6 Commentary on Load Determination and Structural Design Criteria

This chapter provides a discussion of the technical and design criteria information presented in Chapter 3. The intent of this chapter is to present commentary and background information on the "how and why" of the design criteria, including where appropriate, the discussion of how the criteria presented in this publication differ from the new ICC-500 Storm Shelter Standard. Commentary related to performance criteria for debris impact is presented in Chapter 7. The design criteria presented in this chapter are based on the best information available at the time this document was published.

CROSS-REFERENCE

See Chapter 10 for a list of the FEMA publications and other reference documents cited here.

6.1 Commentary on the General Approach

Considerable and significant development of safe room design criteria and standards has taken place since the original FEMA 361 was first published in July 2000. Several extreme-wind events have tested buildings specifically designed and used as safe rooms. This has provided an opportunity to assess the performance of these buildings in extreme events and evaluate the criteria and standards used in their design. Additionally, testing laboratories and universities have been evaluating the acceptability of building envelope components such as wall and roof assemblies, window and door units, and glazing systems for both wind pressure and debris impact resistance design criteria. Since FEMA 361 first appeared, storm shelters have become a manufactured commodity with installations occurring in many markets, especially those where tornadoes are a frequent threat. All of this has considerably extended our knowledge and experience in safe room design.

Much of this knowledge has found expression in new building codes and standards, especially the ICC-500 storm shelter standard. However, following the design requirements of the ICC-500, the IBC, the IRC, or ASCE 7-05 alone might not satisfy the safe room design requirements that FEMA has established. Table 2-1 in Chapter 2 is intended to provide a roadmap to the appropriate use of the standards or design guidance documents, so users of these standards can be assured that, depending on the ultimate use and completion strategy chosen for a safe room project, the appropriate guidance has been followed. Following FEMA safe room criteria will provide “near-absolute protection” for individuals seeking safety from a tornado or hurricane. This level of protection, as defined in Chapter 2, is meant to protect safe room occupants from injury or death during an extreme-wind event.

6.1.1 Design Wind Speeds

The development of design wind speeds used in ASCE 7-05 is discussed in the commentary of ASCE 7-05, Section C6.5.4. An excerpt from that commentary about the use of basic wind speeds shown in Figure 6-1 of that standard states:

> *The design-level speed map has several advantages. First, a design using the map results in an ultimate load (loads inducing the design strength after use of the load factor) that has a more uniform risk for buildings than occurred with earlier versions of the map. Second, there is no need for a designer to use and interpolate a hurricane coast importance factor. It is not likely that the 500-yr event is the actual speed at which engineered structures are expected to fail, due to resistance factors in materials, due to conservative design procedures that do not always analyze all load capacity, and due to a lack of a precise definition of “failure.”*
>
> *The wind speed map of Fig. 6-1 presents basic wind speeds for the contiguous United States, Alaska, and other selected locations. The wind speeds correspond to 3-s gust speeds at 33 ft (10 m) above ground for exposure category C. Because the National Weather Service (NWS) has phased out the measurement of fastest-mile wind speeds, the basic wind speed has been redefined as the peak gust that is recorded and archived for most NWS stations. Given the response characteristics of the instrumentation used, the peak gust is associated with an averaging time of approximately 3 s. Because the wind speeds of Fig. 6-1 reflect conditions at airports and similar open-country exposures, they do not account for the effects of significant topographic features such as those described in Section 6.5.7. Note that the wind speeds shown in Fig. 6-1 are not representative of speeds at which ultimate limit states are expected to occur. Allowable stresses or load factors used in the design equation(s) lead to structural resistances and corresponding wind loads and speeds that are substantially higher than the speeds shown in Fig. 6-1.*

Until more research on the gust structure of tornadoes is conducted, wind engineers must use the same ASCE 7-05 provisions to calculate wind pressures from tornado-induced winds as they do for other types of extreme winds, but with the modifications provided in this manual or the ICC-500. It is imperative that engineers exercise good judgment in the design of a building to resist tornadoes and hurricanes so that actual building performance falls within expected or desired ranges. It is important to note that other effects such as debris impact may control the design of an element rather than the direct wind pressure.

The design methodology presented in this publication is based on the wind load provisions of ASCE 7-05 - Method 2, modified only to the extent that the values of some factors have been specifically recommended because of the extreme nature of tornado and hurricane winds. If the values of all coefficients and factors used in determining wind pressures are selected by the user, the results would likely be overly conservative and not representative of the expected building behavior during an ultimate level event (extreme-wind event).

6.1.2 Design Wind Speeds for Tornado Safe Rooms

Historical data were the key tool used to establish wind speeds and zones associated with areas susceptible to tornado occurrence. The Storm Prediction Center (SPC) archives were searched for data on historical tornadoes, including the time, location, and path of tornado occurrence and the intensity of the tornado.

The National Weather Service assigns an intensity F scale measurement to each tornado occurrence. The F Scale was developed by Dr. T.T. Fujita in 1971 (Fujita 1971). The intensity F Scale is based on the appearance of damage to buildings and other structures. Dr. Fujita assigned a wind speed range to each F Scale level of damage and determined that the ranges represent the fastest ¼-mile wind speeds. A modified version of the F Scale was developed by Texas Tech University and a panel of wind engineering experts in 2007. This modified version, called the Enhanced Fujita (EF) Scale, was shown in Table 4-1 along with the original F Scale. The wind speed ranges associated with both scales are based on subjective evaluation of tornado damage that is used to associate tornado intensity and estimated wind speeds with observed damage. The new EF Scale, which was implemented for use in February 2007, has refined damage evaluation methods that consider improvements in the built environment when tornado damage is viewed and used to categorize the tornado and estimate a maximum wind speed associated with the event.

