Validation of Finite-Element Analyses for Storm Shelters

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Abstract: The Federal Emergency Management Agency *Publication 320*, *Taking shelter from the storm: Building a safe room inside your house*, presents a number of prescriptive designs for residential tornado shelters and specifies building materials commonly found in residential construction in the United States. The design and structural analysis of the shelters was based on simplified and conservative analytical methods and on the results of numerous impact tests on shelter components at the Wind Engineering Research Center at Texas Tech University. This paper compares predictions of structural displacements using a commercial finite-element analysis software package with experimental data taken from the full-scale testing of the aboveground concrete masonry unit (CMU) and timber-steel shelters. The reliability and usefulness of the finite-element analysis method in analyzing aboveground residential shelters under extreme wind loading is verified by these results. It is therefore suggested that finite-element analysis has the potential to be used in designing CMU and timber-steel shelters of different sizes and configurations without the need for physical testing to design loads.

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Introduction

Hurricanes, tornadoes, and related hazards cause dozens of deaths and billions of dollars in property damage each year in the United States. Almost every state has been affected by such extreme windstorms. The series of tornadoes that struck the Oklahoma City area on May 3, 1999 (Fig. 1) were some of the most expensive tornadoes in U.S. history, causing over $1 billion in damage and destroying over 2,500 structures.

Poststorm documentation studies conducted by Texas Tech University since 1974 revealed that, occasionally, a small interior room of a residence remained standing even when surrounding buildings were destroyed (Fig. 2). It was observed that such rooms could be designed to offer a very high degree of occupant protection if additional stiffening and hardening were provided in order to increase the structural integrity and debris impact resistance. Hence the concept of an "In-Residence" shelter was born (Fig. 3). The first publication of the concept appeared in *Civil Engineering* magazine in 1974 (Kiesling and Goolsby 1974).

Because of limited resources and manpower, development of the residential shelter concept continued at a slow pace for a number of years, and few people outside the research community were aware of the work being done. Public interest greatly increased with a *Dateline NBC* program following the 1997 Jarrell, Tex. tornado. The program documented the total destruction of the residential subdivision outside Jarrell and then featured photos of debris impacts on shelter components and illuminated the concept of the aboveground storm shelter. The Federal Emergency Management Agency (FEMA) further popularized the concept by publishing a booklet, FEMA 320: *Taking shelter from the storm: Building a safe room inside your house* (Federal Emergency Management Agency 1999). This publication presents performance criteria and several prescriptive designs for small residential shelters. It was available for use following the May 1999 tornadoes that struck the Oklahoma City area. Designs in FEMA 320 are based on 112 m/s (250 mph), ground-level design wind speeds. Based on observed damage, it is felt that the highest ground level wind speeds are approximately 90 m/s (200 mph); hence designing shelters for 112 m/s (250 mph) winds provides a significant margin of safety for worst-case tornadoes. As a result of several field site investigations after tornadoes and hurricanes, it was found that a mean weight of wood missiles corresponded to a 6.8 kg (15 lb), 50 mm × 100 mm (2 in. × 4 in.) timber plank. Many tests have been conducted in the debris impact facility at Texas Tech University. Different barriers and configurations have been tested. The successful barriers, i.e., those that stopped the

Fig. 1. Damage field in Oklahoma City

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Fig. 2. Inspiration of the shelter concept

Table 1. Impact Test Results for Different Shelter Construction Types

Target name	Missile speed (mph)	Damage description
4-in. thick pea-gravel concrete with #4 reinforcement	<162	Missiles repercussed
	162	Threshold observed
	>162	Target perforated
6-in. CMU reinforced with grout and #4 bars every cell	<130	Missiles repercussed
	130–137	Threshold observed
	>137	Target perforated
Two layers 3/4-in. plywood and one layer 14 gauge steel on doubled 2×4 stud frame	<130	Missiles repercussed
	130–133	Threshold observed
	>133	Target perforated

missiles, were incorporated in different documents and design guides such as FEMA 361: *Design and construction guidance for community shelters* (Federal Emergency Management Agency 2000a) and others (Federal Emergency Management Agency 2000b; NSSA 2001). A sample of the impact test results for different types of sections is shown in Table 1. Shelter walls and roof were designed to meet the requirements of the debris impact criterion which is a 6.8 kg (15 lb), 50 mm×100 mm (2 in. ×4 in.) wooden board traveling at 45 m/s (100 mph) and striking normal to the surface impact area. Wind pressures on the shelters were obtained using the provisions of ASCE 7-98 (ASCE 1998). FEMA 361 gives guidance for determining wind-induced pressures on large shelters and allows for lesser wind speeds. Since this publication is intended to guide professional designers, little emphasis is placed on prescriptive designs.

Background and Scope of Research

Formulation of FEMA 320 shelter design was based on several factors. First, the shelter had to remain intact during the extreme wind event (taken as a 112 m/s, 3-s gust). This requirement takes into account performance of the basic shelter structure—walls, roof, attachment to foundation, and door. Second, the shelter had to resist overturning. This is a function of the competency of the attachment of the shelter to its foundation. Third, the walls and door had to resist design missile (debris) impact, protecting the occupants within.

