ON-GRADE REINFORCED CONCRETE FLOOR SLABS FOR STORM SHELTERS

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Extreme wind events such as tornados and hurricanes are among nature’s most destructive forces. Although we cannot control these forces, we can find ways to mitigate their effects. Satellite imagery and weather radar can be used to forecast and track major wind events in order to develop effective warning systems. Another way of enhancing human safety is to build shelters that are safe against the violent forces produced by extreme wind velocity.

In the U.S., extensive research is being carried out in the fields of wind engineering and storm shelters. Above ground, in-residence storm shelters are gaining popularity because of their easy accessibility and effectiveness in protecting occupants from injury or loss of life. The concept evolved from the findings that small interior rooms often survive major tornados or hurricanes. While it is currently prohibitively expensive to make an entire house safe against extreme winds, a small interior room such as a bathroom, closet or utility room can be economically constructed to serve as a readily accessible storm-resistant unit.

Texas Tech University has been actively involved in the development of such shelters since a 1970 tornado caused severe damage to the City of Lubbock. Based on post-event inspection and damage assessment, the concept of in-residence, above ground storm shelters was introduced in 1974, and extensive work has been done since to develop protocols for the development, design and of such shelters. Initial designs were intended for a maximum room size of 8' x 8' x 8', and the Federal Emergency Management Agency (FEMA) began publishing construction details of 8' x 8' in-residence storm shelters made of concrete masonry units, reinforced concrete and timber-steel in 1998.

To extend the usefulness of the concept to larger shelters, researchers felt a need for an analytical method that simulates actual field conditions during an extreme wind event. In 2002, a Texas Tech researcher employed the ALGOR computer program as a finite element analysis (FEA) package to model the FEMA shelter designs. Subsequent research validated the ALGOR software as an acceptable tool for storm shelter modeling and analysis, allowing its use for the design of larger shelters using concrete masonry units and reinforced concrete. Modifications in construction details for the roofs of larger timber-steel shelters have also been suggested.

The National Storm Shelter Association (NSSA), which was formed in 2000 to foster quality in the shelter industry, has developed the Standard for the Design and Construction of Storm Shelters (available online from www.NSSA.cc) pending the finalization of a consensus standard. The NSSA standard served as the starting point for the ICC Consensus Committee on Storm Shelters’ work on the ICC Standard on Design, Construction and Performance of Storm Shelters, with ongoing research providing input to the ICC/NSSA Committee.

Anchorage to the on-grade floor slab is an important part of the structural design of above ground storm shelters, providing a continuous path for loads from the shelter to the soil, and the FEMA standard provides prescriptive designs for reinforced concrete slabs. Following is information about the flexural behavior of these slabs and their ability to anchor a storm shelter during extreme wind conditions. Critical slab-shelter configurations are covered and design stipulations are recommended for each case. These recommendations have been submitted to the ICC/NSSA committee.

On-Grade Floor Slabs

An on-grade concrete floor slab is subject to forces transferred from the storm shelter through slab-shelter connections. Overturning moment is created by the resultant of wind pressures acting on the windward wall, leeward wall and roof of the shelter. The combined uplift and overturning moment forces are transferred to the slab through the slab-shelter connections in the form of “pull” (upward force) on the windward side and “push” (downward force) on the leeward side, as shown in Figure 1. The weights of the shelter and the concrete slab are the main restoring forces acting on the slab; the suction force from the soil beneath the slab is also helpful in resisting the overturning moment.

The resultant forces acting on a storm shelter during an...
Storm Shelters

extreme wind event} have potential of uprooting the shelter and concrete slab from the ground, causing the slab to move with it in the upward direction. This uprooting tendency is greatest in lightweight shelters that cannot provide sufficient dead weight to prevent the overturning of shelter. Further, the concentrated loads at connection points induce high tensile stresses in the concrete. Under the action of these forces, flexural failure and concrete pullout failure can be anticipated.