Engineering analyses of damage since 1970 have shown that observed damage to buildings can be caused by storms with wind speeds of less than 200 mph (Mehta 1970, Mehta et al. 1976, Mehta and Carter 1999, Phan and Simiu 1998). Prior to 1970, engineers associated wind speeds above 300 mph with F4 and F5 tornadoes. Although F4 and F5 tornadoes are intense and can cause devastating damage, the wind speeds traditionally assigned to these Fujita categories may well be too high (Minor et al. 1982). Some evidence suggests that wind speeds in tornadoes at ground level can be higher than 200 mph, but the limited engineering assessment of EF4 and EF5 tornado damage presents very few examples that can be definitively attributed to winds with speeds above 225 to 230 mph. There is also a debate over the wind speeds from tornadoes just above the ground and in the first 60 feet above grade and the maximum wind speed that can be produced when close to the ground. Research meteorologists mostly agree with the conclusion that the maximum possible wind speed may be above 200 mph, but cannot agree on the highest predicted wind speed. Therefore, the wind speed zones are based on the occurrence of intense tornadoes, but the specified wind speeds are not necessarily related to the EF scale. These observations and conclusions are partially confirmed by the EF Scale shown in Table 4-1.

Data used for the development of wind speed zones consist of tornado statistics assembled by the NOAA SPC. The statistics used are for the years 1950 through 2006, almost 60 years of data. Tornado occurrence statistics prior to 1950 are available, but they are considered to be of lesser quality. From 1950 to 2006, a total of 50,096 tornadoes were recorded in the contiguous United States. Each of these tornadoes was assigned an F Scale level. Table 6-1 shows the number of recorded tornadoes and percentages for each F Scale level, as well as the cumulative percentages. As noted in Table 6-1, less than 2 percent of the tornadoes are in the F4 category and less than 1 percent of the tornadoes are in the F5 category.

Table 6-1. Tornado Frequencies in the United States (1950-2006)

Fujita Scale	Number of Tornadoes	Percentage	Cumulative Percentage
F0	20,728	43.68	43.68
F1	16,145	34.03	77.71
F2	7,944	16.74	94.45
F3	2,091	4.41	98.86
F4	491	1.03	99.89
F5	50	0.11	100
Totals	47,449	100	

To develop wind speed zones, the occurrences of tornadoes over the 1950-2006 period were tallied for all 80 km x 80 km squares (correlating to weather forecasting and monitoring measuring limitations) and plotted on a geographic information system (GIS) grid map. The number of EF5 tornado occurrences and combined EF3, EF4, and EF5 tornado occurrences within the 80 km x 80 km (2,470 square miles) squares were tabulated for the whole country and presented in Figure 2-2. These frequencies of occurrence data were used as a key factor to produce the wind speed map in Figure 3-1. Tornado damage paths are less than 5 square miles on the average; thus, the area covered by a tornado on the ground is quite small compared to the size of a 2,470-square mile GIS grid map square.

A 250-mph wind speed zone includes all 2,470-square mile GIS grid squares with two or more F5 tornadoes recorded in the last 56 years. The 250-mph zone also includes areas with 10 or more F4 and F5 tornado occurrences combined during this same period. In Figure 2-2, the darkest zone covers the middle part of the United States, where the most intense tornado damage has occurred. It also includes large metropolitan areas of the midwestern and southwestern United States (e.g., Chicago, St. Louis, Dallas-Fort Worth). This area should use tornado safe room design wind speeds of 250 mph (and was previously designated as Zone IV in the 2000 Edition of FEMA 361).

A 200-mph wind speed area (previously designated as Zone III in the 2000 Edition of FEMA 361) was developed using the statistics of EF3 tornado occurrences. EF3 tornadoes are less intense and are generally smaller (cover less area on the ground). Most areas with 20 to 30 F3 tornado occurrences in a 2,470-square mile GIS grid square also had a sufficient number of EF4 and EF5 tornado occurrences to be included in the 250-mph tornado safe room design wind speed zone identified above. To be conservative, the tornado safe room design wind speed zone for 200 mph is extended to cover areas where more than five F3 tornadoes were identified within a single square. This zone extends along the Gulf and lower Atlantic coastal areas to include hurricane winds. There are a couple of GIS grid squares in New York and Massachusetts that fall outside of this zone even though they have more than five F3 tornado occurrences. They are considered outliers and have had less than 10 F3 occurrences.

A 160-mph tornado safe room design wind speed (previously designated as Zone II in the 2000 Edition of FEMA 361) has been identified for the remaining areas east of the Rocky Mountains. The western border for this 160-mph zone approximately follows the Continental Divide. The wind speed of 160 mph covers all tornadoes of EF2 or lower intensity and is 75 percent higher than the speed specified in ASCE 7-05.

In the areas west of the Rocky Mountains, there are relatively few tornado occurrences, and none have been assigned an intensity scale of EF5. Over the past 56 years, only two tornadoes were assigned an intensity of EF4 and only 10 were assigned an intensity of EF3 over the entire region. Further revisions of the NWS data set between 1998 and 2006 resulted in the reclassification of a number of the tornado events west of the Rocky Mountains and caused the elimination of historical data points; this is apparent when comparing the new occurrence map in the revised edition of FEMA 361 with the occurrence map (Figure 2-3) in the original FEMA 361 publication (2000). It was determined that a tornado safe room design wind speed of 130 mph is sufficient for this zone. This safe room design wind speed is about 50 percent higher than the basic wind speeds specified in ASCE 7-05 for the west coast states.

6.1.3 Design Wind Speeds for Hurricane Safe Rooms

Hurricane intensity is assessed using the Saffir-Simpson Scale comprising five categories, C1 through C5; hurricane category C5 is the most intense, with decreasing intensity for each of the lower categories of storms. There are, on the average, five hurricanes recorded annually in the Atlantic, with the average of 1.7 landfalling hurricanes. NOAA's National Hurricane Center has archived data on hurricanes since 1900. Hurricane data include track, central barometric pressure, diameter of the eye, distance to hurricane force winds, maximum wind speeds, and storm surge height. The hurricane classification system has a range of wind speeds assigned to each category of storm as shown in Table 6-2. This table also illustrates the relationship between the 1-minute sustained wind speeds of the Saffir-Simpson scale and the 3-second gust speeds shown in Figure 3-2.

