

SIZE LIMITS FOR ABOVE-GROUND SAFE ROOMS

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ABSTRACT

Numerous deaths and injuries due to storm events such as tornadoes and hurricanes led the Federal Emergency Management Agency (FEMA) in 1998 to present publication FEMA No. 320, "Taking Shelter from the Storm: Building a Safe Room Inside your House". In this publication, a number of prescriptive designs are presented for safe rooms, small residential storm shelters. Designs were developed and tested primarily on the basis of debris impact resistance using materials and construction methods commonly used in residential construction in the United States. cursory structural analyses were performed to assure structural integrity under wind-induced pressures. These designs were recently reevaluated using finite element analysis methods and full-scale structural tests. The results of the analysis show that FEMA 320 designs for storm shelters are very conservative in resisting wind loads. This paper represents the size limits for storm shelters for some designs shown in FEMA 320 for design wind speeds (3 second gusts) of 200 and 250 mph.

INTRODUCTION

Each year an average of 1200 tornadoes are reported in the United States. Wind storms cause a substantial proportion of total damage due to all types of natural hazards, including floods and earthquake. Whereas the hazards are beyond human control, much can be done to minimize their disastrous effects. Progress is being made to provide accurate early warning systems, safer housing and improved infrastructure.

The in-residence shelter or safe room is one solution developed to offer peace of mind and to reduce the risk of severe injury or death in severe weather events. Shelters are classified according their location: basement, in-ground, aboveground. Aboveground in-residence shelters have the advantage of easy access, relative immunity from flooding, and a lack of construction limitations that can affect the installation of the other two types of shelters. Several types of aboveground shelters were designed and developed by a team consisting of researchers and engineers affiliated with the Wind Science and Engineering (WISE) Research Center at Texas Tech University, FEMA, and Greenhome and O'Mara Inc (WERC, 1998; NSSA, 2001). These shelter designs are presented in the first edition of FEMA 320 (FEMA, 1998) as the following:

- Reinforced concrete shelter
- Concrete Masonry Unit (CMU) shelter
- Wood-frame shelter with plywood sheeting and CMU infill; and
- Wood-frame shelter with a combination of plywood and steel sheeting.

These shelters were designed to a square floor plan with maximum dimensions of 8 ft (2.44m). Roof height is also 8 ft. (2.44 m). The second edition of FEMA 320 (1999) added Insulating Concrete Form (ICF) construction with maximum dimensions of 12 ft (3.65m). Design wind speed is 250 mph (112 m/s). A representative shelter configuration, with incident wind pressures, is shown in Fig. 1.

The aboveground shelters shown in FEMA 320 were designed to withstand the forces and pressures associated with extreme winds and to prevent perforation or deformation from wind-borne debris impacts which could cause serious injuries to the occupants. Post-storm damage documentation studies indicate that all tornado damage can be accounted for by wind speeds below about 200 mph (89 m/s). To give an added measure of safety, design ground level wind speeds of 250 mph (112 m/s) were used in the design of the shelters presented in FEMA 320. Wind pressures on the shelters were obtained using the provisions of ASCE 7- 02 (ASCE, 2002). Shelters were designed to resist the impact of 15 pound 2 × 4 in (6.8 kg 5 × 10 cm) boards at a speed of 100 mph (44.7 m/s) striking normal to the surface.

At present, larger shelters (such as community shelters) are considered to be engineered structures, with design guidelines given in FEMA 361, *Design and Construction Guidance for Community Shelters* (FEMA, 2000). However, recently conducted full-scale tests on two types of shelters have indicated that FEMA 320 designs for storm shelters are very conservative in resisting wind loads (Pierce, 2001; Ahmed, 2003). It can be inferred from these results that larger shelters can be built to resist 250 mph (112 m/s) winds using the 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.

In the present research, studies for various wind speeds were conducted to determine the size limits of FEMA 320 designs. Results of finite element analyses using the ALGOR software package were compared with experimental results obtained from full-scale tests on a plywood/steel stud wall shelter and a concrete masonry unit (CMU) shelter. After the analytical model was validated, it was used to determine size limits for shelter designs for various wind speeds. Three types of aboveground shelters presented in FEMA 320 were analyzed to determine their size limits: reinforced concrete, CMU, and wood-frame with plywood and steel sheeting.

DEVELOPMENT AND CALIBRATION OF THE ANALYTICAL MODEL

The commercial finite element package ALGOR was chosen to facilitate the evaluation of different-size shelters under different loads. Its use for this purpose has been validated through previous work (Long, 2003; Budek et al, 2004). The results of the analysis and output of the program can be viewed either graphically or as an output file. Stress, strain, reactions, and displacements are calculated in ALGOR's post-processor, as are moments and shear forces.