Flexural Failure

Flexural cracks are developed in the tension zone of concrete slabs when the flexural tensile strength of concrete is exceeded. The size and spacing of the reinforcing bars and the depth at which they are provided determine the flexural strength of the slab. The formation of flexural cracks in a concrete member results in reduction of its moment-carrying capacity. In the uncracked condition, the complete cross section of the concrete member contributes to the flexural stiffness. After cracking, the cross-sectional area contributing to the flexural stiffness is reduced. Figure 2 shows the change in flexural stiffness of a reinforced concrete member under loading. Flexural stiffness before cracking is given by the slope of line OA. After cracking at point A, the tensile force taken by concrete reduces with increase in the load. Finally, at point B, the tensile force taken by concrete becomes negligible as compared to that taken by steel and the steel starts yielding. At point B, the fully cracked section is developed. The flexural stiffness of this section is given by the slope of line OB. The moment-carrying capacity of the member between points A and B is the reserve strength of concrete after cracking. The moment-carrying capacity of the member between points B and C is the reserve strength of the slab after yielding of the reinforcing steel is initiated. The reserve strength of the slab after cracking was computed using the effective flexural stiffness of the cracked section. The limiting value of flexural stress was obtained as the sum of modulus of rupture strength and the reserve strength in reinforced concrete between cracking and failure. The limiting value of flexural stress was computed using the following equation.

\[ \phi \sigma_{\text{lim}} = \phi f_r + \phi \sigma_{\text{res}} \]  

(Equation 1)

where:

\[ \phi f_r = \text{factored modulus of rupture strength of concrete} \]
\[ = 7.5 \sqrt{f_c} \text{ psi, per American Concrete Institute (ACI) Standard 318, Building Code Requirements for Structural Concrete}^{9} \]

\[ \phi \sigma_{\text{res}} = \left( \frac{\phi M_n - \phi M_{\text{cr}}}{I_{\text{eff}}} \right) y_{t \text{-cr}} \]

= reserve strength of reinforced concrete section, psi;

\[ \phi M_n = \text{factored nominal moment carrying capacity of slab section, lbf-in}^2; \]

\[ \phi M_{\text{cr}} = \text{factored cracking moment carrying capacity of slab section, lbf-in}^2; \]

\[ I_{\text{eff}} = \text{effective moment of inertia of slab section after cracking, inches}^4 \text{ (ACI 318); and} \]

\[ y_{t \text{-cr}} = \text{distance of maximum tensile stress from the compression face in a cracked section, inches.} \]

To validate the use of Equation 1, slab-shelter models were analyzed in which a load was applied to cause cracking in the slab. The elements in which cracking stresses were reached were reduced in bending stiffness, then further loads were applied up to the design load. Thus, cracked-section stresses could be monitored and maxima for use in the analysis (in excess of the cracking stress) obtained to validate the approach outlined above (for additional details about this method, see Budek et. al.\textsuperscript{10}).

(continued on page 26)
In our analysis, the shelter and floor slab were subjected to separate FEA analyses because of the incompatibility in modeling in the anchor bolt connections of the shelter to the slab in ALGOR. Since the forces were to be transferred to the slab in the form of point loads, it was found to be sufficiently accurate to model the shelter separately. Thus, the reactions of the shelter were used to get the point loads to be applied to the slab. The shelter was assumed to be a stiff box that is tied to the flexible foundation (slab). In order to simulate the rigid body rotation and deflection of the slab caused by the stiff shelter tied to it, elements corresponding to the footprints of the shelter walls were increased in stiffness by a factor of 10 (i.e., the thickness of these elements was increased by a factor of 10).

Construction details furnished by the FEMA standard were followed for modeling the concrete masonry unit shelters and reinforced concrete shelters. The timber-steel shelters were modeled with 1-inch walls and a 1-inch roof, with all structural properties corresponding to those of actual timber sections. Plate elements with 4” x 4” mesh were used to simulate the walls and roofs of the shelters. This mesh size was used to accommodate pin supports at 16-inch intervals; the average spacing of anchor bolts considered for the slab analysis. The loads applied to the shelters were computed in accordance with FEMA 361, Design and Construction Guidance for Community Shelters, for the case of an F5 tornado considering 250-mph, 3-second gust ground-level wind speed. As per Section 5.4.1, the load combination assumed to be most critical for the floor slab analysis was 0.9D + 1.2W (D = dead loads, W = wind load). The shelter was analyzed for support reactions for all the wind directions (i.e., East-West, West-East, North-South, South-North).

