11 WCEE

Copyright © 1996 Elsevier Science Ltd
Paper No. 990. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

LATERAL STRENGTH OF BRICK CLADDED STEEL FRAMES

Stephen P. SCHNEIDER, Assistant Professor, and
Stephen J. FAVIERI, Research Assistant

Department of Civil Engineering, University of Illinois @ Urbana-Champaign
205 North Mathews Avenue, Urbana, IL 61801-5149, USA.

ABSTRACT

This paper describes an experimental program to investigate the in-plane seismic behavior of steel frames with unreinforced masonry infills having large window openings. This experimental investigation was a necessary first step to fully understand stiffness, strut behavior, strength and ductility of these composite systems with extensive cracking in the masonry infill. Test parameters for this study included the height-to-width aspect ratio of the masonry pier, and the number of wythes. Preliminary results suggest that, for some specimens, the contribution of the masonry to the strength and stiffness of the frame was significant up to drifts of approximately 1.4%. At drifts beyond this level, the strength and stiffness of the composite system reduced primarily to that of the bare steel frame. Failure of the pier depended on the pier width. Masonry crushing at the wall toe was observed in narrow piers, while wide piers failed by diagonal compression splitting.

KEYWORDS

Masonry Infills with Large Window Openings, Strut Behavior in Masonry Piers, Steel-Masonry Composite Systems, Seismic Evaluation, Large-scale Quasi-Static Testing.

INTRODUCTION

Steel frames with unreinforced masonry infills having large window or door openings were a common construction type earlier this century. Several of these composite systems however, were damaged near Oakland California during the 1989 Loma Prieta earthquake (Freeman 1994). Although masonry panels were used primarily as an architectural component, many buildings in Oakland are considered structurally unsound due to perceived excessive cracking of the masonry infill. Further, there is a lack of simple analytical methods to reliably estimate the lateral-load resistance of steel frames with brick masonry infills. Consequently, the engineering community needs more information on the behavior of these composite systems, and guidelines to evaluate structural integrity.

This paper describes an experimental study to determine the lateral strength and stiffness of steel frames composite with unreinforced masonry infills having large window openings. Test specimens were designed to isolate and quantify the fundamental seismic behavior of frames with masonry infills. Strut behavior of the masonry infill, flexural stiffening due to the masonry piers, and stiffening of the beam-to-column joint have been identified as the most important influences on the seismic resistance of these structural systems. This paper focuses on the strut action and flexural stiffening portion of the experimental program.

Little analytical or experimental information is available to help structural designers evaluate the seismic behavior of steel-masonry composite systems. Mander *et. al.* (1993, 1994) suggest that steel frames with brick masonry infills exhibit moderately ductile behavior when loaded in the plane of the wall. However, when subjected to large drifts the masonry infill became granulated and may be susceptible to out-of-plane forces. Strut models have been suggested (Freeman 1991, Hamburger 1993, Mander *et. al.* 1994) to estimate the strength of steel frames with masonry infills, however few of these models have been verified experimentally. Further, previous experimental research studied the performance of solid infill panels, while infills in most buildings have large window or door openings.

EXPERIMENTAL PROGRAM

An experimental program was necessary to investigate the in-plane lateral-deformation behavior of steel frames with masonry infills having large window openings. Of particular interest was the contribution of each component to the strength and stiffness of the composite system over a wide range of drift levels. The test specimen and the reaction system for this test program are shown in Fig. 1, and a schematic of the test specimen only is illustrated in Fig. 2. All tests were performed in the Newmark structural engineering lab at the University of Illinois.

