EXPERIMENTAL INVESTIGATION OF THE SHEAR RESISTANCE OF MASONRY PANELS IN STEEL FRAMES

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ABSTRACT Six large-scale masonry-infilled steel frame test specimens were loaded to ultimate by gradually incrementing a horizontal shearing force applied at roof level in each case. Characteristics of the specimens were varied so that the effects of joint reinforcement, wall-to-column tie systems, and the use of bond beams can be evaluated. Some of the more important experimental results such as stiffness deterioration and progressive cracking are presented and discussed. Results from analytical methods suggested by other investigators for predicting mode of failure, pre-cracking stiffness, first crack load, and ultimate load are compared with experimental observations. Conclusions of this study are felt to be equally applicable to concrete and burnt clay masonry infills.

1. INTRODUCTION

Standing alone, a masonry panel is relatively weak and rather non-ductile in its resistance to in-plane shear. When contained by a surrounding ductile steel frame, on the other hand, a masonry panel exhibits a markedly increased strength and ductility because of the containment afforded by the steel frame. This very desirable interaction between masonry infill and steel frame permits an increased utilization of masonry,in some cases limited only by its crushing strength.

Although most engineers are aware, either intuitively or otherwise, of the beneficial aspects of masonry infill in resisting shear loads, they may be somewhat reluctant to take advantage of this reserve of strength perhaps because of inadequate experimental evidence or because of lack of scientific documentation of certain characteristics which may affect it. For example, some design engineers may be unsuspecting of the considerable decrease in ultimate shear capacity that can be caused in masonry infill systems by the presence of a narrow airspace along the top of a panel at a roof beam. Additionally, it may be felt that this problem can be somewhat alleviated by including panel-to-column shear ties when, in fact, recent experimental observations suggest that this is not the case (1,2). Although much theoretical and experimental research into the general problem of the shear behaviour of infilled frame systems has been conducted, a review of the literature has indicated that additional experimental research in the area of large-scale masonry infilled steel frames is justified. In the work presented herein, results are presented and discussed for six large-scale masonry infilled steel frame specimens tested to ultimate. Additionally,experimental observations are compared with theoretical analyses proposed by other investigators.

2. PREVIOUS RESEARCH

Early research in the area of shear strength of masonry infilled frames was carried out by Polyakov (3,4) in the USSR, Benjamin and Williams (5) in the US, and Thomas (6) and Wood (7) in the UK. Based on observations from his experimental work, Polyakov suggested that composite action of infill and

frame could be idealized by replacing the infill with an equivalent diagonal strut. Holmes (8) and Stafford-Smith et al (9,10,11,12) further developed this concept. In 1969 Stafford-Smith and Carter (13) proposed a method of analysis for infilled frames based on the equivalent strut concept and in 1977 Stafford-Smith and Riddington (14) proposed an improved method based on finite element analysis. Liauw and Khan (15,16,17) and Wood (23) developed a method of plastic analysis of infilled frames in which different collapse modes are considered.

The use of finite element methods in analysing masonry infilled frames has been applied with varying degrees of refinement and success by several investigators (15,16,17,18,19,20). Additionally, the results of a number of model and large-scale tests have been reported (3,4,5,6,7,8,18,21). A review of the literature reveals that tests on large-scale masonry infilled steel frames including the effects of reinforcement, panel-to-column shear ties, and bond beams are limited.

3. TESTING PROGRAM

A testing program was set-up in an attempt to evaluate the behaviour of masonry infilled steel frames and in particular, the effects of joint reinforcement, panel-to-column shear ties, and bond-beams. Table 1 is a summary of the panel characteristics of six large-scale specimens tested for this purpose.

