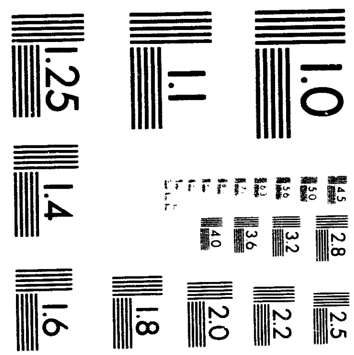
1.0
1.1
1.25
1.4
1.6
1.8
2.0
2.2
2.5
2.8
3.2
3.6
4.0

CONF-931010 2--39

RECEIVED
DEC 07 1993
OSTI

Y/EN 5073

MARTIN MARIETTA

THE EFFECT OF PRIOR OUT-OF-PLANE DAMAGE ON THE IN-PLANE BEHAVIOR OF UNREINFORCED MASONRY INFILLED FRAMES

Prepared for the

Center for Natural Phenomena Engineering

August 25, 1993

For Presentation at
The Fourth DOE Natural Phenomena Hazards Mitigation Conference
Atlanta, GA
October 1993

Prepared by the
Oak Ridge Y-12 Plant
Oak Ridge, Tennessee 37831
managed by
Martin Marietta Energy Systems, Inc.
for the
U.S. DEPARTMENT OF ENERGY
under Contract DE-AC05-84OR21400

MANAGED BY
MARTIN MARIETTA ENERGY SYSTEMS, INC.
FOR THE UNITED STATES
DEPARTMENT OF ENERGY

Y/EN 5073

THE EFFECT OF PRIOR OUT-OF-PLANE DAMAGE ON THE IN-PLANE BEHAVIOR OF UNREINFORCED MASONRY INFILLED FRAMES

R. Craig Henderson
W. Dale Jones
E. G. Burdette
Max L. Porter

Prepared for the
Center for Natural Phenomena Engineering

August 25, 1993

For Presentation at
The Fourth DOE Natural Phenomena Hazards Mitigation Conference
Atlanta, GA
October 1993

Prepared by the
Oak Ridge Y-12 Plant
Oak Ridge, Tennessee 37831
managed by
Martin Marietta Energy Systems, Inc.
for the
U.S. DEPARTMENT OF ENERGY
under Contract DE-AC05-84OR21400

The Effect of Prior Out-of-plane Damage on the In-plane Behavior of Unreinforced Masonry Infilled Frames

R. C. Henderson and W. D. Jones
Center for Natural Phenomena Engineering
Martin Marietta Energy Systems, Inc.
Oak Ridge, TN 37831

E. G. Burdette
223 Perkins Hall
The University of Tennessee
Knoxville, TN 37996-2010

M. L. Porter
416A Town Engineering Bldg.
Iowa State University
Ames, Iowa 50011

In order to address the effect of prior out-of-plane damage on the in-plane behavior of unreinforced masonry infills, two full-scale (24 feet tall by 28 feet long) structural clay tile infills and one frame-only (no infilling) were constructed and tested. The infilled frame, consisting of two wide flange columns surrounded by masonry pilasters and an eccentric wide flange purlin, was identical to many of the infills located at the Oak Ridge Y-12 Plant*. The masonry infill was approximately 12.5 inches thick and was composed of individual four- and eight-inch hollow clay tile (HCT) units. One of the infill panels was tested out-of-plane by four quasi-static actuators - two on each column. The test structure was deflected out-of-plane equally at all four actuator locations in order to simulate the computed deflection path of the top and bottom chords of a roof truss framing into the columns at these locations. Prior to the infill testing, a bare frame was loaded similarly in order to determine the behavior and stiffness contribution of the frame only. Following the out-of-plane test of the infilled panel, the structure was loaded in-plane to failure in order to ascertain residual strength. A second, identical infilled frame was then constructed and tested in-plane to failure. In this way, in-plane behavior with and without prior out-of-plane damage could be established and compared. For both out-of-plane and in-plane testing, reversed-cyclic quasi-static loading was used in order to obtain full tension/compression hystereses. Also, natural frequencies of the first infilled panel were determined before and after the out-of-plane testing. This paper describes the test series and discusses the conclusions pertinent to the effect of out-of-plane cracking on in-plane stiffness and behavior.

* Managed by Martin Marietta Energy Systems, Inc. for the U. S. Department of Energy under contract DE-AC05-84OR21400

INTRODUCTION

The use of structural steel frames infilled with unreinforced masonry (URM) is a common construction technique, particularly in older industrial facilities. Infilled frames are very efficient structural elements in that the construction process is uncomplicated and the resulting structure responds to in-plane lateral loads with significant stiffness and ductility [1, 3]. Because of the stiffness of the infill material and the confinement provided by the surrounding frame, the infilled structure demonstrates much greater strength and ductility than either of the two materials acting independently.

Infilled frames subjected to seismic forces must typically resist earthquake components both in-plane and out-of-plane. Often, when buildings are composed of infilled walls in one direction and moment resisting frames in the orthogonal direction, out-of-plane damage to the URM results from seismic drift (as opposed to inertial forces of the infill material). Tests have shown that small amounts of out-of-plane drift may produce full crack patterns that completely penetrate the thickness of the infill material [2, 3]. Hence, the URM in most infilled frames has likely experienced some degree of post-elastic behavior (i.e., mortar joint cracking, degradation of the frame to infill connectivity, etc.) over the structure's full displacement history. Therefore, the effect of prior damage on the in-plane behavior and capacity of infilled frames is important to understand.

