Y-12

**OAK RIDGE
Y-12
PLANT**

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DEPARTMENT OF ENERGY

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HOLLOW CLAY TILE WALL PROGRAM SUMMARY REPORT

R. Craig Henderson
Gilbert/Commonwealth, Inc.

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MS Technology, Inc.

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Prepared for the
Center for Natural Phenomena Engineering

July 30, 1995

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

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EXECUTIVE SUMMARY

This *Hollow Clay Tile Wall Program Summary Report* was prepared as a final document, summarizing the results of the Hollow Clay Tile Wall (HCTW) Test Program of the U.S. Department of Energy (DOE). The program was conducted for the Oak Ridge Y-12 Plant by the Center for Natural Phenomena Engineering (CNPE) of Lockheed Martin Energy Systems, Inc. (LMES; formerly Martin Marietta Energy Systems, Inc., or MMES).

Many of the Y-12 Plant buildings, constructed during the 1940s and 1950s, consist of steel and/or reinforced concrete framing infilled with hollow clay tile (HCT). 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 in conjunction with the preparation of a site-wide safety analysis report. This analytical work, based on the best available methodology, considered lateral load-carrying capacity of the HCT infill on the basis of building code allowable shear values.

In parallel with the analysis effort, DOE initiated a program to develop natural phenomena capacity and performance criteria for existing buildings, but these criteria did not specify guidelines for determining the lateral force capacity of frames infilled with HCT. The evaluation of infills was, therefore, based on the provisions for the design of unreinforced masonry as outlined in standard masonry codes. When the results of the seismic and wind evaluations were compared with the new criteria, the projected building capacities fell short of the requirements. Apparently, if the buildings were to meet the new criteria, many millions of dollars would be required for building upgrades.

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

1. Once the infilled 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 HCT infill?
4. Was modeling the HCT infill as a shear wall the best approach?

Also, during this time, buildings similar in construction to those at the Y-12 Plant were subjected to the 1985 Mexico City earthquake, and their performance was studied (Miranda and Bertero 1989). This research indicated that the HCTWs at the Y-12 Plant 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 evaluations performed during the early 1980s may have significantly underestimated the performance of the buildings. Therefore, to further quantify the seismic and wind capacity of the HCT-infilled buildings, the test program (Oak Ridge Y-12 Plant December

1991) was established. The test program was formulated to address the four issues related to HCT behavior as well as many peripheral concerns pertinent to the structural analysis of infilled buildings at Y-12. The primary goal, however, 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.

Full-scale testing at Iowa State University of replicas of the infills in Building 9212 confirmed that severe out-of-plane cracking does not diminish the in-plane capacity of an infilled structure (Henderson August 1994). Dynamic shake table tests of moderate-scale infills at the U.S. Army Corps of Engineer's Construction Engineering Research Laboratory showed conclusively that the HCT remains within the framing when subjected to ground acceleration values at frequencies ranging from 4 to 10 Hz. The culmination of all test program research on constitutive properties (i.e., prism compression, bond wrench, etc.) as well as the moderate-scale infill testing at the University of Tennessee demonstrated that the resistance of an infill to in-plane loads is more a function of the compressive strut area and masonry compressive strength than of the masonry shear capacity (Henderson August 1994, Flanagan December 1994).

Also, on the basis of the test results, a piecewise linear equivalent strut formulation was developed for modeling the in-plane resistance of infills at Y-12 (Flanagan 1994). This method, which replaced the previously used shear wall approach, accounts for the nonlinear hysteretic behavior of infills. As the strength of the infill degrades from continued cracking, the effective strut stiffness is reduced as a function of the frame displacement. This strut formulation was chosen over the shear-wall approach because of (1) the need to accurately account for the nonlinear behavior of HCT with increasing lateral load and (2) the inordinate computational effort required to represent the postcracking infill response by using planar finite elements.

In conclusion, after 3 years of static and dynamic experimental research in which more than 700 tests were conducted, the results clearly show that HCT infills resist lateral loads much more effectively than anticipated by conventional code procedures. The experimental data indicate that the combination of a steel frame and infill is very efficient in that 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 HCT, 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.

As part of the Y-12 Safety Analysis Update Program, evaluations of a number of Y-12 structures are currently underway or planned for future fiscal years. The analytical results to date indicate that previously evaluated structures now meet the requirements of DOE Standard DOE-STD-1020-94, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* (previously UCRL-15910). Specifically, the HCTW Program has shown that the modeling techniques and the allowable capacities used for HCTW buildings in the 1970s and 1980s underestimated the performance of those buildings when subjected to seismic and wind loads. The HCTW Test Program has provided a more appropriate method for modeling HCTW structures and for determining their seismic- and wind-load capacities. Using test program results should avoid the expenditure of millions of dollars in unnecessary building upgrade costs.

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ABBREVIATIONS, ACRONYMS, AND INITIALISMS

ASTM	American Society for Testing and Materials
CNPE	Center for Natural Phenomena Engineering
DOE	U.S. Department of Energy
DOE-ORO	DOE-Oak Ridge Operations
HCT	hollow clay tile
HCTW	hollow clay tile wall
IRA	initial rate of absorption
LMES	Lockheed Martin Energy Systems, Inc.
MMES	Martin Marietta Energy Systems, Inc.
NDE	nondestructive evaluation
NIST	National Institute for Standards and Technology
ORO	Oak Ridge Operations
SAR	safety analysis report
UBC	Uniform Building Code
WBS	work breakdown structure

1. INTRODUCTION

1.1 GENERAL

This *Hollow Clay Tile Wall Program Summary Report* was prepared as a final document, summarizing the results of the Hollow Clay Tile Wall (HCTW) Test Program of the U.S. Department of Energy (DOE). The program was conducted for the Oak Ridge Y-12 Plant by the Center for Natural Phenomena Engineering (CNPE) of Lockheed Martin Energy Systems, Inc. (LMES, formerly Martin Marietta Energy Systems, Inc., or MMES).

1.2 BACKGROUND

Many of the Y-12 Plant buildings, constructed during the 1940s and 1950s, consist of steel and/or reinforced concrete framing infilled with hollow clay tile (HCT), as shown in Fig. 1.1. The infilled panels are approximately 13 in. thick and were constructed with adjacent 4- and 8-in. HCT, the cores of which run horizontally (side construction). Each masonry course sits atop a continuous bed joint and was laid with running bond such that head joints are staggered by half a block. The construction of the infills is unique in that the horizontal position of the 4- and 8-in. blocks alternates from course to course, thereby producing a discontinuous collar joint.

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 building in conjunction with the preparation of a site-wide safety analysis report. 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 of 20 psi (Johnson and Walls 1980).

Research in the 1950s and 1960s (Thomas 1953, 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 HCT, 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 frames infilled with HCT. When the results of the 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.

Fig. 1.1. Section of wall infilled with hollow clay tile.

1.3 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 HCT infill?
4. Was modeling the HCT infill as a shear wall the best approach?

1.4 ESTABLISHMENT OF THE HOLLOW CLAY TILE WALL TEST PROGRAM

As analytical work proceeded at the 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, Kariotis et al. 1985a and 1985b, Adham 1985, Woodard and Rankin 1985). This research indicated that the HCTWs at the Y-12 Plant 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 HCT infilled buildings, the HCTW Test Program was established (Oak Ridge Y-12 Plant 1991). The goal of the test program 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.

1.5 ORGANIZATION OF THIS REPORT

A synopsis of the events that led to the inception of the test program is presented in Chapter 2 of this document. Chapters 3 and 4 are organized such that a chronology of the test program—nine major research areas—is provided. The scope and purpose of each of the research areas is outlined in Chapter 3 and the results and conclusions are discussed in Chapter 4. Appendixes A through I provide a more thorough and detailed account of each of the tests associated with the nine research areas.

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2. HISTORICAL PERSPECTIVE

2.1 GENERAL

During the early to middle 1900s, HCT was used extensively as a building construction material throughout the United States. Many buildings were constructed of steel and/or reinforced concrete, and HCT was used as the infill. The methods used to construct these buildings was generally very efficient and inexpensive, and for the most part, the infills within these structures have performed well as a structural element. However, changes in the way that masonry is designed (i.e., the movement from empirical to analytical design approaches), as well as the increasing visibility of seismic-related issues, led to a greater scrutiny of all unreinforced masonry structures—particularly those within the purview of DOE. The way in which this process occurred is directly related to the inception of the test program. Therefore, those events that precipitated the need for the masonry research described herein are outlined in this section.

2.2 CHANGES IN BUILDING CODES

During the past 20 years, more emphasis has been placed on developing better building codes and understanding and evaluating the effects of natural phenomena on critical facilities. In 1970, the International Conference of Building Officials introduced a new seismic-hazard map into the Uniform Building Code (UBC) (International Conference of Building Officials 1970) that placed Oak Ridge in a higher seismic zone. In addition, after the 1971 San Fernando earthquake (U.S. Department of Commerce 1973), the International Conference of Building Officials implemented stricter seismic-design requirements within the UBC. Similar modifications, in addition to the strengthening of the wind design requirements, were subsequently included in the other two national building codes: (1) the Building Officials and Code Administrators' Code (1990) and (2) the Southern Building Code Congress International's Standard Building Code (1990).

The lives and property lost in the 1989 Loma Prieta and 1994 Northridge earthquakes (Earthquake Engineering Research Institute, 1990 and 1994) and in Hurricane Andrew in 1993 have further increased the emphasis placed on building codes and accentuated the need to reduce building damage from natural phenomena. Such emphasis has also increased the demand to show that buildings built before such changes, especially critical facilities, also meet the general requirements of today's codes.

2.3 SAFETY ANALYSIS REQUIREMENTS

Because of an increased emphasis on the safety of critical facilities, DOE began a program to evaluate and document the safety envelope of its facilities through safety analysis reports (SARs). At the Y-12 Plant, this process began in the late 1970s. The SAR requirements mandated the evaluation of existing structures, particularly moderate- and high-hazard facilities, for natural phenomena effects. The first series of formal building evaluations in support of the SAR process began in 1979 (Johnson and Walls 1980) and, as noted in Chapter 1, were based on the best available methodology. The evaluations showed that the performance of some Y-12 structures was unacceptable at earthquake loads as low as 0.07 g when compared with existing criteria and that

a number of the buildings would experience extensive damage at earthquake loads of 0.15 to 0.22 g.

2.4 NEW DOE DESIGN AND EVALUATION CRITERIA

In the early 1980s, DOE began to develop criteria for the design of new facilities and, in a more limited way, criteria for the performance of existing facilities. In 1989, DOE issued the *General Design Criteria*, DOE Order 6403.1A, which specified UCRL-15910 (Kennedy et al. 1985) as the guiding document for natural phenomena analysis and design. When the results of the evaluations conducted in support of the SAR were compared with the DOE criteria, many Y-12 facilities failed to meet the UCRL-specified seismic performance level of 0.19 g. Clearly, many Y-12 structures under the then-current criteria and analytical methodology would require significant upgrade.

2.5 STUDY OF RETROFIT ALTERNATIVES

In 1985, an estimated \$18 million budget line item was submitted to DOE-Oak Ridge Operations (DOE-ORO) for the purpose of upgrading Building 9206 to meet the DOE natural-phenomena requirements (Oak Ridge Y-12 Plant March 1991). Building 9206 has approximately 66,000 ft² of floor space, and the retrofit cost was thus approximately \$273/ft². With that average cost for similar buildings, the estimated cost to upgrade the 9212 building complex (Buildings 9212, 9215, and 9998), which encompasses 605,000 ft², would be \$165 million; for all production facilities at the Y-12 Plant, the cost would be as high as \$700 million. In 1990, another study was conducted to estimate the cost of bringing the Y-12 production buildings into compliance with the DOE natural-phenomena requirements. In this study, different retrofit techniques were assumed and costed, and retrofit-cost experience gained in California was considered (Holmes 1991). The study yielded an estimated upgrade cost of approximately \$600 million.

2.6 DEVELOPMENT OF A REVISED ANALYSIS APPROACH

Originally, the shear capacity of the infills was based on that allowed in the design codes for masonry. These codes indicated that when unreinforced masonry cracks, whether due to in-plane or out-of-plane loads, no remaining resistance may be accounted for. This approach is reasonable for noninfilled unreinforced masonry walls, but it is intuitively unreasonable for masonry infilling confined by steel or concrete frames. However, despite the fact that residual strength remained after cracking, no means of quantifying that strength existed. A literature search that focused on test data for HCT or concrete block was conducted. It was thought that some research may have been performed that either indicated much higher shear strength than the design codes allowed or demonstrated that unreinforced masonry could remain within the steel or reinforced-concrete framing while continuing to carry significant amounts of postcracking lateral load.

Unfortunately, not much was found during the literature search (Williamson 1987); however, during the 1985 Mexico City earthquake Miranda and Bertero (1989) learned that, for some buildings, the unreinforced masonry infills appeared to have prevented buildings from collapsing. Also, in 1983, the National Science Foundation established the Technical Coordinating Committee

on Masonry Research. During 1984 and 1985, this committee conducted limited research on unreinforced masonry (Kariotis et al. 1985a and 1985b, Adham 1985). This research supported the investigative findings related to the performance of infill structures in the Mexico City earthquake. At about the same time as the National Science Foundation research was being conducted, shear tests of unreinforced HCT block wall sections were performed (Woodard and Rankin 1985). These tests showed that the in-plane shear capacity of the HCT infills could be as high as 80 psi (Williamson 1987)—a significant improvement over the 20 psi previously used.

An evaluation approach was developed on the basis of these research results. In this approach, the HCTWs were assumed to remain within the steel or reinforced-concrete framing and to have a shear capacity of 80 psi. This approach was used to conduct a seismic evaluation of the D-Wing in Building 9212 in support of the SAR for the Y-12 Plant's Enriched Uranium Conversion Facility (Beavers et al. 1989). This evaluation still considered the resistivity of the infill by use of finite plane stress elements within the analytical model. The results showed that the D-Wing would indeed meet the DOE requirements of UCRL-15910. However, no rigorous testing or method-development program, as since has been followed, provided the basis for the assumptions used in the evaluation. In addition, no testing program had been conducted at that time to confirm the performance of the buildings in the Mexico City earthquake and to support the results of other corroborative research.

2.7 UNCERTAINTIES IN THE REVISED APPROACH

In the summer and fall of 1989, an MMES subcontractor analyzed A-, B-, and C-Wings of Building 9212 (Lockwood Greene September 25, 1989). They chose not to use the qualitative results of the Mexico City earthquake or other supportive research because of the lack of a rigorous testing program on either HCT or unreinforced masonry infills. Rather, they used a conventional engineering-evaluation approach that, in many ways was even less complex than that used in earlier evaluations. For example, for out-of-plane loading, the seismic capacity of the infill was predicted by using one-dimensional simple beam theory. This approach neglects two-way plate stresses and shallow arching action that contribute significantly to out-of-plane capacity. As a result, this approach showed building performance to be unacceptable at earthquake loads as low as 0.05 g when compared with existing criteria, which suggested that almost all of the buildings would experience extensive damage at earthquake loads of between 0.15 and 0.20 g. These results were similar to those of previous analysts (Johnson and Walls 1980).

On December 9, 1989, the *Washington Post* published an article stating that "... the unreinforced clay tile walls of the main building of Oak Ridge's Y-12 nuclear plant would give way if struck by an earthquake with a lateral force of .12 percent of gravity, well below the .19 percent used nationwide as the 'design basis' ..." (Lippman December 9, 1989). As a result of this publicity, a number of investigations were conducted (Wilkin and Brown December 21, 1989; Barber February 14, 1990; Advisory Committee on Nuclear Facility Safety January 26, 1990) from which the HCTW Test Program began, as described in the *Program Management Plan* (Oak Ridge Y-12 Plant December 1991). The purpose and scope of the test program, as well as its results and conclusions, are outlined in Chapters 3 and 4 of this document, respectively.

3. PURPOSE AND SCOPE OF THE HOLLOW CLAY TILE WALL TEST PROGRAM

3.1 GENERAL

The HCTW Test Program was developed on the basis of the following five major elements: (1) material property and assembly tests, (2) nondestructive evaluation (NDE) tests, (3) in situ and laboratory tests, (4) dynamic tests, and (5) analytical work as described in the work breakdown structure, or WBS (see Fig. 3.1) of the Program Management Plan (Oak Ridge Y-12 Plant November 1991). The CNPE was responsible for developing and conducting the program, the function of which was to conduct a comprehensive investigation into the behavioral characteristics of HCT and its contribution to the strength and stiffness of infilled framed buildings subjected to earthquake and wind loads. The program was originally scheduled from 1990 to 1998; however, as noted in the close-out plan (Oak Ridge Y-12 Plant November 1994), the program is being completed in fiscal year 1995. The reduction in schedule is a result of early success in meeting program goals, the need for alignment with the schedule of the SAR Update Program, and the changing mission of the Y-12 Plant.

3.2 PURPOSE

The purpose of the HCTW Program was to accomplish the following:

1. assess the strength and behavior of HCT infills when resisting earthquake and wind loads, both in-plane and out-of-plane;
2. understand how the HCT infill material interacts with the steel or reinforced concrete framing for in-plane and out-of-plane forces;
3. determine whether in-plane and out-of-plane behavior is coupled such that a three-dimensional method of determining the response is necessary;
4. develop a predictive analytical method that is more representative of the actual performance of buildings constructed of HCT infills than that offered by conventional code approaches;
5. evaluate the feasibility of NDE techniques for determining the strength and existing construction configuration of in situ walls; and
6. provide a mechanism for documenting the information gained from the testing, analysis, and peer review associated with the test program.

3.3 SCOPE OF THE HOLLOW CLAY TILE WALL TEST PROGRAM

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. These 20 original test types are listed in Table 3.1 along with five that were added later. A summary of all planned and completed tests is presented in Appendix J. As the test team's knowledge of masonry research broadened, a clearer understanding of the applicability of individual tests was achieved. From this, a more focused approach developed so that the final research product consisted of the following nine major areas of investigation:

Table 3.1. Comparison of planned and actual Hollow Clay Tile Test Program tests

Test	Number				Appendix where discussed
	Planned		Actual		
	In situ	Lab	In situ	Lab	
1. Unit tile compression	35	20	100	135	D
2. Unit tile splitting tensile	20	20	27	40	D
3. Unit tile initial rate of absorption	20	20	24	64	D
4. Mortar compression	(a)	(a)	18	0	B
5. Mortar compression	(b)	(b)	4	(b)	B
6. Mortar tensile splitting	®	®	0	6	F
7. In situ mortar bed shear test	6	0	18	0	C
8. Deformability	(d)	(d)	2	0	C
9. Flex bond strength	12	18	0	0	F
10. Bond wrench test	(e)	(e)	52	48	C,D
11. Prism compression	12	20	3	73	C,D
12. Prism splitting	18	24	0	0	F
13. Biaxial	(e)	(e)	0	0	F
14. In situ normal flatjack	(d)	(d)	5	0	C
15. Full scale out-of-plane air bag	2	2	1	3	E
16. In-plane HCTW infilled frame	1	13	0	15	E
17. Out-of-plane drift and in-plane	0	6	0	2	E
18. Wall column bending	0	2	0	2	E
19. Shake table test ^f			(f)	(f)	E
Cantilever wall tests	0	14	0	7	
Out-of-plane system	0	6	0	13	
In-plane system	0	6	0	0	
In-plane sine sweep	0	0	0	2	
20. Non destructive	(e)	(e)			F
21. Out binding	2	0	0	0	F
22. HCT coupon tests					F
Compression	0	0		4	
Tensile	0	0		3	
Modulus of rupture	0	0		3	
Moisture absorption	0	0		15	
23. Moisture absorption	0	0	24	24	D
24. Miniature HCT prism compression	0	0	0	4	F
25. Prism compression at an angle	0	0	0	9	D

^aPlanned to obtain random samples from various buildings.

^bTest mortar used to fabricate specimens per ASTM-C780, Annex A6.

^cTest mortar used to fabricate specimens per ASTM-C780, Annex A7.

^d1-3 walls in each building being analyzed.

^eExact number never determined.

^fTests performed on one structure.

1. literature review,
2. investigative studies of in situ conditions,
3. in situ constitutive property tests,
4. laboratory constitutive property tests,
5. moderate- and large-scale tests,
6. supplemental tests,
7. correlation of experimental and analytical results,
8. independent review findings and recommendations, and
9. application of test results to natural phenomena evaluation of DOE facilities.

These areas of investigation represent the chronology and order by which the test program was successfully completed. Each area and its relevance to the test program is briefly explained in the following sections. A more quantitative and detailed explanation of each area is presented in Appendices A through I, respectively.

3.3.1 Literature Review

The behavior of HCT, and particularly HCT infills, was extremely ill-defined at the outset of the test program. However, before the initiation of the experimental research, a thorough literature review was performed to (1) determine whether testing was necessary and (2) if so, the type and extent of testing to be performed. The search was divided into five major areas of focus related to infilled frames: (1) the use of HCT, (2) in-plane testing, (3) out-of-plane testing, (4) in-plane modeling, and (5) out-of-plane modeling. A discussion of the literature findings is presented in Appendix A. A complete listing of all pertinent internal and external references and cross-references is provided in Appendix K.

3.3.2 Investigative Studies of In Situ Conditions

To prepare and complete a successful test program for understanding the behavior of HCT infills, an investigation of the in situ infill conditions at the Y-12 Plant was necessary. This entailed investigating and documenting the strength and composition of the mortar (Butala September 1992), the as-constructed configuration of the HCT (particularly as it relates to the construction of 13-in. panels) and the boundary conditions and frame-to-infill connectivity. Once this was completed, the information was used to formulate tests based directly on the as-built parameters of the Y-12 buildings. A discussion of the investigative studies is presented in Appendix B.

3.3.3 In Situ Constitutive Property Tests

Many of the tests performed for the HCTW Test Program were conducted on specimens constructed in a laboratory with new materials. To correlate the relationship between laboratory and in situ behavior, some in situ testing was necessary. To this end, compressive strength tests were performed on three prisms; bond strength tests were performed on 52 HCT units; low-level elastic properties were determined by using five deformability tests and two normal stress tests; and shear strength was measured with seven push tests. A discussion of the in situ constitutive property tests is provided in Appendix C.

3.3.4 Laboratory Constitutive Property Tests

Because of the expense of extracting, monitoring, and testing in situ samples at the Y-12 Plant, a supplementary laboratory test process was required. Moreover, the lab testing was advantageous in that specific investigations could be performed in a more controlled and focused environment. Therefore, laboratory compressive tests were performed on HCT units and prisms, bond strength tests were performed on 8-in. specimens, and properties were defined for numerous mortar cubes and cylinders. A discussion of the laboratory constitutive property tests is provided in Appendix D.

3.3.5 Moderate- and Large-Scale Tests

One of the primary elements of the test program was the research on both in situ and laboratory moderate- and large-scale test specimens. Because of the similarity of the test structures to the unreinforced masonry infills at Y-12, this research area provided the clearest indicator of the response of the infills to the application of lateral loads. The following tests were performed:

1. In situ airbag test of a wall in Y-12 Building 9207,
2. Full-scale testing of laboratory replicas of the infills in Y-12 Building 9212,
3. Moderate-scale testing of 21 unreinforced masonry infills through the use of quasi-statically applied in-plane and out-of-plane loads, and
4. dynamic shake table testing of moderate-scale unreinforced masonry infills.

A discussion of the moderate- and large-scale tests is provided in Appendix E.

3.3.6 Supplemental Tests

Because of the size and complexity of the test program—as required to define the behavior of HCT infills—much of the information gained from the standard small-, moderate-, and large-scale masonry tests required refinement and clarification through supplementary test processes. For example, modal surveys helped to define the frequency and dynamic behavior of the moderate- to large-scale specimens, NDE provided information related to boundary conditions, and diagonal tension tests defined additional failure modes for masonry specimens (Oak Ridge Y-12 Plant August 1989). Other supplemental tests were coupon tests, miniature prism tests, biaxial tests, and K-25 shake table preparation. Some of the tests outlined in the program were found to be unnecessary or cost-prohibitive; thus, these tests were discontinued. A discussion of the supplementary tests is provided in Appendix F.

3.3.7 Correlation of Results

One of the goals of the HCTW Test Program was to develop analytical approaches that could be used in evaluating the structures and facilities at the Y-12 Plant. Both in-plane and out-of-plane infill behavior needed to be addressed. To determine the most appropriate method for predicting in-plane behavior, several commonly used methods (i.e., classical approaches) were investigated. Also, piecewise linear and nonlinear methods were examined. Once these methods were investigated and refined as necessary, the most applicable and accurate analytical techniques, as defined by the test data, were used in the overall analysis of Y-12 buildings. A discussion of the correlation of experimental and analytical results is provided in Appendix G.

3.3.8 Independent Review

A review committee of national masonry experts was assembled to provide advice on the development and conduct of the HCTW Test Program. The Expert Review Committee consisted of the participants shown in Table 3.2. The committee effectively assisted the test team in developing the program's original plan and also provided advice and guidance throughout the program. Five program reviews were conducted during the course of the test program. After each, the committee prepared a report containing program recommendations and findings that included adding new tests, eliminating tests, and ranking the importance of tests on the basis of timing and funding constraints. A summary of the independent review findings and recommendations is provided in Appendix H.

Table 3.2. Expert review committee members

Member	Affiliation	Location
Dr. James K. Noland	Atkinson Noland Associates	Boulder, Colorado
Dr. Daniel P. Abrams	University of Illinois	Urbana, Illinois
Dr. Samy A. Adham	Agbabian Associates	Pasadena, California
Mr. John Kariotis	Kariotis & Associates	Pasadena, California

3.3.9 Application of Results

The culminating step in the test program plan was the application of the results of the experimental research to the analysis of Y-12 structures (and other DOE facilities with HCT infills). First, parameters that define the ultimate capacity and limiting displacement for in-plane loads on unreinforced masonry infills were developed. Next, equations to predict in-plane stiffness were defined and criteria related to connections, structural damping, and combined loading effects were outlined. Also, out-of-plane loading issues, such as response to both story drift and inertial loads and the effect of in-plane loads on out-of-plane capacity, were investigated and explained. Finally, a recommended seismic evaluation criterion for the analysis of HCT walls was presented. The application of test results to the natural phenomena evaluation of facilities with HCT infills is provided in Appendix I.

4. RESULTS OF THE HOLLOW CLAY TILE WALL TEST PROGRAM

The nine major areas of investigation for which Appendices A through I provide detailed and quantitative explanations are as follows:

1. literature review,
2. investigative studies of in situ conditions,
3. in situ constitutive property tests,
4. laboratory constitutive property tests,
5. moderate- and large-scale tests,
6. supplemental tests,
7. correlation of experimental and analytical results,
8. independent review findings and recommendations, and
9. application of test results to the evaluation of DOE facilities.

Of these nine areas, the first two were preparatory functions whose purpose was to ensure that the test program was necessary and that the experimental research simulated actual conditions at Y-12. Areas 3 through 6 encompass the crux of the test program—that is, the testing of specimens for the purpose of understanding the behavior of unreinforced masonry infills. The function of Area 7 was to develop analytical approaches that are indicative of the behavior defined in Areas 3 through 6. Area 8 consisted of an independent review, and Area 9 addresses practical methods for applying the tests results to Y-12 structures (and other DOE facilities with HCT infills). The scope encompassed by the areas of investigation is outlined in Chapter 3. Results and conclusions for each are briefly presented in this chapter.

4.1 LITERATURE REVIEW

The literature review revealed that the use of HCT was relatively common in the 1940s and 1950s and began to decline dramatically thereafter. Similarly, because of the lack of code-based guidance on the use of unreinforced masonry within surrounding frames, modern construction of these structural systems was infrequent. As a direct result, very limited information was available on the in-plane and out-of-plane behavior of infills and even less on accurate predictive analytical approaches. Hence, to correctly analyze the structures and facilities at the Y-12 Plant, a test program to better understand HCT infills was necessary. A discussion of the literature findings is presented in Appendix A. A complete listing of all pertinent internal and external references and cross-references is provided in Appendix K.

4.2 INVESTIGATIVE STUDIES OF IN SITU CONDITIONS

For the mortar samples taken from Y-12 Buildings 9212, 9206, 9207, and 9204-2, it was concluded that the mortar batch constituents included Portland Cement, lime, and Tennessee River sand; the actual volumetric proportions are as listed in Appendix B. These proportions are very close to an American Society for Testing and Materials (ASTM) C270 Type N mortar. Average 28-day compressive strength for the mortar cubes prepared with the formulated batch proportions was found to be 2190 psi. The actual calculated volumetric proportions were used in all test

program research except the moderate-scale infill tests at the University of Tennessee, in which a standard Type N mortar was used.

The configuration of the HCT within the infilled frames is a parameter that directly affects the behavior of the infill—particularly for 13-in. walls. Several walls were investigated by disassembly and found to be composed of 4-in. and 8-in. HCT. The configuration is unique in that the 4-in. and 8-in. blocks alternate position from course to course, thereby creating a discontinuous collar joint and full composite action. The large-scale testing of replicas of the Y-12 walls (Sect. 4.6 and Appendix E) showed that this configuration is very stable and resistive to the application of lateral loads (Henderson August 1994).

Numerous pilasters and purlins were disassembled in Building 9212. This investigation showed that, for 13-in. infills, panel-to-column connectivity was achieved by placement of mortar and scrap within the confines of the pilaster and column flanges. Purlins were found to be connected to the infill by embedment of the flanges into the HCT and mortar. These connections also proved to be very effective in confining and stabilizing the infill material.

The results of these studies were used in the construction of all applicable test specimens. Therefore, the experimental research provided behavior and capacity data as close to as-built conditions as possible. A complete discussion of the investigative studies is provided in Appendix B.

4.3 IN SITU CONSTITUTIVE PROPERTY TESTS

Prism compression tests, bond wrench tests, flatjack tests, and shear tests were performed on specimens extracted from various buildings at the Y-12 plant. Gross and net compressive strength (f'_m) and modulus of elasticity (E_m) were obtained for three prism specimens. Compressive strength is a key parameter in the prediction of in-plane ultimate capacity and, thus, the resistance to seismic and wind loads.

Bond strength tests were performed on 52 specimens from Buildings 9207 and 9204-2. Bond strength data has been shown to be a good indicator of the resistance of the infill to out-of-plane drift loads. Bond strength was an important parameter in the out-of-plane drift predictive methodology described in Appendix G.

Flatjack testing was used to determine the average in situ normal stress as well as compressive strength of in situ specimens. This technique proved to be very effective in that these parameters may be determined with limited damage to the infilled panel.

Unidirectional and reversed cyclic shear tests were performed on seven in situ specimens. This testing demonstrated that the behavior of mortar beds in shear for an 8-in. HCT infill is similar to that for 13-in. walls.

A complete discussion of the in situ constitutive property tests is provided in Appendix C.

4.4 LABORATORY CONSTITUTIVE PROPERTY TESTS

The laboratory testing of unit blocks provided a good understanding of the properties of individual HCT and how the behavior and capacity of original building tile compared with that of newly constructed tile. Overall, it was concluded that using the new block to construct HCT infilled frames for testing should provide laboratory specimens that would indeed be reasonably representative of the performance of the HCT infills of the Y-12 Plant buildings.

The laboratory testing of prisms provided two key properties needed for the analysis of Y-12 infilled structures—Young's modulus and compressive strength (Boyd 1993, Yancey August 1993a, b). Prism compression strength was an important element in the piecewise methodology developed for determining the overall seismic and wind capacity of the HCTWs in the plane of the wall. Also, one important qualitative result of these tests was that for prisms tested with the load parallel to the cores, the failure mechanism was extremely ductile as compared with the more brittle failure observed when load was applied normal to the cores. Because side construction was typically used for the infills at Y-12, the principal resistance of the HCT against the framing is parallel to the cores for in-plane lateral loads. This phenomenon is believed to have contributed to the observed global ductility experienced in all of the in-plane tests conducted for the test program.

Laboratory bond wrench testing was conducted on a total of 48 8-in. specimens—23 in conjunction with the full-scale Y-12 replica testing and 25 associated with the moderate-scale infilled frames tests. This research provided an indication of the resistivity of the moderate- and full-scale infill specimens to out-of-plane drift loads. Also, bond strength values were used for simulation of the out-of-plane stiffness degradation resulting from out-of-plane drift, as presented in Appendix G. Finally, these values provided a means of comparing bond strength for in situ and laboratory-constructed specimens.

A complete discussion of the laboratory constitutive property tests is provided in Appendix D.

4.5 MODERATE- AND LARGE-SCALE TESTS

An in situ airbag test was performed on a 28-ft-wide by 12-ft-tall infilled wall panel in Y-12 Building 9207. Cracking was first noticed at a pressure level of 0.28 psi. The ultimate capacity of the infill was reached at an airbag pressure of approximately 1.0 psi and a deflection of 1.68 in. From this testing it was concluded that the development of in-plane membrane stresses greatly enhances the ability of the wall to resist lateral pressure loads. Conventional analysis techniques showed that, in the out-of-plane direction, the HCTW would begin to develop unacceptable performance at an earthquake level as low as 0.05 *g* and experience significant damage at a level of around 0.1 *g*. However, the out-of-plane strength of the wall was demonstrated to be many times greater than that predicted by conventional theories that do not account for postcracking mechanisms, especially the arching action of the infill within the steel frame (Huff and Fricke 1992).

Twenty-one moderate-scale (approximate 8-ft nominal width and height) infilled frames were tested with combinations of in-plane forces, out-of-plane drift, and out-of-plane inertial loads. Also, two full-scale (24-ft wide by 23-ft tall) replicas of infilled frames at Y-12 Building 9212 were

constructed in the laboratory and tested to failure with in-plane and out-of-plane drift loads. For both tests series the following conclusions may be drawn:

1. The infill tends to perform as a compressive strut with two observable failure mechanisms: diagonal cracking and corner crushing.
2. After first cracking, significant residual lateral load capacity remained.
3. After the ultimate lateral load was reached, the walls continued to absorb significant amounts of energy.

The moderate-scale tests also showed that in-plane racking reduced the uniform out-of-plane load capacity by 20% (Flanagan December 1994). The full-scale tests demonstrated the following additional conclusions (Henderson August 1994):

1. The frame-to-infill connections and HCT configuration in Building 9212 are more than adequate to ensure full composite behavior of the infill.
2. Despite numerous cracks (induced by out-of-plane loads) that fully penetrated the thickness of the infill, the test structure was still completely stable and resistive to lateral load.
3. Prior out-of-plane damage reduces initial in-plane stiffness but has little effect on in-plane capacity.

Shake-table tests were performed to assure that the results of the larger number of static and quasi-static tests were representative of the infill response to lateral loads. One test structure, consisting of two infills connected by orthogonal trusses, was tested with 22 types of acceleration input. The input consisted of sine sweeps and showed conclusively that the HCT remains within the framing when subjected to ground acceleration values within a frequency range from 4 to 10 Hz. The results of the shake-table tests were indicative of the findings from the quasi-static testing.

A discussion of the moderate- and large-scale tests is provided in Appendix E.

4.6 SUPPLEMENTAL TESTS

Modal surveys were performed on some of the moderate- and large-scale infilled test specimens. These tests provided an estimate of lower-mode frequencies and damping as well as an indication of the extent of damage present after out-of-plane loads were applied. Dynamic input motion produced classical cantilever mode shapes for the infill specimens. Cracking was shown to be extensive after the out-of-plane loads due to the inability of the input function to propagate below the crack patterns. Despite this damage, however, the infills were still stable and resistive under the application of in-plane loads.

NDE techniques (i.e., sonics, ultrasonics, thermography, and microwave) were tested at Y-12, K-25, and ISU to assess the ability of the NDE equipment to determine bond quality, construction irregularities, and local mortar conditions. The most promising techniques, as demonstrated by the investigation, were thermography and microwave technology; however, NDE techniques were not fully explored due to budget constraints (Wynn, Fletcher, and Jones 1992).

Other supplemental tests were performed and some were discontinued. A complete discussion of all planned and performed supplemental testing is provided in Appendix F.

4.7 CORRELATION OF RESULTS

Several methods for the prediction of in-plane stiffness and capacity of infills developed in the 1950s and 1960s are still used frequently today. These "classical approaches" were examined and compared with the full-scale test data to assess their accuracy in predicting the capacity of HCT infills. This research indicated that because of the strength variability and orthotropic nature of HCT, accurate predictions of ultimate capacity by these approaches was difficult (Henderson August 1994).

