Contribution to Numerical Modelling of Concrete-Masonry Interface In Concrete Framed Structures With Masonry Infill

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Abstract

Masonry infills have long been used as interior partitions and exterior walls in buildings. They are usually treated as non-structural elements, and their interaction with the bounding frame is often ignored in design. Nevertheless, infill contributes strength to a structure and will interact with the bounding frame when the structure is subjected to strong lateral seismic loads, when the infill is stressed due to movements of an overlying slab or any other case of in-plane or out of plane lateral loading. This interaction may or may not be beneficial to the performance of the structure, however, and it has been a topic of much debate in the last few decades. The interaction of the infill is governed by the relative stiffness and strength characteristics of each individual component and most importantly the interface characteristics that decide the degree of composite action.

An interface is a special contact plane on which nonlinear relations between stresses and displacement discontinuities are present. Very often initiation and propagation of cracks along these interfaces are the cause of failure of the relevant structures. Similarly, in the case of concrete framed masonry assemblages, the bond between the masonry and the concrete frame is a weak link, through which failure is possible. Therefore to simulate this behaviour, interface elements with a suitable constitutive model can be utilized.

This paper explores finite element models developed to simulate the behaviour of concrete-masonry interface of masonry infill. In this study, brick-concrete couplets were mathematically modelled, using commercially available software ANSYS. The adopted numerical strategy consists of simplifying the concrete-masonry-mortar interface to a zero thick interface, modelling the brick units and the concrete units with three dimensional solid brick elements and modelling the bond using zero thickness interface elements with a cohesive-zone model (CZM) for mixed-mode fracture based on damage mechanics introduced by Alfano and Crisfield(2001).

Key words

Displacement discontinuities, cohesive-zone model, zero thick interface, numerical modelling of masonry, non-linear behaviour.

1. Introduction

Masonry infills have long been used as interior partitions and exterior walls in buildings. They are usually treated as non-structural elements, and their interaction with the bounding frame is often ignored in design. Nevertheless, infill contributes strength to a structure and will interact with the bounding frame when the structure is subjected to strong lateral seismic loads, when the infill is stressed due to movements of an overlying slab or any other case of induced in-plane or out of plane lateral stresses. This interaction may or may not be beneficial to the performance of the structure, however, and it has been a topic of much debate in the last few decades. The performance of such frame structures with infill, during an earthquake has attracted major attention. Even though frame–infill interaction has sometimes led to undesired structural performance, recent studies have shown that a properly designed infilled frame can be superior to a bare frame in terms of stiffness, strength, and energy dissipation. Similarly, in the case of infill cracking due to thermal movements of an overlying slab which is a common problem in a tropical country like Sri Lanka the interaction between concrete beam and masonry wall plays a major role.

According to Ghassan K (2008), experimental investigations of the behavior of masonry-infilled steel and reinforced concrete frames under in-plane and out of plane lateral loading has been the subject of many researchers. Starting back in 1970 with the experimental studies of Fiorato et al who tested 1/8-scale non-ductile reinforced concrete as cyclic lateral loading the investigations were followed by the studies of many other researchers like Klingner and Bertero in 1976, Bertero and Brokken in 1983, Zarnic and Tomazevic in 1985, and Schmidt in 1989. More recently, single-story reinforced concrete frames with masonry infills were studied by Mehrabi et al in 1994 and 1997, Angel et al in 1994, and Al-Chaar et al in 1998 and 2002. Masonry infilled steel frames were tested by Dkanasekar et al in 1985, Dawe and Seah in 1989, Mander et al in 1993, and many others.

All experimental studies cited above have shown that the behavior of an infilled frame is heavily influenced by the interaction of the infill with its bounding frame. In most instances, the lateral resistance of an infilled frame is not equal to a simple sum of the resistance of its components because frame/infill interaction can alter the load-resisting mechanisms of the individual components. At low lateral loading, an infilled frame acts as a monolithic load resisting system and as loading increases, the infill tends to partially separate from the bounding frame. Due to this a compression strut mechanism is formed, as observed in many earlier studies.

