**NUMERICAL METHODS FOR ANALYSIS OF
CLAY TILE INFILLS**

**Prepared for the
Center for Natural Phenomena Engineering**

October 20, 1993

**For Presentation at the
Fourth DOE Natural Phenomena
Hazards Mitigation Conference
Atlanta, GA**

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

MASTER

DISTRIBUTION OF THIS DOCUMENT IS UNLIMITED

DISCLAIMER

This report was prepared as an account of work sponsored by an agency of the United States Government. Neither the United States Government nor any agency thereof, nor any of their employees, make any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, apparatus, product, or process disclosed, or represents that its use would not infringe privately owned rights. Reference herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise does not necessarily constitute or imply its endorsement, recommendation, or favoring by the United States Government or any agency thereof. The views and opinions of authors expressed herein do not necessarily state or reflect those of the United States Government or any agency thereof.

DISCLAIMER

Portions of this document may be illegible in electronic image products. Images are produced from the best available original document.

NUMERICAL METHODS FOR ANALYSIS OF CLAY TILE INFILLS

**Roger D. Flanagan
Michael A. Tenbus
Engineering Division
Martin Marietta Energy Systems, Inc.**

**Richard M. Bennett
Department of Civil Engineering
The University of Tennessee**

**Prepared for the
Center for Natural Phenomena Engineering**

October 20, 1993

**For Presentation at the Fourth DOE Natural Phenomena Hazards Mitigation Conference
Atlanta, GA**

**Oak Ridge Y-12 Plant
Oak Ridge, Tennessee 37831
managed by
Martin Marietta Energy Systems, Inc.
for the**

**U.S. DEPARTMENT OF ENERGY
under contract DE-AC05-84OR21400**

NUMERICAL METHODS FOR ANALYSIS OF CLAY TILE INFILLS

Roger D. Flanagan and Michael A. Tenbus
Martin Marietta Energy Systems, Inc.
P.O. Box 2009, Building 9111
Oak Ridge, TN 37831-8201

Richard M. Bennett
The University of Tennessee
221 Perkins Hall
Knoxville, TN 37966-2010

ABSTRACT

Recent Department of Energy requirements have led to a comprehensive evaluation of the industrial facilities at the Oak Ridge Y-12 Plant. The structures consist of simply connected steel frames infilled with structural clay tile walls. The objective of the evaluation was to determine the stability of the unreinforced infills, and whether they provide the lateral capacity necessary to resist the moderate seismic hazard at the site.

Due to lack of information on the behavior of structural clay tile infills, various large-scale tests were performed to investigate the in-plane, out-of-plane and combined in-plane and out-of-plane behavior. The results of these tests are briefly summarized, and the development of analytical guidelines based on these tests is given. Little interaction between in-plane and out-of-plane loads was observed, both in terms of stiffness and strength. Out-of-plane stability can be examined panel by panel based on arching action. Inter-story drift does not appear to present a stability problem for the type of infill construction investigated. In-plane behavior may be adequately modeled with a nonlinear compression strut.

A typical building is chosen for an illustrative application. The methodology and results of the seismic analysis are presented for this structure.

INTRODUCTION

Unreinforced clay tile infills present a complex structural system that significantly alters the response of a building. Although masonry infills have been studied by several researchers, the state of knowledge of infilled frame behavior is still somewhat limited. This is particularly true of clay tile infills as most of the research has focused on brick and concrete masonry.

To understand the behavior of clay tile infills, a comprehensive testing and analysis program was begun. The experimental program consisted of static testing of various specimens essential to the understanding of clay tile infill response under lateral seismic forces. Initial findings have led to preliminary recommendations for the analysis of clay tile infilled structures.

The typical construction being investigated is steel framing having no moment-resistant connections and infilled with unreinforced structural clay tile panels. The tile is laid with the cores horizontal (side construction) using running bond. Full width bed joints (Type N mortar) are provided but only faceshell mortar is used in the head joints. The masonry is bonded to the framing by packing mortar between the steel and masonry. No reinforcement is present in the masonry.