It was found that a piecewise linear in-plane strut formulation—a modification to one of the classical approaches previously described—was generally effective in simulating the hysteretic behavior of the tested infills. In this approach, the column bearing and the width and area of the compression strut that resists in-plane loads are calculated. This method is currently being used to simulate the stiffness of infilled panels in many of the Y-12 critical facilities (Flanagan December 1994).

Resistance to out-of-plane pressure loads, such as results from inertia of the infilling due to ground motion, was addressed as a function of arching action. The one-way arch model (Chua 1991) predicted the out-of-plane capacity of the Building 9207 airbag test quite closely. The primary conclusion from the experimental testing, in conjunction with the pressure models, was that due to the arching action phenomenon, Y-12 infilled structures can resist out-of-plane pressure loads due to wind and seismic forces at levels much higher than predicted by conventional (code-based) techniques.

Analyses have indicated that for many of the structures at the Y-12 plant—particularly the wings of Building 9212—out-of-plane damage induced by ground motion would generally be a function of seismic drift as opposed to inertia of the infill material. It was determined that few analytical approaches were available for the prediction of the capacity of an infill when subjected to seismic drift. Therefore, a piecewise linear method was developed for predicting stiffness, capacity, and cracking patterns. This method predicted the experimental load/deflection behavior for the full-scale test specimens within 5% (Henderson August 1994).

A discussion of the correlation of the experimental and analytical results is provided in Appendix G.

4.8 INDEPENDENT REVIEW

Throughout the test program, a panel of nationally recognized masonry experts advised the test team on numerous topics related to the experimental and analytical research. Five reports were presented to MMES. This interaction with the test team was very effective both in planning and implementing the testing as well as forming conclusions on the large array of data.

A complete discussion of the independent review is provided in Appendix H.

4.9 APPLICATION OF TEST RESULTS TO NATURAL PHENOMENA EVALUATION OF U.S. DEPARTMENT OF ENERGY FACILITIES

On the basis of the test program, an equivalent strut model has been developed to represent the stiffness of the HCT infill. Through a procedure to evaluate the secant stiffness, the strut stiffness is modified to follow the load deflection relation observed in tests. Nineteen in-plane tests conducted under the HCTW program were reviewed to establish maximum test capacity and associated displacement by using the average load and displacement (average of push and pull) obtained during the peak loading cycle. A prediction equation with parameters of effective masonry strength (geometric average of strength normal and parallel to the HCT cells) and wall thickness closely matches the test data. The behavior at capacity is governed by crushing of the HCT in a corner of the wall.

Equivalent damping from wall-load-displacement hystereses has been estimated to be on the order of 5% equivalent viscous damping. System damping used for seismic response analyses should be determined by adding the effective hysteretic damping to the elastic material damping. A value of 10% system damping is reasonable for HCTW buildings.

The testing seems to support the contention that only the wall thickness contained within the steel frame is effective in creating a compressive strut. It is recommended that only the steel column EI be considered as the effective EI of the pilaster in the evaluation of equivalent strut properties.

The behavior of walls to imposed out-of-plane drift and the behavior of walls with out-of-plane lateral loading was considered in the test programs. HCT walls appear to be able to withstand story drifts on the order of 1% without experiencing any significant distress.

A discussion of the application of results is provided in Appendix I.

5. CONCLUSIONS

Following a 3-year testing program in which over 700 tests were conducted, the results clearly show that the HCT infills at the Y-12 plant are significantly more stable and laterally resistive than previously anticipated by conventional, code-based approaches. The infill material provides considerable lateral stiffness and the steel frame adds ductility and confinement to the overall system. Furthermore, the infill material remains within the surrounding frame when subjected to seismic and wind loads at levels much higher than previously estimated. That is, the infill remains in place for development of the full capacity of the walls in-plane. Similarly, the significant differential displacement between the frame and the infilling that was recorded for the full-scale specimens provided additional ductility and energy absorption.

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 with no explosive or brittle failure of the HCT.

The large-scale testing demonstrated that applied, out-of-plane differential displacement (top to bottom), as would be induced by seismic drift, causes cracking almost exclusively in the bed joints of the infill. The bed joint cracking propagates such that, with large displacements, the infill behaves as a series of horizontal beams separated by numerous complete through-cracks. This cracking has little or no effect on the stability of the infill or on the ultimate in-plane capacity. Prior damage to the infill does, however, reduce the initial in-plane stiffness. The moderate-scale testing indicated that in-plane and out-of-plane behavior is decoupled.

Diagonal cracking and corner crushing were identified as the predominant in-plane failure mechanisms. During postpeak cyclic loading, the infill remains resistive and continues to absorb significant energy while the HCT cracks and some crushing of individual tile occurs at the frame corners.

On the basis of the testing, the in-plane stiffness of the infill is to be incorporated into the full-building model as a piecewise linear representation of an equivalent in-plane strut. As the infill wall reaches various levels of strength degradation due to continuous cracking, the strut effective stiffness is reduced as a function of the frame displacement. This strut formulation was chosen over the shear-wall approach because of (1) the need to accurately account for the nonlinear behavior of HCT with increasing lateral load and (2) the inordinate computational effort required to use planar finite elements to represent the postcracking infill response.

Several nondestructive evaluation techniques were evaluated during the test program for the purpose of determining frame-to-infill connectivity, mortar condition, etc. These methods were found to be of limited value due to the intrinsic complexity of the materials involved.

During 1993 and 1994, new evaluations of a number of the Y-12 Plant buildings were conducted. The results of these analyses have shown that many of the buildings previously evaluated do indeed meet the requirements of DOE Standard DOE-STD-1020-94, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* (previously UCRL-15910).

Finally, the HCTW Test Program has shown that the modeling techniques used in estimating the capacities of HCTW buildings in the 1970s and 1980s significantly underestimated the performance of those buildings. The HCTW Test Program has provided a more appropriate methodology for modeling the HCT infilled buildings and determining their earthquake and wind load capacities. As determined by a thorough peer review (EQE International, Inc. 1995), the HCTW Test Program provided extremely high-quality and well-documented experimental data, useful not only to Y-12 but to the engineering community in general. Using the results of the HCTW Test Program in the evaluation of buildings at the Y-12 Plant should avoid the expenditure of millions of dollars in unnecessary building upgrade costs.

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Appendix A

LITERATURE REVIEW

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ABSTRACT

The behavior of hollow clay tile (HCT), and particularly HCT infills, was extremely ill-defined at the onset of the Hollow Clay Tile Wall Test Program. However, before embarking on a comprehensive process of experimental research, a thorough literature review was performed to (1) determine whether testing was necessary and (2) if so, the type and extent of testing to be performed. The literature review revealed that the use of HCT was relatively common in the 1940s and 1950s and began to decline dramatically thereafter. Similarly, because of the lack of code-based guidance on the use of unreinforced masonry within surrounding frames, modern construction of these structural systems was infrequent. As a direct result, very limited information was available on the in-plane and out-of-plane behavior of infills and even less on accurate predictive analytical approaches. Hence, to enable correct analysis of the structures and facilities at the Y-12 Plant, a test program to better understand HCT infills was necessary.

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Appendix A

LITERATURE REVIEW

The behavior of hollow clay tile (HCT), and particularly HCT infills, was extremely ill-defined at the onset of the Hollow Clay Tile Wall Test Program. However, before embarking on a comprehensive process of experimental research, a thorough literature review was performed to (1) determine whether testing was necessary and (2) if so, the type and extent of testing to be performed. The results of the literature review are described in this appendix.

A.1 USE OF HOLLOW CLAY TILE¹

Some limited information on HCT materials is available in the public literature. The majority of the information found describes testing of tile units and small assemblages (ASTM C-10 1924, ASTM C-10 1926, Parsons 1931, Plummer 1962, Hathcock and Skillman 1918). Some testing of larger assemblages and wall sections has been performed (Matthys 1972; Johnson and Matthys 1973; Parsons and Watstein 1937; Stang, Parsons, and Foster 1926; SCPI 1967; SCPI 1969; Whittemore and Hathcock 1923; Whittemore, Stang, and Parson's 1938).

Little information on testing of structural framing infilled with clay tile was found. Some historical data on the behavior of infilled framing in natural disasters are available. A comprehensive review of building performance in the 1906 San Francisco and 1925 Santa Barbara earthquakes indicated that properly constructed HCT infills performed reasonably well (CPIC 1929).

A.2 IN-PLANE TESTING OF INFILLED FRAMES²

Structural frames infilled with masonry walls have been shown to provide significant in-plane strength and stiffness in resisting lateral loads. Laboratory in-plane tests of clay brick, concrete masonry, and microconcrete infills have been conducted by several researchers. For this type of construction, the rigidity of the masonry infill stiffens the more flexible steel framing. The steel framing confines the more brittle masonry, thereby providing a higher ultimate strength and allowing greater displacement of the infill wall than it would withstand if not enclosed in the steel frame.

In-plane behavior and capacity are directly related to the modulus of elasticity and compressive strength of the masonry infill. Prism compression testing on masonry subassemblages and the testing of individual unit blocks are the generally accepted methods for determining modulus and compressive strength values. Numerous papers have been presented on testing and analysis methods as well as the effect that strength and modulus have on generalized panel

¹Portions of this section were extracted from Flanagan (December 1994) with permission from the author.

²Portions of this section were extracted from Flanagan (December 1994) and Henderson (August 1994) with permission from the authors.

capacities (Ganesan and Ramamurthy 1992, Hathcock and Skillman 1918, Hegemier et al. 1978, Page and Shrive 1988).

Early research on the in-plane behavior of concrete-encased steel frames infilled with brick and clinker-block was conducted by Wood (1958, 1978). The infill testing was part of a larger study of frame stability and included monotonic racking tests of near-full-scale specimens. Several failure modes were reported. A shear or shear-rotation mechanism was observed in tests with strong frame members. These failure modes included panel cracking followed by the formation of plastic hinges at the frame joints, rigid-body rotation of unrestrained panel corners, and diagonal tension failure in the panel. Large frame and panel distortions were observed in these tests. For stronger walls, the panel distortions were accompanied by the formation of plastic hinges along the beam length. For weaker frames (and stronger walls), compression bands in the brickwork were observed as the infill developed significant diagonal cracks. Diagonal-compression or corner-crushing failures were then observed. Of note is that the strength and stiffness of even the weaker frames were quite high compared with those of the bare steel framing. Also, the large frame distortions resulting from the composite action indicate a relatively strong infill material.

Polyakov (1960, 1963) investigated various parameters affecting the behavior of infilled steel frames. He performed 65 large-scale tests in which he varied masonry units, mortar, method of loading (monotonic or cyclic), and wall openings. In addition to individual panels, he tested a three-story, three-bay frame of about one-quarter scale. Monotonic load-unload cycles to ultimate capacity were performed with a hydraulic jack at a single upper joint of this test structure. Diagonal cracking occurred first in the corner panel nearest the load input, but similar cracking appeared in the other panels as the loading progressed. The deformation characteristics of the nine-panel specimen compared well with the characteristics of a single identical panel used for calibration. Polyakov concluded that the multistory, multibay infill system behaved as a group of individually braced frames and not as a single monolith.

Stafford-Smith (1962, 1966, 1969) investigated the behavior of relatively small mortar-filled steel frames subjected to diagonal and racking loads. For these specimens, he observed two dominant failure modes: diagonal cracking and corner crushing. Much of this research focused on determining the contact length at the frame-infill boundary and approximating an effective equivalent strut width. Stafford-Smith (1968) also investigated the effect of combined vertical-pressure and racking loads on infills. He concluded that, at moderate vertical load increases, the panel horizontal stiffness and strength increased because of a greater contact area and a more even distribution of stress in the masonry. The optimum vertical pressure was found to be about one-half that of a purely vertical failure load. For combined-loading panels with a vertical load less than one-half the ultimate strength, failure modes were similar to those for horizontal loading only. For combined-loading panels with a vertical load greater than one-half the ultimate strength, the failure modes were similar to those of vertical loading only (i.e., general compression failure, sometimes preceded by vertical cracks).

Klingner and Bertero (1978) tested a one-third-scale subassembly of an 11-story infilled frame building. This three-story prototype consisted of reinforced concrete framing with fully grouted and doubly reinforced concrete-masonry infills. Repeated cycles of static loading gradually decreased the strength of the infills to that of the bare frame. The increases in strength and energy dissipation afforded by the infill exceeded the potentially detrimental effects of increased inertial forces produced by the added stiffness of the infill.

Liauw and Kwan (1983) performed a series of small-model microconcrete-infill tests. On the basis of the experimental work, they developed a plastic-collapse theory for masonry infills. The plastic-collapse load is dependent on the properties of the infill, the aspect ratio, and the relative stiffnesses of the frame and infill. Three failure modes were identified for nonintegral frames: corner crushing with failure in the columns, corner crushing with failure in the beams, and diagonal crushing with failure toward the interior of the panel.

Riddington (1984) investigated the influence of initial gaps on infilled-frame behavior. He tested a series of approximately half-scale specimens with no gaps, top gaps, and top and side gaps. Significantly lower stiffness and slightly lower ultimate capacities were observed for the frames with initial gaps. As in previous research, diagonal-cracking and corner-crushing failure modes were found. Riddington concluded that initial gaps were largely undesirable and that the practice of providing initial gaps between masonry and concrete frames to offset creep effects should be reconsidered.

Dhanasekar, Page, and Kleeman (1985) tested steel framing with grouted brick infills to verify a biaxial, masonry constitutive model. The specimens were tested under increasing monotonic racking loads. The capacity of infills with stiffer frames was somewhat larger than those with weaker frames. This was attributed to greater contact length along the stiffer frame. After initial failure of the infill, the composite stiffness approached that of a corresponding bare frame yet continued to resist a portion of lateral load by frictional shear in the planes of failure.

Klingner, Beinev, and Amrhein (1987) investigated the 1985 Mexico City earthquake. They found that modern medium-rise and high-rise structures with masonry infills generally performed better than otherwise similar structures without infills. Masonry infills generally appeared not to have been designed as structural elements. The infills were usually unreinforced and were not connected to the surrounding frames. In some instances, the infills suffered severe damage; however, they appeared in many cases to have prevented building collapse.

Dawe and Seah (1989a) investigated the behavior of concrete-masonry-infilled steel frames by testing 28 large-scale specimens with monotonic racking loads. Their intention was to study a broad spectrum of characteristics that might affect infill stiffness and strength. Major observations from the testing are as follows: (1) in general, off-diagonal cracking led to reduced capacity, and therefore narrow bands of tension cracking along the diagonal are desirable; (2) tightly packed mortar between the infill and column flanges increased initial stiffness but did not affect initial panel cracking load or capacity; (3) panel-to-column ties increased the initial stiffness but did not affect initial panel cracking load or capacity; (4) reduction of panel-to-frame interface bond with a polyethylene membrane resulted in lower first panel crack load and ultimate capacity; (5) horizontal-joint reinforcement reduced random cracking in the panel but did not affect the capacity of the infilled frame; (6) reinforcement of the compression diagonal increased the first major cracking load and ultimate load; (7) bond beams increased the precracking stiffness and increased the initial major panel cracking load to near ultimate; (8) a 20-mm gap between the panel and top beam significantly reduced the infill stiffness and strength; (9) openings in panels reduced the initial major crack load and ultimate capacity; and (10) vertical reinforcing around openings increased the initial stiffness but not the initial major panel crack load or ultimate capacity.

El-Ouali, Houde, and Tinani (1991) performed cyclic static tests of seven half-scale infills. Lime-and-sand bricks and clay bricks were used in the specimens. They observed a general

decrease in rigidity of the infilled frame with repeated cyclic loading. Cracking was generally stabilized within three cycles of the same imposed displacement.

Schuller et al. (1994) performed in-plane testing of reinforced-concrete moment-resisting frames infilled with hollow or solid concrete-masonry units. Strong and weak frames were tested. Vertical load was applied to the columns and in some instances to the top beam. Static monotonic and cyclic loadings were employed. Preliminary observations were that stronger infills exhibited higher load-resistance and better energy-dissipation capabilities. However, the postpeak strength of stronger infills dropped more rapidly than that of weaker infills as the displacement increased. Stronger infills were also recognized to be more likely to cause brittle shear failure in the concrete columns.

Mander et al. (1993) and Mander, Aycardi, and Kim (1994) studied the effect of brick infill on steel framing. A three-story, three-bay frame subassemblage was tested statically with the center bay infilled in each story. For the cyclic in-plane loading, good energy-absorption capabilities were observed for the infill with relative stability between successive hysteretic loops of the same displacement magnitude. Primary diagonal-strut action was observed in the infill, and secondary struts developed as the loading continued. In-plane strength relationships based solely on the prism's compressive strength were provided. Both reinforced and unreinforced ferrocement retrofits were applied to the infills after initial testing. For the unreinforced ferrocement retrofit, only a slight strengthening was achieved, but additional energy dissipation was reported. Ferrocement retrofit with diagonal reinforcement strengthened the panels by about 40% with continued hysteretic energy absorption. Most of the strength increase was caused by the tensile capacity of the reinforcement; however, a portion of the increased lateral capacity was attributed to additional confining action of the diagonal strut.

Infills may have particular benefit in moderate seismic zones. The October 12, 1992, Cairo, Egypt, earthquake demonstrated the positive attributes of infilling in a moderate earthquake region (Adham 1994). Reinforced concrete buildings with unreinforced masonry infills performed well in this earthquake. The infills served to increase the stiffness of flexible-frame buildings on soft soils and provided a redundant load path for both horizontal and vertical loads. Similar behavior was seen in the March 13, 1992, Erzincan, Turkey, earthquake (Bruneau and Saatcioglu 1994). The infills controlled the seismic drift and, in many cases, resisted the seismic forces elastically. Several problems were noted where lack of infills in a lower floor created a soft story, and debris from damaged infills blocked exits.

A.3 OUT-OF-PLANE TESTING OF INFILLED FRAMES³

Numerous tests have demonstrated that infilled frames are very efficient structures in that the masonry provides significant stiffness to the system, whereas the framing members add considerable ductility (Dawe and Hatzinikolas 1988). The state of current research on out-of-plane behavior of masonry-infilled frames is limited almost exclusively to pressure loads on moderate-scale test specimens (i.e., planar dimensions of 8 to 10 ft). Some large-scale and/or in situ pressure tests have been performed (Cavanaugh, Webb, and Hodgkinson 1982 and 1986), although the

³Portions of this section were extracted from Flanagan (December 1994) and Henderson (August 1994) with permission from the authors.

number of these is very limited. The application of pressure loads to masonry infills was first attempted to simulate wind or seismic inertial forces by applying numerous lateral point loads via a series of hydraulic jacks (Thomas 1953). Later, large pressure loads were determined to be applied more easily and uniformly with airbags, and this method is used almost exclusively for current research (Dawe and Seah 1988, 1989a,b; Drysdale and Essawy 1988).

Thomas (1953) tested large-scale brick panels bounded by concrete-encased steel framing. The infill-to-frame interface consisted of filling the panel perimeter with mortar. Sixteen points of lateral load were used to simulate uniform distribution. Significant capacity after panel cracking was observed. The importance of panel-edge restraint and vertical load in developing postcracking capacity was noted. One infill tested with both top and side gaps yielded a much lower capacity.

McDowell, Mckee, and Sevin (1956) were among the first to define arching action as the mechanism by which masonry exhibits much greater strength than predicted by conventional three-point bending analysis. Their research involved lateral load tests of brick beams constrained axially between rigid supports. One-way arching developed as beams were loaded at the third point. Failure occurred as the compression zones (at the ends and center of the span) of the arch crushed.

Gabrielson and Kaplan (1977) examined arching of unreinforced masonry walls under blast loadings in a shock tunnel. The brick panels cracked in flexure but withstood tremendous additional forces because of arching. After cracking, specimens resisted maximum equivalent accelerations greater than 1 g as subsequent rebound shock pulses were applied. Some panels exhibited "rigid arching" where the panels were tightly fitted between rigid supports, and developed a symmetrical arch that failed by crushing. Other panels exhibited "gapped arching" where the panels were separated from a support by a small gap and developed an unsymmetrical arch that failed by tensile spalling. Gabrielson and Kaplan also cited examples of arching action in several structures surviving the 1967 Caracas and the 1971 San Fernando earthquakes.

Dawe and Seah (1989b) tested nine large-scale concrete-masonry-infilled steel frames. Monotonic uniform lateral loading was provided with air bags. Significant arching behavior was observed for each specimen, even a dry-stacked, mortarless infill.

Abrams, Angel, and Uzanski (1993) conducted tests of large-scale clay- and concrete-block infills in concrete frames. These tests focused on the transverse behavior and strength of previously damaged infills. Specimens were first subjected to in-plane load reversals and then subjected to monotonic static lateral pressures from an air bag. The effect of varying the mortar type and the infill's height-to-thickness ratio were investigated. It was concluded that prior in-plane damage can reduce the out-of-plane strength of slender brick panels by as much as one-half. Although few data points were obtained to support the conclusion, it was suggested that the transverse strength may be proportional to the inverse of the square of the h/t ratio.

Mander et al. (1993) and Mander, Aycardi, and Kim (1994) tested brick-infilled steel frame under out-of-plane dynamic sinusoidal sweep motion. Portions of the specimens were tested both with and without prior in-plane loading. Significant out-of-plane capacity was observed with some loss of strength because of prior in-plane loading. Virgin panels with h/t ratios of 18 withstood out-of-plane response accelerations of approximately 10 g.

Hill (1994) tested three in situ, unreinforced concrete masonry infills and one in situ, unreinforced, clay tile infill out-of-plane with point loads distributed by means of bearing plates.

The goal was to produce full-scale test data from these actual building structures. Significant panel thrust forces developed from arching, as indicated in the measurable floor displacements.

Most research on out-of-plane loading of masonry infills has centered on uniform panel loads that attempted to emulate wind or inertial effects of the wall (Lawrence 1979). However, Benedetti and Benzoni (1984) have suggested that out-of-plane, interstory drift may be a more severe loading of walls of typical masonry buildings than inertial loading. During a seismic event, in-plane forces create out-of-panel drift displacements in orthogonal walls. Although the contribution to in-plane stiffness by orthogonal walls is typically ignored, the relative top-to-bottom displacements of the orthogonal walls must be considered. Stability of the orthogonal walls must be maintained as they drift out-of-plane. Considering an earthquake component in the building's other principal direction, the in-plane capacity of walls subjected to orthogonal drift displacements must also be determined.

Little or no empirical information is available regarding the effects of out-of-plane drift on infills. However, some important conclusions can be drawn from the existing experimental and analytical research regarding pressure loads on infills. This information indicates that the behavior and ultimate capacity of infilled frames are dependent to a large degree on the panel-to-frame interface characteristics (Adham 1985; Cavanaugh, Webb, and Hodgkinson 1982, 1986; Dawe and Seah 1988) and the material properties of the infill (Essawy and Drysdale 1986, Fried, Anderson, and Smith 1988, Page 1988). The panel-to-frame interface is generally dependent on the in situ construction techniques and may be determined by destructive or nondestructive evaluation methods. Various reliable methods exist for determining masonry and mortar properties through the use of small-scale testing, as outlined in compendiums on masonry testing by Hendry (1987) and Mayes and Clough (1975). The rupture strength of the masonry-to-mortar interface is of particular importance in understanding and simulating out-of-plane loads. The primary methods for determining this property include the flexural bond test (Baker 1979) and the bond wrench method (ASTM 1072-86, Brown and Palm 1982, Copeland and Saxer 1964, Hughes and Zsemsery 1980, Sarker and Brown 1987).

A.4 IN-PLANE MODELING OF INFILLED FRAMES⁴

Although considerably more experimental research has been performed on the behavioral response of infilled frames to in-plane loading (as compared with out-of-plane loading), few testing sequences have culminated in accurate and definitive means of predicting this response (Dawe and Seah 1989, Priestly 1987). Moreover, little or no information or test data are available regarding the effects of prior damage on the in-plane strength of infilled frames.

The first published research on infilled frames subjected to racking loads was by Thomas (1953). Both in-plane and out-of-plane testing performed at the Building Research Station in London were described in this article. The in-plane tests consisted of four sets of 9- by 11-ft infilled frames with unreinforced-masonry infilling, which were tested by applying racking loads at the top corner. The infilling for the tests consisted of 4.5-in. brick, 4.5-in. brick with a central door opening, 3-in. hollow clay tile, and 3-in. clinker block. Interestingly, the 3-in. hollow clay

⁴Portions of this section were extracted from Henderson (August 1994) and Flanagan (December 1994) with permission from the authors.

tile had a capacity consistent with that of the in-plane testing described in this research; however, the concrete-cased steel frame was significantly stiffer.

Polyakov (1956) reported findings from research on 65 moderate-scale infilled-frame racking tests. Of these infills, 33 were approximately 4 by 4 ft, and the remaining ones were approximately 6.5 by 10 ft. Various test parameters, including loading method, masonry type, and mortar-mix characteristics, were studied, as well as the effect of openings on lateral resistance. Polyakov indicated that infills exhibit three stages of behavior as loading is increased toward failure. In the first stage, the infilling and steel frame behave compositely. In the second stage, infill-frame separation results from shortening and lengthening of the compression and tension diagonals, respectively, and culminates in stair-step cracking of the compression diagonal. In the third stage, excessive widening of the compression diagonal cracks occurs, and any practical contribution from the infill becomes limited. The theory of elasticity for isotropic materials was used to predict strains in the infilling and ultimate strength of the panels. Polyakov's subsequent research (1960) was conducted on a three-story, three-bay frame containing nine panels approximately 4 by 4 ft. From this study, Polyakov concluded that the infilling could be replaced with an equivalent compression diagonal. This work appears to be the inception of what has come to be known as the equivalent-diagonal-strut method.

Replacing the infill with an equivalent diagonal was further advocated in a paper by Holmes (1961) in which he suggested that the strut area should be a function of the infill thickness and diagonal length because of the linear variation of the reaction along the planar dimensions of the frame. Stafford-Smith (1962) published the results of tests on small-scale mortar panels and modeled the experimental results by applying finite-difference approximations to the biharmonic equation. He also concluded that infilled frames could be represented by equivalent diagonal struts. After numerous papers, Stafford-Smith culminated his strut research in a paper with Carter (1969) indicating that strut width and thereby in-plane stiffness and ultimate strength are functions of the column-to-infill contact area. Numerous authors have attempted to reproduce his test results with the classical strut methods (Chrysostomou, Gergely, and Abel 1988; Dawe and McBride 1985; King and Pandey 1978; Leuchars and Scrivener 1976) or modifications thereto (El-Ouali, Houde, and Tinawi 1991, Gergely et al. 1993, Klingner 1980, Thiruvengadam 1985, Wolde-Tinsae 1987) with varying degrees of success.

As might be expected, analytical methods, including the compression strut, have difficulty in predicting actual behavior and ultimate capacity as measured in experiments. Other analytical methods include the plastic-collapse theory (Liauw and Kwan 1983, 1984, 1985), partial-plastic-collapse theory (Wood 1978), and various finite-element algorithms (Ganesan and Ramamurthy 1988; Wolde-Tinsae, Lhadj, and Colville 1987). Studies by McBride (1984) as well as Dawe and Yong (1985) on 18 masonry infills produced results that varied widely between theory and experiment. These authors found that the equivalent strut method was inaccurate by 50% to 450%, whereas the finite-element methods produced excellent results before cracking but much less accurate results for postcracking behavior. Dawe concluded that the most accurate method, when compared with the experimental results, was the partial-plastic-collapse theory. This method suggests that a penalty factor should be applied to the analysis results to account for the panel's nonideal plasticity under increasing load.

As indicated by Stafford-Smith and Carter (1969), the verification and refinement of the compression strut method as well as other practical analysis procedures is important in that, because of the variability of masonry construction and materials, the use of cumbersome theoretical

methods will not result in greater accuracy. For example, the most frequently observed failure modes that result from static or cyclic (racking) in-plane tests are corner crushing, shearing of block or mortar, or diagonal tension (Dawe and McBride 1985). However, minor inconsistencies in the testing parameters (gaps, openings, confinement by the surrounding frame, etc.) can produce significant variations in the overall load-deflection slopes and magnitudes. Studies indicate that slight gaps between the confining purlin and the infill can reduce the ultimate load by as much as 50% (Dawe and Hatzinikolas 1988; Wolde-Tinsae et al. 1987). Conversely, the addition of ties between the masonry panel and surrounding frame appears to have little effect on the ultimate in-plane capacity (Dawe and McBride 1985).

A.5 OUT-OF-PLANE MODELING OF INFILLED FRAMES⁵

Laboratory testing in conjunction with correlative modeling attempts indicates that current analytical simulation techniques are, at best, very approximate (Essawy and Drysdale 1987). The principal methods for the analysis of inertia and wind loads on infills, which include the elastic-plate theory, strip method, yield-line analysis, and principal-stress method, are most effective when the infill's increased lateral resistance because of arching is included (Cohen and Laing 1956; Lawrence 1979; McDowell, McKee, and Sevin 1956).

Dawe and Seah (1989b) proposed a modified fracture-line method of calculating the ultimate strength of infill panels subjected to uniform out-of-plane loading. This procedure incorporated arching effects and the effects of frame stiffness and deformation. Tensionless infill properties and a plastic-stress block for masonry in compression at regions of contact along yield lines and at panel boundaries were assumed. Various fracture-line patterns were proposed on the basis of the infill boundary conditions. In comparison with experimental results, this method proved quite accurate in predicting the ultimate capacity. From a series of analytical parametric studies, Dawe and Seah concluded that variations in the compressive strength of the masonry and confining frame restraint would significantly alter the ultimate capacity of the infill. The effects of modulus of elasticity, face-shell thickness, and relative airbag size were found to be less significant.

A.6 CONCLUSIONS

The initial literature review revealed that very limited information on HCT behavior was available. Numerous tests on infilled frames subjected to in-plane loads have been performed; however, the complexity of and differences in support conditions, frame-to-infill connectivity, type of material, orientation of the masonry, application of the loads, etc., make extrapolation of past test results unacceptable for application to the Y-12 structures.

In these earlier tests static or quasi-static out-of-plane tests were confined to pressure loading in which the airbag was the primary mode of application. This type of loading is an excellent means of replicating wind forces on infilled frames or, for that matter, inertial forces of the infill material itself. However, airbag pressures are not adequate for addressing the stability of Y-12 infills when the panel is subjected to trauma by excessive deflection (drift). Also, few infills with

⁵Portions of this section were extracted from Flanagan (1994) and Henderson (1994) with permission from the authors.

the size (i.e., h/t ratio) or material configuration of those at Y-12 had been tested. Likewise, little or no empirical information was available regarding the residual in-plane strength of infills following some degree of damage in an orthogonal direction. These data are particularly important in view of code requirements specifying directional combinations of seismic forces (UBC 1991; UCRL-15910 1990).

Finally, the methods of analysis used in conjunction with earlier tests for both in-plane and out-of-plane evaluation were not sufficiently refined or compared with empirical results. Hence, to enable accurate analysis of the structures and facilities at the Y-12 Plant, a test program to better understand HCT infills was necessary.

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Appendix B

INVESTIGATIVE STUDIES OF IN SITU CONDITIONS

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ABSTRACT

To prepare and complete a successful test program for understanding the behavior of hollow clay tile (HCT) infills at the Y-12 Plant, an investigation of the in situ infill conditions was necessary. This entailed investigating and documenting the strength and composition of the mortar, the as-constructed configuration of the HCT (particularly as it relates to the construction of 13-in. panels), and the boundary conditions and frame-to-infill connectivity. Once this work was completed, the information was used to formulate test series based directly on the as-built parameters of the Y-12 facilities.

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Appendix B

INVESTIGATIVE STUDIES OF IN SITU CONDITIONS

To prepare and complete a successful test program for understanding the behavior of hollow clay tile (HCT) infills at the Y-12 Plant, an investigation of the in situ infill conditions was necessary. That investigation is described in this appendix.

B.1 MORTAR CHARACTERIZATION

B.1.1 Introduction

Mortar and HCT unit blocks are the primary components that make up an infilled-wall panel; therefore, the mortar property is of fundamental importance. Very early in the development of the Hollow Clay Tile Wall (HCTW) Test Program plan, the impossibility of performing all testing on in situ components was recognized. This led to the conclusion that a mortar characterization study was necessary to determine a comparable mix design that would provide the same basic physical properties as in situ mortar. This design, prepared by Testwell Craig Materials Consultants, was then used to construct the many laboratory test specimens needed to completely determine the constitutive properties of the HCTWs located throughout the Oak Ridge site (Butala and Jones 1993). Testwell Craig also performed compressive-strength tests on in situ mortar removed from Building 9212, as reported in Appendix D.4 of this document.

B.1.2 Objective

The objective of the mortar characterization study was to develop a mortar-mix design that would provide the same basic physical properties as in situ mortar.

B.1.3 Scope and Methodology

The scope of the study was to perform petrographic and chemical analyses on hardened mortar samples taken from various wall locations throughout the Oak Ridge Y-12 Plant. Twelve mortar samples were initially collected for analysis from three walls in each of four buildings (9212, 9207, 9206, and 9204-2). Because the results of the chemical analyses of these samples showed a large variation in the mortar ingredients, especially in Building 9206, additional chemical analyses of six mortar samples removed from this facility were performed.

B.1.4 Results and Conclusions

All mortar samples except those taken from Building 9207 were made with natural siliceous sand, Portland cement, and hydrated lime. Hydrated lime was not detected in mortar samples taken from Building 9207. A mortar mix was prepared with the average composition of the initial 12 samples and manufactured sand. The average 28-day compressive strength of the laboratory-prepared mortar cubes was 2190 psi. The results indicated that the mortar exceeds the requirements of a Type N [American Society for Testing and Materials (ASTM) C-270] mortar in terms of strength.

Chemical analyses of the additional six samples taken from Building 9206 and the compressive strength test of the four in situ samples were completed by Testwell Craig. The additional six mortar samples were made with Portland cement, hydrated lime, and natural siliceous sand. The volumetric proportion was similar to that used in previous analyses.

The final mortar mix was designed with the average composition of the 12 analyzed mortar samples that were taken from Buildings 9212, 9206, 9207, and 9204-2. Tennessee River sand, lime, and Portland cement (Type-I) were used for the laboratory mix design. The volumetric proportions of the mix meet the requirements of ASTM C270 Type N mortar. The average 28-day compressive strength of mortar cubes was 2540 psi with a standard deviation for a single operator of 71.9 psi (2.8%). The standard deviation falls within the range required by ASTM standards. The mortar batch proportions are shown in Table B1.1.

Table B1.1. Mortar batch proportions

Material	Volumetric ratio	Proportions (g)
Portland Cement (Type - 1)	1	473.3
Lime	0.86	290.0
Sand (Tennessee River sand)	4.38	1737.0

B.2 HOLLOW CLAY TILE CONFIGURATIONS¹

B.2.1 Introduction

In the spring of 1989, a seismic analysis of Building 9212 D-Wing was initiated to support the design of the Enriched Uranium Conversion Facility. This analysis was subsequently peer reviewed by an independent team selected by the U. S. Department of Energy (DOE) from the maintenance and operating contractors of various DOE sites. Many of the concerns raised by the peer reviewers had to do with the assumed material properties of the HCT walls used in the analysis. Before the analysis could be completed, the peer reviewers required that some limited testing to determine the bed-joint shear capacity of the HCT wall be completed. This requirement of the peer reviewers eventually led to the HCT Wall (HCTW) Test Program. This early test program led to the removal of some unit tile for compression and splitting tensile strength tests. Also, three wall panels were removed and transported to Nashville, Tennessee, to be tested for diagonal tension (shear) strength as described in Appendix F of this document. This early work led to a better understanding of the HCT configuration throughout the plant. Figure B2.1 is a photograph of a wall in Building 9296, which was in the process of being modified. This figure shows how a 12-in.-nominal wall is constructed.

¹Portions of this section were extracted from Henderson (August 1994) with permission from the author.

B.2.2 Objective

The purpose of the HCT configuration investigation was to determine the orientation of the cores, how the 12-in.-nominal walls were constructed, and the dimensions of the head and bed joints.

B.2.3 Methodology

Individual blocks were removed at selected locations to determine how the blocks were oriented, the placement and thickness of head joints (how many face shells were mortared), and the thickness and extent of the bed joints.

Fig. B2.1. Photograph showing construction of 12-in.-nominal wall.