A 3.5-inch-thick slab was analyzed because the International Residential Code permits slabs of such thickness. The slab was modeled in ALGOR using plate elements with properties of 3,000 pounds per square inch normal weight cast-in-place concrete. For computing the moment carrying capacity of the member, the concrete was considered to be reinforced at mid-depth with grade-60 steel bars placed in both the directions at an equal spacing of 12 inches (for newly constructed floor slabs, the FEMA standard indicates minimum #4 steel bars at 12 inches on center in each direction). Mesh size for the finite elements was adopted as 4” x 4” in order to match the connection points of the shelter. Surface elastic boundary elements were used to model the soil support for the slab. The soil was modeled with translation-type elastic compression springs. The springs acted in compression only—soil suction (spring in tension to counter the upward deflection of slab) was not modeled. The spring stiffness was taken equal to the modulus of subgrade reaction, the relationship between soil pressure (q) acting on a foundation member and its deflection ($\delta$), due to
applied force on the member. It is defined as follows.

$$k_s = \frac{q}{\delta} \quad \text{(Equation 2)}$$

The empirical equation that was used for computing modules of subgrade reaction is as follows.\(^8\)

$$k_s = 12(SF)q_a \text{ k/ft}^3 \quad \text{(Equation 3)}$$

where:
- \(SF\) = factor of safety; and
- \(q_a\) = allowable bearing capacity of soil, ksf.

For the slab analysis, \(SF = 2.0\) was assumed. FEMA suggests a minimum soil bearing capacity of 2,000 psf.

In ALGOR, the slab was fixed on at least one edge to provide structural stability in the horizontal plane. The fixity was also provided to simulate continuity. (It should be noted that all construction joints or sawn joints were modeled as slab edges, even though sawn and some types of construction joints do permit vertical shear of adjacent slab components to act across the joint.) Static stress linear material model analysis was then used to analyze the concrete floor slab.

The different slab-shelter configurations used for flexure analysis are shown in Figure 4. The following assumptions were made while analyzing the floor slab and were aimed at achieving conservative designs.

- The shelter was assumed to be rigid and intact under the extreme wind event (250-mph wind speeds), thus capable of transferring all the loads to the slab effectively. This also allowed the shelter weight to counteract the uplift of slab.
- The anchor bolt connections were assumed to be intact and structurally sound; again, transferring the loads from the shelter to the slab efficiently.
- The slab was modeled using solid plate elements. The reinforcement was not modeled; thus, the behavior of slab after cracking (in plastic range) was inferred but not specifically examined. The maximum stresses obtained from ALGOR analyses were compared with the stresses computed using Equation 1 and verified as outlined above.
- Slab turn-downs at the edges were not considered. Turn-downs add significantly to the weight and edge strength of slabs, and would therefore represent additional margins of safety.
- The reinforcement was assumed to be at the center of the slab depth. This is conservative because in practice most slab reinforcement might be placed closer to the bottom of the slab; where it would be more effective in resisting uplift moments.
- As noted, soil suction was not considered. Thus the soil was modeled as a bed of compression springs. This is a conservative assumption because in reality soil suction

Figure 4. Slab-shelter configurations.
will resist uplift of the slab caused by wind loads.

- The connection of the slab under the shelter to laterally adjacent areas of slab (consider the offsets of configurations C-2 and C-3) was assumed to remain competent. The slab moment capacity for both configurations was greater than the moment applied by that part of the slab “lifting” with the shelter.

**Slab-Shelter Configuration**

A storm shelter can be provided at the most suitable location in a new home through proper planning, but manufactured shelters are also often installed in existing buildings. In either case, the slab-shelter configuration is a very important aspect of the analysis of floor slabs. The configurations considered in our analysis are explained below.