To model the shelter, the walls and roof of the shelter were discretized into 6in × 6in (15cm × 15cm) plate elements. This element size was chosen after several iterations were performed to calibrate the model with laboratory testing of CMU and wood-frame shelters. The percentage of error for the 6in × 6in (15cm × 15cm) size element was 0.85%.

Further validation of the analytical model for analyzing the shelter was satisfied by comparing the results of the deflections in the walls and the roof of the shelter from the full-scale tests on CMU and wood-frame shelters to the results obtained from the finite element analysis using ALGOR program. The results from ALGOR analysis gives close comparison to the results obtained in the full-scale tests. In the CMU shelter, the comparison produced close results up to and beyond the design wind pressure. Figures 2 and 3 show comparisons of the deflection in the windward and the roof of the CMU shelter respectively. In the wood-frame shelter, the comparison of the deflection in the centers of the roof and the side wall opposite to the door are shown in Figures 4 and 5, respectively.

ANALYSIS FOR SHELTER SIZE LIMITS

To analyze shelters of varying size, the walls and roof thicknesses were set, the design material properties chosen, and boundary condition for walls and roof were determined.

Construction details for CMU and reinforced concrete shelters were taken from FEMA 320, with the expectation of two roof thicknesses evaluated – the standard 4-in roof, and a roof thickness of 6-in. The critical region was the edge of the reinforced concrete roof slab. (Cracking is forecast in the shelter walls around the doorframe, but the doorframe itself is steel, and its attachment would serve to limit cracking in the adjacent wall.)

Both the reinforced concrete and CMU shelters used cracking as the criterion for the onset of failure. There are several reasons for this. First, retaining the full structural integrity of the shelter offers the advantage of continued protection through protracted severe weather events (such as the passage of a hurricane's eye, in which the initial winds die out, and then return with renewed intensity from the opposite direction). The shelter can also provide expedient housing, if needed. There is also a potential economic benefit in that the shelter need not be rebuilt and can be incorporated into a rebuilt structure.

Second, using cracking as a limit state also places a de facto limit on structural deflections, which will minimize the possibility of internal fittings in the shelter being displaced and possibly causing injury.

Third, limiting cracking will limit potential spalling on the inside surfaces of the shelter. Material which has seen 'incipient spalling' (i.e., cracking has nearly separated it from the walls or roof) can be dislodged by debris impact. Under these circumstances, the spall material can be dislodged with some force and velocity, and could cause injury to the shelter's occupants.

Since the highest stresses occur at the roof edge cracking would initiate there (this was verified in full-scale testing). The boundary condition used to represent the roof-wall connection is fixed, until cracking. When cracking is modeled in the analysis and the roof-wall connection is allowed to 'hinge', stresses from continued loading are redistributed incrementally around the perimeter of the roof, until the roof effectively

becomes a 2-way slab with pinned edges. It was considered conservative to use a limiting tensile stress equal to 115% of the modulus of rupture as the failure criterion. This will be explained below.

As an example, consider a 4-in thick 12-ft by 12-ft 3000 psi slab (102 mm thick, 3.66 x 3.66 m, 20.7 MPa), modeling a shelter roof. In Figure 9, the stress field for a fixed-edge boundary condition under suction loads resulting from incident wind loading is shown. The fixed-edge boundary condition is representative of the shelter before cracking occurs. The highest stresses are at the center of the wall-roof connection. It is here that cracking will first occur. As the section cracks, the individual elements that represent the cracked areas can be considered as having hinged, rather than fixed, boundary conditions (the hinges are actually stiffened with rotational springs, to model the decrease in flexural stiffness of the cracked section). In the case shown, cracking begins at 200 psf. (9.57 kPa)

As more elements along each wall face reach the cracking stress, and their boundary conditions are changed to hinges, the roof's edges 'unzip' – cracking propagates toward the corners. The edges are fully cracked at a pressure of 350 psf (16.75 kPa) (Fig. 10).

The remaining capacity of the slab can be determined using yield-line analysis (MacGregor and Wight, 2005). For a square slab, yield-line analysis gives the capacity-demand equation

$$\phi m_n = \frac{w_u L^2}{24} \quad (1)$$

in which ϕ is the strength reduction factor (taken as 0.9 for flexure), m_n is the nominal moment capacity, w_u is the ultimate distributed load (force/area), and L is the slab side dimension. For the example considered, m_n is 21.7 k-in (2.44 kN-m), L is 12 ft. (3.66 m), and w_u is 23 psi (0.155 MPa).

When the appropriate load combination of (0.9 D+1.2 W_x), where D = dead load, W_x = extreme wind load, is applied, the maximum allowable wind load is 18.13 psi (.124 MPa).

Clearly, the ultimate capacity of the roof slab (and thus the shelter) is far in excess of any credible wind loading.