Both single and double wythe construction are prevalent. Single wythe construction consists of nominally 12"x12"x8" thick tile. The double wythe panels are made of 8" and 4" thick tiles with their location staggered in alternating courses. A gap of approximately 1.5" separates the wythes with no significant collar joint present.

MATERIAL PROPERTIES

Several tests were conducted to determine material constitutive properties. Results of unit tile testing is presented in Flanagan et al. [1]. Significant uniformity in the tile was observed for dimensional and compressive test parameters. Somewhat less uniformity was found for tension, suction, and bond strength parameters.

Masonry prisms consisting of 2' by 4' assemblages have been tested under uniaxial compressive loading. The behavior is markedly different in the instance where the compressive force is normal to the cores of the specimen versus being parallel to the specimen cores. As indicated in the stress-strain relationship of Figure 1, failure of prisms with the cores normal to the load is abrupt following a relatively linear behavior. Conversely, Figure 2 indicates a more gradual failure mechanism with significant postpeak capacity for prisms with the tile cores parallel to the compressive force. Table 1 gives a summary of the compressive test results for prisms of both orientations based on gross area.

IN-PLANE BEHAVIOR

Thirteen in-plane racking tests were performed on eleven large-scale clay tile infill specimens to determine the behavior. Two of the infills were repaired or retrofitted after the first test and retested under in-plane loads. The specimens consisted of portal frames constructed similar to the existing field conditions. The specimens were loaded cyclically in-plane until failure. Particular emphasis was placed on evaluating strength and stiffness degradation after cracking.

The tests were designed to evaluate a variety of engineering conditions including varying frame stiffness, single and double wythe construction, varying panel aspect ratio, offset panels, weak column/infill interface, and corner openings. A summary of the tests is presented in the schematic of Figure 3. Ultimate in-plane tension (pull) and compression (push) forces are shown for each test.

Specimens 1 through 3 and 4 and 5 were designed to envelope the in-situ infill stiffnesses for 8" and 13" walls respectively. A measure of infill stiffness, λ_h , proposed by Stafford-Smith and Carter [2] was used to estimate relative panel to frame stiffness. Specimen 2 which had a moderately stiff frame was used as a control specimen when evaluating other test parameters. With the exception of specimen 9, all of the test frames were oriented with the infill between the flanges and the columns bent about their weak axis under in-plane loading.

Specimen 9 represented infill framing in which the wall is butted against the outside face of the column flange. The column is bent about its strong axis under in-plane panel loading. This very stiff specimen represents an extreme instance of relative frame to panel stiffness.

Seismic forces in an actual structure are transferred from the steel framing to the clay tile walls. Thus, the in-plane loading of all specimens was through the centroid of the steel frame. Several types of interfaces exist between in-situ clay walls and the enclosing frames. Specimen 7 was designed to investigate the condition where the wall is offset from the centerline of the framing.

Specimen 10a was constructed with an approximate 1" gap between the infill and the columns. The beam interface was snugly packed with mortar similar to the other specimens. The resulting weak interface was devised to bound the highly variable construction practice and quality of workmanship in the field. After completion of the test, the wall was repaired. All damaged tile were replaced, and the corners reconstructed with solid concrete masonry units and Type S mortar, with snug mortar packing along the column web and beam flange. This new specimen, 10b, was retested as a potential retrofit scheme to strengthen the infill corners.

Specimens 2, 21a, and 17 enveloped the typical infill aspect ratios found in the field. Each panel was nominally 8 feet high with lengths of 8, 10, and 12 feet respectively. Specimen 21a was repaired after testing with all visibly damaged tile replaced using normal clay tile construction methods. The new specimen, 21b, was then tested in-plane to failure to assess the capacity of a repaired infill previously damaged by earthquake forces.

Finally specimen 24 is similar to specimen 21a except that a 2 feet square opening was left in the upper loaded corner of the panel. This configuration is typical of industrial facilities with openings for air ducts and other mechanical equipment.

