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A TEST PROGRAM TO DETERMINE THE STRUCTURAL PROPERTIES OF UNREINFORCED HOLLOW CLAY TILE MASONRY WALLS AT THE DOE OAK RIDGE PLANTS

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ABSTRACT

The recent Department of Energy (DOE) Order 6430.1A "General Design Criteria" has emphasized the importance of determining the adequacy and safety of both new and existing facilities to natural phenomenon hazards. Many of the buildings at the DOE Oak Ridge facilities are constructed with unreinforced masonry hollow clay tile infill walls - in some cases these walls comprise a substantial part of the lateral force resistance for a building. In order to perform a realistic assessment of the strength of the buildings to seismic events it is important to accurately predict the behavior of these walls. Very little information is currently available on hollow clay tile masonry, its structural properties and behavior. As the in-situ condition of these walls throughout the plants is suspect due to their age and exposure to numerous chemicals, a test program was initiated at the Oak Ridge plants to obtain material properties for use in the natural phenomena hazards analysis. This paper presents the preliminary results of that testing program. The following tests on clay-tile walls, units, and panels were performed: (1) in-situ mortar bed shear strength, (2) compression strength, (3) splitting tensile strength, and (4) diagonal tension (shear) strength of panels which had been removed from existing walls. The testing program is ongoing, is being expanded, and will include not only in-plane tests, but out-of-plane bending testing as well.

INTRODUCTION

The Department of Energy (DOE) plants located in Oak Ridge, Tennessee (the Oak Ridge Y-12 Plant, the Oak Ridge Gaseous Diffusion Plant, and the Oak Ridge National Laboratory) are operated by Martin Marietta Energy Systems, Inc. The three plants were constructed during the mid-1940's and have expanded since then. During the past twenty years the design criteria

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for natural phenomenon hazard events (seismic and wind/tornado) in the Oak Ridge area has steadily become more demanding. In 1970 the Uniform Building Code (UBC) (5) introduced a new seismic hazard map which placed Oak Ridge in Seismic Zone 2 rather than Zone 1 where it had been located in earlier editions of the Code. The increase of one zone level effectively doubled the lateral force requirements for the seismic design of structures. The UBC-88 places Oak Ridge in zone 2A with Z = 0.15. In 1987 the DOE issued in draft form General Design Criteria, Order 6430.1A (9); it references a document, UCRL-15910 (6) which provides guidelines for the evaluation and design of DOE facilities against natural phenomenon hazards. UCRL-15910 provides three levels of effective peak ground acceleration levels for the Oak Ridge area depending on the hazard classification of the facility: 0.15g for general use facilities; 0.19g for important, low and moderate hazard facilities; and 0.32g for high hazard facilities. The wind design criteria also becomes more demanding the higher the hazard level, and includes tornado and missile criteria for the moderate and high hazard categories. The majority of the critical facilities at these plants fall into the moderate hazard categories.

A direct result of DOE Order 6430.1A is that a greater emphasis is being placed on determining the impact of natural phenomenon hazard events on the on-site and off-site consequences to the general public, the plant operators, and the environment. As a consequence, the criteria are being used not only for the design of new facilities, but are also being applied in the structural evaluation of existing facilities. Every time that a new facility or process is installed into an existing building, the safety analysis requires that the building's resistance to natural phenomenon hazards be evaluated and upgraded accordingly. The philosophy being used to perform an evaluation of an existing facility has been to obtain a "realistic" assessment of the actual strength of the facility in order to estimate the corrective actions needed to increase the seismic (or wind) resistance to an acceptable risk level. One of the biggest question marks in these analysis has been the properties and behavior of the unreinforced masonry (URM) hollow clay tile (HCT) masonry infill walls that exist throughout the plants. Many of the older buildings at the Oak Ridge plants are industrial type structures, steel frame with reinforced concrete floors, constructed with URM infill walls. The masonry walls are concrete block, brick, and HCT. This study deals only with the HCT infill walls that exist in the plants.

