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SUMMARY OF LARGE- AND SMALL-SCALE UNREINFORCED MASONRY TEST PROGRAM

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ABSTRACT: A five-year, large- and small-scale, static and dynamic experimental research program, in which more than 700 tests were conducted, has demonstrated that unreinforced masonry infills are more ductile and resist lateral loads more effectively than anticipated by conventional code procedures. The tests were conducted both in the laboratory and on existing structures at the Department of Energy's Y-12 National Security Complex[*]. The experimental data indicate that the combination of a steel frame and infill material efficiently resists lateral loads – the infilling provides significant lateral stiffness while the surrounding frame adds ductility and confinement to the overall system. The results from approximately 25 moderate- and full-scale tests on infills showed that with simulated seismic loads, the frames confined the masonry, and the load-carrying capacity of the infill was considerably above the load that caused initial cracking. This finding was a significant departure from classical code approaches that assumed first cracking to be failure of an unreinforced masonry wall. The experimental program, performed for the US Department of Energy, consisted of the following large-scale tests on infills: in situ airbag pressure testing, shake-table tests, and the application of quasi-static in-plane and out-of-plane drift loads. This paper provides a summary of the overall experimental methodology and results.

INTRODUCTION

An article in *Civil Engineering* entitled "Earthquakes: A New Look at Cracked Masonry" (Langenbach, November 1992) describes a dilemma familiar to many who design or retrofit masonry structures – particularly masonry infills. That is, building codes equate cracked masonry with structural failure. The author, writing about conditions in Oakland, California three years after the Loma Prieta earthquake, says "Economics, fears of liability, and a strict damaged buildings repairs ordinance have contributed to an extensive delay in the repair of masonry infill buildings … several of the most significant historic downtown office buildings remain abandoned and threatened with demolition." He links the problem with the approach taken by structural codes regarding unreinforced masonry in seismic zones: "Unreinforced masonry is not allowed. Rather than giving the masonry the credit it is due, the code encourages engineers to treat it only as dead load or use very conservative values for its strength."

Though it has been almost ten years since the above-mentioned article was written, building codes still regard unreinforced masonry infills in much the same manner. Therefore, retrofit is often inordinately expensive, and engineers do not consider infills as a design option for new buildings despite their construction efficiency. The analysis of infilled frame buildings at the Department of Energy's (DOE) Y-12 National Security Complex (referred to hereafter as Y-12) met with similar constraints due

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to lack of code guidance on these types of structures. Ultimately, a five-year, $10 million test program was completed which addressed many of the pertinent strength and behavior issues (Henderson et al 1995). This first paper in a two-part series describes the components of the research program and respective results. The second paper details the translation of the test results into an analysis protocol for Y-12. It is anticipated that the test results described herein may assist in the formation of pertinent infill design code.

Historical Perspective

Many of the buildings at Y-12, constructed during the 1940s and 1950s, consist of steel and/or reinforced concrete framing infilled with structural clay tile (SCT), as shown in Fig. 1. The infill was intended to provide for building enclosure and was not designed to have vertical or lateral load-carrying capacity. During the late 1970s and early 1980s, seismic and wind evaluations were performed on many of these buildings. To make the analytical modeling as accurate as possible, the stiffening effect of the infills was incorporated by assuming that the infill would respond as an elastic shear wall until cracking (shear failure) occurred. The capacity of the shear wall was based on building code allowable shear values.

FIG. 1. Section of wall Infilled with Structural Clay Tile

Research in the 1950s and 1960s (Holmes 1961, Stafford-Smith 1966) on unreinforced masonry (brick) infills had shown that the walls could be modeled as a strut formulation. However, because of the unique infill construction in the Y-12 facilities and the fact that no research had been conducted on the strut formulation for SCT, analysts investigating Y-12 buildings believed that a shear wall representation would be more appropriate.

In parallel with the analysis effort during the 1970s and 1980s, DOE also began developing the *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* (UCRL-15910, Kennedy et al. 1985). This document specified requirements for new buildings and, for the first time, specified natural phenomena capacity and performance criteria for existing buildings. However, the document did not specify guidelines for determining the lateral force capacity of infilled frames. When the results of the initial seismic and wind evaluations were compared with the new criteria, the projected building capacities fell short of the requirements. Apparently, if the buildings were going to meet the new criteria, many millions of dollars would be required for building upgrades.

The Issues

Because the upgrade costs were significant, the assumptions and approaches used in the analyses were reevaluated. Four issues were identified:

1. Once the infill walls cracked, what capacity (nonlinear response), if any, would the walls have to resist earthquake or wind loads applied in the plane of the infill (in-plane)?
2. Would the infilled walls remain within the steel or reinforced concrete framing when subjected to earthquake or high wind loads applied perpendicular to the infill (out-of-plane)?
3. What was the actual shear capacity of the structural clay tile infill?
4. Was modeling the infill as a shear wall the best approach?
5. Was the SCT likely to exhibit explosive failure characteristics under dynamic loads?

Establishment of the Experimental Program

As analytical work proceeded at Y-12 Plant, many buildings similar in construction to those being analyzed were subjected to the 1985 Mexico City earthquake, and their performance was studied (Miranda and Bertero 1989, Adham 1985). This research indicated that the infills at Y-12 might indeed (1) have significant postcracking capacity in-plane; (2) remain in place during out-of-plane loading; and (3) have a shear capacity of up to 80 psi (Williamson 1987). These results suggest that the engineering approach used in the early evaluations may have significantly underestimated the performance of the buildings when subjected to seismic and wind loads. Therefore, to enable further quantification of the seismic and wind capacity of the infilled buildings, the test program was established (Martin Marietta Energy Systems 1991). The goal of the testing was to perform micro- and macro-experimental research on test structures representative of Y-12 buildings so that an analysis methodology more indicative of the actual performance of the walls could be developed.

THE EXPERIMENTAL PROGRAM

Purpose and Overview

The primary purposes of the experimental research program were to accomplish the following:

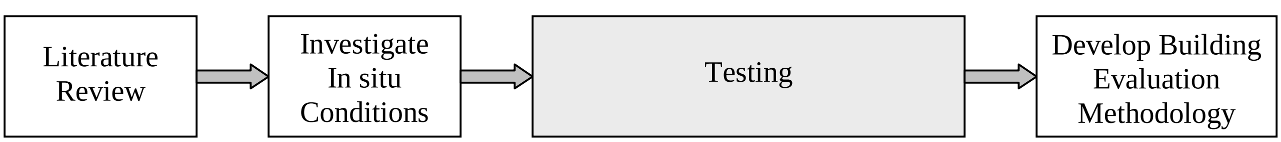
1. Assess the strength and behavior of SCT infills when resisting earthquake and wind loads, both in-plane and out-of-plane;
2. Understand how the infill material interacts with the steel or reinforced concrete framing when subjected to in-plane and out-of-plane forces;
3. Determine whether in-plane and out-of-plane behavior is coupled and, therefore, a three-dimensional method of determining the response is necessary; and
4. Develop a predictive analytical method that is more representative of the actual performance of buildings constructed of SCT infills than that offered by conventional code approaches.