Full-scale structural testing of timber-steel and concrete masonry unit (CMU) shelters under design loads imposed by simulated wind pressures has been performed (Pierce 2001; Zain 2003), as have debris impact tests on shelter wall specimens. Qualification of shelter design has therefore been an empirical process. Experimental verification was considered necessary to validate the design methods used. This is an expensive and time-consuming process, and slows the pace of development for the design of larger (i.e., "community") shelters. Validation of analytical modeling techniques through calibration against existing shelter test data can therefore speed adoption of shelter designs of different sizes and configurations without requiring extensive testing.

The purpose of this study was therefore to validate the use of a representative commercial finite-element analysis software package to represent the response of a storm shelter under its design wind loading. To this end, the ALGOR software package was used to model full-scale testing of both CMU and timber-steel shelters tested at Texas Tech University. It should be noted that the current research dealt with structural wind loads; debris impact loading was not considered.

Full-Scale Tests

The full-scale CMU shelter was tested at Texas Tech University, Zain (2003). The CMU shelter was based on the prescriptive designs and construction plans presented in FEMA 320. The shelter was 2.44 m×2.44 m×2.44 m (8 ft×8 ft×8 ft) and was built onto an existing 304.8 mm thick (1 ft) reinforced concrete slab. Fig. 4 shows the plan view of the shelter. The roof of the shelter

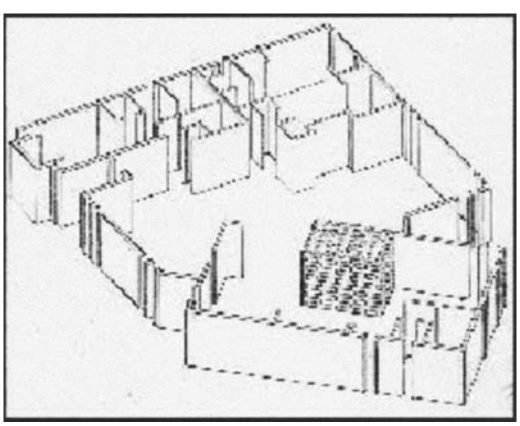

Fig. 3. In-residence shelter concept

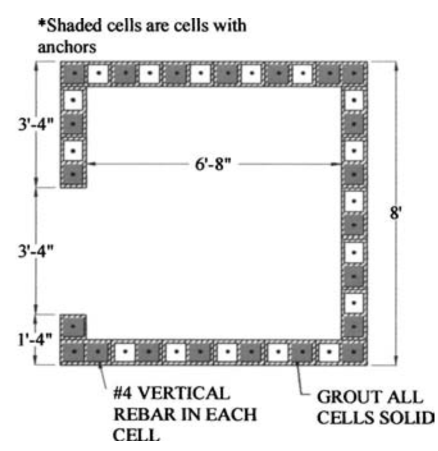

Fig. 4. Plan view of CMU shelter

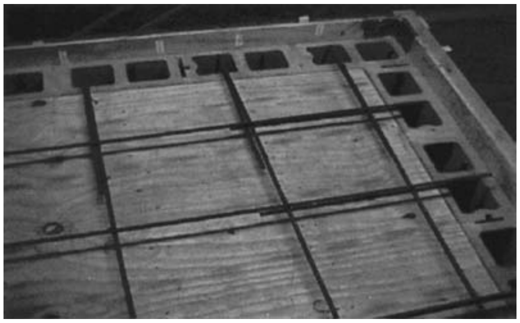

Fig. 5. Roof reinforcement in CMU shelter

was a reinforced concrete roof slab 101.5 mm (4 in.) thick. Reinforcement (#4 Grade 60 bars) was placed at the midthickness of the slab in two directions at 406 mm (16 in.) o.c. The roof slab was anchored to the tops of the walls by extending and bending the wall rebar into the slab. The slab reinforcement and the bent rebar were connected for a length of 508 mm (20 in.) to provide the required anchorage length. Fig. 5 shows the roof slab reinforcement of the CMU shelter. Eight-in.-thick concrete masonry blocks were used to build the walls of the shelter. The walls were reinforced by placing #4 rebar vertically in each cell of the concrete masonry. The cells were grouted with pea gravel concrete. The walls of the shelter were anchored to the slab by #4 rebar dowels. These dowels were placed along the center of the wall at 406 mm (16 in.) o.c. Fig. 6 shows the shelter walls and dowels.

A $914.4\ \text{mm} \times 2{,}032\ \text{mm} \times 44.5\ \text{mm}$ ($36 \times \text{in.} \times 80\ \text{in.} \times 1\text{-}3/4\ \text{in.}$) steel door was used. The door frame was fixed into the masonry wall using masonry tee (waffle style) anchors. The door was fastened with three latched bars to ensure that it would not fail under the pressure during the test (this test was intended to validate the use of FEA modeling for the overall shelter structure, not the door). The latched bars were placed at the top, middle, and bottom of the door.

