AN INVESTIGATION OF FACTORS INFLUENCING THE BEHAVIOUR OF MASONRY INFILL IN STEEL FRAMES SUBJECTED TO IN-PLANE SHEAR

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ABSTRACT An investigation of various factors which may affect shear strength and behaviour of masonry infilled steel frames is presented. Findings of this study are believed to be equally applicable to panels constructed of burnt clay masonry units as well as to panels constructed of concrete masonry units. Factors such as openings in panels, bond and friction between panel and frame, provision of airspace between roof beam and panel, and column-to-panel ties are investigated using large-scale test specimens. A computer-aided analytical technique combining a frame stiffness analysis with a finite element panel analysis is used to determine stress distributions in the panels in the pre-cracking range.

1. INTRODUCTION

The beneficial aspects of masonry in providing a load-sharing structural system in concrete and steel frame structures have become increasingly apparent since about the early fifties. Generally, much research has been done in the area of burnt clay masonry and concrete masonry units with regard to their structural strength, mechanical properties, and overall behaviour in engineered masonry structures. Many researchers have contributed widely to theoretical and experimental understanding of the general problem of shear behaviour of infilled frames. Some of this work relating directly to masonry infilled frames is briefly summarized elsewhere in these proceedings (1). In the case of masonry infilled steel frames, most engineers are aware that masonry infilled panels in steel frames provide a large reserve of resistance to in-plane shear loads and that, in return, the steel frame provides considerable ductility to the system. The purpose of the investigation presented herein is to add, both analytically and experimentally, to the knowledge and understanding of the strength and behaviour of masonry infilled steel frames subjected to in-plane loading. In particular, effects of joint reinforcement, mortar strength, panel openings, and interface conditions between panel and frame, are investigated. The analytical procedure consists of a frame stiffness analysis combined with a finite element analysis of the masonry infill with simple 2-node elements at the interface to account for friction, ties, and compressive contact stresses. The results of twelve tests on large-scale masonry infilled steel frame specimens are presented, discussed, and compared with theoretical observations where valid comparisons can be made.

2. ANALYTICAL TECHNIQUE

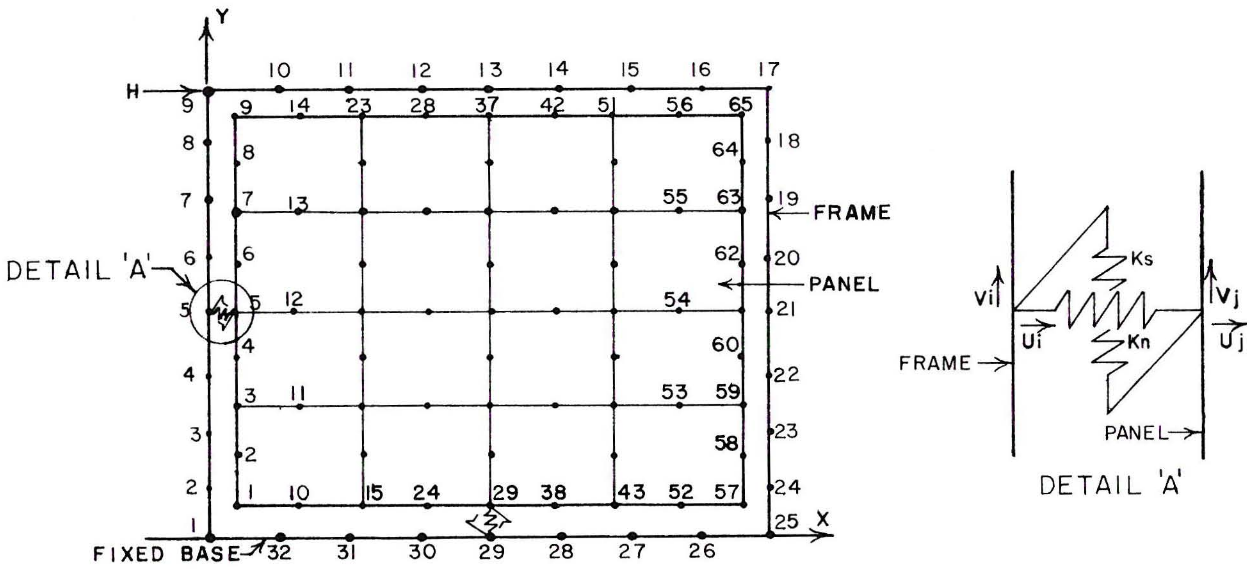
2.1 Introduction

Large-scale specimens considered in this investigation consist of simple rigid portal steel frames with infilled masonry panels. The analytical technique, however, is generally applicable to multi-storey, multi-bay planar infilled frames and would be limited only by computer time and cost considerations. The analysis includes the effects of horizontal joint reinforcement, pinned or rigid frame connections, panel-to-frame ties, friction between panel and frame, and provision of an airspace between roof beam and

panel. An iterative technique is used to determine the interaction between panel and frame with regard to friction and separation. Up to the occurrence of first major cracking in the large-scale specimens tested, experimental observations appear to confirm results predicted theoretically. It is evident that the analytical technique may be reliably used to evaluate stiffness as well as stress distributions in a panel and frame prior to initial major cracking.