Toward this end, Martin Marietta Energy Systems, Inc. and Iowa State University have tested two, full-scale frames infilled with hollow clay tile (HCT) as shown in Figure 1 and one, full-scale steel frame without infilling. Reversed-cyclic, sequentially-displaced, quasi-static loads were applied to the bare frame out-of-plane in order to determine a baseline stiffness and rotational spring constants for the column to floor connections. The first infilled frame was subjected to significant out-of-plane, quasi-static deflection followed by in-plane quasi-static loading to failure. The second infilled frame was tested to failure by applying in-plane, quasi-static loads without prior damage. This paper discusses the effect that prior damage has on the in-plane behavior and capacity of infills.

TEST STRUCTURE

The type of construction used in this testing is unique in that an attempt was made to replicate as closely as possible the in situ conditions of many of the walls at the Department of Energy's Y-12 plant (circa 1940). The construction process, also described in Reference [2], was as indicated in the following paragraphs. The two wall specimens were full-scale (24 feet tall by 28 feet long) hollow clay tile infills constructed with cores running horizontally. The wall panels were double wythe and built with a mortar mix conforming to ASTM C 270 Type N mortar. The walls were composed of four- and eight-inch HCT with a 0.75-inch collar joint so that the full wall thickness was approximately 12.5 inches. The HCT were laid with running bond; however, the construction was somewhat atypical in that the four- and eight-inch block alternated positions from course to course (i.e., there was no full-height collar joint), thereby creating composite wall behavior. Bed joints were 1/2-in. thick, full and continuous, and head joints were 3/8-in. thick with mortar applied to the face shells only. No reinforcement was used in the masonry.

Red clay (terra cotta) tile units manufactured to comply with ASTM C 34 Grade LBX (structural clay load-bearing wall tile) specifications were used to construct the infills. These cored tile units were sampled and tested in accordance with ASTM C 67. Likewise, representative mortar cube specimens were made during construction of the walls and tested at 7, 14, 21 and 28 days in accordance with ASTM C 109 as were six-inch mortar cylinders. The average 28-day mortar compressive strength for the cylinders and cubes was 1700 psi and 1900 psi, respectively. Additionally, twelve bond wrench specimens for the first wall and eleven for the second wall, for a total of twenty-three specimens, were constructed and tested per ASTM C 1072. The average ultimate bond strength was 110 psi with no distinguishable difference between the four- and eight-inch blocks.

Also, a parallel study on the prism compression strength of the block and mortar combination used in the full-scale wall testing was performed by the National Institute for Standards and Technology as per ASTM E 447. Twenty, 12.5-inch prisms were constructed and tested (10 parallel to the cores and 10 normal to the cores). The average parallel and normal compressive strengths on the gross section were 880 psi and 480 psi, respectively.

Wall-to-column connectivity was developed via 25-inch wide by 16.5-inch thick pilasters that were composed of four- and six-inch HCT. The four- and eight-inch blocks used in the wall construction did not extend into the column flange area; however, studies of the in situ wall-to-column interfaces, from which the test structure was constructed, indicated that considerable keying of the wall to the column existed as a result of substantial scrap and mortar inside the column flanges as shown in Figure 2. The effectiveness of the wall-to-column connection was further augmented by horizontal wall cores and vertical pilaster cores, thereby allowing for the transfer of mortar from one course to an adjacent course during the construction process.

Minimal rotational resistance at the column to floor connection was desired as a further means of replicating in situ boundary conditions. However, extremely large uplift forces coupled with the high magnitude in-plane racking loads, precluded the use of typical "pinned" type connections. In an effort to satisfy high uplift forces and minimal rotational capacity, a portion of the column web was removed so that a single anchor bolt could be used as a tie down (See Figure 3). The out-of-plane shear resistance, lessened by the removal of a portion of the column web, was replaced by the connection of shear members to the column flanges. The anchor bolts extended through the base plate, a rocker plate, and the laboratory floor. The rocker plate had an area slightly greater than the cross-section of the W10 column and was used as a means of precluding prying action of the base plate and to more closely represent the rotational resistance of the in situ column-to-floor connections.

The structural steel frame for the frame-only testing as well as the infill tests consisted of two wide-flange columns (W10X33) and a wide-flange purlin (W16X36) as shown in Figure 4. The W16 purlin had coped flanges at the ends of its length and its web was bolted to the flanges of the W10 columns. The web of the W16 purlin was flush with the face of the wall (i.e., the inner flanges of the W16 were embedded in the wall) thereby creating an eccentricity in the in-plane load path. The columns were oriented such that weak-axis bending was in the plane of the wall (See Figure 1). The beam was oriented with its strong axis resisting vertical loads (See Figure 4).