Due to the presence of a vast range of geometrical and structural configurations of masonry, use of physical models to investigate masonry is costly and difficult. As a result finite element method (FEM) has been widely used in the analysis of concrete and steel framed masonry structures. A number of different analytical models have been developed to evaluate these infills. Dhanasekar and Page (1986) and Liauw and Lo (1988) have used linear and nonlinear beam elements to model the behaviour of steel frames, and interface elements to model the interaction between the infill and the frame. Dhanasekar and Page used a nonlinear orthotropic model to simulate the behaviour of brick infills, and Liauw and Lo used a simple smeared crack model to simulate the behavior of micro-concrete infills. Schmidt (1989) used smeared crack elements to model both reinforced concrete frames and brick infills. In all those analyses, infill panels have been modelled as a homogenous material before fracture, and the effects of mortar joints have been smeared out. These models for concrete-masonry framed structures have a major deficiency of sufficient attention not being paid to simulate the interaction through the concrete beam and masonry infill junction.

Further in Engineering practice some of the commonly used interface types at the beam wall junction include (i) leaving a gap between the frame and the infill in order to avoid transfer of load between frame and infill (ii) breaking of bond between frame and infill (no bond/non integral) (iii) connecting the frame and the infill by provision of shear connectors (integral interface) (iv) connecting the frame and the infill by cement mortar (conventional type) and (v) using of non-structural materials like lead sheet, cork etc. Therefore a numerical approach has to be adopted to simulate the concrete masonry infill interface behaviour and to explore the effect of bond between concrete and masonry infill on the global behaviour of these framed structures.

An interface is a special contact plane on which nonlinear relations between stresses and displacement discontinuities are present. Therefore to simulate this behaviour interface elements with a suitable constitutive model can be utilized and essentially this should be verified with adequate experimental data. The failure mechanisms of Couplets under direct tension and shear loading can be differentiated in to the following two kinds of rupture:

- Rupture occurring along the mortar/brick interface for machine made bricks and some of the hand-made brick specimens (see *Figure 1*),
- Rupture beginning along the interface and crossing through the mortar layer and crushing of brick (see *Figure 2*).

Figure 1: Rupture modes in couplet specimens with machine made bricks

Figure 2: Rupture modes in couplet specimens with hand-made bricks

The interface models used should be able to simulate these situations. An interface may be treated as a completely bonded interface or an incompletely bonded interface (Song and Kawakami 1998). The completely bonded interfaces should prevent large relative displacement from occurring at the contact plane. The incomplete bonded interfaces may have the behaviours of sliding, de-bonding and re-bonding, rotation etc.

Further, in the context of the finite element method, there are two major groups of interface elements/models known as the *"zero thickness"* interface element and *"thin layer"* interface elements. Athanasios (2003) has conducted a detailed study about these techniques of modelling interfaces in discontinuous systems. According to him, the explicit representation of discontinuities by means of FEM and the so called interface element goes back to the work of Goodman, Zienkiewicz, Mahtabh and Gaboussi in the 1970s. According to Rots (1997) **t**he method was first developed and applied to solid masonry by Page in 1988.

Several interface models have been proposed in literature to study crack propagation in these incompletely bonded cementitious materials and at bimaterial interfaces. According to the studies of G. Alfano (2006), some of these recent advancements of the interface modelling are results of the work of many researchers as Carol et al, Ceˇrvenka et al, Ruiz et al, Alfano and Crisfield, Marfia and Sacco and Cocchetti et al. Among them, Alfano and Crisfield have presented an interface model for mixed-mode fracture based on damage mechanics which has applications in the delamination analysis of laminated composites and which therefore provides the theoretical background for the damage and debonding model of interface elements in several commercial software as ANSYS[[ANSYS 12.0, Theory reference manual].

In the micro modelling of masonry special attention has been paid to model the brick-mortar joint interface. A number of plasticity-based continuous-interface models have been developed to model the tension and shear behaviour of masonry mortar joints (Rots 1997, Lorenco and Rots 1997). Those models account for the interaction between normal compression and shear as well as the shear dilatation often observed in experiments. Mehrabi and Shing (1997) have developed an interface model for analyzing masonry infills that accounts for the increase of contact stress due to joint closing, the geometric shear dilatation, and the plastic compaction of a mortar joint. The failure surface of the model is based on a hyperbolic function proposed by Lotfi and Shing (1994), and is capable of modelling damage accumulation at mortar joints under increasing displacement and cyclic loading. This is reflected by shear strength reduction and mortar compaction (loss of material) at interfaces. The model has been used to analyze the infilled frames tested by Mehrabi et al. (1994). The application of these models to simulate the concrete masonry interface is yet to be explored. Also, studying about the effect of concrete-masonry interface on the performance of masonry has become subject of many researches. Attempts have been made numerically (Ibrahim and Suter, 1990, Mehrabi et. al., 1997, Asteris, 2008) as well as experimentally (Dias 2005).