B.2.4 Results and Conclusions

All walls were constructed in a running bond pattern with the cores laid horizontally (side construction). As seen in Fig. B2.1, the 12-in.-nominal walls were constructed with 8-in. and 4-in. blocks placed side by side in a row but offset by one-quarter of the length of a block so that the head joints were not aligned. Additionally, 4- and 8-in. blocks switch position in alternate rows, thereby forming a discontinuous collar joint. This type of construction forms a cross-section that acts homogeneously to resist loads.

Early work by Fricke and Jones (1989) showed that the head joint had mortar applied only on the outside face shell; however, because the bed joints covered the full width of the blocks, mortar would protrude partially into the top and bottom face shells. The average bed and head-joint

thickness is approximately 0.5 in. The average block dimensions, as discussed in Appendix D of this document, are shown in Fig. B2.2.

Fig. B2.2. Average dimensions of 4-, 6-, and 8-in. tile.

B.3 INFILLED FRAME CONFIGURATIONS/BOUNDARY CONDITIONS²

B.3.1 Introduction

Before, and to a certain extent during, the test program, some concern existed regarding the in situ boundary conditions that defined the infilled walls at 9212 and elsewhere. This concern stemmed from attempts at forming analytical models, as well as test models, based on design drawings with insufficient detail. The primary area of ambiguity existed in the specification in the wall-to-column connections and the wall-to-purlin connections. Drawings indicated that 25-in.-wide pilasters were used for this purpose; however, little information as to actual "fit-up" was available. Hence, requisite behavioral data (e.g., keying of the wall to the columns and rotational/translational stiffness) for modeling was unavailable. Likewise, existing drawings showed the W16 purlin used in the wings of Building 9212, but little information was provided indicating the connection shear strength. To address these issues, several pilasters and beams in the 9212 complex were removed and observations concerning them were concurrently documented.

²Portions of this section were extracted from Henderson (August 1994) with permission from the author.

B.3.2 Objective

The purpose of this study was to clarify the infill-to-column and infill-to-beam connections in the wings of the 9212 complex so that the wings could be better modeled experimentally and analytically.

B.3.3 Scope and Methodology

Three external pilasters and two purlin-to-infill connections were disassembled. Because asbestos is present in the paint covering the pilasters, a glovebag was required for the removal of the outer face shells. A small jackhammer was used for the remaining disassembly of the pilasters. Because penetration into the 9212 wings was anticipated, an armed guard was required to be present at all times. Continuous videotaping as well as still photography were performed during the disassembly process.

Purlin-to-infill disassembly was performed inside the 9212 wings. No asbestos paint was used in this area; therefore, the operation was much less cumbersome. Still photographs were taken to thoroughly document existing conditions.

B.3.4 Results and Conclusions

The still photos and video of the disassembly process (see Fig. B3.1) indicate that the configuration of the pilaster and purlin connections are as shown in Figs. B3.2 and B3.3, respectively. The fundamental conclusion resulting from this study was that although 4- and 8-in. HCT is not continuous into the column flanges within the pilaster, the scrap, cut block, and mortar material provide extremely durable and strong connectivity. Likewise, although a quantitative evaluation of pilaster-connection strength would have been preferable, the qualitative results indicated well-constructed walls. Additionally, void space inside and outside the column flanges is effectively eliminated by partial blocks, tightly packed scrap pieces, and liberal application of mortar. It is important to note that in most cases the removal (by jackhammer) of the scrap and mortar around the flanges and web of the W10 column was much more difficult than the removal of actual block, which indicates that some considerable keying action exists (see Fig. B3.3.).

The purlin study indicated that the W16 purlin is flush with the face of the infill material (i.e., the inner flanges of the W16 purlin are embedded in the wall), thereby creating an eccentricity in the in-plane load path.

Fig. B3.1. Photograph showing column-to-infill connectivity.

Fig. B3.2. Pilaster configuration.

Fig. B3.3. Purlin configuration.

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Appendix C

IN SITU CONSTITUTIVE PROPERTY TESTS

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ABSTRACT

Many of the tests performed for the Hollow Clay Tile Wall Test Program were conducted on specimens constructed in a laboratory with new materials. To establish the relationship between laboratory and in situ behavior, some in situ testing was necessary. Therefore, compressive strength tests were performed on three prisms, bond strength tests were performed on 52 hollow clay tile units, low-level elastic properties were tested for with five deformability tests and two normal stress tests, and shear strength was measured with seven push tests.

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Appendix C

IN SITU CONSTITUTIVE PROPERTY TESTS

Many of the tests performed for the Hollow Clay Tile Wall Test Program were conducted on specimens constructed in a laboratory with new materials. To establish the relationship between laboratory and in situ behavior, some in situ testing was necessary. That testing is described in this chapter.

C.1 PRISM COMPRESSION TESTS¹

C.1.1 Introduction

In situ hollow clay tile (HCT) prism compression tests were performed to determine the compressive strength and modulus of elasticity of the hollow clay tile walls (HCTWs) as they exist in Y-12 Plant buildings (Jones and Butala 1993). Data describing load vs displacement behavior, including the maximum load and postpeak deformation characteristics, were obtained. The prism test is the standard test used to determine values for f'_m (specified compressive strength at 28 days), which are then used to obtain code-allowable values.

Tests were made on prism specimens oriented in two directions: normal and parallel to the cores. In the Y-12 Plant buildings, the walls are typically constructed so that the cores in the HCT units run horizontally (side construction). Loading normal to the cores simulates vertical loading (gravity and vertical earthquake motions) on the walls, and loading parallel to the cores simulates the earthquake forces applied to a building wall in the horizontal direction. Regardless of the orientation of the cores, the prisms were tested so that the loading was always parallel to the long direction of the prism.

Early testing to support the analysis of the Enriched Uranium Conversion Facility (Beavers et al. 1989) indicated that removal and testing of in situ prisms would require removal and handling fixtures and a testing apparatus that could be located inside a process building from which the in situ prisms were to be removed. To avoid damaging the specimens, procedures describing in considerable detail how the in situ specimens were to be removed, handled, and capped were required (Jones and Hornyak 1992). Fig. C1.1 is a photograph of cap preparation.

C.1.2 Objectives

The primary purpose of the in situ prism tests was to determine the compressive strength of HCTWs. However, secondary objectives were to determine the difficulty of extracting and performing the prism compression test, write a procedure for in situ compression testing of prisms, and establish a benchmark for estimating the costs of performing in situ prism compression tests. The results from the tests were also to provide comparative values for evaluating the results reported by outside laboratories on similar prisms.

¹Portions of this section were extracted from Fricke and Butala (1992) with permission from the authors.

C.1.3 Scope and Methodology

Both 8- and 12-in. prisms were to be tested with cores both normal and parallel to the load. Because of the difficulty of the extraction process, testing of laboratory prisms was to supplement the in situ testing (See Appendix D). The basic testing procedure would meet the requirements of American Society for Testing and Materials (ASTM) E-447. Two test frames were built. One was to be located inside a process building, and removed prisms were to be taken to that location and tested. This procedure would ensure that any contamination would remain inside the process area. The other frame was to be located in Building 9207 or some other noncontaminated area and to be used to develop and understand testing requirements. The testing to be performed is summarized in Table C1.1.

Three prisms were removed from walls located at the Y-12 Plant and tested in a specially designed and fabricated frame (see Fig. C1.2). Two normal prisms (one 8- and one 13-in.) and one 8-in. parallel prism were tested. Instrumentation on the normal prisms consisted of three vertical linear variable displacement transducers (LVDTs), two on one side and the third on the back side, plus three horizontal LVDTs. The parallel prism had two vertical and two horizontal LVDTs on each face. The test setup for a 13-in. normal prism is also shown in Fig. C1.2.

Fig. C1.1. Photograph showing prism capping.

Because these prisms were removed in situ from patched walls, obtaining specimens that completely matched the construction of the laboratory-built prisms was not possible. Anomalies were noted as they were encountered. (For example, on the two bottom courses, two tiles were

oriented in the wall with the cores running vertically, which results in a stiffer prism at those locations.) Also, in the laboratory-built prisms, obviously cracked tiles were not selected.

Table C1.1. Planned prism compression tests

	8-in. walls		12-in. walls		Total	
Test direction	Wall	Built	Wall	Built	Wall	Built
Normal to cores	3	4	3	6	6	10
Parallel to cores	3	4	3	6	6	10
Total	6	8	6	12	12	20

Fig. C1.2. Photograph showing prism test setup.

C.1.4 Results and Conclusions

No prisms were ever removed from the process buildings mainly because a suitable location for the testing fixture was lacking. Also, approval for taking potentially contaminated prisms to Building 9207 could not be obtained. Eventually, three experimental prisms were removed from Building 9207 and used to check and improve procedures. Test results are given in Tables C1.2 and C1.3.

Table C1.2. Description of in situ prism tests

Prism #	Prism ID	Extraction date	Test Date	Ultimate load (kips)
1	PP8S1	5-91	7-13-92	52.2
2	PN12S1	4-12-91	5-28-92	110.6
3	PN8S1	5-91	12-18-92	64.0

Table C1.3. Results of in situ prism tests

Test	P_u (kips)	f'_m gross (psi)	f'_m net (psi)	E33 gross (ksi)	E33 net (ksi)
13-in. Normal	64.0	323	1312	930	3777
8-in. Normal	52.2	252	647	1402	6762
8-in. Parallel	110.2	349	861	1126	2781

C.2 BOND WRENCH TESTS²

C.2.1 Introduction

Masonry flexural bond-strength (bond-wrench) tests are performed to determine the flexural tensile strength at the interface between two masonry courses or units. Because flexural cracking of masonry generally occurs at the block-mortar juncture, bond wrench data is helpful in predicting the bending capacity of unreinforced masonry components (e.g., walls and pilasters). The test apparatus consists of a clamping bracket with a loading arm that is clamped to the top of the masonry unit selected for testing (Hardin 1992a). Load is applied at a uniform rate to the loading arm until the bond fails. The applied load is measured with a compression load cell. A digital multimeter, with the capacity to save and display the maximum applied load, is used to continually monitor the applied load during testing. The maximum applied load is used to calculate the flexural bond strength.

²Portions of this section were extracted from Hardin (1992a, b, c) with permission from the author.

In situ bond-wrench testing was performed in Y-12 Buildings 9207 and 9204-2 (Hardin 1992b, c). The wall panel selected for testing in Building 9207 was located on the ground floor along column line D between E-W column lines 2 and 3. This wall had been previously tested by airbag pressure loads (see Appendix E). The wall panel selected for testing in Building 9204-2 was located along N-S column line 9 between E-W column lines M and K. This panel was being removed to allow for the installation of equipment. For both wall panels, the elements were HCT units $12 \times 12 \times 8$ in. (nominal) constructed with the cores running horizontally (side construction) and with running bond.

C.2.2 Objective

The purposes of conducting bond-wrench tests in Building 9207 on a wall that had been previously tested with an airbag were to (1) determine if the method outlined in the test procedure was a practical method for in situ testing; (2) compare the method with masonry prism procedures of ASTM C1072-86; and, to a lesser extent, (3) determine masonry bond strength for the units in that panel. The primary objective of the testing in Building 9204-2 was strictly to determine the flexural bond strength of the HCT bed joints.

C.2.3 Scope and Methodology

Building 9207. At the conclusion of the airbag test, all but five courses of the HCTW units were removed from the wall panel. The head joints were removed from the top course with a 12-in.-diam circular masonry saw. Bond-wrench testing was started on December 6, 1991. During testing of the first wall unit, transfer of the applied load along the bed joint to adjacent units was observed. Inspection of the head joints revealed that the head joints were not completely removed and that the bed joints needed to be completely sawed through to the bottom of the joint. A linear (reciprocating) saw was used to saw completely through the bed joint. Testing resumed on December 17, 1991, and 27 HCTW units were tested.

Building 9204-2. Bond-wrench testing was started on April 13, 1992, and completed on the same day. Twenty-four HCTW units were tested.

The flexural tensile strength of the bond is calculated by use of a simple equation that takes into account the stresses induced by the bond wrench and the applied load, the section modulus of the specimen, and the compressive stress caused by the mass of the test device and specimen resisting the bending forces. The flexural bond strength may be calculated as follows:

$$F_s = \frac{6(PL - P_1 L_1)}{bd^2} + \frac{P - P_1}{bd} \quad (C2.1)$$

where

- F_s = flexural tensile strength (pounds per square inch),
- P = maximum load applied (pounds),
- P_1 = weight of bond wrench and wall unit (pounds),
- L = distance from loading point to center of wall unit (inches),

L_1 = distance from centroid of bond wrench to center of wall unit (inches),
 b = average length of wall unit (inches),
 d = average width of wall unit (inches).

The values used for the bond wrench testing of the two buildings are as follows:

P_1 = 75.90 lb, d = 8.12 in.,
 L = 24.00 in., b = 12.00 in.,
 L_1 = 1.937 in., d = 8.12 in.
 b = 12.00 in.,
 d = 8.12 in.

C.2.4 Results and Conclusions

Building 9207. The 9207 test yielded valuable information as to the proper method of removing head joints and completely sawing through the bed joint to isolate each wall unit prior to testing. Also, a method to eliminate slippage of the clamping bracket was identified. Two methods of applying load, hydraulic and mechanical, were tested, and both proved to be satisfactory. Individual flexural bond strengths determined by this test series are shown in Table C2.1. The average bond strength was 11.244 psi. These values should not be interpreted as the flexural bond strength of a wall constructed of the same material because of the possible damage caused by airbag pressures.

Building 9204-2. Individual flexural bond strengths determined by this test series are shown in Table C2.2. These strengths average 10.745 psi.

Table C2.1. Flexural tensile strength, Building 9207

Tile I.D.	Fs (psi)	Tile I.D.	Fs (psi)	Tile I.D.	Fs (psi)	Tile I.D.	Fs (psi)
212	5.950	172	2.337	199	7.142	240	6.950
213	18.101	171	26.945	198	20.024	241	11.565
214	14.256	169	7.720	197	12.910	242	6.566
215	7.720	168	7.142	196	32.906	243	4.259
216	13.487	167	8.489	195	9.834	244	6.373
217	13.295	201	6.181	238	2.144	245	10.795
218	3.106	200	31.560	239	4.835		

Table C2.2. Flexural tensile strength, Building 9204-2

Tile I.D.	Fs (psi)	Tile I.D.	Fs (psi)	Tile I.D.	Fs (psi)
1	7.488	9	0	18	10.143
2	6.437	11	7.811	19	11.849
3	38.528	12	4.189	20	9.502
4	6.892	13	6.136	21	11.788
5	40.810	14	4.525	22	3.009
6	20.410	15	5.508	23	5.852
7	5.414	16	35.367	24	3.351
8	3.878	17	5.255	25	3.726

C.3 FLATJACK TESTS³

C.3.1 Introduction

Flatjack tests were performed in Building 9207 in April-May 1991 to provide the HCTW team with experience in their installation and use (Fricke 1993). Two types of tests were performed:

- **Normal-Stress Test** - determines the existing compressive stress in the wall at the location of the flatjack; and
- **Deformability Test** - provides deformability and strength characteristics for the in situ wall.

Both of these tests are commonly being used in the nondestructive evaluation of existing masonry structures. The flatjack was initially developed in the field of rock mechanics for determining stresses and material deformability in the rock structures of mines and tunnels (Panek and Stock 1964). A good summary of the flatjack method is provided by both Rossi (1990) and Noland et al. (1990).

A flatjack is an envelope-like bladder with inlet and outlet ports that may be pressurized with hydraulic oil (or air at low pressures). The shape and thickness of the flatjack is determined by the function and nature of the masonry walls being tested. Flatjacks have been fabricated in rectangular, semicircular, and semirectangular shapes (Noland, Alkinson, and Shuller 1990). The flatjacks used in the HCTW Test Program are rectangular in shape and were designed to fit into slots cut in the walls being tested. Two sizes of flatjacks were designed and procured for use in the program, one to be used in the double-wythe 13-in. wall, the other in the single-wythe 8-in. walls.

³Portions of this section were extracted from Fricke (1993) with permission from the author.

Typically, when testing a multiwythe wall, only one wythe at a time is tested, but because of the interweaving construction of the HCT units in the walls, testing of the double-wythe 13-in. wall required that a full-width flatjack be designed. General details of the flatjacks are shown in Fig. C3.1.

C.3.2 Objectives

The objective of the normal-stress test is to determine the magnitude of the existing compressive stress (normal stress) in the wall at a point in the wall (i.e., at the location of the slot). This value will determine whether the wall is supporting only its own weight or effectively acting as a bearing wall. Additional normal stress beyond that caused by the weight of the wall should increase the shear capacity of the clay tile wall.

The specific objective of deformability testing was to measure the compressive deformation properties of the existing in situ unreinforced HCT walls. Deformation (and strength) properties are measured on the masonry prism between the flatjacks. The general objective of this test, for the HCTW Test Program, was to obtain relatively low-level elastic properties (primarily modulus of elasticity, E_m) without damaging the masonry panel and then to correlate the value obtained with the results of a prism-compression test made from the same test panel.

Fig. C3.1. Flatjack configuration.

Finally, the feasibility of performing both the normal and deformability tests and the performance of the flatjacks required investigation.

C.3.3 Scope and Methodology

Five in situ normal stress tests were performed in Building 9207 at the Y-12 Plant. Three of these were performed on 13-in. HCT walls and two on 8-in. HCT walls. This test is primarily

a problem of stress release, which involves measuring the deformations around a slot cut in the wall. The test setup is shown in Fig. C3.2, and a schematic of the test procedure is given in Fig. C3.3. When a slot is cut and formed in the masonry wall, the compressive stresses, and hence strains, that existed in the wall are relieved, and the spacing in the wall at the slot decreases. The original compressive stresses may be approximately determined by using flatjacks.

Prior to cutting the slot for the flatjack, two horizontal reference lines with four gage points each were established, one above the proposed slot location, and the other below. The spacing between the gages was 6 in. The ASTM standards specify a minimum of 0.3 times the length of the flatjack, which is 24 in. or 7.2 in.; however, the 12-in. length of the unit tiles makes the smaller spacing more optimal for these tests, as shown in Fig. C3.2. The initial distance, d_1 , between vertical, spaced gage points was recorded (Fig. C3.3), and the dial gages removed. After the slot was cut and cleaned, the flatjack was inserted into the slot and snugged in place (with shims as necessary). The new distance between gage points was measured, $d < d_1$ (Fig. C3.3). Because the distance between the gage points measured immediately after the slot was cut (before insertion of the flatjack) and after the flatjack was installed differed slightly, the postinstallation reading was used to establish closure of the wall because of the cut in the wall. The value of $d - d_1$ was obtained for each dial gage, and that reading was set on the dial.

Fig. C3.2. Details of normal stress test setup.

The internal pressure in the flatjack was gradually increased until the original distance between the preestablished gage points was restored (when $d = d_1$, or when the dial gage read zero). This distance indicated that the original compressive strains in the masonry were restored. The value of normal stress obtained in this manner is a very good approximation of the original normal stress that existed in the wall at the slot prior to cutting the slot; the original stress state is never exactly recovered because of the effects of Poisson's ratio.

Deformability tests were performed at two locations previously tested for normal stress. The specimens were approximately 2 ft apart on the same wall. To perform these tests, two

flatjacks (Fig. C3.4) were inserted into parallel slots separated by a number of masonry courses. The flatjacks were then pressurized, thereby producing compressive stress in the masonry between them. By gradually increasing the pressure and measuring the deformation of the masonry between the flatjacks (with four dial gages, as shown), load-deformation (stress-strain) properties and the maximum compressive strength were obtained. The standard procedure is provided in ASTM C-1197.

Fig. C3.3. Schematic diagram of normal stress test.

There is one major difference between a prism tested in situ and a prism tested under laboratory conditions. A prism tested in the lab is restrained only at the ends (i.e., the vertical sides are free to expand), whereas a prism tested in situ in the wall is constrained along the sides (biaxial loading), which should theoretically make it stronger. This difference was considered when correlating the data.

The stress in the masonry wall caused by pressurization of a flatjack is approximately equal to the internal flatjack pressure multiplied by two factors that account for the physical characteristics of the jack: the bearing area of the jack in contact with the masonry and the bearing area of the slot. In equation form, this situation becomes

$$\sigma = p \cdot K_m \cdot K_a \quad (C2.1)$$

where

- σ = corrected pressure in masonry wall (pounds per square inch),
- K_m = flatjack calibration constant,
- K_s = ratio of the flatjack surface area to the surface area of the slot,
- p = flatjack hydraulic pressure (pounds per square inch).

This general equation is used to determine stresses in the walls in both the normal stress test and the deformability test.

Fig. C3.4. Photograph showing deformability testing.

C.3.4 Results and Conclusions

Normal Tests. Results for the five normal stress tests are shown in Table C3.1. Normal stress readings (psi) were recorded at four dial-gage locations as shown in the table. A mean of these values was then calculated and is listed as the average in situ normal stress.

As mentioned earlier, the general objectives of the deformability test as it relates to the HCTW Test Program were to obtain "relatively" low-level elastic properties (primarily modulus of elasticity, E_m) without damaging the masonry panel, to extract the prism from the wall, to perform a prism-compression test, and to correlate the values obtained for E_m . Unfortunately, damage to the prisms made removing them intact from the wall impossible, and subsequent prism compression testing was therefore not performed.

Table C3.1. Normal stress test summary

Test	Test date(s)	Dial gage				Average in situ normal stress (psi)
		#1	#2	#3	#4	
N1-13-9207	5-16-91	44	30	31	30	34
N2-8-9207	4-30-91	121	89	**	58	89
N3-8-9207	5-3 & 5-6-91	100	103	134	141	112
N4-8-9207	4-29-91	44	30	31	30	34
N5-13-9207	5-2-91	8	7	8	5	7

** = Dial gage lost during testing

The measured insitu normal stresses varied widely throughout the building. The measurements ranged from a low of 7 psi to a high of 112 psi. Test N3-8 was performed on a wall that, prior to the test, had large openings cut into it in three locations. These openings probably redistributed the existing stresses in the wall, and the location of the test (in one of the remaining piers) only compounded the measurement. Test N2-8 was fairly close to a building column, and in situ normal stresses may have been higher at this location. Also, the very low value (7 psi) for test N5-13 was certainly caused by the one-story height that was used for this test. The pretest measurements, cutting the slot, the post-slot measurements, and the mounting and installation of the dial gages probably takes between 1 and 2 h (when everything runs smoothly and the crafts personnel are on hand to cut the slot). The test itself takes barely 5 min to run once everything is installed and ready. In general, because of the low pressures, the flatjacks were reusable immediately after each test. No flatjack lost integrity during any of these tests.

Deformability Tests The deformability test results are summarized in Table C3.2. A single cycle of testing was run for D1-13-9207, and two cycles of testing were run for test D2-13-9207. Both testing cycles were terminated because it became apparent that the prisms were being damaged internally (loud cracking was clearly audible) and also because one or two of the blocks immediately above the prisms were being damaged (both internal noises and some slight shell-face

spalling were occurring). Even though both prisms were taken to a 55 psi masonry stress, the strain in the D2-13 test was more than double the strain in the D1-13 test, probably because the two tests were set up very close to each other. After test D1-13 was run, that part of the wall was partially damaged and was not fully participating as a load path for the wall. Upon the application of pressure to the second in situ prism, the wall was less stiff, and the smaller modulus was measured as a result. The 55 psi that was reached was not the ultimate stress level for either prism.

Table C3.2. Deformability test summary

Test	Test date	Maximum stress achieved (psi)	Maximum strain achieved (in./in.)	Secant modulus (psi)	
				$0 - P_{max}/2$	$P_{max}/2 - P_{max}$
D1-13-9207	5-09-91	55	.00015	573,000	279,000
D2-13-9207	5-10-91	55	.00033	183,000	153,000

The deformability test was intended to demonstrate the feasibility of performing a *low-level* pressure test *without* damaging the panel, then removing the tested panel and moving it, undamaged, to a test machine to perform a prism compression test. This process was shown to be very difficult. However, it is considered uncomplicated to perform a standard deformability test intended only to determine the in situ biaxial properties of the wall. Because of the poor condition of the wall where these tests were performed, meeting the original objective may still be possible if the test is performed on a better-constructed specimen. The question, however, of having unintentionally damaged the prism (in a manner that is not visually obvious) will always remain, and this fact will result in questionable comparisons between the in situ test and the prism test.

C.4 PUSH (SHEAR) TESTS⁴

C.4.1 Introduction

Seven tests to determine the shear strength of mortar-bed joints (this test is also known as the shove test or push test) were conducted on an HCTW located in Building 9204-2 (Fricke 1992a). The wall was scheduled to be demolished by the middle of April 1992, and its availability and large, accessible areas permitted the performance of both the push tests described in this report and some bond-wrench tests. The shear tests were performed between March 31 and April 3, 1992. The tests were performed per Procedure HCTP-1 (Fricke 1992b).

The push test is a partially destructive test designed to measure the shear strength of a mortar-bed joint by applying a horizontal load to a single HCT unit until a shear failure of the two mortar beds occurs, thereby allowing the block to slide. The load at which the shear or bond failure first occurs is measured. Instrumentation of the test block yields the load-deflection curve, which provides a means of studying the behavior of a single block under load.

⁴Portions of this section were extracted from Fricke (1992a) with permission from the author.

C.4.2 Objective

The primary objective of the push test is to determine the strength of the mortar-bed joint in direct shear (i.e., to determine the force required to fail the bond between the mortar bed and either the test unit or the surrounding wall). A second objective is to determine the behavior of a single unit after bond failure. Eventually, it was hoped, a relationship could be determined between the behavior of a single block under shear loads and the behavior of the overall HCT-infilled wall.

C.4.3 Scope and Methodology

The wall in Building 9204-2 where the push tests were performed runs north and south and is located between N-S column lines 9 and 12. The wall is approximately 97 ft long and 19 ft high and is constructed in a running-bond arrangement with a single wythe of 8-in.-thick tiles. Preparation of the 8-in. wall requires removing tiles adjacent to the test block and on the same course as the test block, removing the bed joints in the open areas, removing the exposed head joints, and installing steel plates for applying the load uniformly to the test block. The block-removal plan for this size wall is shown in Fig. C4.1. The instrumentation required for this testing included a hydraulic ram with a pressure transducer, two linear variable differential transformers (LVDTs), one XYY' plotter (an XY plotter with two pens), and a load cell (providing a second load measurement). The load-deflection curves were recorded on the XY plotter.

Each push test took 2 to 3 min to perform. The persons involved in each test generally included two observers stationed at the wall (one per side), one person running the hydraulics, one person monitoring the XYY' plotter to make certain that the response of the plotter is acceptable, one person monitoring the load cell and calling out the load, and someone taking photographs or running a video camera. The load is applied with an electronic pump connected to the hydraulic ram located in the wall. Load is applied slowly and fairly uniformly until the mortar-to-tile bond has failed and the block has moved approximately $\frac{1}{4}$ in. ($\pm$, just visually estimated) on each side of the wall, at which point the load is reduced to zero.

A cyclic-loading test was performed in Tests 5 and 7. The purpose of this cyclic testing was to study the effects of load reversal on the mortar-to-block interface behavior. After the first part of the test was completed, the LVDTs were removed along with the hydraulic ram and the load cell. The ram and the load cell were then reinstalled in the opening on the other side of the test block, and the LVDTs were attached, this time facing in the opposite direction from that of the original test. Load was then applied until the block had moved back to its original location and moved about another $\frac{1}{4}$ in. beyond that point. The load was then removed. In both tests, the ram and other equipment were again moved to the original side and this procedure was performed again for a total of three loading cycles.

C.4.4 Results and Conclusions

Seven push tests were run in descending order relative to the location number. The tests are summarized in Table C4.1. In the table, the load at bond failure is taken as the load that occurs at a deflection corresponding to the intercept of the initial and the final slopes of the curve. The exceptions to this criterion are the cases (labeled with a p) where the curve shows a definite initial peak; for these cases, the peak value is used. Test #1 was terminated at a load of 40 kips because the blocks behind the load cell, which were being used as reaction points, were being crushed. The

load-deflection curve for push tests often has a curved transition from straight-line elastic to zero-slope plastic. As a result, ascertaining the failure load of the unit as a function of displacement is often difficult. The methods suggested in the literature are as follows:

1. First Movement (approximately equal to 1/32 in.).
2. Intercept of Initial Slope and Post-Failure Slope. In this method, the intercept point for the lines associated with elastic and plastic behavior is determined. If the postyield curve has a zero slope, then the intercept load will correspond to the load after the bond failure.
3. Load at 1/16-in. Movement. In this method, the load at a deflection of 1/16 in. is determined.

For the values in Table C4.1, Method 2 was used.

Fig. C4.1. Photograph showing push test.

Clearly, a block in a wall exhibits a substantial amount of resistance to movement even after the initial mortar-bond failure. In the majority of the tests, the load-deflection curve continues to demonstrate an increase in load with increasing deflection. In the few cases where a sudden drop in load has occurred, the average drop has been a small percentage of the peak load (perhaps 10% to 20%), and frequently the load has then increased again, sometimes beyond the previous peak level. The reason for this stiffening of the joint has to do with the breaking up and grinding down of the minute mortar particles and the friction created thereby. In two cases, a load reversal was performed to better understand the behavior of the tiles undergoing loading in both directions. The general result was that, on each succeeding load-reversal cycle, the peak load decreased from that of the previous cycle. This same behavior has been observed in previous tests where a load

reversal was run. As cycles of load reversal are applied, the shear resistance is rapidly reduced, and after only a few cycles (perhaps 10 or less) will approach zero.

Table C4.1. Summary of 9204-2 push tests

Test no.	Date	Type of loading	Failure load (kips)	Shear stress (psi)
7	3-31-92	Reverse (3 cycles)	20.0	110
6	3-31-92	Unidirectional	8.7	48
5	4-2-92	Reverse (3 cycles)	18.5	102
4	4-2-92	Unidirectional	15.2	84
3	4-3-92	Unidirectional	29.6 p	163
2	4-3-92	Unidirectional	27.6 p	152
1	4-3-92	Unidirectional	40.0+	221+

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Appendix D

LABORATORY CONSTITUTIVE PROPERTY TESTS

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1892-1893

1893-1894

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ABSTRACT

In situ constitutive property tests are very useful in providing properties for materials and components as they exist in the field. However, because of the expense of extracting, monitoring, and testing in situ samples of hollow clay tile (HCT) at the Y-12 Plant, a supplementary laboratory test process was required. Moreover, the laboratory testing was advantageous in that specific investigations could be performed in a more controlled and focused environment. Therefore, laboratory compressive tests were performed on HCT units and prisms, bond strength tests were performed on 8-in. specimens, and properties were defined for numerous mortar specimens.

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Appendix D

LABORATORY CONSTITUTIVE PROPERTY TESTS

In situ constitutive property tests are very useful in providing properties for materials and components as they exist in the field. However, because of the expense of extracting, monitoring, and testing in situ samples of hollow clay tile (HCT) at the Y-12 Plant, a supplementary laboratory test process was required. Moreover, the laboratory testing was advantageous in that specific investigations could be performed in a more controlled and focused environment.

D.1 UNIT BLOCK TEST¹

D.1.1 Introduction

Many structures within the Y-12 Plant were constructed in the 1940s and early 1950s with walls made of unreinforced HCT. Before the beginning of the Hollow Clay Tile Wall (HCTW) Test Program described in this document, little was known about the properties and strength of HCT. To help supplement the knowledge base and compare the properties of old and new tiles, unit-block testing was initiated at Singleton Laboratories in Knoxville, Tennessee.

D.1.2 Objectives

The purposes of these 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.

D.1.3 Scope and Methodology

Of the 336 tiles tested, 172 were 4-in. tiles and 172 were 8-in. tiles (see Fig. D1.1). The tiles were tested for (1) weight, size, void area, and net and gross areas; (2) initial rate of absorption; (3) absorption; (4) modulus of rupture; (5) splitting tensile strength; and (6) compressive strength (40 tiles were instrumented to measure stress and strain relationships). Half of the tiles were "new tiles" recently purchased from D'Hanis corporation of Texas, and the remaining half were "old tiles," removed from the existing buildings. Prior to any tests, all the old tiles were cleaned with a wire brush to remove loose mortar and paint. The tiles selected for testing were representative of the range of color, texture, and size. The tiles were marked for identification on the basis of source (e.g., the building from which the tile was removed), test procedure planned, size of tile, and orientation of the cores.

The physical measurements (length, width, height, and weight) of 144 new and 144 old tiles were recorded. The dry weight of each HCT unit was measured with a calibrated scale. Sizes of all HCTs were measured in accordance with American Society of Testing and Materials (ASTM) C 67-90a, Sect. 11. For all HCT units, void areas were measured in accordance with ASTM C 67-90a. Also, nine density and three specific-gravity tests were performed on Ottawa sand.

Forty-eight HCT units were tested for initial rate of absorption (IRA) in accordance with ASTM C 67-90, Sect.9 (see Fig. D1.2). Absorption tests were performed on the same 48 HCT units in accordance with ASTM C 67-90a, Sect. 7 (see Fig. D1.3).

¹Portions of this section were extracted from Butala (1993) with permission from the author.

Fig. D1.1. Photograph showing hollow clay tile blocks.

Modulus-of-rupture tests were performed on 48 HCT units in accordance with ASTM C 67-90a, Sect. 5, with the exception that the supporting rods used were free to rotate only in the transverse direction. Splitting tensile strength tests were performed on 40 HCT units in accordance with ASTM C 1006-84. Two variations of load applications were conducted: one-half of the 40 HCT units (10 8-in. and 10 4-in.) with loading parallel to core and the other half (10 8-in. and 10 4-in.) with loading normal (perpendicular) to core.

Compressive strength tests were performed on 200 HCT units in accordance with ASTM C 67-90a. Two variations of loading direction were conducted: one with the loading parallel to the core voids and the other with the loading normal to the core voids. Among the 200 HCT units, 40 HCT units were tested with strain measurement on both sides, and 160 HCT units were tested

without strain measurements. The surface coating of 20 old HCT units was removed, and the surfaces where strain gages would be installed were cleaned thoroughly. A calibrated SATEC universal testing machine with a maximum load capacity of 600 kips was used for the compressive-strength tests. HCT size and loading direction for the 200 compressive strength tests are summarized in Table D1.1.

Table D1.1. Hollow clay tile size and loading direction for compressive strength tests

No. of tests	HCT Size	Loading direction
25	8-in.	Parallel
25	8-in.	Normal
25	8-in.	Parallel
25	8-in.	Normal
25	4-in.	Parallel
25	4-in.	Normal
25	4-in.	Parallel
25	4-in.	Normal

Fig. D1.2. Photograph of initial rate of absorption test.

Fig. D1.3. Photograph showing absorption test.

Prior to finalizing the HCT Test Program, Engineering Design and Geosciences Group, Inc. (1989) tested seven tiles and the University of Tennessee tested 40 tiles (Flanagan and Bennett 1992). The results of these tests fall within the range of the results reported by Butala (1993).

D.1.4 Results and Conclusions

Measurement of Hollow Clay Tile Units. The outside dimensions of old tiles are larger than those of new tiles, but the webs and face shells of old tiles are thinner than those of new tiles. The weight of an 8-in. old tile is greater than that of an 8-in. new tile, whereas the reverse is true for the 4-in. tile. The overall coefficient of variation is less for new tiles than for old tiles. To compute the mean

and coefficient of variation, the net area of each tile was calculated with its average dimensions. Another method would have been to multiply the gross area by the percent void (coring), which would yield slightly different values. The old tiles, which were possibly manufactured at different plants in the United States, exhibit color variations. About 15% of the old tiles were ribbed on three sides and about 15% were ribbed on two sides.

Initial Rate of Absorption. The IRA does not provide any strength information for the unit. Rather, it has been correlated to the bond strength between the mortar and the masonry unit. The values observed for the new tile are in the lower to middle portion of the optimum range of 5 to 20 g per min per 30 in.² recommended by Plummer (1962). The old tile exhibited IRAs from 6 to 41 g per min per 30 in.², depending on the size and orientation of the specimen. Most of the specimens required extensive cleaning to remove the paint and mortar. The effects of these conditions on the capillary action of clay are not known. However, it is known that voids sealed with paint and mortar will reduce suction and that abrasive cleaning to remove coatings may open voids and roughen smooth finishes, thereby increasing suction. See Fig. D1.2.

Absorption. Absorption is higher for the boiling condition than for the cold condition for the same test duration. The difference between the boiling and cold conditions is more profound for old HCT units than for new HCT units. Saturation coefficients varied from 0.3 to 1.0 for all HCT units. However, the coefficients of new units are essentially the same — approximately 0.9.

