. The influnce of taking the average value of the normal to shear stiffness ratio has been investigated and found to be negligible (Sarhosis, 2011). Also, since the normal stiffness is directly related to the shear stiffness, only the joint normal stiffness and the joint tensile strength were consideres as independent parameters and included in the optimization process. A factorial design of 28 experiments has been proposed for each of the short (S1 to S4) and long panels (L1 and L2) referred in Table 1. The material parameters used in the computational experiments for the Level 1 optimization are shown in Table 3. Such ranges have also been adopted in the literature (Hendry, 1998; Rots, 1991 & 1997; Van der Pluijm, 1999). While the joint normal stiffness, joint shear stiffness and joint tensile strength values were varied, the rest of the input parameters were assumed to be constant and equal to the values reported by Lourenço (1996). As has been identified from the sensitivity analysis, such values do not have significant influence to the behaviour of the panel up to the occurrence of first crack.

Table 3. Range of brick and mortar joint properties used in UDEC models.

	Brick Properties	Symbol	Value	Units
Elastic parameters	Density	d	2000	Kg/m^3
	Elastic modulus	E	6050	MPa
	Poisson's ratio	ν	0.14	-
	Mortar Joint Properties			
	Joint normal stiffness	JK_n	10, 25, 50,..., 150	GPa/m
	Joint shear stiffness	JK_s	$JK_n/2.3$	GPa/m
Inelastic parameters	Joint friction angle	Φ	36.8	Degrees
	Joint cohesion	J_{coh}	0.375	MPa
	Joint tensile strength	J_{ten}	0.09, 0.1, 0.11, 0.12	MPa
	Joint dilation angle	Ψ	0	Degrees

The least squares differences between the experimental and computational test results , up to the occurrence of first cracking, for each of the short and long panels were then estimated. All the response quantities were considered to be equally weighted for the formulation of the objective function. A surrogate model was constructed with the use of the Moving Least Squares (MLS) approximation method. The predicted response surface created using Altair HyperStudy 10 (Altair, 2010) is shown in Figure 7.