This paper explores finite element models developed to simulate the behaviour of concrete-masonry interface of masonry infill. In this study, brick-concrete couplets were mathematically modelled, using commercially available software ANSYS. The adopted numerical strategy consists of simplifying the concrete-masonry-mortar interface to a zero thick interface, modelling the brick units and the concrete units with three dimensional solid brick elements and modelling the bond using zero thickness interface elements with a cohesive-zone model (CZM) for mixed-mode fracture based on damage mechanics introduced by Alfano and Crisfield(2001).

2. Methodology

2.1 Modelling assumptions and the numerical models used in the study

A typical masonry brick- concrete block couplet was considered. The bricks were of the standard size of 210x105x50mm and the grade 25 concrete block modelled was of a similar size. The thickness of the mortar joint was considered as 10mm which is commonly used in construction practice. One micro model was created with solid elements to model the brick, concrete block and the mortar joint separately as show in *Figure 3*. In this no interface elements were used for the interfaces between the mortar-concrete and mortar-brick and the junctions were considered to be fully bonded (model Type1). The other was created with zero thickness contact target elements and fully bonded option (model Type 2) as given in *Figure 5*. The two types of models for couplets were analysed for the cases of direct shear and direct tension. All degrees of freedom at the bottom of the brick and the two faces in the direction parallel to the load application in direct shear test were restrained in order to model the direct shear test method boundary conditions (*Figure 5)*. In the case of the couplet under direct tension only the bottom surface was considered fixed (*Figure 6)*.

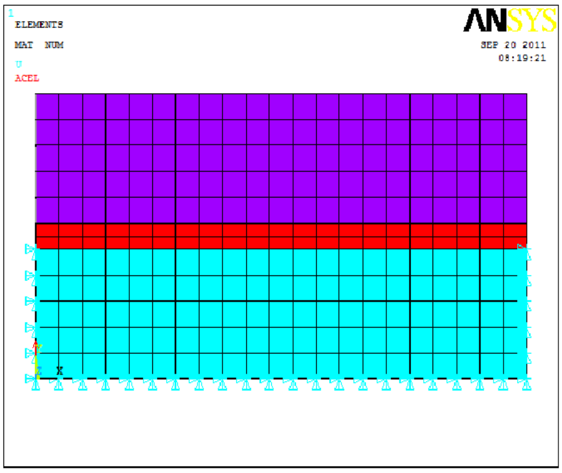

Figure 3: Micro model Type1

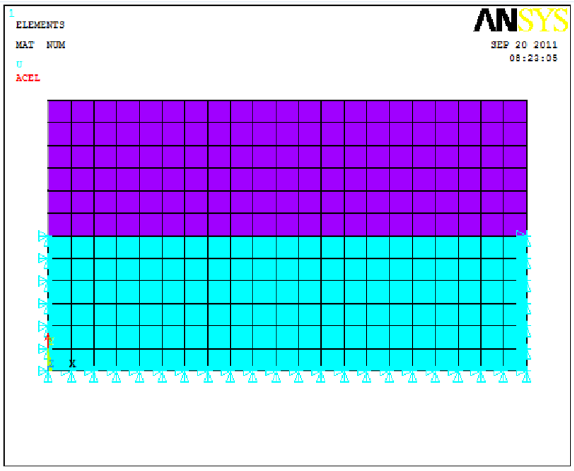

Figure 4: Micro model Type2 (With interface)