Modulus of Rupture (Flexure). The results from the modulus-of-rupture testing are shown in Table D1.2. As indicated, the 4-in. tiles appear to be stronger in flexure than the 8-in. tiles.

Splitting Tensile Strength. The results from the splitting tensile strength testing are shown in Table D1.2.

Compressive Strength and Strain Measurement. The results from the compressive strength testing are shown in Table D1.2. Generally, the compressive strength is higher for the parallel loading than for the normal loading. The strength of one HCT unit exceeded the capacity of the testing machine. Almost all new tile specimens produced a cracking sound throughout the testing, which indicates that the new HCT units are much more brittle than the old units.

The results of the compressive strength test for new tiles were substantially higher than those for old tiles. This result may be caused by the improvements in the grinding and firing process. The strength of new 4-in. tiles is substantially higher than that of old 4-in. tiles because of a smaller void area. The variation in Poisson's ratio was small, from 0.14 to 0.17 for old tiles and from 0.16 to 0.21 for new tiles. The gross elastic modulus varied from 1.65 to 1.95 for old 4-in. tiles and from 1.33 to 2.05 for new tiles. For 8-in. tiles, the results for old tile were in general agreement with those for new tiles.

In conclusion, the test results indicate that the old tiles are larger in outside dimensions than new tiles, but the net area of the old tiles is less than that of the new tiles. Also, new 4-in. tiles are heavier than old 4-in. tiles, but the new 8-in. tiles are lighter than old tiles. IRA results varied widely, probably because only six specimens were tested from each category. The saturation coefficient showed less variation: the average was 0.8. For loading parallel to the cores, new tiles appear stronger than old tiles. For loading perpendicular to the cores, the new 4-in. tiles appear to be much stronger than old 4-in. tiles, possibly because the new tiles have thicker webs and face shells.

Table D1.2. Summary of test results

Parameter	Old tiles				New tiles			
	4-in.		8-in.		4-in.		8-in.	
	Mean	V(%)	Mean	V(%)	Mean	V(%)	Mean	V(%)
Ultimate compressive load (kips) [horizontal ¹ , normal ²]	80.2 (25)	24.43	144.0 (25)	33.13	159.8 (25)	6.99	211.2 (25)	16.02
Ultimate compressive load (kips) [vertical ³ , parallel ⁴]	206.5 (25)	28.18	369.4 (25)	25.93	345.6 (25)	11.05	411.4 (25)	29.60
Gross compressive strength (psi) [horizontal, normal]	1771 (25)	28.49	1550 (25)	35.93	3736 (25)	7.06	2317 (25)	15.99
Gross compressive strength (psi) [vertical, parallel]	4815 (25)	32.98 (25)	3985 (25)	29.87	8048 (25)	11.19	4507 (25)	29.5
Net compressive strength (psi) [horizontal, normal]	5214 (25)	24.75 (25)	6025 (25)	33.46	9139 (25)	7.14	8572 (25)	16.12
Net compressive strength (psi) [vertical, parallel]	9116 (25)	29.52	9488 (25)	28.10	14590 (25)	11.41	10285 (25)	32.41
Gross elastic modulus (ksi) [horizontal, normal]	1.61 (5)	29.13	0.97 (5)	21.69	1.33 (5)	7.37	1.09 (5)	8.50
Gross elastic modulus (ksi) [vertical, parallel]	1.95 (5)	22.43	1.46 (5)	40.10	2.05 (5)	2.36	1.49 (5)	3.37
Net elastic modulus (ksi) [horizontal, normal]	4.07 (5)	21.39	3.74 (5)	19.11	3.24 (5)	8.0	4.06 (5)	8.10
Net elastic modulus (ksi) [vertical, parallel]	3.85 (5)	19.44	3.55 (5)	31.76	3.72 (5)	2.57	3.39 (5)	3.14
Split. tens. load (kips) [horiz]	11.9 (5)	25.7	22.3 (5)	40.4	16.7 (5)	59.4	18.7 (5)	30.4
Comp. Str. (psi) [normal]	518	28.08	592	39.2	612	59.0	482	29.8
Split. tens. load (kips) [vert]	17.5 (5)	40.5	20.5 (5)	20.9	18.9 (5)	64.1	33.3 (5)	15.0
Comp. Str. (psi) [parallel]	749	45.25	560	23.17	698	64.3	869	14.8
Modulus of rupture (kips) ultimate load	6.173 (12)	18.8	10.2 (12)	14.65	7.129 (12)	17.71	7.904 (12)	21.84
Modulus of rupture (psi) ultimate stress	177 (12)	18	136 (12)	17	203 (12)	22	108 (12)	22

¹Horizontal: Cores horizontal

²Normal: Load applied normal to cores

³Vertical: Cores vertical

⁴Parallel: Load applied parallel to cores

For modulus-of-rupture and splitting tensile strength tests, the old 8-in. tiles appear to be stronger. The compressive strength of new tiles is considerably greater than that of old tiles. One reason for this difference may be that the tiles might have been damaged during removal. A second reason for the greater compressive strength of the new tiles is that the manufacturing process may have improved over the years. The gross and net modulus of elasticity and Poisson's ratio for old and new tiles are comparable.

D.2 PRISM COMPRESSION²

D.2.1 Introduction

The prism compression testing was implemented to determine the compressive strength of an HCT prism and to provide an indication of the compressive strength of the infilled frames located in the critical facilities at Y-12. The compressive strength of the wall could then be used as a standard index for other properties. The laboratory prism compression tests were performed at the National Institute for Standards and Technology (NIST) (Yancey 1993a, b) and at the University of Tennessee (UT) (Boyd 1993). The NIST testing was divided into two tasks: 6 HCT prisms were tested in monotonic, uniaxial compression to failure in Task 1, and 41 were tested in Task 2. At UT, 31 HCT prisms were tested in uniaxial compression to failure. Laboratory prism testing was performed in accordance with ASTM E447 and was intended to provide a benchmark for the in situ prism testing described in Appendix C.

D.2.2 Objective

National Institute for Standards and Technology Task 1. The objectives of the NIST Task 1 series were to (1) choose between two methods of capping the prisms, (2) demonstrate and gain approval for the NIST prism-handling assembly and technique, (3) establish the loading rate(s) for Task 2 testing, (4) establish the data sampling rate for Task 2 testing, (5) determine the minimum number of LVDTs needed to define the specified mechanical properties of the prisms, and (6) confirm an acceptable means of transferring data files from NIST to MMES.

National Institute for Standards and Technology Task 2. The objectives of the NIST Task 2 test series were to (1) test all prisms to failure and obtain their compressive strength, (2) obtain the secant modulus of elasticity, (3) obtain Poisson's ratio for each prism, (4) study and understand the behavior of this type of masonry prism, (5) determine whether workmanship during construction of the prisms significantly affects the strength, and (6) compare the results of using load control versus displacement control while loading the prism.

The University of Tennessee. The objective of the UT testing was to document the uniaxial compressive capacity and load-deflection behavior of 8- and 13-in. prisms with various bed-joint angles.

²Portions of this section were extracted from Yancey (1993a and b) with permission from the author.

D.2.3 Scope and Methodology

National Institute for Standards and Technology Task 1. Six HCT prisms were constructed at NIST, cured for a minimum of 28 days, and tested in monotonic, uniaxial compression to failure. Four prisms were nominally 13 in. thick and two were nominally 8 in. thick. Three prisms were tested with the compressive load directed normal (i.e., normal prisms) to the axis of the hollow cores, and the other three were subjected to compressive load directed parallel (i.e., parallel prisms) to the axis of the cores. Eight linear variable displacement transducers (LVDT) were attached to all normal prisms and 8-in. parallel prisms, and ten LVDTs were attached to the 13-in. parallel prisms. Also, one LVDT was attached to the upper crosshead of the universal testing machine to record crosshead displacement. The displacements and load were recorded continuously on an electronic data-acquisition system until the prisms failed.

National Institute for Standards and Technology Task 2. Forty-one HCT prisms were constructed at NIST, cured for a minimum of 28 days, and tested in monotonic, uniaxial compression to failure. Twenty prisms were nominally 13 in. thick and twenty-one were nominally 8 in. thick. Twenty-one prisms were tested with the compressive load directed normal (i.e., normal prisms) to the axis of the hollow cores, and the other twenty were subjected to compressive load directed parallel (i.e., parallel prisms) to the axis of the cores. Eight LVDT were attached to all normal prisms and 8-in. parallel prisms, and ten LVDTs were attached to the 13-in. parallel prisms. Also, one LVDTs was attached to the upper crosshead of the universal testing machine to record crosshead displacement. The displacements and load were recorded at the rates of 20 or 30 scans per second on an electronic data-acquisition system until the prisms failed. All NIST specimens were capped according to ASTM procedures with the capping and handling fixture developed by Jones and Hornyak (1992). See Figs. D2.1 and D2.2.

The University of Tennessee. Thirty-one HCT prisms were tested at UT in uniaxial compression to failure. Twenty-three of these prisms were 8 in. thick, and the remainder were 13 in. thick. The prisms were constructed with running bond, a full-bed joint, and face-shell bedding for the head joint. The 8-in. prisms were tested at various angles to the bed joint (see Fig. D2.3). The 13-in. prisms were tested with the bed joint either horizontal or vertical. Stress-strain relationships were determined by LVDTs over gauge lengths that included mortar joints. Several of the prisms were loaded, unloaded, and reloaded during the test so that the effect of the cycling load could be observed.

D.2.4 Results and Conclusions

National Institute for Standards and Technology Task 1. The results of the six prism tests are summarized in Tables D2.1 and D2.2. On the basis of quantitative and qualitative results, the following conclusions may be drawn:

1. The NIST prism-handling assembly and procedures for lifting, rotating, leveling, and moving prisms are acceptable for Task 2 prisms.
2. The NIST capping procedures are recommended for Task 2 prisms.
3. The loading rate for Stage 1 of loading should be between 0.05 and 0.10 in./min.
4. To meet the loading-rate requirements of ASTM E 447, the Stage 2 loading rates should be 0.020 in./min for normal prisms and 0.030 in./min for parallel prisms.

5. On the basis of reducing the data by a factor of 100, it is concluded that a scan rate of 10 scans/s is sufficient to capture all of the critical events occurring during a given test.
6. There was relatively close agreement between the vertical-displacement measurements obtained by the outside LVDTs and those obtained by the center LVDTs. This observation notwithstanding, three vertical LVDTs attached to each of faces 1 and 2 should be used in Task 2.

Fig. D2.1. Photograph showing capped prism.

Table D2.1. Summary of NIST task 1 load and displacement results

Prism I.D.	Age of prism (days)	Loading rate (in./min)		Duration of test (s)	Scan rate (scan/s)	Max. load (lbf)	Max. disp (in.)
		Stage 1	Stage 2				
N13PT1	29	0.050	0.015	470	1000	223,194	0.153
N13PT2	32	0.075	0.030	22	200	206,974	0.105
N8NT1	40	0.050	0.020	329	50	207,500	0.067
N8PT1	40	0.050	0.020	240	50	150,000	0.053
N13NT1	28	0.140	—	152	1000	138,348	0.025
N13NT2	28	0.05	0.015	130	200	95,250	0.01

NIST: National Institute for Standards and Technology

Fig. D2.2. Photograph showing NIST parallel prism.

National Institute for Standards and Technology Task 2. The results of the 41 prism tests are summarized in Table D2.3. The failure mode exhibited by the normal prisms differed significantly from that of the parallel prisms. Both the 8-in. and 13-in. 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 HCT webs) were heard at load levels between approximately 25% and 50% of the maximum load. The frequency and volume of the popping sounds 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. Rubble consisting of broken pieces of HCT units, sandwiches of HCT units and mortar joints, and, in some cases, a portion of the bottom course of HCTs remained after these tests.

Fig. D2.3. Photograph showing angled prism compression test.

Table D2.2. Summary of NIST task 1 strength and modulus results

I.D.	Max load	Gross area	Net area	fmt gross	fmt net	E33 gross	E33 net
N13PT1	222,825	293.41	92.54	759	2408	529	1678
N13PT2	206,974	295.65	92.54	700	2237	776	2478
N8NT1	204,873	186.52	49.60	1099	4083	863	3245
N8PT1	150,008	187.52	51.34	800	2922	792	2893
N13NT1	138,348	291.31	84.50	475	1637	1016	3502
N13NT2	95,250	291.19	84.50	327	1127	1123	3873

NIST: National Institute for Standards and Technology

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 HCT. Thereafter, the prisms remained virtually intact and the faces contained a number of vertically oriented cracks indicative of splitting failure.

On the basis of the quantitative and qualitative results, the following conclusions may be drawn:

1. The values recorded by the center vertical LVDTs agreed closely, which indicates that the axial load was applied uniformly.
2. For the normal prisms, comparison of the output from the center and outside LVDTs extends from zero loading up to 60% of the expected maximum load. Generally, over the limited range of comparison, close agreement was seen between center and outside LVDTs for the normal-prism tests. This observation led to the conclusion that the center LVDTs alone can be used to measure the vertical shortening of the prisms.
3. The displacement readings from the horizontal LVDTs mounted at mid-height of the 8- and 13-in. normal prisms remained virtually unchanged over the entire load range to which they were exposed. The approximate average readings were 0.002 in. for 8-in. normal prisms and 0.0008 in. for 13-in. normal prisms. Because the readings were constant throughout a given test, no conclusion is drawn with regard to a Poisson effect. However, the conclusion was drawn that a Poisson effect existed in the parallel prisms. However, no historical data on Poisson's ratio for HCT prisms against which these results can be compared are known.
4. No evidence exists for a transverse Poisson's effect. Horizontal deflection values along the two edges were relatively insignificant for most of the 13-in. parallel-prism tests.
5. The secant moduli values listed in Table D2.3 indicate that the normal prisms were about 67% stiffer than the parallel prisms.
6. As indicated by either gross or net area, the 8-in. normal prisms had the highest compressive strength, and the 13-in. normal prisms had the lowest.
7. As indicated by either gross or net area, the compressive strengths of the 8-in. and 13-in. parallel prisms were comparable.
8. The relatively low strengths exhibited by NIST task 2 prisms indicate that masonry workmanship can significantly affect the strength realized by the prisms.
9. No significant difference was found between the failure loads obtained under displacement control and those obtained under load control. However, the number of prisms tested under load control within each prism type is too small to enable a rigorous statistical analysis to be performed.

Table D2.3. Summary of strength and modulus test results

I.D.	fmt gross	fmt net	E33 gross	E33 net	E50 gross	E50 net
N8PT1	799.91	2921.70	792	2892	670	2447
N8P1	833.96	3005.45	546	1967	545	1963
N8P2	821.46	2999.61	587	2142	587	2143
N8P3	483.74	1753.02	259	940	270	980
N8P4	539.74	1957.54	234	850	296	1075
N8P5	820.99	2980.13	528	1917	561	2038
N8P6	631.15	2298.40	394	1436	454	1654
N8P7	789.27	2843.79	670	2414	659	2375
N8P8	750.37	2697.31	710	2554	689	2478
N8PL1	773.97	2814.57	623	2264	606	2203
N8PL2	697.93	2522.40	494	1785	504	1821
8PAVG.	722.05	2617.63	531	1924	531	1925
N8NT1	1098.54	4131.05	863	3245	864	3247
N8NA1	1093.53	4082.66	808	3015	799	2982
N8NB1	1014.58	3802.42	793	2973	797	2988
N8NA2	1212.83	4547.38	835	3131	815	3056
N8NB2	940.71	3506.05	846	3155	804	2996
N8NA3	880.31	3296.37	1291	4385	1038	3886
N8NB3	1025.32	3829.64	800	2989	804	3002
N8NA4	1019.55	3806.45	798	2978	801	2991
N8NB4	786.91	3092.74	768	3018	740	2910
N8NA5	1093.25	4093.75	807	3023	796	2982
N8NAL	1110.87	4189.52	1271	4794	1284	4842
N8NBL	931.73	3477.82	857	3199	848	3166
8NAVG.	1017.34	3821.32	895	3363	866	3254
N13PT1	759.35	2407.61	529	1678	583	1849
N13PT2	699.98	2236.33	776	2478	751	2400
N13PA1	997.70	3146.75	618	1951	673	2122
N13PB1	908.68	2849.58	535	1677	614	1925
N13PA2	550.15	1754.92	1287	4105	1149	3665
N13PB2	520.80	1653.34	763	2421	700	2223
N13PA3	701.08	2229.31	457	1453	441	1401
N13PB3	850.63	2728.55	468	1503	507	1628
N13PA4	800.26	2516.75	514	1618	558	1755
N13PB4	838.85	2655.07	520	1645	564	1784
N13PAL	862.87	2720.45	537	1694	527	1660
N13PBL	1021.00	3220.23	878	2769	853	2689
13PAVG.	792.61	2509.91	657	2083	660	2092
N13NT1	474.92	1635.34	1016	3498	882	3036
N13NT2	327.11	1125.89	1123	3867	921	3169
N13NA1	390.98	1349.88	1488	5136	1195	4127

Table D2.3 (continued)

N13NB1	455.06	1563.83	1323	4547	1060	3643
N13NA2	394.39	1362.88	1128	3896	1208	4175
N13NB2	463.97	1596.93	883	3039	851	2930
N13NA3	458.35	1598.11	1007	3513	988	3445
N13NB3	404.70	1400.71	1542	5336	1062	3676
N13NA4	427.39	1459.81	1241	4238	1132	3865
N13NB4	487.41	1670.21	1009	3457	951	3258
N13NAL	441.72	1518.91	1347	4632	1143	3929
N13NBL	489.71	1678.49	1137	3898	1039	3560
13NAVG.	434.64	1496.75	1187	4088	1036	3568

D.3 BOND WRENCH TESTS³

D.3.1 Introduction

The bond wrench method of testing provides a simple and economical means for determining comparative values of flexural bond strength. Laboratory bond wrench testing was performed at Iowa State University in conjunction with the full-scale Y-12 replica testing described in Appendix E. The bond wrench procedure used for this research was as outlined in ASTM C 1072-86. This procedure, modified from the technical paper by Brown and Palm (1982), was originally intended for the bond wrench testing of brick-masonry units and, thus, the test fixture was modified to accommodate larger masonry specimens. These bond-wrench tests were the only ones performed on laboratory-constructed specimens. The in situ bond-wrench testing performed at the Y-12 Plant is documented in Appendix C.

D.3.2 Objective

The purpose of this testing was to provide flexural-capacity values (ultimate stress) at the unit-to-mortar interface for masonry specimens. These data would then be qualitatively and analytically correlated with the out-of-plane drift testing on full-scale Y-12 replicas described in Appendixes E and G of this document.

D.3.3 Scope and Methodology

Twenty-three bond wrench specimens were tested. Six 4-in. and six 8-in. specimens were built during the construction of the first wall. Six 4-in. and five 8-in. specimens were built during the construction of the second wall. All specimens were made during the infill construction process, and mortar for the specimens was sampled uniformly. The construction process for the two-block specimen was identical to that used for Wall 1 and Wall 2. That is, blocks were laid with the cores running horizontally and were separated by a 1/2-in. bedjoint. Specimens were allowed to cure for a minimum of 28 days before testing.

³Portions were of this section were extracted from Henderson (August 1994) with permission from the author.

The test apparatus consisted of a clamping bracket with a loading arm that is attached to the masonry unit selected for testing, as shown in Fig. D3.1. 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 between the upper and lower blocks. The applied load was measured with a compression load cell in conjunction with a digital multimeter. The multimeter continually monitored the applied load during the test process and saved and displayed the maximum load value. From this maximum applied load, a value of flexural bond strength was calculated.

Fig. D3.1. Bond wrench apparatus.

The flexural/tensile strength of the bond is calculated by use of an equation that incorporates the stresses induced by the bond wrench and the applied load, the section modulus of the specimen, and the compressive stress caused by the mass of the test device and specimen resisting the bending forces. For the arrangement shown in Fig. D3.1, the flexural bond strength may be calculated as follows:

$$F_s = \frac{6(PL + P_1 L_1)}{bd^2} - \frac{P + P_1}{bd} \quad (D3.1)$$

where

- F_s = flexural tensile strength (pounds per square inch),
- P = maximum applied load (pounds),
- P_1 = weight of the bond wrench and wall unit (pounds),

- L = distance from the loading point to the center of the wall unit (inches),
 L₁ = distance from centroid of bond wrench to the center of the unit (inches),
 b = average length of the wall unit (inches),
 d = average width of the wall unit (inches).

D.3.4 Results and Conclusions

The results of the bond wrench testing are shown in Table D3.1.

Table D3.1. Bond wrench data

Specimen #	Wall #	Block size	Course #	Load (lb)	Modulus of rupture (psi)
1	1	8	1	466	79
2	1	4	3	350	252
3	1	8	5	604	102
4	1	4	7	71	54
5	1	8	9	719	121
6	1	4	11	87	65
7	1	8	13	1064	179
8	1	4	15	103	76
9	1	8	17	798	135
10	1	4	19	160	117
11	1	4	21	107	79
12	1	8	23	422	72
13	2	4	1	139	102
14*	2	4	1	230	167
15	2	8	3	688	116
16*	2	8	3	512	87
17	2	4	5	81	61
18*	2	4	5	156	114
19	2	8	7	370	63
20*	2	8	7	680	115
21	2	4	9	136	100
22	2	8	11	753	127
23	2	4	13	348	251
* Indicates cleaned block				Average	389
					114

The data shown in Table D3.1 were calculated by using Eq. (D3.1) and the descriptive test machine information shown here:

		<u>4-in. Block</u>	<u>8-in. Block</u>
L	=	19.25	21.38
L ₁	=	10.75	12.75
b	=	11.56	11.56
d	=	3.688	7.875
weight	=	18.00	35.00
P ₁	=	43.00	26.00

As indicated, 12 4-in. blocks and 11 8-in. blocks were tested (a total of 23 specimens). The size of the blocks tested had little effect on the rupture stress. The average modulus of rupture was 120 and 109 psi for the 4- and 8-in. blocks, respectively. The overall average for the 4- and 8-in. blocks combined was 114 psi. Additionally, 4 of the 23 blocks tested were cleaned before the testing. This process also had little effect on the overall bond wrench results. In general, the process of constructing and testing the bond wrench specimens was very uniform, yet results from the testing are quite scattered: the standard deviation is 54 psi and the coefficient of variation is approximately 47 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.

D.4 TESTS OF MORTAR PROPERTIES

D.4.1 Introduction

The strength of the mortar used in masonry construction is an extremely important constituent in the overall behavior of structures or specimens. For this reason, compressive strength tests were performed on mortar used in many of the individual laboratory test series including (1) NIST prism testing (Yancey 1993a, b) (2) Iowa State University (ISU) large-scale replica testing (Henderson August 1994) and (3) UT moderate-scale tests (Boyd 1993). Also, compressive strength tests were performed by Testwell Craig Materials Consultants on in situ mortar removed from pilasters during boundary-condition investigations at the Y-12 Plant (see Appendix B, Butala 1992, and Butala and Jones 1993).

D.4.2 Objective

The purpose of the mortar testing at NIST, ISU, UT, and Testwell Craig was to determine the compressive strength of the mortar used to construct the larger-scale specimens (i.e., masonry prism and infilled frames).

D.4.3 Scope and Methodology

National Institute for Standards and Technology. Mortar compressive-strength tests were performed on 114 specimens in conjunction with the prism testing performed at NIST. The test

procedure specified testing three cubes at 28 days and three cubes on the day of an associated prism test. The test procedure was followed in all but a few batches for which the 28-day strength was not obtained. The cubes were tested in accordance with ASTM C 780-90.

Iowa State University. Mortar was sampled uniformly, in the form of mortar cubes and mortar cylinders, throughout the construction for each infilled frame tested at ISU (see Appendix E). The number of mortar cubes and cylinders sampled during the 2-day construction processes for each wall is listed in Table D4.1. Mortar used for the construction of both walls was mixed in accordance with ASTM C270, Type N, and had volumetric relationships resulting from the Testwell Craig petrographic and chemical analyses described in Appendix B. Tennessee River sand was used. The sampling and testing of the mortar specimens were called for in ASTM C 109-84. Cubes sizes were $2 \times 2 \times 2$ in., and the cylinders were 3 in. in diameter and 6 in. in length. Cubes and cylinders were tested at 7, 14, 21, and 28 days.

Table D4.1 Summary of mortar testing

	Wall 1		Wall 2	
	Cubes	Cylinders	Cubes	Cylinders
Day 1	46	—	45	45
Day 2	42	20	50	54

The University of Tennessee. Mortar tests were conducted for each of the infilled specimens tested at UT. The masonry specimens were constructed with Type N mortar of approximately 1:2.5 mix proportions of mortar cement and natural river sand. The only exception to this was the Type S mortar used in the retrofit of Specimen 10b. (The mortar mix proportions specified by Testwell Craig were not used because their results were unavailable at the start of the UT testing.)

The infills were allowed to cure in the ambient temperature and moisture conditions of the laboratory testing facility. Mortar cubes (2-in. $\times$ 2-in. $\times$ 2-in.) were molded according to ASTM C 109-88 and allowed to cure 2 days in the laboratory testing facility before being placed in a moist room for the time remaining until testing. Mortar-cube specimens were tested according to ASTM C 780-91 at approximately 30 days curing and on the days of infill or prism testing.

Testwell Craig. Six large pieces of mortar were removed from Building 9212 (D-wing) during the investigation of boundary conditions described in Appendix B.3. Compressive strength tests were performed on four of these extracted specimens. (Two samples were damaged in shipping.)

D.4.4 Results and Conclusions

National Institute for Standards and Technology. Average compressive strengths ranged from 2242 to 2574 psi. Most of the series of mortar-cube tests were tightly banded with coefficients of variation of approximately 3% or less. The highest coefficient of variation was 6.2%.

Iowa State University. Average 28-day cube strength for both test structures was between 2000 and 2250 psi. Cube strength, in general, was considerably higher than cylinder strength at each of the four test days for both walls; however, a much closer relationship between cube and

cylinder strength existed for Wall 2. This difference was probably caused by the limited number of cylinders tested for Wall 1 (i.e., 20) as compared with Wall 2 (i.e., 100). The difference between cylinder and cube strength for Wall 2, where virtually the same number of cubes and cylinders were tested, can be attributed to the significantly higher confinement present in cube testing.

The University of Tennessee. Mean compressive strengths of the 8-in. prism mortars were 2428 psi (COV = 29%) and 1931 psi (COV = 23%) for cores normal to the load and cores parallel to the load, respectively. Mean compressive strengths of the 13-in. prism mortars were 1807 psi (COV = 18%) and 1723 psi (COV = 14%) for cores normal to the load and cores parallel to the load, respectively. Mean 30-day mortar compressive strengths for the infill testing ranged from 1240 psi (COV = 6.1%) for infill Specimen Number 23 to 3040 psi (COV = 10.7%) for infill Specimen Number 17.

Testwell Craig. The average compressive strength of the four pieces of mortar (removed from Y-12 Building 9212) ranged from 2870 psi to 4390 psi. The average was 3580 psi.

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Appendix E

MODERATE- AND LARGE-SCALE TESTS

 
Bob Johnson	1992

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ABSTRACT

One of the primary elements of the test program was the research on both in situ and laboratory moderate- and large-scale test specimens. Because of the similarity of the test structures to the unreinforced masonry infills at Y-12, this research area provided the clearest indicator of the response of the infills to the application of lateral loads. The following tests were performed:

1. In situ airbag test of a wall in Y-12 Building 9207,
2. Full-scale testing of laboratory replicas of the infills in Y-12 Building 9212,
3. Moderate-scale testing of 21 unreinforced masonry infills through the use of quasi-statically applied in-plane and out-of-plane loads, and
4. dynamic shake-table testing of moderate-scale unreinforced masonry infills.

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Appendix E

MODERATE- AND LARGE-SCALE TESTS

One of the primary elements of the test program was the research on both in situ and laboratory moderate- and large-scale test specimens. Because of the similarity of the test structures to the unreinforced masonry infills at Y-12, this research area provided the clearest indicator of the response of the infills to the application of lateral loads. The tests are described in this appendix.

E.1 AIRBAG TEST OF Y-12 BUILDING 9207 INFILLED FRAME¹

E.1.1 Introduction

An existing unreinforced-masonry (URM) infill wall was load tested out-of-plane with an airbag. The test wall was located on the ground floor of Building 9207, a five-story steel-frame structure built in 1945, which was used for many years as a biology laboratory. The airbag test was a very large undertaking that started more than a year prior to actually performing the test. Site preparation of the wall and adjacent building areas, design and field fabrication of test supporting structures, procurement of equipment and instrumentation, development of supporting test procedures and checklists, installation of more than 70 linear variable differential transducers (LVDTs) and strain gages, development of computer programs for use in the data-acquisition systems, extensive review of the existing engineering literature, discussions with researchers who had prior experience performing tests with airbags, coordination with the building operators, plant-safety reviews, and dry runs of the test itself were all part of the test preparation.

E.1.2 Objectives

The purpose of the airbag test was to determine the wall's out-of-plane capacity to resist wind and seismic loadings by applying lateral pressure (with an airbag) to an existing wall. This result was useful to the overall objective of the HCTW Test Program of gaining a better understanding of how buildings with hollow clay tile (HCT) infill walls respond to these types of loadings. The test also served the major purpose of demonstrating that URM infill walls are inherently stronger than would normally be 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. determine the existing boundary conditions (connectivity of the wall to the steel frame),
3. establish crack patterns,
4. obtain data for correlation with computer analyses for use in developing accurate behavioral models for future seismic analyses of buildings having similar wall construction,
5. demonstrate that this type of wall is capable of resisting larger out-of-plane loads than would be calculated by normal means, and

¹Portions of this section were extracted from Huff and Fricke (1992) with permission from the authors.

6. address U.S. Department of Energy (DOE) design and evaluation requirements for natural-phenomena hazards.

E.1.3 Scope and Methodology

The wall panel was 28 ft wide and 12 ft high (floor to top of wall), and consisted of single-wythe unreinforced construction made from HCT units $12 \times 12 \times 8$ in. thick (wall thickness) with the cores running horizontally and with a running-bond construction. The panel was located on the ground floor of Building 9207 along N-S column line D between column lines 2 and 3. The building is a five-story steel-frame structure with reinforced-concrete floor slabs built around 1945.

The wall was infilled between W14x142 columns with a W30x108 overhead beam and concrete slab floor beneath. The wall tiles were placed up against the column's webs, but because the distance between the inside of the flanges ($12\frac{1}{2}$ in.) exceeded the width of the wall (8 in.), the wall was not fully captured in the columns. Thus, rotation of the wall about the inside corner of the column flanges was possible. The wall from the airbag side is shown in Fig. E1.1 (in the picture, the reaction frame for the airbag is seen in front of the wall). The upper beam is oriented with the strong axis in the plane of the wall. 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 main overhead beam has a W21x59 beam framing in at midspan on each web face.

The airbag test was run on Saturday, October 5, 1991. Two preliminary low-level tests (part of the final checkout procedures) to approximately 0.554 in. H_2O (0.02 psi) airbag pressure were run prior to the actual test. These pretest runs 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 test process are recounted in the following paragraphs.

Pressurization of the airbag for the main test began at 11:12 on October 5. The first sounds indicating possible cracking were audible approximately 40 min later at a pressure of 0.28 psi. Displacement at the center of the wall was measured to be 0.066 in. The first vertical crack, which occurred in a mortar joint, became visible at this time.

A horizontal crack near the top left beam interface was noted at a pressure of 0.49 psi and a centerline deflection of 0.104 in. At a pressure of 0.84 psi and a centerline deflection of 0.246 in., the beam and mortar joint separated. Stair-step cracking patterns began to develop in the upper quadrants of the infill at 0.96 psi.

At 1:12 pm, a pressure of 1.007 psi was reached; centerline deflection was 0.60 in. This was the maximum pressure attained during the test. Almost immediately, the pressure began to fall, and for the next 25 min, as air continued to flow into the airbag, the pressure fluctuated between 0.92 and 0.99 psi. During this period, wall movement was obvious, and the centerline deflection increased more than 1 in. to 1.68 in.

At 1:37 pm, the XY-plot of the stress in the overhead steel beam and the centerline displacement indicated the presence of nonlinear behavior; therefore, an air-pressure dump was initiated, and the test was terminated. Release of the pressure from the wall took about 30 s. Data were read for a few more minutes after the pressure was at zero to allow the LVDTs to stabilize.

E.1.4 Results and Conclusions

The airbag test lasted about 2½ h. Wall and steel-frame deflections and strains were automatically read and recorded at approximately 2-s intervals, so that about 4000 data records (each with the data from 94 channels) were produced. Cracking of the wall was first noticed at a pressure level of about 0.28 psi. The airbag pressure reached a maximum of 1.0 psi approximately 2 h into the test, after which the pressure fell and then fluctuated between 0.90 psi and 0.96 psi for the next 26 min. During this time, the LVDTs indicated that large displacements were taking place, even though major distress was not visible in either the wall or the frame.

Finally, the test was terminated because the LVDTs measuring the behavior of the overhead steel beam indicated that large lateral displacements and possible rotations were occurring in that member. The maximum deflection of the center of the wall, just prior to unloading, was 1.68 in. See Figs. E1.2 and E1.3.

Fig. E1.1. Photograph showing reaction frame.

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. During the dismantling of the wall, the presence of wall sections composed of two 4-in.-thick blocks (double-wythe construction) was discovered in an area of the wall where a door had once existed and the entire first course was sitting on the floor.

In-plane loads that develop during the out-of-plane loading greatly enhance the ability of the wall to resist the lateral loads. Some of the factors that affect the out-of-plane strength of URM

infill walls 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 surrounding frame.

Fig. E1.2. Vertical displacement at midwidth.

Fig. E1.3. Horizontal displacement at midheight.

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 postcracking mechanisms, especially the arching action of the wall within the steel frame, which provides a substantial increase in the predicted capacity. These methods predict "failure" of the wall because of ground acceleration at pressures of 0.07 to 0.1 psi and wind speeds significantly less than the straight design wind speed of 80 mph. Because the maximum pressure obtained in the test was 1.0 psi, the statement can be made that

infills similar to the one tested are highly unlikely to fail out-of-plane as a result of either extreme wind or a seismic event. This is true despite amplification of the ground acceleration at higher elevations within a building. This wall is probably not atypical of many of the other HCT infills at Y-12. Despite the variable (and rather poor) construction encountered within this wall, it still demonstrated a remarkably high lateral-load capacity. The wall from the measuring frame side is shown in Fig. E1.4.

Fig. E1.4. Photograph showing measuring frame and instrumentation.

E.2 QUASI-STATIC TESTS OF MODERATE-SCALE INFILLS AT THE UNIVERSITY OF TENNESSEE²

E.2.1 Introduction

To evaluate the behavior of HCT infills, an experimental program on half-scale infilled frames was formulated. The dimensions of the tested HCT infills varied, but the minimum nominal dimension was 8 ft, incorporating at least seven tile units. Twenty-one moderate-scale infills were tested to determine their behavior and correlate the results with other available experimental data.

E.2.2 Objective

The objective of the experimental program was to test a broad range of steel-frame specimens infilled with HCT 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. These techniques were used for the analysis of Y-12 structures as described in Appendix J of this document.

E.2.3 Scope and Methodology

To determine the behavior of the infills, 21 half-scale specimens consisting of steel frames infilled with unreinforced structural clay tile were tested. The in-plane test setup of a typical solid panel is shown in Fig. E2.1. All of the infills consisted of solid panels except one that had a 2-by 2-ft opening in the upper corner. The test specimens consisted of portal frames infilled with HCT masonry of either single-wythe or double-wythe construction. Single-wythe panels were constructed of nominally 8-in-thick units, and double-wythe panels were constructed of 8-in. and 4-in. tile units, thereby creating a nominally 13-in.-thick wall.

The panels were built with running bond with American Society for Testing and Materials (ASTM) Type N masonry cement. Mix proportions by volume of the masonry cement and natural river sand were approximately 1:2.5. The clay tiles were laid with their cores horizontal (side construction) with approximately 3/4-in. full-bed joints and only face-shell mortar on the head joints. A full width of mortar was placed between the concrete structural floor and the bottom course of each panel. For all tests except one weak column-interface test, each infill was bonded to the column and beam by mortar snugly packed between the steel and the tile. Mortar was not placed between the panel and the flanges of the enclosing columns. No reinforcement was used in the masonry. For all tests except one offset-panel test, each infill was constructed with the centerlines of the panel and the framing coincident. The panels were cured in the ambient temperature and moisture conditions of the testing laboratory for a minimum of 30 days prior to testing.