The older facilities were not designed for seismic forces, and only for a nominal wind loading and, therefore, in many cases the HCT walls comprise a substantial portion of the existing lateral force resistance in these buildings. The construction specifications for most of the older facilities are no longer available. Very little information on HCT, its structural properties and behavior have been found in the engineering literature. Although data from the literature (1,7,10) have been used to estimate the shear, bending, and tensile strengths of the URM-HCT walls, the use of this data for analysis has been questioned due to (1) the inherent uncertainties in the state of the facilities (the condition of the walls throughout the plants is suspect due to their age and exposure to numerous chemicals), and (2) the fact that much of the published information is not specifically on HCT, but on other types of masonry. In order to perform an assessment of the seismic strength of the buildings it is important to accurately predict the behavior of these walls, and, therefore, a testing project was initiated to obtain this information.

CURRENT TESTING PROGRAM

The testing program was initially intended to provide immediate answers in support of a safety analysis review of a facility that was awaiting approval by an independent reviewer pending verification of the assumed material properties of the HCT used in the analysis. However, because of the importance of the HCT walls to the lateral resistance of the facilities and the lack of available data, a more comprehensive testing project to develop a substantial data base was approved and funded.

The program developed in support of the initial goal consisted of the following tests (tests 1-5 have been completed and are described in this paper, while the last two are still incomplete as of this writing):

1. In-situ mortar bed shear strength
2. Compression strength
3. Splitting tensile strength
4. Diagonal tension (shear) strength of clay tile panels
5. Compression modulus of elasticity of the HCT
6. Poisson's ratio
7. Tension modulus of elasticity of the HCT

The Geologic Associates Division of the Engineering, Design & Geosciences (EDGE) Group, Inc. performed tests 2-4 under contract to Energy Sytems, and submitted a report of their tests and results (11). The remainder of the tests have been performed in-house.

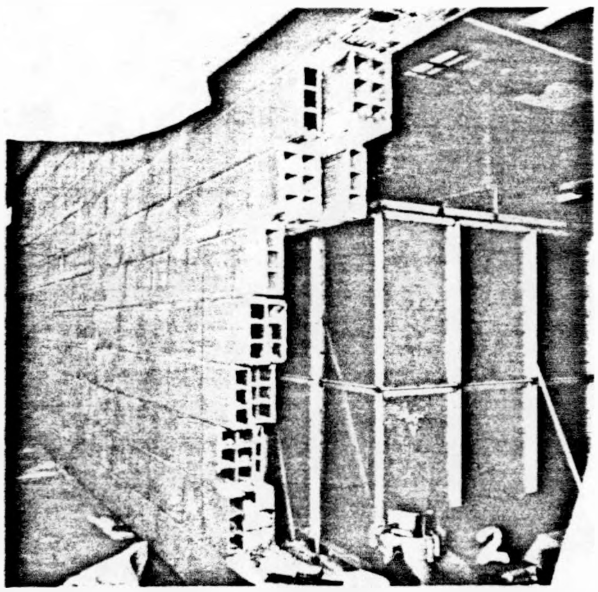

Fig. 1 - Typical 12-in. HCT Wall

Three sizes of URM-HCT infill walls are found in the buildings. The outside walls are 12-in. (304.8 mm) thick, while the interior walls are either 12-in., 8-in. (203.2 mm), or 6-in. (152.4 mm) thick. The 12-in.

thick walls are constructed (Fig. 1) from 12 x 12 x 8-in. tile and 12 x 12 x 4-in. (101.6 mm) tile, using a running bond side construction, with the 8-in. and 4-in. tiles alternating rows. The in-situ wall testing and the panel testing was done using the 12-in. walls, while the lab testing used both 8-in. and 4-in. HCT units. All the units and panels tested were removed from existing facilities.

IN-SITU MORTAR BED SHEAR STRENGTH TESTING

Test Setup

The testing procedure for performing the in-situ mortar bed shear test is basically that described by Schmid et. al. (8). The in-situ test is a partially destructive in-plane test which measures the shear strength of the mortar bed joint between a masonry block and the adjacent blocks on the top and bottom by displacing the test block horizontally with a hydraulic ram (Fig. 2). Nine locations were selected for testing, three locations in each of three walls.