Table 6-2. Saffir-Simpson Hurricane Scale

Saffir-Simpson Scale	1-Minute Sustained Wind Speed (mph)*	3-Second Gust Wind Speed (mph)**
C1	74-95	90-116
C2	96-110	117-134
C3	111-130	135-159
C4	131-154	160-188
C5	155+	189+

Conversion: 1 mph = 0.447 m/s

* Powell 1993

** Durst 1960 (ASCE 7-98)

The wind speeds associated with each category of storm are considered to be 1-minute sustained wind speeds (Powell et al. 1994). These wind speeds are converted to equivalent 3-second gust speeds using Figure C6-1 in the Commentary of ASCE 7-05 (Durst 1960). The 3-second gust speed permits the development of a unified map for wind speed, as well as use of ASCE 7-05 for determining wind loads. The total number of hurricanes rated category C3, C4, or C5 that struck each U.S. Gulf and Atlantic coast state during the period of 1900 to 2006 were also identified and included in the preparation of Figure 3-2. The data show that no hurricanes of intensity C4 and C5 have made landfall north of the North Carolina coast. Also, during the last 100 years, only three category C5 storms have made landfall – an unnamed hurricane struck Florida in 1935, Hurricane Camille made landfall in Mississippi and Louisiana in 1969, and Hurricane Andrew in September 1992 (although Andrew has been classified both a C4 and a C5, it is currently categorized a C5 by NOAA).

Based on those historical data, and hurricane simulation models, a hurricane hazard map was developed to identify appropriate hurricane design wind speeds for shelters for the ICC-500. Simulation model results for the development of design wind speed maps have been incorporated in the United States wind loading standards since 1982. The ICC-500 standard committee recognized that using computer-based simulation models is the accepted method for developing hurricane hazard curves, where "hazard curves" are wind speed contours representing the expected maximum hurricane-induced wind speed for a location as it makes landfall and moves inward away from the water.

The hurricane hazard curves or wind speed contours were generated using the hurricane simulation models as described in Vickery et al. 2000[1] and Vickery et al. 2006.[2] The version

[1] Vickery, P.J., P.F. Skerlj, and L.A. Twisdale Jr., "Simulation of hurricane risk in the U.S. using an empirical track model," *Journal of Structural Engineering*, ASCE, Vol. 126, No. 10, October 2000.

[2] Vickery, P.J., J.X. Lin, P.F. Skerlj, and L.A. Twisdale Jr., "The HAZUS-MH hurricane model methodology part I: Hurricane hazard, terrain and wind load modeling", *Natural Hazards Review*, ASCE, Vol. 7, No. 2, May 2006.

of the model described in Vickery et al. 2000 forms the basis of the wind speeds given in the Basic Wind Speed Map (Figure 6-1) in ASCE 7-98 through ASCE 7-05. The version of the model described in Vickery et al. 2006 is a refinement to the model discussed in Vickery et al. 2000 and forms the basis of the hurricane hazard model which is used by FEMA's HAZUS-MH software. Further, the Vickery et al. 2006 model was used to develop the wind speed contours, and thus the shelter design wind speeds, for the hurricane hazard in the ICC-500. It should be noted that the model from Vickery et al. 2006 yields slightly lower wind speeds in the Florida panhandle as compared to the model described in Vickery et al. 2000.[3]

As part of the preparation of the ICC-500, the ICC-500 committee had several wind speed maps prepared using the Vickery et al. 2006 model. The maps that were prepared and considered by the ICC-500 committee had mean recurrence intervals of 1,000, 2,000, 5,000, and 10,000 years. In order to develop the wind speed contours presented on the maps, the model was run where wind speeds associated with 100,000 years of simulated hurricanes were recorded at a number of grid point locations near the hurricane coast. At each location, a hurricane hazard curve was constructed using the methodology outlined in Vickery et al. (2000) and the 1,000-, 2,000-, 5,000, and 10,000-year mean recurrence interval (MRI) values were extracted from the curves and used to develop the contour maps. Each of the maps was then reviewed by the committee to determine if the appropriate ultimate wind speed from the hurricane event was being identified.

After considering the historical nature of landfalling hurricanes, their decay (or weakening) as these storms move inland, and the frequency at which these storms have and may occur, the ICC-500 committee selected maps to represent the ultimate hazard from the hurricane hazard. The map, the *Shelter Design Wind Speeds for Hurricanes Map*, Figure 304.2.2 of ICC-500 has wind speeds ranging from a minimum of 160 mph to 225 mph for the contiguous United States (see map for island and U.S. territory wind speeds) and is associated with the 10,000-year MRI.

The FEMA review committee responsible for the safe room publications update reviewed the work used to prepare the ICC-500 hurricane hazard map. The work of the ICC-500 committee was the best information regarding hurricane hazard mapping that was available for consideration. After a thorough review of the information and coordination with several members of the ICC-500 standard committee, the FEMA review committee concurred with the findings of the ICC-500 committee and agreed to move forward with the hurricane hazard map developed as part of the ICC-500 standard development because it provided defendable, scientific data to revise the previous wind hazard data used in the hurricane-prone regions. The hurricane safe room design wind speed map presented in Figure 3-2 (and the additional, enlarged regional maps) have been included here with permission from the ICC-500 standard committee. This map should be used to select the appropriate hurricane safe room design wind speed to be used in the design calculations presented in Chapter 3.

[3] A revised version of the model discussed in the Vickery et al. 2006 paper is being proposed to determine new maps for upcoming editions of ACSE 7-10. However, since the revisions were not accepted at the time this map was produced for the ICC-500, the proposed updates to the model were not used.

6.1.4 Wind Speeds for Alaska

The State of Alaska does not experience hurricanes and is not prone to a significant number of tornadoes, but it does experience extra tropical cyclone winds and thunderstorms. Since there are no specific records of extreme storms in Alaska, the safe room design wind speeds are based on contours shown on the map in ASCE 7-05. It is recommended that wind speeds of 160 mph be used for areas shown on the ASCE 7-05 basic wind speed map with wind speeds of 110 mph or higher. For the interior areas where the ASCE 7-05 basic wind speeds are less than 110 mph, the safe room design wind speed of 130 mph is recommended; these safe room design wind speeds are shown in Figure 3-1.