TABLE 1 Description of Panels

Specimen	Age (days)	Reinforcement Plain	Reinforcement Joint	Tie System	Bond Beams
1	32		√		
2	14		√		
3	14		√		
4	14	√			
5	15		√	√	
6	14		√		√

3.1 Testing Frame

Details of the testing frame are shown in Figure 1. The frame consisted of a rigid triangular loading frame mounted on a stiffened built-up support beam securely bolted to the laboratory strongfloor. The specimen, mounted on the supporting beam as shown, was loaded at roof level by means of an 1 800 kN ram. A lateral bracing system (not shown) used roller-bearing contacts at the upper corners to prevent the test frame from deflecting out of plane.

3.2 Test Specimens

Apart from the general characteristics given in Table 1 all large-scale specimens consisted of 3.6 m long by 2.8 m high masonry panels made of 200 x 200 x 400 concrete masonry blocks in intimate contact with a surrounding steel frame consisting of W 250 x 58 columns and a W 200 x 46 beam. Panels were mortared snugly into the roof beam flange and column webs and flanges. In reinforced panels, truss type joint reinforcement

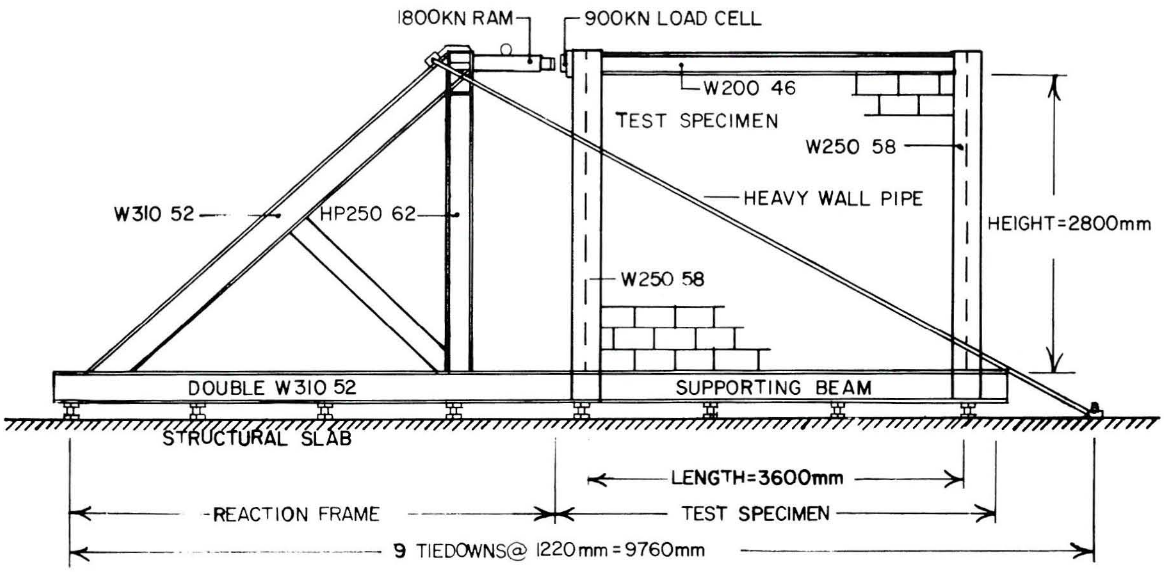


Figure 1. Testing Frame With Specimen in Place.

(9-gauge wire, 3.66 mm diam.) was placed in alternate bed joints starting with the second course. In specimen 6, two bond beams placed at the 1/3 and 2/3 levels of the panel height consisted of bond beam blocks containing two 15 M deformed bars and filled with mortar. Specimen 5 included a 50 x 3.2 mm flat bar tie system. L-shaped ties were welded to the column webs with the bent portions extending into the second cell from the end of a panel. This cell was filled with mortar along its height. Ties were placed at alternate courses beginning with the second course.

3.3 Instrumentation and Loading

LVDT's at the tops of the columns and 0.001-mm dial gauges at other points around the frame were used to monitor frame deflections at each load increment. Inclinometers were used to measure rotations of the columns at the level of the roof beam centerline. Strain gauge instrumentation was used to monitor shear and axial stresses in the columns. Separation along the infill-frame interface was monitored using applied gauge points and a multi-position gauge. Similar instrumentation was used to monitor strain within the masonry panel primarily along its compressional diagonal.