The specimens were constructed of A36 steel, with the columns welded continuously to 1-in. $\times$ 2 ft 6 in. $\times$ 2 ft 6 in. baseplates with a 1/4-in. fillet weld. The baseplates were anchored to the concrete structural floor with 1.25-in. tie rods. Tube sections covering the baseplates and extending

²Portions of this section were extracted from Flanagan (December 1994) with permission from the author.

Fig. E2.1. In-plane test setup.

to tiedown locations beyond the frame were anchored to the floor to provide additional base restraint and uplift capacity. Beam-to-column connections were framed with double clip angles so that a partially restrained connection resulted. The connection consisted of 1-in.-thick angles with a 1-in.-diam A325 bolts (beam web) and 3/4-in. A325 tension bolts (column flange). The bolts were installed as slip-critical connections with the turn-of-the-nut method.

Eleven steel frames infilled with HCT 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 HCT infilled frames were tested out-of-plane to failure with uniform lateral loading. These tests were designed to simulate inertial effects of the infill normal to its plane. Sequential bidirectional 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 another direction. A combined bidirectional test was performed on a single infilled frame whereby airbag and racking loads were applied simultaneously. The tests performed under this phase of the test program are summarized in Table E2.1. Photographs of the in-plane racking, out-of-plane drift, and airbag test setups are shown in Figs. E2.2, E2.3, and E2.4, respectively.

E.2.3 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.

In-plane tests that evaluated varying frame stiffness, single- and double-wythe construction, varying panel aspect ratio, weak column-infill interface, and repaired panels exhibited remarkably different stiffness characteristics but relatively similar ultimate strengths. The contribution to total strength by the framing is small, and the contribution of the infill is nearly constant.

Infills with the panel offset from the frame centerline developed peak capacities proportional to the effective net area enclosed in the framing. An infill with a significant upper-corner opening developed a compressive strut lower in the panel. The result was a 60% loss of capacity in the corresponding direction.

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. Typically, vertical arching developed until failure of the top- and bottom-course tiles. The horizontal arching that then developed maintained the stability of the panel. The failure mechanism of the masonry along the panel boundary indicated a combined shear and compression failure in the tile units.

The stability of HCT 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.

A sequential test of in-plane racking followed by uniform lateral 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.

A sequential test of uniform lateral loading followed by in-plane racking 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.

A combined in-plane-racking and uniform-load test resulted in reduced lateral-pressure capacity. Lateral loads producing thrust forces around the panel perimeter and in-plane loads producing strut forces along the diagonal combined to create high vertical compression near the panel base and cause failure of the bottom course tiles. However, even after much damage to the masonry panel, the infills remained remarkably stable under combined loading.

Under uniform out-of-plane load, the infills cracked along the mortar joints and developed membrane forces. Tremendous out-of-plane capacity was observed as the panels arched vertically and then horizontally, remaining stable after ultimate capacity was reached. Under sequential and combined bidirectional loading, the panels remained stable with little interaction of the in-plane and out-of-plane behavior, particularly in the frame-member forces.

Table E2.1. Summary of the University of Tennessee testing

Test no.	Panel thick (in.)	Panel length (in.)	Panel height (in.)	Column size	Beam size	Description (including column orientation)
1	7.67	88	88	W10x12	W12x35	Single wythe; weak axis
2	7.67	88	88	W10x30	W12x35	Single wythe; weak axis
3	7.67	88	88	W10x45	W12x35	Single wythe; weak axis
4	13	88	88	W16x26	W18x76	Double wythe; weak axis
5	13	88	88	W16x40	W18x46	Double wythe; weak axis
7	4.67	88	88	W10x30	W12x35	Single wythe; weak axis; 3.5" offset with 8" tile
9	7.67	88	88	W10x30	W12x35	Single wythe; weak axis
10a	7.67	112	88	W10x30	W12x35	Single wythe; weak axis; column/panel gap = 1"
10b	7.67	112	88	W10x30	W12x35	Panel 10a retrofit
17	7.67	136	88	W10x30	W12x35	Single wythe; weak axis
21a	7.67	112	88	W10x30	W12x35	Single wythe; weak axis
21b	7.67	112	88	W10x30	W12x35	Panel 21a repair
24	7.67	112	88	W10x30	W12x35	Single wythe; weak axis; corner opening - 2' x 2'
18	7.67	88	88	W10x30	W12x35	Single wythe; weak axis
22	13	88	88	W16x40	W18x76	Double wythe; weak axis
25	3.66	88	88	W10x30	W12x35	Single wythe; weak axis
11	7.67	88	88	W10x30	W12x35	Single wythe; weak axis; Drift then Racking
13	7.67	88	88	W10x30	W12x35	Single wythe; weak axis; Drift then Racking
19	7.67	88	88	W10x30	W12x35	Single wythe; weak axis; Racking then Airbag
20	7.67	88	88	W10x30	W12x35	Single wythe; weak axis; Airbag then Racking
23	7.67	88	88	W10x30	W12x35	Single wythe; weak axis; Airbag then Racking

Fig. E2.2. Photograph showing in-plane racking test.

Fig. E2.3. Photograph showing out-of-plane drift test.

Fig. E2.4. Photograph showing airbag test.

E.3 QUASI-STATIC TESTS OF Y-12 REPLICAS AT IOWA STATE UNIVERSITY³

E.3.1 Introduction

To enable better understanding of the complex behavior of HCT infills, an experimental program on full-scale infilled frames was formulated. The structures tested were laboratory replicas of an existing facility at the Y-12 Plant. Furthermore, loading methods were designed to simulate seismic loads as they would be applied in situ.

E.3.2 Scope and Methodology

The experimental phase of the research consisted of (1) out-of-plane testing on one bare frame without infilling, (2) out-of-plane testing of one infilled frame, (3) in-plane testing of the infilled frame previously damaged by out-of-plane drift, and (4) in-plane testing of an infill with no prior damage as summarized in Fig. E3.1. In addition to the full-scale tests, numerous correlative tests were performed including (1) 183 cube tests, (2) 119 cylinder tests, (3) 168 unit-block tests, (4) 20 prism-compression tests, (5) 23 bond wrench tests, and (6) 2 out-of-plane modal surveys. Items (2) and (3) are described in Appendix D of this document.

The in situ infilled construction, as replicated in each of the full-scale tests, is somewhat complex and differs from modern masonry construction in several ways. The HCT infill for the test structure had a 13-in. nominal thickness that was composed of individual 4- and 8-in. blocks. The HCT was oriented so that the cores ran horizontally (side construction) and in the direction parallel to the plane of the wall. Bedjoints were full and continuous with a 1/2-in. thickness. A 1/2-in. bedjoint was also placed between the floor and the first course.

Headjoint mortar was applied to the face shells only and was 3/8 in. thick. The 4- and 8-in. blocks alternated position from course to course and were laid in running bond (1/2-block offset). The collar joints of the test structure had a nominal thickness of 0.75 in.; however, because of the alternating block position, the collar joints were discontinuous at the interface with successive courses. No mortar was placed inside the collar joints. Four- and six-inch blocks were used for construction of the pilaster.

The face of the HCT infill was flush with one of the W10x33 column flanges, as was the web of the connective purlin. As a result, the construction process found to be most efficient entailed aligning the outer rows of blocks with the outer edge of the column flanges and then laying the adjacent row relative to the first. This assured vertical and horizontal plumbness of the infill. Bedjoint mortar was spread in sections of 3 to 4 ft at a time, and the blocks were laid and tapped into place. Construction of the infills is depicted in Fig. E3.2.

³Portions of this section were extracted from Henderson (August 1994) and Henderson et al. (1993) with permission from the authors.

Fig. E3.1. Summary of large-scale testing.

One infilled frame 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 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 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 at the purlin location 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 to obtain full tension-compression hysteresees. Also, natural frequencies of the first infilled panel were determined before and after the out-of-plane testing. The out-of-plane test setup and the final damage resulting from in-plane loads are shown in Figs. E3.3 and E3.4, respectively.

Fig. E3.2. Photograph showing infill construction.

E.3.3 Objective

The specific objectives of the research 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. determine the in-plane capacity and load-deflection relationships of an infilled frame with prior out-of-plane damage;
4. determine the in-plane capacity and load-deflection relationships of an infilled frame without prior out-of-plane damage;

5. By comparison, establish the effect of prior damage on the behavior and capacity of infilled frames;
6. establish out-of-plane and in-plane crack patterns;
7. establish the behavioral influence of the existing boundary conditions; and
8. obtain data for correlation with and improvement of analytical models.

Fig. E3.3. Photograph showing out-of-plane test setup.

E.3.4 Results and Conclusions

The bare frame and the first infill panel were tested out-of-plane in a quasi-static fashion by applying loads at four actuator locations to simulate drift loads applied by the top and bottom chords of a roof truss. The maximum applied out-of-plane displacement for wall 1 was approximately 2.6 in. Cracking damage to the first infill 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 and second infilled frames were then tested in-plane to failure. The in-plane displacement required to cause considerable corner crushing was approximately 3 in. for both infills. The maximum in-plane loads were 64 kips for the first infill (predamaged) and 61 kips for the second infill (no prior damage). Crack patterns and final damage states were very similar for both infilled frames. The conclusion from this testing was that prior damage has little effect on the in-plane capacity of infills provided confinement by the steel frame is maintained. In addition to the full-scale testing, numerous correlative tests were performed

including mortar-cube, mortar-cylinder, unit-block, prism-compression, and bond wrench tests as well as out-of-plane modal surveys.

Fig. E3.4. Photograph showing final in-plane damage.

On the basis of the experimental and analytical research described in the preceding section, the following conclusions regarding the behavior of unreinforced masonry infilled frames can be drawn.

1. The frame-to-infill connections used in the wall 1 and wall 2 construction were more than adequate to ensure composite behavior. The pilasters, which were composed of blocks with vertical cores, provided an extremely resilient connection that allowed the frame and infill to

withstand large out-of-plane and in-plane displacements with only minute separation. Similarly, the purlin beam contributed the necessary stability to the overall structure for loading in both directions, despite its eccentricity with respect to the columns and infill. Also, the use of 4- and 8-in. block, which alternated from course to course in running bond, augmented the compositeness of the test structure by eliminating the continuous collar joint typical of most double-wythe construction.

2. The out-of-plane behavior of wall 1 was, essentially, as anticipated by preliminary analyses; however, certain key analytical conclusions were confirmed. Infilled structures with continuous bedjoints react to seismic drift by forming cracks almost exclusively in the bedjoints at the locations of maximum moment. Cracks, generally, propagate from the columns inward toward the centerline. As deflection increases and cracks propagate along a bedjoint, stress is relieved at that vertical location, and a new crack begins at a new point of maximum moment. This may happen repeatedly until the infill becomes a series of horizontal beams separated by numerous complete through-cracks.
3. Though wall 1, at the maximum out-of-plane displacement, contained six or more cracks that fully penetrated the thickness of the infill, the test structure was still completely stable and able to resist load. 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. This philosophy applies an extremely conservative and unrealistic penalty to structures, particularly existing structures that may have been cracked numerous times over a long displacement history. This research indicates that masonry infills perform well under out-of-plane and in-plane loads despite numerous through-cracks.
4. Limited separation and subsequent differential movement of the frame relative to the infill material occurred for both the out-of-plane and in-plane testing. Observation of the test suggests that this frame slippage may have been helpful in reducing shear or bearing damage to the infill material by the structural steel frame. Likewise, the movement at the frame-to-infill juncture is suspected to enhance the ductility and energy absorption of infills. This observation is corroborated in research by Dawe and Hatzinikolas (1988), which indicates that if differential movement is restrained by ties, for example, damage to the structure is amplified.
5. A comparison of in-plane data for wall 1 and wall 2 clearly shows that prior damage to the infill reduces the in-plane initial stiffness. Also, walls with prior damage respond to in-plane loads in a much less brittle fashion than those with no prior damage. Prior out-of-plane damage results in sliding of the infill along preformed crack patterns. 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 an explosive fashion through brittle, diagonal X-cracking. Though more research is necessary, it would appear that virgin structures may actually be subjected to more damage and have a reduced long-term stiffness than predamaged structures under seismic drift. The research demonstrated conclusively that infills with no prior out-of-plane damage reach ultimate capacity early in the testing and suffer a decline in load for a given applied displacement thenceforth. Predamaged walls tend to build to ultimate capacity more slowly—behavior that could actually be advantageous under long periods of strong ground motion.

6. Major damage (i.e., severe block cracking and spalling) occurred in somewhat different sequences for wall 1 and wall 2. For the predamaged wall, the process of block spalling began directly at the column-to-purlin juncture of the infilled frame and progressed toward the center of the wall. For the undamaged wall, spalling began more toward the center of the structure and progressed, primarily, toward the corners. This seems to have occurred because the predamaged wall could slide and settle in the corners, thereby increasing the bearing forces in those locations. The end result, however, was essentially the same for both walls: two large corner openings.
7. In general, prior damage to infill structures has little effect on the overall in-plane performance or the final damage state. This fact is very important to the analysis of most older structures, which have been subjected to some cracking. An appropriate response to the conclusions drawn from this research would be a reassessment of the current design standards for infilled masonry construction.

E.4 DYNAMIC SHAKE-TABLE TESTS OF INFILLS AT THE U.S. ARMY CONSTRUCTION ENGINEERING LABORATORY⁴

E.4.1 Introduction

Dynamic shake-table testing of HCT infills was conducted at the U.S. Army Construction Engineering Research Laboratory (USA-CERL), located in Champaign, Illinois, during the summer of 1993. This work was accomplished through an Interagency Agreement between DOE and USA-CERL.

E.4.2 Objectives

The objectives for performing these tests were to accomplish the following:

1. investigate structural capability relative to ground-motion design spectra for a test wall that is representative of HCT wall at the Oak Ridge Y-12 Plant;
2. correlate conclusions resulting from 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 HCT-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 pseudostatic techniques.

⁴Portions of this section were extracted from Fowler (1994) with permission from the author.

E.4.3 Scope and Methodology

The test structure as shown in Fig. E4.1 consisted of two HCT-infilled frames spaced approximately 10 ft apart and connected by steel trusses and a concrete roof slab. Three types of dynamic tests were performed on the specimen (Fowler 1994). The first set of tests were performed with the walls oriented out-of-plane with respect to the loading direction. The specimen was then rotated 90° 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. E4.2.

Fig. E4.1. Photograph showing test structure atop shake table.

Out-Of-Plane Testing. Seven tests, as listed in Table E4.1, were performed on the structure in the out-of-plane direction. Three of the tests were performed by exciting the model with an Oak Ridge, Tennessee, site-specific seismic time history as shown in Fig. E4.3. The displacement record from Fig. E4.3 was programmed into the biaxial shake table and, in turn, used to excite the structure. As indicated in Table E4.1, the seismic tests were performed at 1, 2, and 4 times the full-scale value of the time-history displacements.

Fig. E4.2. Plan view showing test orientation.

Table E4.1. Out-of-plane testing sequence

Test	Test level	Test type
1	1.6% Full scale	Low-level random test
2	1 × Full scale	Seismic time history test
3	2 × Full scale	Seismic time history test
4	1.6% Full scale	Low-level random test
5	4 × Full scale	Seismic time history test
6	1.6% Full scale	Low-level random test
7	1.6% Full scale	Low-level random test

Due to the displacement limit of the CERL table of 2.75 in. peak, the acceleration time history shown in Figure E4.3 was filtered to remove all frequencies 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. Acceleration response spectra of the table motion achieved were calculated and plotted in Figure E4.4. Three levels of acceleration response are plotted along with the design and acceleration spectra calculated from the approved time history. These plots clearly show the effects of filtering below 0.75 hz. Prior to the filtering process, a low-level excitation was input to the table to determine resonances of the specimen to be tested. This confirmed that the lowest frequency was around 4 hz, which was well above the 0.75 Hz; therefore, the filtering did not effect the response of this specimen.

The other four tests consisted of using low-level, random-vibration input 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. These tests provided

just enough vibration amplitude to produce a noticeable response in the specimen. At the end of each out-of-plane test, crack maps were drawn to document the progression of mortar cracking.

Fig. E4.3. Seismic time history.

Fig. E4.4. CERL Acceleration response spectra (5% damping).

In-Plane Testing. Thirteen tests were performed with the structure oriented in the in-plane direction, as indicated in Table E4.2. In the in-plane testing, six tests were conducted with the same site-specific time-history record that was used for the out-of-plane tests. As Table E4.2 indicates, these tests 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.

Table E4.2 In-plane testing sequence

Test	Test level	Test type
1	1.6% Full scale	Low-Level random test
2	1 × Full scale	Seismic time history test
3	1.6% Full scale	Low-level random test
4	2 × Full scale	Seismic time history test
5	1.6% Full scale	Low-level random test
6	4 × Full scale	Seismic time history test
7	1.6% Full scale	Low-level random test
8	8 × Full scale	Seismic time history test
9	1.6% Full scale	Low-level random test
10	12 × Full scale	Seismic time history test
11	1.6% Full scale	Low-level random test
12	16 × Full scale	Seismic time history test
13	1.6% Full scale	Low-level random test

Sinusoidal Sweep Tests. 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 a 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 masonry panel began to fail, and the test was stopped at a frequency of about 12.7 Hz.

E.4.4 Results and 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.

Out-of-Plane Behavior. 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 a 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 panel and frame. 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 midheight 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.

In-Plane Behavior. 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. This might be because the large mass of the roof slab did not respond to the input base motion as drastically as did the column. 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 ($12 \times FS$). 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 locate the natural frequency of the structure, which was much lower than initially thought. The panels were eventually failed by using an input acceleration of 3.0 g, although the panels may possibly have failed because of fatigue.

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 still provided some load resistance. Therefore, infills can be expected to enhance the seismic resistance of otherwise laterally weak structures.

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Appendix F
SUPPLEMENTAL TESTS

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ABSTRACT

Because of the size and complexity of the Hollow Clay Tile Wall Test Program — as required to define the behavior of hollow clay tile infills — much of the information gained from the standard small-, moderate-, and large-scale masonry tests required refinement and clarification through supplementary test processes. For example, modal surveys helped to define the frequency and dynamic behavior of the moderate- to large-scale specimens, nondestructive evaluation provided information related to boundary conditions, and diagonal tension defined additional failure modes for masonry specimens. Some of the tests outlined in the test program were found to be unnecessary or cost-prohibitive; thus, these tests were discontinued.

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Appendix F

SUPPLEMENTAL TESTS

Because of the size and complexity of the Hollow Clay Tile Wall (HCTW) Test Program — as required to define the behavior of hollow clay tile infills — much of the information gained from the standard small-, moderate-, and large-scale masonry tests required refinement and clarification through supplementary test processes. Those supplemental tests are described in this appendix.

F.1 MODAL SURVEYS¹

F.1.1 Introduction

Two important parameters in the dynamic response of a structure to seismic loading are the structure's lower-mode frequencies and damping. To better understand these parameters, modal surveys were performed on Iowa State University (ISU) wall 1 and the University of Tennessee (UT) test specimens 11 and 13 prior to and after the out-of-plane, quasi-static testing. From these tests, an estimate of system damping, a frequency-response function, and a plot of the lower modes were obtained.

F.1.2 Objective

The purpose of the modal surveys was to obtain an estimate of lower-mode frequencies and damping as well as to gage the extent of damage to the infill caused by the out-of-plane loading.

F.1.3 Scope and Methodology

Iowa State University Testing. An impact hammer was used for exciting the wall panel, and subsequent accelerations were measured at 56 locations (i.e., eight rows of seven accelerometers per row), spread uniformly over the purlin side. Because of a limited number of accelerometers as well as data acquisition system channels, the structure was excited, and two rows (i.e., 14 channels) of accelerations were recorded. This process was then repeated four times to achieve the full 56 data points. The vertical location of the impact hammer was chosen to be at the centerline of the W16x36 purlin so that damage to the brittle hollow clay tile (HCT) by the impact would be avoided. The horizontal location of the impact hammer blows was approximately 9 ft from the left column and was chosen such that modal node points would be avoided. Finding the location of test structure node points was accomplished by varying the impact locations and assimilating the resulting structural frequencies. No out-of-plane restraints existed at the truss-chord points.

A full-response function, which plots frequency in cycles per second on the abscissa vs the magnitude of the response (i.e., acceleration in mg per pound force) on the ordinate, was produced for each accelerometer (i.e., 56 for the ISU testing) during the test. To ascertain the correct

¹Portions were of this section were extracted from Henderson (August 1994) with permission from the author.

frequencies, each of the 56 plots was overlaid, and the time-domain responses of the plots were qualitatively compared for shape, symmetry, and frequency magnitude.

University of Tennessee Testing. The UT testing was performed in a manner similar to that described for the ISU tests. Accelerometers were located on the HCT and steel frame. Varying the driving and recording points on the structures helped to confirm the consistency of the modal analysis technique despite the lack of homogeneity and linearity of the specimen (Flanagan December 1994).

F.1.4 Results and Conclusions

Iowa State University Testing. In general, the response functions for each of the accelerometer locations had similar shape and frequency content. However, certain accelerometer locations produced completely atypical responses. Generally, these occurrences were caused by the location of the instrument relative to the force point or a dynamic node point and were considered outliers for the qualitative comparison. The first four frequencies isolated from the response function as well as a description of the modal behavior are shown in Table F1.1.

Table F1.1. Out-of-plane frequencies of Y-12 infill replicas

Mode	Frequency (Hz)	Description
1	12.0	Low mode cantilever
2	15.8	Horizontal bending
3	18.0	High mode cantilever
4	23.4	Torsional cantilever

The second modal survey was performed subsequent to the out-of-plane quasi-static testing. As indicated in Appendix E of this document, the out-of-plane testing was performed up to a limiting displacement of 2.6 in. at each of the actuators and resulted in numerous horizontal bedjoint cracks that penetrated the thickness of the infill. These through-cracks were concentrated at the lower actuator location (i.e., at a height of approximately 12 ft) and, as a result, hindered the wave propagation to the lower accelerometers during the second modal survey. In other words, the modal behavior was isolated at the top half of the structure because of the damage caused by the out-of-plane testing. This modal information is extremely useful because it indicates a complete separation of the top and bottom portions of the test structure. Though this type of behavior was thought to have been observed during the out-of-plane testing, the post-test modal survey provides definitive confirmation.

University of Tennessee Testing. The types of modal response for the UT tests were similar in nature to the results obtained at ISU. However, the fundamental frequency of the frame and the infilling differed by 3%, which indicates that a good bond existed between the frame and infill both before and after drift loads were applied. This result is somewhat of a departure from the ISU tests, which showed a complete separation between framing and infill material. The first out-of-

plane frequency decreased by approximately 17% because of the drift loads for both specimens (Flanagan December 1994).

The conclusion may be drawn from both test series that significant damage resulted from the out-of-plane drift applied to the structures. This was indicated by the drop in fundamental frequency and the shape of the output-response function.

F.2 NONDESTRUCTIVE EVALUATION²

F.2.1 Introduction

Because most of the buildings at the Y-12 Plant were constructed in the mid-1940s, many of the buildings have undergone modifications or additions, and documented details of the early construction have been obscured with time. Although many construction details relative to structural capacity can be obtained by visual inspection, some important details may not be accessible for visual examination without extensive penetration or removal of the HCT by conventional methods. Thus, the development of suitable nondestructive evaluation (NDE) techniques was necessary (Wynn, Fletcher, and Jones 1992; and Bates and Phillips 1993).

Experiments have been conducted with sonics, ultrasonics, infrared thermography, and microwave NDE techniques on HCT-masonry construction at Y-12. Many of the techniques that showed promise in benchtop experiments proved to be disappointing for in situ evaluations. For the problem definitions specific to the HCTW Test Program, at least two NDE techniques continue to justify funding for further development of a useful method: infrared thermography and low-power microwave spectrography.

F.2.2 Objective

The purpose of NDE was to accomplish the following:

1. assist in determining the bond quality of the interface between the HCT and the surrounding steel columns and beams and concrete floor diaphragms,
2. assist in determining significant variations or departures from regular construction in the unreinforced masonry (URM) panels, and
3. assist in determining the local strength and condition of mortar in the URM panels.

F.2.3 Scope and Methodology

The initial list of NDE techniques that were potentially capable of addressing test-program objectives were determined from a review of current technologies. The list consisted of the following techniques:

- sonics: low-intensity applications of mechanical wave energy in the sonic range;
- ultrasonics: application of mechanical wave energy above the sonic range;
- infrared thermography: application and detection of radiant thermal energy; and

²Portions of this section were extracted from Wynn, Fletcher, and Jones (1992) with permission from the authors.

- microwaves: application of low-power narrow- and broad-band continuous wave.

Plans for the NDE part of the HCTW Test Program were formulated to develop and deploy an effective NDE method. The task would proceed in three orderly steps:

1. evaluate each of the techniques on the short list, and select one or two of the most promising;
2. develop test standards and probabilities of detection for the techniques picked in Step 1 and formulate an inspection method; and
3. procure a field-tested system and train inspectors for deployment into the Y-12 buildings.

Researchers and specialists in each of the techniques at the Y-12 Plant, the K-25 Site, Oak Ridge National Laboratory, and UT's Engineering Science and Mechanics Department submitted proposals to conduct experiments for the initial evaluation of the techniques on the short list, submit reports of results on the experiments, and recommend further development of the respective techniques as warranted.

F.2.4 Results and Conclusions

Experiments with HCT material in which sonics and ultrasonics were used provided predictable results with relatively new tile tested alone in free suspension. The average compression-wave propagation speed in the fired clay material was measured to be 16,750 ft/s. The average transverse-wave propagation speed was measured to be 6970 ft/s.

"A Span" ultrasonic measurements on the tile with transducers matched for ceramics inspection easily defined the face-shell thickness and the core-web junctions to the face shell. This phenomenon, in itself, could be used to determine the orientation of tile cores in situ. The determination of overall tile thickness in situ (i.e., is the tile 4-in. or 8-in. tile?) was uncertain because of severe attenuation of the reflected ultrasonic pulse from the backside surface of the tile.

Mechanical impulse-response testing on the exposed face shell of free tile showed that this information could be easily interpreted to enable determination of the overall thickness of the tile and the orientation of the cores. However, when similar tests were performed on tile and mortar prisms, very little usable information was obtained. The response-pattern auto-spectrums and input-response cross-spectrums were ambiguous relative to the reliable free-block tests. Even with two free blocks joined by one mortared head joint, the apparent attenuation of compression waves was so severe that detection of wave transmission on framed HCT panels was established before and after destructive testing. The purpose of wave transmission detection was to record first and second out-of-plane whole-panel modes so that modal frequency shifts could be correlated with the degradation of the structural integrity of the panels. The data trends were determined to be inconclusive, except that the trends do show definite downward shifts in modal frequencies occurring with degradation.

Initial experiments in one of the Y-12 buildings with thermography showed that irregularities were visible in the postprocessed thermal images, even through painted walls. However, interpretation of all of the irregularities on the thermal image was reliable. A buried steel lintel left behind from an old door and junctions of recent HCT construction to old construction were readily detected. Both transient and steady-state experiments were performed on in situ panels. A 6-ft-high by 8-ft-wide test panel of HCT-URM, incorporating many of the flaws and

irregularities typical of infilled frames, was constructed inside the flanges of an exposed steel column. This panel was used to correlate thermal images to flaws and to assess the feasibility of evaluating the bonding quality between the panel and the steel column. The test panel was also used to evaluate microwave applications.

The conclusion may be drawn that the most difficult problem is evaluating the quality of the bond of the HCT to the surrounding steel frame and diaphragm. The infill nested between the column flanges eludes attempts by most simple techniques to determine the quality of the bond. The combination of structural steel, mortar, and HCT in this junction complicates or obscures most wave detection and interpretation.

Developing an inspection method by using a single NDE technique or a combination of two NDE techniques along with visual inspection appeared, from the experiments, to be the most promising approach for detecting variations within the HCT. Little or no additional progress was made on this topic because of budget cuts and the reallocation of NDE development funds to destructive testing. Destructive testing was given a higher priority because verification of HCT strength and sensitivity to flaws and construction variations will come from the results of ultimate and postultimate tests on structures and assemblages.

F.3 DIAGONAL TENSION TESTS³

F.3.1 Introduction

Prior to planning the HCTW Test Program, three coupons approximately 4 ft square were removed from walls in Building 9206 for diagonal tension testing (Oak Ridge Y-12 Plant 1989). This testing was to be done to support the design of the Enriched Uranium Conversion Facility, or EUCF (Beavers et al. 1989).

Lessons learned from this early work helped to define the requirements for the future HCTW Test Program. Two important lessons learned were that (1) contaminated samples could not be cleaned and shipped offsite to be tested because of damage to them during cleaning (sand blasting) and (2) handling and removal equipment for extracting wall specimens would have to be specially designed and fabricated to prevent damage to the specimens.

F.3.2 Objective

The original objectives as reported by Bennett and Flanagan (1992) were to remove in situ specimens and to perform prism tensile tests. These tests would be used to obtain in-plane tensile strength, which would help to define the cracking strength of the wall. The cracking strength was considered a primary limit state of the HCT walls.

³Portions of this section were extracted from Oak Ridge Y-12 Plant, Y/EN-4619 (August 1989) with permission from the authors.

F.3.3 Scope and Methodology

In situ and laboratory-built specimens were to be tested as shown in Table F3.1. Because of difficulties in removing in situ specimens, testing of laboratory-built specimens would be used to supplement the data obtained from testing the in situ specimens. In the table, the in situ specimens are identified in column "Wall" and the laboratory-built specimens in column "Built." The specimens in the table defined as "45° to cores" are the classical diagonal tension (shear) specimens as defined by American Society for Testing and Materials (ASTM) E 519.

Table F3.1. Prism tensile tests

Test Direction	Quantity to be tested					
	8" Walls		12" Walls		Totals	
	Wall	Built	Wall	Built	Wall	Built
Normal to cores	3	4	3	4	6	8
Parallel to cores	3	4	3	4	6	8
45° to cores	3	4	3	4	6	8
TOTALS	9	12	9	12	18	24

F.3.4 Results and Conclusions

No prism tensile tests were ever completed during the HCTW Test Program. However, the testing by Engineering, Design and Geosciences Group, Inc. (EDGE) for the Oak Ridge Y-12 Plant to support the design of the EUCF is considered significant and is reported here. The ultimate shear stress as reported by the Oak Ridge Y-12 Plant (August 1989) is given in Table F3.2.

Table F3.2. Results of diagonal tension (shear) tests

Specimen	Ultimate shear (psi)	Shear strain (in./in.)
A	16.5	0.002543
B	25.1	0.002684
C	36.3	0.001423

EDGE tested three specimens at 45° to the cores. These were removed from Building 9206, sand blasted, and packaged for shipment by Martin Marietta Energy Systems, Inc. The specimens were picked up by EDGE and transported to Nashville for testing. After the samples arrived in Nashville, EDGE cut the specimens to a square shape to remove uneven edges; consequently, each specimen had a slightly different size. Specimen A was 13 in. thick by 32.8 in. square; specimen B was 12.7 in. thick by 36.5 in. square; and specimen C was 12.8 in. thick by 42.1 in. square.

The specimens were tested per ASTM E 519. EDGe reported that the specimens appeared to be damaged, especially specimen A, and suggested that prisms be laboratory built and tested with old tile and new mortar.

F.4 DISCONTINUED TESTING

Outbuilding Tests. During the beginning of the HCTW Test Program, two small HCT buildings at the Y-12 Plant (9743-2 and 9982) were scheduled for demolition and were possible candidates for full-scale static and dynamic testing. These structures were composed of 8-in. HCT and, because of their construction and roof masses, were good choices for low- and high-amplitude dynamic excitation. Although infills were not used in these structures, the results from this test series would be used to better understand the behavior of HCT systems and weak links in the global structural response to dynamic loads.

In the draft test plan (Bennett and Burdette 1989), it was proposed that these structures be dynamically tested by means of mechanical-vibration exciters (i.e., rotating mass shakers) attached to the concrete roof or constructed diaphragm element. The vibrational input on the system would be produced by rotating counterweights where the force is proportional to the rotating speed. Displacement and cracking of the structure at critical locations would be monitored throughout the testing.

After review by the Expert Review Committee (ERC), it was decided that this test series was somewhat impractical from a cost-benefit perspective. Moreover, the structures had been painted with asbestos paint, the removal of which would be difficult and expensive. For these reasons, this testing was canceled and not included in the final test plan (Bennett and Flanagan 1992).

Flexbond Tests. The flexbond technique for determining the bond strength of a masonry structure entails the removal of representative masonry specimens from existing walls or the building of laboratory specimens for the purpose of subjecting them to flexural loads by a method similar to that recommended by ASTM E 518. Flexbond testing was considered an important element in the overall HCTW Test Program and initially was given a high priority.

The objective of the test was to determine the flexural-bond strength at the clay-tile-to-bed-joint interface. Two variations of the test were to be performed, one with tension induced normal to the bed joints and one with tension parallel to the bed joints. Tension normal to the bed joints would simulate a vertically spanning masonry beam, and tension parallel would simulate a horizontally spanning masonry beam. Specimens removed from walls or laboratory-built specimens would be tested by bending in a horizontal position to avoid axial loads. The results of the tests would be the determination of the modulus of rupture, out-of-plane stiffness (flexural and shear), and ultimate strength. These data would then provide stiffness and strength data for use in analytical models.

The numbers of specimens to be tested for both 8- and 12-in. wall thicknesses are shown in Table F4.1. The removed wall specimens are designated as "Wall" and the laboratory-built specimens as "Built" in the table. Because most of the load-resisting walls were the external 12-in. walls, more laboratory specimens were to be tested.

Table F4.1 Planned Flexbond Tests

	8-In. walls		12-In. walls		Total	
Test direction	Wall	Built	Wall	Built	Wall	Built
Normal to cores	3	3	3	6	6	9
Parallel to cores	3	3	3	6	6	9
Total	6	6	6	12	12	18

The specimens to be tested normal to the cores were to be 24 in. wide by 72 in. long by the thickness of the wall. The specimens parallel to the cores were to be 36 in. wide by 72 in. long by the thickness of the wall. The specimens were to be tested in a horizontal position with loads applied at the third-points. A schematic of a test setup of an 8-in. wall specimen loaded with tension normal to the bed joints is shown in Fig. F4.1.

Fig F4.1. Schematic of flexbond test.

It was clear from the beginning that extreme care would be needed to protect the integrity of a specimen during removal, handling, and testing. Consequently, the design, fabrication, and procurement of equipment to do this was given a high priority. Jones and Hornyak (1992) wrote a procedure for the training of craftsmen and the technical testing of personnel in the use of this equipment.

All handling and removal equipment was designed, fabricated, or procured for flexbond testing. No specimens were tested, however, because as the testing program proceeded it became clear that the primary resistance of HCT walls to out-of-plane loading was arching action and not

bond strength. Also, bond-wrench testing proved adequate to develop the necessary knowledge of the bond strength between mortar and the unit-tile block.

K-25 Shake Table. Shake table tests were planned as a part of the HCTW Test Program. This approach was appropriate because the walls were required to resist earthquake-induced forces. Also, because most of the testing was to consist of static tests of components and subassemblies, dynamic tests were to be used to correlate with those static test results. Other parameters and issues that could not be determined by static testing but could be determined by dynamic testing were natural frequency, damping, and energy absorption; verification of analytical methods; and behavior after cracking, such as dynamic stability and rocking.

Before test plans were finalized, Clark and Bennett (1991, 1992) reviewed the literature on shake table testing of masonry structures. Citations from 18 authors were presented, covering the complete state-of-the-art of shake table testing of masonry structures. None of the work cited was directly applicable to HCT walls; however, the work gave good insight into the type of testing that should be done to satisfy the objectives of the HCTW Test Program.

After the literature review was completed, the facilities available and their capabilities for performing dynamic testing were determined (Walls et al. 1991). This information, along with the results of the literature review, could then be used to plan the dynamic-testing phase of the program.

One of the facilities to be considered was the Centrifuge Seismic Test Facility located in Building K-1600 at the former Oak Ridge Gaseous Diffusion Plant (now the K-25 Site). Design criteria and performance data for the shake table were collected and analyzed. From the outset, it was recognized that this shaker facility could not perform some of the testing that would be required; however, because of the facility's proximity to the Y-12 Plant and because it could be 100% dedicated to HCT testing, the decision was made to recommission the facility and to perform many of the tests that were within its capabilities. Information on the capacity of the shake table is given in Table F4.2 and Fig. F4.2.