The wood-frame with plywood and steel sheathing full-scale test was performed at the WERC at Texas Tech University by Pierce (2001). The dimensions of this shelter were $2.44\ \text{m} \times 2.44\ \text{m} \times 2.44\ \text{m}$ ($8\ \text{ft} \times 8\ \text{ft} \times 8\ \text{ft}$). The wall framing members were constructed of nominal $50.8\ \text{mm} \times 101.6\ \text{mm}$ ($2\ \text{in.} \times 4\ \text{in.}$) double studs. These were placed at 406 mm (16 in.) on centers. The top and the base plates were also made of nominal $50.8\ \text{mm} \times 101.6\ \text{mm}$ ($2\ \text{in.} \times 4\ \text{in.}$) double studs. The studs were nailed to the base plates with 16d nails during construction, and then connected using "H6" type Simpson Strong Tie connectors. The roof framing was built of joists with nominal $50.8\ \text{mm} \times 152.4\ \text{mm}$ ($2\ \text{in.} \times 6\ \text{in.}$) lumber placed at 406 mm (16 in.) on center. This spacing lined up each joist to be above a set of double studs. The joists were connected to wall studs using "H6" Simpson Strong Tie connectors. The sheathing material for the entire shelter consisted of a layer of 14-gauge steel and two layers of 19 mm (3/4 in.) plywood. The layer of steel was the first layer attached to the studs and joists. The plywood layers were attached such that the stiff directions in bending were perpendicular. The sheathing was attached with 76 mm (3 in.) long number 8 screws. The sill plates of the shelter were connected to the concrete slab using 12.7 mm (1/2 in.) diameter expansion anchors. The anchors were spaced at 406 mm (16 in.) on centers.

Fig. 7. External reaction frame

In both tests, steel reaction frames (external, and within the shelter) were used to support airbags, which were pressurized to simulate the wind loads on the shelter, between the load frames and the walls and roof. Internal and external reaction frames were used in the test. The external reaction frame (Fig. 7) placed outside the shelter was used to support the pressurizing airbags placed between the frame and windward wall of the shelter to simulate the positive pressure on the windward wall. The $2.44\ \text{m} \times 2.44\ \text{m}$ ($8\ \text{ft} \times 8\ \text{ft}$) exterior reaction frame was placed with 101.5 mm (4 in.) between it and the shelter. The interior frame was used to support airbags that simulated the negative wind pressures on the side and leeward walls, and roof of the shelter. Fig. 8 is a drawing of the general layout of the reaction frames and shelter.

Waterbed bladders were tested as airbags and found able to apply the needed pressures. The airbags were pressurized using an air compressor and tank that was connected to a manifold and a system of plastic tubes. This system of tubes allowed for different loading pressures corresponding to the four different loading

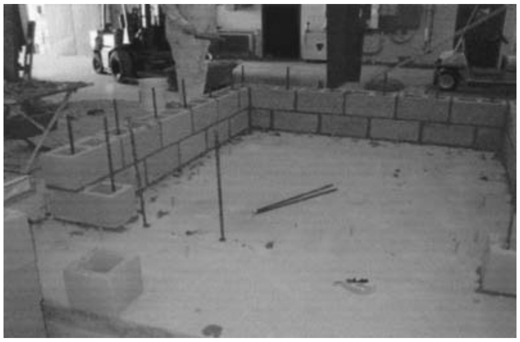

Fig. 6. Shelter walls dowels

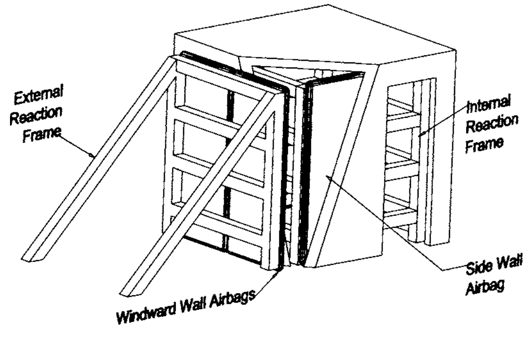

Fig. 8. Reaction frame and airbag loading system (door, and door and roof airbags omitted for clarity)

Fig. 9. Full-scale CMU specimen with instrument frame

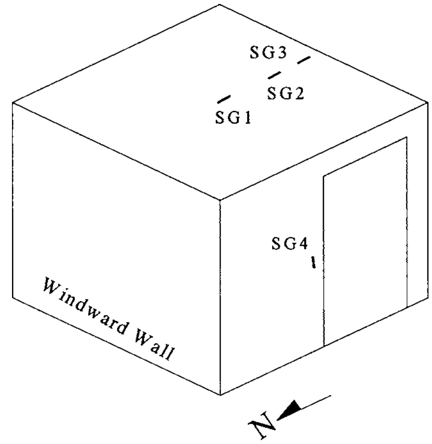

Fig. 11. Strain gauge locations

zones of the shelter (windward and leeward walls, sidewalls, and roof). Ten $1.22 \text{ m} \times 2.44 \text{ m} \times 152.4 \text{ mm}$ $(4 \text{ ft} \times 8 \text{ ft} \times 6 \text{ in.})$ airbags were used (two along each wall and two loading the roof). Differential pressure transducers were used to measure the amount of pressure that was applied to each loading zone.