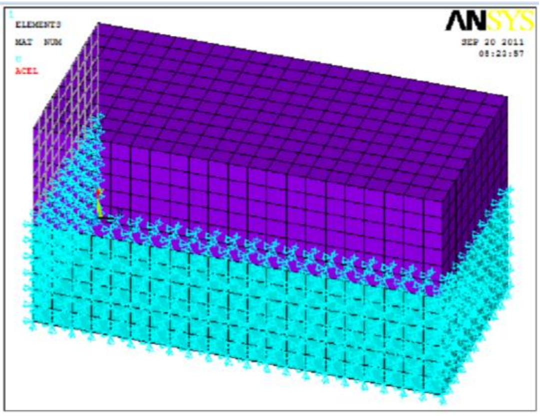

Figure 5: Boundary conditions for shear

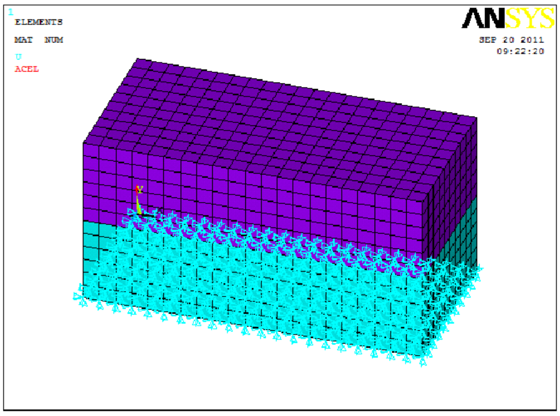

Figure 6: Boundary conditions for direct tension

2.2 Numerical modelling of cracking

A three dimensional eight noded isoparametric element SOLID65 was employed for the modelling of both concrete and masonry elements. The element is capable of cracking (in three orthogonal directions), crushing, plastic deformation and creep. The material model of SOLID 65 in ANSYS, having a five parameter Williams-Warnke failure criterion [ANSYS 11.0, Theory reference manual], is implemented to measure cracking or crushing of the material (*Figure 7)* – symbols are defined in ANSYS 11.0, Theory reference manual. Since there is no possibility of cracking of the concrete elements, those elements were created without the crack model.

The SOLID65 element has eight Gauss integration points at which cracking and crushing checks are performed. The element behaves in a linear elastic manner until either the specific tensile or compressive strength is exceeded. If cracking or crushing occurs at an integration point, the cracking is modeled through an adjustment of material properties which effectively treats the cracking as a smeared band of cracks. In numerical routines the formation of a crack is represented by the modification of the stress-strain relationships of the element to introduce a plane of weakness in a direction normal to the crack face. Also, a shear transfer coefficient is used to represents a shear strength reduction factor for those subsequent loads that induce sliding across the crack.

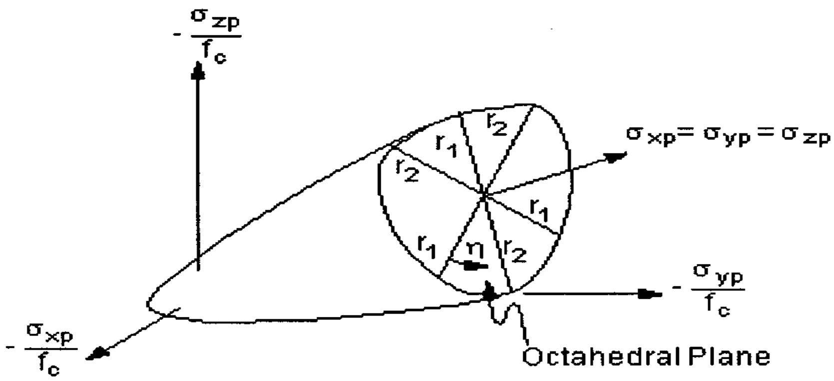

Figure 7: The failure surface described in three dimensional principal stress space

2.3 Numerical modelling of interface

The total behaviour of the couplet is governed by the relative stiffness and strength characteristics of each individual component. Out of these factors the interface characteristics are the vital ones that decide the degree of composite action.

ANSYS provides two methods to model separation of interfaces - i.e. an interface element with cohesive zone material model and a contact element with bonded contact option and a cohesive zone material (ANSYS 11.0, Contact technology guide). In this study contact element with bonded contact option and a cohesive zone material was used to model interface.

For the modelling of the masonry-concrete interface, the surface to surface contact element of “CONTAC173” with target element “TARGE170” was used. ANSYS provides two cohesive zone material models with bilinear behaviour to represent debonding. The material behaviour, defined in terms of contact stresses (normal and tangential) and contact separation distance (normal gap and tangential sliding), is characterized by linear elastic loading followed by linear softening. Debonding allows three modes of separation;

1. Mode I debonding for normal separation

2. Mode II debonding for tangential separation

3. Mixed mode debonding for normal and tangential separation.

Debonding is also characterized by convergence difficulties during material softening. Artificial damping is provided to overcome these problems. After debonding is completed, the surface interaction is governed by standard contact constraints for normal and tangential directions. The cohesive zone material model with bilinear behaviour is defined as:

$$P = K_n U_n (1-d)\ ,\ \tau_y = K_t U_y (1-d) \text{ and } \tau_z = K_t U_z (1-d)$$

where; P- Normal contact stress (tension), τ_y - tangential contact stress in Y direction, τ_z - tangential contact stress in Z direction, K_n - Normal contact stiffness, K_t - Tangential contact stiffness, U_n - Contact gap, U_y - Contact slip distance in Y direction, U_z -Contact slip distance in Z direction, d - Debonding parameter.