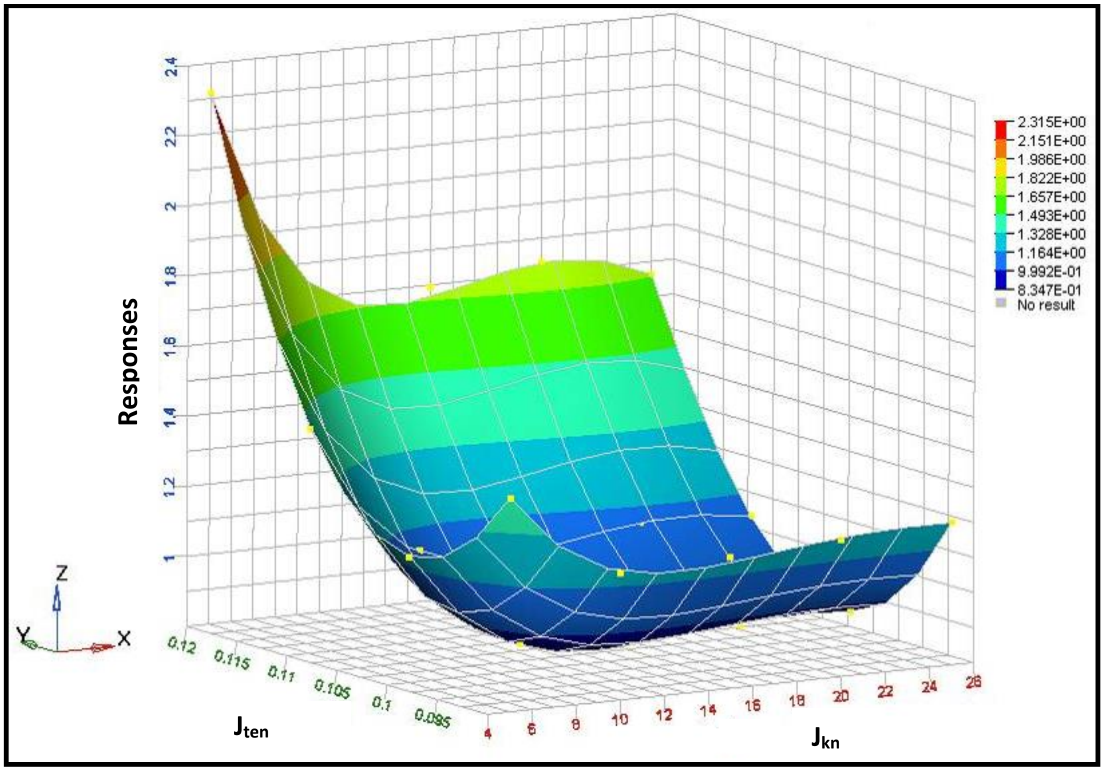


Fig. 7. Response surface for relating the objective function with the normal stiffness and the tensile strength of mortar joint interface as derived from the MLS approximation.

An optimization study has also been carried out using Altair HyperStudy 10. An evolutionary algorithm method i.e. the Genetic Algorithm (GA) in HyperStudy 10 has been adopted. According to Toropov and Yoshida (2005), GA is more likely to find a non-local solution (i.e. the global minimum) when compared to other gradient based methods such as Sequential Quadratic Method or the Adaptive Response Surface method. Further details of the optimization process are described elswhere (Sarhosis 2011). From the Level 1 optimization exercise it was found that: **JKn = 13.5GPa/m; JKs = 5.87GPa/m and Jten = 0.101MPa.** These values and the brick properties shown in Table 3 were then used in the Level 2 optimization.

5.3.4 Level 2 Optimization

For the Level 2 optimization, a factorial design of 175 experiments was carried out for each of the short and long panels used in the level 1 optimization. The values of the interface parameters used when planning the computational experiments are shown in Table 4 (Rots, 1998; Pluijm, 1997). The procedure adopted was similar as per Level 1 Optimization. Response surfaces were created as shown in Figure 8 and an optimization study was carried out using a genetic algorithm in Hyperstudy 10. From the optimization analysis, the results conmverge to: **Jfric = 40^0; Jcoh = 0.062MPa and Jdil = 40^0.**

Table 4. Range of brick and interface material properties.

	Brick Properties	Symbol	Value	Units
Elastic parameters	Density	d	2000	kg/m^3
	Elastic modulus	E	6050	MPa
	Poisson's ratio	v	0.14	-
	Mortar Joint Properties			
	Joint normal stiffness	JK_n	13.5	GPa/m
	Joint shear stiffness	JK_s	5.87	GPa/m
Inelastic parameters	Joint friction angle	Φ	20 to 40	Degrees
	Joint cohesion	J_{coh}	0.04 to 0.016	MPa
	Joint tensile strength	J_{ten}	0.101	MPa
	Joint dilation angle	Ψ	0 to 40	Degrees