The reinforced concrete shelters were modeled with a wall thickness of 6in (15cm). Roof thicknesses of 4in (10cm) 6in (15cm) were evaluated. In the FEMA 320 reinforced concrete shelter designs, the reinforcement is at mid-depth of the walls and roof. As outlined above, the attainment of a stress level commensurate with the onset of cracking was used as a conservative failure criterion. Thus, the reinforced concrete walls and roof was assumed isotropic with modulus of elasticity of 3.2×10^6 psi (2.21×10^4 MPa).

For the CMU shelter, the wall thicknesses were 8in (20cm), and the roof slab was 4in (10cm) or 6in (15cm). Figures 6 and 7 show a drawing of the wall section of the CMU shelter and picture of the reinforcing dowels connecting the walls to the floor slab. The modulus of elasticity of the CMU shelter that was used in the analysis was 1.35×10^6 psi (9.31×10^3 MPa) for walls and 4.5×10^6 psi (3.1×10^4 MPa) for roof. As for the reinforced concrete shelter, cracking was used as a conservative failure criterion.. Therefore, isotropic plate elements were used.

The connections between adjoining walls, and between walls and roof, were assumed fixed. The connection between walls and floor slab was a hinge in both the reinforced concrete and CMU shelter models.

For the wood-frame shelter, the walls and roof were modeled as plate elements with a thickness of 1 in (2.5cm). Figure 8 shows a plan view of the wood frame shelter. Due to the lack of information regarding the behavior of the composite cross-section and its material properties, several tests on the shelter's wall section were completed to determine the modulus of elasticity of the equivalent plate section. The section of the walls and roof was assumed orthotropic. Therefore, the modulus of elasticity in two directions was used for the walls and roof plates. For the walls the modulus of elasticity used in the strong direction was 7.0×10^6 psi (4.8×10^4 MPa) and in the weak direction was 7.3×10^5 psi (5.02×10^3 MPa). For the roof, the modulus of elasticity was 2.0×10^7 psi (1.37×10^5 MPa) in the strong direction and 9.3×10^5 psi (6.4×10^3 MPa) in the weak direction. In contrast to the boundary condition used for the reinforced concrete and CMU shelters, pinned connections for wall to wall and wall to roof were used. Pinned connections at 12in (30cm) on center for shelter to foundation was assumed to represent the anchor bolts connecting the shelter to the foundation.

In the wood-frame shelter, according to the full-scale test results, the deflection was linear until an equivalent wind speed of 370 mph (165 m/s) was reached. After this point, there was sudden jump in the roof deflection. It was assumed that at this point the shelter roof started to fail. By using the finite element analysis for the same size of shelter, the bending moments of the roof and the walls sections were calculated at the wind speed of 370 mph (165 m/s). To be conservative, plate bending moments resulting from loads equivalent to a wind speed of 350 mph (156m/s) were taken as the limiting factor for the size limits of the shelter analysis. Bending moment at the center of the roof and walls for larger shelters was compared to the limiting bending moment to determine the size limits of the wood-frame shelter. Thus, since pressure varies as the square of the velocity there is a 'effective' factor of safety of 1.96 for a tornado shelter designed for wind speeds of 250 mph (112 m/s), and 3.06 for wind speeds of 200 mph (89 m/s) (Shamsan, 2002; Davidson, 2004).

For each shelter type, the door was simulated as a rigid element transferring its surface area loads to the shelter structure as point loads through its hinge and latch locations. Tests were designed so that the full pressure was applied to the door.

In the analytical analyses, wind direction was simulated as blowing on each of the four sides of the shelter to determine the worst case loading. Wind speeds were incremented to maximum in steps to develop a load history for each case.

RESULTS

Different sizes of CMU, wood-frame, and reinforced concrete shelters were analyzed under the design wind load of 250 mph (112 m/s) and 200 mph (89 m/s). Shelters were analyzed using load combinations recommended in FEMA 360. The load combination used in the size limits analysis was $(0.9 D+1.2 W_x)$, where D = dead load, W_x = extreme wind load. In the CMU and reinforced concrete shelters, the stresses obtained from the finite element analysis of the larger shelters were compared with the moduli of rupture of the concrete masonry and reinforced concrete. Using appropriate design strengths (1500 psi (10.35 MPa) CMU and 3000 psi (20.7 MPa) concrete), these

are 135 psi (0.93MPa) and 410 psi (2.8MPa) respectively (ACI, 2005). The failure criterion for the wood-frame shelter was taken as a maximum moment recorded in full-scale testing corresponding to a wind speed of 350 mph (156 m/s).

Maximum tensile stresses for a 250 mph wind speed are shown in Table 1 for CMU shelters of various plan dimensions and roof heights. The roof edges reach the cracking stress before the walls do, and are therefore the critical structural element. Using a criterion of acceptance of a maximum stress of 115% of the modulus of rupture for the concrete roof slab, the ‘acceptable’ shelter sizes are marked thus: \surd . Briefly, a 4-in roof slab is no longer acceptable for shelters larger than 12 ft. (3.66 m) square.