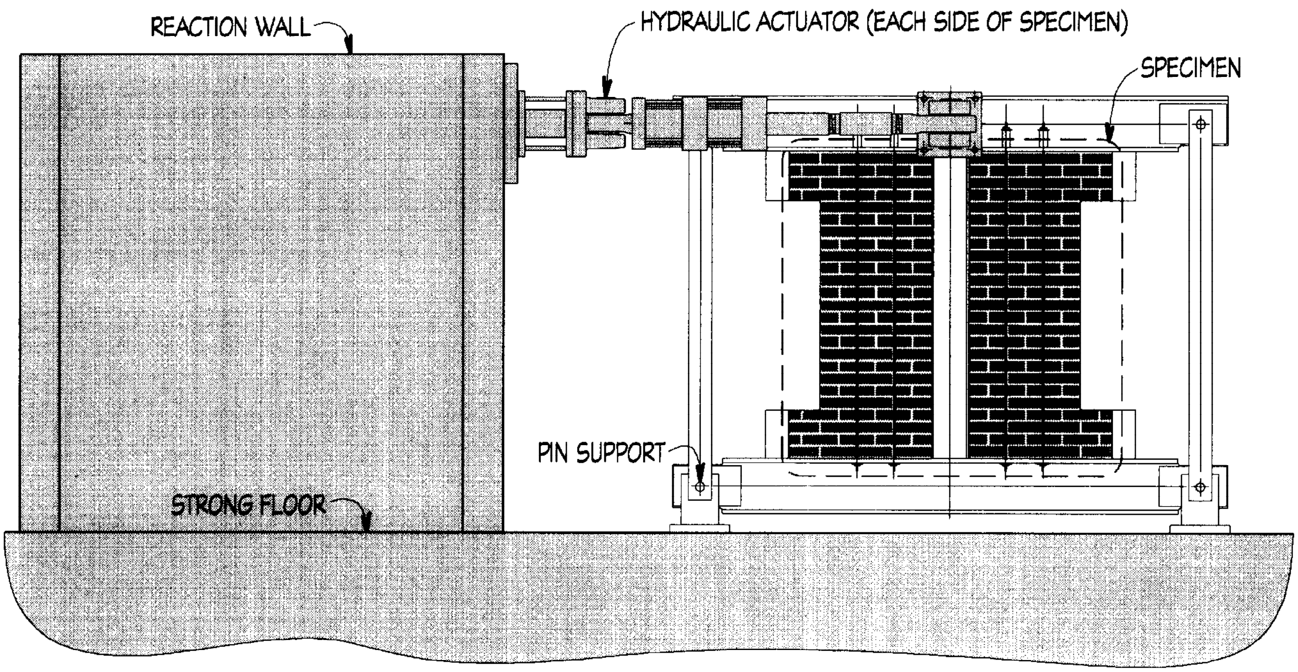

Fig. 1. Test Apparatus for Experimental Program

To investigate desired behavior, a test specimen was chosen to represent an interior steel column with masonry piers on either side. A W8x31 was selected for the steel column of the test specimen. The W8x31 was attached, using full-penetration groove welds, to the center of two W12x87 beam elements. The W12x87s were sufficiently stiff, relative to the W8x31, to approximate a fixed-fixed end condition for the steel column. The upper W12x87 was supported at its ends by struts to simulate roller supports, while the bottom W12 was supported at its ends by pin supports that were anchored to the strong floor.

Clay-unit masonry was placed on each side of the W8 steel column. Symmetry was maintained to simulate an interior column. To provide full bearing, a mortar bed was placed between the bricks and the steel elements at each interface. Structural steel tubes were welded to the W12s at four locations. The masonry was placed with full bearing against these steel tube "blocks" to help develop the thrust that can occur in the brick infill due to the adjacent masonry pier. Window openings were simulated by the absence of thrust that can develop in the masonry over the window opening height [h]. Tie rods were tensioned between the W12s to produce a compressive axial stress in the masonry infill. Compressive stress was applied to account for some live load that may be distributed to the infill from the floor gravity loads, and from the self-weight of the masonry infills above the floor. The axial stress applied by the tension rods was adjusted to compensate for the load shared by the W8 steel column and the struts.