A 305 mm (1 ft) thick concrete slab supported the shelter and both reaction frames. The slab dimensions were $3.66 \text{ m} \times 4.88 \text{ m}$ $(12 \text{ ft} \times 16 \text{ ft})$ with reinforcement bars at 304.8 mm (12 in.) on center each way at 81 mm (3-3/16 in.) below the surface and at 152 mm (6 in.) on center each way at 219 mm (8-5/8 in.) below the surface. The reaction frames were connected with epoxy cement to the slab using anchor bolts. The shelter was anchored to the slab according to the construction plans in FEMA 320.

Once the shelter was complete, a wooden frame was built around it to hold the instrumentation (Fig. 9). Linear variable differential transducers (LVDTs) measured deflections of the walls and roof (Fig. 10). Foil-resistance strain gauges measured roof strains and strains adjacent to the door frame (Fig. 11). A total of 15 LVDTs and four strain gauges were used. LabView (2002) was used to record and display the data. Pressurizing of the airbags was done incrementally to pressures corresponding to wind speeds ranging from 44.7 to 200 m/s (100 to 450 mph) (Table 2).

During the two tests, all the measurements of the LVDTs were written to a text file through LabView. To ensure that the displacement readings of the LVDTs correlated with the estimated pressures, the pressures at each equivalent wind speed were kept constant for 5 s, and the time of each increment of pressure was recorded manually.

The shelters passed through the test program, which included loading to 180% of design capacity, with no visible damage. There was some nonlinear performance, indicating yielding, at a corresponding wind velocity of 192 m/s (430 mph), or 172% of the design wind velocity.

Finite-Element Analysis

The complexity of the structure and loading of a storm shelter are clearly not amenable to classical, closed-form solution; numerical methods [specifically finite-element analysis (FEA)] are required.

In the past 20 years, finite-element analysis has found widespread use for the analysis of structural systems under extreme loading, such as seismic, blast, and wind (Adams and Azkenazi 1998). Numerous commercial software packages are available that facilitate the use of FEA techniques. These programs provide streamlined procedures for prescribing nodal point locations, element types and locations, boundary constraints, steady and/or time-dependent load distributions, and the specific solution information desired. Several of these packages (notably ALGOR, ANSYS, DIANA, and ABAQUS), are sufficiently sophisticated to have been accepted as basic research tools.