The behavior of CMU and reinforced concrete shelters was quite similar. Table 2 shows the maximum recommended sizes for CMU and reinforced concrete shelters built using FEMA 320 construction detailing, for wind speeds of 200 and 250 mph.

Roof deflection was also checked at the maximum sizes. For shelters larger than 14ft \times 14ft, roof-edge beams built into the tops of the walls are recommended to limit deflection.

In the case of the wood-frame shelter, roof joists and wall studs were checked and redesigned (if necessary) for maximum size shelters. Doubling joists or studs was not seen to be a violation of the ‘spirit’ of FEMA 320 design. Joist and stud spacing of 16-in (400 mm) was retained in all cases. Table 3 shows results of the size limit analysis of the wood-frame shelter. The limiting sizes of the wood-frame shelters are seen to be slightly smaller than either CMU or reinforced concrete.

CONCLUSIONS

Based on the results of the full-scale tests that conducted on the FEMA 320 shelters, it was postulated that the shelters could be built larger than 8ft \times 8ft \times 8ft (2.44m \times 2.44m \times 2.44m). The design approach used in FEMA 320 for reinforced concrete, CMU, and wood-frame shelters was considered conservative for resisting the design wind loads. After performing parametric finite-element analyses, it was determined that a larger shelter could be built with the same design philosophy, materials and detailing presented in FEMA 320. CMU and reinforced concrete shelters can be increased in floor-plan dimension by 50% and 75% respectively for 250 mph (112 m/s) winds, and by 100% for 200 mph (89 m/s) winds. Wood-frame shelter plan size can be increased by 50% for tornado shelters and 75% for hurricane shelters, with minor modification (doubling joists and studs).

The authors recommend that more research be performed on the properties and capacity of the wood-frame shelter walls and roof. Also, it is recommended that studies be undertaken of shelter performance when missile impact occurs simultaneously with peak wind pressure. This is of particular interest in the case of a hurricane shelter, which is expected to withstand high sustained winds.

ACKNOWLEDGEMENTS

The authors would like to gratefully recognize the support of NIST in this project.

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Table 1: Maximum tensile stresses in reinforced concrete and CMU shelters for 250 mph wind speed

Shelter Size B (ft) x L (ft) x H (ft)	Roof Thickness (in)	Maximum Tensile Stress (psi)		
		Roof Center	Roof Edge	Walls
12 x 12 x 8 √	4	267	433	135
12 x 12 x 10 √	4	270	437	158
14 x 14 x 8	4	382	638	225
14 x 14 x 10	4	332	540	173
14 x 14 x 8 √	6	217	362	233
14 x 14 x 10 √	6	173	280	171

Table 2: Size Limits of Reinforced Concrete and CMU Shelters for 200 and 250 mph wind speeds

Shelter Type	Shelter Size Limits					
	Wind Speed (mph)	Roof Thickness (in)	Wall Dimensions			
			Length (ft)	Width (ft)	Height (ft)	Thickness (in)
Reinforced Concrete	250	4	12	12	10	6
		6	14	14	10	6
	200	6*	16	16	10	6
CMU	250	4	12	12	8	8
		4	12	12	10	8
	200	6	14	14	10	8
		6*	16	16	10	8

* Roof with edge beams built into the tops of the walls is recommended for this size of shelters.

Table 3: Size Limits of Wood Frame Shelter

Shelter's Type	Shelter Size Limits					
	Wind Speed (mph)	Roof Joist Dimensions (in)	Wall Dimensions			
			Length (ft)	Width (ft)	Height (ft)	Wall Studs Dimensions (in)
Wood Frame	250	Single 2 × 6	10	10	8	Double 2 × 4
		Double 2 × 6	10	10	10	Double 2 × 4
		Double 2 × 6	12	12	8	Double 2 × 4
	200	Double 2 × 6	12	12	10	Double 2 × 4
		Double 2 × 6	14	14	8	Double 2 × 4

List of figures

Fig. 1 – Shelter conceptual drawing with incident wind pressures (side wall pressures omitted for clarity)

Fig. 2 - Windward wall deflection of CMU shelter

Fig.3 - Roof deflection of CMU shelter

Fig.4 - Roof deflection of wood-frame shelter

Fig.5 - Side wall deflection of wood-frame shelter

Fig.6 – Plan view of CMU shelter

Fig.7 - Reinforcement within the shelter walls

Fig. 8 - Plan view of wood-frame shelter

Fig. 9 – Von Mises stress field in shelter roof at onset of cracking (arrow indicates location of highest stress)

Fig. 10 – Roof slab with nearly-full edge cracking (arrow indicates highest stress)