TEST PROCESS

Similar to the wall construction process, which was specifically intended to replicate the existing infill structures at the Department of Energy's Y-12 plant, the intent of the test process was to simulate the method of application and intensity of load that can be expected from seismic forces in situ. The testing process, also described in Reference [2], was as indicated in the following paragraphs. The in situ out-of-plane forces are resisted by trusses that connect to the exposed flanges of the infilled frame columns at the top and bottom chord locations. Because of the stiffness of these trusses (in the plane of the truss) there is little differential, horizontal movement at the top and bottom chord point where the truss frames into the infill column. Furthermore, because of the stiffness of the trusses as compared to the out-of-plane stiffness of the infill, the truss drift essentially governs the out-of-plane motion (behavior) of the system. For these reasons, the infilled frame drifts out-of-plane in response to the lateral loads applied by the trusses. These out-of-plane forces can be simulated by four actuators -- one at each of the top and bottom chord locations -- that cycle with equal deflections. Thus, the bare frame and the first infill panel were tested out-of-plane by four quasi-static actuators -- two on each column (See Figure 5).

The in situ in-plane forces are transmitted from the steel trusses and the roof diaphragm into the purlin which connects to the infill as well as to the W10 columns as shown in Figure 4. The in-plane deflection-controlled loading was applied at the purlin location through an extension arm by a 200-kip tension/compression actuator. The following sections describe the out-of-plane followed by in-plane testing of the first infilled panel as well as the in-plane testing of the second infilled panel.

OUT-OF-PLANE TESTING (1st INFILL)

The first infilled panel was deflected out-of-plane equally at all four actuator locations in order to simulate the computed deflection path of the top and bottom chords of a roof truss framing into the columns at these locations (the computed deflection assumes no arching action of the infill). The out-of-plane, quasi-static loading process consisted of incrementally increasing cyclic displacements until the predetermined deflection or a desired limit state was achieved.

The instrumentation used in the first infilled frame test consisted of 28 displacement transducers (8 on the pilasters and 20 on the wall as shown in Figure 5), 15 quarter-bridge strain gages (6 on the outer flanges of each column and 3 on the outer flanges of the purlin), and 4 crack-o-meters for measuring horizontal crack width at the base and lower actuator locations. The out-of-plane deflection limit for the infilled panel was 2.6 inches and was selected such that significant damage would be applied to the infill out-of-plane before continuing the test sequence with in-plane loading. Reversal of the cycling direction occurred at every 0.05-inch displacement increment.

The four actuators were driven by four, independently-acting hydraulic hand pumps. Hand pumps were used to maintain control of the applied deflections to within 0.02 inches of differential displacement (less than 1% of full displacement). The minimization of differential displacement at the actuator locations was necessary to avoid torsion of the test structure and to ensure uniform and symmetrical behavior as would occur in situ.

IN-PLANE TESTING (1st and 2nd INFILL)

In-plane, quasi-static loading was applied to both of the test structures at the purlin location as shown in Figure 6. The W16 purlin was extended out and attached to a 200-kip load cell and actuator for application of the in-plane load. Eight string potentiometers, four on each end of the test structures, were used to measure lateral displacement. Two diagonal deformation gages measured diagonal shortening and lengthening of the panel orthogonal to the compression strut. Additionally, 15 strain gages were placed at the outer flange edges of the columns and purlin in order to measure in-plane strain, and ultimately stress, in the frame.

The in-plane actuator was driven by a hand pump at the onset of the testing and, later, by a single electric pump. The hand pump was used initially in order to apply finely controlled reversing cycles at every 0.05 inches. Later, as the in-plane stiffness of the test structure degraded, an electric pump applied the load required for displacement cycles of 0.1 inches. Both infilled frames were cycled to a maximum displacement of 3.0 inches in tension and compression; however, only qualitative data (i.e., crack propagation and damage levels) were recorded for the first infill after an applied displacement of 2.0 inches.

TEST RESULTS

The out-of-plane and in-plane test results for both infills, also described in Reference [3], were as indicated in the following paragraphs.

OUT-OF-PLANE (1st INFILL)

Thorough preliminary analyses of the test structure using finite elements were performed prior to testing in order to predict the magnitude of actuator loads as a function of applied deflection as well as to determine behavior and crack patterns. Damage to the test structure as a result of applied out-of-plane displacements was, generally, as anticipated. Overall panel damage was primarily confined to bedjoint cracking (i.e., horizontal cracks formed at the mortar interface between structural clay tile courses); however, some headjoint cracking (i.e., vertical cracks formed at the mortar interface between adjacent masonry units in the same course) did occur. Damage to the HCT units themselves was extremely rare and consisted of localized, vertical microcracking at the edges of the HCT blocks near the bed joints.

At the onset of testing the infill behaved as if fixed at the base; thus, the highest bedjoint stresses, those resulting from negative moment, were between the floor and the first course. A base crack began to appear in the regions near both columns at an applied displacement of 0.1 inches and propagated inward toward the centerline. As the migration of the base crack from each column was approximately half complete, a second crack pattern at the lower actuator location began near each of the columns. Because cracking at the base relieved some of the flexural stress, a continuous transition from double-curvature bending to single-curvature bending ensued. In a similar fashion, as complete crack patterns formed, flexural stresses were redistributed until the infill structure behaved, essentially, as a series of horizontal beams. The final crack pattern, at an applied displacement of 2.6 inches, is shown in Figure 7.