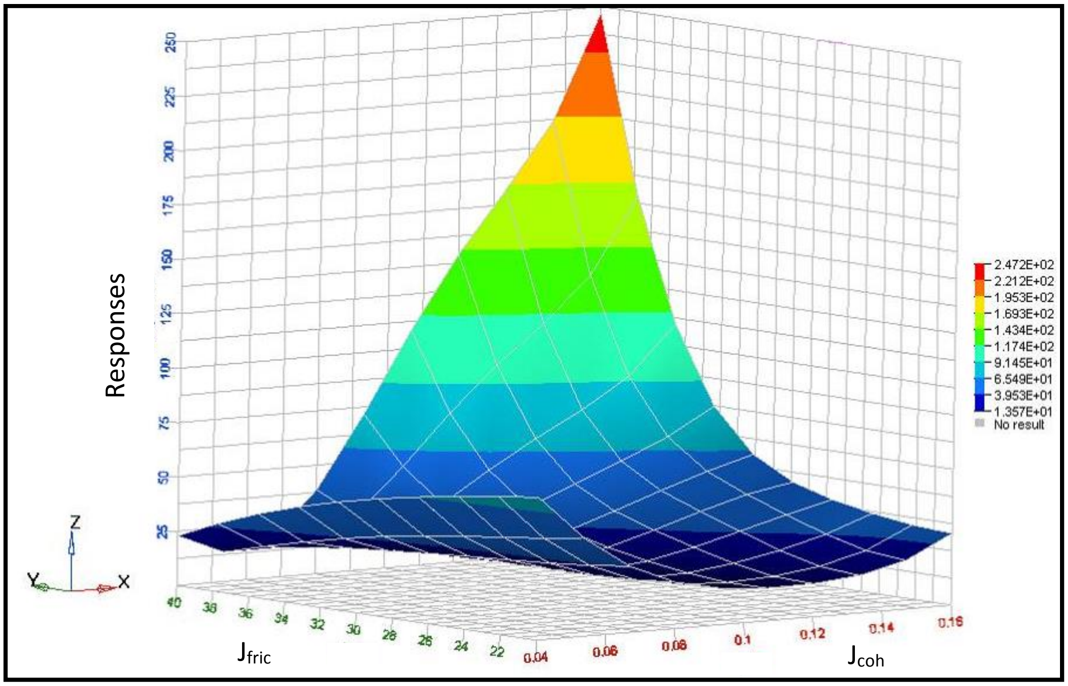


Fig. 8. Response surface for relating the objective function with the cohesive and frictional characteristics of the mortar joint interface (the dilation angle is 35 degrees).

5.3.5 Verification study

In order to verify the effectiveness of the optimization process, the experimental results for the short and long panels were compared with the results obtained from the UDEC modelling results using the optimized material parameters, as can be seen in Figures 9 and 10. Bearing in mind the inherent variations that occur in masonry, a good level of correlation between modelling and test results was achieved for the load at first crack and the ultimate load that the panel can carry.