The original plan for the test program consisted of 20 test types with multiple tests in each area. Not all of the tests were completed as originally planned, and in a few cases, a much larger number of specimens was tested. A summary of all completed tests is presented in Table 1. As the test team's knowledge of masonry research broadened, a clearer understanding of the applicability of individual tests was achieved.

Table 1. Test Program Summary

	In situ	**Lab**
Unit tile compression	100	135
Unit tile splitting tensile	27	40
Unit tile initial rate of absorption	24	64
Mortar compression	18	
Morter tensile splitting	0	6
In situ morter bed shear test	18	0
Deformability	2	0
Bond wrench	52	48
Prism compression	3	73
In situ normal flatjack	5	0
Full scale out-of-plane air bag	1	3
In-plane infilled frame	0	15
Out-of-plane and in-plane drift	0	2
Shake table (one structure) • Out-of-plane system • In-plane system • In-plane sine sweep	 0 0 0	 7 13 2
Coupon tests • Compression • Tensile • Modulus of rupture • Moisture absorption		 4 3 3 15
Moisture absorption	24	24
Miniature prism compression	0	4
Prism compression at an angle	0	9

The following four phases of investigation represent the chronology and order by which the test program was successfully completed:



The intent of this paper is to provide an overview of the primary components of the testing phase of the program, including:

1. Constitutive property tests.
2. Large-scale in situ air bag pressure testing;
3. Large-scale quasi-static drift tests;
4. Dynamic shake-table tests; and
5. Moderate-scale laboratory tests.

Constitutive Property Tests

Test Description: Many of the tests performed for the overall test program were conducted on specimens constructed with new materials. To correlate the relationship between new and in situ material behavior, constitutive property tests were performed on specimens made from new structural clay tile and specimens constructed from existing structural clay tile removed from Y-12 buildings. The three primary constitutive property tests that were performed on both old and new material specimens were: 1) unit block; 2) prism compression; and 3) bond wrench. Also, mortar characterization tests were performed to ascertain the mix ratio of the existing mortar, and numerous mortar property tests were performed on the new mortar used in the construction of the large-scale test specimens. A Type N masonry cement was used for the mortar throughout the test program, based on the mortar characterization of the in situ conditions at Y-12. Finally, low-level elastic properties were determined using flatjacks (i.e., normal stress testing and deformability testing) on in situ structural clay tile walls; and push tests were conducted to determine in situ shear capacity at the bed joint. A summary of the primary constitutive testing that was performed on old and new specimens is given in Table 2.

Table 2. Summary of Primary Constitutive Property Tests

Test	Old Materials	New Materials
Unit Block	<u>168 SCT Units</u> The specimens, two sizes of 10.16 and 20.32 cm (4 and 8 in.), were grouped to test for one or more of the following behavioral characteristics: • Initial Rate of Absorption • Absorption • Modulus of Rupture • Splitting Tensile Strength • Compressive Strength • Elastic Modulus	<u>168 SCT Units</u> The same number, sizes, and test types were performed on new SCT units as listed for old SCT units.
Prism Compression	<u>3 Prisms</u> The prisms were extracted (sawed) from existing walls and had the following characteristics: • Prism 1: 33.02 cm (13 in.) thickness with compression normal to the core • Prism 2: 20.32 cm (8 in.) thickness with compression normal to the core • Prism 3: 20.32 cm (8 in.) thickness with compression parallel to the core	<u>78 Prisms</u> The prism specimens were grouped according to one or more of the following characteristics: • 20.32 cm (8 in.) thickness • 33.02 cm (13-in.) thickness constructed of 10.16- and 20.32-cm (4 and 8 in.) units • Compression normal to core • Compression parallel to core • Compression at angle to core
Bond Wrench	51 8-in. specimens	23 Bond wrench specimens (12 4-in. and 11 8-in.)

Because many of the large-scale structures were comprised of 10.16- and 20.32-cm (four- and eight-inch) SCT, effort was made to test statistically appropriate samples of each block size. The compression test results of unit blocks and either constructed or in situ prisms are indicative of the strength of the larger structural element. Fig. 2 shows a photo of a prism being tested to failure under an applied compressive load. The 33.02-cm (13-inch) prism, constructed of 10.16- and 20.32-cm (4- and 8-in.) SCT in side construction, was built in the laboratory to match in situ conditions and then capped with gypsum. The specimens were instrumented with strain gages and linear variable displacement transducers connected to a data acquisition system.

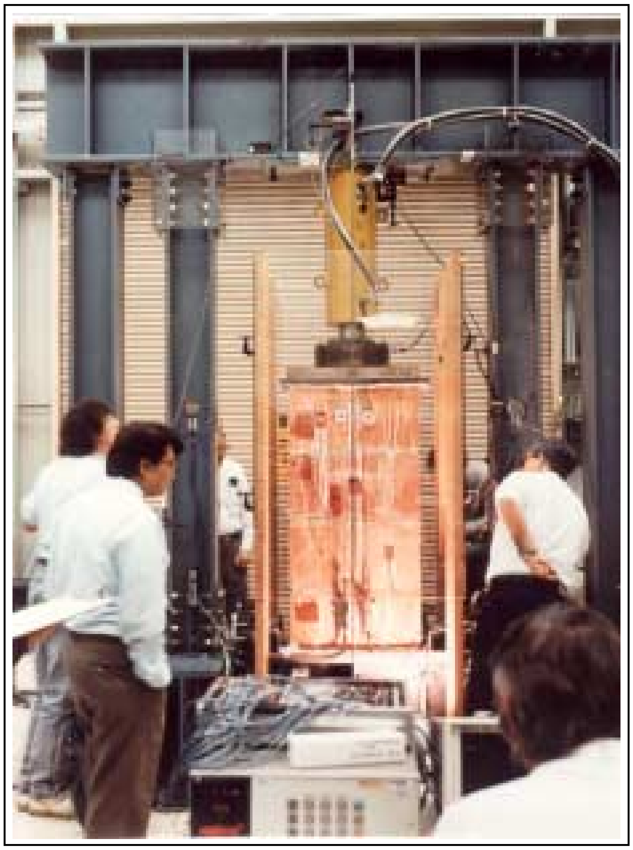


FIG. 2. Prism Test Setup

Bond wrench testing provides a measure of the flexural capacity (strength of the bond between mortar and unit) of an unreinforced masonry wall. The test apparatus consisted of a clamping bracket with a loading arm that is attached to the masonry unit selected for testing. Because of the eccentricity of the applied load, flexural/tensile stresses were maximized at the bed joint of the two-course specimen. Load was uniformly applied to the loading arm until failure of the connective bond occurred between the upper and lower blocks. The applied load was measured with a compression load cell in conjunction with a digital multimeter.