Figure 8 shows the out-of-plane, load-displacement hysteresis at the lower actuators. Initially, the infill panel was quite stiff, resisting the applied out-of-plane loads at 90 kips/inch. However, stiffness was significantly reduced with the formation of each bedjoint crack. At the completion of the out-of-plane testing, the lateral stiffness had been reduced to 18 kips/inch -- twenty percent of the original value. This final stiffness was approaching that of the bare frame, which was determined to be 10.8 kips/inch. The applied out-of-plane drift resulted in numerous fully-penetrating crack patterns, in addition to a corresponding loss in stiffness. However, because of the confinement provided by the structural steel frame, the infill panel was still completely stable and laterally resistive.

IN-PLANE (1st INFILL)

The first visible response to the applied displacement was widening of the vertical headjoint cracks that existed as a result of the out-of-plane loading. Soon thereafter, significant sliding of the upper portion of the structure was observed along the preformed, out-of-plane crack patterns shown in Figure 7. Differential movement between the top and bottom of the test structure was approximately 1/8 inch early in the loading sequence with maximum values of near 1/4 inch later in the testing.

Up to 0.75 inches of applied displacement, only small to moderate amounts of additional damage to the infill panel occurred. Cracks that did occur during this displacement range were characteristic mortar joint stair-step-cracks, propagating generally from the lower corners toward the upper corners. However, this cracking tended to be discontinuous and bounded by the pre-formed horizontal bedjoint cracks. Thus, it appears that when through-cracks of significant length are present, sets of discontinuous compression struts replace the single, diagonal compression strut typical of in-plane loads on a virgin structure [3].

The displacement range from 0.75 inches to 2.0 inches was marked by the onset of HCT block damage. The most severe block damage began and was concentrated in the upper corners on the purlin side and resulted from compression and buckling of the face shells caused by bearing of the steel frame on the infill material. The onset of full face shell spalling began at an applied displacement of just over 2.0 inches and propagated from the column/purlin juncture toward the center of the infill. Final damage to the infill at a displacement of approximately 3.0 inches was marked by large, symmetrical openings in the infill as indicated in Figure 9.

Figure 10 shows the in-plane, load-displacement hysteresis for the first infilled panel. Initial tensile stiffness was 790 kips/inch while the initial compressive stiffness was 390 kips/inch. This difference, attributable to greater out-of-plane damage on the compression side, diminished to near zero after the first few cycles. Loads gradually increased to the peak values of 49 kips (Δ = 1.15 inches) and 64 kips (Δ = 0.75 inches) for tension and compression, respectively. As illustrated by the smoothness of the hysteresis (and corroborated by the recorded cracking) damage occurred in a very flexible fashion without sudden stair-step-cracking and accompanying sharp drops in load or increases in deflection.

IN-PLANE (2nd INFILL)

The second infill panel was extremely stiff upon the application of in-plane loading and little or no visible movement or damage was recorded for the first two cycles (up to about 0.10 inches). However, as a deflection of 0.15 inches was reached on the compression stroke, loud popping occurred and stair step cracks, emanating from the compression toe diagonally to the center of the wall, were recorded on both sides of the test structure. This same phenomenon occurred on the tension stroke with a diagonal crack propagating from the opposite corner to the center of the panel. Separation of the infill material from the surrounding steel frame occurred at the same time or slightly before the first lower-structure diagonal crack and may have actually been a catalyst for this behavior.

As in-plane applied displacement was increased, numerous brittle diagonal cracks were formed in the upper portion of the infill. This cracking occurred on both sides of the test structure and was symmetrical in the upper corners. At approximately 1.3 inches of applied displacement, spalling of the face shells began on the frame side of the panel. This spalling began soon thereafter on the pilaster side and for both cases propagated from the center toward the column/purlin juncture. Though the interim behavior of the second infill was considerably different than that of the first, the final damage conditions were virtually identical (i.e., very similar to that shown in Figure 9).

Figure 11 shows the in-plane, load-displacement hysteresis for the second infilled panel. Initial tensile and compressive stiffnesses were 820 and 930 kips/inch, respectively. The maximum tensile and compressive loads of 51 kips ($\Delta = 0.10$ inches) and 61 kips ($\Delta = 0.12$ inches) were achieved on the third hysteretic loop. Immediately thereafter, a dramatic loss in load capacity occurred which corresponded to the first incidence of diagonal cracking on the tension and compression strokes. After reaching approximately 1.2 inches of displacement, the load declined almost linearly for both tension and compression with a final in-plane stiffness of approximately 5 kips/inch.

CONCLUSIONS

The comparison of in-plane data for the two infilled panels indicates that prior damage reduces initial in-plane stiffness. Also walls with prior damage respond to loads in a much less brittle fashion than those with no prior damage. This is in direct contrast to virgin structures that store considerable potential energy under the first few cycles of applied displacement and release it in a sudden fashion through brittle diagonal stair-step-cracking. The primary conclusion from the complete test sequence is that prior damage to infill structures has little effect on the overall in-plane performance or the final damage state as long as confinement by the frame is maintained. This fact is very important to the analysis of most older structures, which have, typically, been subjected to some cracking [4]. The response that would logically come from conclusions drawn from this research is a reassessment of the current design standards for infilled masonry construction.