A 900-kN flat load cell placed between the loading ram and steel frame at roof beam level was used to monitor applied load, which was incremented slowly from zero to ultimate in steps of 22.2 kN. At each load increment the system was allowed to stabilize before readings were recorded. Crack patterns were noted and marked clearly as testing progressed.

4. TEST RESULTS

4.1 Specimen Behaviour

The overall behaviour during testing was similar for the majority of specimens. As an example, the behaviour of specimen 1 is summarized. At 44.5 kN, hairline cracks developed at the interface of bottom beam and infill near the column on the load side of the specimen. At 89 kN there was an audible cracking sound and upon close inspection, a separation crack between the roof beam and infill was noticed. At 111 kN, cracking of the masonry occurred in the upper and lower tension corners. Cracking along approximately 50% of the mortar joint between the top and second courses of the infill occurred at 133 kN. At 342 kN a major crack extending stepwise along the compressional diagonal was accompanied by a sudden drop in load to 320 kN. Beyond this point, loading resumed with no apparent decrease in stiffness until the maximum load was approached. The maximum load reached for this specimen was 471 kN. A summary of test results for all specimens is given in Table 2.

TABLE 2 Summary of Test Results

Col 1 Specimen	Col 2 Age (days)	Col 3 Major Crack Load (kN)	Col 4 Ult. Load (kN)	Col 5 Stiffness (kN/mm)
1	32	342	471	73
2	14	356	440	82
3	14	200	463	74
4	14	211	476	63
5	15	334	445	113
6	14	483	489	87

The stiffness value shown in Table 2 for each specimen represents the initial slope of the straight-line portion of the load displacement curve for the specimen. (Load-displacement curves are shown in Figure 2).

4.1.1 Load-Displacement Curves. Figure 2 shows the graphical relationship between horizontal load and corresponding deflection measured at the load point for each of the six specimens. The jagged portions of these curves correspond to initial major cracking which was usually accompanied by a noticeable drop in load. Specimen 6 had two such occurrences at approximately 350 kN and 483 kN, the higher value being recorded in Table 2. Ultimate capacity for each specimen was usually characterized by excessive cracking, greatly enlarged diagonal crack widths, spalling of the masonry at crack locations, and an inability of the system to carry additional load as deformations were further increased.

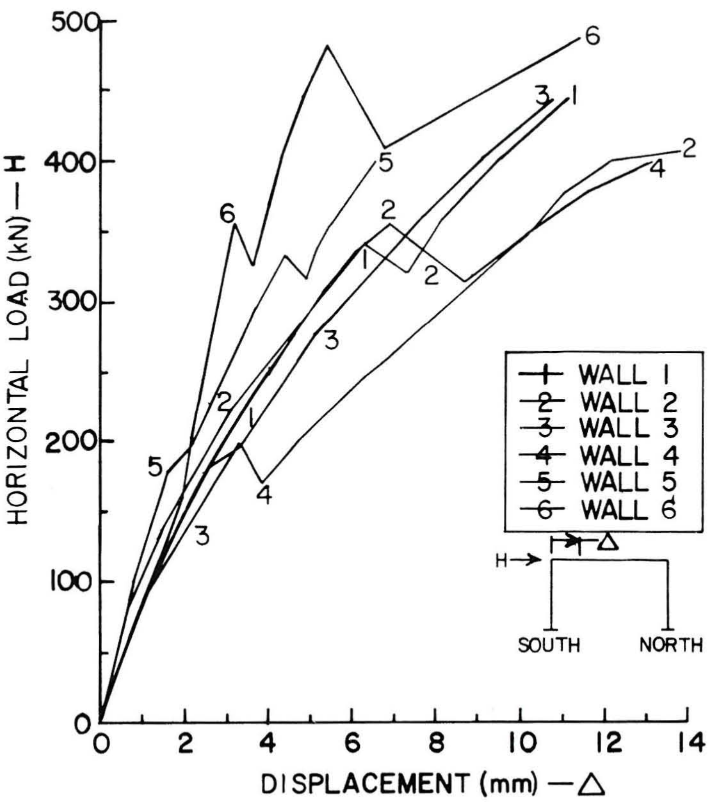
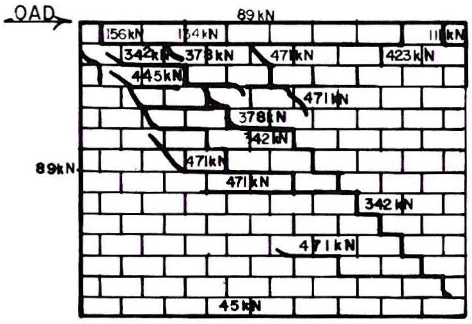