A partial in-plane hysteresis curve is shown in Figure 4. At moderate to high loading, the infilled frame exhibited a pinched hysteretic curve composed of two stages. For small displacements, the infilled frame hysteretic curve was nearly identical to that of the bare frame. Larger displacements caused the infill to engage, greatly increasing the stiffness of the combined system. The pinching was due partly to interface degradation at the frame-panel boundary, but primarily due to plastic deformations of the masonry.

Six limit states of the clay tile infilled frame are identified in Figure 4. Although, these limit states occurred

at different load and displacement levels for other specimens, similar behavior was observed. As the incremental increasing displacements were applied to the frames, mortar cracking and separation at the base and top of the panel was observed, followed by cracking in the mortar joints of the upper corners. Next, diagonal mortar joint cracks developed in both directions of the cyclic loading. In the ensuing load cycles, parallel off-diagonal cracks developed, as the initial diagonal crack width was reduced. Subsequently, mortar crushing and tile cracking occurred in the upper corners. Ultimate capacity of the infilled frame was reached coincidentally with splitting of the upper corner tiles along the intersection of the web and faceshell. Finally, many of the upper course tiles split and the corner crushing continued as whole tiles were destroyed in the upper corners.

Preliminary observations from the series of in-plane tests are as follows.

(1) In-plane strength of the steel framing is increased dramatically by the addition of the clay tile infill. For many specimens, the ultimate strength of the infilled panel was an order of magnitude higher than the bare frame.

(2) In-plane stiffness of the steel framing is also increased dramatically with the addition of the clay tile infill. Under increasing displacement cycles, the secant stiffness reduced gradually from the beginning of the loading. The hysteretic loops were stable as indicated by the second cycle of a given displacement typically following the path of the first. For the larger specimens, there was a noticeable reduction in stiffness coincident with the onset of major diagonal cracking. Panel diagonal cracking occurred at a shear stress of 15-20 psi measured on the gross horizontal cross-section of the infill. For all specimens, there was a marked shift in stiffness as ultimate capacity of the infill was reached.

(3) Varying the steel frame stiffness had a significant effect on the stiffness of the infilled frame system. However, variations in frame stiffness had only a marginal effect on the ultimate capacity of the infilled frame system, after accounting for the different capacities of the steel frames. For the stiffest infill specimens, the ultimate capacity was reached at lower displacement levels than for the softer infilled frames.

(4) Varying the aspect ratio of the specimens by increasing the infill length increased the capacity of the infilled frame system slightly. The longer specimens also exhibited a more ductile post-peak behavior.

(5) Double wythe panels exhibited a similar failure pattern as the single wythe specimens. A significant separation of the wythes of the upper courses was observed as the tiles of each wythe failed independently in the latter portions of the load history. This follows logically as there was no appreciable collar joint or other special binding to prevent the wythes from responding separately. Little additional strength was observed in the double wythe panels as compared with the single wythe panels. Similar strength comparisons can be made for single and double wythe prisms with the load parallel to the tile cores.

(6) The in-plane stiffness and strength of an infill partly enclosed in the framing was approximately that of a concentrically constructed specimen times the ratio of the net area enclosed over the total net area.

(7) The presence of a weak interface consisting of nominal 1" gaps between the columns and the infill resulted in a softer specimen throughout the in-plane loading. As expected the lower displacement cycles were dominated by the frame behavior but exhibited a pinched hysteresis early in the loading due to the snug boundary at the beam/infill interface. The ultimate strength of the weak interface panel was only marginally smaller than that of a similar specimen with solid boundaries.

(8) An opening in the upper loaded corner of an infill resulted in a soft hysteretic curve for the push cycles of the in-plane loading. The push cycles were similar to those of the weak interface test. The stiffer pull cycles of the in-plane hysteresis were similar to those of a solid panel. A more dramatic postpeak strength degradation was also observed for the panel with an opening. The presence of a corner opening on the loaded side of the panel reduced the ultimate in-plane capacity by 25% in the pull direction and over 50% in the push direction.