Test Objectives: The purposes of the <u>unit block</u> tests were (1) to determine the properties of new tiles to be used in laboratory testing of assemblages (prisms, infilled frames, etc.) and (2) to correlate the properties of the new and old tiles to aid in analyzing the existing structures.

Likewise, the prism compression tests were conducted in order to determine the compressive strength of SCT walls and to compare the behavior of prisms extracted from in situ conditions with those newly constructed in the laboratory.

Finally, bond wrench tests were conducted to provide flexural capacity values (ultimate stress) at the unit-to-mortar interface for masonry specimens. These data were then qualitatively and analytically correlated with the out-of-plane testing on full-scale specimens.

Results and Conclusions: The laboratory testing of unit blocks provided a good understanding of the properties of individual SCT and how the behavior and capacity of original building tile compared with that of new tile. Overall, it was concluded that using the new block to construct SCT infilled frames for testing should provide laboratory specimens that would indeed be reasonably representative of the performance of the SCT infills in the Y-12 Plant buildings. Though many block properties were investigated (Butala and Jones 1993), compressive strength and elastic modulus are discussed here. Two hundred of the total units were tested for these properties. Variations in the loading direction and the use of gross or net area in the calculation of strength and modulus are also included. The results from the compressive strength testing are shown in Table 3.

Table 3. Average Unit Compressive Strength and Elastic Modulus

Parameter	Old tiles		New tiles	
	10.16 cm (4 in.)	**20.32 cm (8 in.)**	**10.16 cm (4 in.)**	**20.32 cm (8 in.)**
Gross compressive strength, KPa (psi) [Normal to core]	12,210 (1771)	10,687 (1550)	25,760 (3736)	15,976 (2317)
Gross compressive strength, KPa (psi) [Parallel to core]	33,199 (4815)	27,477 (3985)	55,490 (8048)	31,076 (4507)
Net compressive strength, KPa (psi) [Normal to core]	35,951 (5214)	41,542 (6025)	63,013 (9139)	59,104 (8572)
Net compressive strength, KPa (psi) [Parallel to core]	62,855 (9116)	65,420 (9488)	100,598 (14,590)	70,915 (10285)
Gross elastic modulus, MPa (ksi) [Normal to core]	11,101 (1610)	6688 (970)	9170 (1330)	7516 (1090)
Gross elastic modulus, MPa (ksi) [Parallel to core]	13,445 (1950)	10,067 (1460)	14,135 (2050)	10,274 (1490)
Net elastic modulus, MPa (ksi) [Normal to core]	28,063 (4070)	25,787 (3740)	22,340 (3240)	27,994 (4060)
Net elastic modulus, MPa (ksi) [Parallel to core]	26,546 (3850)	24,477 (3550)	25,649 (3720)	23,374 (3390)

Generally, the compressive strength was higher for the parallel loading than for the normal loading. Almost all new tile specimens produced a cracking sound throughout the testing (absent for the most part during the old tile tests), which indicates that the new SCT units are more brittle than the old units.

The results of the compressive strength test for new 10.16-cm (4-in.) tiles were higher than those for old 10.16-cm (4-in.) tiles. This result is due to the smaller void area in the new tiles (i.e., larger net area) and may also be linked to improvements in the grinding and firing process. For 20.32-cm (8-in.) tiles, the results, though somewhat lower for old tiles, were in general agreement with those for new tiles.

The laboratory testing of prisms provided two key properties needed for the analysis of infilled structures – Young's modulus and compressive strength. Three in situ prisms were extracted and subjected to compressive loads to failure. The ultimate load, compressive stress, and modulus of elasticity were quite low, likely due to damage resulting from the extraction process. Though the number of the in situ test specimens was inadequate to form statistically valid quantitative conclusions, the behavioral characteristics for in situ prisms were similar to those constructed and tested in the laboratory.

Seventy-eight prisms were constructed and tested in the laboratory with varying thickness and loading angles. Of those 78 specimens, 47 were cured for 28 days and tested in monotonic uniaxial compression to failure. Approximately half of the 47 specimens were constructed from 20.32-cm (8-in.) SCT and half from 20.32- and 10.16-cm (8- and 4-in.) SCT [forming a composite 33.02-cm (13-in.) prism]. Approximately half of the 47 specimens were tested with the load parallel to the cores and half with the load normal to the cores. Average compressive strength and modulus of elasticity on the gross and net sections for the 47 uniaxial prism tests are shown in Table 4.

Table 4. Average Laboratory Uniaxial Prism Test Results

ID	f_m gross KPa (psi)	f_m net KPa (psi)	E_m gross MPa (ksi)	E_m net MPa (ksi)	Number Tested
8-in. parallel to core	4978 (722)	18,044 (2617)	3661 (531)	13,265 (1924)	11
8-in. normal to core	7012 (1017)	26,345 (3821)	6171 (895)	23,188 (3363)	12
13-in. parallel to core	5461 (792)	17,300 (2509)	4530 (657)	14,362 (2083)	12
13-in. normal to core	2992 (434)	10,315 (1496)	8184 (1187)	28,187 (4088)	12

For the laboratory test results, the failure mode exhibited by the normal prisms differed significantly from that of the parallel prisms (Bennett et al, 1997). Both the 20.32-cm (8-in.) and 33.02-cm (13-in.) normal prisms experienced brittle failures that resulted in the almost complete destruction of the prism as the maximum load was attained. Popping sounds (apparently indicating cracking of the SCT webs) were heard at load levels between approximately 25% and 50% of the maximum load. The frequency and volume of the cracking increased as the peak load was approached. After reaching peak load, the failure was sudden and characterized by total or nearly total disintegration of the test specimens. The failure mode for the parallel prisms was generally more ductile as characterized by a gradual decrease in the applied load after the peak load was reached. Typically, maximum load was associated with localized failures in the form of spalled or bulged face shells along the edges of the SCT. Thereafter, the prisms remained virtually intact and the faces contained a number of vertically oriented cracks indicative of splitting failure.

