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Numerical micro-modeling of masonry in filled frames

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ABSTRACT

Reinforced concrete frames are usually infilled by masonry walls, but in most design, the shear strength response of these walls is ignored. Hence the nonlinear numerical modeling of masonry infilled frames is one of the most complicated problems in structural engineering especially when such structure is under the influence of lateral loads. However previous experimental test results have indicated that infilled mansonry have a significant influence on the shear strength of the infilled frame, but this process is very expensive and time consuming. In this study a proper numerical model is made using explicit finite elements method to study the behaviour of the masonry inflilled reinforced concrete frames. The basic characteristic of this model is that the complexity which is attributed to the varying mechanical and geometrical properties, existence of joints as a source of weakness and the interaction between the reinforced concrete frame and infill is properly taking into account. In order to undertake this analysis, a suitable computer programme was also developed. To ensure the ability of this method to precisely stimulate the shear strength response of masonry infilled panel, results obtained from experimental test are compared with that obtain using this finite element method.

Keywords: Mansonry infilled frame, finite element method, shear strength, stresses.

INTRODUCTION

In most seismic regions of the world, reinforced concrete infilled frame structures which are associated with high rise buildings have raised serious concerns to designers. Infill walls may also be built between columns of single story frame buildings. In these high rise buildings the interaction between infill and the frame under in-plane load depends on the infill being constructed Snugly inside the surrounding frame, hence care is required to construct a right fit in the frame, however, thermal and moisture expansions of the infill wall and shrinkage of the frame have been shown to cause structural distress and, in such cases, a movement joint must be provided (Suter and Hall 1976). The interaction mechanism between an infill wall and the surrounding structural frame also depend on the area of contact at the interface of the two components; thus the extent of composite action will depend on the level of lateral load, degree of bond or anchorage at the interfaces, geometric and stiffness characteristics of the two components (Sabnis, 1976). Extensive experimental research by (Smith 1960, Dawe and Seah 1989, Harris et al. 1993, Mehrahi et al. 1996, Buonogame and White 1999) have shown results that indicated that for very strong infilled reinforced concrete frames, the columns can fail in shear, hence proper anchorage of the masonry infill to the surrounding frame would reduce separation and cause failure to take place in the masonry without causing premature shear/flexure failure of the column.

Full scale cyclic test on single story frames infilled with brick masonry have been conducted by (Flanaga et al. 1993) to study seismic response. The study show that for small value of lateral loads a gap existed between the infill wall and the frame. Test on multistory small-scale infilled reinforced concrete frames showed the same pattern. It is pertinent to note that the need to consider the response of infilled frames to shear loads became necessary after earthquake in the 80's when buildings failed predominantly in corner crushing patterns (Mehrahi et al. 1996, Amrhein et al. 1985). Significant experimental and analytical research has been conducted to develop analytical methods for typical infilled frames (Polyakov 1963, Leuchars and Scrivener 1975).

Numerical methods utilizing the finite element method was not easily conducted early because of uncertainty in defining the boundary conditions of these structures; hence approximate methods using equivalent diagonal struts to replace the infill panel was initially proposed by (Polyakov 1960, 1963) and further developed by (Stafford – Smith and Riddinton 1976). A research was also carried out on equivalent diagonal strut stimulation of the influence of masonry infills on the seismic response of reinforced concrete frame structures by (Manos et al. 1983). In his work which is on test on small-scale two story infilled reinforced concrete frames under-simulated earthquake loads he concluded that the diagonal strut analogy would yield reasonable prediction of lateral response. In this method, the system is modeled as a braced frame where the infill walls provide the web elements (equivalent diagonal struts). The effective length of the diagonal strut can be taken as equal to one-third of the diagonal length of the panel (Holmes 1990) while the strut maintains the same mechanical and physical property as the infill panel. A pin jointed equivalent strut system would not be able to indicate the bending moments and shearing forces in the frame members (Saneinejad and Hobbs 1995) and this has lead to the proposition of more complex macro-models to handle the complex behaviour of infilled frames, especially the response of brickwork to seismic forces.