The following material constants were used (see Table 1) in order to define the material behaviour with traction and separation.

Table 1: Material constants for defining interface behaviour

Constant	**Symbol**	**Meaning**
C1	σ_{max}	Maximum normal contact stress
C2	U_n^c	Contact gap at the completion of debonding
C3	τ_{max}	Maximum equivalent tangential contact stress
C4	U_t^c	Tangential slip at the completion of debonding
C5	η	Artificial damping coefficient
C6	β	Flag for tangential slip under compressive normal contact stress

Here U_n^c and U_t^c were defined in ANSYS as;

$U_n^c = 6U_n$ and $U_t^c = 6U_t$.

For brittle materials like concrete, U_n and U_t are the displacements corresponding to σ_{max} and τ_{max} respectively. The values for K_n, K_t, σ_{max} and τ_{max} were the same as the values defined earlier in this chapter. The values of η and β were defined according to the instructions of ANSYS manual as;

η = 1000 x modulus of elasticity and $\beta = 0$

2.4 Simplification of concrete beam masonry infill junction in to a zero thickness interface

The method developed by Rots (1997) to model the masonry-mortar interface was used in this study to model the concrete–masonry interface. In this model the constitutive behaviour of the unit is described by stress-strain relations for the continuum element. In the linear elastic range, the stress (σ) vs the strain (ε) relationship can be described according to Hooke’s law. The presence of a 10 mm thick mortar layer between concrete and masonry was assumed in this simulation. The simplification of the concrete-masonry-mortar interface to a zero thick interface is illustrated in Figure 5

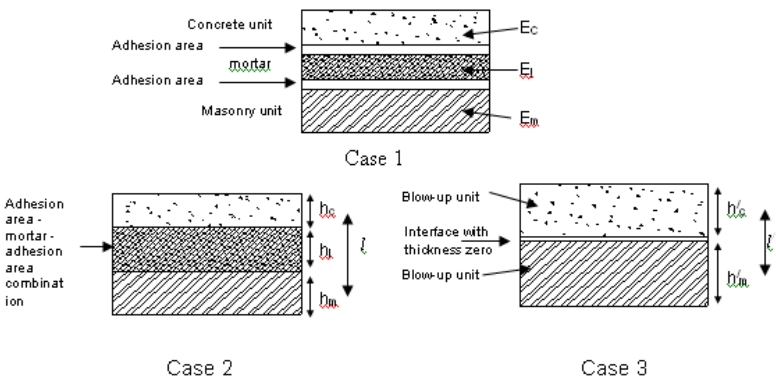

Figure 8: Simulation of the concrete-masonry joint

Case 1 (Figure 8) shows the actual situation with adhesive areas on both sides of the mortar layer. In Case 2 (Figure 8) a compound interface has been created, accounting for both adhesion areas and the mortar layer. Finally in Case 3 (Figure 8) the concrete and masonry units were “blown up” to create an interface with zero thickness but with the properties of the adhesion area-mortar layer-adhesion area combination.

The total lengthening across half the units and joint should be equal in both Case 2 (over a length l) and Case 3 (over a length l')

For Case 2

$$\Delta\ell = \sigma\left[\frac{h_c}{2E_c} + \frac{h_j}{E_j} + \frac{h_m}{2E_m}\right]$$

where h_c, h_m and h_j are the thicknesses and E_c, E_m *and* E_j are the moduli of elasticity of the concrete, masonry and joint respectively.

For Case 3 (the simplified model)

$$\Delta\ell' = \sigma\left[\frac{h'_c}{2E'_c} + \frac{1}{k_n} + \frac{h'_m}{2E'_m}\right]$$

however, Δl should be equal to $\Delta l'$ and we have also assumed that blown up units have the same E value as the real units (i.e $E_c = E'_c$ and $E_m = E_m'$)

Also, $h'_m = h_m + \dfrac{h_j}{2}$ and $h'_c = h_c + \dfrac{h_j}{2}$

Therefore the normal stiffness, k_n, of the interface element will become:

$$k_n = \frac{4E_cE_jE_m}{h_j\left(4E_cE_m - E_mE_j - E_cE_j\right)}$$

Similarly the shear stiffness, k_t, of the interface element is given by:

$$k_t = \frac{4G_cG_jG_m}{h_j\left(4G_cG_m - G_mG_j - G_cG_j\right)}$$

where $G = \dfrac{E}{2(1+\nu)}$

2.5 Material Properties and Loading

The material properties (See Table 2) were taken from earlier studies done on numerical modelling of cracking in masonry (Dilrukshi & Dias 2008).