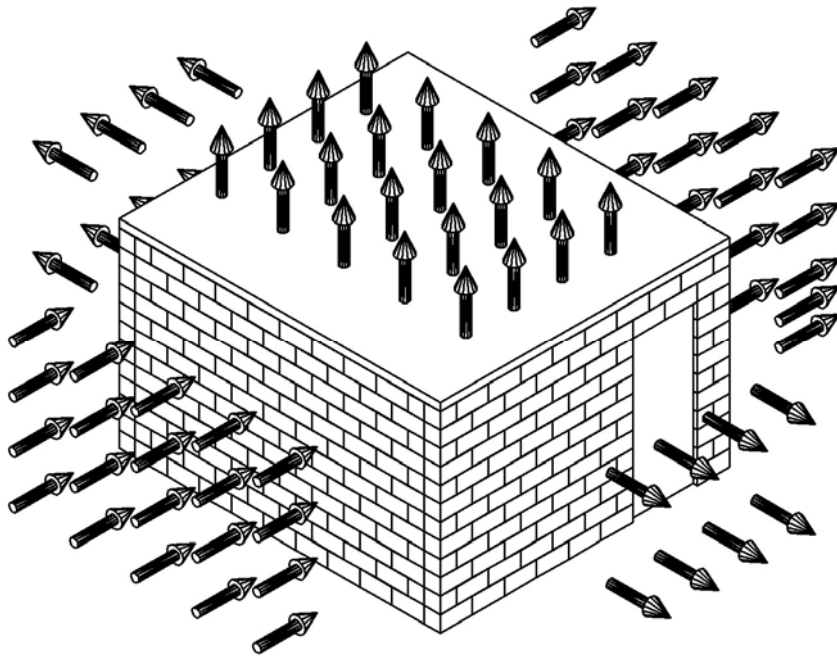


Fig. 1 – Shelter conceptual drawing with incident wind pressure vectors

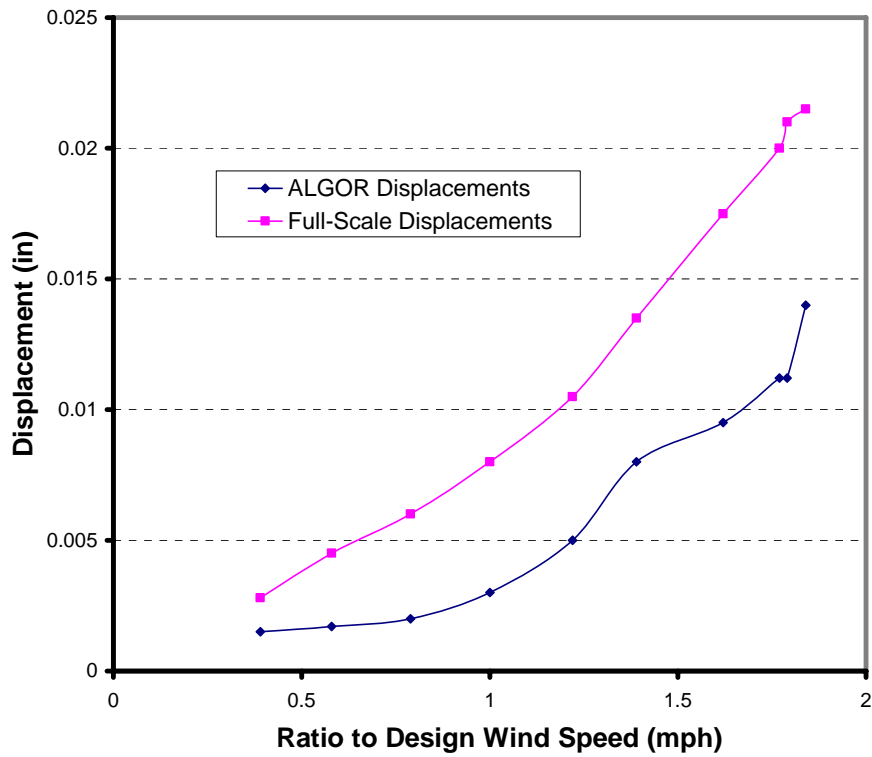


Figure 2 - Windward wall deflection of CMU shelter

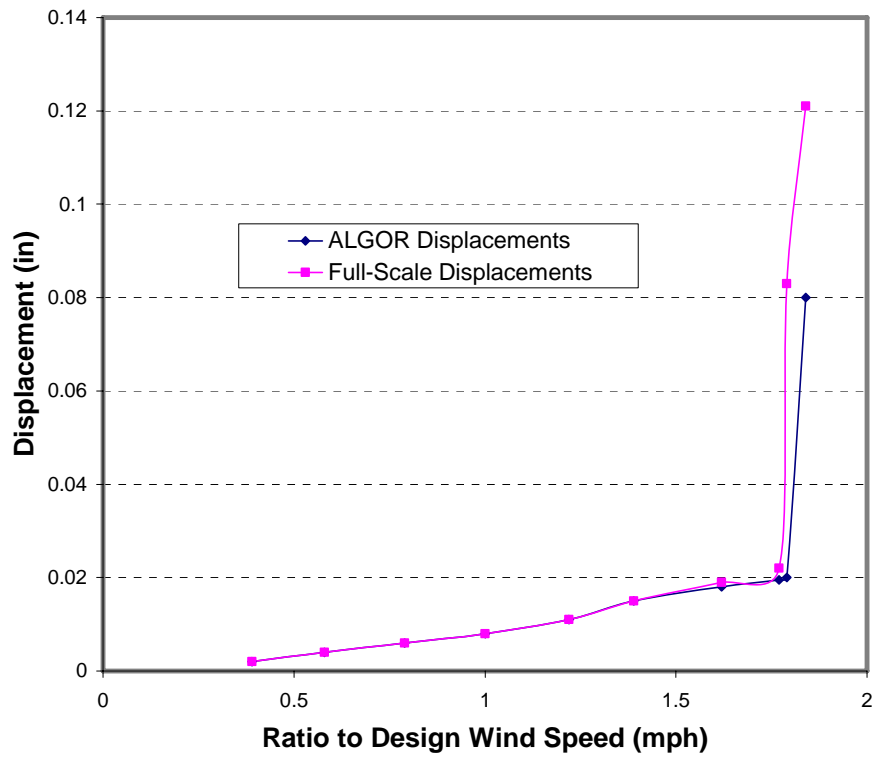