2.2 Frame Analysis

The frame is analysed by a direct stiffness technique using frame elements of two nodes at which axial, transverse, and rotational displacements are defined. As shown in figure 1(a), frame element nodes are defined to correspond



(a) FINITE ELEMENT DISCRETIZATION

(b) INTERFACING ELEMENT

Figure 1. Finite Element Model of Frame and Panel.

to nodes on the panel periphery. Following the usual procedure, stiffness matrices for the frame elements are assembled directly into the frame stiffness matrix. Hinges may also be inserted at the nodes and general in-plane member loads can be accommodated.

2.3 Panel Analysis

Figure 1(a) shows a finite element mesh for a panel which is discretized into 8-node rectangular elements with two degrees of freedom at each node. The 16 x 16 stiffness matrix for each element is determined and assembled directly

into the panel stiffness matrix. Because of the size of stiffness matrix of each element (256 numbers per matrix) a mesh size analysis was carried out to determine the optimum number of elements to use for the size of large-scale specimens tested. It was determined that 16 rectangular elements gave accurate results as well as economy of computer costs for the iterative procedure required.

2.3.1 Reinforcement. The effects of horizontal reinforcement in the panel are also included in the analysis. It is assumed that reinforcement is integrally bonded to masonry units at the nodes of rectangular panel elements through which the reinforcement passes. Two-node line elements are used to model the reinforcement between corresponding rectangular element nodes. The 2 x 2 stiffness matrix of a 2-node reinforcement element tangent to, or intersecting a rectangular element at its nodes, is added directly to the rectangular element stiffness matrix. The resulting stiffness matrix is then assembled into the structure stiffness matrix.

2.4 Panel - Frame Interaction

At the time of assembly, the panel and frame stiffness matrices are uncoupled. The interaction between a frame node, i, and a panel node, j, is modelled by horizontal and vertical 2-node line elements each of an arbitrarily negligible length and of stiffness k_n and k_s, respectively, as shown in figure 1(b). The horizontal element is used to model bond forces as well as the effects of ties between the masonry and steel at corresponding nodes, while the vertical element is used to model friction between the panel and frame or shear in ties.

2.5 Analytical Procedure

Although more general loads may be considered, loading for the analysis of specimens of the type described herein consists of a single horizontal force applied at node number 9 of the frame as shown in figure 1(a). Interaction of the frame and panel are realized in the computer model by use of the interface elements described previously. During the first step of the analytical procedure, all interfacing friction elements are assumed to have zero stiffnesses while high nominal stiffnesses are assigned to all interfacing normal elements. The horizontal and vertical displacements of nodes i and j in figure 1(b) are u_i and u_j and v_i and v_j, respectively. The friction spring stiffness, k_s, parallel to the interface, varies with the force in a normal element whose stiffness is k_n. After the first iteration, all nodal displacements are known and a normal force between frame and panel nodes, i and j, is given by,

$$F_n = k_n(u_j - u_i) \qquad \text{——— (1)}$$

where a negative result indicates compression. If F_n is greater than the tension bond force between the mortar and frame, k_n is set equal to zero for subsequent iterations unless the panel and frame again come into bearing contact in which case k_n is replaced by its original nominal value. If F_n is less than the tension bond force (or less than zero if the bond has been previously broken), k_n is given its previous nominal value for the next iteration. If F_n is less than zero, the friction element is given a stiffness value of,

$$k_s = \frac{\mu\, k_n\, |(u_j - u_i)|}{|(v_j - v_i)|}\ , \qquad |(v_j - v_i)| > 0 \qquad \text{——— (2)}$$

for the next iteration where μ is the coefficient of friction between the masonry and the steel frame. In any iteration, if tension develops at a node or if $|(v_j - v_i)| = 0$, k_s is given a value of zero for the next iteration. If a tie exists at a node, k_s and k_n are given high nominal values and shear and tension forces at any iteration are checked against the breaking strength of the tie. If the tie breaks at any iteration the corresponding nodes are subsequently treated as untied. The iterative technique is continued until there are negligible differences between two successive iterations.

3. EXPERIMENTAL INVESTIGATION

3.1 Introduction

Experimental results for 12 large-scale masonry infilled steel frame specimens are presented and discussed and compared with theoretical findings where applicable. Overall dimensions for the masonry panels of all large-scale specimens were 3 600 mm long by 2 800 mm high. The panels consisted of 200 x 200 x 400 concrete masonry blocks placed in running bond within a surrounding rigid steel frame consisting of W250 x 58 columns and a W200 x 46 roof beam. Further overall specimen characteristics as well as a description of testing apparatus, instrumentation,and testing procedure are available elsewhere (1,2).