(9) The repair of an infill (previously tested to failure) by replacing all damaged tile (upper two courses) resulted in a somewhat softer in-plane hysteresis and approximately 80% of ultimate capacity of an undamaged infill. The retrofit of an infill with a weak column/infill interface (previously tested to failure) by adding solid concrete masonry units in the upper corners increased the infill stiffness to that of a solid panel with no weak column/infill interface. The ultimate strength of the retrofitted panel was approximately 75% of an undamaged panel.

OUT-OF-PLANE BEHAVIOR

Eight infill panels were tested normal to the plane of the wall under either simulated drift or uniform lateral

pressure. Figure 5 provides a summary of the out-of-plane testing.

INTER-STORY DRIFT

The stability of infill panels under out-of-plane inter-story drift was evaluated by testing two large-scale specimens. Details of this study are contained in Flanagan and Bennett [3]. The specimens were subjected to cyclic out-of-plane displacements imposed at either the top or midheight of the columns. The specimens were then loaded in-plane to failure to determine the effect of the out-of-plane drift on in-plane stiffness and strength.

Specimen 11 was loaded near the top of the columns producing a cantilever type curvature. Increasing increments of displacement were imposed up to approximately 1.7 percent of the panel height. Minimal damage to the masonry was observed in the form of hairline cracks in the mortar joints.

Specimen 13 was first loaded near the midheight of the columns with the top of the specimen restrained. This produced a beam type curvature. Secondly, the upper restraint was removed and the infill was subjected to a cantilever curvature loaded near midheight. The maximum midheight displacement imposed was approximately 1.2 percent of the height of measurement. Again, minimal damage to the masonry was observed.

There was little relative movement of the infill panels with respect to the steel framing as the out-of-plane drift loading was applied. Infill panels constructed snugly to the framing, but without ties or other reinforcing, were shown to remain stable when subjected to cyclic out-of-plane drift displacements within the range that a typical infill might experience.

The results of the following in-plane tests were compared to specimen 2 which was similar except having been loaded only in-plane to failure. The in-plane failure modes were similar to that of the control specimen. Only minimal degradation of in-plane stiffness and strength resulted from the out-of-plane drift displacements.

UNIFORM LATERAL PRESSURE

Three infill panels were tested out-of-plane with a uniform lateral loading supplied by an air bag. The tests were designed to simulate the inertial effects of the panel normal to its plane. Each infilled frame was nominally 8 feet by 8 feet. Three different wall thickness were tested; two single wythe panels of 4" and 8" and one 13" double wythe panel.

Static load-unload cycles were applied to the specimens until ultimate capacities were reached and overall panel stability began to deteriorate. Two cycles were performed at each increasing pressure increment. The pressure versus midpanel displacement relationship for specimen 18 is shown in Figure 6.

The behavior was dominated by arching action of the masonry. With the presence of panel edge restraint, membrane forces developed in the infill as fracture lines divided the panel into separate segments. As the segments moved out of the plane of the wall and rotated about their boundaries, arching forces continued to develop until failure in the masonry occurred along the boundaries.

Typically vertical arching developed as horizontal cracking appeared in the bedjoint at midheight of the panels. Ultimate capacity was reached as the upper course tiles failed in a shear and tension mode. After reloading the specimen, a similar type failure of the bottom course tiles occurred. With the upper and lower course tiles damaged, horizontal arching developed allowing for considerable postpeak strength and stability.

The test specimens were designed to envelope the range of typical infill height to thickness ratios. Allowing that each specimen was the same height but with varying thickness, the ultimate capacities appeared to vary with square of the panel thickness.

Two additional specimens were tested using sequential in-plane racking and out-of-plane air bag loadings. Specimen 19 was first loaded to approximately 75% of ultimate in-plane capacity using incremental cyclic displacements. This partially damaged infill was then subjected to uniform lateral pressure loadings. The ultimate lateral capacity was approximately 85% of a similar undamaged specimen.