6.1.5 Probability of Exceeding the Design Wind Speed

The design wind speeds chosen for safe room guidance were determined with the intent of specifying “near-absolute protection” with an emphasis on life safety. Historically, most tornado deaths have occurred in storms that have been classified as either F4 or F5 (now EF4 or EF5). For hurricanes, the largest storms have typically been the deadliest; however, it should be noted that most of these deaths are associated with storm surge inundation. For either hazard, such intense storms are very rare. Even in the areas of the middle of the country where the risk of EF4 and EF5 tornadoes is greatest, the annual probabilities that a particular structure will be impacted by an EF4 or EF5 tornado are no more than 0.00002 (a 50,000-year MRI). For the purpose of “near-absolute protection,” the safe room design guidance must address these extremely rare events.

Tornado probabilities have typically been based on historical records of tornado observations and classifications within large areas surrounding the site. These areas have ranged from 80 km by 80 km squares to 1 degree latitude and longitude squares. Consequently, they are subject to considerable uncertainty, particularly for the rare EF4 and EF5 storms. The annual probability of 0.00002 was selected as the nominal return period for addressing the tornado wind risk. However, it should be pointed out that the safe room design wind speed contours on the maps have been smoothed and rounded upwards, which reflects the limited number of observations and the large variability that occurs when 40 or 50 years of tornado experience are used to extrapolate very long return periods for these very low probability events. The reanalysis of tornado wind speeds required to produce observed damage has resulted in a decrease in wind speeds assigned to EF Scale-rated events as opposed to F Scale-rated events. The 250-mph 3-second gust design wind speed chosen for the areas with greatest risk from the most intense tornadoes corresponds to a value near the upper end of the old F4 Scale and is actually above the upper end of the current EF5 Scale, so annual probability of 0.000001 may be provided by the maps in some instances. This provides a conservative design wind speed for the riskiest region and allows reductions in wind speeds for other east coast zones based on relative risks while maintaining the lowest tornado design wind speed for that region close to the bottom of the EF4 range.

Selection of an annual probability of 0.00002 (50,000-year MRI) for the hurricane risk was considered unreasonable and the extrapolation methodology based on available data was considered unreliable. The most credible estimates of 3-second gust wind speeds in the intense Category 5 hurricanes that have occurred in this century are slightly above 200 mph. A 2,000-year return period produced 3-second gust design wind speeds for "near-absolute protection" that were on the order of 190 mph in the areas of the country most prone to intense hurricanes. Maps showing annual probability of 0.0001 (10,000-year) for hurricanes produced 3-second gust wind speeds on the order of 210 mph to 225 mph in the most hurricane-prone regions, with a maximum of 255 mph for some Pacific islands. This probability of exceeding the design wind speed was selected for the hurricane design wind speeds in the ICC-500 standard. It produces higher values in the areas most prone to hurricanes than those given in the earlier version of FEMA 361 and provides a consistent risk-based design approach for hurricane shelters and safe rooms. The lower limit of the hurricane safe room and shelter design wind speed was set at 160 mph, which is close to the upper limit of the EF3 Scale and at the upper limit of the old F2 Scale. Since nearly all observed tornadoes spawned by hurricanes have been classified as F3 or lower, this lower limit provides reasonable and conservative design criteria for safe rooms that are intended for use during a hurricane.

6.2 Commentary on Load Combinations

This section presents discussion on the load combinations used in safe room design. Strength Design load combinations are presented first, followed by those for Allowable Stress Design.

6.2.1 Strength Design

The concept used in the development of design methodology for safe rooms is that the designers must use wind loads for extreme events, with mean recurrence intervals as high as 20,000 to 1,000,000 years. Section C2.5 in ASCE 7-05 describes how ASCE has treated the load combinations for extraordinary events and a similar approach is being used in this design guidance. The current load combinations for strength design are (new/revised values are in bold):

Load Combination 1: $1.4(D + F)$

Load Combination 2: $1.2(D + F + T) + 1.6(L + H) + 0.5(Lr \text{ or } S \text{ or } R)$

Load Combination 3: $1.2D + 1.6(Lr \text{ or } S \text{ or } R) + (L \text{ or } \boldsymbol{0.5\, W_x})$

Load Combination 4: $1.2D + \boldsymbol{1.0W_x} + L + 0.5(Lr \text{ or } S \text{ or R})$

Load Combination 5: $1.2D + 1.0E + L + 0.2S$

Load Combination 6: $0.9D + \boldsymbol{1.0W_x} + 1.6H$

Load Combination 7: $0.9D + 1.0E + 1.6H$

Exceptions (from ASCE 7-05):

1. N.A.

2. The load factor on *H* shall be set equal to zero in load combinations 6 and 7 if the structural action due to *H* counteracts that due to *W* or *E*.

3. In combinations 2, 4, and 5, the combination load *S* shall be taken as either the flat roof snow load or the sloped roof snow load.

The recommended changes are as follows:

In load combination 3, replace 0.8W with 0.5 W_x

In load combinations 4 and 6, replace 1.6W with 1.0 W_x

Exception 1 shall not apply, and Exception 3 is added per ASCE 7-05.

The primary reason for the reduction of load factors for extreme events is that the use of load factors for standard design wind speeds is not justified because the proposed wind speeds used in safe room design are considered to be very low probability events. The level of adjustment that the load factors provide for load combinations using the standard design wind speeds is not necessary for load combinations with extreme-wind speeds. It should be noted that the section on load combinations for extraordinary events in ASCE 7-05, Section C2.5 indicates that the extraordinary load be taken as A_k with no load factor applied, because such loads encompass the uncertainty of extraordinary events and are necessarily very conservative; therefore, no adjustment is needed. These are basically the cases presented above as load combinations 4 and 6. Load combinations 3 and 4 are both for vertical forces. Except for the dead load, the other components of these two combinations are reversed. The reduction of the load factor on W_x in the vertical or uplift direction means that uplift loads are being reduced for extremely high wind speeds. Load combination 6 is used for lateral loads and the reduction in load factor in this case reduces the overturning effect caused by wind.

In all load combinations, the W symbol is used to indicate standard design wind pressures from ASCE 7-05. The W_x symbol is used in load combinations for extreme events (safe room design) to indicate extreme wind pressures.