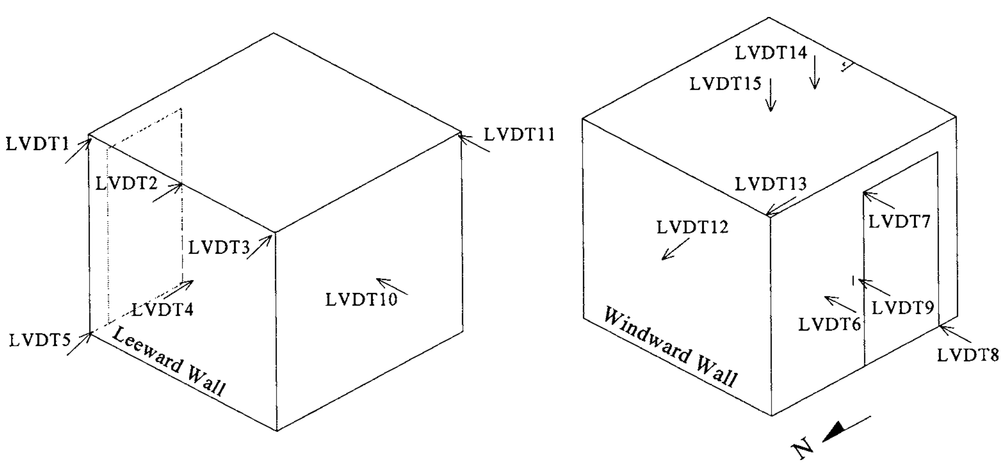

Fig. 10. LVDT locations

Table 2. Summarized Wind Loads for Internal Pressure Coefficients of ±0.55

Velocity (mph)	Windward wall net pressure (psf)	Leeward wall net pressure (psf)	Side wall net pressure (psf)	Front roof net pressure (psf)	Back roof net pressure (psf)
	Gcpi=+0.55/−0.55	Gcpi=+0.55/−0.55	Gcpi=+0.55/−0.55	Gcpi=+0.55/−0.55	Gcpi=+0.55/−0.55
100	3.79/−27.73	−21.82/2.12	−25.76/−1.82	−37.58/−13.64	−25.76/−1.82
125	5.92/43.32	−34.09/3.31	−40.24/−2.84	−58.71/−21.31	−40.24/−2.84
150	8.53/62.38	−49.09/4.77	−57.95/−4.10	−84.55/−30.69	−57.95/−4.10
175	11.61/84.91	−66.81/6.49	−78.88/−5.58	−115.08/−41.77	−78.88/−5.58
200	15.16/110.91	−87.27/8.48	−103.03/−7.28	−150.30/−54.56	−103.03/−7.28
225	19.19/140.37	−110.45/10.73	−130.39/−9.22	−190.23/−69.05	−130.39/−9.22
250	23.69/173.29	−136.36/13.24	−160.98/−11.38	−234.85/−85.25	−160.98/−11.38
260	25.62/187.43	−147.48/14.32	−174.12/−12.31	−254.01/−92.20	−174.12/−12.31
270	27.63/202.13	−159.05/15.45	−187.77/−13.27	−273.93/−99.43	−187.77/−13.27
280	29.72/217.38	−171.05/16.61	−201.93/−14.27	−294.59/−106.93	−201.93/−14.27
290	31.88/233.18	−183.48/17.82	−216.61/−15.31	−316.01/−114.71	−216.61/−15.31
300	34.11/249.54	−196.35/19.07	−231.81/−16.39	−338.18/−122.76	−231.81/−16.39
310	36.43/266.45	−209.66/20.36	−247.52/−17.50	−361.10/−131.08	−247.52/−17.50
320	38.81/283.92	−223.41/21.70	−263.75/−18.64	−384.77/−139.67	−263.75/−18.64
330	41.28/301.94	−237.59/23.08	−280.49/−19.83	−409.20/−148.53	−280.49/−19.83
340	43.82/320.52	−252.21/24.49	−297.75/−21.05	−434.37/−157.67	−297.75/−21.05
350	46.43/339.65	−267.26/25.96	−315.52/−22.3	−460.30/−167.08	−315.52/−22.30
360	49.12/359.34	−282.75/27.46	−333.81/−23.60	−486.98/−176.77	−333.81/−23.60
370	51.89/379.58	−298.68/29.01	−352.61/−24.93	−514.41/−186.73	−352.61/−24.93
380	54.73/400.37	−315.04/30.60	−371.93/−26.29	−542.59/−196.96	−371.93/−26.29
390	57.65/421.72	−331.84/32.23	−391.76/−27.69	−571.52/−207.46	−391.76/−27.69
400	60.65/443.62	−349.07/33.90	−412.11/−29.13	−601.21/−218.23	−412.11/−29.13
410	63.72/466.08	−366.74/35.62	−432.97/−30.61	−631.65/−229.28	−432.97/−30.61
420	66.86/489.10	−384.85/37.38	−454.35/−32.12	−662.83/−240.60	−454.35/−32.12
430	70.09/512.66	−403.40/39.18	−476.24/−33.66	−694.77/−252.20	−476.24/−33.66
440	73.38/536.79	−422.38/41.02	−498.65/−35.23	−727.46/−264.06	−498.65/−35.25
450	76.76/561.46	−441.80/42.91	−521.57/−36.87	−760.91/−276.20	−521.57/−36.87

Shelter Structural Analysis Using ALGOR

ALGOR is a commercially available finite-element program that is capable of analyzing a broad range of engineering problems, including stress and vibration analysis, fluid flow analysis, electrostatic analysis, and mechanical event simulation. Using ALGOR, engineers can analyze stresses and displacements in complex parts due to static or dynamic loading applied constantly or varying with time. Effects of large deflections under these loadings can be analyzed using a variety of nonlinear material models. The availability of a mechanical events simulator (MES) package allows the modeling of highly transient effects (ALGOR 2002).

ALGOR was chosen for this project because it has a wide range of utilities that allow the modeling and analysis of the shelter to be performed quickly and efficiently. Changes in the model and loading can be quickly implemented. The structural model can be created in one of several commercially available computer-aided design (CAD) packages and imported into ALGOR. The structure's surfaces can be defined within the original CAD model, and these are preserved by ALGOR. Mesh generation is automatic, and several different types of elements can be specified. Constitutive models can be chosen from ALGOR's built-in library, or they may be user-defined. All types of connections between elements, including rotational springs and "gap" elements, can be defined. This is particularly useful in modeling composite members such as the FEMA 320 timber-steel shelter, which has a wood-frame covered with plywood and steel sheeting. Additionally, the MES module will allow modeling of debris impact as a future research objective.

Wind effects on shelters are modeled by applying surface pressures normal to the walls and roof. The quantification of pressure produced at a given wind speed is prescribed in ASCE 7-02, as are the pressures assigned to the windward, leeward, and side walls as well as the roof (as ratios of the basic wind-induced pressure). Since both tornadoes and hurricanes contain strong vortices, it is necessary to run analyses modeling wind from all of the main structural axes. ALGOR provides output (i.e., stress, strain, reactions, and displacement) in the form of data files, which can be represented graphically as multicolored "fields" on a drawing of the structure (Figs. 12–15). This is a very useful tool that allows the researcher to quickly identify critical aspects of the structure's response. Also, the graphical representation facilitates understanding of the issues involved in shelter design for a lay audience.

For the purpose of this research project, 2.44 m × 2.44 m × 2.44 m (8 ft × 8 ft × 8 ft) CMU and timber-steel shelter models were selected. The models were created in ALGOR with plate elements for the walls and roof. For the CMU shelter, the element thickness for the wall was 203 mm (8 in.), and 101.5 mm for the roof (4 in.). In the timber-steel shelter, the walls and roof were modeled as plate elements of thicknesses of 25.4 mm (1 in.). The elements in both models were a 152.4 mm × 152.4 mm (6 in. × 6 in.) mesh, produced by ALGOR's mesh generation utility.