3.2 Specimen Descriptions

Twelve large-scale specimens were tested during this investigation. Specimen characteristics were varied to determine the effects of mortar strength, panel-to-column ties, friction between panel and frame, panel openings, and airspace between roof beam and wall. The details required to study these effects are outlined in Table 1 for each specimen. All panels had horizontal truss-type

TABLE 1 Specimen Details

Specimen Number	Description
1A	Mortar packed between column flanges and panel
2A,3A	Mortar omitted between column flanges and panel
4A	20 mm airspace between roof beam and panel
5A	Same as 4A but with flat bar panel-to-column ties
6A	Same as 2A & 3A but with flat bar panel-to-column ties
1B	Same as 2A & 3A but with polyethylene membrane between panel and frame to reduce friction and bond
2B	Same as 1B but with a purposely very weak mortar
3B & 4B	Opening at center of panel (see Figure 2)
5B	Opening offset towards load (see Figure 2)
6B	Opening offset away from load (see Figure 2)

joint reinforcement at alternate courses. Details relating to joint reinforcement and panel-to-column ties are available elsewhere in these proceedings (1). Panel opening details are shown in figure 2.

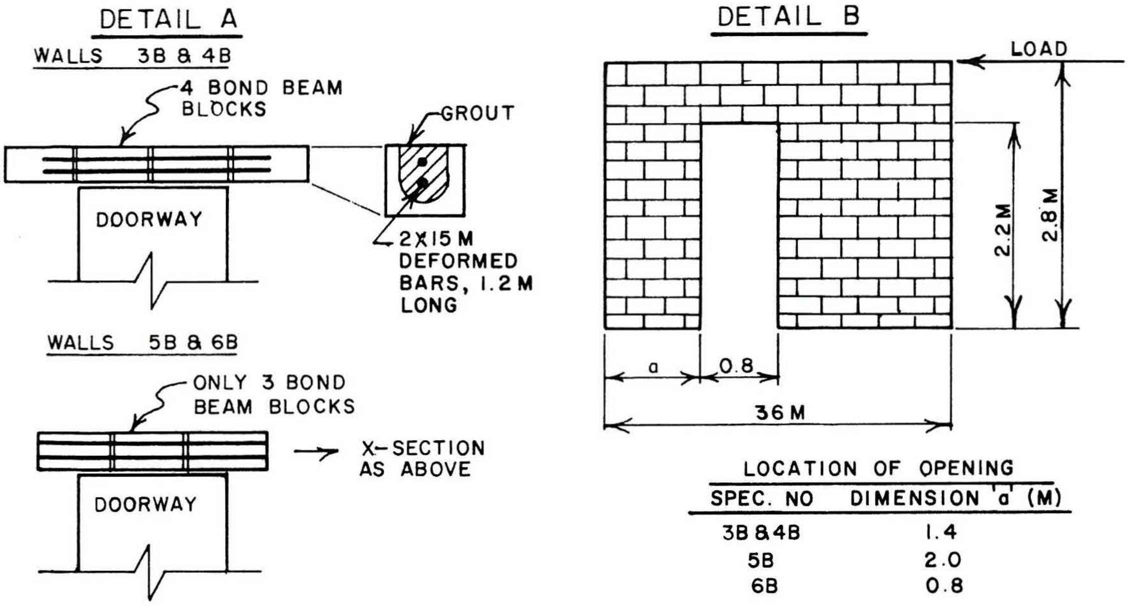


Figure 2. Panel Opening Details.

3.3 Experimental Results

Each specimen was mounted in a test frame and a horizontal load applied at roof level was gradually incremented in steps of 22.2 kN up to the ultimate capacity of the specimen. Recorded data included progressive cracking patterns, loads, deflections at various points along the frame, and slippage and separation between panel and frame. Loads at initial major cracking and ultimate as well as initial wall stiffness and mortar compressive strength are summarized in Table 2 for all specimens tested.

TABLE 2 Summary of Test Results

Specimen Number	Initial Major Crack load (kN)	Ultimate Load (kN)	Initial Stiffness ($\frac{kN}{mm}$)	Mortar Compressive Strength (MPa)
1A	245	449	71.9	20.2
2A	413	538	47.1	19.5
3A	307	556	43.8	17.7
4A	169	209	26.9	18.9
5A	200	231	34.4	24.9
6A	311	423	72.5	38.6
1B	270	420	40.9	24.4
2B	155	310	45.5	12.4
3B	55	285	34.2	17.5
4B	66	335	34.2	-
5B	90	245	38.5	16.5
6B	67	365	50.0	-

3.3.1 Load-Deflection Studies. Load-deflection curves for all 12 specimens are compared in Figure 3. In each case the horizontal load at roof level is plotted on the vertical axis and the horizontal deflection of the specimen at the point of load is plotted on the horizontal axis. Load-deflection curves for specimens 1A, 2A, and 6A are compared in figure 3(a). The panel of Specimen 6A was mechanically tied to the columns whereas the panel of

specimen 1A was tightly bonded by mortar between column flanges. The panel of specimen 2A on the other hand was free to rotate and slip relative to the frame. Consequently, as loading increased, it was able to assume optimum interaction with the deflected frame with regard to equivalent strut behaviour (3,4,5). Of the three, this panel had the highest initial major crack load and the highest ultimate load (table 2). (A major initial crack load was taken as the load at which a large visible crack, usually accompanied by a noticeable drop-off in load, occurred).