For example, if all of the loads were 1,000 pounds and the basic wind speed was 130 mph, but the safe room design wind speed was 200 mph (and all other wind design parameters were as proposed), then W from 130 mph speed is based on velocity pressure (q). So, designating q_{130} as the velocity pressure, we can calculate the following: $q_{130} = 0.00256 \times K_d \times K_{zt} \times I \times V^2$. Then, from ASCE we know W = q_{130}(p=q) x (area of the building – assumed to be 1,000 square feet for this simple example). For simplicity, assume there is no difference between the design coefficients.

For 130-mph “basic” wind speed:

Load Combination 3: 1.2(1,000) + 1.6(1,000 or 1,000 or 1,000) + (1,000 or 0.8 *42.29* [-1,000]) = -31,032 lbs

Load Combination 4: 1.2(1,000) + 1.6*42.29*(-1,000) + 1,000 + 0.5(1,000 or 1,000 or 1,000) = -64,964 lbs

Load Combination 6: 0.9(1,000) + 1.6*42.29*1,000 + 1.6(1,000) = 70,164 lbs

For a safe room design wind speed of 200 mph, the velocity pressure q_{200} is calculated in the same way except that the K_d (directionality factor) is now 1.0 instead of a value specified for 'basic' wind design in ASCE 7-05 Table 6-4. Therefore, for 200-mph "safe room" wind speed:

Load Combination 3: 1.2(1,000) + 1.6(1,000 or 1,000 or 1,000) + (1,000 or 0.5*102.4*[-1,000]) = -48,400 lbs

Load Combination 4: 1.2(1,000) + 1.0*102.4*(-1,000) + 1,000 + 0.5(1,000 or 1,000 or 1,000) = -99,700 lbs

Load Combination 6: 0.9(1,000) + 1.0*102.4*1,000 + 1.6(1,000) =104,900 lbs

This example illustrates the increase in loads on the building only by the increase in the wind speed and the modification of wind parameter K_d from that used in 'basic' hurricane wind design to that used in tornado safe room design. It should be borne in mind that the load increase resulting from the increase in wind speed for safe room design is tempered by the revised (reduced) load factors, thereby providing much more realistic design loads.

The designer should also consider the appropriate seismic load combinations in Section 2.3.2 of ASCE 7-05. Where appropriate, the most unfavorable effects from both wind and seismic loads should be investigated. Wind and seismic loads should not be considered to act simultaneously (refer to Section 9.2.2 of ASCE 7-05 for the specific definition of earthquake load, E). From the load cases of Section 2.3.2 of ASCE 7-05 and the load cases listed above, the combination that produces the most unfavorable effect in the building, safe room, building component, or foundation shall be used.

NOTE

When a safe room is located in a flood zone, the following load combinations in Section 3.2.1 should be considered:

In V zones and coastal A zones, the $1.0W_x$ in combinations 4 and 6 should be replaced by $1.0W_x + 2.0F_a$.

In non-coastal A zones, the $1.0W_x$ in combinations 4 and 6 should be replaced by $1.0W_x + 1.0F_a$.

6.2.2 Allowable Stress Design (ASD)

The building code in effect should indicate the load combinations to be considered for the design of a building. In the absence of a building code, the designer should use the load combinations of Section 2.4.1 of ASCE 7-05 to ensure that a complete set of load cases is considered. For the MWFRS, C&C, and foundations of extreme-wind safe rooms, designers should also consider the current load cases for ASD (new/revised values are in bold):

Load Combination 1: D + F

Load Combination 2: D + H + F + L + T

Load Combination 3: D + H + F + (Lr or S or R)

Load Combination 4: D + H + F + 0.75(L + T) + 0.75(Lr or S or R)

Load Combination 5: D + H + F + (**0.6W_x** or 0.7E)

Load Combination 6: D + H + F + 0.75(**0.6W_x** or 0.7E) + 0.75L + 0.75(Lr or S or R)

Load Combination 7: 0.6D + **0.6W_x** + H

Load Combination 8: 0.6D + 0.7E + H

The recommended changes for use in FEMA 361 are as follows:

In load combinations 5, 6, and 7, replace W with 0.6W_x.

As illustrated for the strength design load combinations, the ASD combinations will yield the same approximate relationships, so if all terms of the equations are 1,000 pounds except the wind speed is varied from 130 mph to 200 mph and the wind tributary area is 1,000 square feet, the results follow:

For 130-mph wind speed:

Load Combination 5: D + W_x + L = 1,000 + 42,290 + 1,000 = 44,290 lbs

Load Combination 6: D + H + F + 0.75W_x + 0.75L + 0.75(Lr or S or R) = 1,000 + 1,000 + 1,000 + 0.75*42,290 +0.75*1,000 + 0.75*1,000 = 36,218 lbs

Load Combination 7: 0.6D + W_x + H = 0.6*1,000 + 42,290 + 1,000 = 0.6*1,000 + 42,290 + 1,000 = 43,890 lbs

where D = dead load, L = live load, and W_x = extreme-wind load based on wind speed selected from Figure 2-2.

NOTE

When a safe room is located in a flood zone, the following load combinations in Section 6.2.2 should be considered per ASCE 7-05:

In V zones and coastal A zones, 1.5F_a should be added to load combinations 1 and 2.

In non-coastal A zones, 0.75F_a should be added to load combinations 1 and 2.

For 200-mph safe room wind speed:

Load Combination 5: D + 0.6W_x + L = 1,000 + 0.6*102,400 + 1,000 = 63,440 lbs

Load Combination 6: D + H + F +0.6W_x + 0.75L + 0.75(Lr or S or R) = 1,000 + 1,000 + 1,000 + 0.6*102,400 + 0.75*1000 + 0.75*1,000 = 65,940 lbs

Load Combination 7: 0.6D + 0.6W_x + H = 0.6*1,000 + 0.6*102,400 + 1,000 = 63,040 lbs

As mentioned in Section 6.2.1, wind loads determined from the safe room design wind speed maps presented in Figures 3-1 and 3-2 are considered extreme loads. Safe rooms are required to protect their occupants during extreme windstorms. When live load (transient load) is to be combined with wind load, live load is multiplied by a factor of 0.5, but no reduction should be taken for wind loads except as specifically shown above for the extreme-wind event being considered. In addition, allowable stress should not be increased for designs based on the wind loads specified in this document.

