OUT-OF-PLANE BEHAVIOR OF HOLLOW CLAY TILE WALLS INFILLED BETWEEN STEEL FRAMES

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OUT-OF-PLANE BEHAVIOR OF HOLLOW CLAY TILE WALLS INFILLED BETWEEN STEEL FRAMES

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ABSTRACT

Several Buildings at the Department of Energy (DOE) Oak Ridge Y-12 Plant* rely on unreinforced hollow clay tile walls (HCTW) infilled between unbraced, non-moment resisting steel frames to resist natural phenomena forces, seismic and wind. One critical building relies on moment resisting steel frames in one direction while relying on unreinforced HCTWs infilled between the columns in the orthogonal direction to resist these forces. The HCTWs must act as shear walls while maintaining out-of-plane lateral stability. In assessing the safety of these buildings to seismic forces, several models to study the in- and out-of-plane effects were made and analyzed. The study of the moment resisting steel framed building indicated that bending stresses in the walls were induced by building drift and not by inertial forces per se. The discovery of this phenomenon was somewhat of a surprise in that the analysis performed is not typically used in design of these structures. The study indicated that the walls began to crack at their interface with the foundation at a low “g” level and that horizontal cracking at different elevations continued until the walls exhibited little bending resistance.

This paper presents the results of the study for out-of-plane behavior of unreinforced HCTWs infilled between adjacent moment resisting steel frames and discusses the problems of assessing the in-plane behavior given the horizontal cracks induced by building drift in the out-of-plane direction.

INTRODUCTION

The purpose of this study is to evaluate the behavior of a typical hollow clay tile wall (HCTW) panel infilled between steel trussed frames of Building 9212. The 9212 building (Figure 1) which is located at the Oak Ridge Y-12 Plant in Oak Ridge, Tennessee was constructed in the late 1940s and early 1950s. The original structure was completed in 1947 and consisted of the two story head house and four one story wings (A, B, C, and D) with basements. All of the wings were constructed of load carrying steel truss frames and non load bearing hollow clay tile walls between the frames from floor to roof. The roof construction is composed of trusses and purlins and supports a gypsum deck with built-up roofing. The basement, first floor and the footing are of concrete constructions. The footing bearing on rock which is a part of the Rome Formation of interbeded sandstone, siltstone, shale and dolomite. The basement, which is only seven feet deep from the first floor to the basement slab is a very stiff structure.

Two interior rows of piers spaced 12 foot centers support the first floor. The basement walls are ten

* Managed by Martin Marietta Energy System, Inc. for the U.S. Department of Energy under contract DE-AC05-84OR121400.
inches thick and are cast integrally with the piers supporting the main building columns. In 1948 the space between the wings was filled with the addition of wings A-1, B-1, C-1 and D-1 wing. These wings share the common HCTWs clay tile walls of the original A-D wings. A steel frame structure was constructed to support the new roofs, which are constructed of purlins supporting a gypsum deck with built-up roofing. There are no major structural ties of the newer wings with the older wings. The focus of this study is centered on the behavior of a typical HCTW panel wall of an original wing.

**MODELING CONSIDERATIONS**

The typical wing for analysis consideration is 264-ft long, 36-ft wide and 22-ft high. Eight feet deep rigid roof trusses spanning 36 ft between the column spaced 24 ft center to center along the length of the wing provide lateral stability. The bottom chord of the trusses are 12 ft 6-inch above the floor. To provide longitudinal stability the space between the frames was infilled with 13 inch HCTW as shown in Figure 2. Because the wing is long, narrow and symmetrical, it was found that an isolated single bay with proper boundary condition could be used for analysis to represent the wing.

This conclusion was reached after an initial analysis of a simplified planar steel model, Model A (Figure 3), was made. In this model wall mass was lumped at nodes 2, 3, 12, and 13 and the roof mass lumped at nodes 3, 5, 7, 11 and 13. Here 100% of the mass participates in the first mode having a frequency of 0.73 Hz. In the past, other consultants have also used this type of approach.
Figure 2: Isometric of a Typical HCTW Panel.

Figure 3a: Model A

Figure 3b: Mode Shape
Properties of HCTW.
The HCT wall was assumed to be homogeneous and isotropic.

The following material properties of HCTW were assumed for all models:

Modulus of Elasticity \( E_c = 1.21 \times 10^6 \)  
Weight Density \( \rho = 2.84 \times 10^{-2} \)  
Poisson’s Ratio \( \mu = 8.00 \times 10^{-2} \)  
Modulus of Rigidity \( G = 5.6 \times 10^5 \)  
Tensile Strength of Mortar = 40 psi

Typical Bay
Figure 2 shows an isometric of an isolated steel frame infilled with a HCTW panel. The steel frames consist of vertical W10x33 columns connected to eight feet deep rigid trusses at the top and bolted to concrete piers at the bottom. The construction of infilled HCTW is unique. The 13 inch thick HCTW consist of 12x12x8 and 12x12x4 inch thick tiles, using a running bond type construction, with the 8 and 4 inch tiles staggered vertically and horizontally as shown in Figure 4. The W10 column is encased in the wall as shown in Figure 5. Since the cores run horizontally, the mortar bed joint is 12 inches wide on both the top and bottom surfaces of tile.