Table 2: Material properties

	Concrete	**Masonry**	**Mortar**
Weight per unit volume (kN/m^3)	23.6	20	20
Modulus of elasticity (kN/m^2)	$25x10^6$	$1x10^6$	$1x10^6$
Poisson ratio	0.2	0.2	0.2

The tensile and shear strength of mortar joint was considered as 0.2 N/mm^2 and 0.13 N/mm^2 respectively.

The self load was applied at the first load step and a direct shear load of 275kg with no pre-compression was then applied to the couplets at a number of sub steps for Case1. In Case 2 for both models with and without interface elements, additional to the self load a direct tensile load of 40kg was applied. The analysis was performed at each sub step and results of each sub step were recorded.

3. Results and Discussion

The results of the two representations of the couplets for both load cases are according to the Figures 9-18 and Table 3 summarises the stresses transferred to the top of the brick surface through the concrete block-brick interface. It is observed that no significant variations in the results have occurred for both models.

3.1 Couplet behaviour under direct shear

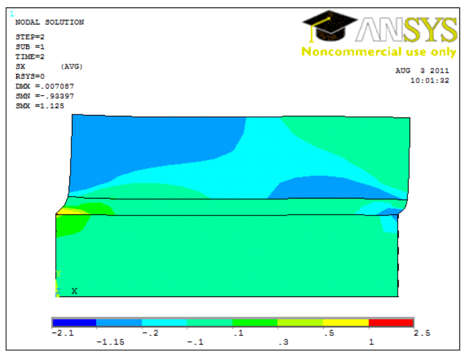

Figure 9: X direction stresses for model Type1