Specimen 20 was first loaded with an uniform out-of-plane pressure of approximately 75% of ultimate. The infill was then subjected to in-plane racking forces and achieved approximately the same ultimate capacity as a similar undamaged specimen.

BI-DIRECTIONAL BEHAVIOR

Specimen 23 was loaded with combined in-plane and out-of-plane forces. After a series of cyclic racking loads were completed, the in-plane drift was held as uniform lateral pressure load-unload cycles were applied. This sequence was repeated until ultimate in-plane capacity of the infill was reached.

The maximum out-of-plane load obtained was 2.3 psi, or 60% of the pure out-of-plane capacity, under an in-plane drift of 0.002. However, the system was still able to withstand an out-of-plane load of 1.2 psi at an in-plane drift of 0.015. Although some interaction of in-plane and out-of-plane forces exists, this interaction does not appear to be significant at low to moderate levels of loading.

ANALYSIS METHODOLOGY

For analysis of complex industrial facilities constructed of steel framing infilled with structural clay tile, it assumed that a three dimensional analysis may be required due to geometric or mass eccentricities contributing to torsion. Typically seismic analyses of such facilities are performed using equivalent static or response spectra modal analysis. The methodology for seismic evaluation of clay tile infills described below is applicable for these instances as well as simpler conditions where two dimensional analysis or cruder approximate techniques are employed.

The process of evaluating infilled frames is as follows. The masonry infill is idealized as an equivalent compression strut in the plane of the wall. Because the stiffness of the infill is a function of the previous load history, the strut model is necessarily nonlinear. After seismic loads are determined, the in-plane capacity of the infill is assessed based on allowable strain in the masonry. The resulting out-of-plane drift stability is assessed by comparison with allowable ratios. Out-of-plane inertial strength is evaluated by comparing maximum panel accelerations with equivalent acceleration levels computed from uniform lateral load equations incorporating arching action. Frame members are evaluated based on the controlling in-plane drift and any significant thrust forces due to arching.

The conditions required for a compression strut to develop must be present. Openings, interface gaps, and other discontinuities may affect development of a compression diagonal. Equivalent strut methods are typically formulated without consideration of vertical forces. Thus, vertical loads should be omitted during lateral load analysis and superimposed later.

Attention to detail in modeling the compression strut is also important. For instance, the use of a nonsymmetric strut may result in erroneous axial forces in members at reentrant corners of buildings. It is recommended that a compression only strut be used in each direction. If a compression only element is not available, a tension-compression truss member may be used with half the strut area in each diagonal direction.

The compression strut formulation suggested is similar to that developed by Stafford-Smith and Carter [2]. The method has been extended to include nonlinear behavior of the infill. The contribution to lateral stiffness may be computed using the relationships of Equation 1-4. Refer to Figure 7 for parameter details.

The length of column bearing on the infill, α (in), is given by

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

where

$$\lambda = \sqrt[4]{\frac{E_m t \sin 2\theta}{4 E_s I_c h'}} \quad (2)$$

in which E_m is the gross elastic modulus of the masonry (psi), $E_s I_c$ the flexural rigidity of the column (psi, in⁴), θ the slope of the infill diagonal to the horizontal, t the infill gross thickness (in), and h' the infill height (in). The width of the equivalent strut, w (in), is then determined by

$$w = \frac{\alpha}{\cos \theta} \quad (3)$$

and the area of the equivalent strut, a (in²) by

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

It is suggested that the length of contact be limited to 20% of the panel height (h'). The parameter C varies with the in-plane drift displacement. Typical values of C are given in Table 2.