Specimen 4A and 5A each had an airspace along the underside of the roof beam. The panel of specimen 4A was not mechanically attached to the frame while specimen 5A had panel-to-column ties. Although the overall behaviour was similar for these specimens, as shown in figure 3(b), it appears that ties may have contributed to an increase in initial cracking load of about 18% but only a slight increase in ultimate load is evident. Curves shown in figure 3(c) are for Specimen 3A which had joint reinforcement and no ties, Specimen 1B which was similar but with a polyethylene membrane placed between the panel and frame, and Specimen 2B which was similar to 1B except that a purposely weak mortar was used. Comparing these curves, it appears that reducing friction and bond between panel and frame does not significantly reduce the cracking load although there is an indication that ultimate load may be significantly decreased (see table 2). Also it appears that a very weak mortar considerably reduces the stiffness beyond initial cracking as well as the ultimate strength.

Figure 3(d) shows the load-deflection curves for Specimens 3B to 6B which had openings as described in table 1 and figure 2. Specimens 3B and 4B each had a centrally-located door opening and did not experience a drop in load although first cracks appeared at 55 kN and 66 kN respectively. Specimens 5B and 6B had larger initial stiffnesses than panels with central openings. Specimen 5B, which had an opening offset from center towards the loaded side, had the lowest ultimate load, while Specimen 6B, which had the same size opening offset from center towards its unloaded side, had the highest ultimate load of the three. These observations are somewhat at variance with the findings of Mallick and Garg (6) who found that an opening remote from the loaded side of such a specimen tends to decrease its capacity. Figure 3(e) shows the effects of cycling Specimen 6 four times to ultimate. On the first loading, the ultimate load was 365 kN and after three additional cycles approximately 85% of this load was still attainable.

3.3.2 Crack Studies. Progressive crack patterns for Specimens 1A to 6A are shown in figure 4 while those for Specimens 1B to 6B are shown in figure 5. Comparing crack patterns of Specimens 2A and 3A, which had no mechanical panel-to-frame ties, with those of Specimen 1A, in which the panel was mortared snugly between column flanges, and Specimen 6A which had flat bar ties, it appears that panel-to-frame ties induce random off-diagonal cracking. More tightly banded, compressional diagonal crack patterns occur for panels that are not tied to the columns. Such panels would be expected to more closely approximate an equivalent strut type of behaviour and to have overall better behaviour in load-reversal situations. The crack pattern shown in figure 5(a) for Specimen 1B which was similar to Specimens 2A and 3A except for a polyethlene membrane between panel and frame to reduce bond and friction stresses, also clearly displays equivalent strut type of behaviour. Specimen 2B which had a weak mortar has more extensive off-diagonal random cracking than Specimen 1B which was otherwise similar.