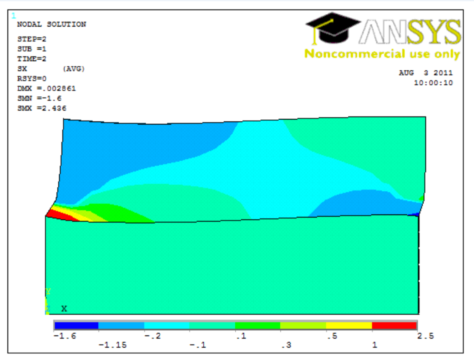

Figure 10: X direction stresses for model Type2

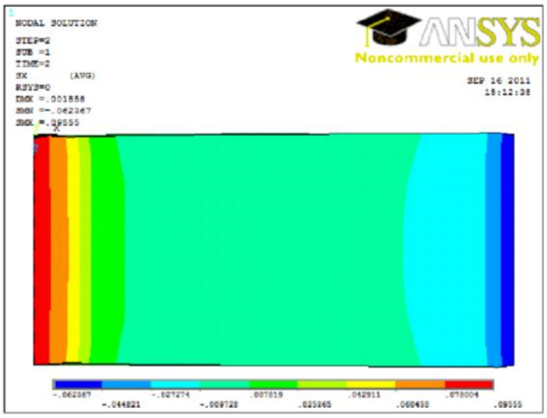

Figure 11: X direction stresses at top of brick surface for model Type1

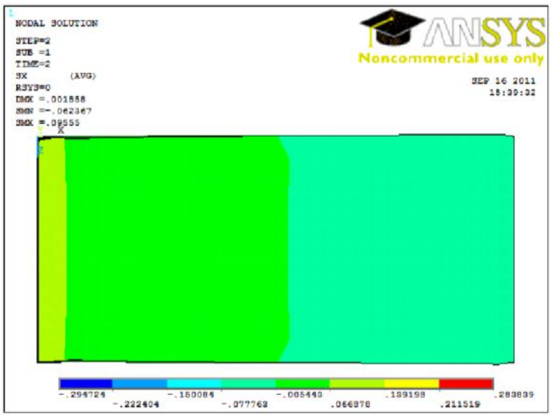

Figure 12: X direction stresses at the top surface of the brick for model Type2

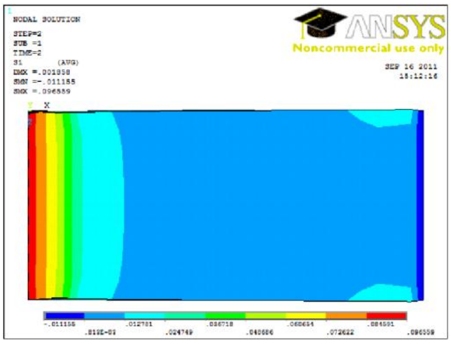

Figure 13: 1st principle stresses at top of brick surface for model Type1

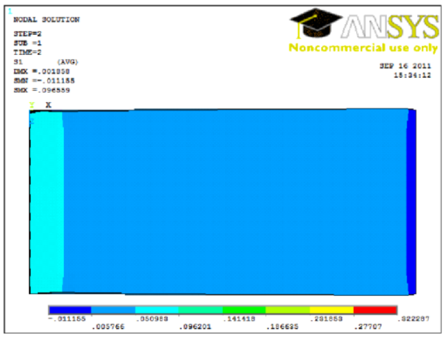

Figure 14: 1st principle stresses at the top surface of the brick for model Type2

3.2 Couplet behaviour under direct tension

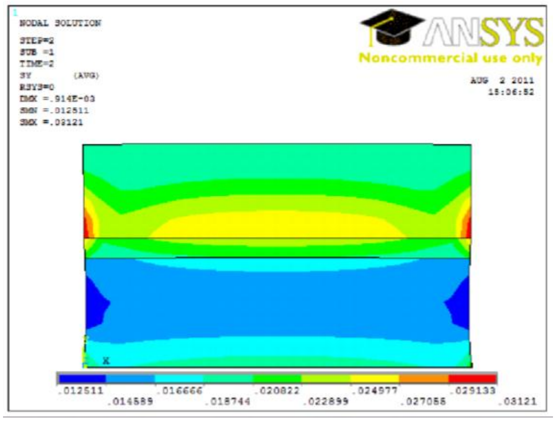

Figure 15: Y direction stresses for model Type1

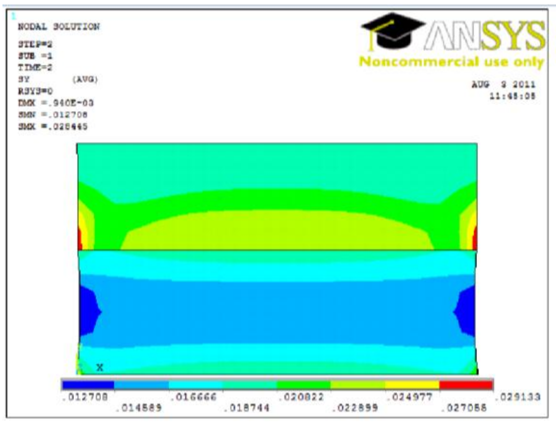

Figure 16: Y direction stresses for model Type2

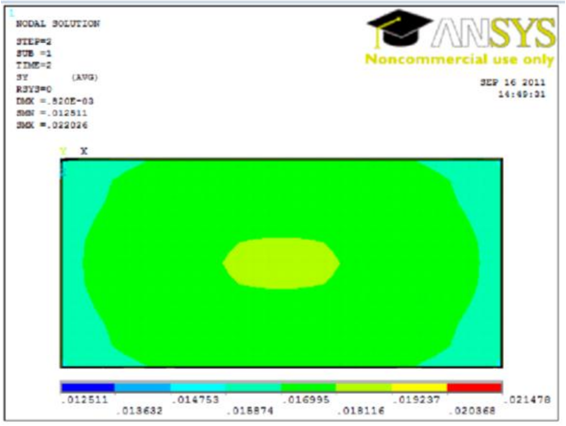

Figure 17: Y direction stresses at top of brick surface for model Type1

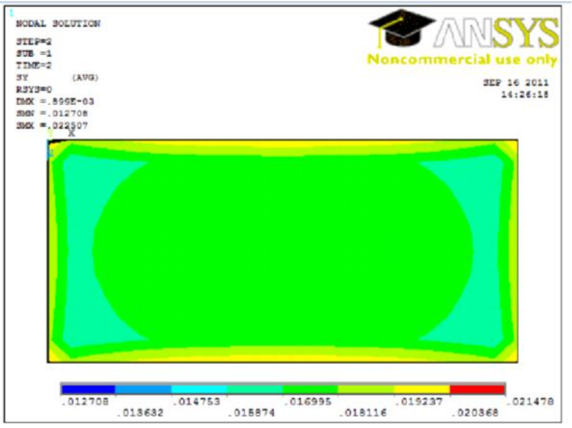

Figure 18: Y direction stresses at the top surface of the brick for model Type2

Table 3: The stresses at a point in the middle of the top of the brick surface for both load cases