For specimens in the range of height tested, ultimate in-plane capacity of the clay tile infills should be limited by a 1" in-plane drift displacement. This corresponds to an in-plane drift of approximately 1%. Caution should be exercised in extrapolating this drift limit for extremely tall panels. In a recent test of a 24 feet tall by 28 feet long clay tile infill specimen of similar construction [4], the ultimate capacity was reached at a much smaller in-plane displacement. It is believed that this ultimate load corresponds to the panel cracking load exhibited in the smaller specimens. In the large panel, the panel cracking load was reached early in the loading sequence and was greater than the corner crushing mode of failure. This type of failure may be less prevalent in slightly cracked panels typical of existing masonry infill construction. Precracked

panels will more likely be controlled by corner crushing modes of failure and should exhibit more gradual stiffness reductions and strength increases through cycles of in-plane loading.

An upper limit of 50 kips for single wythe panels and 60 kips for double wythe panels should also be used for the in-plane capacity of the infills (maximum horizontal component of equivalent strut force). Stiff frames creating a greater length of contact along the infill will be limited by this criteria and will reach predicted ultimate capacity at a lower in-plane displacement.

Beyond peak capacity the in-plane strength of the infill is assumed to reduce to 75% of peak at an in-plane drift of 1.5 times the displacement at peak. Postpeak testing of the infilled frames indicates significant capacity well beyond a displacement of 1.5 times that reached at peak capacity. However, this displacement serves as a practical limit in predicting repairable damage levels of the masonry as well as an indicator of potential damage to the columns.

Specimen 17 has been used to illustrate the nonlinear compressive strut method. The results of this equivalent strut method compare well with the envelope of observed in-plane hysteretic behavior. Figure 8 presents a comparison of test versus analytical methodology. The test specimen consisted of partially restrained connections, so a fixed base and pinned base calculation with the compressive strut formulation is given. The masonry elastic modulus used in the calculations is from the prism tests with the compressive load parallel to the tile cores. Although the state of stress in the corners of the masonry panels is biaxial at low precracking load levels, the behavior of the panel appears to be more influenced by the compressive behavior in the lateral direction at higher load levels.

Out-of-plane story drifts due to the relative top-bottom displacements of orthogonal walls may be evaluated based on an allowable drift ratio of 1%. Panels drifts exceeding 1% may be stable but require more detailed evaluation.

Out-of-plane inertial loads are evaluated for individual panels based on the maximum acceleration computed for a given elevation. The uniform lateral load capacity, q (psi), of an individual infill may be determined from

$$q = 0.8 f'_m{}^{0.75} t^2 \left(\frac{\alpha}{l'^{2.5}} + \frac{\beta}{h'^{2.5}} \right) \quad (5)$$

in which f'_m is the masonry gross compressive strength (psi) and l' the infill length (in). Originally developed by Dawe

and Seah [5], this relationship incorporates arching action for infills supported on all four boundaries.

The parameter α is a measure of the relative stiffness of the bounding columns

$$\alpha = \frac{1}{h'} (E_s I_c h'^2 + G_s J_c t h')^{0.25} < 35 \quad (6)$$

in which G_s is the column shear modulus, and J_c the column torsional constant. The parameter β is a measure of the relative stiffness of the bounding beam

$$\beta = \frac{1}{l'} (E_s I_b l'^2 + G_s J_b t l')^{0.25} < 35 \quad (7)$$

in which I_b is the beam moment of inertia and J_b is the beam torsion constant. Effective acceleration limits may be computed from the ultimate static pressures and compared with the results of the dynamic analysis.

Figure 9 presents a comparison of this uniform lateral capacity formulation for various h/t ratios versus the results of three static pressure tests. Good correlation of test data is made with these arching equations. The masonry strength used in the calculations is from the prism tests with the compressive load normal to the cores. This is appropriate as the primary behavior mode of the panels was vertical arching (horizontal cracking) prior to peak capacity.

Frame member forces may be computed from the in-plane and out-of-plane behavioral conditions. Forces in the columns due to in-plane loading may be estimated based on the in-plane drift of the frame and the contact forces computed along the column infill boundary. Similarly, uplift forces in the beam may be computed based on the contact along the beam/infill boundary. Depending on the construction, the bending forces in the beam resulting from lateral loading may be negated by those resulting from vertical loads.