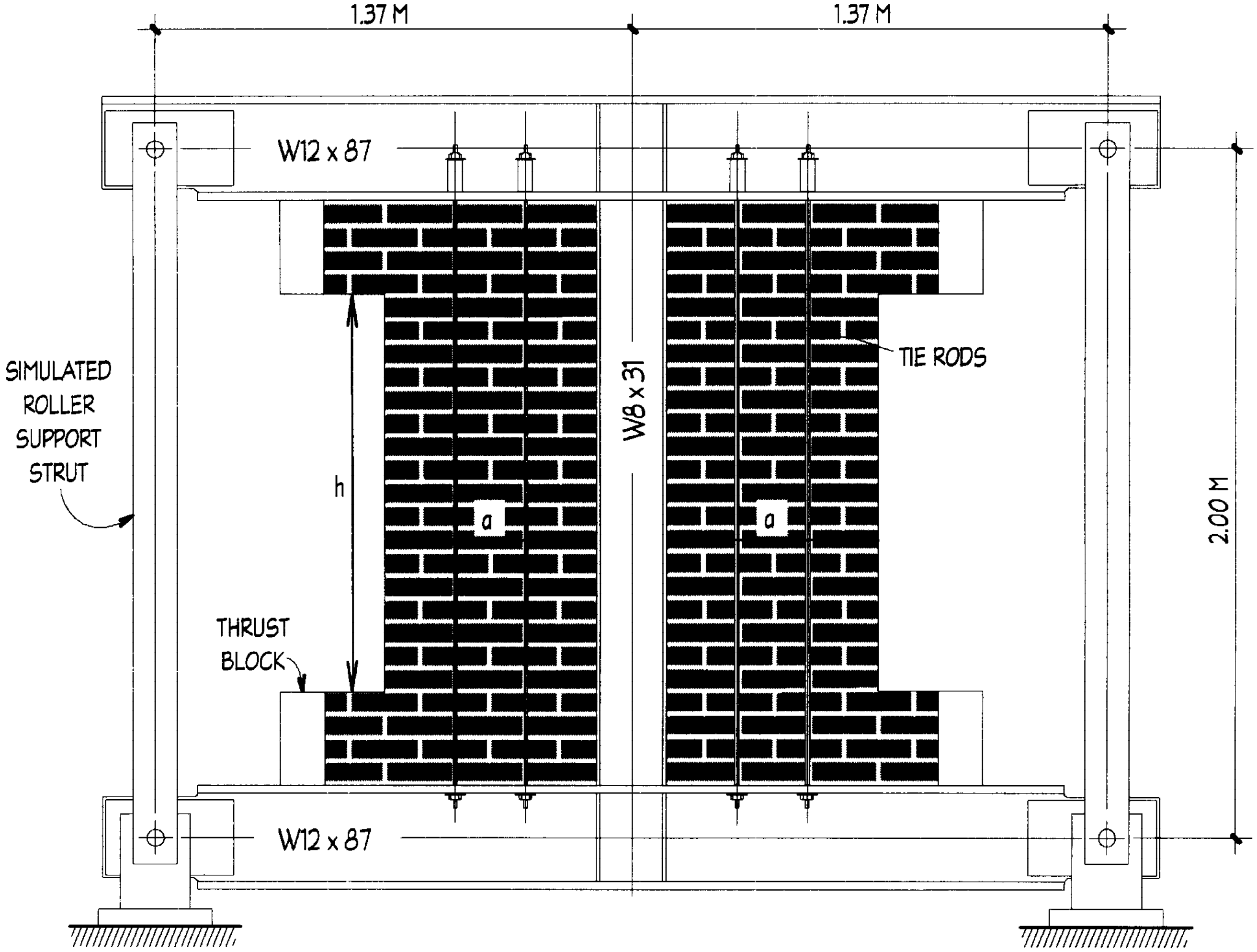

Fig. 2. Schematic of Test Specimen

The primary parameters for this test program were the width [*a*] for each side of the masonry infill and the number of wythes. The pier height was fixed with a window opening height of *1.15M.* Type N mortar was used for all specimens, and all bricks were from the same shipment. Compressive stress for each test was maintained at *420Pa*, which represents gravity loads from about four floors. Test parameters for each specimen are listed in Table 1.

Table 1. Variation of Height-to-Width Ratios for Test Specimens

Test Specimen	Axial Stress (Pa)	*a* (cm)	*h/a*	Wall Thickness (No. of Wythes)	Wall Area (cm^2)
1	420	40	2.88	1	365
2	420	60	1.92	1	550
3	420	60	1.92	2	1,100
4	420	80	1.44	1	750
5	420	40	2.88	2	730

Load cells, displacement transducers, and strain gages were placed on the test frame and the specimen to determine the distribution of the lateral forces in each element of the composite system. Of particular importance was the amount of lateral shear distributed to the steel column compared to the shear in the masonry pier, and the increase in stiffness of the steel frame due to the masonry infill.