Figure 2. Horizontal Load Versus Displacement.

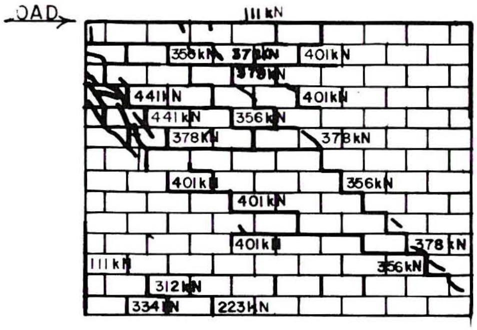
4.1.2 Crack Patterns. Progressive crack patterns are shown in Figure 3 for each of the six specimens. Major cracks occurred at the loads indicated in Table 2 and correspond to the jagged portions of the load-displacement curves of Figure 2. Some minor cracking, not accompanied by a drop in load, was recorded at lower loads. In Figure 3, the loads at which cracking occurred are shown on the cracks outlined in each case.

4.2 Discussion of Test Specimen Behaviour

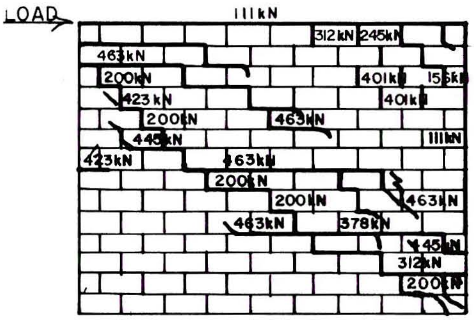
Referring to Table 2 and Figure 2, Specimens 1, 2, and 3 which were similar and included horizontal joint reinforcement had similar initial stiffnesses with an average value of 76 kN/mm. The lowest stiffness of 63 kN/mm occurred for Specimen 4 which had no joint reinforcement but was otherwise similar to Specimens 1, 2, and 3. The highest stiffness of 113 kN/mm occurred for Specimen 5 which was similar to Specimens 1, 2, and 3 except for the inclusion of panel-to-column ties. Specimen 6 which was similar to Specimens 1, 2, and 3 except for the inclusion of bond beams, had an intermediate value of stiffness of 87 kN/mm.



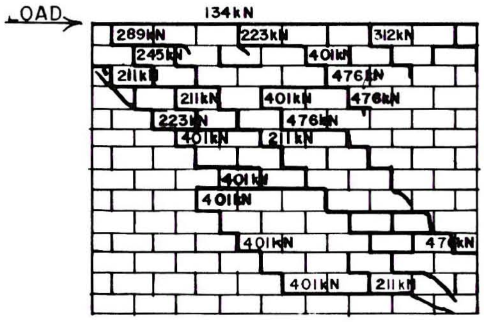
Specimen 1
(a)



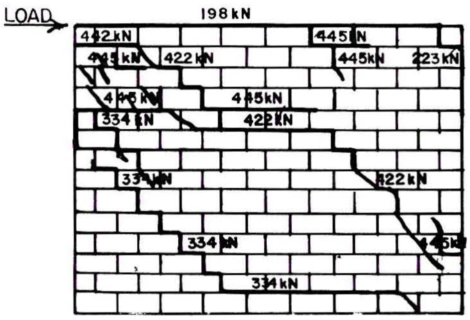
Specimen 2
(b)