Recommissioning the facility required many activities, such as returning all the utilities to service; cleaning the hydraulic power supply; cleaning the table bearing ways; purging the nitrogen distribution system; flushing the system and preparing it for normal operations; cleaning the circuit cards; powering up the shaker and performing startup testing; refurbishing the digital control system; and installing computer operating software.

Fourteen cantilever wall tests were originally planned. Seven of these were plain walls (not infilled) of different thicknesses and height-to-width ratios. Seven were infilled frames, again with different thicknesses, height-to-width ratios, and different column and beam sizes. A cantilevered wall specimen mounted in position for testing is shown in Fig. F4.3. A special table adapter extended the table width to 12 ft.

After all startup activities were completed, it was discovered that the system could not read the magnetic tapes on which the operation software was stored during the shutdown period. This finding effectively stopped the testing effort, and no tests were conducted.

Table F4.2. Shake table description

Actuator	Major features	Shake table capabilities
Modified 204.63 s (hor)	6 × 6 ft table	Frequency range 1-20 Hz
Force rating 22 kips (hor)	Specimen wt 15,000 lb	Displacement +/- 7.6 in.
Stroke: 19 in. (hor)	Reaction mass 1.6 million lb	Velocity up to +/- 12.0 in./s
Modified 204.63 s (vert)	Limiting interlocks	Acceleration up to 0.25 g
Force rating 27 kips (vert)	Over travel shock absorbers	Tilt < 0/001 rad
Stroke: 19 in. (vert)	Retaining structure	125 Horsepower
		Overturning moment 125 K-ft

Fig. F4.2. Shake table capacity envelope.

Fig. F4.3. Typical cantilevered-wall specimen on shake table.

F.5 OTHER TESTING

Other tests performed under the auspices of the test program include (1) mortar tensile splitting (2) HCT coupon tests and (3) miniature HCT prism compression tests. Results from this supplemental testing are reported by Columber (1994). Also biaxial testing was outlined in the original program plan by Bennett and Burdette (1989) but due to budget constraints was not carried out.

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Appendix G

CORRELATION OF EXPERIMENTAL AND ANALYTICAL RESULTS

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ABSTRACT

One of the goals of the Hollow Clay Tile Wall Test Program was to develop analytical approaches that could be used in evaluating the structures and facilities at the Y-12 Plant. Both in-plane and out-of-plane infill behavior needed to be addressed. To determine the most appropriate method for predicting in-plane behavior, several commonly used methods (i.e., classical approaches) were investigated. Also, piecewise linear and nonlinear methods were examined. Few analytical approaches were available for predicting the capacity of an infill when subjected to out-of-plane pressure loads or seismic drift. Resistance to out-of-plane pressure loads, such as result from inertia of the infilling due to ground motion, is addressed as a function of arching action. Capacity and behavior of infills when subjected to differential displacement resulting from seismic drifted is also examined by using a piecewise linear model.

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G.1 IN-PLANE CLASSICAL METHODS¹

G.1.1 Introduction

In support of the Hollow Clay Tile Wall (HCTW) Test Program, a literature review was performed to determine any approaches that existed for the prediction of infilled frame capacity. These methods were then evaluated as to their applicability and compared with experimental results. Any of these "classical" methods found to be good predictors of in-plane infill capacity could be used in actual building evaluations and no further analytical development would be required. Thus, the four most commonly used classical approaches are explained in this section and, later, results are compared with experimental data.

Experimental research on the in-plane capacity of infilled frames began in the early 1950s (Thomas 1953, Polyakov 1956). The first published analytical method for predicting in-plane strength on the basis of test results was by Polyakov (1960). He suggested that the infilling could be replaced with an equivalent compression diagonal (equivalent strut). Since that time, extensions of and modifications to the equivalent strut method have been proposed, and more rigorous methods (i.e., linear and nonlinear finite elements, theory of elasticity procedures, etc.) have been developed and discussed. However, as indicated by Holmes (1961) and Stafford-Smith (1969), the inherently variable nature of infilled frame construction as well as the practicality required for full building analysis dictate that the simplest (and still reasonably accurate) methods be used for predicting infill frame stiffness, behavior, and ultimate capacity. As a result, some researchers have attempted to validate and/or improve upon (on the basis of existing test data) the classical analysis methods and, in so doing, have improved the field of knowledge regarding in-plane loads on masonry infills (McBride 1984).

G.1.2 Objectives

The purpose of this analytical research was to describe and document the classical analytical approaches for predicting ultimate capacity and type of failure mechanism for in-plane loads on unreinforced masonry infilled frames. These methods are evaluated by comparing their analytical results [with unreinforced hollow clay tile (HCT) infill parameters as input] with actual test results as described in Appendix E.

G.1.3 Scope and Methodology

Four classical methods for predicting the in-plane capacity and failure mechanisms of unreinforced masonry infills have been evaluated. These include the Holmes equivalent strut method (Holmes 1961), the Stafford-Smith equivalent strut method (Stafford-Smith 1969), plastic analysis (Liauw and Kwan 1985), and the plastic penalty factor approach (Wood 1978).

Compression Strut (Holmes 1961). In his paper, Holmes presents a method for calculating the ultimate load and corresponding deflection of a steel frame infilled with concrete or brickwork. He then compares the results with data obtained from the testing of 13 infill panels

¹Portions of this section were extracted from Henderson (1994) with permission from the author.

by Wood (1958) and Benjamin and Williams (1958). Holmes postulates that the infill material may be replaced by an equivalent strut and that the lateral load to cause failure is calculated as a function of the shortening of the strut and the failure strain in the infill, as indicated in Eq. (G1.1).

$$S = \frac{(H - F \cos \phi)h^3}{24 EI} \left(1 + \frac{I}{I_0} \cot \phi\right) \cos \phi = \epsilon_m d \quad (\text{G1.1})$$

The term $(H - F \cos \phi)$ represents the lateral shear forces on the steel frame alone. By replacing the resistive infill force, F , by the product of the cross-sectional area of the equivalent strut (A) and the crushing strength of the infill material (f_m), Holmes solves for the horizontal load and deflection to cause crushing failure of the infill as shown in Eqs. (G1.2) and (G1.3), respectively.

$$H = \frac{24 EI \epsilon_m d}{h^3 \left(1 + \frac{I}{I_0} \cot \phi\right) \cos \phi} + A f_m \cos \phi \quad (\text{G1.2})$$

$$\delta_H = \epsilon_m d \cos \phi \quad (\text{G1.3})$$

where

- E = modulus of elasticity for steel,
- I = column moment of inertia,
- I_0 = beam moment of inertia,
- ϵ_m = failure infill strain,
- d = length of diagonal strut,
- h = height of steel frame,
- ϕ = angle diagonal makes with horizontal,
- A = strut area, $t * d/3$,
- f_m = crushing strength of infill.

The first portion of Eq.(G1.2) represents the lateral load resisted by the steel frame, whereas the second portion represents the load resisted by the infill material. The width of the diagonal strut used in the calculation of the cross-sectional area (A) is assumed by Holmes to be one-third of the diagonal length. He chooses this value under the assumption that the

reaction between the steel frame and the infilling varies linearly from a maximum at the loaded corners down to zero at the unloaded corners.

Compression Strut (Stafford-Smith 1969). Stafford-Smith performed much of his early research on infilled frames concurrently with Holmes, and in a general sense came to many of the same conclusions. In his first published work on infills (1962), Stafford-Smith referenced Holmes and concurred with his conclusion that the ultimate capacity of an infilled panel can be accurately predicted by replacing the infill material with an equivalent strut. However, Stafford-Smith found that the strength of materials approach used by Holmes gave poor correspondence with his own experimental results and thus felt that a more exact approach was necessary. As a result, Stafford-Smith used a two-dimensional theory-of-elasticity method to solve for the magnitude and distribution of strains and stresses in the infill continuum. The approach involved solving Airy's stress function [represented by X in Eq.(G1.4)] by applying finite difference approximations to the biharmonic equation, as shown here:

$$\frac{\partial^4 X}{\partial x^4} + \frac{\partial^4 X}{\partial y^4} + \frac{2}{\partial x^2 \partial y^2} \frac{\partial^4 X}{\partial x^2 \partial y^2} \quad (G1.4)$$

Stafford-Smith found that this method gave results nearly identical to data obtained from a series of tests on unframed mortar panels. He then used energy methods to convert the strain data into an effective strut width, as did Holmes.

Improvements to this method were suggested in two subsequent papers (Stafford-Smith 1966 Stafford-Smith, Carter 1969) in which the area of the equivalent strut was said to be a function of the infill's length of contact, α , with the surrounding frame. The contact length refers to the infill to column contact and, on the basis of his experimental work, Stafford-Smith assumed the beam-to-infill contact to be one-half of the beam length. Likewise, the contact length is dependent on the pseudo-stiffness, λ , of the frame relative to the infill material as indicated in Eqs. (G1.5) and (G1.6). Also, because the interaction between flexural members and plane stress elements of an infill are similar to the behavior of beams on an elastic foundation, λ is similar to the "characteristic" term in elastic foundation theory.

$$\alpha = \frac{\pi}{2\lambda} \quad (G1.5)$$

where

$$\lambda = \sqrt[4]{\frac{E_m t \sin 2\theta}{4E_s h}} \quad (G1.6)$$

E_m	=	infill modulus of elasticity,
t	=	thickness of the infill material,
θ	=	slope of the infill diagonal to the horizontal,
E_s	=	steel modulus of elasticity,
h'	=	height of the infill material.

Stafford-Smith and Carter (1969) indicate that the estimation of the lateral capacity of an infilled frame is determined by selecting the weakest of the various failure modes for the frame and infill. For masonry infills, these failure modes include the following:

1. local crushing of the masonry or mortar (or both) in one of the compression corners of the infill
2. tension cracking through the mortar joints and masonry and
3. shear cracking along the interface between the masonry units and mortar.

The authors suggest that compression failure occurs gradually and that the crushing area extends down the contact length, α , of the column. From a series of lateral load tests on infills (1968), they concluded that the diagonal crushing strength of the infill, R_c , may be predicted as indicated in Eq.(G1.7).

$$R_c = \frac{t f_m \pi}{2 \lambda \cos \theta} \quad (G1.7)$$

where

t	=	thickness of the infill,
f_m	=	crushing strength of the infill,
λ	=	pseudostiffness of the infill,
θ	=	slope of the infill diagonal to the horizontal.

On the basis of studies by Carter (1968), the authors concluded that the diagonal tensile strength of the infill was limited by the tensile strength of the mortar. From this the authors empirically determined the ultimate tensile strength, R_t . The authors recommend that the tensile strength, f'_t , be taken as one-tenth of the compressive strength.

Carter and Stafford-Smith indicated in previous research (1967) that the diagonal load to cause shear failure for particular length to height proportions of a panel may be represented by the following equation:

$$R_s = \frac{100 f'_{bs} h t}{8 f_{st} [1 - (\mu f'_{st}) / (f'_{st})]} \quad (G1.8)$$

f'_{bs}	=	Bond shear failure stress of the infill
μ	=	Coefficient of internal friction of the masonry
f_{st}	=	Theoretical maximum shear stress
f_{nt}	=	Theoretical maximum vertical stress.

Each of the three failure modes proposed by Stafford-Smith and Carter predict a value for the diagonal capacity of the infilled frame. The lowest of these values is presumably the controlling failure mode and is to be converted to the corresponding lateral load for failure as follows:

$$H_u = \frac{R \cdot L}{\sqrt{h^2 + L^2}} \quad (G1.9)$$

Plastic Analysis (Liauw and Kwan 1985). Liauw began experimental and analytical research on infilled frames in the early 1970s (1970, 1972). This research consisted of elastic models with isotropic and homogeneous infill material completely bonded to the surrounding frame. Later, Liauw and Kwan (1982) began investigating the nonlinear behavior of infilled frames with and without shear connectors. Their method involved using finite gap and slip elements to model the frame-to-infill interface. They found that nonlinearities arise along the beam-to-infill connection and in the compression corners for frames with and without shear connectors, respectively. The data from the nonlinear analyses were compared with experimental values. From the results of the nonlinear finite element models, Liauw and Kwan proposed separate plastic analysis methodologies for predicting the ultimate capacity of integral and nonintegral infilled frames (1983, 1984). This information was subsequently integrated to form their Unified Plastic Collapse Theory (1985), described in the following paragraphs.

Three frame-to-infill configurations -- integral, semiintegral, and nonintegral -- may be analyzed by using the plastic collapse approach, thereby accounting for both the stress redistribution towards collapse and the conditions of the infill-frame interface. Liauw and Kwan suggest that the strength of the infilled frames and the strength of the interface condition govern the collapse shear for fully integral infilled frames; the strength of the interface condition normally governs the collapse shear for semiintegral frames; and only the strength of the infilled frames is pertinent to predicting the collapse shear for nonintegral frames. The four predicted modes of failure and their corresponding collapse shear values are listed in Eqs. (G1.10) through (G1.13). The modes of failure, which are derived for frames with length greater than height, are depicted in Fig. G1.1.

MODE 1: Corner Crushing/Column Failure

$$H_u = \sqrt{\frac{2(M_{pj} + M_{pc})}{\sigma_c \cdot t \cdot h}} \cdot \sigma_c \cdot t \cdot h + S \cdot (L - \frac{h}{2}) \quad (G1.10)$$

MODE 2: Corner Crushing/Beam Failure

$$H_u = \sqrt{\frac{2(M_{pj} + M_{pb})}{\sigma_c \cdot t \cdot h}} \cdot \sigma_c \cdot t \cdot h + \frac{S \cdot h}{2} \quad (\text{G1.11})$$

MODE 3: Diagonal Crushing/Beam-Connection Failure

$$H_u = \frac{4M_{pj}}{h} \cdot \frac{\sigma_c \cdot t \cdot h}{6} + S(L - \frac{h}{2}) \quad (\text{G1.12})$$

MODE 4: Diagonal Crushing / Column-connection Failure

$$H_u = \frac{4M_{pj}}{h} \cdot \frac{\sigma_c \cdot t \cdot h}{6 \tan^2 \theta} + \frac{S \cdot h}{2 \tan \theta} \quad (\text{G1.13})$$

where

H_u	=	collapse shear load,
M_{pj}	=	smaller of the beam or column plastic moment,
M_{pc}	=	column plastic moment,
M_{pb}	=	beam plastic moment,
σ_c	=	crushing stress of panel material,
t	=	thickness of infilled panel,
h	=	height of infilled panel,
S	=	shear strength of interface connection.

All terms in the preceding equations are applicable for frames that are considered fullyintegral; only the shear terms are considered for frames that are semiintegral; and all shear terms are neglected for nonintegral infilled frames. Likewise, according to Liauw and Kwan, the fourth mode (i.e., diagonal crushing/beam connection failure) is not applicable to nonintegral infills.

Plastic Penalty Factor (Wood 1978). Wood (1978) indicated that, although Stafford-Smith's equivalent strut was a worthwhile effort, he felt that no theory of elasticity method was capable of accurately predicting the various collapse modes resulting from racking forces on infills. Thus, he began research into plasticity theory relative to infills and felt that Johansen's square yield line criterion could be modified to include membranes with axial forces only. From this approach, he concluded that infilled behavior is governed by four failure modes: (1) shear, (2) shear rotation, (3) diagonal compression, and (4) corner crushing. Wood presents a postulated penalty factor that can be used with general plasticity analysis of infills. Wood's premise is that a penalty factor is required to account for the nonideal plasticity of the infill material that results as applied loads and displacements are increased on the frame.

Fig. G1.1. Plastic analysis failure modes.

The penalty factor, γ_p , is an empirical value that reduces the ultimate crushing stress, σ_c , and is based strictly on test results by Wood and others before 1977. Some authors (McBride 1984 Dawe and McBride 1985) have found that Wood's penalty factor in conjunction with the plastic collapse theory of Liauw and Kwan provides an accurate means of predicting the ultimate load on infilled frames. In fact, the authors found that though the plastic collapse method alone overpredicted their experimental results by more than 60%, use of the penalty factor provided results equal to the average of the experimental in-plane capacities.

G.1.3 Results and Conclusions

All four of these classical methods were compared with the full-scale test results described in Sect. E.3 of this document. Detailed explanations of the comparative calculations may be found in (Henderson 1994) for the quasi-static tests of Y-12 replicas at Iowa State University (ISU). The Holmes compression strut and plastic analysis with a plastic penalty factor were compared with the test results described in Sect. E.2 of this document. A simplification of Holmes compression strut is also compared with these moderate-scale test results. Detailed explanations of the comparative calculations may be found in Flanagan (1994) for the quasi-static tests of infills at the University of Tennessee (UT). A synopsis of the general findings is presented in the following paragraphs.

When comparing the classical methods with the full-scale tests of Y-12 replicas, infill crushing strain and a net compressive strength parallel to the cores were determined from the prism compression testing at the National Institute for Standards and Technology (see Appendix D). Also, the thickness used in calculating the area of the compression strut was taken to be the thickness of the three structural clay tile webs resisting bearing by the steel frame rather than the full wall thickness.

For comparison with the moderate-scale tests performed at UT, ultimate strain was taken as that measured along the diagonal of the infilled test specimen. Net compressive strength values parallel to the core were determined from testing of prisms at UT. Finally, the thickness of the infill was taken as the portion enclosed in the surrounding steel frame. Note that all of these methods are a function of the compressive strength of the masonry and the thickness of the infill. Because of the nonhomogeneous, orthotropic nature of the infill material, values for masonry strength, f'_m , and thickness, t , may vary depending on the analyst and the method used.

Compression Strut (Holmes 1961). Holmes' method for the calculation of ultimate lateral load and deflection was used in conjunction with the geometric and material property parameters of the infills tested at ISU and UT. On the basis of this input, the predicted lateral load and deflection to cause failure of the infill greatly exceeds the actual values experienced during testing. This overprediction is on the order of approximately 3 times for the moderate-scale tests (UT) and 12 times for the full-scale tests (ISU). As determined by McBride (1984), one of the inadequacies of the Holmes strut is that it accounts for only one mode of failure (i.e., compression failure of the infill). However, because the infills tested in this research failed in compression, this particular inadequacy should not affect the analytical results. Part of the overprediction is evidently due to the assumption that contact stresses degrade linearly over the full length of the columns and beams. This results in an extremely long strut width.

For infilled frames with large planar dimensions and small frame rigidities, the contact stresses degrade much more quickly than the linear assumption indicates. It is clear from the testing that the width of the compression area was much less than that assumed by Holmes. This resulted from the small lateral rigidity of the frame compared with that of the infill material. Thus, the use of the Holmes strut may be inappropriate for large infills where the contribution to lateral rigidity by the steel frame is comparably small.

Similarly, for unreinforced hollow masonry units, determining the appropriate thickness (for area calculations) and crushing strength of the infill is difficult. The Holmes' strut actually resists the resulting vector caused by the horizontal shear force, H (typically a wind or seismic load). As such, the strength and resistivity of the infill is a function of the orthotropic geometric and material properties of the hollow blocks. This is particularly true for infills whose cores run horizontally, as in this research. Thus, the Holmes method may be more appropriate for homogeneous, and generally isotropic, infill material.

Compression Strut (Stafford-Smith 1969). For the full-scale testing of Y-12 replicas, the Stafford-Smith approach was used without modification. As when comparing the Holmes method results with the results from the testing of Y-12 replicas, the thickness used in calculating the compression and tensile capacity was taken to be the thickness of the three structural clay tile webs resisting bearing by the steel frame -- rather than the full wall thickness. However, for shear failure, the full wall thickness was used in that the continuous bed joint would be resisting the lateral shear forces. The friction coefficient for shear calculations was assumed to be 0.6 and the aspect ratio was taken to be 1 to 1. These estimates should be very close to the actual values, and slight discrepancies should not significantly affect the results of this equivalent strut method.

The maximum load achieved by the undamaged infill was 61 kips and the mode of failure was diagonal tension followed later by corner crushing. However, the Stafford-Smith method predicted compression, shear, and tensile failure to be 212, 76, and 241 kips, respectively. Though the 76-kip value is relatively close to the actual value observed, shear failure was not the dominant failure mode. Stafford-Smith's elasticity approach to the calculation of the contact length and ultimately the area of the compression strut appears to be more representative of the observed experimental results than was the Holmes method. However, compression and tensile capacity parameters (i.e., f'_m and t) are difficult to assess for nonisotropic, nonhomogeneous materials such as those commonly used in infilled construction. For instance, because the strut force is diagonal -- with substantial vertical and horizontal components -- the strength and thickness in both directions must be considered when estimating the compressive or diagonal tensile capacity of the infill. Furthermore, it is not clear whether the non-dimensional parametric approach for calculating shear and tensile capacity is applicable for materials and configurations other than those used in the Stafford-Smith research.

A simplification of the Holmes equation in conjunction with the Stafford-Smith approach was used by Flanagan (1994) to predict ultimate capacity. As shown in the following equation, contribution of the frame resistivity has been modified and the area reduced by a factor of two from the original equation. The use of this approach produces results within a range of 70% to 185% of the actual test data.

$$P_n = \frac{6EI}{h^3} \delta + A f'_n \cos \theta \quad (G1.14)$$

Plastic Analysis (Liauw and Kwan, 1985). The plastic analysis approach (without a penalty factor) was evaluated only for the full-scale Y-12 replica tests. All terms under Plastic Analysis of Sect. G.1 are applicable for frames that are considered fully integral; only the shear terms are considered for frames that are semiintegral; and all shear terms are neglected for nonintegral infilled frames. Likewise, according to Liauw and Kwan, the fourth mode (i.e., diagonal crushing/beam-connection failure) is not applicable to nonintegral infills. The method for the calculation of ultimate lateral load (described previously) was used in conjunction with the best estimates of geometric and material properties for the Y-12 replica tests performed at ISU. During the in-plane testing, significant differential movement of the structural steel frame relative to the infilling was noticed early in the test process. Hence, the infill is considered to be nonintegral and the shear strength, S , is set equal to zero.

The plastic analysis method predicted the correct mode of failure (i.e., corner crushing with column failure); however, the magnitude of the predicted ultimate shear load (103 kips) was in excess of the actual in-plane value for the infill with no prior damage (61 kips) by almost 70%. As with the two previous methods, the orthotropic properties of the infill material make determining the appropriate compressive strength and thickness for input into the equations difficult. If the inception of compression failure were assumed to be instigated by the vertical component of the diagonal force, then the compressive strength value normal to the cores (1.5 ksi) from the prism compression testing would be used in the calculation; however, overprediction of the panel's ultimate racking strength would still result.

Plastic Penalty Factor (Wood 1978). Wood's plastic penalty factor, in conjunction with the plasticity method of Liauw and Kwan, was compared with the experimental in-plane capacity data for both the full-scale Y-12 replica tests at ISU and the moderate-scale infill testing at UT. The magnitude of the dimensionless penalty factor, γ_p , is plotted as a function of m , a nondimensional ratio representing frame-to-wall strength. For walls with strong frames controlled by shear failure modes in the infilling, the penalty factor is governed by a 0.45 limit. However, for walls with very small frame strength as compared with the in-plane contribution of the infill material (as is the case for the research described herein) the penalty factor is governed by a best fit quadratic equation representing empirical data by Wood and others. It is important to note Wood's suggestion that non-ideal plasticity will always result in a reduction of the ultimate capacity by at least 55% (i.e., $\gamma_p \leq 0.45$) of the ideal value.

Because of the weak frame and strong infill of the full-scale Y-12 replicas, m was found to be very small (i.e., well less than 0.2) and, thus, γ_p was governed by the best-fit quadratic equation in the region of diagonal compression and corner crushing, as expected. With a penalty factor, $\gamma_p = 0.394$, the predicted failure load due to corner crushing is reduced from 103 kips [using Mode 1 (Liauw and Kwan 1985)] to 64 kips. This value is within approximately 5% of the result obtained from testing of the undamaged infill.

The plastic analysis approach in conjunction with a penalty factor correlated reasonably well with the experimental values for the moderate-scale specimens. Typically, predictions for the shorter specimens were controlled by the third, diagonal crushing mode equation. The use of this approach produces results within a range of 54% to 200% of the actual test data.

G.2 PIECEWISE LINEAR IN-PLANE STRUT FORMULATION²

G.2.1 Introduction

As the HCTW Test Program began to conclude in the spring of 1993, evaluations of some of the Y-12 Plant critical facilities began. In response to the Safety Analysis Report Upgrade Program, a methodology was formulated to address the behavior of the infill material within the steel frames. This methodology was actually a set of preliminary recommendations on the basis of the test data available at the time. The recommendations were issued in a document titled *Appendix A: Recommendations for Clay Tile Infills* and were intended to supplement the knowledge base of clay tile behavior from an analytical standpoint — particularly with respect to stiffness and strength. An approximation to the Stafford-Smith equation presented in Sect. G.1 of this document was used in the approach. This method was a piecewise linear approach in that the contact length term, α , varied with hysteretic damage to the infill.

G.2.2 Objective

The objective of the piecewise linear strut was to provide those who were analyzing buildings at the Y-12 plant with a procedure that accounted for the stiffness of the infilling and gaged the remaining strength on the basis of hysteretic test results.

G.2.3 Scope and Methodology

One of the first considerations of any facility evaluation is the establishment of proper structural performance levels. These recommendations are on the basis of the acceptance of damage levels that may impair architectural or serviceability criteria but do not induce gross instability of the infill.

The *Appendix A* methodology assumes that an equivalent static or response spectrum dynamic analysis is used in the structural evaluation. This provides reasonable computational efficiency if used with a simple infill representation.

Considering combined lateral forces acting on an infill, it is recognized that some level of interaction between in-plane and out-of-plane behavior occurs. However, because the test data indicated that many of the structural mechanisms are somewhat independent, traditional

²Portions of this section were extracted from Flanagan (1994) with permission from the author.

methods of treating the infill response in orthogonal directions separately thus appear to be adequate.

G.2.4 Results and Conclusions

It has been concluded from testing that very little interaction occurs between in-plane and out-of-plane forces. This conclusion leads to the recommendation that an equivalent diagonal compression strut bracing the framing be used for the analysis methodology.

It is further concluded that whether two- or three-dimensional building analysis is used, the HCTW infills may be idealized as two dimensional in-plane components. Although several in-plane approximations might be used, the method recommended is an equivalent diagonal compression strut bracing the framing.

The conditions required for a compression strut to develop must be present. Openings, interface gaps, and other discontinuities may affect the infill behavior. Attention to detail in modeling the compression strut is also important. For instance, the use of a nonsymmetric (single) strut may result in erroneous axial forces at reentrant corners of a building. If a "compression only" element is not available for an equivalent strut, the equivalent strut stiffness may be divided equally between two struts bracing the frame in opposite directions.

It should be recognized that equivalent strut methods have been formulated without consideration of vertical forces. Thus, vertical loads should be omitted during the lateral load analysis and superimposed later.

The length of column bearing on the infill is given by

$$\alpha = \frac{\pi}{C\lambda} \quad (G2.1)$$

where:

$$\lambda = \sqrt{\frac{E_m t \sin 2\theta}{4E_s I_{col} h'}} \quad (G2.2)$$

in which E_m is the gross elastic modulus of the masonry (ksi), $E_s I_{col}$ is the flexural rigidity of the column (kip in²), θ is the angle of the infill diagonal to the horizontal, t is the infill gross thickness (inches), and h' is the infill height (inches). Fricke (1995) recommends that gross section compressive strengths of 735 and 495 ksi be used for 8-in. and 13-in. prisms, respectively. These values are determined as the geometric mean of the orthotropic prism compression strengths and are discussed in further detail in Appendix I.

The width of the equivalent strut is then determined by

$$W = \frac{\alpha}{\cos \theta} \quad (G2.3)$$

and the area of the equivalent strut by

$$a = wt. \quad (G2.4)$$

The length of contact (α) should be limited to 20% of the panel height (h'). Values of C vary with the in-plane drift displacement and are given in Table G2.1.

For very stiff columns, this strut formulation predicts significantly high contact lengths. This leads to an ultimate capacity prediction at smaller displacement levels that is substantiated in the experimental results. For example, in specimen 9 of the UT moderate-scale tests (See Sect. E.2), the limit states of off-diagonal cracking and banded-diagonal cracking were barely observed before corner crushing and tile cracking occurred. It is suggested that the length of contact, α , be limited to 15-20% of the panel height. Similarly, on the basis of both the moderate- and full-scale tests an upper limit of 50 kips for 8-in. infills and 60 kips for 13-in. infills may be appropriate.

Table G2.1. Typical values of C for varying in-plane drift displacements

C	Displacement (in)	Typical infill damage
5	0.0 - 0.05	None
7	0.05 - 0.2	Diagonal mortar joint cracking
11	0.2 - 0.4	Off diagonal mortar joint cracking
14	0.4 - 0.6	Banded diagonal mortar joint cracking
16	0.6 - 0.8	Corner mortar crushing and tile cracking
18	0.8 - 1.0	Tile faceshell splitting (primarily corner regions)
-	1.5 x Ult.Disp	Tile faceshell spalling

Ultimate capacity of the clay tile infill should be limited by a 1-in. plane drift displacement (e.g., 0.0042 h' for $h' = 20$ ft). An upper limit of 50 kips should also be imposed on the peak capacity of the infill (maximum equivalent strut force). Beyond peak capacity, the strength of the infill may be assumed to reduce to 75% of peak at an in-plane drift of 1.5 times the displacement at peak or 1.0-in., whichever is greater.

Figure G2.1 is used to illustrate this piecewiselinear compressive strut method as it compares with test specimen 17 of the moderate-scale testing conducted at UT. These results compare well with the envelope of observed in-plane hysteretic behavior. The masonry elastic modulus parallel to the tile cores is used in the computation. Other in-plane tests have also compared reasonably well with this piecewise linear analysis method.

Fig G2.1 Infilled frame 17 piecewise linear strut analysis.

G.3 NONLINEAR IN-PLANE MODELING³

G.3.1 Introduction

Results of the in-plane testing were compared with a compressive strut formulation in which the stiffness (modulus) of the infill varies and a nonlinear analysis results. The finite element program ABAQUS was used to carry out these analyses. The steel frame was modeled by using two-dimensional plane stress elements with elastic-plastic properties on the basis of the yield stresses given in the mill test reports. Base and beam-column connections were modeled with rotational springs having properties approximated by the methodology of Frye and Morris (1975) and Kishi and Chen (1986) and refined to match the results of in-plane bare frame tests. The infill was replaced by two compression-only diagonal struts having an arbitrary stress-strain relationship. A plasticity material model was used to incorporate isotropic strain hardening and softening of the compression struts (ABAQUS 1993).

G.3.2 Objective

The intent of this analytical work was to investigate the effect of plasticity and degrading infill stiffness as well as to formulate an accurate approach for modeling nonlinear behavior. Also, a comparison of the accuracy of stiff and flexible nonlinear struts in reproducing hysteretic masonry infill behavior was desired.

³Portions of this section were extracted from Flanagan (1994) with permission from the author.

G.3.3 Scope and Methodology

Two stress-strain relationships representing a stiffer and more flexible infill were used in the analyses, as indicated in Fig. G3.1. The stiffer compressive strut properties were derived from average data on the 8-in. prism specimens (cores parallel to the load) tested by Boyd (1993). The areas of the compressive struts were derived from the known capacities of the tests and the average ultimate compressive strength of the prisms. The capacity prediction equations previously discussed in Sect. G.2 might well have been used in lieu of actual capacities.

G.3.4 Results and Conclusions

The more flexible compressive strut properties incorporate a reduction of the prism stiffness. The column top displacements are about $\sqrt{2}$ times greater than the panel compressive displacements. The local interface deformations are $\sqrt{2}$ times panel shortening. To account for this in an average sense, the prism strains were multiplied by $(1 + \sqrt{2})$ to produce the more flexible properties.

Both compressive strut properties were used to approximate the degrading stiffness of the infill. The results of the two analyses are compared in Fig. G3.2 with portions of the actual test hysteresis of specimen 17 (See Sect. E.2). Six hysteretic loops representative of the infill behavior states were used as input to these analyses. The model with stiffer infill properties overestimates the stiffness of the combined infilled frame system. However, the model with the more flexible infill properties does a reasonable job of approximating the hysteretic behavior, particularly during the push (negative load) cycles of the test. From the in-plane tests, either interface softening early in the loading and plastic strain or crushing of the masonry joints later in the loading are believed to account for much of the degrading stiffness.

Fig. G3.1. Nonlinear Compressive Strut Properties.

Fig. G3.2. Comparison of nonlinear strut analysis with specimen 17.

G.4 OUT-OF-PLANE PRESSURE LOADS⁴

G.4.1 Introduction

Until recently, much of the research that has been completed on the resistance of masonry to out-of-plane loading has been performed on concrete and brick masonry. A comprehensive bibliography has been compiled of this research (Bennett and Flanagan 1992). However, very little of this research has been incorporated into the building codes that specify requirements on the basis of beam or plate bending. The previously cited literature indicates that most of the resistance of masonry to out-of-plane loading is due to arching action. Therefore, the HCTW Test Program addressed the analytical modeling of the arching and compared it with test results.

G.4.2 Objective

The objective of the out-of-plane pressure load testing of infilled HCT walls was to calibrate and verify the analytical models for out-of-plane cracking and ultimate load capacity of these walls.

G.4.3 Scope and Methodology

The scope of this work was to develop analytical models to predict the first cracking and ultimate loads of two HCT infilled wall panels, one located in Building 9207 and one in Building 9212 at the Oak Ridge Y-12 Plant. These models were to be checked first by comparison with results from existing tests of concrete masonry and then by comparison with actual in situ test results for the identified panels. Due to the difficulty of testing in Building 9212 this wall panel was never tested.

⁴Portions of this section were extracted from Chua (1991) with permission from the author.

Various methods are available for the prediction of the out-of-plane capacity of infill walls. These range in complexity from simple beam theory to rigorous mathematical treatments of the "arching action" mechanism. Arching action refers to the development of in-plane thrust loads when a specimen of significant thickness is subjected to out-of-plane pressure loads. Simple beam models treat the wall as a flexural member, spanning either horizontally or vertically. Bending stresses are computed and compared with some multiple of code-based allowable stresses to obtain an estimate of ultimate capacity. Some of the earlier evaluations of existing buildings at Y-12 relied on this method.

Ultimate plate theory has also been proposed as a method for analysis. Seah (1988) developed yield-line formulas and compared predictions from these formulas with experimental results. The formulas consistently underestimated actual capacities; the ratio of theoretical to experimental capacities ranged from 0.35 to 0.99. Seah also developed a model that included the effects of arching action. The model was shown to be a good predictor of experimental capacity. Theoretical results ranged from 0.94 to 1.14 times experimental values. To simplify the analysis, Dawe and Seah (1989), also proposed an empirical design formula that includes the effects of arching action. The primary parameters in the formula are the wall panel dimensions and the stiffness properties of the surrounding frame.

Chua (1991) used an orthotropic linear elastic material model to predict first cracking loads for the wall panels in Buildings 9207 and 9212. Shell elements that included membrane and transverse shear deformations were used for the masonry panel. Because the wall panel in Building 9212 was never tested, only the results for the wall panel in Building 9207 are discussed here.

The wall panel was 28 ft wide and 12 ft high (floor to top of wall) and consisted mostly of single wythe unreinforced construction. The panel was made from HCT units, 12 x 12 x 8 in. thick (wall thickness) with the cores running horizontally. A running bond construction was used. The panel was located on the ground floor of Building 9207 along N-S column line D between column lines 2 and 3. The wall was infilled between W14x142 columns with a W30x108 overhead beam and concrete floor slab beneath. A description of the infilled panel and related testing is provided in Appendix E of this document.

Chua found that very little information exists for material properties for HCT panels or assemblages. For the analysis, he needed the modulus of elasticity in both directions, Poisson's ratio, shear modulus, modulus of rupture in both directions, and a failure criterion. For the material properties he used Plummer (1962) and Johnson and Matthys (1973). For the failure criterion he used the elliptical interaction method of Thurlimann and Guggisberg (1988), which he found to give closer results to existing experimental data for concrete masonry. Next Chua chose three boundary conditions for the wall panel. Case 3, which is the only one reported here, appears to be the most reasonable. The boundary condition for this case was simple supports at the top and sides and supports fixed at the base. Chua also ran case 3 with the base simply supported. From these analyses he predicted a uniform pressure on the wall to cause cracking to be 0.26 psi at the base for the fixed base case and 0.29 psi for center panel cracking for the case in which the base is simply supported.