The bond wrench tests provided results that should primarily be used as a qualitative instrument to be coupled with other more accurate evaluative methods. The in situ tests resulted in average modulus of rupture values of approximately 75.8 KPa (11psi) for 20.32- and 10.16-cm (8- and 4-in.) specimens. However, the test values ranged from 20.7 KPa (3 psi) to 275.8 KPa (40 psi) and were evidently dramatically affected by prior damage and age. The 23 new specimens that were tested had an average modulus of rupture of 827 and 752 kPa (120 and 109 psi) for the 10.16- and 20.32-cm (4- and 8-in.) blocks, respectively. The overall average for the 10.16- and 20.32-cm (4- and 8-in.) blocks combined was 786 KPa (114 psi). In general, the process of constructing and testing the laboratory bond wrench specimens was very uniform, yet results from the testing were scattered (though not as scattered as the in situ specimens), with a standard deviation is 372 KPa (54 psi). Likewise, few behavioral groupings may be established on the basis of parameters such as wall number, specimen size, or specimen condition.

The conclusion drawn from this information is that, in general, bond wrench laboratory test values may be good predictors of upper- and lower-bound flexural/tensile capacity; however, incorporation of average values into accurate full-scale analytical models may be ill advised. This conclusion appears to be even more justified when the sample population consists of in situ specimens whose construction, removal, and testing lacks the rigor of laboratory conditions.

Large-Scale In Situ Airbag Pressure Testing

Test Description: An existing SCT infill was load tested out-of-plane with an airbag (Fricke 1992). The test was performed on an infilled frame on the ground floor of a five-story steel-frame structure built in 1945 at Y-12 and, at the time of the test, being used for office space. Because the pressure test was complex and was to be performed in situ while occupied, the preparatory work started more than a year before the actual experiment. The wall panel was 8.5 m (28 ft) wide and 3.7 m (12 ft) high (floor to top of wall), and consisted of single-wythe unreinforced construction made from SCT units 30.5 x 30.5 x 20.3 cm; 20.3 cm wall thickness (12 x 12 x 8 in.; 8 in. wall thickness). The cores were horizontal, and running-bond construction was used. The wall was infilled between the flanges of two W14x142 columns with a W30x108 overhead beam (strong axis in the plane of the wall) and concrete slab floor beneath. The second-floor concrete floor slab was poured around the top flange of the overhead beam to provide continuous lateral support of the top flange. The wall from the airbag side is shown in Fig. 3 (in the picture, the reaction frame for the airbag is seen in front of the wall). Approximately 70 linear variable displacement transducers (LVDT) and strain gages measured behavior of the wall during the test process (See Fig. 4). A high-speed data acquisition system monitored strain and displacement and controlled airbag pressure.

FIG. 3. Photograph Showing Reaction Frame for Airbag Test

Test Objectives: The purpose of the airbag test was to determine the wall's out-of-plane capacity to resist wind and seismic loads by applying lateral pressure (with an airbag) to an existing wall. The test also served the major purpose of comparing the actual strength and behavioral characteristics of infills to that predicted by conventional theory. The primary objectives of the airbag test were to accomplish the following:

1. Determine out-of-plane load capacity and load-deflection behavior of the wall;
2. Document the performance of the existing boundary conditions (connectivity of the wall to the steel frame);
3. Establish crack patterns and failure behavior; and
4. Obtain load, deflection, and strain data for use in developing accurate behavioral models for future seismic analyses of buildings composed of infilled frames.

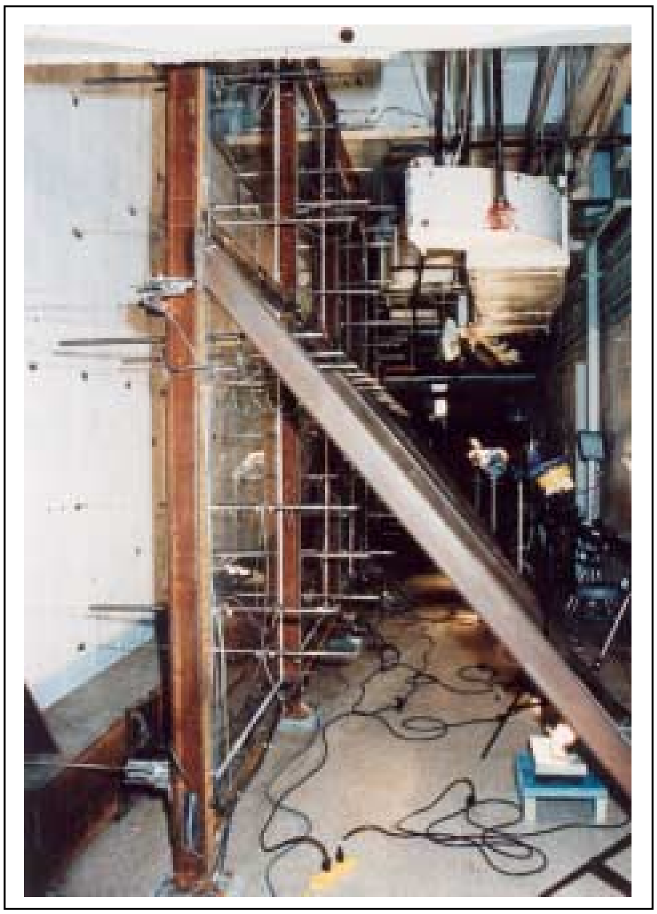


FIG. 4. Photograph Showing Airbag Measuring Frame and Instrumentation

Results: Two preliminary low-level tests were run prior to the actual test. These pretests verified that the control system, the data-acquisition system, and all associated instrumentation were functioning properly and that the airbag was filling as expected. Some major occurrences in the actual test process are recounted as follows:

- The first sounds indicating possible cracking were audible approximately 40 min after the start of the main test process at a lateral pressure of 1.93 Kpa (0.28 psi). At this pressure, displacement at the center of the wall was measured to be 0.17 cm (0.07 in.). The first vertical crack, which occurred in a mortar joint, became visible at this time.