REFERENCES

1) Dawe, J. L., Seah, C. K., "Lateral Load Resistance of Masonry Panels in Flexible Steel Frames," Proceedings, 8th Int'l Brick and Block Masonry Conference, Ireland, 606-616, 1988.

2) Henderson, R. C., Jones, W. D., Porter, M. L., "Factors Affecting the Ductility of Double-Wythe Masonry Infills Subjected to Seismic Drift." Proceedings, 6th North Amer. Masonry Conference, Philadelphia PA, 1433-1438, 1993.

3) Henderson, R. C., "Experimental and Analytical Investigation of Out-of-plane and In-plane Seismic Drift on Unreinforced Masonry Infilled Frames." Ph.D. dissertation (draft), Civil Engr. Dpt., The University of Tennessee, Knoxville, TN, 1993.

4) Langenbach, R., "Earthquakes: A New Look at Cracked Masonry," Civil Engineering, Vol 62, No. 11, 56 - 58, November 1992.

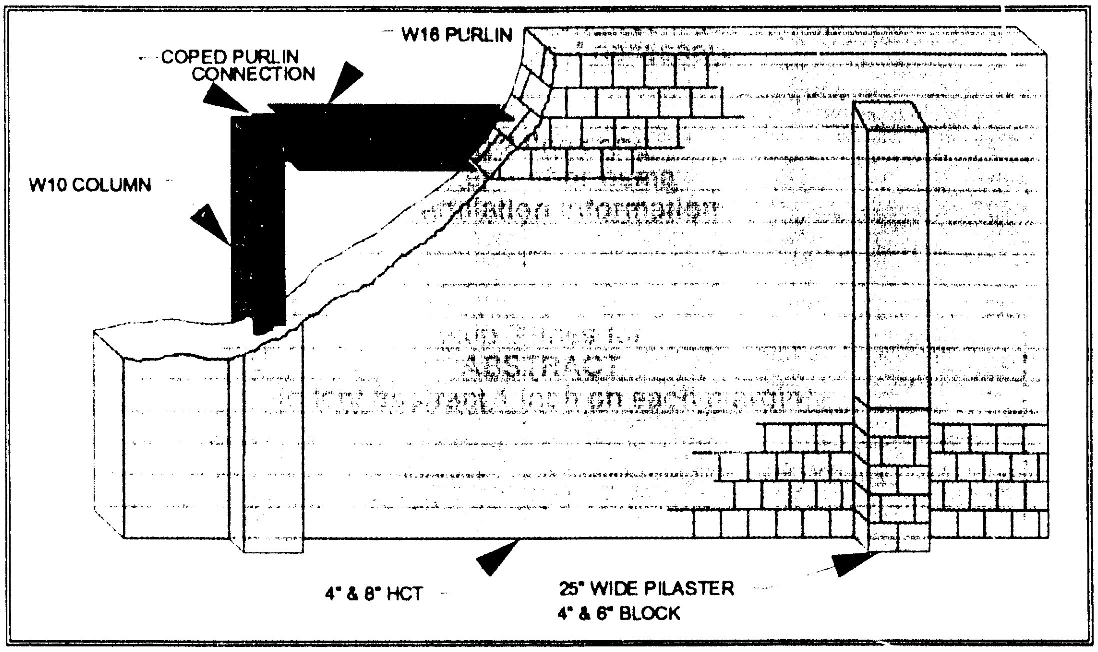

FIGURE 1: INFILL # 1 and # 2

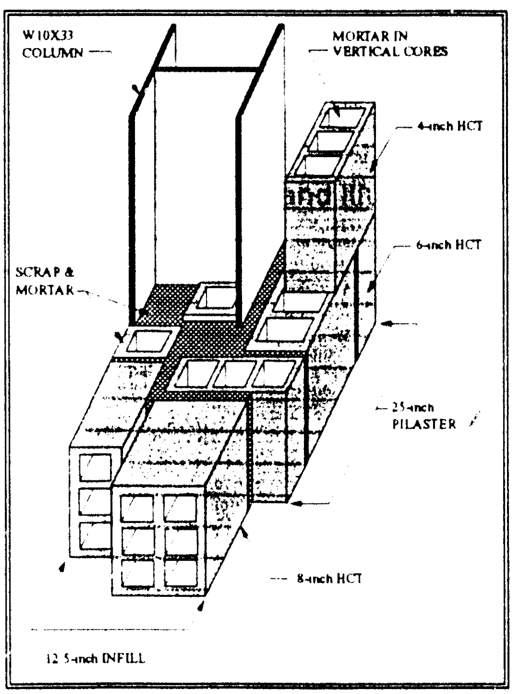

FIGURE 2: PILASTER CONFIGURATION