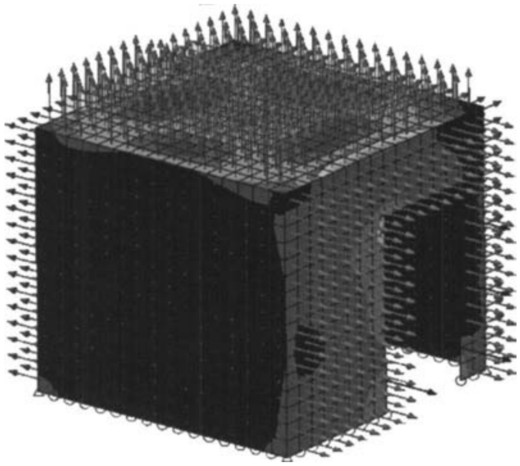

Fig. 12. Load applied on the shelter

Element size was chosen by iterating until results from succeeding analyses, using ever-smaller elements, converged. Pinned boundary conditions were imposed at the nodes connecting the walls to the ground, to the roof, and to one another. Two types of forces were used to represent wind-induced pressure: uniform surface load and nodal force. Surface pressure represented the uniform distributed wind loads on the sides and roof elements. Nodal forces were used to represent the load transferred from the door to the shelter by the door frame anchors to the walls. Fig. 12 shows a representative set of loads applied on the shelter model. Wind direction is shown in Fig. 13; windward wall pressures are represented in Fig. 12 "within" the shelter and therefore are not visible. In this work, 27 load cases (listed in Table 2) were applied to each model (Zain 2003). The loads associated with different wind speeds are determined by *Method 2—Analytical procedure* presented in ASCE 7-02 (ASCE 2002), with coefficients and factors specified by FEMA 361. Linear stress analysis was used. Element specifications were defined. The mass density and the modulus of elasticity of the elements were required to perform this type of analysis. For the CMU shelter, isotropic plate material was chosen for all element types. The modulus of elasticity was 9.31×10^3 Mpa (1.35×10^6 psi). The modulus of elasticity of the concrete roof was determined as the default values of concrete that

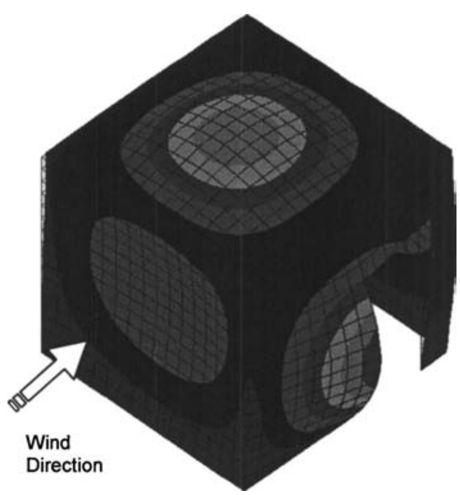

Fig. 13. Displaced shape of the shelter model

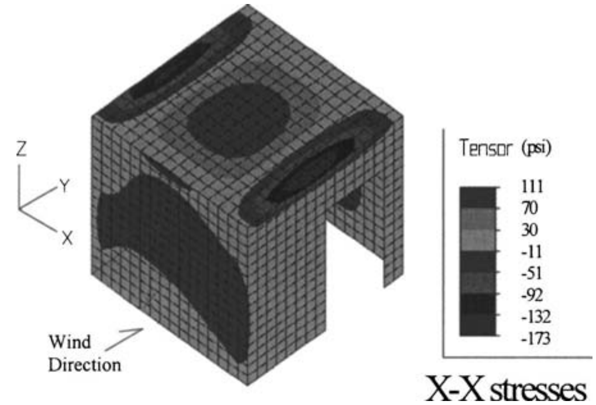

Fig. 14. Stress in x direction in the roof

are available in the program library, which is 3.1×10^4 Mpa (4.5×10^6 psi), respectively.

The walls and roof of the timber-steel shelter consist of studs/joists sheathed with steel and plywood. For modeling purposes, the sections were assumed to be orthotropic. Several panel specimens were tested to characterize the elastic modulus in the orthotropic directions (the strong direction parallel to the studs/joists, and the weak direction normal to them). For the walls, the modulus of elasticity used in the strong direction was 4.8×10^4 Mpa (7.009×10^6 psi) and in the weak direction was 5.02×10^3 Mpa (7.278×10^5 psi). For the roof, it was 1.37×10^5 Mpa (1.983×10^7 psi) in the strong directional and 6.4×10^3 Mpa (9.278×10^5 psi) in the weak direction.

A set of graphical results is presented in Figs. 13–15. Fig. 13 shows the displaced shape of the model with the wind direction as shown (door opening on side wall) for a 112 m/s (250 mph) wind velocity. Figs. 14 and 15 show the stresses in the roof and walls, respectively.