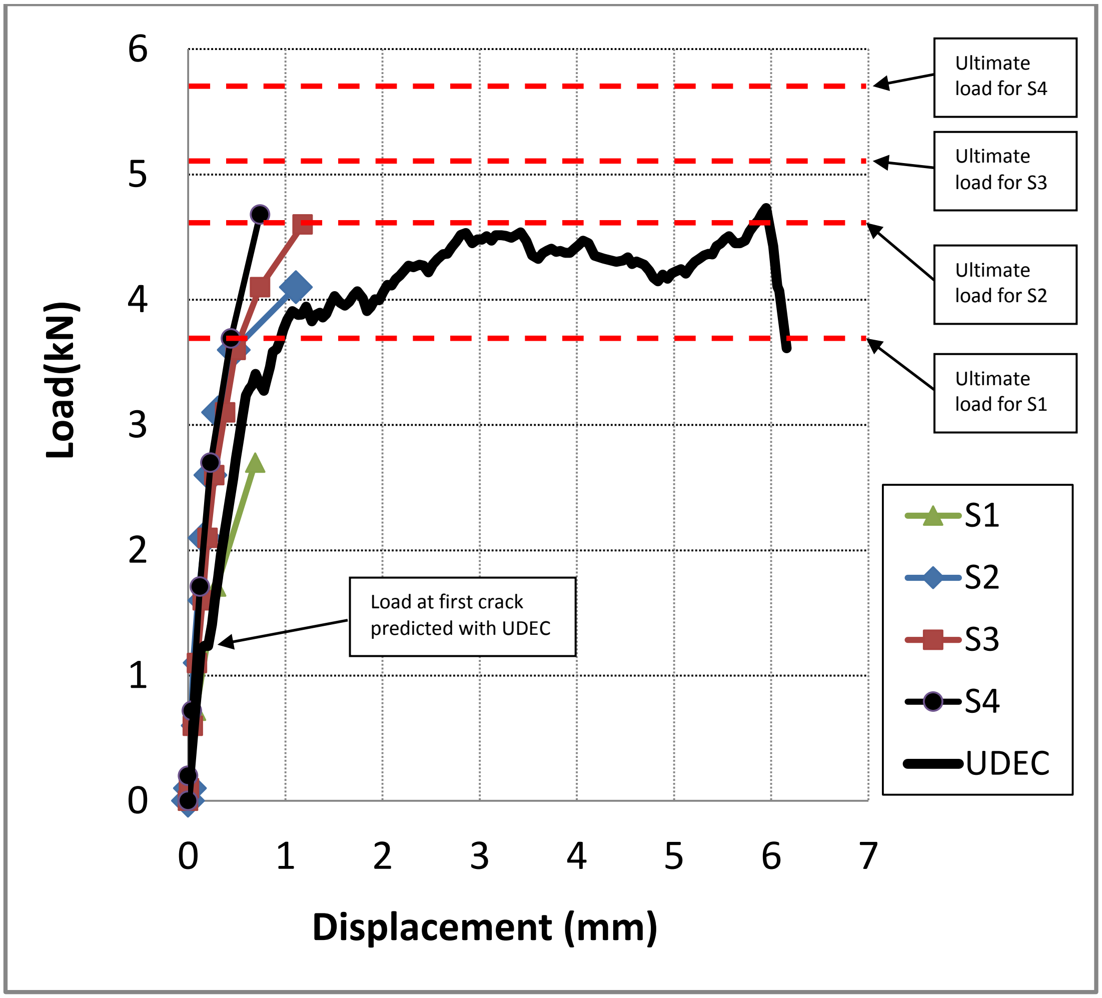


Fig. 9. Comparison of experimental and computational results for the short panels

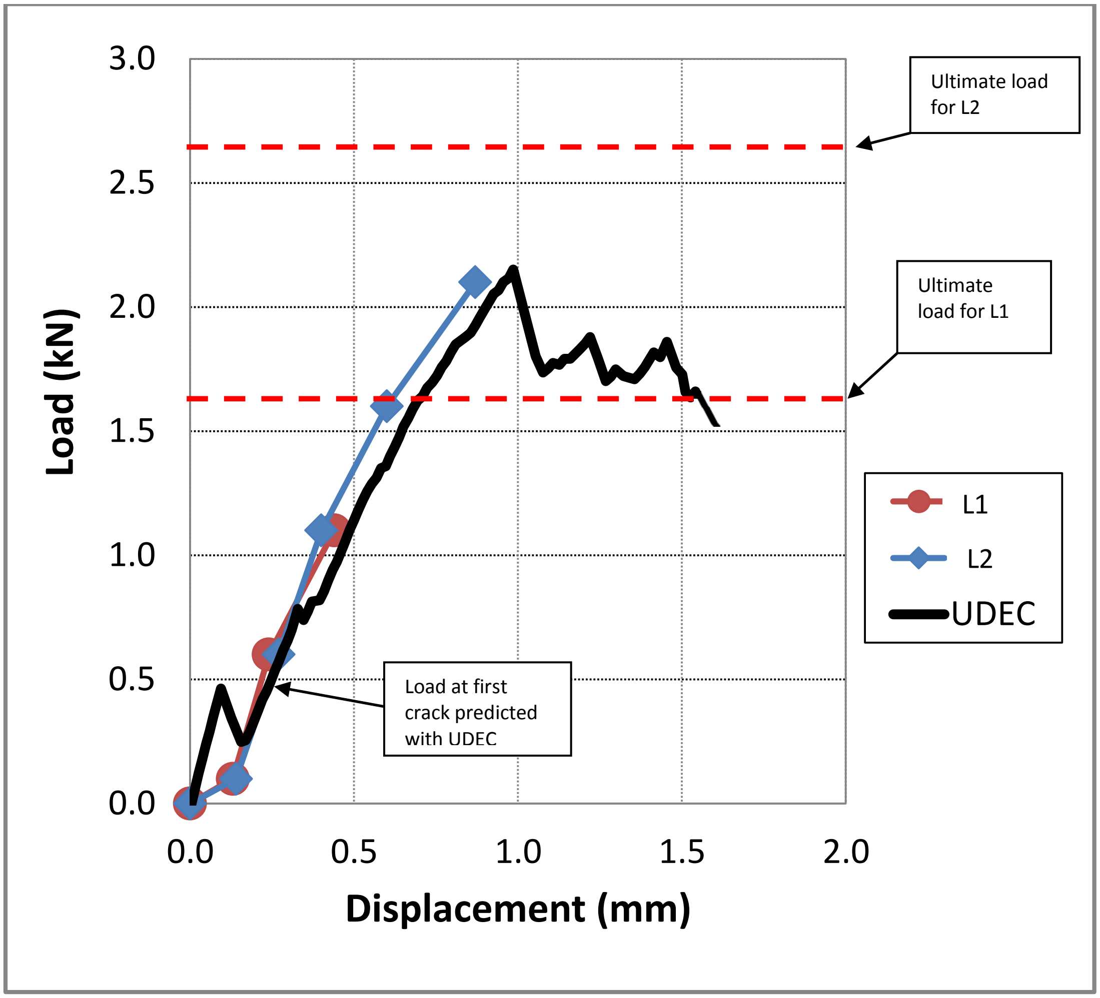


Fig. 10. Comparison of experimental and computational results for the long panels