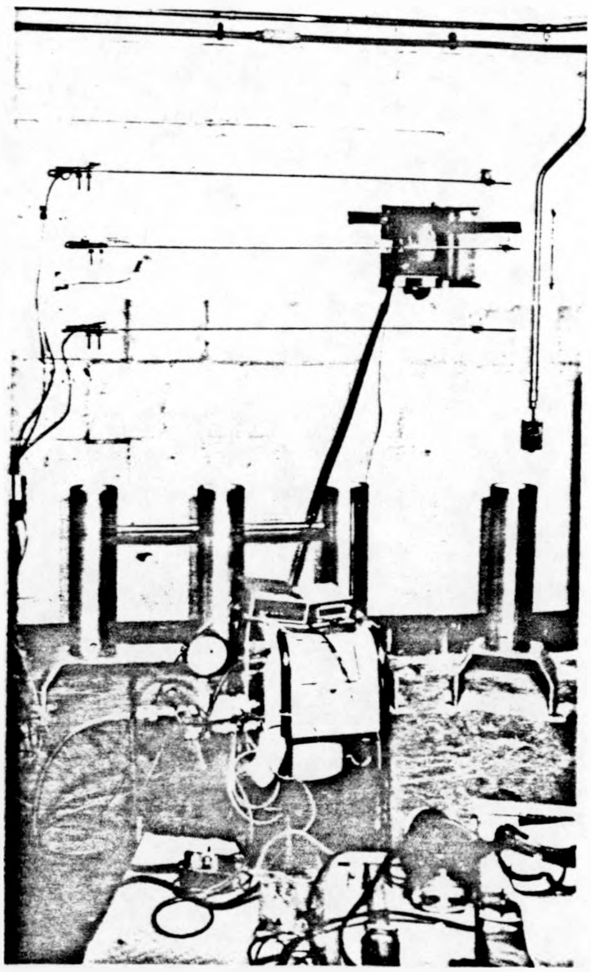

Fig. 2 - Mortar Bed Shear Test Setup

The procedure for performing this test calls for drilling out the mortar around and removing one tile behind the test tile, removing the head joint in front of the test tile. A hydraulic ram inserted in the opening, applies load to the test tile, with the reaction load being taken by the wall behind the ram. The entire cross-section of a 12-in. wall will act as a unit (there is virtually no collar joint between the 8-in. and the 4-in. tiles, but the bed joint is placed across the entire width) and thus the full width of the wall was tested. To be able to test the entire width of the wall it was necessary to modify the set-up procedure in order to obtain a flat surface at each end of the opening. A 4-in. long section of HCT was inserted into the wall at each end of the opening, and the new tile cores and a part of the existing tile cores were filled with grout to ensure a solid joint. A steel plate was placed at each end of the opening. A 30 ton (267 kN) hydraulic ram, placed in series with a load cell, was used to apply the load. The magnitude of the applied load was measured with the load cell and backed up with a pressure transducer connected to the hydraulic cylinder. LVDTs were placed on each side of the wall to measure the displacement of the test tile. The data acquisition system consisted of an HP-9825 computer and an HP-3497A scanner unit. An X-Y plotter provided monitoring of the load-deflection data.

In-Situ Test Results

Load was applied at approximately 10 kips (44.5 kN) per minute, until the applied load increased no further or the head joint gap had been closed. Fig. 3 shows some typical load-deflection curves. A summary of the test results from the in-situ tests are presented in Table 1. The failure load, unless it was clearly defined during the test, was taken to be the load that occurred at a displacement of 0.060-in. (1.524 mm). The general failure surface was between the mortar joint and the tiles above and below the test block, i.e., the mortar bed displaced with the test tile. Mortar was placed over the entire width of the wall on both the top and bottom surfaces (as seen in Fig. 1), so the total nominal mortar bed area for a test block is 288 in.^{2} (185.8 x 10^3 sq. mm). Measurements taken prior to the first set of tests revealed that the area was actually slightly less by about 5% due to damaging the mortar during the setup. In Table 1, the nominal bed area is used in calculating the shear stress. A large variation exists in the shear strength of the bed joints, as is evidenced by the large standard deviation, though this should not be surprising given that there probably had not been much quality control of the construction of the infill walls. There was nothing visually obvious to account for the large deviation.