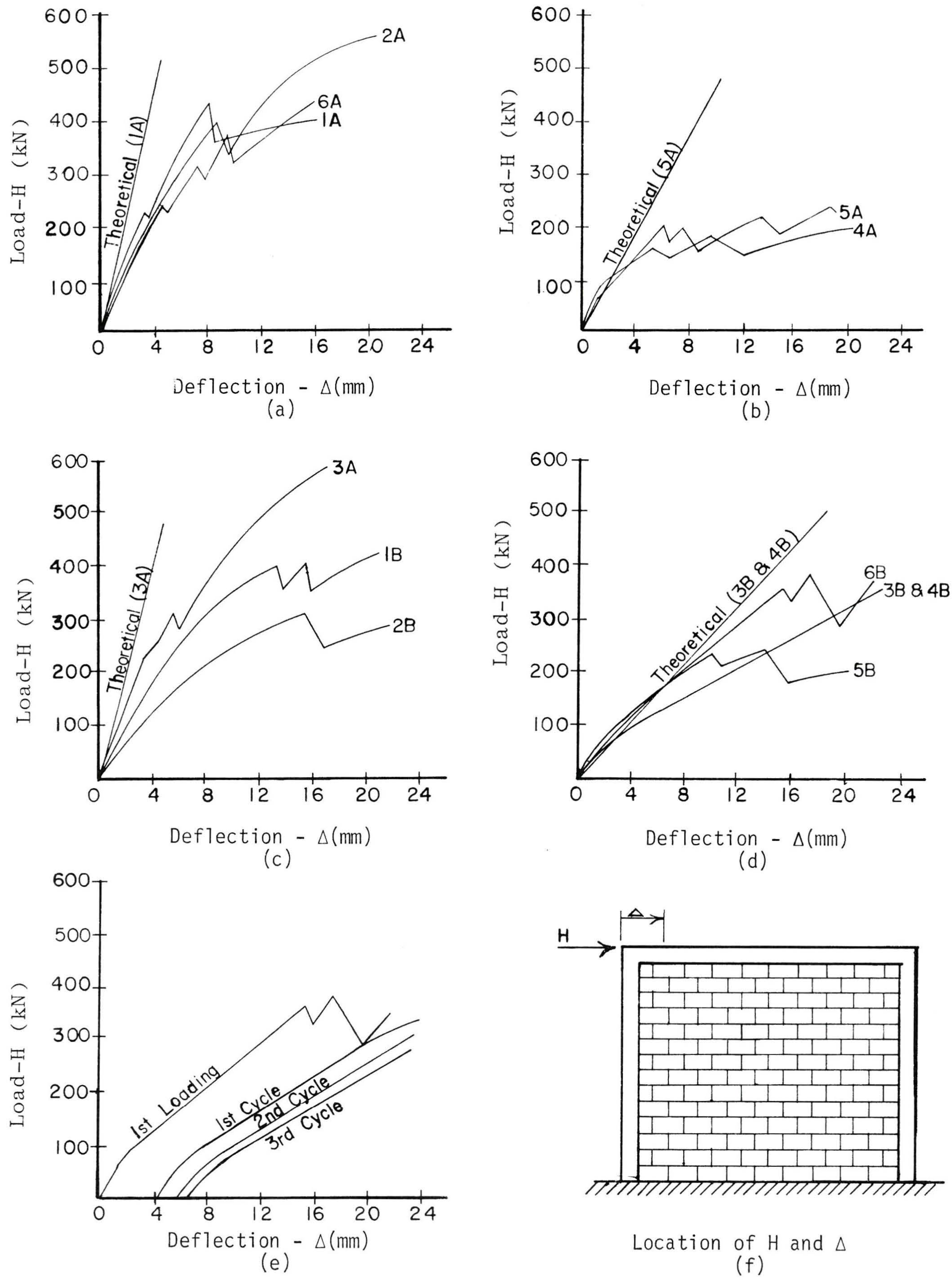
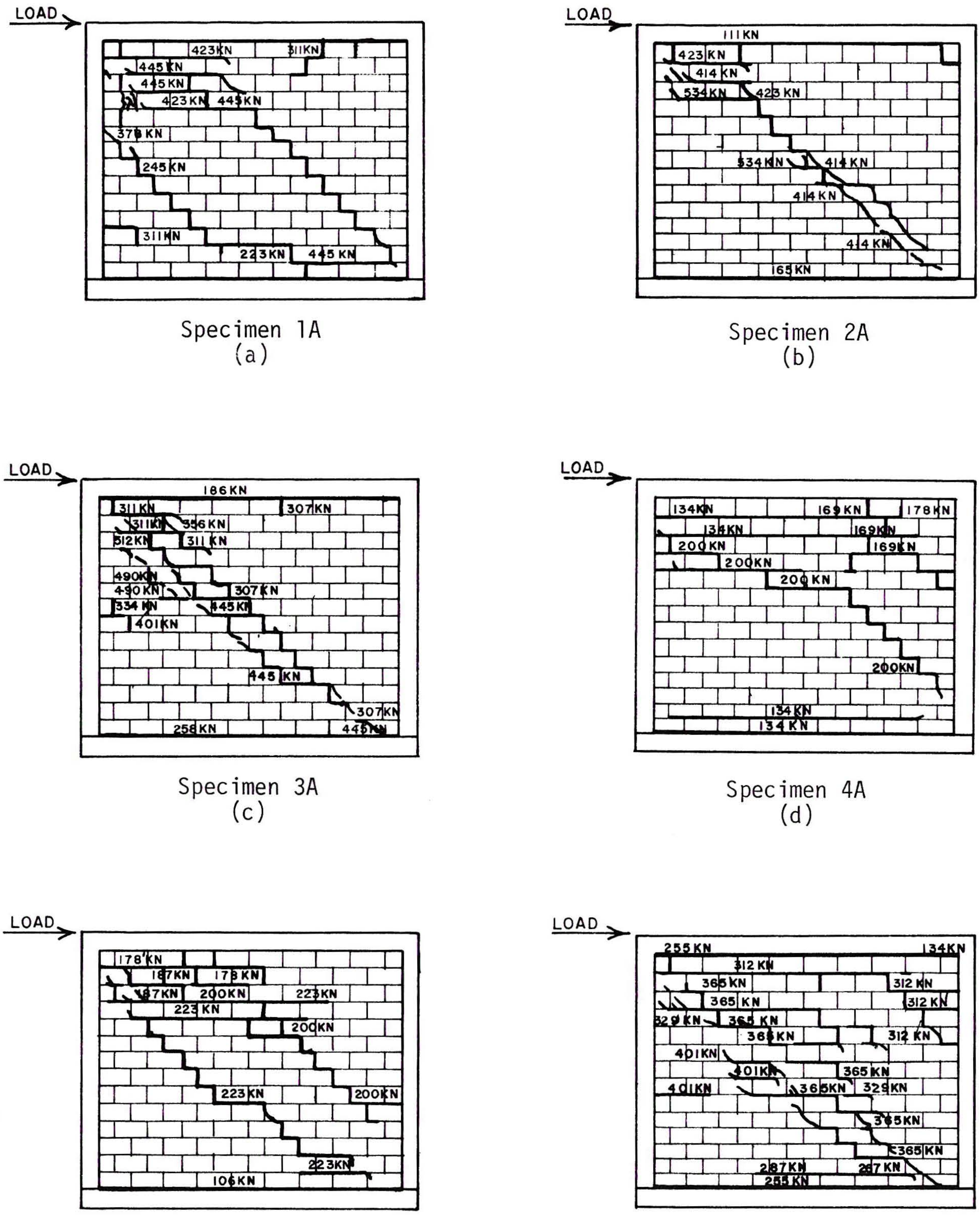


Figure 3. Load-Deflection Curves For Test Specimens.