6.0 Validation study

To validate the material parameter identification procedure, the UDEC model with the optimized parameters was used to predict the behaviour of a new set of deeper wall/beam panel (DS1). Both experimental and computational results contain four notable aspects of behaviour 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 both up (towards the loading point) and down (towards the corner of the opening); and d) collapse as a result of what is usually referred as a shear failure (or excessive diagonal tension). Figure 11 show the distribution of cracks at collapse predicted by UDEC. A similar pattern of cracks was observed in the laboratory testing, as shown in Figures 12, 13 and 14.

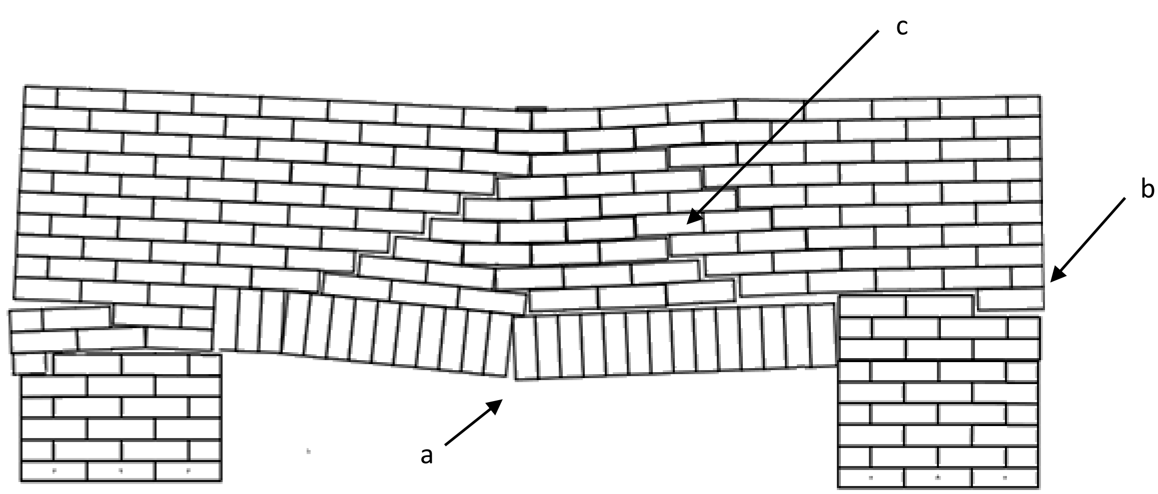


Fig. 11. Failure mode for panel DS1 predicted using UDEC.

Fig. 12. Shear development propagated from the corners of the opening to the top of the panel.

Fig. 13. Flexural crack at the right hand support.