- **C-1**—on the edge of an 8-foot wide slab strip. This configuration represents a shelter mounted at the edge of a strip of slab that has width equal to that of the shelter (8 feet). This case is common when a shelter is to be built adjacent to an existing building or at the end of a driveway. The offset length (distance to the edge of slab from the shelter walls) for configuration C-1 was selected in such a way that the stresses in the slab are not affected by the fixity at the far end. If the offset provided to the strip is less than the values shown in Figure 4, the connection (fixed edge) of the slab should be designed for the fixed end stresses. In this configuration, the contribution of slab self-weight is minimal and restricted to the width of shelter. The soil suction that prevents the slab uplift during the wind event is also restricted by the width of slab available. Furthermore, the slab cannot get the anchorage support of the reinforcement due to absence of slab offset on the sides.

- **C-2**—on the center edge of a 16' x 16' slab. This configuration represents a shelter mounted on the center edge of a corner room in an existing house or on an isolated floor slab of a garage. It provides better contribution of slab self-weight as compared to C-1. (The effect of soil suction would also be enhanced, but was not considered.) Further, the offsets are sufficient to allow the reinforcement to add to the anchorage of the shelter to the slab.

- **C-3**—on the corner edge of a 16' x 16' slab. This represents a shelter mounted at the corner edge of an existing corner room or an isolated floor slab of a garage. This is a very common configuration.

- **C-4**—on an isolated slab. This configuration was considered to cover cases in which the shelter is not attached to a floor slab of the house. The slab thickness for each shelter case was determined by hand calculations, assuming the dead weight to counteract the over-turning moment caused by wind forces. This configuration was checked with different offsets of slab all around the shelter.

**Concrete Breakout Strength**

Concrete breakout strength at the anchor bolt connections was checked in tension per the provisions of ACI-318. The nominal concrete breakout strength of an anchor in tension is given as follows.

\[
N_{eb} = \left( \frac{A_N}{A_{No}} \right) \psi_2 \psi_3 N_b \quad \text{(Equation 4)}
\]

where:

- \(A_N\) = projected concrete failure area of an anchor limited by edge distance, for calculation of strength in tension;
- \(A_{No}\) = projected concrete failure area of an anchor not limited by edge distance, for calculation of strength in tension (for an isolated anchor \(A_N = A_{No}\));
- \(\psi_2\) = modification factor for edge effects (\(\psi_2 = 1\) if smallest edge distance, \(c_{min} \geq 1.5 h_d\); and \(\psi_2 = 0.7 + 0.3 \left( \frac{c_{min} - 1.5 h_d}{1.5 h_d} \right) \) for \(c_{min} < 1.5 h_d\));
- \(\psi_3\) = modification factor for cracking of concrete (\(\psi_3 = 1.25\) for uncracked concrete and \(\psi_3 = 1.0\) for cracked concrete);
- \(N_b\) = basic concrete breakout strength of a single anchor (lbf), given by ACI-318 Equation D-7:
  \[N_b = 24 \sqrt{h_d}\] for cast-in anchors; and
- \(h_d\) = effective anchor embedment depth, inches.

The modification factor for cracking of concrete includes a reduction of 25 percent in concrete strength after cracking.

**Failure Criteria**

Stress tensor components in XX and YY direction (\(\sigma_{xx}\) and \(\sigma_{yy}\)) were considered in order to understand the flexural response of the concrete slabs. These stresses were compared with the flexural tensile strength of concrete in order to locate the elements that indicate the onset of cracking. The limiting stress (Equation 1) was used to determine the failure stress in a slab. The ductile (energy-absorbing) capacity of a slab after the onset of yielding of the reinforcement represents an additional margin of safety.

The slabs designed for bending were checked to ensure adequate strength at the connections. Concrete breakout strength is the magnitude of force that should not be exceeded by any slab-shelter connection during a wind event. The loads at the connections as obtained from the shelter analysis were checked against this maximum value.
Conclusions and Recommendations

As the ALGOR software has been validated and used in the past to analyze and design storm shelters, it can be concluded that the following can be used in the development of criteria for on-grade floor slabs supporting above ground storm shelters.