- A horizontal crack at the top of the wall near the beam-column interface was noted at a pressure of 3.38 KPa (0.49 psi) and a deflection at the center of the wall (i.e., centerline deflection) of 0.25 cm (0.10 in.). At a pressure of 5.79 KPa (0.84 psi) and a centerline deflection of 0.63 cm (0.25 in.), the upper steel beam and mortar joint separated. Stair-step cracking patterns began to develop in the upper quadrants of the infill at 6.61 KPa (0.96 psi).
- The maximum pressure of 6.94 KPa (1.01 psi) was reached at approximately two hours into the main test process at a centerline deflection was 1.52 cm (0.60 in). Almost immediately, the pressure began to fall, and for the next 25 min, as air continued to flow into the airbag (to maintain pressure), the pressure fluctuated between 6.34 and 6.83 KPa (0.92 and 0.99 psi). During this period, wall movement was obvious, and the centerline deflection increased more than 2.54 cm (1 in.) to 4.27 cm (1.68 in.).
- At approximately 2-½ hours after the start of the main test, the XY-plot of the stress in the overhead steel beam and the centerline displacement indicated the presence of nonlinear behavior. At this point, pressure reduction was initiated, and the test was terminated. Release of the pressure from the wall took about 30 seconds. Data were read for a few more minutes after the pressure was at zero to allow the LVDTs to stabilize.

Wall and steel-frame deflections and strains were automatically read and recorded at approximately 2-sec intervals, so that about 4000 data records (each with the data from 94 channels) were produced. The test data were analyzed to provide insights into the behavior of the infilled frame. Crack patterns were mapped. The wall was taken down, block by block, so that details of construction could be documented.

Conclusions: In-plane forces that develop during the out-of-plane loading greatly enhance the ability of the wall to resist lateral pressure. Some of the factors that affect the out-of-plane strength of infills were identified as: (1) in-plane spring stiffness (surrounding frame-member sizes), (2) preexisting normal stresses, (3) wall-assemblage properties, and (4) wall eccentricity with respect to the steel frame.

The out-of-plane strength of the wall was found to be many times greater than that predicted by conventional theories that do not account for post-cracking mechanisms, especially the arching action of the wall within the steel frame. Arching phenomena provided a substantial increase in the predicted capacity. Conventional methods predicted "failure" of the test wall due to ground acceleration at pressures of 0.48 to 0.69 KPa (0.07 to 0.1 psi) and wind speeds less than 80 mph. The maximum pressure obtained on the wall was 6.94 KPa (1.0 psi). This result suggests that infills similar to the one tested are highly unlikely to fail out-of-plane as a result of either extreme wind or inertial forces from a seismic event. This is true despite amplification of the ground acceleration at higher elevations within a building. Despite the variable (and rather poor) construction encountered within this wall, it still demonstrated a remarkably high lateral-load capacity.

Quasi-Static Testing of Large-scale Infilled Replicas

Test Description: To better understand the effects of in-plane and out-of-plane loads on the infill walls at Y-12, full-scale replicas of these walls were constructed and tested at Iowa State University (Henderson et al 1993). The loading methods were designed to simulate seismic loads as they would be applied in situ. The experimental phase of the research consisted of the following four large-scale tests:

(1) Frame-OP – out-of-plane testing of one bare frame without infilling in order to determine the behavior and stiffness contribution of the frame only;

(2) Wall1-OP – out-of-plane drift testing of one infilled frame; [This wall 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 to simulate seismic drift induced by the top and bottom chords of a roof truss framing into the columns at these locations.]

(3) Wall1-IP – in-plane testing to failure of the infilled frame previously damaged by out-of-plane drift in order to determine residual strength after prior damage; and

(4) Wall2-IP – in-plane testing to failure of an infill (identical to Wall 1) with no prior damage.

By comparing Wall1-IP and Wall2-IP, the effect of prior damage on in-plane behavior could be established. For both out-of-plane and in-plane testing, reversed-cyclic quasi-static loading was used to obtain full tension-compression hystereses. Also, natural frequencies of the first infilled panel were determined before and after the out-of-plane testing. The out-of-plane and in-plane test setup is shown in Fig. 5. This test series was preceded by an extensive study of the in situ boundary conditions (such as column-to-wall and beam-to-wall connections) in order to closely reproduce actual conditions.

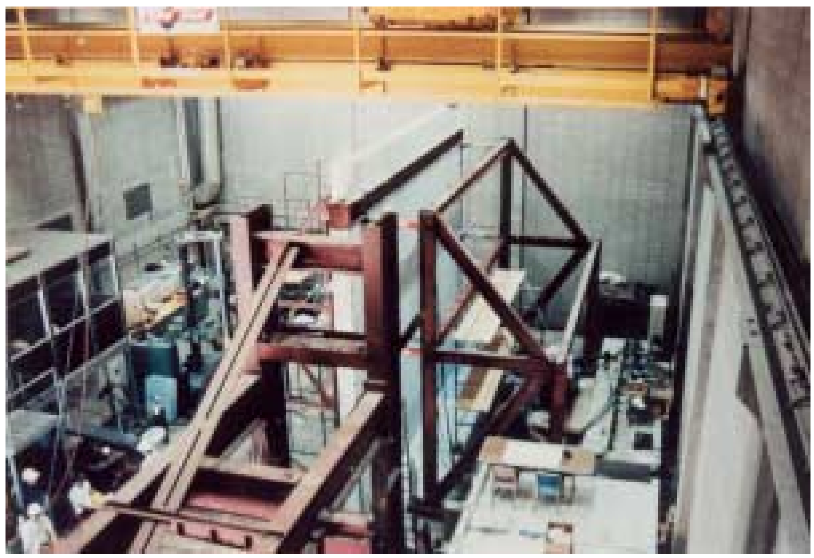


FIG. 5. Photograph Showing Large-scale Replica Test Setup

Each test specimen was 7.32 m (24 ft) long by 6.71 m (22 ft) tall – center-to-center of steel. The infill material (both in the field and for the test series) had a 33 cm (13-in.) nominal thickness, composed of individual 10.2- and 20.3-cm (4- and 8-in.) blocks (see Fig. 1). The SCT was oriented so that the cores ran horizontally (side construction) and in the direction parallel to the plane of the wall as is common at Y-12. Bedjoints were full and continuous with a 1.27-cm (½-in.) thickness. A 1.27-cm (½-in.) bedjoint was also placed between the floor and the first course. Headjoint mortar was applied to the face shells only and was 0.95 cm (3/8 in.) thick. The 10.2- and 20.3-cm (4- and 8-in.) blocks alternated position from course to course and were laid in running bond (½-block offset), thereby creating collar joints that were discontinuous at the interface with successive courses.

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Test Objectives: The primary objectives of the large-scale quasi-static testing were as follows:

1. Determine the out-of-plane behavior and stiffness contribution of the bare frame to the overall infilled-frame stiffness;
2. Determine the out-of-plane load-deflection relationships of an infilled frame;
3. Establish out-of-plane and in-plane crack patterns;

4. Determine and compare the in-plane capacity and load-deflection relationships of an infilled frame with and without prior out-of-plane damage; and
5. Establish the behavioral influence of the existing boundary conditions.