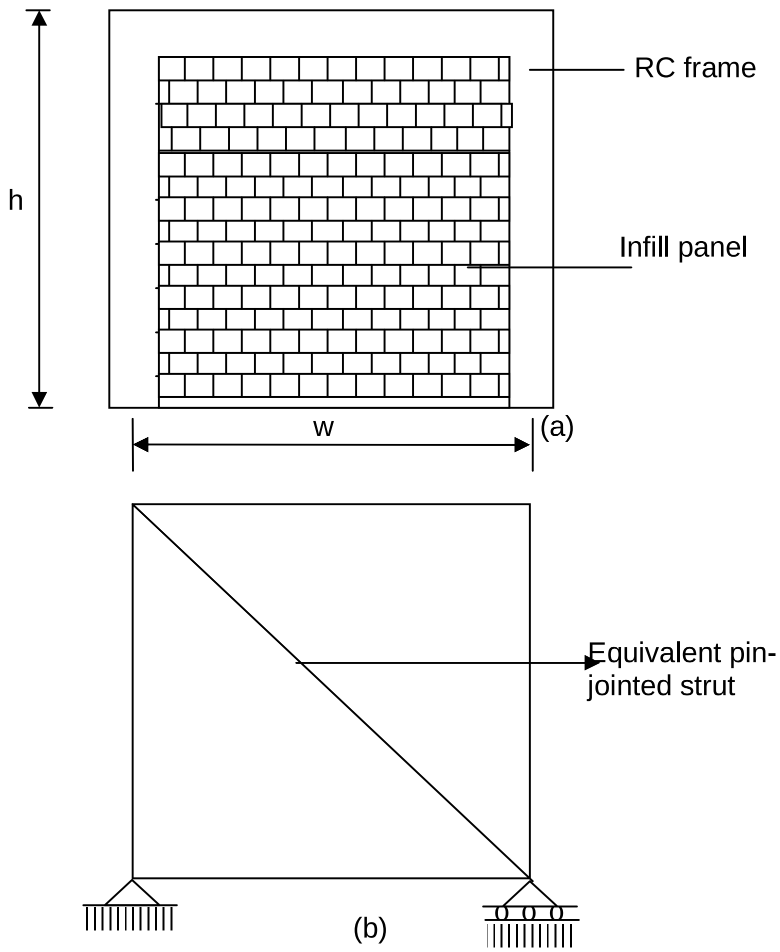

Figure 1: (a) infilled reinforced concrete frame (b) equivalent one-strut, macro-model

The short comings of a macro-model is that they fail to model the whole possible modes of failure while a micro-models considers all possible modes of failure including the boundary conditions of the structural system. The development of the finite element method has to a great extent aided the micro-modeling of infilled frames, both in static and dynamic regime. The finite element method involves using different shapes of elements where the most common shapes are the triamgular and rectangular elements. Mallick and Garg (Marllick and Garg 1971) simulated infill panels using linear elastic rectangular finite elements with two degree of freedom at each node which is situated at the four angular ends of the element, while the frame was simulated assuming a beam element ignoring axial deformation. Also attempt have been made to study the response of infilled frame under monotonic loading using different types of elements (Liauw and Kwan 1984). Here the infill panel was modeled using triangular constant strain elements while the infilled frame modeled using bar type elements. The infill panel continuum was assumed to be isotropic before cracking and then anisotropic after cracking. Their result showed a nonlinear relationship between the stress and strain when the panel is under compression, however using an iterative procedure it was possible to stimulate the response of a one-bay storey infilled frame by adopting an incremental displacement.

It is clear that previous analytical models were not able to consider the real composite behaviour of an infilled frame structure and also model properly the infill/frame boundary problem. In this present study a new finite element method using the displacement approach would be used to model the ideal composite structure. Also a useful computer code would be formulated for quick analysis of the infilled frame. The new finite element approach will formulate a criterion for the solution of the boundary problem at the infill/frame interface and also adopt a constant strain triangular element for the in-plane anisotropic behaiviour of the masonry infill panel.

2. Modeling Strategy

In the work, the major assumptions in modeling masonry behaviour under plane stress and plane strain would be adopted. It is believed that the strength and deformation behaviour of the wall is greatly affected by the resulting stress. Hence for a better understanding of the brick-wall behaviour, a numerical analysis would be necessary.

The model would be considered as a two-dimensional plane stress and strain problem. Hence in this present study the both phases of the material are replaced with an equivalent material property, assuming is to be a homogenized material.

When masonry is assumed to be composite (heterogeneous) material, hence different mechanical properties for brick and mortar region, and hence a heterogeneous model, masonry walls are analyzed as discretizing brick and mortar separated through finite element and/or interface elements, while in the homogenous models, masonry is assumed as a continuous material with properties either obtained from tests or empirical equations.