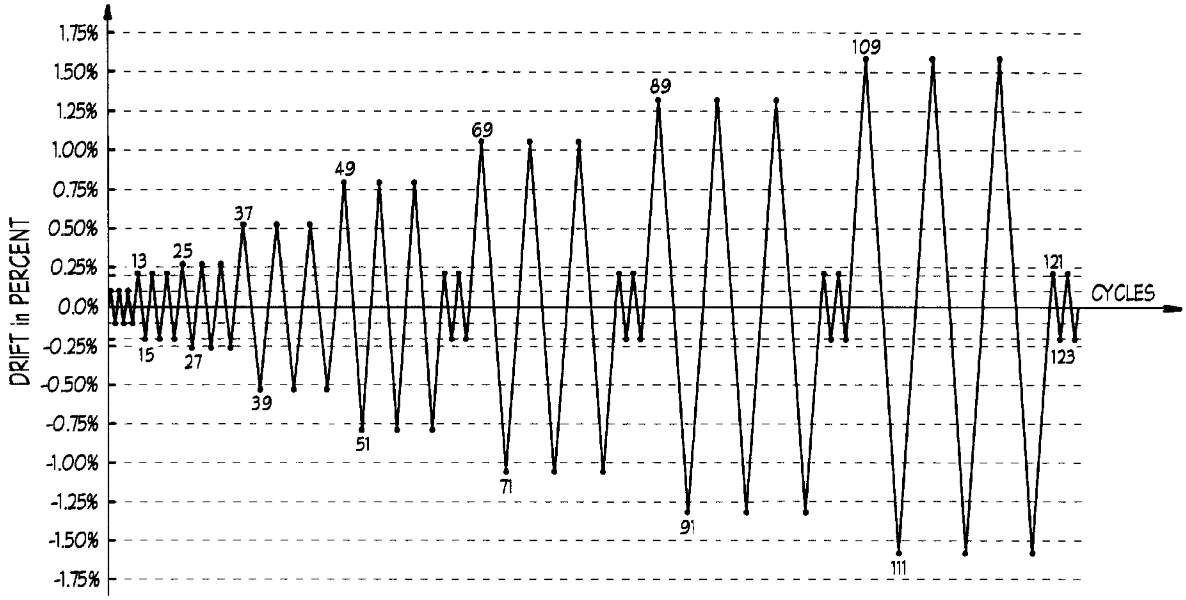

Fig. 3. Imposed Cyclic Deformation History

Earthquake loads were simulated by imposing lateral deformations to the center of the upper W12, using two 500 KN hydraulic actuators. The hydraulic actuators maintained equal forces on each side of the specimen to minimize imposed torsion. Predetermined cyclic deformations were imposed on each test specimen, and a typical deformation history is shown in Fig. 3. To identify crack growth on the specimen during the test, the deformation corresponding to each quarter cycle was numbered sequentially. Various levels of deformation were needed to investigate seismic behavior from initial to extensive cracking, and to eventually study behavior at failure of the masonry infill. In order to determine the deterioration of the elastic strength and stiffness characteristics, two "elastic" deformation cycles were imposed after three cycles of same-amplitude drift. These "elastic" cycles were imposed when the inelastic drifts exceeded 0.75%.

EXPERIMENTAL RESULTS

Experimental results for Test Specimens 2 and 3 are shown in Figs. 4 through 8. For these results, the horizontal load on the test specimen is the sum of the forces in both hydraulic actuators, and the displacement was measured at mid-depth of the upper W12x87.

Figure 4 illustrates the force-displacement behavior of the bare steel frame alone. The bare frame was tested, before the masonry was placed to determine the inelastic cyclic characteristics of the steel frame alone. Further, the bare steel frame test proof-loaded the full-penetration groove welds. Large-displacement cycles were imposed on the steel frame to approximate equivalent amplitudes expected during the test. Three cycles at each deformation-level were imposed, and each cycle indicated repeatable behavior. No deterioration in the inelastic behavior was noted, primarily because no local buckling occurred in the W8x31. Also, the elastic stiffness deteriorated less than 15% after all inelastic cycles were imposed. Therefore, the force-displacement history of the bare steel frame applies to both specimens 2 and 3. After each test, the permanent lateral-displacement was corrected to obtain the original undeformed configuration, and the masonry was placed inside the steel frame.