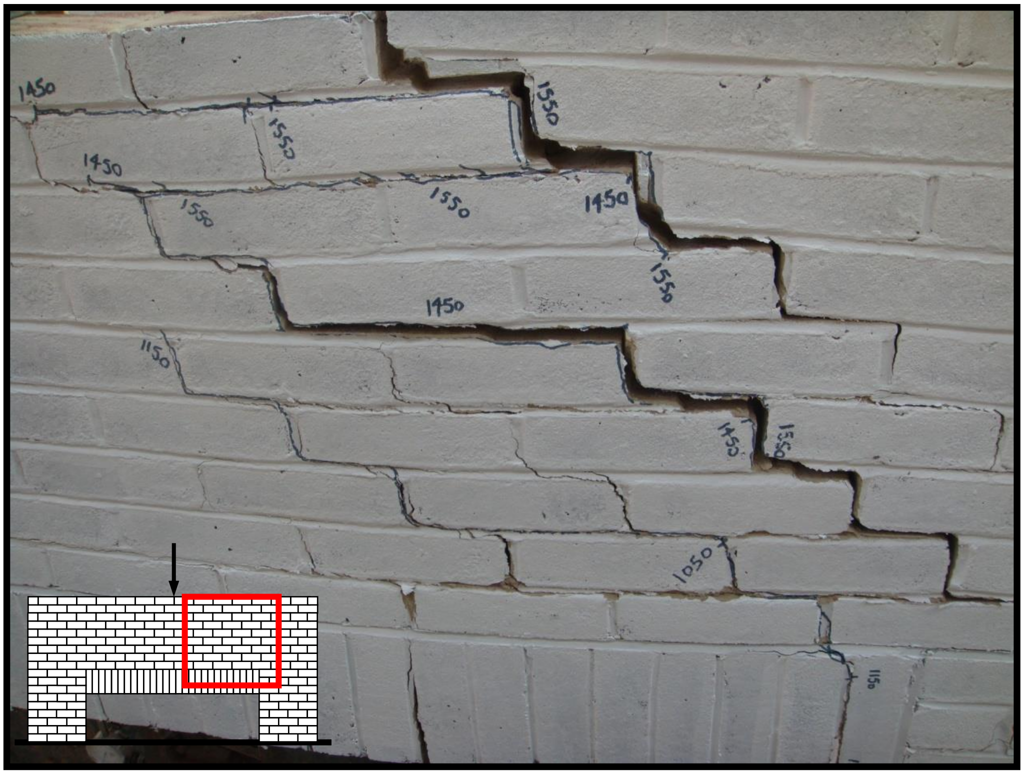


Fig. 14. View of the right hand side of the panel.

Figure 15 shows the observed and the UDEC predicted load versus displacement behaviour of the panel DS1. The UDEC predicted value of the load at first cracking (2.0kN) is close to that observed from the laboratory testing (1.72kN). Also, the ultimate load recorded in the laboratory (10.6kN) compares well with the load predicted by UDEC model (10.4kN). The stiffness of the panel observed from the experiment is similar to that found from UDEC modelling. However, as the load applied to the panel increases the two stiffness results start to deviate from each other. This could be due to short term creep effects and load redistribution that occurred in the panel with the application of load, both are very difficult to record in the lab test. Another factor contributing towards this difference is that as the panel neared a state of impending collapse, the dial gauge reading from the mid span displacement varied a great deal under constant applied load as cracks developed and propagated throughout the panel. This has influenced the accuracy of the record of the test results.