The selection of appropriate and practicable “Numerical model” and feasible finite elements depend on the computational effort required for handling and processing the model and the verification of the desired results. It is a well known fact that homogenous models of masonry with finite elements provide sufficient accuracy with reasonable computational effort. On the other hand homogeneous models with plane elements render better results than that of the former models but at the cost of high computational effort, especially when a real wall or building must be analyzed. It is therefore clear that, for numerical modeling of full scale field model, plane element homogenous models will be employed. Nevertheless, heterogeneous and homogeneous models of brick-mortar couplets have been developed and verified by comparing the empirical and experimental results. The basic objective in present times is to get familiar with the capabilities of the software used and to verify the software results in order to make sure that the final model is perfect.

The best numerical model is the one that represents the maximum characteristics of the actual structure, the process of representation of an actual structure into a numerical model in a particular finite element software environment needs continuous refinement. This can be best achieved when sufficient experimental data of the relevant parameter of the model is available for validation against the result of numerical model.

3. Numerical Method

For the purpose of this study the finite element method of analysis for a continuum would be used. Basic triangular elements shall be used and the formulation adopted in the displacement approach. In using this method the model displacements are the basic unknown, while the stresses and strain are assumed to be constant for each element. The finite element method of analysis used in this paper would involve voluminous numerical works which would be considerably simplified by matrix formulation of the whole problem, hence very suitable for computerization.

Hence for plane elasticity problem the elastic matrix denoted by [D] can be expressed as

$$D = \begin{bmatrix} \dfrac{E_x}{1-v_{xy}\,v_{yx}} & \dfrac{E_x v_{yx}}{1-v_{xy}\,v_{yx}} & 0 \\ \\ \dfrac{E_y v_{xy}}{1-v_{xy}v_{yx}} & \dfrac{E_y}{1-v_{xy}v_{yx}} & 0 \\ \\ 0 & 0 & E_y \big/ 2\,(1+v_{yx}) \end{bmatrix} \qquad (1)$$

where E_x and E_y are the modulli of elasticity in the x and y direction respectively, V_{xy} and V_{yx} are the poisson's ratio in the xy and yx plane respectively.

The element stiffness matrix $[K^e]$ would be a 6 x 6 matrix for the plane elasticity triangle, because there exist two degree of freedom (DOF) at each node of the triangular element see Figure 2, hence the Nodal force vector $[F^e]$ can be related to the displacement vector as in equation 2.

$$\{F^e\} = [K^e]\{\delta^e\} \qquad (2)$$

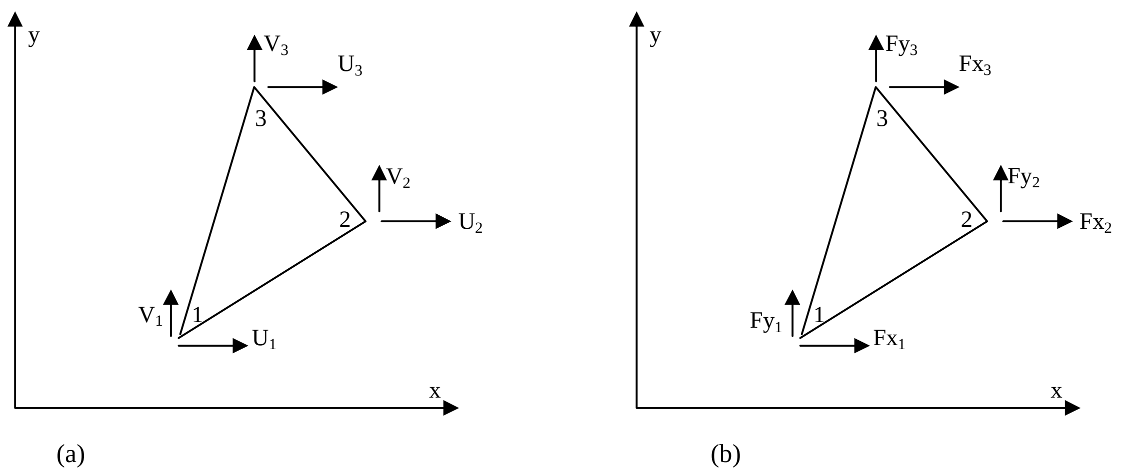

Figure 2: (a) Nodal displacements vector (b) Nodal force vectors, displayed in the Cartesian co-ordinate system.