Chua also used Seah's arching action model and the empirical formula developed by Dawe and Seah to estimate the ultimate load capacity of the wall. To do this, Chua considered four modes of failure, A-D (Fig. G4.1), and performed a parametric study to determine the importance of various parameters. He found that the only important material property parameter was the compressive strength of the masonry. The ultimate minimum pressure loads for the various failure modes were reported as 2.15 psi for Mode A; none computed for Mode B; 0.419 psi for Mode C; and 1.012 psi for Mode D. Chua decided that the most likely failure mode was Mode A. The deflected shape for Mode A is shown in Fig. G4.2. Chua's analyses were performed prior to the actual test and the results were used to develop various aspects of the airbag test, such as the design of the reaction frame for supporting the airbag pressures.

Fig. G4.1. Modes of failure.

G.4.4 Results and Conclusions

A summary of the test results for the wall panel in Building 9207 is given in Appendix E. This summary was taken from the work reported by Huff et al. (1992).

Fig. G4.2. Predicted pressure vs Failure for wall panel in Building 9207.

Out-of-Plane Cracking. In the final test report it was stated that the first sound of audible cracking was heard 40 min into the test at a pressure load of 0.28 psi. This value is in close agreement with Chua's calculation of 0.26 psi when the wall was considered fixed at its base or 0.29 psi when the wall was considered simply supported at its base. Because of the keying at the panel-to-floor interface, the test structure was more nearly fixed at the base. Thus it appears that a linear elastic finite element model can predict first cracking of an HCT wall panel.

Ultimate Load Capacity. It appears that predicting the ultimate load capacity is much more difficult than predicting the first cracking load. As stated previously, Chua pointed out that there are many variables, all of which affect the ultimate load capacity. All of these variables

cannot be known about an existing wall configuration. Good engineering judgement is required to make the proper assumptions. As Chua showed, the assumed failure mode is an important criterion. The 2.15 psi that Chua predicted was on the basis of a two-way arching action. He pointed out that if a one-way arch is considered (failure Mode D), then the ultimate load capacity would drop to 1.012 psi, which is a little less than half of Failure Mode A. This value is in very close agreement with the actual load capacity of about 0.88 psi (scaled due to airbag size) as reported by Huff.

Huff also made a simplified one-way arch model that predicted about the same result. Chua used 1000 psi for compressive strength and 1500 ksi for the elastic modulus, whereas Huff used 725 psi and 3000 ksi for the two constants, respectively. Chua did a parametric study to examine the effect of changing these two constants. Using this study to adjust Chua's ultimate pressure load to the values Huff used indicates Chua was 0.25 psi higher than Huff due to compressive strength but 0.285 psi lower than Huff due to the difference in the elastic modulus used. Applying these two corrections factors to Chua's work for Case D indicates he was low by only 0.035 psi, which leads to a corrected value of 1.047 psi for the ultimate pressure load. This correction is insignificant considering the number of other assumptions made in the analyses.

Huff considered only the results from Chua's two-way arch model and made no reference to Chua's one-way arch model. In his comparisons Huff gave four reasons why Chua's values for two-way arching were probably incorrect.

1. The analytical model Chua employed assumed no stiffness contribution from the second-floor slab.
2. The analytical model had no provision for the inclusion of normal stress effect.
3. The analytical model required the input of wall properties, specifically the compressive strength and elastic modulus of the wall. The benefit of prism test results was not available for input to the model and values had to be assumed.
4. Chua's model made no provision for the fact that the test wall was offset from the center line of the overhead beam and columns.

Of these four reasons only reasons two and four apply to Chua's one-way arch model. Chua's work was never reevaluated on the basis of better known material values and boundary conditions as provided by correlative small-scale testing (e.g., prism compression, bondwrench, etc.). More work needs to be done before sound judgements on the proper parameters that should be used in conjunction with Chua's methodology can be made. It can be concluded, however, that even with very conservative assumptions applied to either of the arching models, predicted capacities offer much higher resistance to seismic and wind loads than conventional (code-based) techniques.

G.5 OUT-OF-PLANE DRIFT LOADS⁵

G.5.1 Introduction

Analyses have indicated that out-of-plane damage to many of the structures at the Y-12 plant as induced by ground motion would generally be a function of seismic drift as opposed to inertial effects on the infill material itself. This is particularly true for the wings of Building 9212 due to the low frequency of the structure and the decoupled frequency of the infilling. A typical wing bay is shown in Fig. G5.1. Because seismic drift is the predominant loading mechanism, the full-scale Y-12 replica test specimens were loaded in a displacement-controlled fashion by actuators at the top and bottom chord locations shown in Fig. G5.2. Similarly, these test structures were also analyzed by using a piecewise linear method for predicting stiffness, capacity, and cracking patterns resulting from applied out-of-plane drift. The framing members were incorporated into the model by supplying the appropriate section and material properties to space frame elements. Because of the composite nature of the infill construction, the infill was modeled as a continuum of discretized finite elements. The results of this out-of-plane analytical approach were compared with experimental data retrieved from the quasi-static testing of Y-12 replicas described in Sect. E.3.

Fig. G5.1. Typical wing bay of Building 9212.

⁵Portions of this section were extracted from Henderson (1994) and Henderson et al (1994) with permission from the authors.

G.5.2 Objective

The objective for this analytical research was to reproduce both the experimental behavior and ultimate capacity of an infill subjected to out-of-plane seismic drift by performing an uncomplicated piecewise linear analysis. The primary drift loads of consequence for the structures at the Y-12 plant occur due to the loads applied by the top and bottom chords of roof trusses framing in at the column locations, as shown in Fig. G5.1. The intent was, thus, to replicate the experimental loading (that is, equal displacements applied at four actuator locations) by: (1) applying equal and increasing displacements at the translational support locations shown in G5.2, (2) monitoring the crack damage caused by the applied displacements, and (3) concluding and remodeling when an appropriate interim limit state was achieved.

Fig. G5.2. Overview of the out-of-plane modeling.

G.5.3 Scope and Methodology

If performed in a step-by-step linear method, the procedure described in Sect. G.5.2 would be a very lengthy and tedious process because the loads and displacements that cause

the limit state are unknown; hence, each model would require numerous iterations (varying the applied displacement) until a final crack pattern was achieved. However, because each stiffness analysis produces linear results (e.g., doubling the applied displacement exactly doubles the stresses at a given location), one analysis, with any arbitrary applied displacement, may be performed and the results factored up or down as a function of some assumed limit on a given output parameter (e.g., bedjoint cracking stress, S_{yy}). A summary of the analysis process is shown in Fig. G5.3. Thus, to determine the displacement and load on the structure when cracking occurred at any joint location, the arbitrary applied displacement (input value) and the loads at the translational supports (output value) were factored by the assumed rupture stress divided by the actual vertical stress, as follows:

$$\Delta_{cracking} = \frac{S_{yy}(cracking)}{S_{yy}(actual)} \Delta_{applied} = \frac{70 psi}{S_{yy}(actual)} \Delta_{applied} \quad (G5.1)$$

$$P_{cracking} = \frac{S_{yy}(cracking)}{S_{yy}(actual)} P_{applied} = \frac{70 psi}{S_{yy}(actual)} P_{applied} \quad (G5.2)$$

Fig. G5.3. Flowchart of the out-of-plane analysis.

Thus, because of the linearity of the analysis, the applied displacement and resulting forces at the actuator locations may be factored by the assumed rupture stress divided by the actual vertical stress at any node location. This results in the applied displacement and force required to cause a crack at that particular node. Also, because of the linearity of the analysis, it is known that the node with the highest vertical stress will crack first and so on. Thus, the order of the cracking in the model is determinable by sorting all of the output (node number, S_{xx} , S_{yy} , S_{xy}) from highest to lowest vertical stress (S_{yy}).

To make the most use of this process, a BASIC computer program was written to (1) read and sort the output file as a function of S_{yy} stresses, (2) read the applied displacement and load and factor them, on the basis of the assumed rupture stress, each time a crack was formed, and (3) plot the structure and graphically depicted the formation of cracks at the correct location.

G.5.4 Results and Conclusions

A comparison between analytical and maximum experimental load/deflection data at the lower actuator locations is provided in Fig. G5.4. Crack patterns for the piecewise linear analyses are shown in Fig. G5.5. Several modifications to the analytical technique were incorporated on the basis of information gained from the experimental testing.

Fig. G5.4. Experimental vs analytical plot of load vs deflection.

The first trial incorporated only the infill and framing members while neglecting the stiffness of the pilaster and rotational rigidity of the column-to-floor connections. The rupture stress was assumed to be 70 psi. The second preliminary trial incorporated the pilaster stiffness by increasing the thickness of the end elements. As shown, trials three and four are the best predictors of the actual test behavior. These models, performed after the out-of-plane testing, incorporated the column-to-floor rotational stiffness ascertained from the out-of-plane testing of the bare frame. This value was shown to be approximately 650 kip-inches/degree. Intermediate points on the load deflection curve are followed quite closely; however, some over-prediction of the ending tensile capacity did occur. This method, which is based on the linearity of stiffness analysis output, predicted the experimental load/deflection results within 15% and, after modifications resulting from the out-of-plane testing, reproduced the out-of-plane behavior within 5%.

Fig. G5.5. Cracking of the piecewise linear models.

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Appendix H

FINDINGS AND RECOMMENDATIONS FROM EXPERT REVIEW COMMITTEE AND NATIONAL WORKSHOP

10

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ABSTRACT

A review committee of national masonry experts was assembled to provide advice on the development and conduct of the Hollow Clay Tile Wall Test Program. The committee effectively assisted the test team in developing the program's original plan and also provided advice and guidance throughout the program. Five program reviews were conducted during the course of the test program. After each, the committee prepared a report containing program recommendations and findings that included adding new tests, eliminating tests, and ranking the importance of tests on the basis of timing and funding constraints. This interaction with the test team was very effective both in planning and implementing the testing as well as drawing conclusions from the large array of data.

A National Workshop on Unreinforced Hollow Clay Tile sponsored by the U.S. Department of Energy (DOE) in September 1992 provided a forum for discussion of hollow clay tile issues. The main objective of the workshop was to bring together researchers and practitioners to begin a dialogue on test methods, structural behavior, and future research needed to develop seismic evaluations of buildings constructed of unreinforced hollow clay tile that would adequately (as judged by national consensus) address the safety issue. The specific goals and objectives of the workshop are described in this appendix.

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Appendix H

FINDINGS AND RECOMMENDATIONS FROM EXPERT REVIEW COMMITTEE AND NATIONAL WORKSHOP

A review committee of national masonry experts was assembled in 1989 to provide advice on the development and conduct of the Hollow Clay Tile Wall (HCTW) Test Program, and a National Workshop on Unreinforced Hollow Clay Tile sponsored in 1992 by the U.S. Department of Energy (DOE) provided a forum for discussion of hollow clay tile (HCT) issues. Findings and recommendations from the committee and the workshop are discussed in this appendix.

H.1 EXPERT REVIEW COMMITTEE¹

H.1.1 Introduction

The goal of the HCTW Test Program was to sufficiently understand the behavior of HCT infills such that these infills could be accurately and efficiently modeled in generalized building analyses of Y-12 structures. As requested by Martin Marietta Energy Systems, Inc. (MMES), the Civil Engineering Department of the University of Tennessee, Knoxville (UT) developed a suite of tests that could be used in the HCTW evaluation process. A preliminary report from UT was issued in December 1989.

The Expert Review Committee (ERC) consisted of the participants shown in Table H1.1. The consultants met with MMES and UT personnel in Oak Ridge on September 29, 1989, to become familiar with the program objectives. Subsequently, the consultants were asked to review the final UT report and comment upon the overall plans and tests.

Table H1.1. Expert review committee members

Member	Affiliation	Location
Dr. James K. Noland	Atkinson Noland Associates	Boulder, Colorado
Dr. Daniel P. Abrams	University of Illinois	Urbana, Illinois
Dr. Samy A. Adham	Agbabian Associates	Pasadena, California
John Kariotis	Kariotis & Associates	Pasadena, California

In May 1990 it was suggested that the ERC members meet to reconcile any differences in their recommendations and to consider any new information and ideas from MMES. John Kariotis

¹Portions of this section were extracted from Abrams et al. (September 1989, 1990; August 1990, 1991, 1993) with permission from the authors.

(Kariotis & Associates, Pasadena, California) was then added to the consultant group, which met in Oak Ridge on August 7 and 8, 1989, to receive current information from MMES regarding the HCTW Test Program, and formulate the consultants' consensus recommendations regarding the HCTW Test Program.

In November 1991, during the intermediate stages of the in situ and laboratory testing, an ERC workshop was held in Oak Ridge. Presentations were given by the MMES principal testing investigators to the ERC so that interim guidance could be provided and modifications implemented if necessary. Also, at a final review workshop held in April 1993 in Oak Ridge, the tests completed to date were reviewed and guidance was provided as to the completion and implementation of the remaining testing.

H.1.2 Objectives

The ERC was convened to provide guidance in the formulation of testing and the application of test results.

H.1.3 Scope and Methodology

The ERC submitted essentially five reports to Energy Systems throughout the course of the test program. Four of the five reports were in response to workshop presentations to the committee by Energy Systems for the purpose of updating the ERC on progress to date. One of the five reports provided review comments on the test program document developed by UT. The documents submitted by the ERC are summarized in Table H1.2.

Table H1.2. Documents submitted by Expert Review Committee

Rpt	Date	Title	Purpose
1	9/89	"Expert Review Committee Comments on September 29, 1989, Meeting" Authors: Abrams, Adham, Noland	Provide guidance on start-up of the Hollow Clay Tile Test Program.
2	4/90	"Expert Review Committee Peer Review of Final Report on Structural Behavior of Clay Tile Walls Proposed Evaluation Program" Authors: Abrams, Adham, Dawe, Noland	Review The University of Tennessee Program document on Hollow Clay Tile testing.
3	9/90	"Expert Review Committee Comments on April 7 & 8, 1990, Workshop" Authors: Abrams, Adham, Kariotis, Noland	Summarize work up to that point. Recommend future course.
4	1/93	"Expert Review Committee Comments on November 21 & 22, 1991, Workshop" Authors: Abrams, Adham, Kariotis, Noland	Answer fifteen questions proposed by Martin Marietta Energy Systems. Provide impressions and recommendations for the program.
5	4/94	"Expert Review Committee Comments on April 13 & 14, 1993 Workshop" Authors: Abrams, Adham, Kariotis, Noland	Address areas of concern posed by the review committee and by Martin Marietta Energy Systems.

H.1.4 Results and Conclusions

Reports 1 & 2. Reports 1 and 2 were compilations of responses from each of the four consultants individually regarding tests applicable to achieving the goals of the test program. Recommendations as to specific tests that should be performed and the method by which these results should be interpreted from an in-plane and out-of-plane behavioral standpoint were provided in report 1. The recommended tests in the *Final Report on Structural Behavior of Clay Tile Walls Proposed Evaluation Program* (Bennett and Burdette 1989) submitted by UT were reviewed in report 2. As for report 1, each of the consultants provided individual comments and therefore overall consensus conclusions were not achieved. It was generally agreed, however, that the Lockwood Greene Analysis (Lockwood Greene September 25, 1989) produced lower bound values for capacity. Further, it was felt that UT had produced a thorough report and the recommended tests were, generally, appropriate.

Report 3. In report 3, the consultants presented a consensus view on the workshop presentation in Oak Ridge on August 7 and 8, 1990. The consultants reviewed possible tests and their effect on achieving the goals of the test program. On the basis of this review, they ranked the tests with regard to the value of each test in helping to provide necessary behavioral information. The rankings presented by the consultants are shown in Table H1.3.

Table H1.3. Summary of test priorities

Name of test	Priority
Push test	High
In situ deformability	High
Prism compression	High
Flexural bond	High
Full-scale airbag	High
In-plane infill shear	High
Tensile splitting panels	High
Boundary condition evaluation	High
Unit compression	Medium
In situ normal stress	Low
In-plane wall shear	Low
Tensile splitting (units)	Low
Out-building	Low
Shake table	Not assigned

Report 4. The primary conclusions from report 4 came from a series of questions posed by MMES and responded to by the ERC. These questions and responses follow, verbatim.

1. A key issue raised in the HCTW test program is the out-of-plane capacity of HCTWs after their strength/stiffness has been degraded from in-plane loads and the in-plane capacity after the strength/stiffness has been degraded from out-of-plane story drift. Given our building configuration does the ERC have any specific recommendations with respect to the loading sequence for quasi-static loading?

The ERC suggests that lab studies be conducted to evaluate both sequences of strength - stiffness degradation. It is not certain which would produce the worst effect, i.e., in-plane cracking before out-of-plane testing or the opposite.

The out-of-plane loads could be applied with point loads rather than by air bag, however, one must be aware that problems may occur if the location of the point loads coincides with the location of a crack; loading plates or pads should be used to apply the loads over an area. The boundary conditions provided by columns in the test set-up are probably conservative, i.e., panel edge rotation would not be restrained as it would be in the building where there are adjacent panels.

2. Some people have the perception that HCTWs in steel frame buildings shatter (explode) suddenly under seismic lateral loads. Our experience does not support this perception, except in a unit tile compressive test. Does the ERC have any comments?

Prism uniaxial compression tests under force control often result in explosive failure. This is due to the sudden release of energy stored in the prism and in the machine. This condition is not replicated in infilled steel frame structures because energy dispersion is gradual due to progressive diagonal tension cracking.

3. The Safety Analysis Report Update Program (SARUP) for evaluation of the Y-12 facilities is scheduled to begin before the HCTW test program is completed. Should the HCTW test program be conducted in a particular manner given this need? Does the ERC have an opinion on how to proceed with the seismic analysis before having sufficient test data to fully validate analytical methods used?

The University of Tennessee testing should be accelerated as much as possible to provide data for model validation. Initiate building model and analyses using "dummy" values for material and component properties. This will enable the model to be functioning when actual data is available. Sensitivity studies can be done using the analytical models to indicate the accuracy's to which material and component properties must be determined. Such a study will identify and/or confirm the data to be obtained by the experimental program.

It may be possible or desirable to conduct the SAR based upon "conservative" values of component and material properties to obtain a lower bound safety estimate. The lower bound may be adequate.

4. Laboratory specimens constructed with "new" hollow clay tiles (HCT) will be used in many of the tests in the HCTW test program. Typically, removal and testing of

specimens from, or in-situ testing in, contaminated areas costs much more. During the workshop, the uses of unit tile/prism tests were proposed to relate to new construction. Do you support this approach?

The ERC supports the approach of testing specimens built with new HCT to relate existing to new construction. It's the most logical way to use experimental results to validate analytical models. The variance in mechanical properties should not exceed that necessary for proper verification of analytical models. Once verified, the analytical models can be used with properties of existing construction to estimate safe lateral force levels.

- 5. What does the ERC believe is considered to be the proper approach to correlate the results of static tests to the dynamic analysis of the existing buildings under our site specific seismic loads? Does the ERC believe that our shake table can be utilized to bridge the gap between the static test results and dynamic behavior of our buildings? Should large scale dynamic tests at outside laboratories be considered?**

Results of static tests are used to estimate values for strength, stiffness, stiffness degradation, and damping. This is the traditional approach and is the most conservative regarding strength and displacement. This was confirmed by analyses and results of Priestley's full-sized flanged wall specimens and also by Abrams and others. Static tests are more demanding on the structure than are dynamic tests.

It may be feasible to test a sample structure dynamically in 2-D on a shaking table. Measured lateral deflections and/or forces could then be imposed on a sibling structure at static rates to provide information for a direct correlation between static and dynamic response. For the MM-ES shake table, model specimens would be required. It is expected that this would reveal that using the static test results of strength, stiffness, and damage will be conservative.

It is conceivably possible to perform large scale dynamic tests. This would be very expensive because of the size of the specimen, cost of the data gathering, experimental equipment, etc. If it is believed that basing models on static test results is conservative, and that no differences have been observed from shake-table testing, then full-scale dynamic tests are not necessary. The cost effectiveness would be low. Such tests are primarily demonstrations. See the response to Item 15.

- 6. (a) Some properties of old mortar used in constructing infilled HCTWs vary among buildings at Y-12. It is proposed to use an "average" mortar surrogate for the lab test program. Is the proposed use of an "average" mortar acceptable?**

(b) Does the ERC recommend natural or manufactured sands be used in the mortar mix? It is generally accepted that the original sand was local river bottom dredgings. This type of sand is still available but must be specified.

The use of an "average" mortar is recommended. It would be difficult to proceed otherwise. However, assembly properties, e.g., of masonry prisms and wallettes should be reasonably matched. In situ mortar can be expected to be quite variable. Sensitivity of HCT masonry can

be established by tests of small assemblages, e.g., prisms and small diagonal compression specimens.

Use of natural sand is recommended. Mortar made with manufactured sand may require higher w/c ratios to achieve the same workability.

- 7. In which test of HCTW specimens is it critical to use constant cross-head speed or displacement control to apply test loads? It is understood that cross-head speed control yields more post-peak information. When is post-peak information critical?**

Use displacement control in tests where the peak strain data is required. True peak strain values are difficult to obtain under force control. Knowledge of post-peak behavior is required to describe system or component deformation behavior and to set required strength requirements.

Displacement control is also useful for identifying an accurate value of peak stress since a specimen tested in force control will fail quite suddenly. Displacement control is critical for tests of prisms, and in-place walls, for example.

- 8. (a) What specific conclusion does the ERC have about "how the HCTW test program relates to the seismic capacity of our buildings?"**

(b) What specific concerns does the ERC have about the analytical methods currently planned and considered for development in the HCTW program?

The HCTW test program is necessary to establish values used to evaluate seismic capacity using analytical models.

It is not clear to the ERC which analytical methods will be explored or used. Analytical methods should be emphasized to guide the experimental work and to develop systems models for final building analyses. The ERC is concerned that insufficient effort is being expended on this topic. The ERC regards the development of building systems models for seismic response evaluation to be a fundamental part of the safety analyses.

- 9. What is the ERC's opinion for the need for 3-dimensional scale models of structures with infilled HCT being tested on a shake table to develop sufficient confidence in the final numerical modeling? (This would be an addition to the current test program.)**

There is little need for shake-table tests of a three-dimensional scale model at this time. It may be difficult to predict or analyze measured response. Such a test may confirm that the models analyzed by pseudo nonlinear methods are satisfactory. However, three dimensional tests of scale models may be very sensitive to flaws and other imperfections in the model. These could introduce undesirable responses such as torsion and anti-symmetry.

- 10. What is the ERC's opinion of the NDE effort made to date? Are there other technologies that should be examined? How should the methods selected for NDE be validated for use in evaluation of our existing facilities? How would you address an argument that the**

NDE activity is premature since we cannot confidently define the limits of the critical variables at this time?

The NDE techniques described at the November 21, 22, meeting appear to require much development to be in a state where they could be confidently used to assess, for example, the integrity of panel edge conditions or relative quality of panels. MMES has made a good effort to investigate the potential of several methods as presented in the first part of this report. However, unless MMES is confident that one or more of the approaches will be reliable with a small amount of additional work, further time and money expenditures in this area should be reconsidered especially considering the budget situation.

No other NDE technologies that could be utilized in this program, without extensive development are known to the ERC.

A program of examination of edge conditions of the infill panels and relative quality of the panels is suggested using physical methods, e.g., test holes, baroscope examinations, visual surveys, and appears to be more appropriate at this time. The number of such observations should be sufficient to support assumptions regarding the conditions throughout a building.

11. What is the ERC's evaluation of the relative worth of the value of the wall-column bending test described to the committee at the workshop?

The ERC considers the wall-column bending test to be important only if the assumption of a flexible diaphragm is correct. This assumption should be verified using a static linear analysis. If the diaphragm is found to be relatively flexible, then the wall-column bending test has worth since it would define the lateral strength and stiffness of a subassembled that is replicated many times throughout the complex. Because the particular structural configuration is somewhat unconventional, measured behavior would be useful in increasing confidence in predicting seismic response of the system.

12. Can decoupled accelerometers be used effectively to measure displacements of structures tested on a shake table? We have inherited several precision servo-accelerometers, and would prefer to double integrate these for global displacements vs using LVDTs with their inherent mounting and dynamic stability problems.

Displacements obtained by double integration may be questionable because of numerical errors that may be prevalent in the integration routines. Such displacement measures should be verified by LVDTs placed at critical response locations on the test specimen. It is possible that MMES accelerometers would be satisfactory, but the ERC feels more comfortable with LVDTs.

13. With respect to our prism compression tests that were described, does the ERC have specific recommendations about the way in which the tests were conducted to improve future tests?

The ERC believes that prism compression tests may be improved by testing under displacement control.

Use of a 4-inch thick steel bearing plate is suggested to approximate uniform a stress application. This assumes that the distance from the outside edge of the test head to the extreme point on the specimen is in the 5- to 6-inch range.

It is suggested that prism specimens carefully made from "new" materials be tested in both servo-control machines and the MMES in situ test frame to verify the accuracy of results obtained using the MMES frame. It would be necessary to show similarity of results for at least 90% of peak strength on the ascending branch of the stress-strain curve.

- 14. What is the ERC'S opinion about tests to replicate the offset beams and pilasters as described in our wall column bending test? Note that this configuration represents one of our more critical facilities, but is not necessarily typical of construction throughout.**

Tests of subassemblages replicating panel edge conditions caused by offset beams and pilasters are recommended to determine a probable reduction of arching action capacity. It would be difficult or impossible to discern this capacity from tests of assemblages without offset beams and pilasters. Since this configuration represents a critical facility, it should be examined even though it may not be typical of construction throughout.

- 15. As a final verification of our analytical methods, we are considering testing a full scale building. At the first workshop, we indicated that we intended to dynamically test two small abandoned buildings by applying large cyclic loads at the roof level. The use of these buildings is no longer feasible due to asbestos problems. To test a full building, it would have to be constructed.. We have available an enclosed space that is approximately 100 ft. long x 60 ft. wide x 120 ft. high with overhead crane (and a), two-feet thick reinforced concrete floor with approximately 200 three-inch diameters threaded steel inserts welded into embedded steel structure. It appears ideal for building and testing a full-scale structure, and for accommodating any required stiff backs. Would the ERC please address that need for considering such tests in out program?**

A good system-level test(s) to observe predictive accuracy and system behavior would be useful. Without this, all that would be available would be the UT frame tests. The system level specimen need not and should not be a complete building, but rather a well-defined superassemblage of an idealized system. A three-bay three-story specimen structure, for example, could be tested statically using reversed displacement cycles. This system would reveal strength enhancements attributable to continuity of infill panels that would not be seen by any other test in the plan.

Dynamic tests of a single story, single-bay specimen consisting of a HCT plan surrounded by a steel frame would reveal strain rate effects, inertia loading distribution, and the relationship to static strength. This may be worth consideration. However, the need for such a test should be confirmed by building model analyses. The tests could involve a pre-damaged panel due to in-plane loading determined by the demand on the existing building. This would be a demonstration of the out-of-plane safety.

Report 5. Report 5 was the final report issued by the ERC and was in response to a summary workshop presented by MMES to the ERC on April 13 and 14, 1993. The report was issued near

the end of the test program and was intended to summarize the testing and data collection to date as well as to address any outstanding concerns. The following ERC consensus observations are taken directly from the report.

ERC OBSERVATIONS Since the last meeting of the ERC with Energy Systems (November 1991), an impressive amount of experimental work has been completed. The experimental phase of the Hollow Clay Tile (HCT) Program is well underway with data from 184 tests of HCT units, 18 tests of mortar, 48 prism compression tests, 18 in-place shear tests, five normal stress tests, two deformability tests, six airbag tests, 14 static in-plane tests of HCT infilled steel frames, 62 bond wrench tests, and two large-scale wall-column bending tests. Plans for future tests include additional prism tests, normal stress tests, deformability tests, bi-axial tests and static and shake-table tests of HCT infilled panel assemblies.

Progress has been made in quantifying mechanical properties of the HCT structures built in the lab and HCT materials. Results have been disseminated via several presentations and papers at conferences and in journals. The principle areas which, in the opinion of the ERC, require attention are in the categories of:

1. Correlation of properties of laboratory HCT masonry to properties of in situ masonry;
2. Static and shake-table testing of a model HCT structure;
3. Behavior of offset HCT infill;
4. Out-of-plane strength of HCT infill;
5. Equivalent strut; and
6. Analytical modeling.

Correlation of Properties: Correlation of properties of the masonry actually in the building (termed "in situ" properties) with the properties of the HCT masonry used in the laboratory studies. Many tests have been conducted in the laboratory. The value of the results should be assessed in terms of reliability and usefulness in analytical models. Properties of laboratory HCT masonry must be correlated to properties of in situ HCT masonry using the types of tests which are the most productive. Refer to the ERC responses to MM-ES issues number 3 and 4 as documented in the ERC report pertaining to the meeting of November 21 & 22, 1991.

Static and Shake-Table Testing: The purpose of the proposed shake-table testing should be defined in terms of the overall evaluation of the Y-12 plant. Usually such tests are done to provide dynamic data for calibration of computational models. The tests, in general, could be useful for developing a better knowledge of how HCT infills respond to earthquake motions than would be achieved by static testing.

Behavior of Offset HCT Infill: The results of the test with the offset HCT was expected. The critical compression zone is in that material adjacent to the beam-column joint. The added thickness in the central body of the panel would add stiffness, but the maximum strength would be related to the area of the contact zone. A few more specimens with the HCT offset would confirm that a percentage of the strength allowed with concentric material can be used in analysis.

Out-of-plane Strength of HCT Infill: Tests at the University of Illinois have shown that as much as a 50 % reduction in out-of-plane strength can occur with in-plane drifts as high as twice the initial cracking drifts. This percentage is dependent on the h/t ratio. MM-ES will need to know of such a reduction factor.

Extrapolation of the UT tests using the empirical curve-fitting procedure may be inaccurate for infill panels of different aspect ratios and h/t ratios. Arching action varies with the square of the h/t ratio and interactions between in-plane and out-of-plane loadings should be dependent upon both ratios. The observation that in-plane stiffness reduces by only a small amount with out-of-plane loading may not be true for infills with much larger h/t ratios. If arching is not so dominant, more out-of-plane cracking may occur which would reduce in-plane stiffness. Specific tests should be done using specimens of different h/t ratios.

Equivalent Strut: The analytical representation of in-plane infill presented in the draft Preliminary Recommendations for Seismic Analysis of Clay Tile Infills dated 4/7/93 is based on stiffness with no concern for strength. It pertains only to infills with no openings. A hybrid approach may be better which has a strength cutoff based upon measured strength estimates. The effect of varying aspect ratios must be considered. **Alternatively, a FEM approach is suggested as being more adaptable and able to handle the case of infill with openings.**

Analytical Modeling: As noted in the ERC report of the November 1991 meeting, an analytical strategy for modeling seismic response of the Y-12 facilities needs to be defined. It is of paramount importance to identify how the modeling will be done for the complete structural system. Preliminary comments by MM-ES personnel suggesting that a spectral response analysis would be done using a conventional structural analysis program in three dimensions with linear elements is a first step in this direction. However, there are many other questions that remain to be resolved such as: (1) What elements of the system will be included or neglected (such as partial infills)? (2) How will stiffness of cracked infills be estimated? (3) Will the wings of the 9212 complex be considered as separated or joined? (4) Can a simple 2-D representation be of use for part of the analysis?

As reported in a previous ERC meeting, some 2-D analysis has been done on a portion of a HCT wall with a single steel frame. Simple models can be very useful in setting analytical strategy, validation of analytical methods and studying local effects. Models should be validated using the two major experimental studies for which information is now available: the UT infill tests and ISU wall-column bending tests. **Models should be based on basic mechanics rather than on curve fitting.**

Three dimensional modeling, as discussed in "Preliminary Impressions" will be important in many ways. Such a model can be developed using the best available data. There is a potential need for experimental data on the properties of the gypsum roof diaphragm.

The preparation of the SAR could be simultaneous with the development and validation of the 3-D model. However, the 3-D model must include reasonable element stiffnesses. The paper included herein as Appendix A, presents a procedure for analyzing infilled

buildings. The California Strong Motion Instrumentation Program found this procedure to be valid. MM-ES should refer to the U.S. Coordinated Program for Masonry Building Research Report 2.3-6 *Comparison of the Dynamic Response of a Damped MDOF Nonlinear Beam Model With an Equivalent SDOF Hysteretic Model* (Kariotis and Waqfi, 1992) for ideas on how to express damping.

Even though not all of the experimental data is available, MM-ES is strongly encouraged to accelerate analytical modeling. Sensitivity studies should be done with a range of input parameters to examine the necessary degrees of certainty for quantifying any one input variable. These studies would develop the knowledge required to assess what additional experimental data may be necessary.

H.2 NATIONAL WORKSHOP ON UNREINFORCED HOLLOW CLAY TILE²

H.2.1 Introduction

In September 1992, DOE Office of Environmental, Safety, and Health sponsored a National Workshop on Unreinforced Hollow Clay Tile. The workshop was chaired by Dales Jones of MMES, Center for Natural Phenomena Engineering (CNPE), and co-chaired by Mark Webster of the Structural Engineers Association of Northern California (SEAONC). The main objective of the workshop was to bring together researchers and practitioners to begin a dialogue on test methods, structural behavior, and future research needed to develop seismic evaluations of buildings constructed of unreinforced hollow clay tile (UHCT) that would adequately (judged by national consensus) address the safety issue. The specific goals and objectives of the workshop are described in the following sections.

H.2.2 Objective

The goals and objectives for the workshop were divided into three areas: (1) in situ characterization, (2) constitutive modeling, and (3) structural system behavior. The goals and objectives for these three areas are more fully discussed in the following sections.

H.2.2.1 In Situ Characterization

The goal for this area was to identify requirements for assessing the characteristics, constructed condition, and retrofit needs of existing buildings constructed with UHCT. The objectives were identified as follows.

1. Evaluate the current in situ testing methods (e.g., push test, bond wrench, etc.). Determine if current testing methods that were developed for brick and concrete masonry can be directly used for UHCT. Propose any necessary modifications or additional tests.

²Portions of this section were extracted from Jones (1993) with permission from the authors.

2. Identify NDE needs, particularly for evaluation of infilled frames. Determine the applicability of the latest NDE techniques to UHCT structures. Determine how the integrity of the infill/frame interface can be evaluated.
3. Determine inspection methods for HCT structures. Determine the key items that need to be examined in facility walkdowns.
4. Identify retrofit techniques that are appropriate for UHCT structures. Determine what techniques are appropriate for various structural systems (e.g., infills, bearing walls).

H.2.2.2 Constitutive Modeling

The goal for this area was to identify requirements for constitutive models for UHCT and testing requirements for parameter determination. The objectives were identified as follows.

1. Determine for which structural systems in-plane and out-of-plane behavior can be treated independently in the constitutive modeling and for which systems the behaviors need to be combined.
2. Identify strengths and weaknesses of current masonry and concrete models for analyzing the in-plane behavior of UHCT. List the requirements of an in-plane biaxial model.
3. Identify the strengths and weaknesses of current models for analyzing the out-of-plane behavior of structural clay tile. Examine appropriate methods for modeling for the different types of out-of-plane behavior, such as interstory drift, panel cracking, and arching.
4. Identify experimental tests and results that are necessary for determining parameters (particularly for HCT) for the models.

H.2.2.3 Structural System Behavior

The goal for this area was to identify system behavior and analytical techniques for evaluating UHCT walls and UHCT infilled frame structures subjected to earthquake loading. The objectives were identified as follows.

1. Examine structural modeling assumptions for idealizing complicated shear wall and infilled frame systems.
2. Assess the applicability of spectral methods for analyzing the seismic behavior of unreinforced clay tile infill structures.
3. Identify the various failure modes of UHCT walls or UHCT-infilled frame structures. Identify methods for evaluating the capacity in each of these failure modes.
4. Recommend future research needed to establish structural response of UHCT wall and infilled frame structures.

H.2.3 Scope and Methodology

The 2-day workshop was convened on September 21-22, 1992. Thirty-six attendees from four countries (including researchers, engineers, educators and administrators who represented the private sector, universities, and local, state, and federal governments) participated.

The workshop was divided into two parts: (1) for the purpose of introduction and to acquaint other attendees as to their particular concerns, each attendee presented a short paper in the Plenary Session; (2) three committees were then formed to address the three specific areas of interest (in situ characterization, constitutive modeling, and structural system behavior). Participants were chosen for these committees on the basis of where they could best contribute to committee goals. The intent was to provide clear and concise guidance on the issues directly related to the assessment of UHCT structures.