Finally, the designer should consider the appropriate seismic load combinations in Section 2.4.1 of ASCE 7-05. Where appropriate, the most unfavorable effects from both wind and seismic loads should be investigated. Wind and seismic loads should not be considered to act simultaneously (refer to Section 9.2.2 of ASCE 7-05 for the specific definition of earthquake load, E). From the load cases of Section 2.4.1 of ASCE 7-05 and the load cases listed above, the combination that produces the most unfavorable effect in the building, safe room, building component, or foundation should be used.

The determination of which load combination method should be used is dictated in part by the materials of construction chosen for the safe room. The masonry, concrete, and steel trade organizations have strength requirements in their respective codes that have been used for several years and there has been research and testing done for these ultimate loads situations. The wood industry has strength requirements in part of the wood design codes, but the use of these requirements is not widespread. Many mechanical connectors still do not give ultimate loads, but rather specify the allowable load with a stress increase to be used for high load but short duration, such as wind and seismic events.

6.2.3 Combination of Loads – MWFRS and C&C

According to ASCE 7-05, the MWFRS is an assemblage of structural elements assigned to provide support and stability for the overall structure and, as a consequence, generally receives wind loading from all surfaces of the building. Elements of the building envelope that do not qualify as part of the MWFRS are identified as C&C and are designed using C&C wind loads. The elements of low-rise buildings are considered part of the building envelope (C&C) or the MWFRS, depending upon the wind load being considered. For example, MWFRS provisions are used to determine the in-plane shear forces for the design of exterior masonry walls, while C&C provisions are used to determine the out-of-plane design bending loads.

The pressure (positive/inward or negative/outward suction) exerted by the wind flowing over and around a building varies with time and location on the building. The highest pressures occur over small areas for a very short time in the regions of a building where the wind flow separation is quite significant. This flow separation can cause small vortices to form that can cause much higher pressures in small localized areas. These flow separation regions generally occur along the edges of the roof and corners of the exterior walls. Therefore, the design wind pressures for the design of the C&C are higher when the tributary area for the element is small and located in a wind flow separation region. The design pressure for a C&C element can be over twice the pressure used to design the structural framing of the building. Proper assessment of the design wind pressures is critical to developing the design of a building's structural frame and the selection of appropriate exterior cladding.

The majority of the wind load provisions are based on wind tunnel modeling of buildings considering non-cyclonic, straight-line winds. Most wind engineers believe that the results from these wind tunnel tests can be used to determine wind pressure from hurricanes. Tornado wind fields are believed to be more complex than the winds modeled in wind tunnel tests that form the basis for the wind loads calculated in ASCE 7-05. However, in investigations of buildings damaged by tornadic winds, the damage is consistent with damage caused by the forces calculated by ASCE 7-05. For this reason, use of ASCE 7-05 provides a reasonable approach to calculating wind loads for tornadoes, even though it is known that these winds are more complex than the wind fields used in the models.

Design wind loads can cause axial, in-plane, and out-of-plane forces to act on the same building element. The combination of these loads should be considered in the design of building walls. For example, consider the exterior reinforced masonry wall shown in Figure 6-1. Depending on wind direction, the building walls carry different combined loads. For wind direction 1, the wall element shown acts as a shearwall and may experience axial, shear, and bending effects (from wind suction pressures) or axial and shear effects only. When either of these conditions exists, the designer should calculate and combine these loads using MWFRS loads. For wind direction 2, however, the loads on the wall are from axial and out-of-plane bending effects. For this condition, the designer should use MWFRS loads to calculate axial loads and C&C loads to calculate the bending loads when combining loads that affect the design of the wall.

It has been previously stated that, when wind blows over a building, a myriad of forces act on the structure. These forces may cause the building to overturn, deform by racking or bending of components, or collapse and fail at the component junctions or joints. Chapter 3 describes how these wind loads affect

NOTE

C&C elements include wall and roof members (e.g., joists, purlins, studs), windows, doors, fascia, fasteners, siding, soffits, parapets, chimneys, and roof overhangs. C&C elements receive wind loads directly and transfer the loads to other components or to the MWFRS.

a building or safe room. To calculate the loads corresponding to the design wind, the design professional should refer to ASCE 7-05 when calculating the wind pressures on the safe room.