LABORATORY TESTS

Compressive Strength Tests

Fifteen HCT bricks were tested (11) per ASTM C-67-85 (2) parallel to the hollow cores. Both 8-in. and the 4-in. wide units were tested. The HCT units were cut in half through the cross-section prior to being tested. The results are presented in Table 2, where the compressive strength is given for both gross and net area. The net area of the tiles is approximately 44% of the gross area, so the net compressive strength values will be about 2.3 times higher than the gross compressive strength.

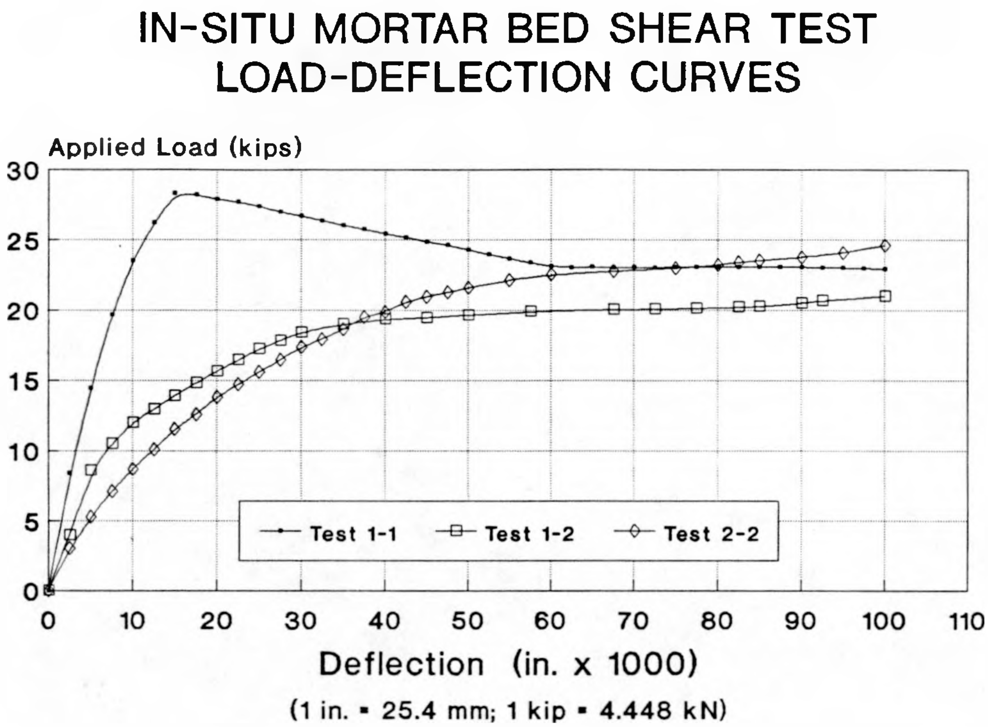

Fig.3 - Typical Load-Deflection Curves for the In-Situ Mortar Bed Tests

TABLE 1

SUMMARY OF IN-SITU MORTAR BED JOINT TESTS

Test No.	Bldg. No.	Failure Load (kips)	Shear Stress (psi)
1-1	1	28.4	99
1-2	1	20.3	71
1-3	1	45.0	156
2-1	2	17.0	59
2-2	2	25.5	88
2-3	2	31.5	109
3-1	2	19.5	68
3-2	2	40.0	139
3-3	2	29.5	102
mean	=	28.5	99
standard deviation	=	9.4	32.5

(1 kip = 4.448 kN; 1 psi = .006895 MPa)