Specimen 1A
(a)

Specimen 2A
(b)

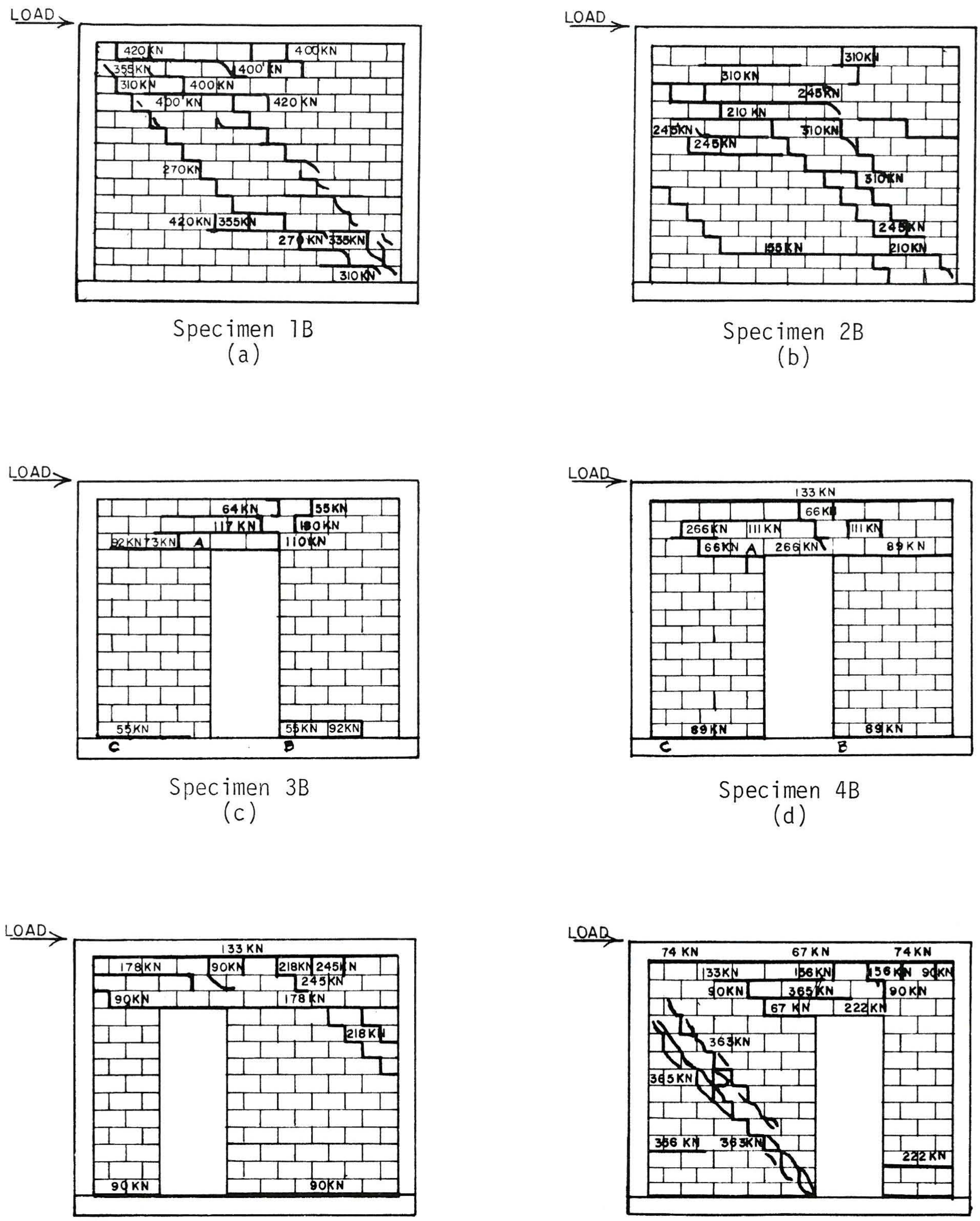
Specimen 3A
(c)

Specimen 4A
(d)

Specimen 5A
(e)

Specimen 6A
(f)

Figure 4. Crack Patterns for Series A Specimens.



Specimen 1B
(a)

Specimen 2B
(b)

Specimen 3B
(c)

Specimen 4B
(d)

Specimen 5B
(e)

Specimen 6B
(f)

Figure 5. Crack Patterns for Series B Specimens.

The effects on cracking of a roof beam airspace along the top of a panel are shown in figure 4(d). The majority of cracks occur along horizontal joints with only a slight suggestion of diagonal cracking on the side remote from the load. The effects on cracking of using ties in such a wall are shown in Figure 4(e) for Specimen 5A. The ties appear to reduce horizontal cracking through the joints while increasing diagonal cracking.