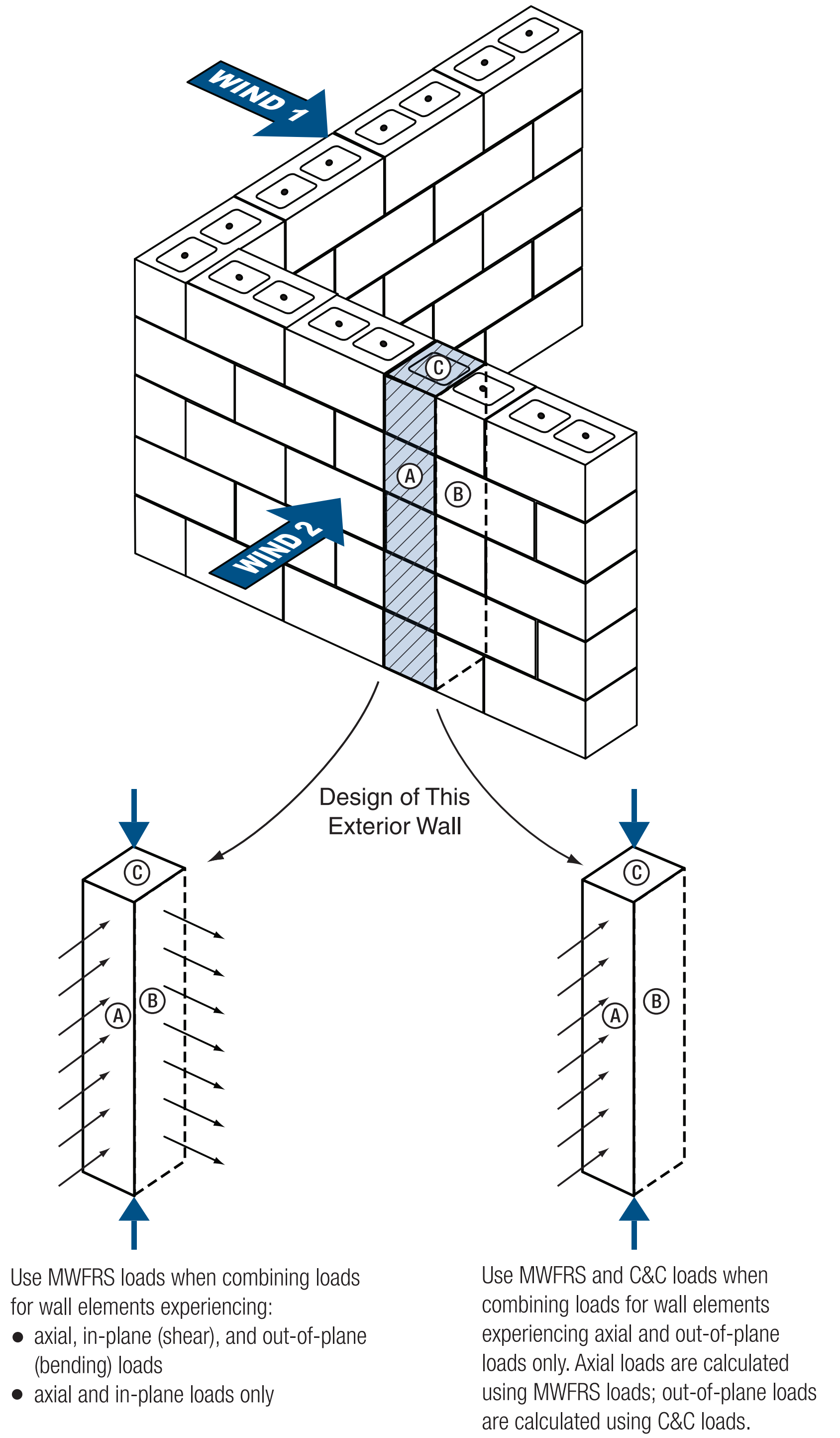

Figure 6-1. MWFRS combined loads and C&C loads acting on a structural member

6.3 Commentary on Tornado Community Safe Room Design Criteria

When wind loads are considered in the design of a building, lateral and uplift loads (discussed in Chapter 3) must be properly applied to the building elements along with all other loads. The design of the safe room relies on the approach taken in ASCE 7-05 for wind loads. For consistency, the designer may wish to use ASCE 7-05 to determine other loads that may act on the safe room. The IBC 2006 and IRC 2006 also reference ASCE 7-05 for determining wind loads. These wind loads should then be combined with the gravity loads and the code-prescribed loads acting on the safe room in load combinations that are presented in Sections 6.2.1 and 6.2.2.

Design wind loads for buildings are generally treated separately for the design of the structural system and the design of the cladding and its attachment to the structural system. Design loads for the structural system of a safe room start with the basic loads from the applicable building code governing the non-refuge use of the safe room. The determination of design wind loads acting on the safe room is based on standard provisions and formulas (equations) for the MWFRS as defined in ASCE 7-05. The design of cladding and its attachment to the structural system is based on standard provisions and formulas for the C&C. Wall and roof panels should also be checked for out-of-plane loading associated with C&C loads for the appropriate tributary areas.

6.3.1 Wind Design Parameters Using ASCE 7-05

After it has been determined that a safe room is needed, the next step in the design process is to select the safe room design wind speed from the maps in Figures 3-1 and 3-2, or the highest of either (for a combined hazard safe room). The four zones on the map in Figure 3-1 have corresponding design wind speeds of 130 mph, 160 mph, 200 mph, and 250 mph. Similarly, the wind contours on the map on Figure 3-2 represent the hurricane hazard as wind speed isobars that range from 160 to 225 mph for the mainland U.S. and increase to up to 255 mph for specific island territories. Depending upon the hazard, one wind speed should be selected as the safe room design wind speed. The safe room design wind speed should be used to determine the wind-generated forces from tornadoes that act on either the structural frame or load-bearing elements of a building or safe room (MWFRS) and the exterior coverings of a building or safe room (C&C).

It is recommended that all wind loads, both MWFRS and C&C, be calculated using the wind load provisions in Section 6 of ASCE 7-05. When ASCE 7-05 is used for the design of tornado or hurricane safe rooms, only *Method 2 – Analytical Procedure* should be used. The design requirements for tornado and hurricane safe rooms do not meet the requirements for using *Method 1 – Simplified Procedure*. In addition, some of the pressure calculation parameters used in the design of a safe room should be different from those listed in ASCE 7-05 because detailed wind characteristics in tornadoes and hurricanes are not well understood. Based on the wind speed selected from Figure 3-1, the following parameters are recommended for the calculation of wind pressures with Method 2 of ASCE 7-05:

Importance Factor (I)	I = 1.0
Site Exposure	C
Directionality Factor (K_d)	K_d = 1.0
Internal Pressure Coefficient (GC_{pi})	GC_{pi} = +/- 0.55
Height of the safe room is not restricted	

The importance factor (I) is set to 1.0. The importance factor for wind loads in ASCE 7-05 is designed to adjust the velocity pressure to different annual probabilities of it being exceeded (different MRIs). Since the wind speeds in Figure 3-1 are already based on very large MRIs (i.e., low exceedence probabilities), they do not need to be adjusted with the importance factor.

It is recommended that site Exposure C, associated with open terrain, be used to determine design wind forces for safe rooms. In severe tornadoes and hurricanes, ordinary structures and trees in wooded areas are flattened, exposing safe rooms to winds coming over open terrain. Also, very little is known about the variation of winds with height in hurricanes and tornadoes. Use of Exposure C is appropriate until the knowledge of localized winds, turbulence characteristics, and boundary layer effects of winds in hurricanes and tornadoes improves.