- Slabs modeled with box stiffness (i.e., elements representing the footprints of shelter walls increased in stiffness to simulate a rigid body rotation of slab) provided a more realistic model for determining the behavior of slabs anchoring storm shelters as compared to slabs modeled with uniform stiffness. This attributed to the stiffness of the shelter walls being very high compared to that of the slab, thus the movement of the shelter governed slab displacement.
- East-West wind direction, defined in Figure 4, governed slab design for all configurations except the C-1 8’ x 8’ x 8’ timber shelter, for which the North-South wind direction was found to be critical.
- Slab offset beyond the shelter edge was found to be a very important parameter. Configuration C-1 (offset on one side) was found to be the most critical case for slab design, followed by configuration C-3 (offset on two adjacent sides) and configuration C-2 (offset on three sides).
- Soil bearing capacity was found to have an appreciable impact on flexural stresses in slabs and their designs. Modeling the soil as a bed of compressive springs is a conservative approach as the resistance to overturning provided by suction is disregarded.
- The duration of the wind event (3-second-gust wind speed was considered) and suction resistance offered by soil are factors that would increase resistance to overturning, but were not accounted for in our analysis. Thus, the procedure employed may be considered conservative.
- A width-to-height aspect ratio of 1:2 ($L:B = 4’ \times 8’$) was considered to be the limiting case. Cases having an aspect ratio less than 1:2 have a greater tendency to overturn and thus require appropriate engineering analysis. All shelters were analyzed for a height of 8 feet.
- Heavy-duty sleeve anchors—$M^{#12}$ mm bit diameter, embedment depth ($h_{ef}$) = 2.916 (65 mm) for 3,000 psi concrete—have the required tensile strength (5,165 lbf) to resist the maximum pullout force of 3,640 lbf obtained for an 8’ x 4’ x 8’ timber shelter.\textsuperscript{1} ACI Standard 318 suggests no reduction of nominal concrete breakout strength in tension of a single anchor if the edge distance provided to an anchor is greater than 1.5 $h_{ef}$ (1.5 times the embedment depth).

Figures 5 through 7 illustrate the flexural behavior of the 3.5-inch-thick slab (with box stiffness) supporting a 8’ x 4’ x 8’ shield for configurations C-1 through C-3, respectively, subjected to an East-West wind with a velocity of 250 mph. It was observed that for C-1 configurations the slab shows maximum stress all along the leeward wall of the shelter. In the case of configurations C-2 and C-3, the maximum stress was located at the corners of the leeward wall of the shelter. The localized high stress at the corner was deemed unlikely to cause progressive structural failure of the slab, so the stress values at the leeward edge of the slab were considered for determining failure. Based on these results, suggested
Storm Shelters

slab thickness and reinforcement are given in Table 1 (the #3 reinforcement is assumed to represent an existing slab; in accordance with FEMA, the minimum size suggested is #4).

The results of dead load calculations for isolated storm shelters clearly indicate the importance of slab offset beyond shelter edges on all sides. It was assumed that the overturning caused by the extreme wind about the leeward edge of slab is equal to the moment of resultant dead load (weight of slab and shelter) about the leeward edge of slab. The lateral resistance offered by the passive pressure and skin friction of soil was not taken into account. An increase in slab offset causes an increase in dead weight, which in turn resists the overturning of shelter and thus ensures better anchorage. The recommended slab thickness for isolated storm shelters (configuration C-4) is given in Table 2.

<table>
<thead>
<tr>
<th>SHelter Size (feet)</th>
<th>Offset (feet)</th>
<th>Slab Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 x 4 x 8</td>
<td>1</td>
<td>2' - 8&quot;</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1' - 6&quot;</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>10'</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>7&quot;</td>
</tr>
<tr>
<td>8 x 8 x 8</td>
<td>1</td>
<td>1' - 6&quot;</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1' - 0&quot;</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>8&quot;</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>6&quot;</td>
</tr>
</tbody>
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9. ACI 318, Building Code Requirements for Structural Concrete, American Concrete Institute, Farmington Hill, MI.