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ANALYSIS OF MASONRY INFILLED FRAME STRUCTURES

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ABSTRACT

Steel and reinforced concrete frame buildings often incorporate masonry wall panel infills within the frame. Studies conducted in the past have provided ample evidence of the beneficial contribution of these panels to the stiffness and strength of the overall system. However, as they exist today, these panels are used primarily as partitions to separate spaces within the building or as cladding to complete the building envelope. Very little attention is given to the structural usefulness of these panels. This is partly due to a lack of design tools and the lack of a universally acceptable theory for the analysis and design of these systems. Design aids for masonry infilled steel or reinforced concrete frame systems are generally not available. This paper presents an analytical technique that may overcome the above shortcoming. In the proposed technique, a general three-dimensional multi-storey, multi-bay building is analysed as an equivalent braced frame structure whereby the infills are replaced by a pair of diagonal springs. The stiffness and strength characteristics of the diagonal springs may be obtained by tests or derived analytically using a more elaborate finite element method. Also included in this paper is the description of a finite element technique for the analysis of masonry infilled panels developed specifically for this study. The correctness of this analytical technique is evaluated by comparison of analytical predictions with available test results reported in the literature.

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INTRODUCTION

Despite the amount of information available, structural designers are still reluctant to include infills in a frame structure as load resisting elements. The work presented herein describes a general analytical procedure for the analysis of frames with masonry infills. The analytical model accounts for elastic - plastic behaviour of frame members and non-linear behaviour of infills. To enable an economical and reasonably accurate analysis of large infilled frame structures, simple beam elements are used to model frame members and infills are modelled using equivalent springs. All frame members are assumed to be perfectly elastic up to the load level where plastic hinges start to develop and remain plastic thereafter. The stiffnesses and strengths of diagonal springs used to model the infills are based on load-deformation curves of diagonally loaded infills confined within frames of similar dimensions and stiffnesses. These load - deformation curves may be obtained from actual test results or they may be generated using a more elaborate finite element analysis. The use of a simple diagonal spring with a load - deformation behaviour that accounts for interaction of frame and infill allows a large, practical structure to be analysed using computing facilities normally available in a typical consulting office. While the program described herein is calibrated using test results of plane frames with masonry infills, it is also capable of analysing general three-dimensional infilled frame structures.

ANALYTICAL MODEL FOR THE EVALUATION OF INFILLED FRAME BEHAVIOUR

The generation of a load - deformation curve for an equivalent diagonal spring used to replace an infill involves a suitable finite element technique which must consider:

1. interaction between frame and infill, including the effects of initial lack of fit, gaps between frame and infill, interface bond and friction, and separation and re-contacting at the frame - to - infill interface;
2. non-linear behaviour of infill resulting from the occurrence of cracks due to shear and tension, and crushing of the infill material possibly under the condition of biaxial compressive stress;
3. non-linear behaviour of surrounding frame members and the formation of plastic hinges due to a critical combination of axial load, shear, and moment in the member section.

Fig. 1 shows a model used to generate load - deformation curves that can satisfactorily account for the behaviour of infill. As shown, a masonry panel in this study is treated as a series of elastic blocks linked together by a system of springs. The elastic blocks are assumed to be linearly elastic up to failure while the springs are introduced to handle tensile and shear stress failure in mortar joints. Standard plane stress rectangular elements having a homogeneous property which accounts for effects of unit geometry and mortar joints were used to model the elastic blocks. As shown in Fig. 2, the aggregate of all linkage elements at a location, identified as a joint element in this study, consists of four nodes with ten springs of zero physical dimensions. Springs 1 to 8 ensure that the nodes of wall elements connected by the joint move in unison when load is applied. Arbitrarily high values are assigned to the stiffnesses of these springs. When joint failure occurs in the form of tensile or shear cracking along mortar joints, the stiffness of one or more springs is reduced to zero to reflect the corresponding failure. Although not absolutely necessary, Springs 9 and 10 are introduced and assigned a small nominal, non-zero value to avoid numerical difficulty during analysis when the stiffnesses of other links are reduced to zero.