Model Type and load case	**X direction stresses (N/mm^2)**	**Y direction stresses (N/mm^2)**	**1st Principle stresses (N/mm^2)**
Direct Shear for model Type1	-0.55774E-02	-	-
Direct Shear for model Type2	-0.53425E-02	-	-
Direct Tension for model Type1	0.39342E-02	0.18178E-01	0.18178E-01
Direct Tension for model Type2	0.40954E-02	0.17993E-01	0.17993E-01

(For the Shear case the stresses are for the final load step of a direct shear load of 275kg.
For the tensile case the stresses are for the final load step of a direct tensile load of 40kg)

Results do not show significant variation of stresses for both types of models. Therefore the compatibility between the two models is verified for the case of fully bonded interface. The use of the method developed by Rots (1997) to simplify the masonry-mortar interface in to a zero thickness interface is valid for this case.

The deformations at the brick surface from one edge to the other of the brick along the longitudinal axis for both load cases at nodes of 10mm interval are as shown in Figure 19 and 20.

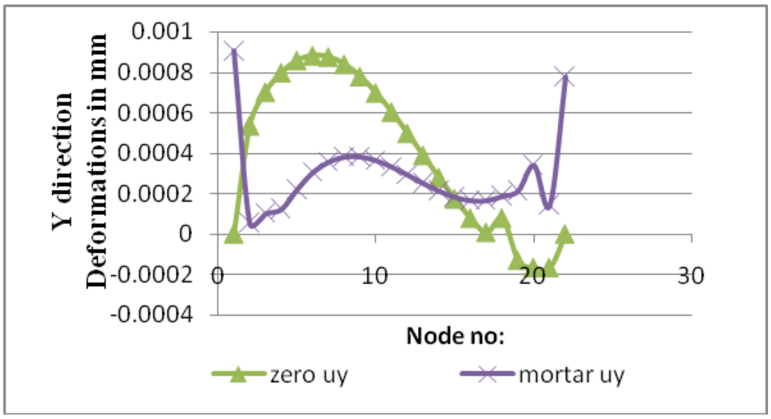

Figure 19: Horizontal deformations along the brick surface for direct shear load case

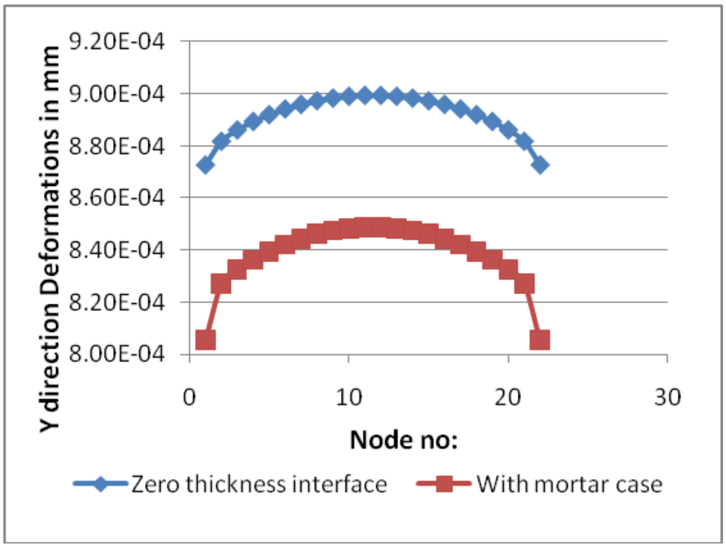

Figure20: Vertical deformations along the brick surface for direct tensile load case

The vertical and horizontal deformations along the brick surface are shown in Figures 19 and 20. The differences in both types of deformations for the considered cases are less than 10% and hence considered acceptable.

4. **Conclusions**

- The use of the method developed by Rots (1997) to simplify the masonry-mortar interface in to a zero thickness interface is valid for this case.

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