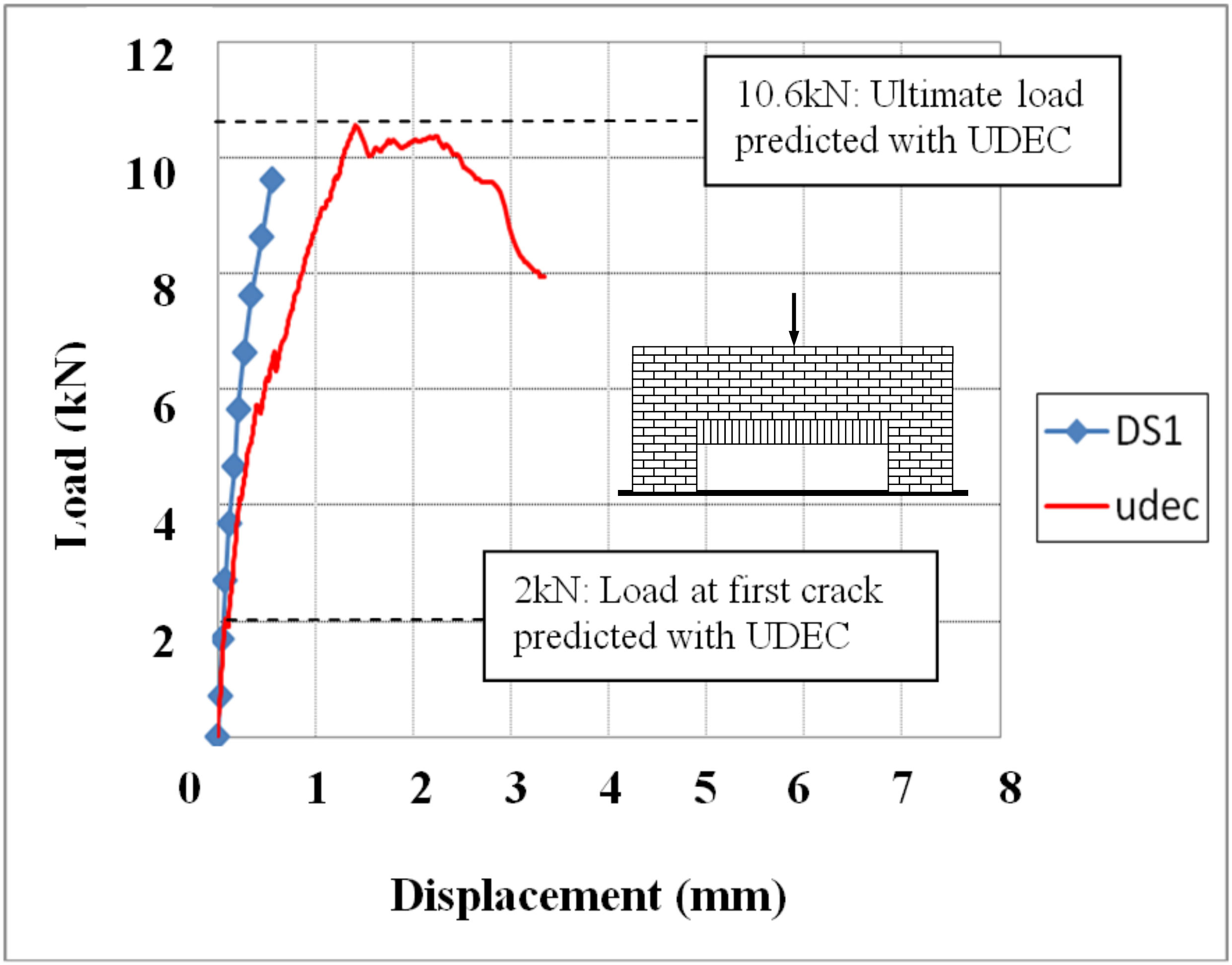


Fig. 15. Observed and UDEC-predicted load versus displacement relationships for panel DS1.

6.0 Conclusion

A study of the identification of material parameters for modelling masonry structures with UDEC has been carried out and reported in this paper. Traditionally, the material parameters used for modelling masonry in computational models are based on the results of small scale tests that do not reflect the more complex boundary conditions and stress-state types that exist in a real masonry structure. A method which is considered likely to provide more representative material parameters for masonry constitutive models has been proposed. The method involves the computational analysis of large scale experimental tests on masonry structures, and an optimization process to fine tunethe masonry material parameters by minimizing the difference between the responses measured from the large scale lab tests and those obtained from the computational simulation.

This research has used the results of a series of low bond strength masonry wall panels with opening tested in the laboratory. Such panels have also been modeled with a DEM software UDEC. The material parameters for UDEC models were "tuned" using an optimization process in order to achieve similar responses to those obtained in the laboratory. The tuning was based on the need to achieve good correlation between the pre-cracking, post-cracking and near-collapse behaviour of the masonry wall panels. Surrogate models that relate the output responses of the UDEC model to the input material parameters have been constructed to minimize the computational costs. Optimization of the material parameters was then performed using the surrogate model and a single set of optimized material parameters were obtained by this process. The use of surrogate modelling for creating approximations has proved to be a useful approach as it resulted in less computational iterations, and led to substantial saving of computational resources and time. To validate the optimization results, the material parameters obtained from the optimization process were then used in a UDEC model to predict the structural response of different wall panels. In spite of the inherent variability of masonry, good correlation was achieved between the predicted behaviour from UDEC model and that observed in the laboratory. The developed method provides a more rigourous tool for the identification of the material parameters for modelling masonry structures, and has reduced the uncertainties associated with the traditional methods. As for the future study, the accuracy of the optimization method can be improved by using larger number of test data and the developed computational model for masonry can be used to study the strengthening of masonry structures.

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