For out-of-plane drift, the frame forces are estimated directly from the level of frame drift. In the case of out-of-plane inertial loads, the frame thrust forces due to arching action require more detailed evaluation. Depending on the vertical loading in the beam and the presence of adjacent infills along the columns, these thrust forces may also be negated.

ILLUSTRATIVE EXAMPLE

A three dimensional modal analysis of a prototypical industrial facility was performed using the previously described analysis methodology. Although many

simplifications have been made in the details of the structure, this prototypical building is of the same general size, mass, eccentricity and construction as many existing facilities.

A schematic showing plan and elevation views of the structure is given in Figure 10. The building includes three full stories with a penthouse creating part of a fourth floor. W16x67 columns are used throughout with W24x76 floor beams and W16x45 roof purlins. The steel frame is infilled with structural clay tile around the perimeter of the building and in two interior bays. A open area with no masonry is assumed in the two bays comprising the reentrant corner of the building.

Infill walls in the first story are 13" double wythe construction and the top three stories are 8" single wythe construction. The double wythe walls have been assumed to weigh 52 psf and the single wythe walls 32 psf. Total floor loads have been taken as 120 psf and roof loads taken as 30 psf.

Two seismic inputs were used in the analysis. The first is a 0.15g Newmark-Hall response spectra and the second is a 0.18g site specific response spectra developed for the Oak Ridge, TN area and incorporating eastern U.S. earthquake characteristics and the latest Department of Energy hazard criteria. Figure 11 provides a graph of the two input spectra for 10% damping. These spectra were used for both horizontal acceleration inputs and the vertical input was assumed to be 2/3 of the horizontal. The modal analysis results for different spectra directions and modes were combined using the square root of the sum of the squares method (SRSS).

Fundamental frequencies of the structure are presented in Table 3. There is a significant difference in the building frequencies for the two analyses since the stiffness of the infills is nonlinear and based on the strain in the masonry. Sufficient modes were used in the analysis to obtain at least 95% mass participation. An amplified displaced shape of the structure under Newmark-Hall is shown in Figure 12. For the in-plane evaluation of the masonry infills, column line 3 bay CD has been highlighted. See Tables 4 and 5 for details. The Oak Ridge site specific earthquake has less acceleration input for lower frequency ranges and thus the in-plane drift and compression strut forces are less than for the Newmark-Hall spectra. Using the in-plane drift displacements or C values, the masonry damage levels may be roughly correlated with those in Table 2. For Newmark-Hall, the lower stories are near ultimate capacity and some tile cracking in the corners would be expected. For Oak Ridge, only diagonal cracking would be expected.

Maximum out-of-plane nodal accelerations and their locations are tabulated for both seismic inputs in Table 6. Since the frequency of the panel is significantly higher than the building structure, the maximum panel acceleration is approximately the nodal acceleration. Using equation 5, the capacity is 0.44 psi or 2g, which is significantly greater than predicted panel accelerations. Therefore, no panel stability problems are expected.

CONCLUSIONS

A rational method for seismic analysis of steel frames infilled with structural clay tile is presented. The methodology has been validated with a significant amount of large-scale experimental results. The method accommodates standard techniques for dynamic analysis while incorporating the nonlinear behavior of masonry infills in a computationally efficient manner.

ACKNOWLEDGEMENTS

This work was sponsored by the Center for Natural Phenomena Engineering, Martin Marietta Energy Systems, Inc., under contract with the U.S. Department of Energy.