Response Spectrum
The Nemack-Hall (1) response spectrum shape was used for analysis. It was computed for 12% damping and followed the recommendations given in Reference 2. It is anchored to 0.19g ground acceleration and the end points are given in Table 1. The response spectrum is shown in Figure 6.

MODELING TECHNIQUE
The analytical model is shown in Figure 7. The model consist of 108 node points, 72 plate bending elements, 24 beam elements and 20 truss elements. Proper boundary conditions, \( (\theta_x, \theta_y, \Delta_z) = 0 \) were applied to simulate the normal curvature of the wall under applied inertia loading considering bending in the first mode. The model was restrained against displacement in the Z- direction (i.e., only lateral modes were considered). The bottom of the wall was restrained in \( R_x; R_y; R_z; M_x; \) or \( M_z \) direction depending on the model. The steel columns were modeled as an integral part of the HCT wall. The wall was considered to be constructed of nominal 12 inch X 12 inch x 12 inch blocks using side construction with

<table>
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<th>POINT</th>
<th>FREQUENCY (Hz)</th>
<th>ACCELERATION (g's)</th>
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<tr>
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<td>0.19</td>
</tr>
<tr>
<td>A</td>
<td>33</td>
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</tr>
<tr>
<td>B</td>
<td>8</td>
<td>0.29</td>
</tr>
<tr>
<td>C</td>
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<td>D</td>
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<td>0.05</td>
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<tr>
<td>E</td>
<td>0.10</td>
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</table>
Figure 6: Response Spectrum Shape Scaled to 0.19g.

Figure 7: Analytical Model
nine cells, each 3 inch square. The actual dead load on the roof was obtained by field walkdowns, and included the weight of all equipment, utilities and roof. Several analytical models (1 – 4) with different boundary condition and nodal configuration were analyzed. A summary of the analyses is presented in Table 2. Figure 8 shows the mode shape of the various analytical models.

**Model A**

As previously stated this Model consist of a rigid steel frame with truss and all the wall mass was lumped at nodes 2, 3, 12, and 13 and the roof mass lumped at nodes 3, 5, 7, 11, and 13 as shown in Figure 3. The results of this analysis are shown in Table 2. Here 100% mass participates in the first mode having a frequency of 0.73 Hz.

**Model 1**

This model represents a typical model with steel frame and infilled HCTW fixed at the base. The maximum plate bending stress (188 psi, plate 19) occurred at the base. The “g” level to cause cracking is computed as follows:

\[ g = \left( \frac{40}{188} \right)(0.19) = 0.0409 \text{ “g”s} \]

where 40 is the tensile strength of the mortar and 0.19 is the “g” level to produce the 188 psi bending stress.

The fundamental frequency is 4.17 Hz as shown in Table 2. The mode shapes are shown in Figures 8a and 8b.

**Model 2**

Since, in Model 1, the base cracked at only 0.04g, Model 2 was analyzed with a pinned base condition. A maximum stress of 347 psi occurred at plate 22, the junction of the wall to the bottom chord of the truss. The “g” level to cause cracking is computed as follows:

\[ g = \left( \frac{40}{347} \right)(0.19) = 0.022 \text{ “g”s} \]

Because this “g” level is less than the fixed base case, the wall will more than likely crack shortly after the base has cracked. The mode shape is shown in Figures 8c and 8d and a frequency of 2.05 Hz.

**Model 3**

For this model it was assumed that the crack that formed in Model 2 had propagated across the entire length of the wall. The maximum stress of 399 psi occurred in plate 22. The “g” level to cause another crack is only 0.019 which is again lower than the previous model, Model 2. The mode shape for this model is shown in Figures 8e and 8f with frequency of 1.22 Hz.
Figure 8a: Model 1. Frequency – 4.17 Hz

Figure 8c: Model 2. Frequency – 2.05 Hz

Figure 8e: Model 3. Frequency – 1.22 Hz

Figure 8f: Model 4. Frequency – 0.98 Hz

Figure 8: Analytical Models Mode Shapes.
Model 4

Since it is obvious that the stiffness of structure is degrading very rapidly the final model was analyzed with an additional crack midway between the bottom of the truss and the base. The maximum bending stress of 183 occurred in plate 16. The 'g' level to cause yet another crack is only 0.042 computed as before. The mode shape for this model is shown in Figures 8g and 8h and has a frequency of 0.98 Hz which is approaching the 0.73 Hz, as computed in Model A.

CONCLUSIONS AND FINDINGS

The stresses in the wall, which cause stiffness degradation, are due to bending of steel column due to story drift and not inertial forces of the wall itself.

The lateral stiffness of the entire wing degrades rapidly with formation of horizontal cracks in wall.

The original Model A can be used to represent the actual behavior of the structural system for computing stress in the steel members.

The longitudinal stability of the wing depends on severely horizontally cracked walls. Further work is needed to determine the in-plane strength of infilled, cracked HCTW.

FOLLOW UP TESTING

The Center for Natural Phenomena Engineering, CNPE at Martin Marietta Energy System Inc. is involved in a Hollow Clay Tile Wall testing Program. A part of this program is to perform out-of-plane full scale testing of steel frames infilled with HCTW, followed by in-plane Testing to determine their residual capacities.

REFERENCES