Fig. 3 - Roof deflection of CMU shelter

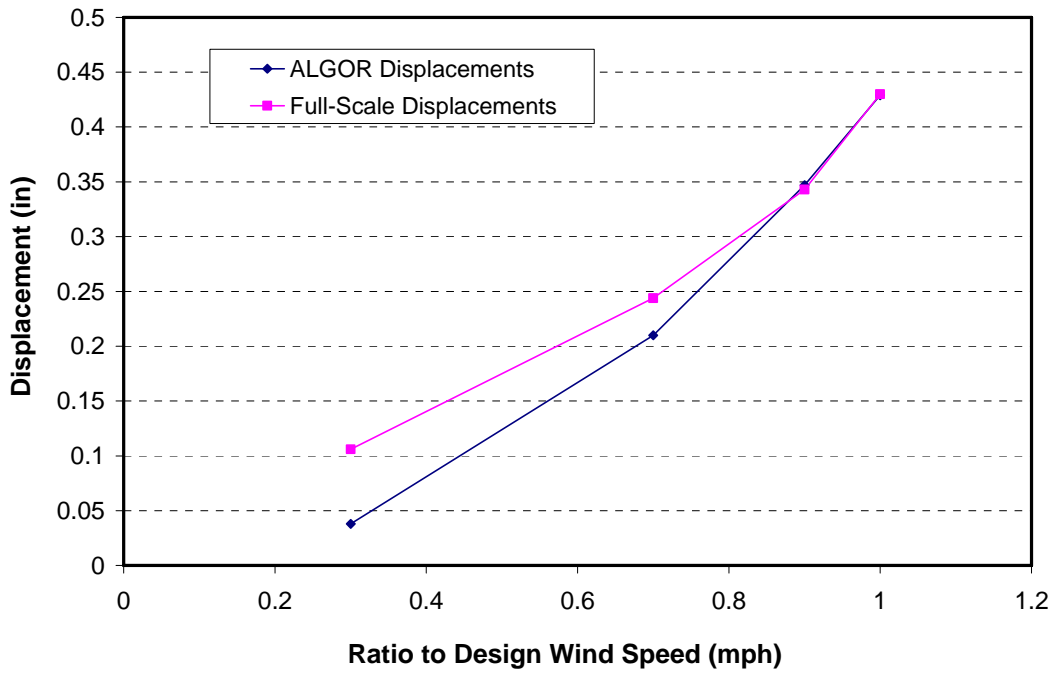


Fig. 4 - Roof deflection of wood-frame shelter

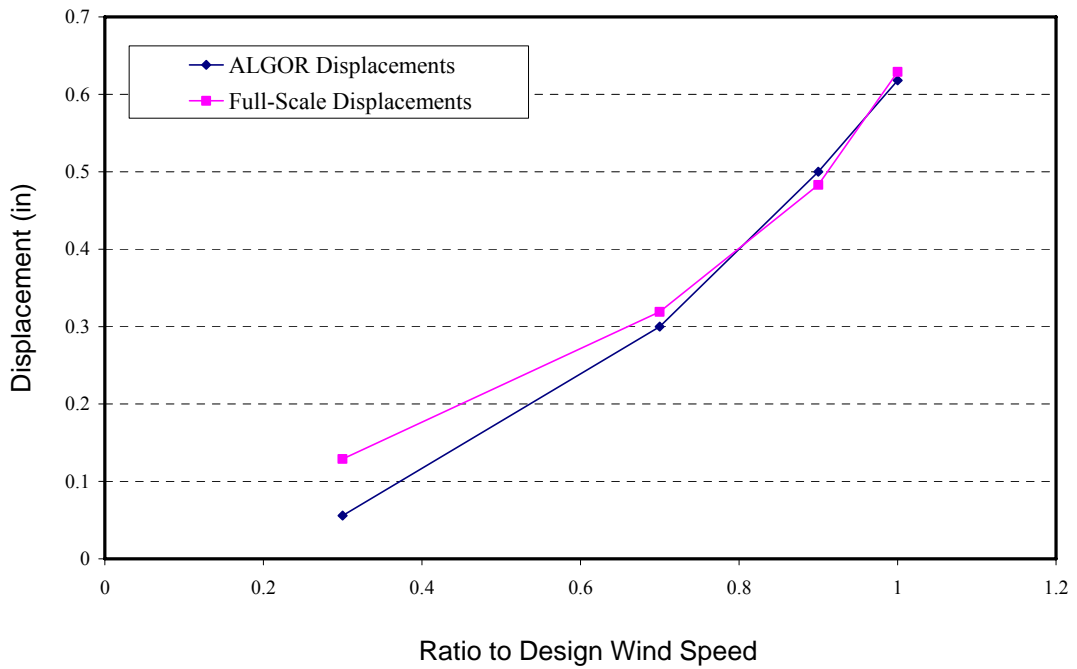


Figure 5 - Side wall deflection of wood-frame shelter