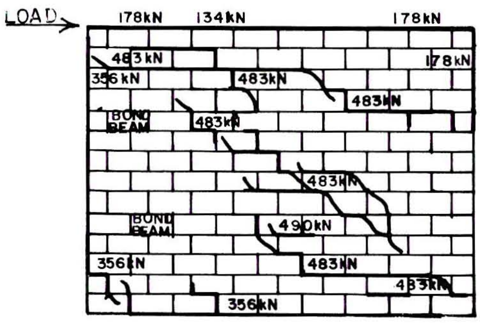
Specimen 3
(c)



Specimen 4
(d)



Specimen 5
(e)



Specimen 6
(f)

Figure 3. Crack Patterns for Infilled Masonry Panels

Referring to column 4 of Table 2, it appears that horizontal joint reinforcement, panel-to-column ties, and bond beams have little effect on the ultimate load of the specimens. (The average ultimate load of the six specimens is 464 kN with an overall maximum deviation from this value of about 5%). Although Specimen 3 had a low initial major crack load as compared to Specimens 1 and 2 which were similar, Figure 2 shows that the load-deflection behaviour was almost identical for all three specimens up to about 350 kN. Beyond initial cracking, Specimen 4, which had no joint reinforcement, exhibited the least stiffness for all specimens as shown in Figure 2. Specimen 6 which included bond beams was the overall stiffest of the specimens tested. Additionally, as shown in Table 2, initial major cracking for this specimen virtually coincided with the ultimate load.

Figures 3(a), (b), and (c) show progressive crack patterns respectively for specimens 1, 2, and 3 which had identical specifications. The crack patterns are generally similar with cracks extending primarily along the main compressional diagonal through the mortar joints. At lower loads there is evidence of separation between panel and frame as well as some block separation in the panel at the tension corners. At loads near ultimate, extensive cracking of the masonry occurs at the loaded corners. Figure 3(d) shows the crack pattern for Specimen 4 which had no joint reinforcement. There is some indication that the absence of joint reinforcement leads to more extensive and random cracking than occurs in panels with joint reinforcement. The crack pattern shown in Figure 3(e) for Specimen 5 which had column-to-panel ties indicates a more random, off-diagonal pattern than occurs for panels without ties. It appears that, at the initial major crack load, a triangular portion in the lower tension corner directly below the applied load was pulled away from the remainder of the panel. Beyond that load level the panel appeared to behave somewhat as an untied specimen. The crack pattern for Specimen 6 is shown in Figure 3(f). The bond beams appeared, to some extent, to divide the overall major cracking into three regions at the top, middle, and bottom of the panel, respectively. The crack pattern in each region primarily extends through horizontal joints and diagonally through the units.

4.3 Comparison With Theory

Several theoretical techniques relating to mode of failure, crack load, ultimate load, and stiffness of masonry infilled frames are compared with experimental observations for the type of system studied herein.

4.3.1 Equivalent Strut Concept. Holmes proposed a procedure for calculating the ultimate load of a brickwork or concrete infilled steel frame by replacing the panel with an equivalent strut (8). The proposed formula requires a value of failure strain and compressive strength of the masonry at failure. Using values of strain and of compressive strength as determined from prism tests, Holmes' method predicts an ultimate load of 2 100 kN (about 4.5 times the average test value). This large discrepancy probably results from the fact that the method assumes crushing of the compression diagonal as the mode of failure whereas the test specimens failed primarily by tensile and shear cracking of the infill.