Comparison of Analytical and Test Results

The ratio of applied to design pressure versus displacement at the centers of the walls and roof that were recorded in the full-scale test of the CMU shelter were compared to the displacements of the nodes at the same locations resulting from the finite-element analysis. The comparisons are shown in Figs. 16–20. These are worst-case results—that is, the differences between analytical and

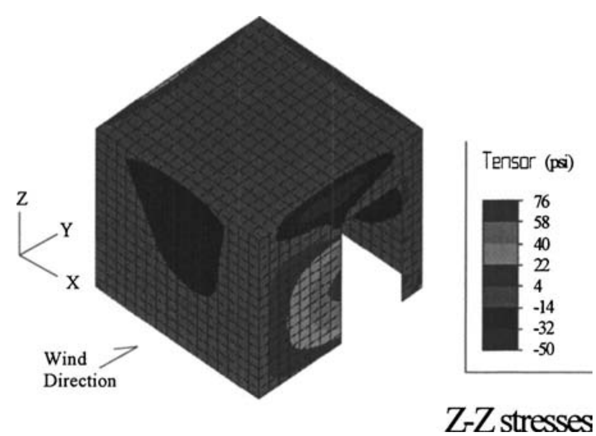

Fig. 15. Stress in z direction in the walls

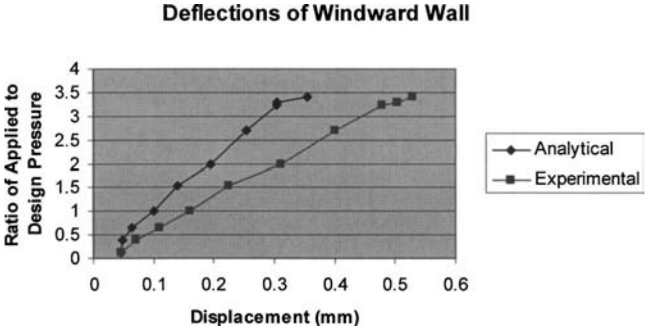

Fig. 16. Comparison of analytical to experimental displacement results on the windward wall

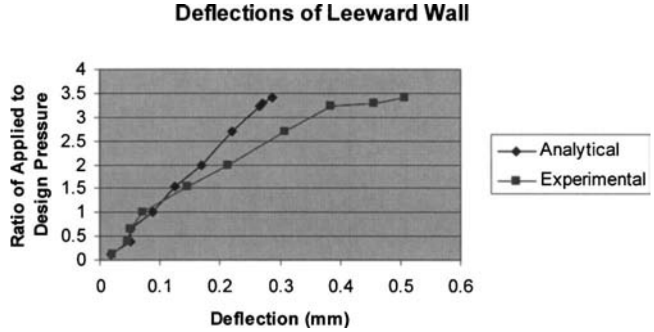

Fig. 18. Comparison of analytical to experimental displacement results on the leeward wall

experimental results are the largest. (Recall that pressure varies as the square of wind velocity—the CMU shelter was tested to pressures simulating a wind speed of 200 m/s, while design was 112 m/s.)

In the analyses of the CMU shelter, the differences between analytically predicted and observed deflections at the center of the windward wall were numerically small (Fig. 16). Experimental displacement was greater with the maximum difference being 0.25 mm (0.01 in.). The difference between predicted and recorded pressure at the design wind velocity of 112 m/s (250 mph) was less than 0.125 mm (<0.005 in.). Both analytical and experimental results show generally linear behavior until the highest pressures investigated were reached. While ALGOR did well in describing the actual value of the displacement and its qualitative "shape," the predicted stiffness in the linear range was about twice as great as that observed. This higher initial stiffness has two likely causes. First, microcracking between the CMUs and the mortar beds would have reduced the wall's membrane stiffness. Second, the use of plate elements in modeling the wall typically results in higher out-of-plane shear stiffness (Hughes 1987), which would have reduced the modeled wall deflection.

The comparison between experimental and analytical results showed no differences on the side wall without door opening (Fig. 17). There was also agreement between analytical and experimental results on the leeward wall (Fig. 18) until well above the design wind loading. The results are almost the same at the center of the leeward wall up to a pressure equivalent to a wind speed of 133 m/s (300 mph). Beyond this point the observed deflections are larger than those predicted, probably due to cracking in the mortar joints.

On the sidewall with the door (Fig. 19), the difference between predicted and observed deflections was small at the lower pressures, but increased to and beyond the design load. The wall's actual stiffness was initially significantly greater than that assumed, but the stiffness of the experimental specimen approached that of the model as the design pressure was approached. As mentioned above, the door was braced closed during the test, and the bracing structure is assumed to have enhanced the initial stiffness of the wall system and forced microcracking in the mortar beds, which allowed the stiffness to drop.

Finally, there was excellent agreement between the experimental and the analytical results for the roof (Fig. 20).

For the timber-steel shelter, displacements of the center of the wall opposite the door and the center of the roof are plotted versus the ratio of applied-to-design pressure, in Figs. 21 and 22, respectively. In both cases, the initial observed displacements are larger than those predicted. For the wall, observed and predicted displacements match at about 80% of the design pressure. For the roof, agreement comes at about 90% of design pressure. In both cases the quantitative difference is small (about 1.5 mm) but the overall displacements are also in that range. The reason the model is stiffer at lower wind speeds is that the properties modeled are the average for the entire wall or roof cross section, while at lower speeds only studs in walls and joists in roof (not the entire cross sections) are resisting the applied forces (Davidson 2004).

The results from these analyses show that the finite-element method can be successfully used to model extreme wind pressures on a storm shelter. Certain modeling issues remain unresolved, but the magnitude of the errors recorded in this case at design wind velocities is not significant to the overall safety and performance of the structure.