Standard plane frame line elements located along centrelines of members are used to model beams and columns. A typical plane frame element has three degrees of freedom at each node and has moment, shear and axial load capacity. The frame element is assumed to be linearly elastic and all inelastic behaviour is assumed to be concentrated at nonlinear hinges located at the ends of the member. Hinge elements as shown in Fig. 3 are introduced at the ends of frame members to model non-linear behaviour as described above. Referring to Fig. 3, k_n, k_s, and k_r are stiffnesses of the normal, tangential, and rotational springs, respectively. Initially, these are assigned a large arbitrary value to ensure that the two points connected by the hinge element deform in unison with no relative displacements or rotation. When end forces of the frame member, to which a hinge element is attached, reach their peak values, a small nominal value is assigned to k_n, k_s, or k_r depending on the type of failure. For example, if the plastic moment capacity of the frame member is reached, k_r is assigned a small value so that the hinge element allows the frame to rotate with very little increase in load. A pair of equal but opposite moments is applied at the two end nodes of the hinge element to account for the plastic moment sustained by the frame member. A similar technique is used to account for shear and axial load failure by reducing the stiffness k_n and k_s, respectively.

An interface element consisting of a pair of normal and tangential springs is used to model the conditions of the frame - to - infill interface. In this study, the normal spring is assumed to have infinite compressive capacity with a tensile capacity depending on the adhesive bond between frame and infill. Crushing of the infill is handled by reducing the elastic constants of plane stress elements used to model the elastic blocks. A high stiffness value is assigned to the normal spring when the frame is in bearing contact with the infill. When tension capacity is exceeded, separation occurs and the stiffnesses of both the normal and tangential springs are reduced to zero to allow the frame and infill to move independently. The strength of the tangential spring depends on the shear bond and friction that exist in the interface and its stiffness is approximated incrementally as follows :

$$k_s^{i+1} = \frac{\mu F_n^i}{\Delta_s^i} \tag{1}$$

where k_s^{i+1} is the stiffness of the shear spring to be used in the next iteration of computation, μ is the coefficient of friction of the joint, F_n^i is the force in the normal spring, and Δ_s^i is the relative shear displacement of nodes tied by the spring under consideration. This permits the wall to slip when the shear force in the interface exceeds the shear capacity of the frame - to - panel interface. If the panel is in contact with the frame and the shear bond of the interface is not exceeded, a high stiffness value is assigned to the tangential spring.

MATERIAL MODEL

Material models as described in this section are used to describe constitutive relationship for frame and interface elements. The beams and columns of the frame are assumed to have a tri-linear load deformation behaviour as shown in Fig. 4. In this figure, the ordinates can be end moment, shear, axial tension, or compression while the abscissa is the corresponding associated deformation. The masonry panel model used in the present study is assumed to be homogeneous and linearly elastic up to failure. Additionally, the material is assumed to be orthotropic in directions parallel and normal to bed joints. The assumption of linear elastic behaviour is based on experimental evidence available in the literature (Fattal and Cattaneo 1976; Drysdale and Hamid 1979) which tends to confirm that masonry behaves linearly almost up to failure. The failure criteria for masonry as proposed by Lourenço (1996) is adopted in this study. The primary reason for this is that these criteria can be readily adapted for other masonry materials and types of construction. Generally, there is a wide regional variation in the geometry of masonry units, manufacturing materials used, and construction techniques throughout the world. It is hoped that the use of these failure criteria will make the analytical technique proposed herein readily adaptable by others.