Crack patterns for Specimens 3B and 4B which had centrally-located doorway openings are shown in figure 5(c) and 5(d) respectively. Cracks occurred primarily through horizontal joints and, as loading increased, very large crack widths developed at points A, B, and C as noted. Figure 5(e) shows the crack pattern for Specimen 5B which had a doorway opening close to the loaded side of the specimen. Again, horizontal shear cracking through the mortar joints was quite evident although some diagonal cracking developed in the side remote from the load. In contrast, Specimen 6B, which had an opening remote from the load, developed extensive diagonal cracking in the panel between the opening and the load. This apparent diagonal strut action resulted in a much higher ultimate load than that obtained for Specimen 5B.

3.4 Comparison with Theory

3.4.1 Load-Deflection Curves. Theoretical comparisons are valid only in the elastic pre-cracking load ranges. Theoretical load-deflection curves in figure 3 are compared with corresponding curves obtained experimentally. Generally, theoretical curves agree with test curves in the lower, pre-cracking load ranges. As the loads increase and cracking occurs,theoretical and experimental curves deviate markedly. The theoretical method gives a reasonably accurate estimate of initial stiffnesses in most cases.

3.4.2 Stress-Contours. The stress contours presented herein are valid only up to initial cracking of the masonry panel. However, there appears to be some correlation between theoretical stress contours and observed crack patterns. A complete study of stress contours and crack patterns for several of the specimens tested as well as for many hypothetical cases is available elsewhere (7). As examples, figure 6 shows the stress contours for major principal stresses and maximum shear stresses for Specimens 2A, 6A, and 3B. The arrows in these figures show the directions normal to major principal stresses and parallel to maximum shear stresses,respectively. Specimens 2A and 6A were standard specimens with horizontal joint reinforcement and differed only in that Specimen 6A also had panel-to-column ties. The presence of these ties is evident by comparing stress contours for these specimens which were assumed to have a load of 220 kN applied at roof level. A comparison of the stress contours for Specimen 2A with the corresponding crack patterns shown in figure 4 reveals a correlation between high maximum shear stresses and horizontal cracking along the roof and floor beams as well as a correlation between diagonal cracks and maximum major principal stresses and their directions. For Specimen 6A the crack pattern is much more erratic due to the presence of panel-to-column ties. Again, there is reasonable correlation between the normal and shear stress contours and the cracking pattern observed. Diagonal cracks appear to have formed along directions normal to maximum major principal stress directions while horizontal shear cracks appear to have formed along maximum shear stress paths. The action of ties suggested by the major principal stress contours is also somewhat noticeable in the crack pattern. Major principal stress and maximum shear stress contours are also shown in figure 6 for Specimen 6B which had a centrally located doorway opening. A correlation between these contours and the experimentally determined crack pattern shown in figure 5(c) is also evident.

4.0 CONCLUSIONS

Twelve large-scale specimens consisting of masonry infilled steel frames have been tested to determine their behaviour and strength when subjected to an in-plane horizontal shear load at roof level. The specimens were varied to evaluate effects of panel-to-column ties, mortar strength, panel-to-frame friction and bond, airspace along roof beam, and openings in the panel. Initial stiffnesses and panel stress contours obtained from a proposed analytical technique were compared with experimental findings.

In view of many random variables such as micro-cracking, variation in mortar strength in a panel, and stress concentrations, etc., which affect the strength and behaviour of a masonry panel, correlation between theory and test results was considered to be satisfactory. Column-to-panel ties result in extensive off-diagonal cracking which is felt to be undesirable in a load-reversal situation. (It is felt that a crack pattern tightly banded about the compressional diagonal results in a more desirable structural behaviour). Additionally, ties appear to increase the initial stiffness but do not appear to be beneficial in increasing either the initial major cracking load or the ultimate strength of masonry infilled steel frames. Provision of an airspace along the roof beam at the top of a panel infill results in a decrease in initial major cracking load and ultimate load of about 50% and 60%,respectively, of that observed for similar specimens with no airspace. The use of panel-to-column ties in these specimens does not appear to significantly improve their in-plane strength and behaviour. Comparing Specimens 1B and 2B of table 2, it is indicated that a reduction in mortar strength of about 50% results in a reduction of cracking strength and ultimate strength of about 40% and 25%,respectively. Comparing results for Specimens 2A and 3A with those of Specimen 1B in table 2, there is some indication that a reduction in panel-to-frame bond and friction tends to reduce the initial crack load and the ultimate strength.

Openings in a masonry infill panel markedly reduce the initial major cracking load while the ultimate load may not as significantly be affected. When a doorway opening is remote from the applied load, the ultimate load is less affected than when the opening is placed towards the loaded side of the panel. It appears that if the opening must interrupt the compressional diagonal, it is preferable that it be placed so that as much panel material as possible is between it and the point of load. This gives a diagonal strut effect an opportunity to develop in that part of the panel. Since, in practical cases, loads will come from either side of a panel, it appears that the best location for a doorway opening is at the center of a panel.

5.0 ACKNOWLEDGEMENTS

Funding for this research was provided by the Canadian Masonry Research Foundation and the University of New Brunswick through its research grants program. This support is gratefully appreciated.

REFERENCES

(1) Dawe, J.L., and McBride, R.T., "Experimental Investigation of Shear Behaviour of Masonry Panels in Steel Frames", Proceedings, 7th International Brick Masonry Conference, Melbourne, Australia, February, 1985.