In applying their version of the equivalent strut concept, Stafford-Smith and Carter (13,22) recognize three failure modes dependent on the relative stiffness of the infill to frame. For the specimens tested herein this value of relative stiffness was calculated as 6.75 neglecting effects of

joint reinforcement, ties, and bond beams which are not considered by the proposed analysis. For each of the three modes of failure, reference 13 presents graphical relationships in which diagonal force to cause crushing at the loaded corners, diagonal force to cause shear failure, and diagonal force to cause diagonal tension failure in the infill are given in non-dimensional form as functions of the relative stiffness. Using this procedure, the predicted horizontal failure loads corresponding to crushing, shearing, and diagonal tension failures of the specimens tested herein are 550 kN, 401 kN, and 135 kN respectively. Thus, a diagonal tension failure is predicted at 135 kN. This is the correct mode of failure for the specimens tested herein. However, the predicted load of 135 kN is rather low when compared to the average cracking load of 289 kN (excluding the results for the specimen which had bond beams). The method proposed by Stafford-Smith and Carter for the estimation of stiffness for infilled frames was used to determine a relationship between the horizontal load and the displacement at the point of applied load. As shown in Figure 4, the theoretical results of this method show good agreement with test results for Specimens 1, 2, and 3. (These three specimens were selected because they are similar and do not include variations such as bond beams or ties).

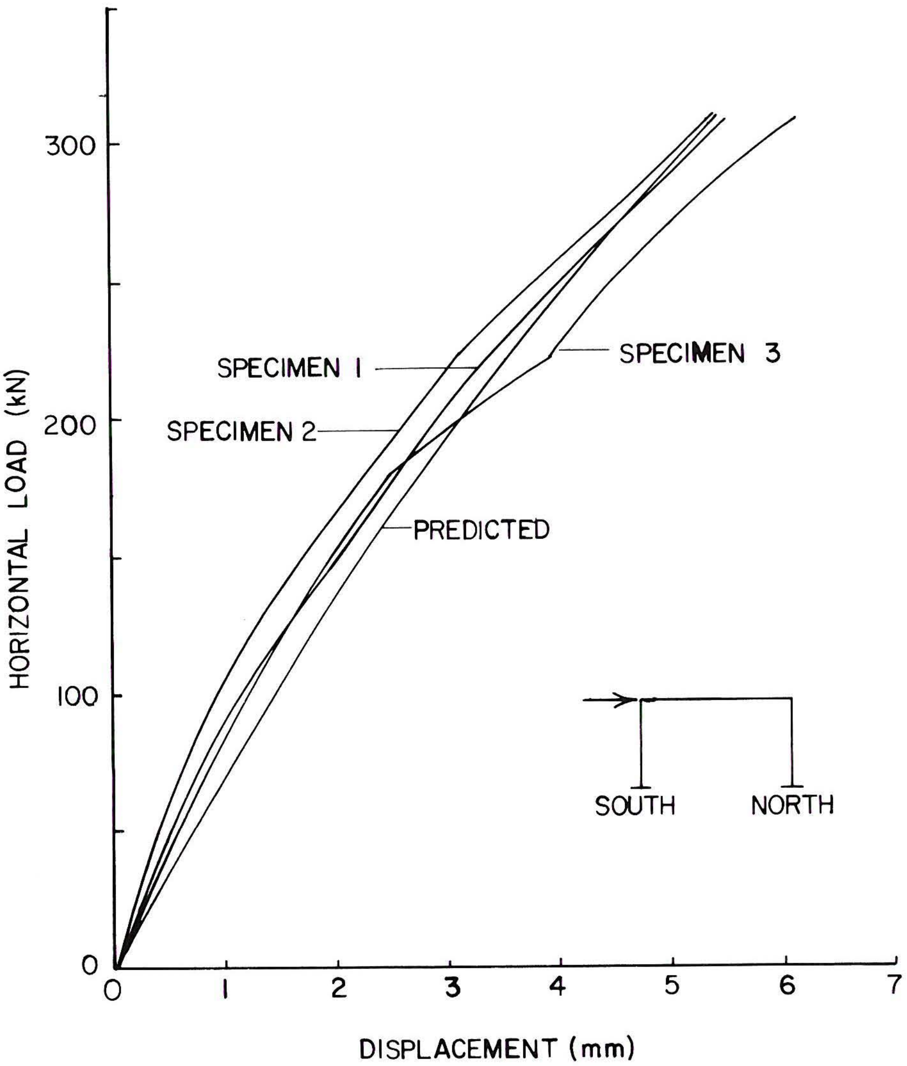


Figure 4. Comparison of Test Results With Predicted Values (13,22).