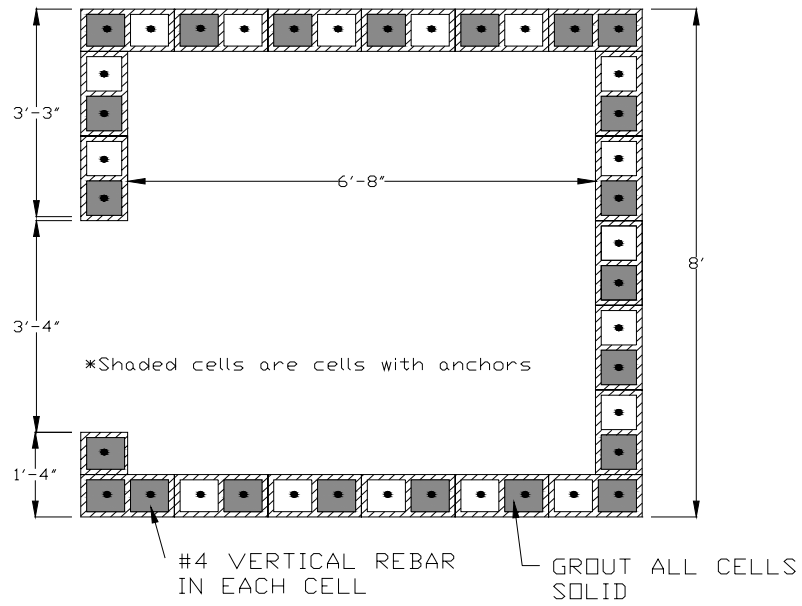


Fig. 6 – Plan view of CMU shelter



Fig. 7 - Reinforcement within shelter walls

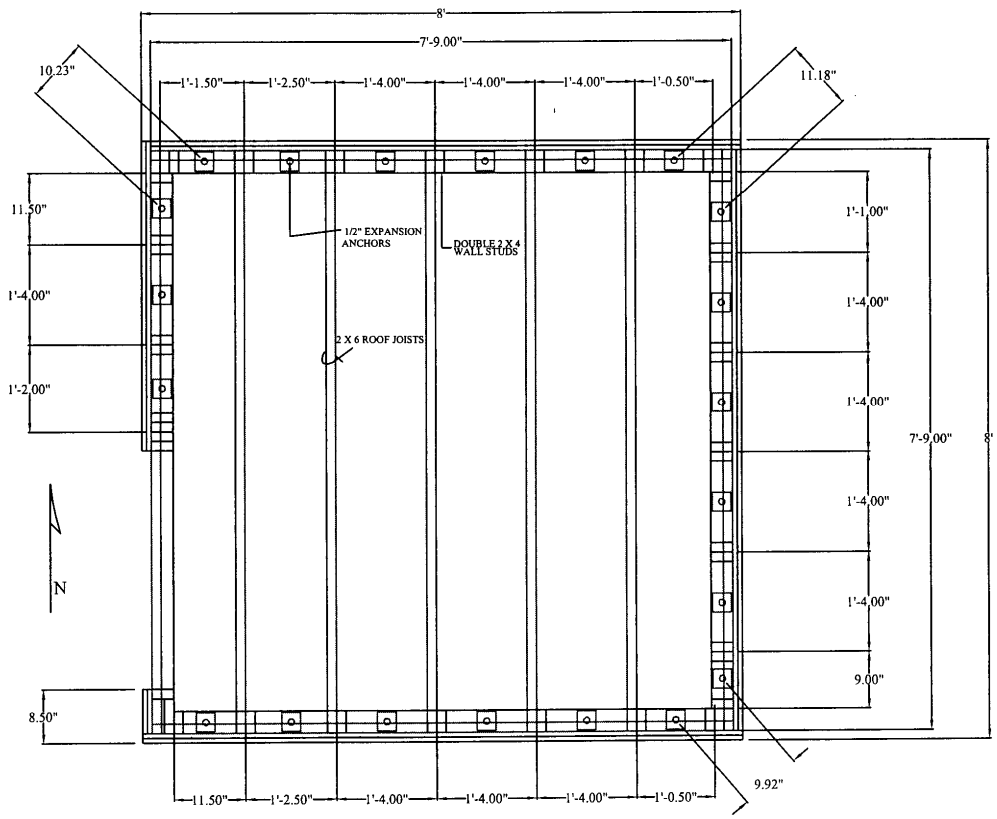


Fig. 8 - Plan view of wood-frame shelter

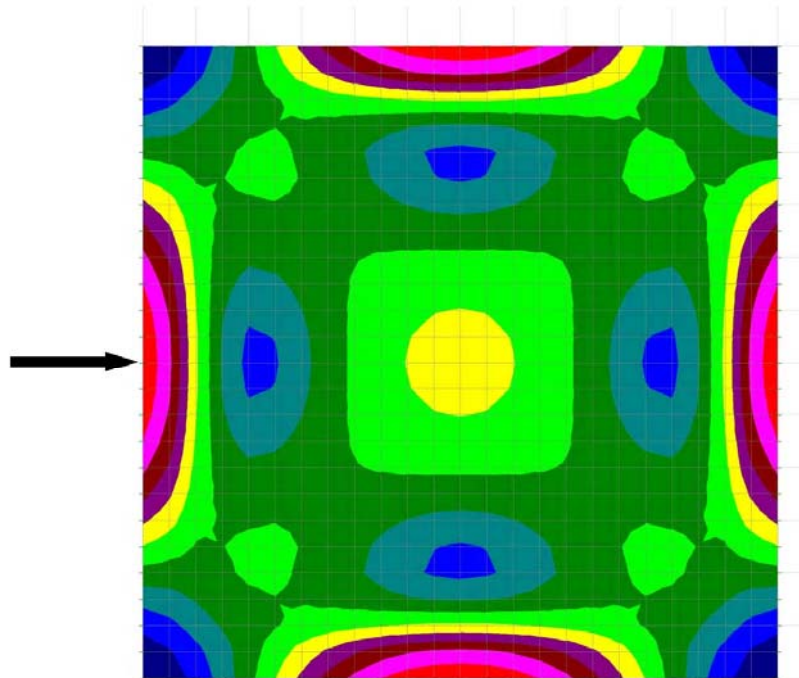