TABLE 2

SUMMARY OF COMPRESSION STRENGTH TESTS

No.	Width (in.)	Length (in.)	Height (in.)	Area ($in.^2$)	Load (kips)	Compressive Strength Gross (psi)	Compressive Strength Net (psi)
1	8.19	6.00	11.72	49.14	188.5	3826	8745
2	8.13	6.00	11.94	48.78	122.0	2501	5717
3	8.25	6.04	12.25	49.83	96.5	1937	4427
4	8.19	6.13	12.19	49.14	90.0	1832	4188
5	8.19	6.13	12.38	50.20	77.5	1544	3529
6	8.25	6.13	12.38	50.27	90.0	1780	4069
7	8.13	6.00	12.25	48.78	92.5	1895	4331
8	8.07	5.94	12.13	47.94	106.5	2222	5079
9	8.00	5.75	12.13	46.00	128.0	2783	6361
10	7.88	6.25	12.19	49.25	97.0	1980	4526
11	4.00	5.81	12.50	23.24	85.0	3657	8359
12	3.94	6.13	12.31	24.15	57.5	2381	5442
13	3.94	6.00	12.25	23.64	79.0	3342	7639
14	4.00	5.88	12.25	23.52	66.0	2806	6414
15	4.13	6.25	12.00	25.81	111.5	4320	9874
			mean		=	2587	5913
			standard deviation		=	849	1942

(1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 psi = .006895 MPa)

Splitting Tensile Strength

Seven HCT bricks were tested (11) per ASTM C-1006 (3) in order to determine the splitting tensile strength of the individual units. Four were original clay tiles (dark red) removed from the walls and three were newer clay tiles (light red). The units were nominally 12-in. long x 12-in. deep and came in 8-in. and 4-in. widths. The bricks were tested in the same spatial orientation that they have in the walls (the holes running horizontally), with the line load running perpendicular to the axis of the holes and parallel to the brick width. Table 3 presents a summary of the splitting tensile tests. The splitting tensile strength, T, is calculated from the equation, $T = 2P/\pi A_{net}$, where P is the maximum applied load, and the net area is calculated as (WH - area holes), where the nominal hole dimensions are 3-in. x 3-in. The 8-in. clay tiles have six holes and the 4-in. clay tiles have three. The walls in the older buildings consist primarily of the older clay tile, though with facility modifications and occasional repairs, walls generally contain a small percentage of the newer tile. It can be seen from these tests that the newer clay tile appears to be weaker than the older tile.

TABLE 3

SUMMARY OF SPLITTING TENSILE STRENGTH TESTS

No.	Color	Gross Tile Dimensions W (in.)	L (in.)	H (in.)	Net Area (in.^{2})	Load (kips)	Split Tensile (psi)
1	LT. RED	8.27	12.34	12.40	41.3	162.6	251
2	DK. RED	8.24	12.21	12.19	40.3	241.5	382
3	LT. RED	8.27	13.00	12.28	39.0	127.5	208
4	DK. RED	8.14	12.09	12.10	38.3	292.0	486
5	DK. RED	8.00	12.24	12.13	38.4	259.0	430
6	LT. RED	4.17	12.59	12.66	21.5	76.0	225
7	DK. RED	4.28	11.75	11.81	24.5	147.0	382
				mean		=	338
				standard deviation		=	109

(1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 psi = .006895 MPa)

Diagonal Tension (Shear) Tests of Masonry Panels

The diagonal tension test was run on three masonry panels (11) removed from an existing facility wall. The panels were roughly 4-ft (1.22 m) square when removed from the wall, but were cut to a smaller dimension in order to smooth the edges. The test procedure, per ASTM E-519 (4), tests for the diagonal tensile (or shear) strength of the masonry panel by loading it in compression along one of the diagonals. The loading, which produces and equal amount of shear and normal stress, results in a diagonal tension failure with the panel splitting apart along the diagonal parallel to the direction of the load. Table 4 shows the results of these tests.

TABLE 4

DIAGONAL TENSION TESTS OF MASONRY PANELS

No.	W (in.)	H (in.)	t (in.)	Gage Length (in.)	Net Area (in.^{2})	Max. Load (lbs)	Shear Strain (in./in.)	Shear Stress (psi)	G (psi)
A	32.8	32.8	13.0	46.4	188.5	4391	.002543	16.5	6476
B	36.5	36.2	12.7	51.4	194.8	6922	.002684	25.1	9358
C	42.1	42.1	12.8	59.0	241.6	12407	.001423	36.3	25501

(1 in. = 25.4 mm; 1 lb = 4.448 N; 1 psi = .006895 MPa)

The tests all resulted in failure through the mortar joints, rather than through the webs of the clay tile units. While this is certainly the result that we expected, the data may be misleading Due to health and safety regulations, the panels were subject to a decontamination process, which included sandblasting and cleaning with powerful chemical cleansers and brushes. The researchers noticed significant damage to the wall samples including fractures at a few of the mortar-tile interfaces, damage which could have been caused not only during the cleansing process, but also during shipment to Nashville. The extent to which the damage affected the results is not obvious, but the shear stress results appeared to be low in comparison to the previous data.