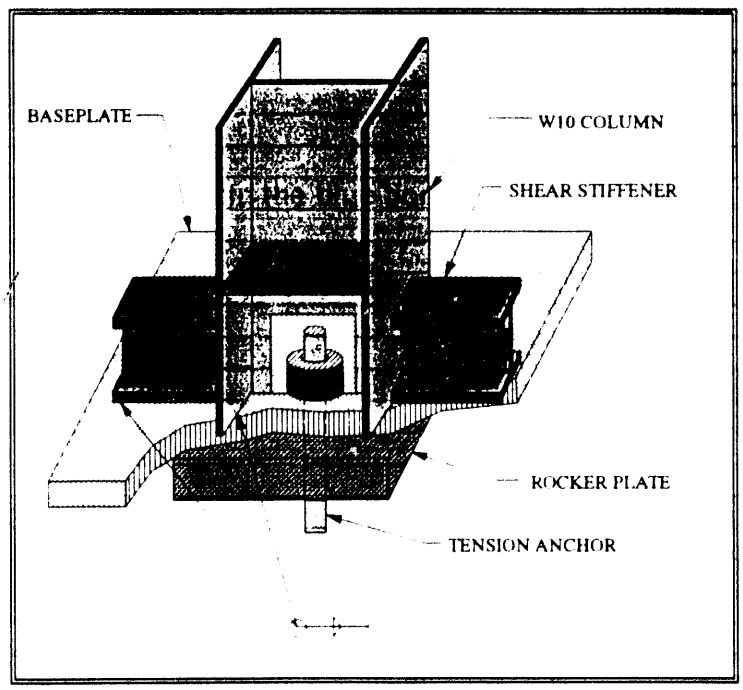

FIGURE 3: COLUMN TO FLOOR CONNECTION

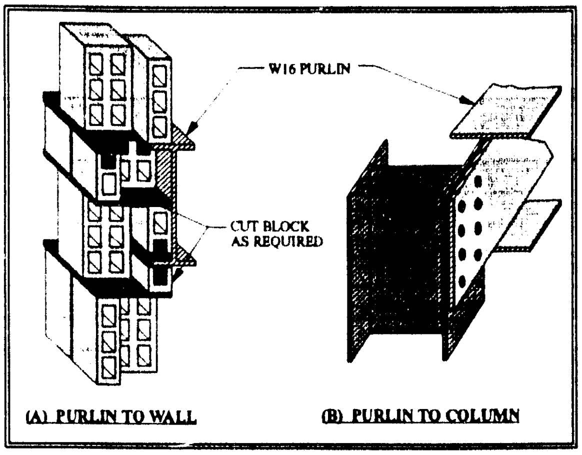

FIGURE 4: PURLIN CONNECTIONS

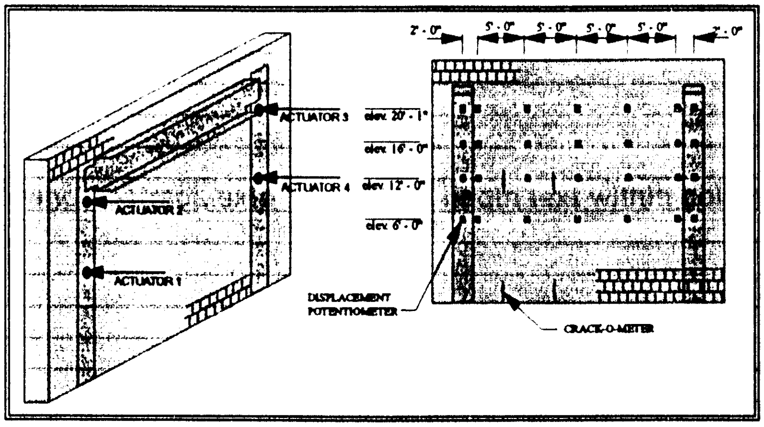

FIGURE 5: OUT-OF-PLANE EQUIPMENT

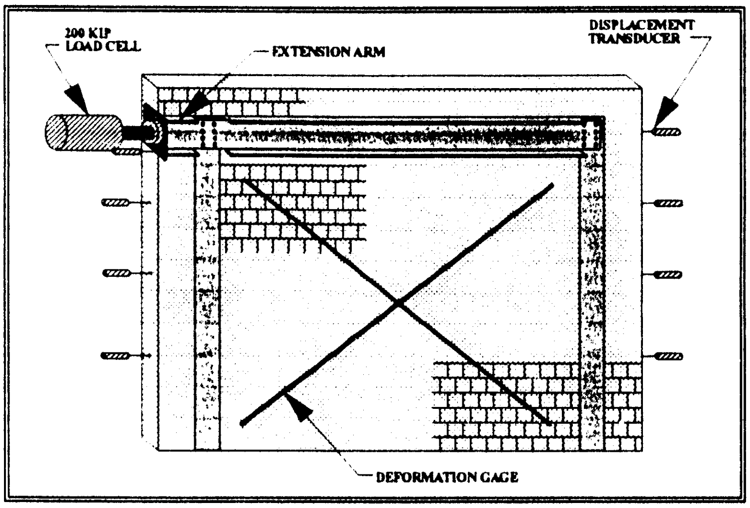

FIGURE 6: IN-PLANE EQUIPMENT

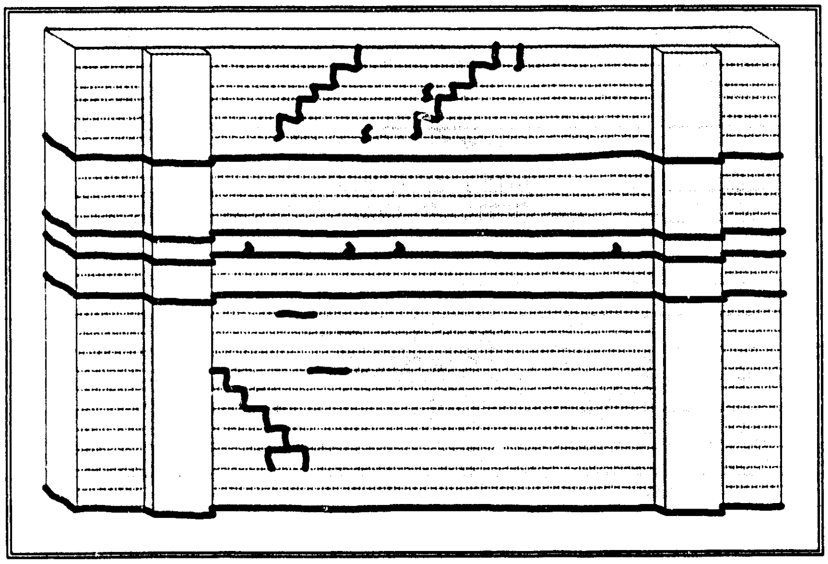

FIGURE 7: OUT-OF-PLANE CRACK PATTERN

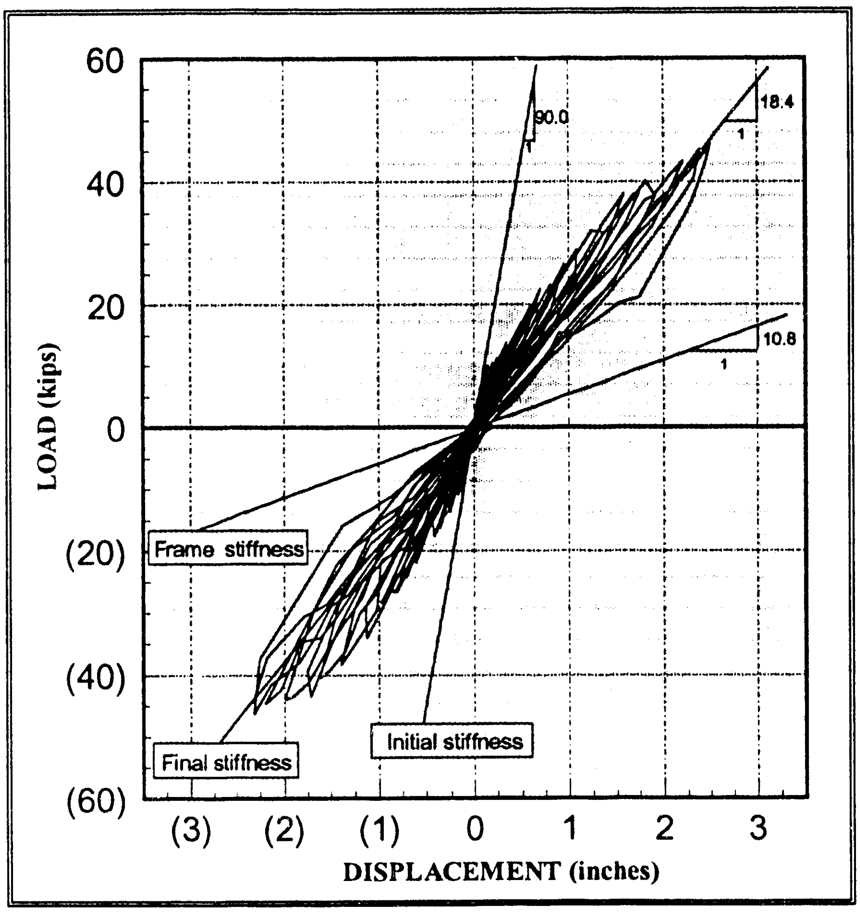

FIGURE 8: OUT-OF-PLANE HYSTERESIS (INFILL # 1)