REFERENCES

- [1] Flanagan, R. D., Butala, M. B., and Bennett, R. M. "Properties of Structural Clay Load-Bearing Wall Tile", Proceedings of the Sixth North American Masonry Conference, Philadelphia, PA, 87-98, 1993.
- [2] Stafford-Smith, B., and Carter, C. "A Method for the Analysis of Infilled Frames." Proceedings of the Institution of Civil Engineers, 44, 31-48, 1969.
- [3] Flanagan, R. D., and Bennett, R. M. "Inter-Story Drift Effects on the In-Plane Capacity of Infilled Frames," Proceedings of the Third International Masonry Conference, London, England, 1992.
- [4] Henderson, R. C., Jones, W. D., and Porter, M. L. "Factors Affecting the Ductility of Double-Wythe Masonry Infills Subjected to Seismic Drift," Proceedings of the Sixth North American Masonry Conference, Philadelphia, PA, 1433-1438, 1993.
- [5] Dawe, J. L., and Seah, C. K. "Out-of-Plane Resistance of Concrete Masonry Infilled Panels," Canadian Journal of Civil Engineering, 16, 854-864, 1989.

Table 1 Summary of Prism Compression Tests

Parameter	8" Prism		13" Prism	
	Parallel to cores	Normal to cores	Parallel to cores	Normal to cores
f'_m (psi)	436	810	372	332
E_m (ksi)	329	768	410	731
Strain at f'_m	0.00222	0.00135	0.00125	0.00088

Table 2 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 Disp @ Ultimate	Tile Faceshell Spalling

Table 3 Summary of Fundamental Natural Frequencies

0.15g Newmark Hall Spectra				0.18g Oak Ridge Site Specific Spectra			
NS	EW	VER	% Mass	NS	EW	VER	% Mass
	1.2 Hz		90		1.6 Hz		88
2.2 Hz			78	2.6 Hz			56
				2.8 Hz			31
		11.9 Hz	26			12.1 Hz	28

Table 4 Summary of In-Plane Forces for Column Line 3 Bay CD - 0.15g Newmark Hall

Elevation	Story	C	Δ (in)	δ (in)	Strut Force (k)	Strut Allowable (k)
1060			1.36			
	4	7		0.08	8.3	58.3
1045			1.28			
	3	7		0.15	16.5	58.3
1030			1.13			
	2	14		0.54	28.4	58.3
1015			0.60			
	1	16		0.60	46.6	70.0
1000			0			

Table 5 Summary of In-Plane Forces for Column Line 3 Bay CD - 0.18g Oak Ridge

Elevation	Story	C	Δ (in)	δ (in)	Strut Force (k)	Strut Allowable (k)
1060			0.35			
	4	5		0.03	7.7	58.3
1045			0.32			
	3	5		0.05	8.8	58.3
1030			0.27			
	2	7		0.13	14.1	58.3
1015			0.14			
	1	7		0.14	24.1	70.0
1000			0			

Table 6 Summary of Maximum Nodal Accelerations

0.15g Newmark Hall Spectra				0.18g Oak Ridge Spectra			
Direction	Acceleration (g)	Location	Elevation	Direction	Acceleration (g)	Location	Elevation
NS	0.39	A1	1045	NS	0.25	C1	1060
EW	0.35	B5	1045	EW	0.33	A5	1045
VER	0.23	C2	1060	VER	0.35	C2	1060

Figure 1 Prism Compression (Cores Normal to Load)

Figure 2 Prism Compression (Cores Parallel to Load)

Figure 3 Summary of In-Plane Testing

Figure 4 Partial In-Plane Hysteresis of Specimen 17

Figure 5 Summary of Out-of-Plane Testing

Figure 6 Specimen 18 Uniform Lateral Load

Figure 7 Equivalent Compressive Strut

Figure 8 Specimen 17 In-Plane Hysteresis

Figure 9 Uniform Lateral Load Analysis

Figure 10 Illustrative Example

Figure 11 Response Spectra Input

Figure 12 Structure Displaced Shape

DISTRIBUTION:

J. E. Beavers
R. M. Bennett
D. R. Denton
R. D. Flanagan (2)
K. E. Fricke
M. S. Guthrie (RC)
R. J. Hunt
W. D. Jones
A. K. Lee/DOE-OTSI (2)
W. C. Stoddart
Y-12 Central Files