The displacement vector for the element may be expanded as follows

$$\{\delta^e\} = \begin{Bmatrix} \delta_1 \\ \delta_2 \\ \delta_3 \end{Bmatrix} = \begin{Bmatrix} u_1 \\ \upsilon_1 \\ u_2 \\ \upsilon_2 \\ u_3 \\ \upsilon_3 \end{Bmatrix}$$

The corresponding force vector can similarly be represented below

$$\{F^e\} = \begin{Bmatrix} F_1 \\ F_2 \\ F_3 \end{Bmatrix} = \begin{Bmatrix} F_{x1} \\ F_{y1} \\ F_{x2} \\ F_{y2} \\ F_{x3} \\ F_{y3} \end{Bmatrix}$$

A suitable displacement function is chosen to define the displacement at any point in the element. This is simply represented by two linear polynomials functions containing six unknown coefficients $(\alpha_1, \alpha_2 \cdots \alpha_6)$ representing the six degrees of freedom in the case of a plane triangular element.

$$\left.\begin{aligned} u &= \alpha_1 + \alpha_2 x + \alpha_3 y \\ v &= \alpha_4 + \alpha_5 x + \alpha_6 y \end{aligned}\right\} \qquad (3)$$

Hence the triangular element stiffness matrix [K^e] is represented by

$$\{K^e\} = \left|[B]^T [D][B]\Delta t\right| \qquad (4)$$

Where matrix [B] constraints constant linear dimensional values, Δ represents area of triangle and t represents the thickness of the triangular elements. It is simpler in practice to perform the matrix multiplications of equation (4) numerically in the computer.

To determine the element stresses from the element nodal displacements, the relationship below is considered where

$$\{\sigma(x,y)\} = [D]\,[B]\,\{\delta^e\} \qquad (5)$$

Where the stress-displacement matrix [H] equates to the product of matrix [D] and [B]
[H] = [D] [B]

$$\{\sigma(x,y)\} = [H]\ \{d^e\} \qquad (6)$$

where σ is the component of normal stress (σ_x, σ_y) and shear stress (τ_{xy}).

4. Structural Model

The structural model for this investigation will consist of a single-bay single-storey masonry infilled reinforced concrete frame shown in Figure 3 with a 30kN horizontal load acting of the top corner of the structure. A typical triangular mashed micro model for the structure is also shown in Figure 4.

In order to use a suitable mechanical property for masonry, values had been obtained through experimental work conducted by the author during doctorate research work, see Table 1

Table 1: Material elastic properties

Material	Moduli of elasticity		Poisson's ratio	
	E_x (kN/m^2)	E_y (kN/m^2)	V_{xy}	V_{yx}
Concrete	2.9 x 10^7	2.9 x 10^7	0.20	0.20
Masonry	4.4 x 10^6	7.41 x 10^6	0.22	0.33

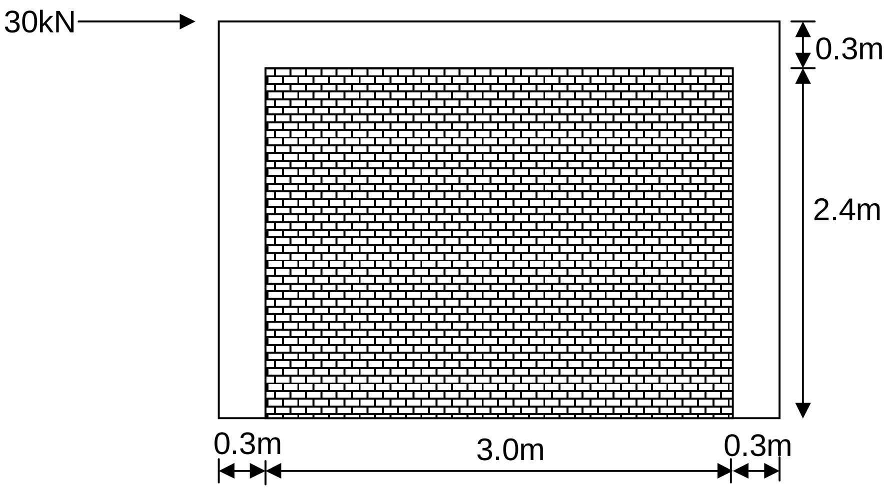

Figure 3: Infilled reinforced concrete frame structure under the action of a static horizontal load