Fig. 9 – Von Mises stress field in shelter roof at onset of cracking (arrow indicates location of highest stress)

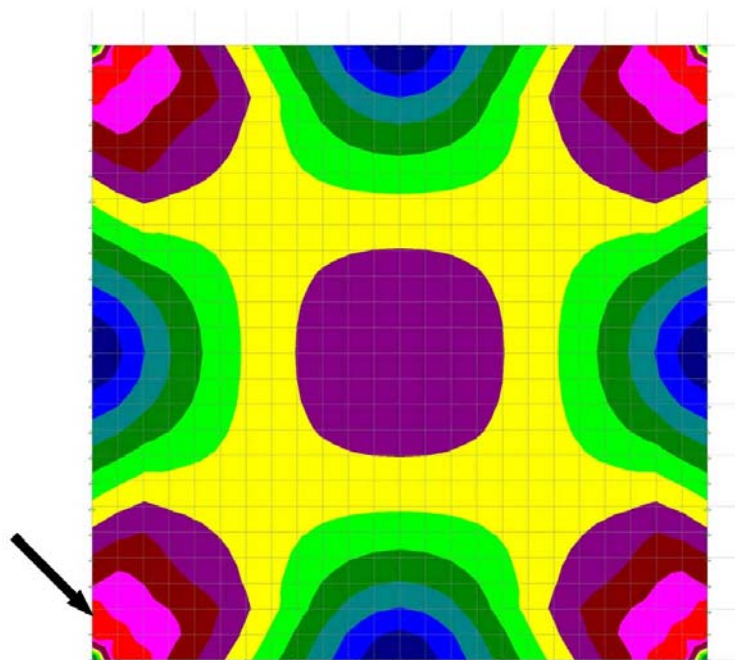


Fig. 10 – Roof slab with nearly-full edge cracking (arrow indicates highest stress)