Results and Conclusions: The maximum applied out-of-plane displacement for the first infill (Wall1-OP) was approximately 6.60 cm (2.6 in.). Out-of-plane cracking damage to this wall was extensive and included numerous complete through-cracks; however, the structure was still completely stable and laterally resistive upon discontinuance of the out-of-plane loads. The first infilled frame was then tested in-plane to failure (Wall1-IP) where the displacement required to cause considerable corner crushing was approximately 5.08 cm (2 in.). The second wall with no prior damage (Wall2-IP) also sustained significant in-plane corner crushing at approximately 5.08 cm (2 in.). The maximum in-plane loads were 64 kips for the first infill (predamaged; Wall1-IP) and 61 kips for the second infill (no prior damage; Wall-2 IP). Crack patterns and final damage states were very similar for both infilled frames.

Some conclusions related to the test objectives follow:

1. Comparing Frame1-OP with Wall1-OP, indicates that the bare frame contributed approximately 10 percent of the total out-of-plane stiffness at the start of loading. The relative lateral contribution of the frame increased to approximately 50 percent as loading progressed and nonlinear behavior (i.e., cracking) of the masonry progressed.
2. The tension / compression load cycles produced hysteretic behavior under out-of-plane loads. However, the load-deflection behavior was significantly more linear in the out-of-plane direction, indicating less inelasticity and energy absorption than for in-plane loading.
3. Each infilled frame (Wall1-OP, Wall1-IP and Wall2-IP) developed multiple fully-penetrating cracks during the test process. Yet, the structures were completely stable and capable of resisting significant lateral load at displacements of 5 cm (2 in.) and greater. [This information points to the importance of addressing current masonry and building code deficiencies. Current codes do not recognize masonry infills as anything more than unreinforced masonry walls, which, to remain viable, should not be cracked.]
4. A comparison of in-plane data for the infilled frames (Wall1-IP and Wall2-IP) clearly shows that prior damage to the infill reduces the in-plane initial stiffness. However, after the first few tension/compression cycles, Wall1-IP and Wall2-IP showed very similar load deflection behavior, including the shape and magnitudes of the load-deflection plots. Prior damage has little effect on the in-plane capacity of infills provided confinement by the steel frame is maintained.
5. The frame-to-infill connections used in construction of the test specimens were more than adequate to ensure composite behavior.

Dynamic Shake-Table Tests

Test Description: Dynamic shake-table testing of SCT infills was conducted at the U.S. Army Construction Engineering Research Laboratory (USA-CERL). The test structure as shown in Fig. 6 consisted of two SCT-infilled frames spaced approximately 3.05 m (10 ft) apart and connected by steel trusses and a concrete roof slab (Bennett et al 1996). Three types of dynamic tests were performed on the specimen. The first set of tests was performed with the walls oriented out-of-plane with respect to the loading direction. The specimen was then rotated 90^{o} on the shake table so that the second set of tests could be performed with the walls oriented in-plane with respect to the loading direction. After the in-plane seismic tests were complete, two in-plane sinusoidal sweep tests were performed on the specimen. The orientation of the specimens with respect to the reference frame and table motion during both the in-plane and out-of-plane tests is shown in Fig. 7.

Three tests were performed on the structure in the out-of-plane direction by exciting the model with an Oak Ridge, Tennessee, site-specific seismic time history at 100%, 200%, and 300% of full-scale, respectively. Due to the displacement limit of the CERL table of 6.99 cm (2.75 in.) peak, the acceleration time history was filtered to remove all frequencies of 0.75 Hz and below. This acceleration time history was then double integrated to get the displacement time history used by CERL to control table motion. Four low-level random-vibration tests (at 1.6% of full-scale) were also performed in the out-of-plane direction in order to measure the natural frequency of the structure after each of the seismic tests. The purpose of this sequence was to observe the amount that the frequency changed with each progressive test. At the end of each out-of-plane test, crack maps were drawn to document the progression of mortar cracking.

FIG. 6. Shake-table Test Setup

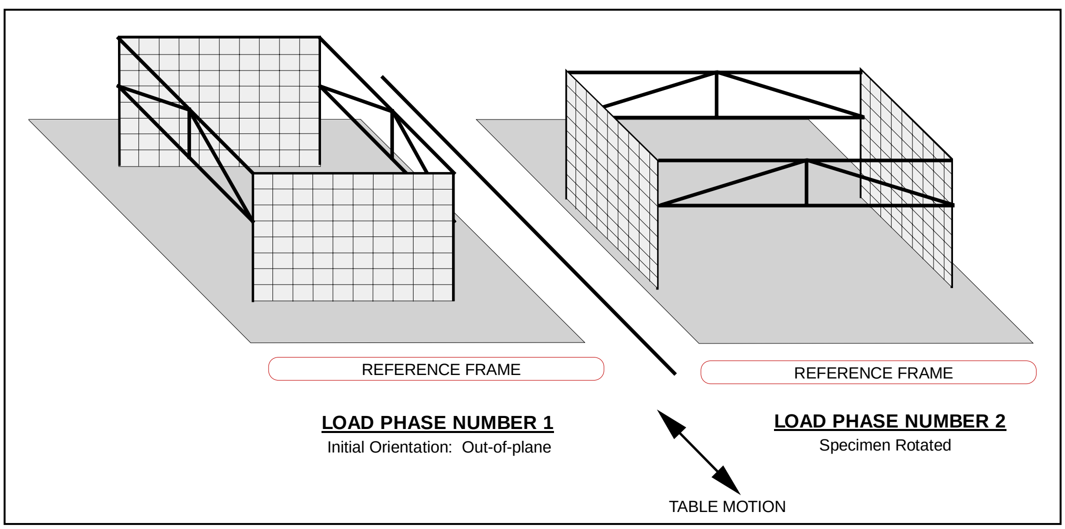

FIG. 7. Shake-table Test Orientation

Six in-plane tests were conducted with the same site-specific time-history record that was used for the out-of-plane tests, and were performed at 1, 2, 4, 8, 12, and 16 times the full-scale time-history record. Seven low-level random-vibration tests were performed to determine change in frequency of the structure as the testing progressed. At the end of each of the in-plane tests, crack maps were drawn. After all of the out-of-plane and in-plane tests were completed, two in-plane sinusoidal sweep tests were conducted to fail the structure. During the sweep tests, the biaxial shake table was driven in a horizontal direction at constant amplitude with sinusoidal acceleration. This acceleration was swept over a predetermined frequency range. For the first test, the frequency started at 10 Hz and was increased up to 20 Hz at an amplitude of 1.0 *g*. For the second test, the frequency started at 15 Hz and was to be decreased to 5 Hz at an amplitude of 3.0 *g*. During the second test, however, the SCT panel began to fail, and the test was stopped at a frequency of about 12.7 Hz.