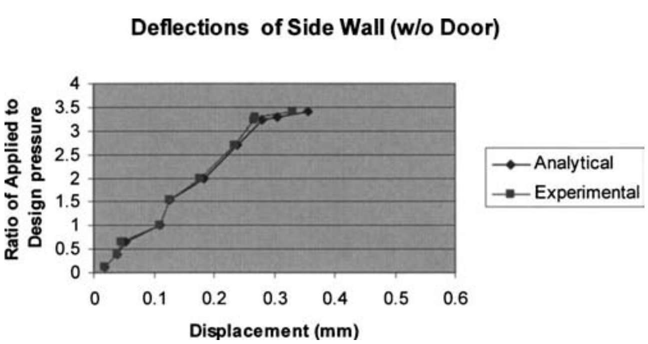

Fig. 17. Comparison of analytical to experimental displacement results on the sidewall without door

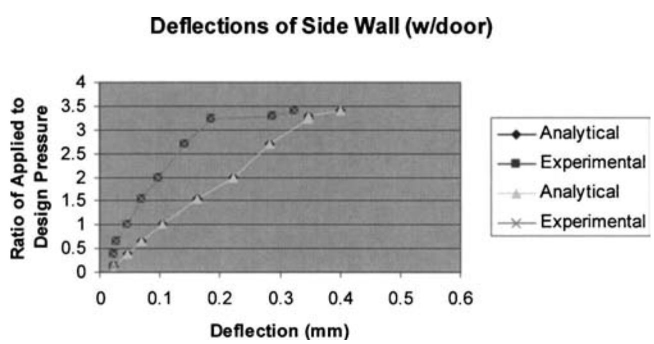

Fig. 19. Comparison of analytical to experimental displacement results on the sidewall with the door

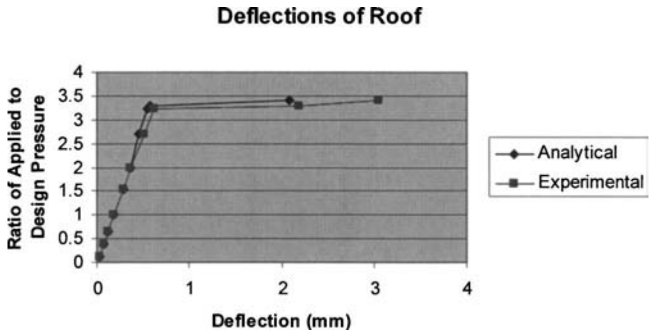

Fig. 20. Comparison of analytical to experimental displacement results on the roof

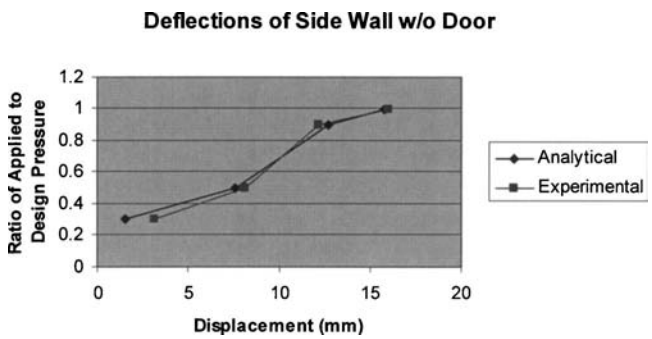

Fig. 21. Comparison of displacements of wood-steel shelter on the sidewall without the door

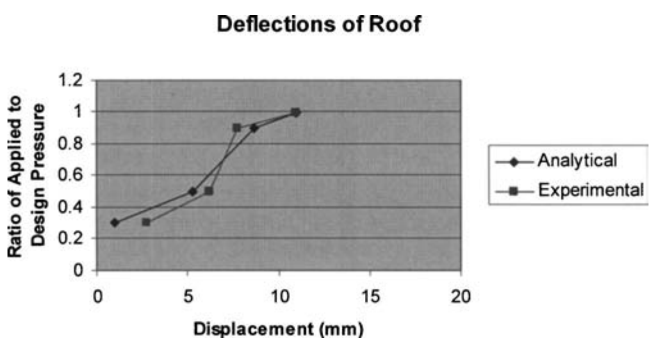

Fig. 22. Comparison of displacements of wood-steel shelter on roof

Conclusions

Full-scale testing of a CMU shelter indicated that FEMA 320 designs for storm shelters are very conservative in resisting wind loads. The fact that the FEMA 320 design remained visibly intact at 170% of the design loads implies that larger shelters can be built to resist 112 m/s (250 mph) winds using the same designs and construction methods presented in FEMA 320. Even larger shelters can be built for lesser design wind speeds appropriate to some geographic areas, including hurricane regions.

The close correlation of predicted and observed results for the CMU shelter at design wind speeds indicates that FEA analysis shows the potential for modeling shelters of different configurations, and different sizes. It can provide adequate and accurate analysis of shelter behavior under the required wind loadings. The need for some testing may, therefore, be eliminated at a significant cost savings. It will allow for the determination of the maximum size shelter that can safely withstand the design wind loads. Full-scale testing of the timber-steel shelter showed that its design is adequate under design loads, and that FEA modeling can successfully predict overall structural displacements under design loads.

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