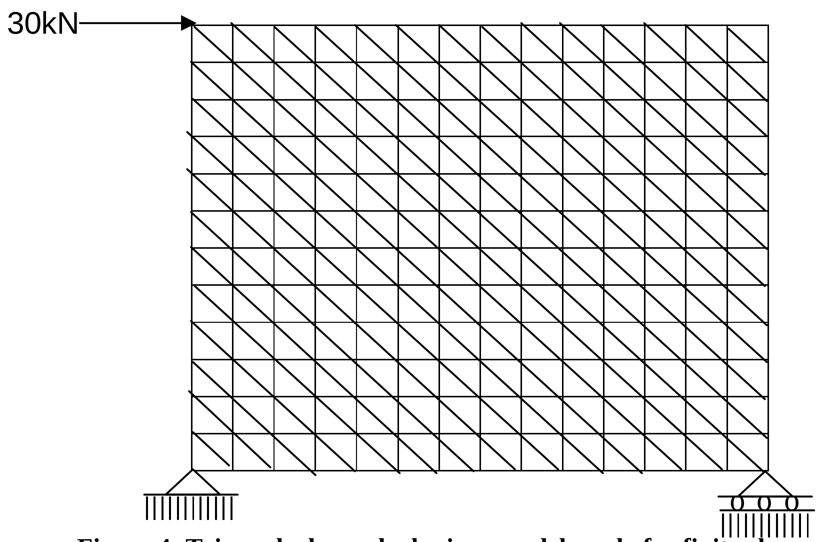

Figure 4: Triangularly meshed micro-model ready for finite element analysis

5. Computer Program Formulation

In other to implement the finite element method, a computer programme for two dimensional finite element analysis developed by the author would be used. The computer program is divided into two parts (subroutines). The first part consists of the routines for the control numbers and data input modulus, the second part consists of routines for tabulated nodal displacements and element stresses. The basic steps to obtain the element stiffness matrix $[K^e]$ and stress matrix [H] have already been discussed in details and would involve voluminous numerical work, hence this processes were well built up in the subroutines to take care of the overall analysis.

The input data consists of specifying the geometry of the idealized structure, its mechanical properties, the loading and the support condition. The data also includes certain control numbers that would help the efficiency of the program such as the total number of nodes and elements.

RESULTS AND DISCUSSION

The complex state of stress induced in the structural model subjected to shear loads was studied by finite element analysis described in this work. The state of displacement at any point of the structure in two directions δ_x and δ_y at any point of the structure is obtained. Also the components of stresses σ_x, σ_y and τ_{xy} was also obtained, with particular interest on the maximum shearing stress. A study of the distribution of these stresses reveals that the state of stress induced has a controlling influence on the failure of the infilled frame structure. More information about the failure criterion of the structure can be obtained by the loading history of the structure. Hence, to investigate the shear response of the structure, the shear stress path for any point on the structure is studied by observing the shear stress τ_{xy} against the normal stress σ_x from zero shear load upto a reasonable failure load obtained by the finite element elastic analysis on the solid infilled frame structure.

The study of the distribution of these stresses for the rigid frame reveals that the state of stress induced has an effect on the failure criteria of the infilled panel. The distribution of the normal and shear stresses from the result outputs, are seen to be in good agreement with previous experimental results by (Mehrahi et al 1996) and previous analytical works by (Galanti et al 1998). The results indicate higher values of stresses in zones around the compressed diagonal and the loaded corners. Hence two modes of infill failure are readily observed from the results. The first failure mode is observed as higher stresses that will initiate cracks extending from the center of the infill along the diagonal towards the loaded corners, while the second failure mode occurs at the loaded corners and the extent of the crushed region limited to the length of contact length.

Also, stress path obtained from finite element analytical modeling of the structural model can be compared against the stress path obtained from results of experimental data obtained by (Galanti et al 1998).
The maximum value for shear stress τ_{max} for the different models considered can be obtained from the result of finite element analysis. These results also show that the shear stress τ_{xy} is a function of factors such as the applied lateral force, the dimensions and the elastic properties of the structure. Hence, more general formula for shear strength and hence ultimate shear load will consist of one which includes such variables.

CONCLUSION

This work presents a simple method of micro modeling of the complicated behaviour of infilled frames under lateral loads. Using the finite element analytical technique supported by useful computer programme software developed by the author the behaiovur of single-storey single-bay to multi-storey single-bay masonry infilled frames under lateral loads has been investigated.

The proposed finite element analytical technique is easier and very practical to apply and requires much less computational time than other techniques based on discretizing the infill panel as a series of plane stress elements interconnected by a series of springs or contact elements.

From the forgoing a study of the shear response of brick masonry infill in a framed structure subjected to in-plane lateral load has proved that there exist two modes of masonry infill failure. Higher stresses initiating from the centre of the infill and proceeds towards the loaded corner in a diagonal pattern constitutes the first failure model while the second failure mode is seen as higher stresses at the loaded corners and noticed to be closely limited to the size of the contact length.

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