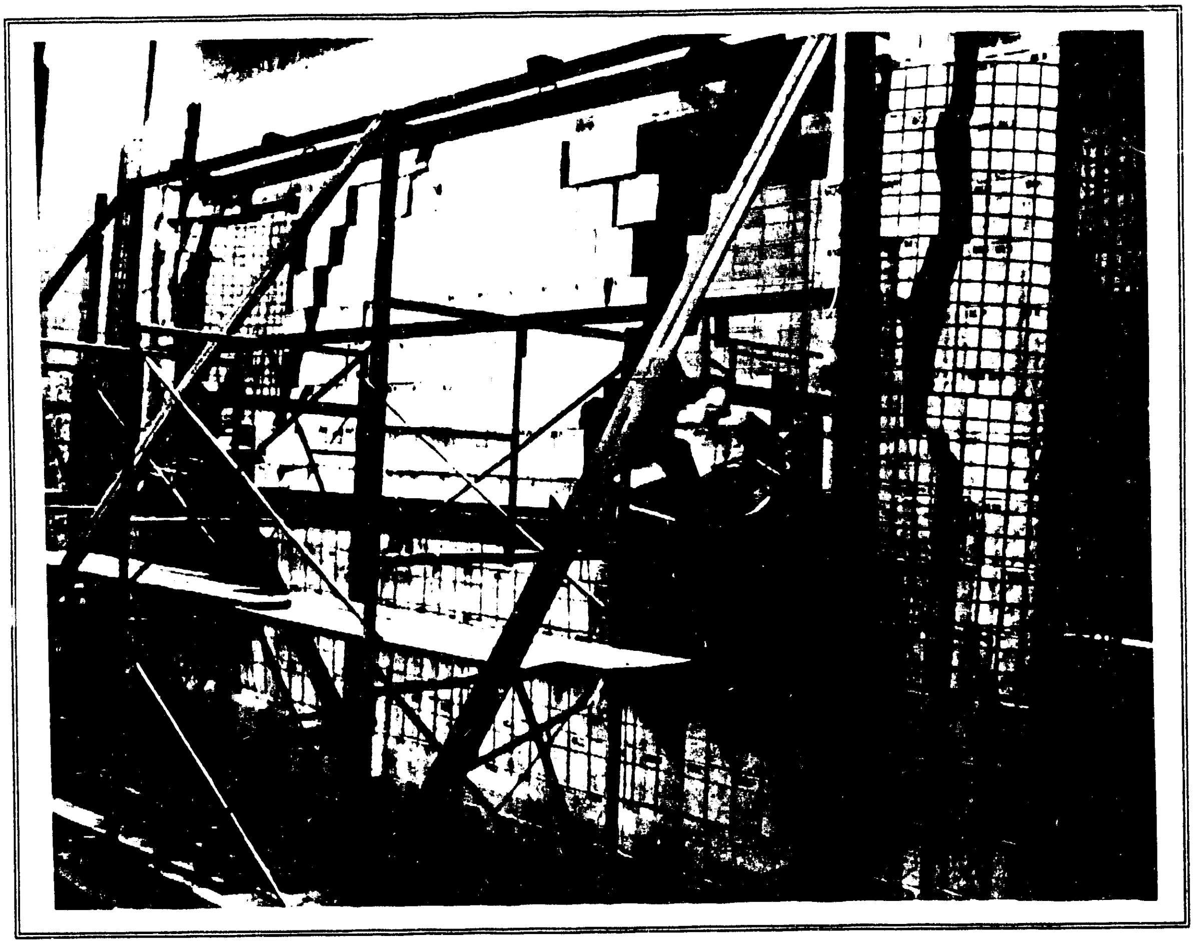

FIGURE 9: FINAL IN-PLANE DAMAGE

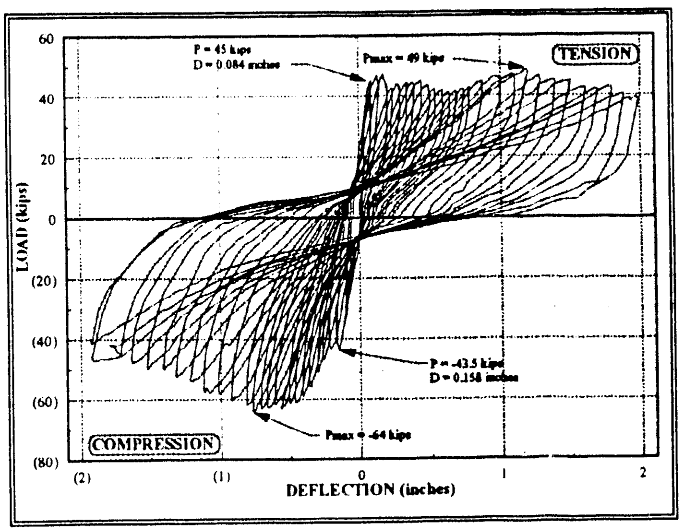

FIGURE 10: INFILL # 1 HYSTERESIS (I-P)

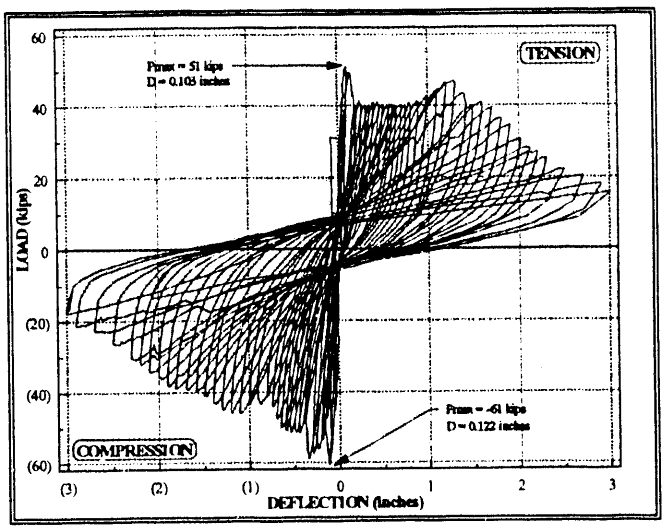

FIGURE 11: INFILL # 2 HYSTERESIS (I-P)

DISTRIBUTION

J. E. Beavers	9207, MS 8083
M. S. Guthrie (2)	9739, MS 8117
R. C. Henderson	9207, MS 8083
W. D. Jones	9207, MS 8083
Y-12 Central Files	9711-5, MS 8169
A. K. Lee/DOE-OSTI (2)	9731, MS 8175

DATE
FILMED
1 / 14 / 94

END