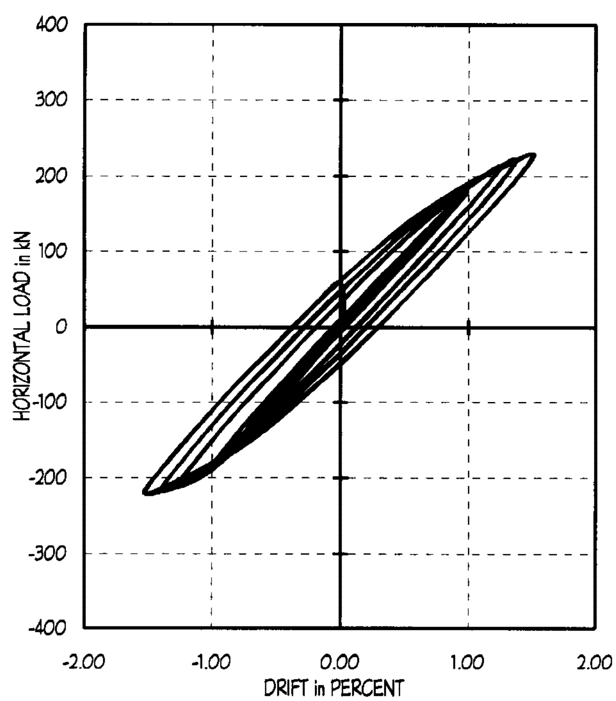

Fig. 4. Force-Displ. Behavior of Bare Steel Frame

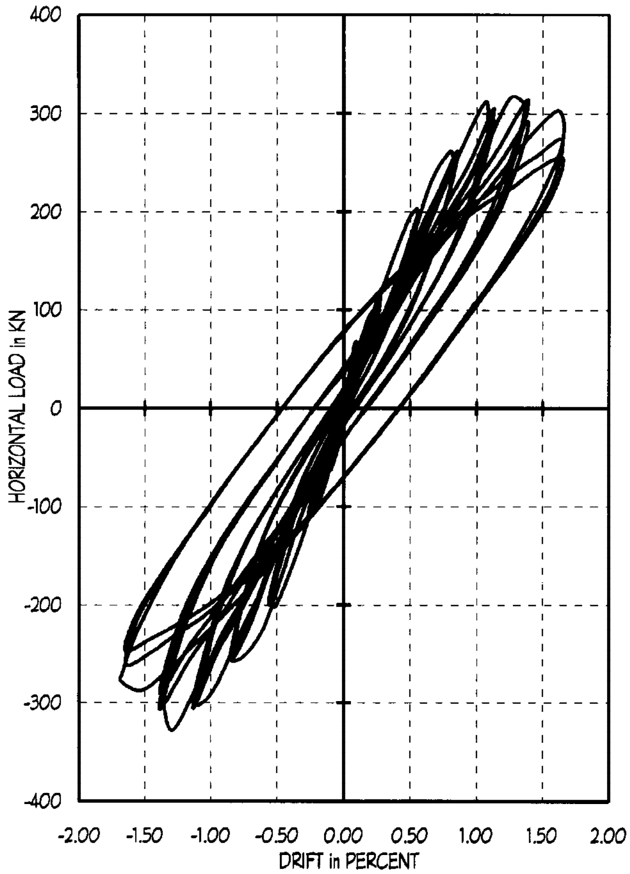

Fig. 5a. Composite Force-Displ. Behavior, Specimen 2

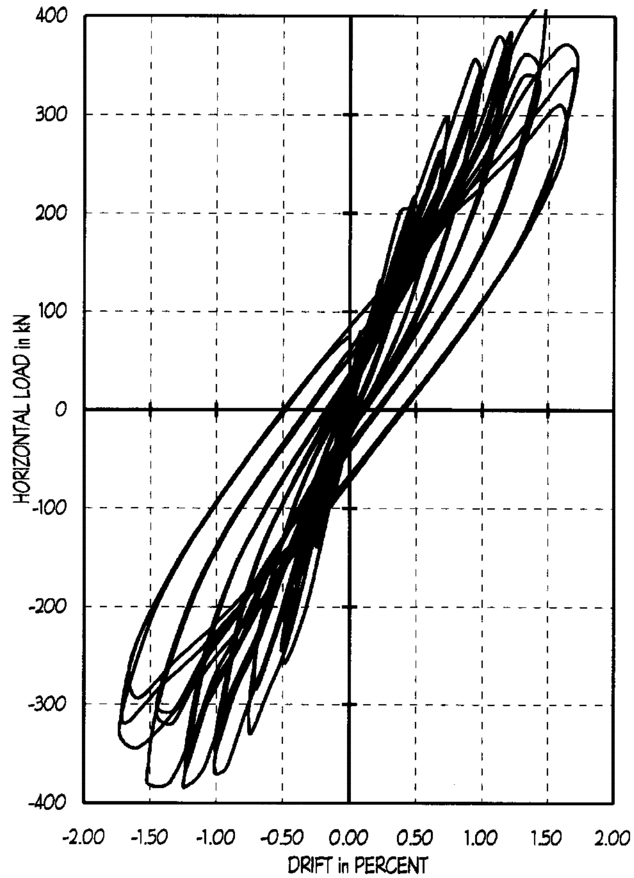

Fig. 5b. Composite Force-Displ. Behavior, Specimen 3

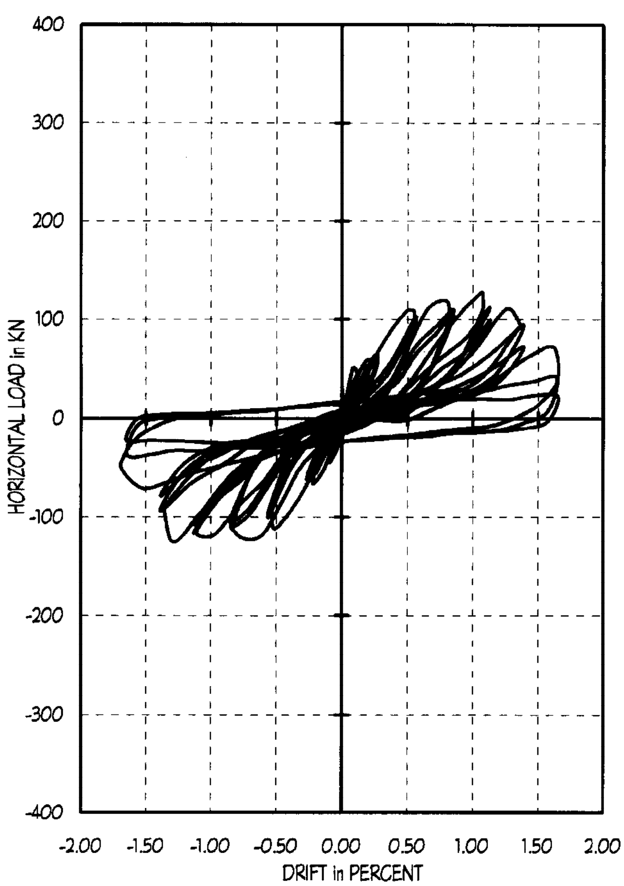

Fig. 6a. Masonry Force-Displ. Behavior, Specimen 2

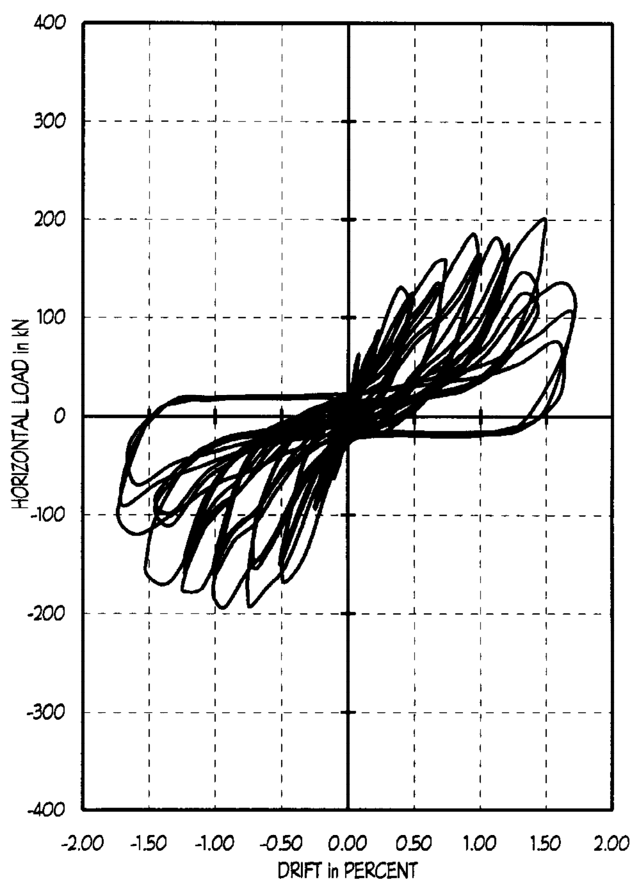

Fig. 6b. Masonry Force-Displ. Behavior, Specimen 3

Figure 5 illustrates the load-displacement behavior of both specimens. In general, the composite system exhibited reasonably ductile behavior. The first of the three same-amplitude displacement cycles absorbed the most inelastic energy, while this energy absorption capacity reduced for the remaining two cycles. This reduction in inelastic energy absorption capacity is due to the opening and closing of the cracks formed on previous cycles.

Figures 6 illustrates the contribution of the masonry infill to the strength of the steel-masonry composite system. Results for this hysteretic behavior were obtained by subtracting the bare steel frame results from those of the composite test specimen. These results indicate the approximate strength contribution of the masonry infill to the composite system. Comparison of the graphs in Fig. 6 illustrate that the strength contribution of the double wythe wall was almost twice the strength contributed by the single wythe wall. However, because of the steel frame the overall strength of the double wythe structural system was only about a 30% increase over the strength of the single wythe system. The contribution of the masonry for the single wythe wall tends to diminish at smaller drift levels compared to the double wythe wall. Strength in the single wythe pier began to decay at approximately 1.0% drift, while the shear contribution of the double wythe masonry pier was significant up to about 1.4% drift. Little contribution from the masonry was observed for either specimen at the maximum 1.7% drift, particularly on the last two cycles of the imposed deformation.

Figure 7 illustrates the overall strength of the composite system, and the percentage of the strength contributed by each structural component. These graphs illustrate the absolute value of peak strengths near, or at, the peak displacement. Clearly, the double wythe masonry infill offered a larger contribution of shear strength, at larger drift levels, than the single wythe infill.