ANALYTICAL PROCEDURE

In the present study, it is desirable to obtain the entire range of load - deformation behaviour of infilled frames loaded to ultimate failure. Generally, such curves would include a rising and falling branch, and a plateau which indicates the plastic strength and ductility of the structure, if any. This curve may also contain one or more intermediate load drops associated with localized failures. To obtain the load - deformation curve of an infilled frame system loaded to failure, successively increasing loads are applied at a pre-selected node. At each load step, stresses in the structure are examined and checked for failure using appropriate failure criteria. If failure is detected, the stiffness of the structure is modified to reflect the change caused by the failure and the analysis is repeated until no new failure is detected. At this stage, the load is in equilibrium and the deflection of the structure recorded. An incremented load is then applied and the process repeated to obtain the next pair of load - deflection coordinates. When the load reaches its peak value, a further increase cannot be applied because the state of equilibrium cannot be reached at a higher load level. In order to overcome this difficulty so that the descending portion of the curve can be obtained, an augmented structure as shown in Fig. 5 is used. Fig. 6 gives a graphic illustration of the iterating procedure for the augmented structure adopted herein. In the first iteration, the stiffness matrix corresponding to the undeformed structure is used. Stresses in the elements of the structure are then computed and checked for failure. If required, the structural stiffness is re-evaluated reflecting any failure of elements and new displacements and stresses are computed. This process is repeated until no further change in structural stiffness is encountered. Graphically, this process is shown as progressing from a to b and eventually to c in Fig. 6. At point c, the structure is in equilibrium with the externally applied load and the force on the infilled frame is then computed by taking the difference between the total applied load, F, and the force in Spring A, labeled as P, in Fig. 5. The applied load may then be increased by a predetermined increment and the process repeated. Stiffness values computed in the preceding iteration is used to initiate the computations for load step (i+1). Using this technique, the entire load - deflection curve can be generated.

COMPARISON OF ANALYTICAL AND EXPERIMENTAL RESULTS

Fig. 7 shows a comparison of a load - deformation curve generated using the above procedure and a corresponding curve obtained experimentally for a concrete masonry infilled steel frame specimen (Richardson 1986). A similar comparison of analytical and experimental results for a reinforced concrete frame with clay brick infill is shown in Fig. 8. Reasonable prediction of peak load and post peak behaviour are obtained in both cases. Although Figures 7 and 8 show the load - deformation curves of infilled frame specimens under horizontal racking load, the technique described herein can also be used to generate load - deformation curves of diagonally loaded infilled panels.

ANALYSIS OF GENERAL FRAMED STRUCTURES WITH MASONRY INFILLS

Fig. 9 shows a model that can be used for the analysis of a general three-dimensional framed structure with masonry infills. Standard beam elements with six degrees of freedom per node are used for frame members. Infills are replaced with equivalent diagonal springs which are activated only when the diagonals are in compression. Stiffnesses of the diagonal springs are based on load - deformation curves of infills that can be obtained using procedures described in the preceding section or determined experimentally. A typical curve as shown in Fig. 10 is based on test results by Dukuze (1998) for a diagonally loaded infill panel confined within a frame. Also shown in Fig. 10 is the corresponding curve determined analytically using the technique presented in this paper. It is believed that the curve accounts for the effects of infill-frame interaction and therefore the diagonal spring can replace the infill in the analysis of a large multi-panel structure.

A computer program called EPIFRAME was developed for the analysis of general three-dimensional framed structures with infills as described above. A combined incremental and iterative technique similar to that described previously was adopted in this program. As mentioned earlier, diagonal infill springs are activated only when in compression and their stiffnesses and strengths are based on their load - deformation curves. The program checks for bending forces in frame members at the end of each load increment and the stiffness matrix of the structure is modified, if required, during the iterative computations.

EXPERIMENTAL PROGRAM

Racking tests on one-third scale three-bay, three-storey reinforced concrete frames with brick masonry infills were conducted by Dukuze (1995, 1998). Fig. 11 shows the overall dimensions of the test specimen and the total shear load distributed to the top, second, and first storey levels in the ratio of $\frac{1}{2}:\frac{1}{3}:\frac{1}{6}$. A summary of the cross-sectional dimensions of the frame members is shown in Table 1.