Using formulae presented by Stafford-Smith and Riddington (12,14), horizontal loads of 630 kN for failure by crushing, 395 kN for failure by shear cracking along the interface of mortar and units, and 142 kN for failure by diagonal tension cracking through the mortar joints and units were predicted. The predicted failure load of 142 kN is only slightly above that predicted by Stafford-Smith and Carter while the predicted mode of failure is the same.

4.3.2 Collapse Theories. The plastic collapse theory of Liauw and Khan (15,16,17) recognizes four possible collapse modes in predicting the ultimate capacities of infilled frames. The four modes of collapse considered are: 1. corner crushing with plastic hinges in the columns and connections, 2. corner crushing with plastic hinges in the beams and connections, 3. diagonal crushing with failure along the columns, and 4. diagonal crushing with failure along the beams. For the specimens tested herein, the predicted ultimate loads are 760 kN, 1 200 kN, 970 kN, and 1 500 kN for modes 1, 2, 3, and 4 respectively. The lowest load of 760 kN predicted for mode 1 is considerably higher than the average ultimate load of 464 kN for the six specimens tested herein. The specimens did not develop plastic hinges at the time the maximum load was reached. Rather, the infill had deteriorated extensively at this point and additional frame deflections resulted in further deterioration of the infill rather than a load increase.

Wood (23) suggests, based on investigations similar to that of Liauw and Khan, that a penalty factor be applied to reduce the effective crushing stress of the infill to account for the fact that masonry panels are not ideally plastic. For the specimens tested herein, this penalty factor was determined to be 0.367. Using this factor, the re-computed collapse loads for modes 1, 2, 3, and 4 are 464 kN, 701 kN, 471 kN, and 502 kN respectively. Again, the lowest load is for mode 1 and the predicted load of 464 kN compares well with ultimate loads ranging from 440 kN to 489 kN for the six specimens tested. A mode 1 failure is also consistent with laboratory observations which indicated that a plastic hinge would develop in the loaded column at approximately 1 400 mm from its base and the masonry in the compression corner was undergoing extensive cracking.

5. CONCLUSIONS

Six large-scale masonry infilled rigid steel frame specimens with varying panel characteristics have been tested and results compared to some available theoretical analyses. Quite notably, and as also recognized by others, masonry infill greatly increases in-plane shear resistance of steel frames. In a reciprocal fashion, the surrounding steel frame ensures adequate ductility of the system. The presence of column-to-panel ties, horizontal joint reinforcement, or bond beams,does not significantly affect ultimate loads although crack patterns can be significantly different. Initial cracking loads do not appear to be affected by ties or horizontal joint reinforcement whereas the use of bond beams leads to a major cracking load quite near the ultimate.

Holmes' proposed method based on a crushing mode of failure was found to be unsuitable for the specimens tested in the study presented herein. The method is probably more suitable to more flexible frames with homogeneous infill. The methods of Stafford-Smith et al predict the correct mode of failure but a rather conservative cracking load for the specimens studied herein. The pre-crack stiffness as predicted using the method of

Stafford-Smith and Carter shows good agreement with experimental results. The plastic collapse theory of Liauw and Khan predicts much higher ultimate loads than obtained experimentally although the predicted mode of failure agrees with that observed during testing. When modified by Wood's penalty factor however, the ultimate load predicted by the plastic theory is in good agreement with test results. It is felt that the conclusions of this study are equally applicable to concrete masonry and burnt clay masonry infills in steel frames.

ACKNOWLEDGEMENTS

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