Modulus of Elasticity in Compression

Tests were performed on 0.5-in. (12.7 mm) square x 1.0-in. (25.4 mm) long specimens cut from the original clay tiles. Nineteen specimens, cut from nine different tiles, were tested. In addition to the compression modulus of elasticity, the ultimate compressive strength was obtained. Table 5 presents the results of these tests. The compressive strengths obtained from these tests were greater than those previously measured using the ASTM C-67 method described earlier and tabulated in Table 2. The modulus of elasticity values in compression are similar to the data presented in Table 3-5 of Reference 7 for clay tile masonry.

TABLE 5

MODULUS OF ELASTICITY (COMPRESSION) RESULTS

Block No.	Area ($in.^2$)	Ultimate Load (lbs)	Compression Strength (psi)	Modulus of Elasticity (psi)
B	.2513	2745	10,925	3.40E+6
C	.2512	2805	11,170	3.51E+6
D	.2510	2610	10,400	3.12E+6
E	.2515	2850	11,333	3.22E+6
F	.2506	1830	7,304	2.09E+6
H	.2511	1845	7,348	2.25E+6
I	.2513	2625	10,447	2.88E+6
K	.2513	2635	10,485	3.02E+6
L	.2506	2640	10,535	3.02E+6
N	.2512	2805	11,168	3.29E+6

(1 in. = 25.4 mm; 1 lb = 4.448 N; 1 psi = .006895 MPa)

Note: All data is the average of tests on 2 samples, except for sample B, which had only one sample.

FUTURE TEST PROGRAM

The testing program for the HCT walls is still in the process of being defined as of this writing. The Department of Civil Engineering at The University of Tennessee will be serving as consultants to the project. In addition to the university personnel, Energy Systems has contacted consultants intimately familiar with masonry and masonry testing and expect to use them in a review capacity to the program. The number one priority is to find some HCT walls that can be used for testing. Most of the walls in these facilities are not readily available for full testing because they are in current use and have miscellaneous piping and electrical conduits hanging off them, which would be very difficult to remove. We are presently conducting a building to building search to identify walls that have potential for testing. The following type of tests are being considered for the project:

1. Low-amplitude impact tests to determine the natural frequency, mode shapes, and damping of the walls out-of-plane. The data will also be useful in evaluating the boundary conditions that are appropriate to use in numerical models.

2. In-situ testing to measure (a) the normal stresses existing in the walls using flat jacks, (b) in-plane bed joint shear tests similar to the ones already discussed, but also performed at the base of the wall and at the top girder to evaluate boundary conditions at these locations, (c) out-of-plane flexural tensile and compressive strength of the walls, and (d) in-plane wall flexural and shear strength and stiffness of the walls performed on both isolated masonry panels and on infilled steel frames.

3. Material property tests

4. Dynamic tests. There is a 2-axis shaker table available at Energy systems which could possibly be used to obtain some data. The table has been shut down for about five years, and would require some work to restart, but that prospect is currently being evaluated.

Once some walls have been found, then the type of testing that is suitable for each will be determined. While many of the existing walls are not available for full-scale tests because of the miscellaneous equipment hanging on them, it is possible that some testing, such as using the flat jacks to measure the in-situ compressive stress, could easily be performed on these walls.

SUMMARY

The data from the testing program is still being evaluated. The data from the diagonal tension tests appear to be highly suspect due to the poor condition of the panels at the time of the tests. There is also some question as to the value of the diagonal tension test considering that the panels have four free edges (except at the loading diagonal corners) which does not represent a realistic situation for an infill wall. The in-situ mortar bed shear tests need to be evaluated to determine the correlation between the results for the single block and the entire wall. The results of the other tests appear to be match reasonably with published data for hollow clay tile. We expect to be started on the rest of the testing program by the spring of 1990.

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