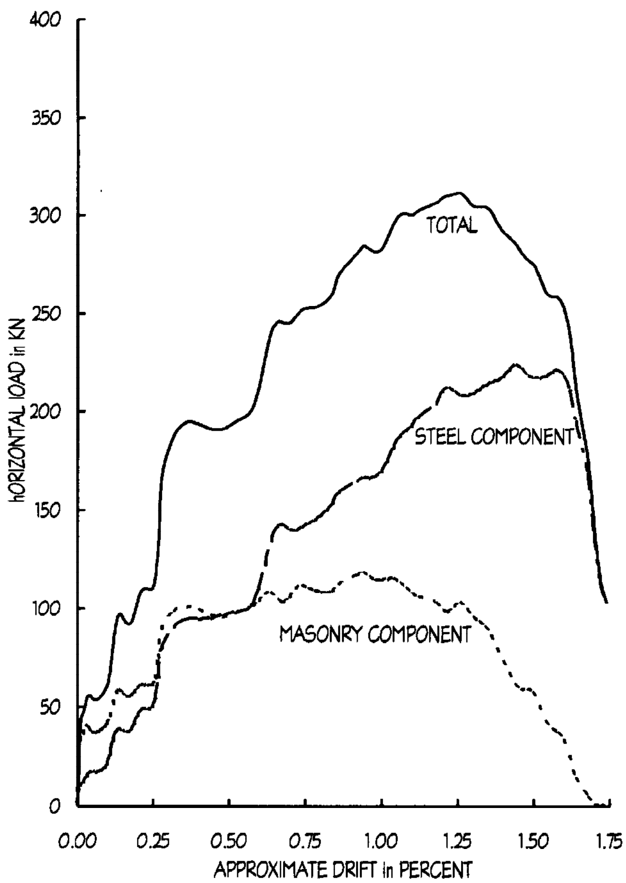

Fig. 7a. Shear Distribution, Specimen 2

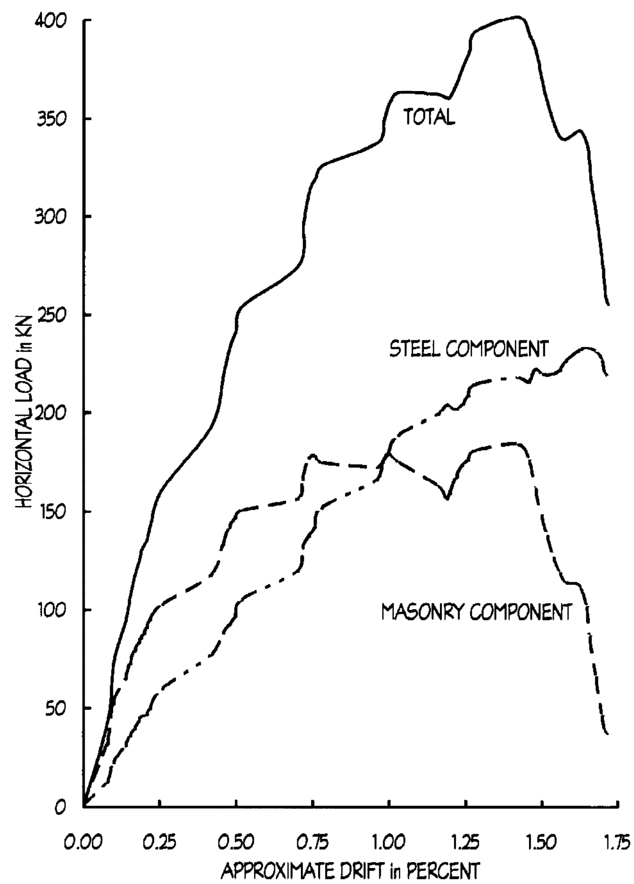

Fig. 7b. Shear Distribution, Specimen 3

The cyclic deformation history in Fig. 3 illustrates two cycles of elastic-level drift, of approximately 0.2%, between each three-cycle set of large displacements. These elastic-level drift cycles exhibited little influence from the masonry pier. The stiffness and strength during these deformations were close to that of the bare steel frame. This was due to the amount of cracking exhibited by the large-deformation cycles prior to the initial elastic-level drift. Enough cracking occurred in the masonry to minimize its influence on the steel frame at small-deflection levels.

These results suggest that after significant damage occurred, the strength and stiffness of the bare steel frame should be used to determine the characteristics of a composite system at small deformations.

Figure 8 illustrates typical crack patterns at the end of the test for masonry piers with a height-to-width ratio of approximately 2. For Specimen 2 the left infill developed diagonal compression splitting due to strut action. Eventually this cracking reduced the effective width of the masonry pier because some bricks lost cohesion with the remaining infill. At large deformations, the portion of the masonry between the diagonal crack and the window opening crumbled, and was on the verge of collapse. The crumbled portion of masonry is indicated by the darker shaded brick. The right pier exhibited a different failure mode. Extensive toe crushing of the masonry infill occurred in the vicinity of the initial flexural cracks, just above the lower corner of the window opening. The crushed portion of masonry is shown by the darker shaded brick on the right side of the column. This extensive masonry crushing clearly limited the amount of diagonal strut action that developed in the piers, and explains the deterioration of the strength and stiffness of the composite system to that of the bare steel frame alone. For Specimen 3, both sides of the infill developed diagonal compression splitting. Failure modes for both piers were almost exactly the same as the diagonal compression splitting observed in the left pier for Specimen 2. Extensive cracking appeared, and crushing when present, occurred at drifts larger than 1.1%.