Comparison of predicted and experimental load deformation behaviour of two three-storey, three-bay frame specimens tested by Dukuze (1998) are shown Figures 12 and 13. As evident in Fig. 12, EPIFRAME correctly predicts the ultimate load of Specimen S331 (See Table 1). However, the predicted initial stiffness is greater than that obtained experimentally. The lower stiffness obtained experimentally can be partly attributed to the fact that Specimen S331 was subjected to repeated loading in initial attempts to test the specimen (Dukuze 1995) and cracking had occurred in some panels prior to the final test. EPIFRAME gives reasonable prediction of the initial stiffness and ultimate load of Specimen S335. In both specimens, the load - deformation curves of equivalent diagonal springs used to replace the infill were obtained analytically and are similar to that shown in Fig. 10.

CONCLUSIONS

Comparisons between predicted and experimental data were made for reinforced concrete frames with brick panel infills and steel frames with concrete masonry infills. Satisfactory correlation between experimental results and analytical predictions were obtained. This work was then extended to obtain the load-deformation behaviour of diagonally loaded infill panels to provide information for the analysis of more general multi-storey, multi-bay infilled frames. The computer program developed for this purpose accounts for the elastic - plastic behaviour of frame members and non-linear load - deformation characteristics of the infill. The infills in the computer model are replaced by non-linear diagonal springs while standard beam elements are used for frame members. Findings indicate satisfactory correlation between experimental results and predicted behaviour. Preliminary results indicate that this technique is suited for the analysis of three-dimensional frames with masonry infills.

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Canadian Standards Association, 1994. CSA A23.3 -94 : Design of concrete structures, Canadian Standards Association, Rexdale, Ontario.

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Dukuze, A, and Dawe, J.L. 1995. In-plane behaviour of three-storey three-bay RC frames with URM panels, Proceedings, 4th Australasian Masonry Conference, pp. 208-217.

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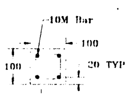
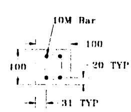
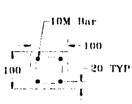
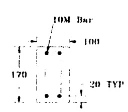
Fattal S.G. and Cattaneo, L.E. 1976. Structutal performance of masonry walls under compression and flexure, National Bureau of Standards Building Science Series 73. National Bureau of Standards, Washington D.C. 57 pp.

Lourenço, P. B. 1996. Computational strategies for masonry structures. Delft University Press, Thesis, Delft University of Technology. The Netherlands.

Richardson, J. 1986. The behaviour of masonry infilled steel frames. M.Sc. thesis, Department of Civil Engineering, University of New Brunswick, Fredericton, N.B.

Table 1: Test Specimens: Three-Storey, Three-Bay Frame (Dukuze 1998)

	Column	Beam	Infill dimensions W (mm) x H(mm)	Ultimate Load Predicted / Experimental
S331	 M_p = 5.4 kN-m	 M_p = 5.4 kN-m	900 x 900	0.97
S335	 M_p = 5.4 kN-m	 M_p = 11.1 kN-m	900 x 900	0.92

Note: M_p = Moment capacity based on CSA 23.3-94 (Canadian Standards Association 1994). f'_c = 20 MPa, f_y = 400 MPa