H.2.4 Results and Conclusions

Each of the three committees developed a summary and recommendations, as well as a conclusion section. The results of the three committees were then combined into a concluding observation section.

Committee 1, In Situ Characterization, made 13 recommendations relating to further research needs. Many of these recommendations dealt with the development of standardized testing techniques, including the development of a concentrated and correlated experience data base on UHCT masonry performance. Others recommendations dealt with the need to continue educating the engineering community. Development of a standardized retrofit cost/effect data base to present to building owners was also recommended. The most significant conclusion reached was that many of the tests that have been developed for brick and concrete masonry could be used for UHCT masonry evaluation; however, these tests should be validated in controlled laboratory experiments. The committee also concluded that nondestructive evaluation (NDE) test requirements (invasive and noninvasive) should be based on information required to analyze or predict full building or substructure performance. The committee agreed that, with regard to infill retrofit, one goal is to ensure that sufficient connection between the UHCT infill and framing members is achieved.

Committee 2, Constitutive Modeling, concluded that in-plane and out-of-plane behavior can, in general, be treated separately. However, sufficient connection between infill and frame is required so that the out-of-plane arching effect is ensured. The committee recommended that macro modeling techniques using the finite-element methodology for analyzing in-plane behavior are appropriate. Again, as stressed by Committee 1, Committee 2 concluded that a pressing need exists for an intense laboratory testing program at the fundamental level. This program should consist primarily of biaxial prism tests in the compression-compression and compression-tension regions. Testing should be controlled such that a finite-element model for in-plane behavior can be developed and used to predict nonlinear and progressive local cracking effects.

Committee 3, Structural System Behavior, concluded that, in general, the use of three dimensional analyses that include the HCT infill as a two-dimensional in-plane component would

be an appropriate evaluation technique. The recommended method for in-plane analysis using this technique is the compression strut, if the conditions required for its development are present.

The committee also concluded that spectral techniques may be used with linear stiffness approximations representing the infill properties. This could be done by using piecewise, linear models to represent infills with degrading stiffness. Apparent natural frequencies as they vary with displacement amplitude would then be determined. In making its conclusions the committee grouped the infill failure mechanisms into two categories: (1) panel blowout or slippage caused by out-of-plane inertial loads and (2) corner crushing, diagonal bed joint shear, or sliding caused by in-plane loading. Finally, the committee members generally agreed that methods for predicting capacity in these failure models are not well established.

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Appendix I

APPLICATION OF TEST RESULTS TO NATURAL PHENOMENA EVALUATION OF U.S. DEPARTMENT OF ENERGY FACILITIES

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ABSTRACT

The culminating step in the test program plan was the application of the results of the experimental research to the analysis of Y-12 and other U.S. Department of Energy facilities with hollow clay structures. Parameters that define the ultimate capacity and limiting displacement for in-plane loads on unreinforced masonry infills were developed. In-plane infill stiffness prediction equations were then defined and criteria related to connections, structural damping, and combined loading effects were outlined. Also, out-of-plane loading issues such as response to both story drift and inertial loads and the effect of in-plane loads on out-of-plane capacity were investigated and explained. Finally, a recommended seismic evaluation criterion for the analysis of unreinforced hollow clay tile structures.

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Appendix I

APPLICATION OF TEST RESULTS TO NATURAL PHENOMENA EVALUATION OF U.S. DEPARTMENT OF ENERGY FACILITIES

The culminating step in the test program plan was the application of the results of the experimental research to the analysis of Y-12 and other U.S. Department of Energy facilities with hollow clay structures.

I.1 IN-PLANE WALL BEHAVIOR¹

I.1.1 Wall Load Capacity and Limiting Displacement

Nineteen in-plane tests were conducted under the HCTW Test Program — 17 at the University of Tennessee (UT) and 2 at Iowa State University (ISU) (Flanagan 1994, Henderson 1994). The maximum load (peak) and associated displacement obtained during cyclic testing of a specimen were used as the capacity of the specimen by both Flanagan and Henderson. The average load and displacement (average of push and pull) obtained during the peak loading cycle is considered to be the appropriate capacity and associated displacement that should be assigned to a given test. The net horizontal in-plane wall loads (i.e., wall only; the frame test stiffness times displacement has been subtracted from the test loads) are compared in Table I1.1 for all 19 tests. The numbered tests are those conducted at UT. The tests denoted W1 and W2 are the two full-scale Y-12 replica tests conducted at ISU.

The shaded tests represent special boundary conditions or loading combinations (test 10a had an intentional 1-in. gap between the wall and the column; test 10b was specimen 10a with corner damage repaired by using concrete masonry; test 21b was specimen 21a repaired after failure with new HCT units in the upper courses; test 24 had an opening in one corner; and test 23 was the "combined" loading test). A tabulation of test specimen evaluation parameters for the 14 nonshaded tests indicated in Table I1.1 are provided in Table I1.2. The characteristic parameter, λ , for the column bearing on an elastic foundation is also listed in Table I1.2 and is calculated as follows:

$$\lambda = (E_m t \sin 2 / (4 E_s I_{co} l h'))^{1/4} \quad (I1.1)$$

The values of E_m shown in Table I1.2 are prism test values taken from Yancey (1993) and Boyd (1993). Because the tests indicate that the panel failure mode is crushing of the upper corner HCT units, these corner areas are judged to be in a state of biaxial compression due to the resultant diagonal thrust. The prism tests indicate that the compression strength is different for loading parallel to the HCT cells vs normal to the HCT cells. The effective ultimate compression stress, f'_{m-eff} , has been taken as the geometric mean of the normal and parallel prism tests in Table I1.2. The tabulated f'_{m-eff} values in Table I1.2 are determined from the UT prism test results for the UT

¹Portions of this section were extracted from Oak Ridge Y-12 Plant, Y/EN-5371 (1995b) with permission from the authors.

panel tests (tests 1 through 20) and from the average of all National Institute for Standards and Technology prism tests for the ISU panel tests (W1 and W2). Review of Flanagan (1994) and Henderson (1994) indicates that the test walls were constructed with a wide range of mortar strengths.

If the equivalent strut idealization can be used to represent capacity, then the test loads should be approximately inversely proportional to λ and directly proportional to $f'_{m-eff} \times t$ such that

$$P = f'_{m-eff} \times t \times 8.3 \quad (I1.2)$$

Comparison of this estimator with the test capacities yields an average ratio of unity and a COV of approximately 0.11. Eq. (1.2) was derived for panel length-to-height ratios between 1.0 and 1.5. This equation may not be conservative for length-to-height ratios significantly below 1.0 and excessively conservative for length-to-height ratios greater than 1.5.

Table I1.1. In-plane net horizontal panel test capacities and displacements

Test no.	Test load	Panel capacity P-net (-)	Displacement d (-)	Panel capacity P-net (+)	Displacement d (+)	Panel capacity P-net (avg)	Displacement d (avg)
1	In-plane only	27.0	1.08	36.9	0.77	32.0	0.93
2		35.2	1.10	38.9	1.08	37.1	1.09
3		31.3	0.67	34.5	0.57	32.9	0.62
4		48.3	0.86	31.6	0.68	40.0	0.77
5		39.5	0.62	33.4	0.57	36.5	0.60
7		29.6	0.84	27.4	0.73	28.5	0.79
9		40.4	0.45	38.0	0.53	39.2	0.49
10a		38.6	0.97	26.3	0.84	32.5	0.91
10b		30.3	1.10	31.0	0.90	30.7	1.00
17		44.9	1.22	39.9	1.62	42.4	1.42
21a		35.0	1.48	41.6	1.31	38.3	1.40
21b		33.3	1.44	25.8	1.24	29.6	1.34
24		13.0	0.76	27.7	0.64	27.7/13	0.70
W2		60.1	0.76	50.3	1.00	55.2	0.88
11	Out-of-plane	32.9	0.42	32.3	0.62	32.6	0.52
13	drift then	40.2	0.75	35.9	0.66	38.1	0.71
W1	in-plane	58.6	0.82	40.8	1.00	49.7	0.91
20	Out-of-plane	39.6	0.71	29.5	0.73	34.6	0.72
23	pressure then in-plane	21.9	0.56	18.2	0.56	20.1	0.56

Notes:

- Panel capacity is the net panel capacity equal to the test load minus the frame load.
- Capacities and displacements are in the horizontal direction.
- Capacities are kips, displacements are in inches.
- Test 10a has 1-inch column gap such test displacements are reduced by one inch.
- For tests 24, 11, 13, 20, and 23, the frame stiffness of test 2 was used.
- For tests W1 and W2, frame stiffness of 7 kips/inch was used.
- Shaded tests have special boundary conditions or loading combinations.

Table I1.2. Test specimen evaluation parameters

Test no.	Panel width (inches)	Panel height (inches)	θ (degrees)	Column	I_{col} (in. ⁴)	Wall effective thickness, t	E_m (psi)	f'_{m-eff} (psi)	λ
1	88	88	45.0	W10x12	2.2	7.67	329,000	594	0.1032
2	88	88	45.0	W10x30	16.7	7.67	329,000	594	0.0620
3	88	88	45.0	W10x45	53.4	7.67	329,000	594	0.0464
4	88	88	45.0	W16x26	9.6	13	410,000	351	0.0859
5	88	88	45.0	W10x40	28.9	13	410,000	351	0.0652
7	88	88	45.0	W10x30	16.7	4.67	329,000	594	0.0548
9	88	88	45.0	W10x30	170.0	7.67	329,000	594	0.0347
17	136	88	32.9	W10x30	16.7	7.67	329,000	594	0.0606
21a	112	88	38.2	W10x30	16.7	7.67	329,000	587	0.0616
W2	288	245	40.4	W10x33	66.0	10	660,000	594	0.0432
11	88	88	45.0	W10x30	16.7	7.67	329,000	594	0.0620
13	88	88	45.0	W10x30	16.7	7.67	329,000	594	0.0620
W1	288	245	40.4	W10x33	66.0	10	660,000	587	0.0432
20	88	88	45.0	W10x30	16.7	7.67	329,000	594	0.0620

I.1.2 Equivalent Strut Model of Wall Stiffness

Although the characteristic parameter λ did not seem to correlate with the test capacity data as well as the simple expression of Eq. (I1.2), the strut model does seem to provide a reasonable basis for modeling the infill stiffness. The strut model may be modified to reflect the wall stiffness associated with a given level of displacement as indicated by Flanagan (1994). By such an approach, the effective strut stiffness, or the product of modulus times area $(EA)_{eff}$ of the equivalent strut is reduced by a parameter $(4/C)$ as shown in Eq. (I1.3), where C is a function of the horizontal wall displacement. Note that this formulation is numerically equivalent to the approach presented in Flanagan (1994) and Lockheed Martin Energy Systems (1995a).

$$(EA)_{eff} = E_m (4/C) wt \quad (I1.3)$$

The value of C (minimum = 5 for near-elastic response levels and a maximum of 18 for response associated with panel capacity) is a function of the secant stiffness of the load deflection

curve, as given in Table I1.3. The effective length of column-bearing contact should be limited to no more than $0.2 \times$ wall height, h' .

Table I1.3. Values of C for simulating nonlinear wall behavior

Displacement (in.)	C	Typical infill damage
0.0-0.05	5	None
0.05-0.2	7	Diagonal mortar joint cracking
0.2-0.4	11	Off- diagonal mortar joint cracking
0.4-0.6	14	Banded diagonal mortar joint cracking
0.6-0.8	16	Corner mortar crushing and tile cracking
0.8-1.0	18	Tile faceshell splitting (primarily corner regions)
$1.5 \times$ Displacement @ ultimate	—	Tile faceshell spalling

I.1.3 Framing Members and Connections

Full force transfer (i.e., without friction) of the forces induced by the resultant diagonal thrust (effective compression strut) should be assumed in evaluating all connections. Although the testing (both the HCTW Test Program and other masonry infill tests) indicates that frictional force transfer occurs between the infill and frame, these forces cannot be reliably quantified.

The vertical component of the diagonal strut is resisted by the confining beam of the frame. The beam-to-column connection should be considered to transfer this entire shear. Further, in a structure, the in-plane horizontal force is applied to the wall by diaphragm or floor slab drag forces along the beam. The beam/column connection should also be considered to transfer this resultant tension (or compression). At the base reaction end of the effective strut, column anchors should be considered to transfer both the horizontal thrust and any vertical uplift induced by the adjacent infill panel.

I.1.4 Equivalent Damping

Examination of the hysteresis loops associated with peak loading indicates that the effective viscous damping represented by the hysteretic energy dissipated per cycle, or the enclosed area of the loop, is approximately 5% for the moderate-scale walls and 10% for the full-scale walls. If the simple structure (two independent infilled frames) subjected to the shake table testing (Fowler 1994) is viewed as an approximate single degree of freedom system, then the 5% damped response spectrum (an assumption; the response spectra given by Fowler are not labeled) for the $16 \times$ reference motion is approximately 10 g. Fowler indicates that the maximum wall loading for this input was approximately 48 kips and that the slab weight was 9.5 kips; thus $48/(9.5/2) = 10$ g, which indicates that the effective hysteretic damping was approximately 5% during the test.

Note that the preceding estimates of damping represent the hysteretic energy of an effective element consisting of a single infilled frame bay. NUREG CR-3805 indicates that, for hysteresis

loops that have strongly pinched behavior (such as the HCTW load deflection curves), the energy dissipation can be accounted for in a secant model by adding the effective hysteretic damping to the elastic material damping. Elastic material damping for HCT masonry construction is judged to be on the order of 4% to 7%. These are the Response Level 1 and 2 (i.e., elastic behavior) damping values for masonry construction from U.S. Department of Energy (DOE) Standard DOE-STD-1020-94. Hence, the total damping (hysteretic plus elastic material) to be used for response analyses is expected to be between about 10% and 15%.

I.1.5 Effect of Out-of-Plane Loads on In-Plane Capacity

The effect of prior 1% out-of-plane drift response of the wall did not appear to have a significant effect on in-plane capacity. Likewise, the effect of prior out-of-plane loading did not appreciably affect the in-plane capacity. However, the effect of concurrent lateral loading and in-plane loading did affect in-plane test capacity. The failure mode of the wall was a compression failure at the bottom course of the wall due to the combined vertical compression component of the strut thrust and the compression induced by vertical arching due to the lateral out-of-plane load. On the basis of these observations, it is concluded that the interaction between in-plane and out-of-plane loads should be considered.

I.1.6 Effect of Column-Wall and Beam-wall Gaps

A test in which a 1-in. gap is present between the wall and the column indicated that the cyclic behavior of the wall is similar to the behavior of a wall without a gap except for the effect of a reduced secant stiffness caused by the additional displacement associated with the gap. The effect appears to be an approximate 10-15% reduction in capacity.

The HCTW Test Program did not consider the effect of beam/wall gaps. Dawe and Seah (1989a) indicate that capacity is reduced 50% for concrete masonry infills tested with a 20-mm gap.

I.1.7 Effect of Wall Openings

The basis of the wall opening criteria given by Lockheed Martin Energy Systems, Inc. (1995) is not based on the test results of the HCTW program but rather on review of May and Ma (1985) and Amos (1985). The only opening considered in the HCTW program was an opening in the corner of the infilled panel. This opening dramatically affected the infill behavior: capacity was reduced by approximately 60%.

I.1.8 Effects of Beam-Column and Column-Base Connections

For all tests specimens (moderate- and full-scale), slip-critical A325 bolted connections that were overstrength to prevent connection behavior issues were used. This is reasonable because the focus of the test program was the strength behavior of the masonry infill and not the strength of the frames. However, the actual Y-12 frames are vintage (1950s) riveted connections that are considered as bearing connections and subject to slip. Flanagan (1994) and Henderson (1994) indicated that although the connections were designed as slip critical, considerable connection slip did actually occur as the test specimens were cycled at load levels approaching capacity. Thus, the test capacity levels are judged to include the effects of connection slip.

I.2 OUT-OF-PLANE WALL BEHAVIOR

I.2.1 Response to Story Drift

The out-of-plane test behavior of walls subjected to drift imposed by either a diaphragm or a floor slab was considered in both Henderson (1994) and Flanagan (1994). Both test programs tested the walls with a cyclic imposed drift ratio of 1% and concluded that wall capacity was not affected by drifts of this order.

I.2.2 Response to Lateral Inertial Loads

The primary data for the behavior of HCT walls subjected to lateral out-of-plane inertia loading as simulated by static airbag loading are provided in Flanagan (1994) and Huff et al. (1992). Flanagan subjected the test walls to cyclic pressure application at each increment of loading whereas Huff et al. subjected the test wall to increasing static pressure only. Note that the vertical wall edge conditions were such that horizontal arching was minimal. All of the walls failed by one-way vertical arching between the floor and confining beam.

Both Flanagan and Chua (1991) used the formulation proposed by Dawe and Seah (1989b) to express wall capacity under arching conditions. The formulation proposed by Dawe and Seah is based on a parametric analytical study without direct comparison with test data. The formulation has been checked for a few of the Dawe and Seah test results and seems to predict the maximum lateral wall test loads with reasonable accuracy.

I.2.3 Effect of In-plane Loading on Out-of-plane Capacity

Prior out-of-plane drift response results in cracking at the base course of the wall. This cracking did not seem to affect the in-plane capacity of either the moderate- or full- scale test specimens. The converse condition of the effect of drift on walls damaged by in-plane loading or out-of-plane lateral loading was not tested. One out-of-plane pressure test conducted for a wall with prior in-plane loading resulted in an approximate 20% reduction in out-of-plane capacity.

I.3 RECOMMENDED SEISMIC EVALUATION CRITERIA FOR UNREINFORCED HOLLOW CLAY TILE WALLS

I.3.1 Evaluation of Demand for Hollow Clay Tile Wall Buildings

Evaluation of Overall Seismic Response. A dynamic model of the building should be developed for seismic analysis representing all beams and columns as beam elements and all infilled HCT panels as diagonal struts. An equivalent diagonal strut used to represent infill wall panels is illustrated in Figure I3.1. The panels may be represented in the analytical model either as a single strut or as two struts in an "X-bracing" configuration with each strut at $\frac{1}{2}$ the total wall stiffness. It should be recognized that the equivalent strut approach has been formulated without consideration of vertical forces. Thus, vertical loads should be omitted during the lateral load analysis and superimposed onto frame members later.

Linear elastic seismic analyses may be performed by using the response spectrum modal superposition method. Spectra in compliance with DOE-STD-1020-94 at 10% of critical damping

should be used for evaluation of buildings where HCT infill walls are the lateral-force resisting system.

Fig. I3.1. Equivalent diagonal strut representation of an infill.

Evaluation of Equivalent Stiffness. HCT walls exhibit nonlinear behavior during large seismic response as wall stiffness decreases with increasing wall displacement. The strut model may be formulated to provide wall stiffness associated with a given level of displacement. By such an approach, the effective strut stiffness, or the product of modulus times area $(EA)_{\text{eff}}$ of the equivalent strut, is reduced by a parameter $(4/C)$ as given by Eq. (I3.1). The value of c (minimum = 5 for near elastic response levels and a maximum = 18 for response associated with panel capacity) is a function of the secant stiffness from the wall load-deflection curve, as given in Table I1.3.

$$(Ea)_{\text{eff}} = E_m (4/C) wt \quad (\text{I3.1})$$

The width of the equivalent strut is then determined by

$$w = \alpha / \cos \theta \quad (\text{I3.2})$$

The length of column bearing on the infill is given by

$$\alpha = \pi / 4 \lambda \quad (\text{I3.3})$$

where the parameter, λ , is given by

$$\lambda = (E_m t \sin 2 \theta / (4E_s I_{col} h'))^{1/4} \quad (I3.4)$$

in which E_m is the gross elastic modulus of the masonry (ksi); $E_s I_{col}$ is the flexural rigidity of the column (ksi $\times$ in⁴); θ is the slope of the infill diagonal to the horizontal; t is the infill gross thickness (inches); and h' is the infill height (inches). The gross elastic modulus of the HCTW, E_m , may be taken as 350 and 400 ksi for 8-in. and 13-in. walls, respectively. Eq. (I3.3) is appropriate for frames with type 2 connections (zero moment). The effective length of the column bearing contact, α , should be limited to no more than 0.2 times wall height, h' .

For HCT infills offset from the framing centerline, use the full gross thickness, t , if the panel is totally enclosed in the framing. If part of the panel is outside the framing, use the gross thickness of the portion of the panel enclosed in the framing, provided that at least one cell is enclosed. In this context, "framing" includes masonry pilasters. These rules for thickness, t , apply both to stiffness determination as described previously and to capacity evaluation as described in the following section. Estimates of building deformations are made to establish the secant stiffness for the equivalent strut. The resulting deformations from the response analysis must be compared with the estimated deformations and an iteration may be needed such that calculated and assumed displacements are sufficiently close that the building modal properties are consistent with the response.

Effect of Openings on Strut Stiffness. The presence of openings may significantly alter the in-plane stiffness and strength of HCTW panels. Several opening configurations are illustrated in Fig. I3.2. The approach employed to account for openings is to evaluate the stiffness and capacity of the panel without openings and then apply reduction factors for stiffness and strength as given in Table I3.1.

The rules for panel cases 1 and 2 apply if the door openings are approximately in the middle one-third of the panel width. For panel cases 5 and 6, the strut model should be on a diagonal across the solid portion of the panel (e.g., partial height panel modeled with partial height strut).

Table I3.1. Stiffness reduction factors

Panel case	Stiffness reduction (%)	Strength reduction (%)
1	100	100
2	100	100
3	75	50
4	75	75
5	75	50
6	75	50

Fig. I3.2. Hollow clay tile wall opening cases.

Evaluation of Out-of-Plane Demand on Infill Walls The overall seismic response analysis gives out-of-plane displacements and accelerations at model node points. The out-of-plane displacements may be used to compute story drifts that may be compared with story drift limits. The out-of-plane accelerations may be used to develop seismic inertial loads on the wall by separate calculations. It is recommended that out-of-plane wall demand be determined as the inertial load corresponding to the average of the floor spectral accelerations from the floor above and the floor below at the natural out-of-plane frequency of the wall. The out-of-plane natural frequency is calculated using the arching action of the wall at ultimate capacity (Lockheed Martin Energy Systems 1995b). If floor spectral accelerations are not available, it is recommended that the average of the Zero Period Accelerations (ZPA) from the floor above and the floor below be multiplied by 2 and used as input for computing inertial loads.

If the out-of-plane demand is less than $\frac{1}{2}$ the out-of-plane capacity, interaction with in-plane behavior need not be considered. Where interaction is considered, a linear interaction relation shall be used. Note that out-of-plane and in-plane demands should be combined in a probabilistic manner such as is represented by the 100-40-40 rule in ASCE 4-86.

1.3.2 Evaluation of Capacities of Hollow Clay Tile Buildings

Acceptance criteria for unreinforced infilled HCTW at Oak Ridge have been developed on the basis of the target performance goals of DOE Order 5480.28. It is recommended that horizontal in-plane HCTW panel capacity be determined from

$$C_c = 8.3 \Phi * t_{eff} * f'_{m-eff} / F_M = 4.15 * t_{eff} * f'_{m-eff} \quad (I3.5)$$

where Φ is a strength reduction factor equal to $2/3$, f'_{m-eff} is the geometric average of the median masonry strength normal and parallel to the cells in the masonry units, F_M is a factor of safety on the median masonry strength equal to $4/3$, and t_{eff} is the thickness of the wall that does not protrude out from the frame members. The value given previously is the capacity of the HCTW panel only. For those cases where the frame has moment connections, the capacity of the frame may be computed from its stiffness times the response displacement and added to capacity just given to produce the total capacity of the infilled HCTW element. For Y-12 construction, f'_{m-eff} values of 647 psi for 8-in. walls and 420 psi for 13-in. walls are recommended.

Frame connections (column-base and beam-column) shall be required to take the full level of force in the strut representing the wall. Any friction between the wall and the frame shall be ignored in accordance with DOE-STD-1020-94 for the appropriate materials of construction, taking F_μ to be unity.

For Performance Category 3, in-plane horizontal displacement of walls shall be limited to 0.5 in. to be consistent with the performance goals for this category. For buildings in Performance Category 1 or 2, in-plane horizontal displacement is permitted out to 0.75 in. provided that maximum load is limited to 75% of the level given by Eq. (I3.5). If maximum load per Eq. (I3.5) is reached in a wall element before the maximum limit displacement is reached, reducing the stiffness of that element by some rational approach and rerunning the demand calculation allowing redistribution of loads to other elements is permissible. The limit state is reached when the element reaches the limit displacement or other elements are not able to resist the increased loads from the redistribution.

The out-of-plane capacity for inertial seismic loads may be estimated from the prediction equation presented as Eq. (I3.6).

$$q = 0.8 * \Phi * (f'_m / F_M)^{0.75} * t^2 * \beta / h^{1/2.5} \leq 3 \text{ psi} \quad (13.6)$$

where q is the uniform out-of plane pressure capacity in pounds per square inch of a wall that is primarily acting in one-way action spanning vertically, $\Phi = 2/3$, $F_M = 4/3$, f'_m is the median masonry prism strength normal to the masonry unit cells in psi, t is the infill gross thickness in inches, h' is the infill height in inches, and β is a measure of the relative stiffness of the bounding beam. The term for horizontal arching can be used if a mechanism exists to prevent slippage of the wall along the column. If the above approach is too conservative, a more rigorous approach should be used to calculate out-of-plane capacities (Lockheed Martin Energy Systems 1995b). Out-of-plane response to story drift is expected to be governed by the story drift limits from DOE-STD-1020-94 for buildings.

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Appendix J

SUMMARY OF HOLLOW CLAY TILE WALL TESTING

The following tables are an expanded version of Table 2 in Section 3.5 of this report. For each test planned or performed as part of the Hollow Clay Tile Wall Test Program at the Oak Ridge Y-12 Plant, a brief description and purpose is given. The specimen size, test location(s), number of tests planned, and number of tests performed are also listed.

Unit Tile Compression

Description	Compressive tests on in situ and new 4- and 8-in. hollow tile units.
Purpose	Determine ultimate compressive strength and modulus of elasticity.
Specimen	4- and 8-in.-thick (12-in. wide by 12-in. high) HCT units.
Test location	UT, EDGe, Singleton.
Number planned	55
Number tested	235

Unit Tile Splitting Strength

Description	In situ and new 4- and 8-in. thick by 12-in. wide by 12-in. high hollow tile units.
Purpose	Determine splitting strength of unit tiles.
Specimen	4- and 8-in.-thick (12-in. wide by 12-in. high) HCT units.
Test location	EDGe, Singleton
Number planned	40
Number tested	67

Unit Tile Initial Rate of Absorption

Description	In situ and new 4- and 8-in.-thick by 12-in. wide by 12-in. high HCT units.
Purpose	Measure the capillary action or suction of water into the tile unit to determine mortar bond strength.
Specimen	4- and 8-in.-thick (12-in. wide by 12-in. high) HCT units.
Test location	UT, Singleton
Number planned	40
Number tested	88

Mortar Characterization

Description	In situ mortar removed from existing walls at Y-12 Plant.
Purpose	Determine existing mortar composition to enable development of a comparable lab mix.
Specimen	Mortar removed from bed joints in HCTWs at Y-12 Plant.
Test location	TCMC.
Number planned	Random samples from various buildings.
Number tested	18

Mortar Compression

Description	2-in. mortar cubes made from laboratory mixes.
Purpose	Determine compressive strength and control mixes for quality assurance purposes.
Specimen	2-in. cubes
Test location	TCMC, UT, ISU, NIST, USACERL
Number planned	As required by ASTM to control mortar mixes.
Number tested	As required by ASTM for each mortar mix batch at each test location.

Mortar Tensile Splitting

Description	Laboratory made 2-in.-diameter by 4-in.-long specimens.
Purpose	Determine splitting tensile strength of laboratory specimens built with approved mortar mix.
Specimen	2-in.-diameter by 4-in.-long mortar cylinders.
Test location	UT
Number planned	Number was to be based on the number of laboratory mixes and ASTM C 780.
Number tested	6

In Situ Mortar Bed Shear

Description	Remove an existing tile from a wall and insert hydraulic ram to perform the shear test.
Purpose	Determine interface shear strength of bed joint.
Specimen	An isolated unit tile is loaded (pushed) until mortar shears.
Test location	Y-12 Plant
Number planned	6
Number tested	18

Deformability

Description	Remove the bed joint mortar at the top and bottom of a section of an existing wall and insert flatjacks.
Purpose	Determine the in situ compressive deformation properties of HCTWs.
Specimen	A section of wall two blocks wide by four rows high.
Test location	Y-12 Plant
Number planned	One to three specimens in each building being analyzed.
Number tested	2

Flex Bond Strength

Description	A relative large section of a HCTW that has been removed from an existing wall or has been laboratory constructed.
Purpose	Determine the flexural bond strength of mortar to the clay tile.
Specimen	A section of wall 3 ft wide by 6 ft long.
Test location	None tested
Number planned	30
Number tested	0

Bond Wrench Test

Description	Clamp and isolate clay tile unit with a specially designed tool to apply a load that wrenches the unit from its bed joint.
Purpose	Measure the flexural bond strength of mortar to block.
Specimen	One isolated clay tile unit still set in its undisturbed mortar bed joint.
Test location	Y-12 Plant
Number planned	Exact number was never determined.
Number tested	48

Prism Compression

Description	2-ft by 4-ft wall specimens removed from existing walls or shop fabricated.
Purpose	Determine compressive strength (f'_m) and stress/strain relationship (E_m) of masonry prisms in two directions.
Specimen	13- and 8-in.-thick HCT wall specimens 2 ft wide by 4 ft high oriented vertically or horizontally with respect to a wall.
Test location	UT, NIST, Y-12 Plant
Number planned	32
Number tested	76

Prism Splitting

Description	4-ft by 4-ft wall specimens removed from existing walls or shop fabricated.
Purpose	Determine the splitting tensile strength of masonry prisms in two directions representing vertical and horizontal directions in a wall.
Specimen	13- and 8-in.-thick HCT wall specimens 4 ft wide by 4 ft high.
Test location	None tested
Number planned	42
Number tested	0

Biaxial	
Description	4-ft by 4-ft wall specimens removed from existing walls or shop fabricated.
Purpose	Determine biaxial stress-strain relationships and strength information to be used in finite element analyses of infills as a check on the recommended analytical models.
Specimen	13- and 8-in.-thick HCT wall specimens 4 ft wide by 4 ft high.
Test location	None tested
Number planned	Exact number was never determined.
Number tested	0

In Situ Normal (Flatjack)	
Description	Remove the bed joint mortar at the top and bottom of a section of an existing wall and insert flatjacks.
Purpose	Determine the existing dead load stresses in existing walls.
Specimen	A section of wall two blocks wide by four rows high.
Test location	Various buildings at the Y-12 Plant.
Number planned	One to three walls in each building being analyzed.
Number tested	5

Full Scale Out-of-Plane Air Bag	
Description	Load a wall panel uniformly out-of-plane by using a bag sequentially inflated with air pressure.
Purpose	Determine the ultimate strength and failure mode of HCT-infilled wall panels.
Specimen	Existing wall panels or laboratory-built infilled panels.
Test location	Y-12 Plant, UT
Number planned	4
Number tested	4

**In-Plane HCTW
Infilled Frame**

Description	HCT walls enclosed within steel frames (infilled) to simulate existing infilled walls at the Y-12 Plant.
Purpose	Determine failure modes, ultimate strength, and hysteretic behavior (damping) of infilled wall panels.
Specimen	Approximately 8-ft by 8-ft infilled walls either 8 or 12 in. thick.
Test location	UT
Number planned	14
Number tested	15

Out-of-Plane Drift and In-Plane

Description	HCT walls enclosed within a steel frames (infilled) to simulate existing infilled walls at the Y-12 Plant.
Purpose	Determine failure modes, ultimate strength, and hysteretic behavior (damping) of infilled wall panels subjected first to out-of-plane drift then to in-plane loading. Subject other frames to in-plane followed by air bag pressure loading.
Specimen	Approximately 8-ft by 8-ft infilled walls either 8- or 12-in. thick.
Test location	UT
Number planned	9
Number tested	5

Wall Column Bending

Description	Full-size testing of laboratory-built replicas of infills in Building 9212. This includes the pilaster at the columns and the offset purlin beam.
Purpose	Subject a full-size wall having the same construction as existing walls to out-of-plane drift loads to determine crack patterns and stiffness degradation. Then subject the wall to in-plane loading to determine ultimate compression and hysteretic behavior. Compare this to an identical wall subjected only to in-plane loading.
Specimen	13-in.-thick walls (24 ft wide by 23 ft high) infilled into a steel frame identical to that of Building 9212. The specimen included the HCT pilaster around the column.
Test location	ISU
Number planned	2
Number tested	2

Shake table Tests

Description	Specimens to be similar to in-plane infilled frame tests conducted at UT. Cantilevered walls were to be tested at the K-25 Site Seismic Shake Facility (K-1600). Similar infills were to be constructed as a full building system and tested at a commercial test lab. The planned K-25 testing was replaced by shaketable tests at USA-CERL.
Purpose	Determine dynamic failure modes and g-levels to achieve failure. Determine damping and stiffness degradation and then compare results with static tests of similar specimens.
Specimen	Infilled cantilevered walls approximately 8 ft by 8 ft and 13 in. thick connected by a floor structure to form a three-dimensional shape.
Test location	K-25 Site, USA-CERL
Number planned	26
Number tested	22 separate tests on a single structure

Nondestructive	
Description	Investigate NDE methods such as thermography, microwave, sonic and ultrasonic wave propagation, acoustic emission, and modal surveys.
Purpose	Develop a methodology to determine material flaws in wall construction, tile-to-steel-frame interface connections, and thickness of collar joints.
Specimen	Existing walls at the Y-12 Plant and laboratory control specimens.
Test location	Y-12 Plant, K-25 Plant, UT, ISU
Number planned	As required to qualify wall parameters for use in analytical models of buildings.
Number tested	9

Out Building	
Description	Small buildings constructed of HCT at the Y-12 Plant that had been decommissioned were to be tested to failure by applying dynamic loads to them. These tests were discontinued due to budget constraints.
Purpose	To show that the analysis techniques developed could adequately predict response to natural phenomena forces (seismic and wind) on a full structure.
Specimen	Small building constructed of HCT at Y-12 Plant that had been decommissioned.
Test location	Y-12 Plant
Number planned	2
Number tested	0

HCT Coupon	
Description	Small pieces removed from the face shells of HCT blocks were machined and tested to failure.
Purpose	To determine the basic properties of the HCT block material such as maximum tension and compression strength, modulus of elasticity, modulus of rupture, and absorption.
Specimen	The specimens were approximately 2.25 in. wide by 12 in. long by the thickness of the face shell (approximately 0.75 in.).
Test location	UT
Number planned	0
Number tested	25

Moisture Absorption	
Description	Submerge a whole unit tile (one block) in boiling water for 1 or 5 h.
Purpose	To determine the saturation coefficient of a unit tile.
Specimen	One unit tile 8 or 12 in. thick by 12 in. wide by 12 in. high.
Test location	Y-12 Plant, K-25 Plant, UT
Number planned	0
Number tested	48

Miniature Prism Compression	
Description	Miniature prisms were laboratory constructed and tested in compression.
Purpose	To compare results with the larger prism test results.
Specimen	Two 8-in. HCT units laid up in stack bond with a 3/4-in. bed joint.
Test location	UT
Number planned	0
Number tested	4

Prism Compression at an Angle

Description	Two-ft by 4-ft prisms were tested at various angles to the bed joint (22.5, 45, and 67.5 degrees). Special fixtures were constructed to allow the prisms to be constructed with bed joints horizontal.
Purpose	Check ultimate load capacity and stress strain of these prisms.
Specimen	Two-ft by 4-ft prisms constructed at various angles to the bed joint (22.5, 45, and 67.5 degrees).
Test location	UT
Number planned	0
Number tested	9

ACRONYMS

ASTM	American Society for Testing and Materials
EDGE	Engineering Design and Geosciences Group, Inc.
HCT	hollow clay tile
HCTW	hollow clay tile wall
IS	Iowa State University
NDE	nondestructive evaluation
NIST	National Institute for Standards and Technology
TCMC	Testwell Craig Materials Consultants, Inc.
USA-CERL	U.S. Army Construction Engineering Research Laboratory
UT	University of Tennessee, Knoxville

Appendix K
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Appendix K

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