(2) McBride, R.T., Yong, T.C., Dawe, J.L., and Valsangkar, A.J., "Behaviour of Masonry Infilled Steel Frames Subjected to Racking", Canadian Society of Civil Engineering, Proceedings, National Engineering Conference, Halifax Nova Scotia, May, 1984.

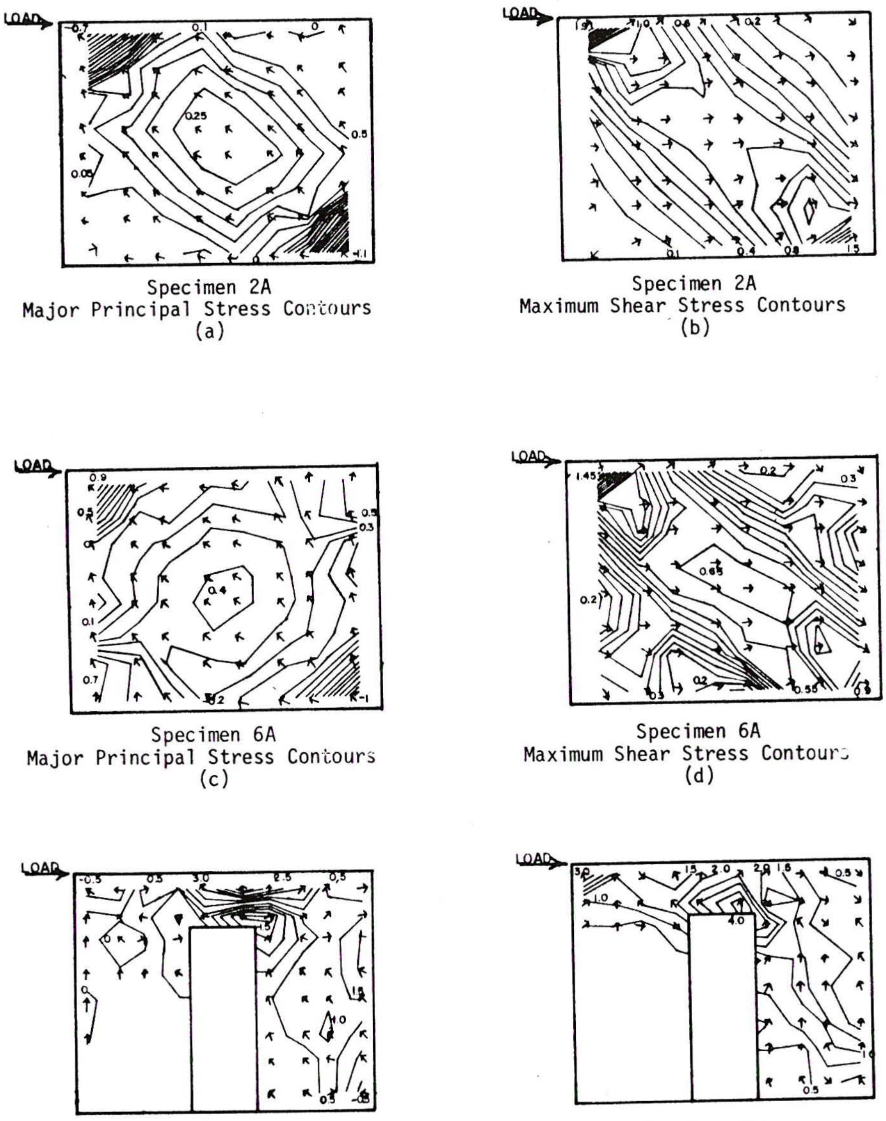
(3) Polyakov, S.V., "On the Interaction Between Masonry Filler Walls and Enclosing Frame when Loaded in the Plane of the Wall", Translations in Earthquake Engineering, pp. 36-42, Earthquake Engineering Research Institute, San Francisco, 1960.

(4) Holmes, M., "Steel Frames with Brickwork and Concrete Infilling", Proceedings of the Institution of Civil Engineers, pp. 473-478, vol. 19, August, 1961.

(5) Stafford-Smith, B., "Lateral Stiffness of Infilled Frames", Journal of the Structural Division, ASCE, vol. 88, ST6, pp. 183-199, 1962.

(6) Mallick, D.V., and Garg, R.P., "Effect of Openings on the Lateral Stiffness of Infilled Frames", Proceedings of the Institution of Civil Engineers, London, England, vol. 49, pp. 193-209, 1971.

(7) Dawe, J.L., and Yong, T.C., "In-Plane Behaviour of Masonry Panels in Steel Frames", Report submitted to the Canadian Masonry Research Council, c/o University of Calgary, Calgary, Alberta, April, 1984.



Specimen 2A
Major Principal Stress Contours
(a)

Specimen 2A
Maximum Shear Stress Contours
(b)

Specimen 6A
Major Principal Stress Contours
(c)

Specimen 6A
Maximum Shear Stress Contours
(d)

Specimen 3B
Major Principal Stress Contours
(e)

Specimen 3B
Maximum Shear Stress Contours
(f)

Figure 6. Maximum Normal and Shear Stress Contours.