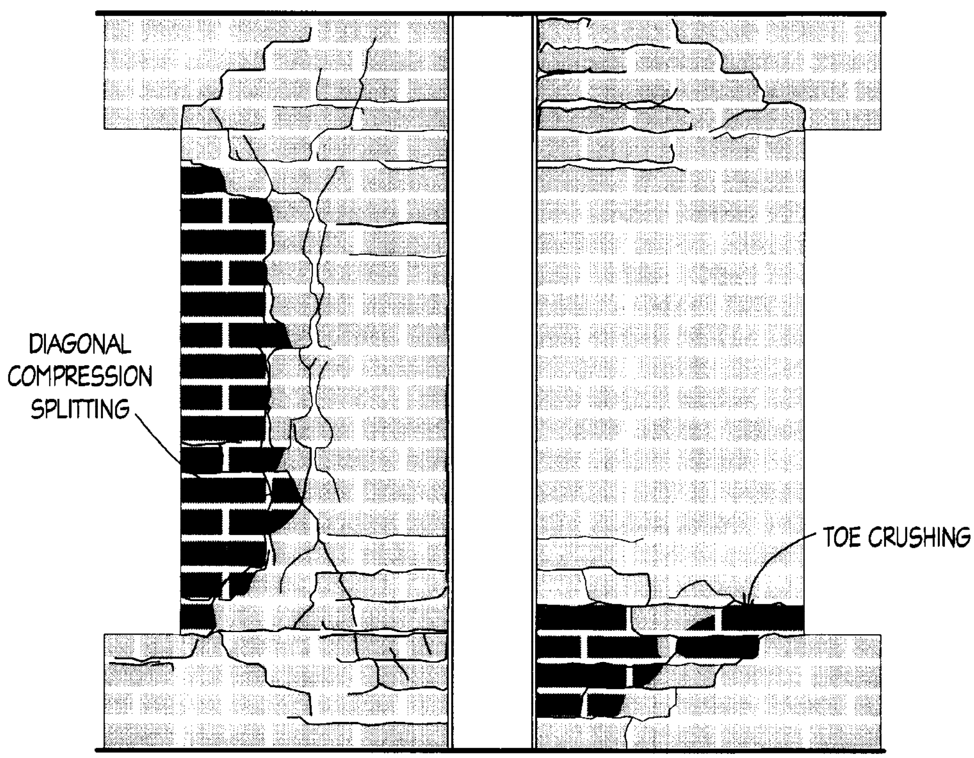

Fig. 8. Typical Crack Patterns, Specimens 2 and 3

CONCLUDING REMARKS

Although results from only two specimen of a more comprehensive test program were presented, several conclusions can be inferred:

1. Steel frames with unreinforced masonry infills exhibited reasonably ductile behavior. Most of the hysteretic energy was dissipated on the first cycle of each set of three same-amplitude displacement cycles. Subsequent cycles at the same drift level exhibited less energy absorption due to the opening and closing of cracks. However, after the frame experienced large drifts only the ductility of the bare steel frame was apparent. The masonry infill at these large drift levels was extensively cracked, and relatively ineffective with respect to the ductility of the composite system.

2. Load sharing of each component of the composite system depended on the drift level. At small drift levels the masonry pier shared as much as 90% of the total lateral load. After drifts of approximately 1.4% were imposed however, the total horizontal load shared by the masonry piers was reduced to about 30% or less.

3. Of the strength contributed by the masonry infill, the double wythe infill was almost twice the strength of the single wythe infill. However, the double wythe infill offered a larger share of the lateral shear strength, at higher drifts, compared to the single wythe infill.

4. The elastic stiffness of the initial composite system was significantly larger than that of the bare steel frame. The contribution from the masonry pier on the stiffness however, was minimal after the composite system experienced approximately 0.8% drift.

5. Diagonal struts created diagonal compression splitting that precipitated the loss of bricks adjacent to the window opening, and hence reduced the effective pier width. This minimized the participation of the masonry infill to the strength and stiffness of the steel-masonry composite system.

While experimental results from only two specimens are shown, similar trends in test results were observed from the other specimens.

ACKNOWLEDGMENTS

This research is sponsored by the National Center for Earthquake Engineering Research Center [NCEER] under grant SBC SUNY 93-3113 & 94-3111 NSF. This funding is gratefully acknowledged. Opinions expressed in this paper are those of the authors, and do not necessarily reflect the opinion of NCEER or NSF.

REFERENCES

Freeman, S.A. (1991). Behavior of Steel Frame Buildings with Infill Brick. *Proceedings of the 6th Canadian Conference on Earthquake Engineering*, Toronto, Canada.

Freeman, S.A. (1994). The Oakland Experience During Loma Prieta -- Case Histories. *Proceedings from the NCEER Workshop on Seismic Response of Masonry Infills, Technical Report NCEER 94-0004,* National Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, N.Y.

Hamburger, R.O., and A.S. Chakradeo (1993). Methodology For Seismic Capacity Evaluation of Steel-Frame Buildings with Infill Unreinforced Masonry. *Proceedings of the 1993 National Earthquake Conference; Earthquake Hazard Reduction in the Central and Eastern United States*, Memphis, TN.

Mander, J.B., B. Nair, K. Wojtkowski, and J. Ma (1993). An Experimental Study on the Seismic Performance of Brick-Infilled Steel Frames With and Without Retrofit. *Technical Report NCEER 93-0001*, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, N.Y.

Mander, J.B., L.E. Aycardi, and D.K. Kim (1994). Physical and Analytical Modeling of Brick Infilled Steel Frames. *Proceedings from the NCEER Workshop on Seismic Response of Masonry Infills, Technical Report NCEER 94-0004,* National Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, N.Y.