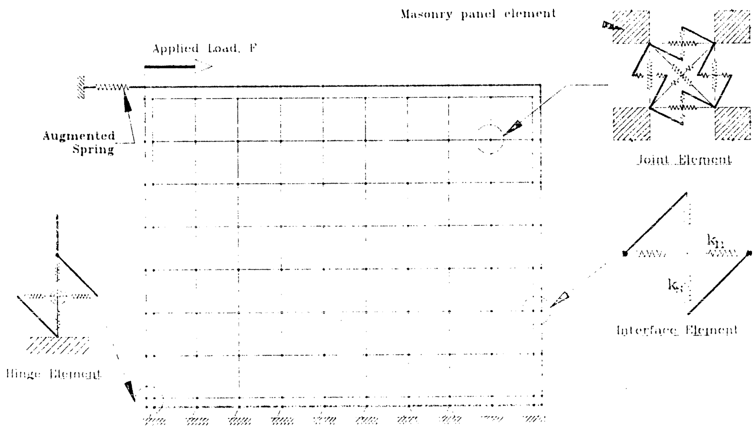


Figure 1: Infill Frame Model

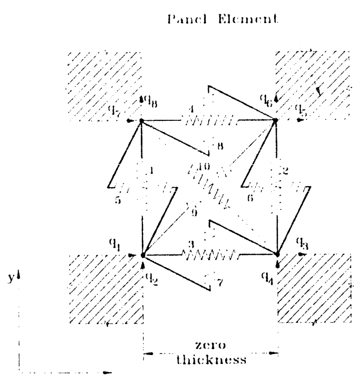


Figure 2: Joint Element

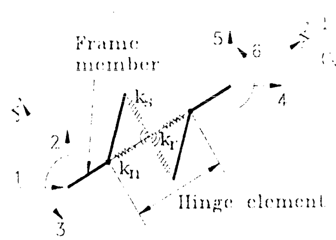


Figure 3: Hinge Element

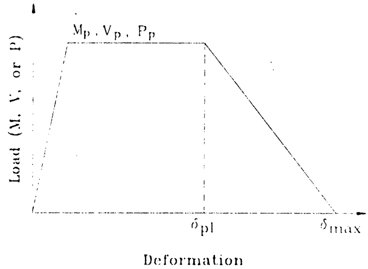


Figure 4: Load Deformation Behaviour of Frame Member

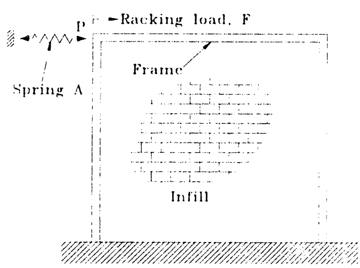


Figure 5: Augmented Structure

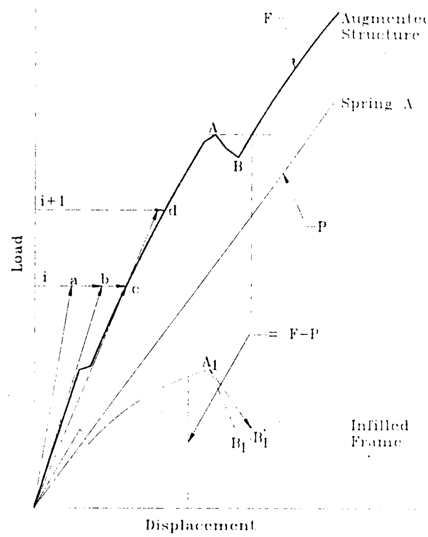


Figure 6: Iterative Procedure

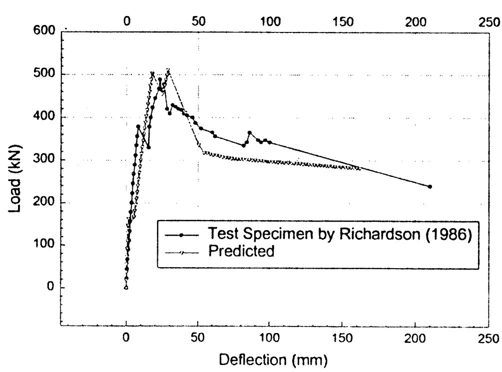


Figure 7: Comparison of Experimental and Predicted Behaviour - Concrete Masonry Infilled Steel Frame

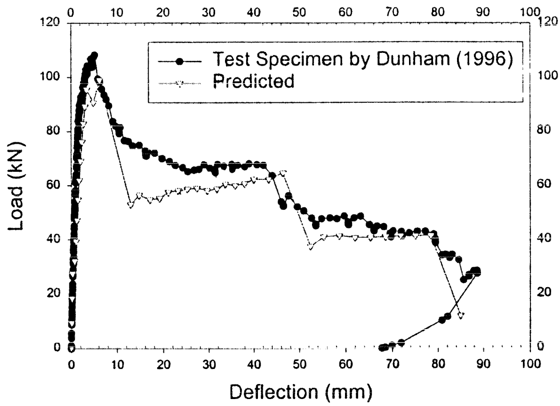


Figure 8: Comparison of Experimental and Predicted Behaviour - Reinforced Concrete Frame with Brick Masonry Infill