Test Objectives: The primary objectives of the shake-table testing were as follows:

1. Investigate the structural capability relative to the ground-motion design spectra for a test wall that is representative of SCT walls at Y-12;
2. Correlate conclusions resulting from the static tests;
3. Perform a series of tests to enable characterization of nonlinear behavior, including damping;
4. Take test data that can be used to correlate analytical modeling techniques for full-scale modeling and local-unit evaluation;
5. Identify and characterize coupling mechanisms between drift and SCT-wall inertial effects;
6. Design a test specimen with a drift mode high enough in frequency to demonstrate coupling behavior;
7. Select and position sensors to measure relative and absolute characteristics to facilitate data reduction; and
8. Extend testing beyond quasi-static techniques.

Conclusions: The following conclusions are based on the observations made by analyzing data obtained from the out-of-plane, in-plane, and sinusoidal sweep tests. The areas of interest in the out-of-plane direction were acceleration amplifications, relative and absolute displacements, column deformations, and frequency degradation. The panels of the specimen had larger acceleration amplification than either the roof slab or beam. The largest acceleration response occurred in the panels near the top of the walls, in which an input amplification factor of about 3 was calculated.

The relative- and absolute-displacement transducers indicated very little relative movement between the clay tile panels and frame for the out-of-plane testing. This result suggests that the panel and frame were essentially moving together as a single unit, although frequency analysis indicated that the panel and frame were responding at two separate natural frequencies. The deformed shapes show that the largest relative displacement between the panel and frame occurred midway up the columns. The truss system that connects the two walls added considerable stiffness to the top of the specimen. This caused the maximum relative displacement to occur at mid-height of the columns and may have caused the frame and panel to respond at two different frequencies.

As a result of the out-of-plane testing, the structure’s stiffness decreased slightly, but no evidence was observed of the panel's "walking-out" of the frame. The panel was still very stable after completion of the out-of-plane testing.

The in-plane analysis also involved looking at acceleration amplification, relative and absolute displacements, and frequency degradation. Also, load-deflection behavior was investigated. Analysis of acceleration data indicates that considerably larger acceleration amplification occurred in the columns than in the roof slab. The load-deflection analysis indicated that the stiffness of the structure decreased with each progressive in-plane test.

In comparison with static tests, higher stiffness was seen in the dynamically tested structure and could be attributed to the vertical load being applied by the slab. Ultimate-strength characteristics were very similar to those observed from static in-plane tests. Load-column force analysis indicated the formation of a diagonal compressive strut during loading that caused high lateral strains in the panels. Panel load-strain curves indicated that significant cracking of the panel did not occur until the fifth seismic test (1200% of full scale). The frequency degradation that occurred with increasing seismic input indicates that the panel was cracking and losing stiffness.

The sinusoidal sweep tests that were performed failed to precisely locate the natural frequency of the structure; however, it was much lower than initially thought. The panels were eventually failed by using an input acceleration of 3.0 *g*. During all of these out-of-plane and in-plane tests, the panels did not "walk out." Up until the last sinusoidal sweep test, the infill remained stable and continued to provide some load resistance. Therefore, infills can be expected to enhance the seismic resistance of otherwise laterally weak structures. Furthermore, the infills exhibited a slow progression of inelasticity rather than explosive failures that were thought possible.

Moderate-Scale Laboratory Tests

Test Description: Twenty-one half-scale specimens consisting of steel frames infilled with unreinforced structural clay tile were tested at the University of Tennessee in this portion of the research. The in-plane test setup of a typical solid panel is shown in Fig. 8 (Flanagan and Bennett 1999a). All of the infills consisted of solid panels except one that had a 0.61- by 0.61-m (2- by 2-ft) opening in the upper corner. The test specimens consisted of portal frames infilled with SCT of either single-wythe or double-wythe construction. Single-wythe panels were constructed of nominally 20.32-cm (8-in) thick units, and double-wythe panels were constructed of 20.32-cm (8-in) and 10.16-cm (4-in) tile units, thereby creating a nominally 33.02-cm (13-in) thick wall (side-construction).

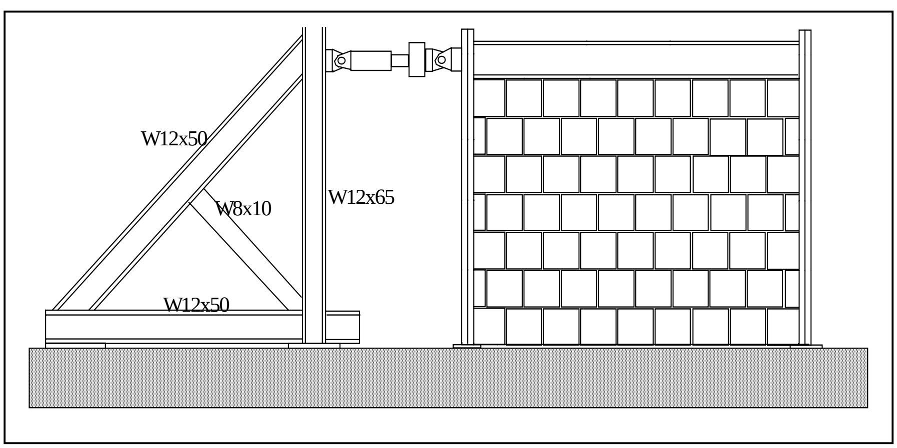

FIG. 8. Moderate-scale In-plane Test Setup

Eleven steel frames infilled with SCT were tested to failure in the plane of the panels. Two of the panels were tested twice, one repaired and the other retrofitted prior to retesting. Three structural SCT infilled frames were tested out-of-plane to failure with uniform pressure loads. These tests were designed to simulate inertial effects of the infill normal to its plane. Sequential bi-directional tests were performed on four specimens. In these tests, load cycles were applied in one direction up to a predetermined limit, and the specimen was unloaded. Subsequent tests were then performed as load was applied in the other direction (Flanagan and Bennett 1999b). A combined bi-directional test was performed on a single infilled frame whereby out-of-plane (airbag) and in-plane loads were applied simultaneously. A photograph of the out-of-plane drift test setup is shown in Fig. 9.