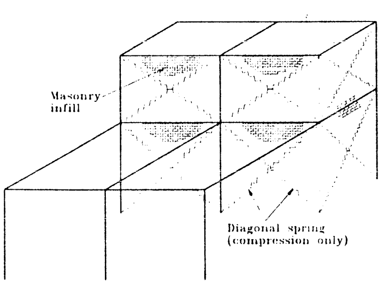


Figure 9: Diagonal Spring Model

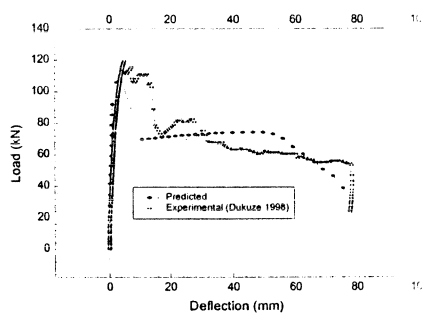


Figure 10: Load-Deformation Curve of Typical Infilled Panel

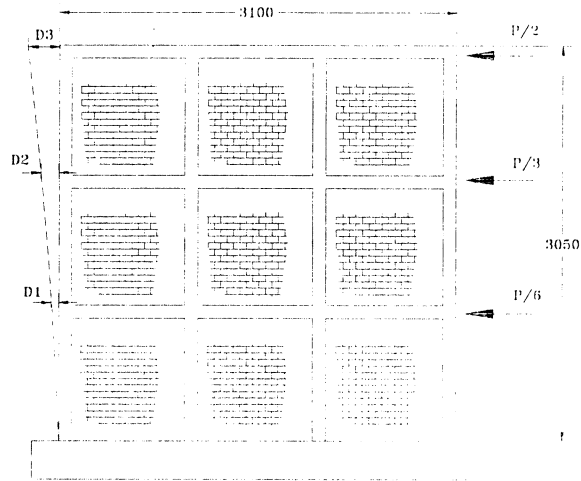
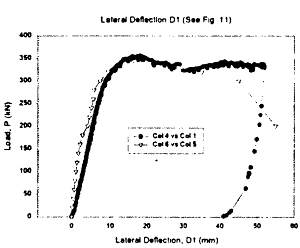
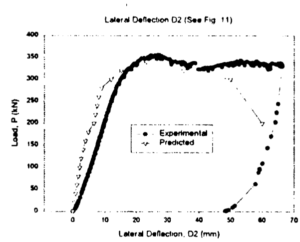


Figure 11: Three-Storey, Three-Bay Infilled Frame Specimen





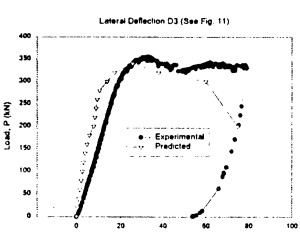
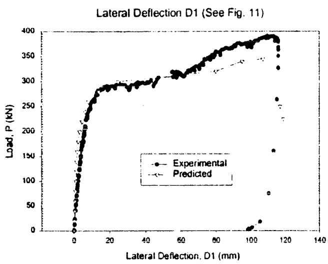
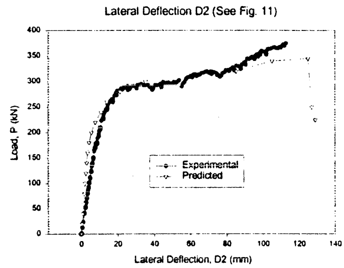


Figure 12: Comparison of Experimental and Predicted Results for Specimen S331





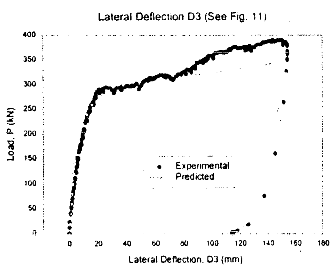


Figure13: Comparison of Experimental and Predicted Results for Specimen S335