Test Objective: The objective of the experimental program was to test a broad range of steel-frame specimens infilled with SCT to determine their structural behavior and the sensitivity of that behavior to a variety of geometric and construction parameters. Furthermore, the research was intended to provide experimental data for validating proposed analytical techniques.

FIG. 9. Moderate-scale Out-of-plane Drift Test

Results and Conclusions: The in-plane behavior was characterized first by diagonal panel cracking at a shear stress of approximately f'_m followed by corner crushing at ultimate capacity. The presence of the infill stiffens the steel framing, and the framing confines the masonry, thereby allowing greater strength and ductility. The infill tends to perform as a compressive strut that braces the otherwise unstable framing. The significant postpeak strength that was observed indicates continued energy-absorption capability. Two in-plane failure mechanisms were observed: diagonal cracking and corner crushing.

Infills with the panel offset from the frame centerline developed peak capacities proportional to the effective net area enclosed in the framing. Infills of different thicknesses were tested out-of-plane with a uniform lateral loading to simulate the inertial effects of the panel normal to its plane. The behavior was dominated by the arching action of the masonry.

A sequential test of in-plane loads followed by out-of-plane airbag loading reduced the out-of-plane capacity by 20%. The out-of-plane stiffness was also reduced by the prior in-plane loading, thereby resulting in a 65% increase in midpanel displacement.

The stability of SCT infills under out-of-plane drift loading and the subsequent effect on in-plane behavior were investigated. Little relative movement of the infill panels with respect to the steel framing occurred, and stability of the infill was maintained. Consequently, degradation of in-plane and out-of-plane loads was not significant, particularly at moderate levels of loading.

Airbag loading followed by in-plane loads resulted in no in-plane strength degradation as compared with a test of a similar specimen loaded purely in-plane. Interface degradation and plastic strains from the prior out-of-plane loading resulted in a 50% decrease in initial in-plane stiffness. Prior lateral loading cracked the panel mortar joints, thereby eliminating the in-plane diagonal cracking limit state.

CONCLUSIONS

Following a 3-year testing program in which over 700 tests were conducted, the results clearly show that the SCT infills are significantly more stable and laterally resistive than previously anticipated by conventional, code-based approaches. Prior cracking reduces the initial stiffness, but has little effect on capacity. Furthermore, the infill material remains within the surrounding frame when subjected to simulated seismic and wind loads at levels much higher than previously estimated.

None of the test specimens exhibited explosive failure of the SCT. On the contrary, the infill material provides considerable lateral stiffness and the steel frame adds ductility and confinement to the overall system. During postpeak cyclic loading, the infill remains resistive and continues to absorb significant energy while the SCT cracks and some crushing of individual tile occurs at the frame corners.

From the constitutive property testing, it was concluded that using the new units to construct SCT infilled frames for testing should provide laboratory specimens that would indeed be reasonably representative of the performance of older SCT infills.

For uniform out-of-plane loading, as would be induced by wind pressures or seismic inertial forces, tremendous capacity was observed due to vertical and sometimes horizontal arching action of the wall panel (infill). The infills invariably remained stable after ultimate capacity had been achieved.

Diagonal cracking and corner crushing were identified as the predominant in-plane failure mechanisms. Therefore, shear capacity of the masonry does not govern the capacity of the infill. Rather, modeling the behavior of the infill as a compression strut is likely to be the best predictor of overall wall capacity.

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REFERENCES

Adham, S.A. 1985. "Out-of-Plane Response of Masonry Walls," pp. 1-14 in *Proceedings of the 3rd North American Masonry Conference*, Univ. of Texas, Arlington, 47.

Bennett, R.M., Fowler, J.J., and Flanagan, R.D. (1996). "Shake Table Testing of Structural Clay Tile Infilled Frames." *Proceedings of the 7th North American Masonry Conference*, Notre Dame, 497-507.

Bennett, R.M., Boyd, K.A., and Flanagan, R.D. (1997). "Compressive Properties of Structural Clay Tile Prisms." *Journal of Structural Engineering*, ASCE, 123(7), 920-926.

Butala, M.B., and Jones, W.D. 1993. *Material Properties of Hollow Clay Tile and Existing Mortar Characterization Study*, Y/EN 4687, Martin Marietta Energy Systems, Inc., Oak Ridge Y-12 Plant, Oak Ridge, Tennessee.

Flanagan, R.D., and Bennett, R.M. (1999). "In-Plane Behavior of Structural Clay Tile Infilled Frames." *Journal of Structural Engineering*, ASCE, 125(6), 590-599.

Flanagan, R.D., and Bennett, R.M. (1999). "Bi-Directional Behavior of Structural Clay Tile Infilled Frames." *Journal of Structural Engineering*, ASCE, 125(3), 236-244.

Fricke, K.E., Jones, W.D., Huff, T.E. (1992). "In situ Lateral Load Testing of a URM HCT Wall." Proceedings of the 6th Canadian Masonry Conference, Saskatoon Saskatchewan Canada.

Henderson, R. C., Beavers, J. E., Jones, W. D., Schaefer, K. E. (1995). "Lessons Learned From the Y-12 Plant Hollow Clay Tile Wall Test Program." *Proceedings of the* 5th *DOE Natural Phenomena Hazards Mitigation Conference*. Atlanta, GA.

Henderson, R. C., Jones, W. D., Porter, M. L. (1993). "Out-of-Plane and In-Plane Testing of URM Infills." *Proceedings of The ASCE Structures Congress*, Irvine, CA.

Holmes, M. 1961. "Steel Frames with Brickwork and Concrete Infilling," pp. 473-478 in *Proceedings of the Institution of Civil Engineers*, 19.

Kennedy, R. P., et al. 1985. *Draft Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards*, Office of the Assistant Secretary for Environment, Safety, & Health, Office of Safety Appraisals, U.S. DOE.

Langenbach, R. 1992. "Earthquakes: A New Look at Cracked Masonry," *Civil Engineering*, **62**(11), 56-58

Miranda, E., and Bertero, V.V. 1989. "The Mexico Earthquake of September 19, 1985—Performance of Low-Rise buildings in Mexico City," *Earthquake Spectra*, Earthquake Engineering Research Institute, 5(1), 121-143.

Stafford-Smith, B. 1966. "Behavior of Square Infilled Frames," *Journal of the Structural Division*, American Society of Civil Engineers, 92(ST1), 381-403.

Williamson, D.H. 1987. *Reevaluation of Seismic Resistance Capacity of Y-12 Plant Facility 9206*, Y/EN-1871, Martin Marietta Energy Systems, Inc., Oak Ridge Y-12 Plant, Oak Ridge, Tennessee.