The directionality factor (K_d) is conservatively set at 1.0. This is done because wind directions may change considerably during a tornado or higher intensity hurricane and a building may be exposed to intense winds from its most vulnerable direction. Therefore, the reduction of this factor allowed in ASCE for normal building design is not recommended for the design of a safe room.

The ASCE 7-05 equations for determining wind loads also include the topographic factor K_{zt}. Damage documentation in hurricane disasters suggests that buildings on escarpments experience higher forces than buildings otherwise situated. No specific observations on topographic effects in tornadic events are available. The designer is advised to avoid siting safe rooms in locations that are likely to experience topographic effects. If it is necessary to locate a safe room on top of a hill or an escarpment, requirements given in ASCE 7-05 for the topographic factor can be used when calculating wind pressures on safe rooms that are being designed for hurricane winds only.

The design wind loads/pressures for the MWFRS or the C&C of a building are based on the following factors: velocity pressure, an external gust/pressure coefficient, and an internal gust/pressure coefficient. These coefficients are derived from several factors related to the wind field, the wind/structure interaction, and the building characteristics.

The velocity pressure equation (Equation 6-15, ASCE 7-05) is shown in Formula 6-1. The equation for pressure on a building surface for MWFRS for buildings of all heights (Equation 6-17, ASCE 7-05) is shown in Formula 6-2.

formula

Formula 6-1 Velocity Pressure*

$$q_z = (0.00256)(K_z)(K_{zt})(K_d)(V^2)(I)$$

where: q_z = velocity pressure (psf) calculated at height z above ground
K_z = velocity pressure exposure coefficient at height z above ground
K_{zt} = topographic factor
K_d = directionality factor = 1.0
V = safe room design wind speed (mph) (from Figure 3-1 or 3-2)
I = importance factor = 1.0

*From ASCE 7-05

formula

Formula 6-2 Pressure on MWFRS for Low-Rise Buildings*

$$p = (q)(G)(C_p) - (q_i)(GC_{pi})$$

where: p = pressure (psf)
q = q_z for windward walls calculated at height z above ground
q = q_h for roof surfaces and all other walls
G = gust effect factor
C_p = external pressure coefficients
q_i = q_h = velocity pressure calculated at mean roof height
GC_{pi} = internal pressure coefficients = ±0.55

*From ASCE 7-05

The velocity pressure is related to height above ground, exposure, wind directionality, wind speed, and importance factor. Several of these factors account for the boundary layer effects of wind flowing close to the surface of the earth where it interacts with the terrain, buildings, and vegetation.

Values of the exposure factor (K_z) are presented in tabular form in ASCE 7-05. The value of K_z selected should be based on the height of the safe room above grade and the building exposure (Exposure C). The terrain speedup factor (K_{zt}) is based on the acceleration of straight winds over hills, ridges, or escarpments. As previously mentioned, the ASCE provisions for K_{zt} should be followed.

For the MWFRS, the gust effect factor (G) depends on wind turbulence and building dimensions. The gust effect factor can be calculated, or, for a rigid building, G = 0.85 is permitted by ASCE 7-05. The external pressure coefficient (C_p) for the design of the MWFRS is based on the physical dimensions and shape of the building and the surface of the building in relation to a given wind direction.

The equation for pressures on C&C and attachments (Equation 6-22, ASCE 7-05) is shown in Formula 6-3.

Formula 6-3 Pressures on C&C and Attachments*

$$p = (q_h)[(GC_p) - (GC_{pi})]$$

where: **p** = pressure (psf)
q_h = velocity pressure calculated at mean roof height
GC_p = external pressure coefficients
GC_{pi} = internal pressure coefficients = ±0.55

*From ASCE 7-05

The internal pressure coefficient (GC_{pi}), which incorporates the gust factor (G), accounts for the leakage of air entering or exiting the building where the building envelope has been breached. This leakage creates a pressure increase or a decrease within the building. The recommended value of GC_{pi} is ±0.55. This value, associated with partially enclosed buildings and applicable to both the MWFRS and C&C components, was selected for the following reasons:

1. In tornadic events, as discussed in Section 3.3.1, maximum wind pressures should be combined with pressures induced by atmospheric pressure change (APC) if the building is sealed or, like most safe rooms, nearly sealed. Although most buildings have enough air leakage in their envelopes that they are not affected by APC, safe rooms are very "tight" buildings with few doors and typically no windows. A building designed to nullify APC-induced pressures, which usually qualifies as a partially enclosed building as defined by ASCE 7-05, would require a significant number of openings in the safe room to allow pressures to equalize. Allowing wind to flow through the safe room through large openings to reduce internal pressures (venting) could create an unsatisfactory environment for the occupants, possibly leading to panic, injury, or even death. It is important to note that ventilation is needed to ensure that safe room occupants have sufficient airflow to remain safe, but that code-compliant ventilation is not sufficient to nullify APC-induced pressures. Designers who wish to eliminate the need for venting to alleviate APC-induced pressures should use higher values of GC_{pi} (in safe room design, GC_{pi} = ±0.55 is recommended). Design pressures determined using wind-induced internal and external pressure coefficients are comparable to the pressures determined using a combination of wind-induced external pressure coefficients and APC-induced pressures. Thus, the resulting design will be able to resist APC-induced pressures, should they occur.

2. In hurricane events, tornadic vortices are often embedded in the overall storm structure. These tornadoes are considered small and less intense than tornadoes occurring in the interior

of the country. However, swaths of damage reminiscent of tornado damage have been noted in several hurricanes. It has not been confirmed whether these swaths are caused by localized gusts or unstable small-scale vortices. As a conservative approach, to design safe rooms better able to resist long-duration wind forces associated with landfalling hurricanes, designers should use high values of $GC_{pi.}$ This approach will provide reliable and safe designs. It is particularly important that none of the C&C elements (e.g., doors, windows) fail during a windstorm and allow winds to blow through the safe room. The consequences could be the same as those described above for tornadoes.

Additionally, the value of GC_p for C&C elements is related to the location on the building surface and the effective wind area of the element. For systems with repetitive members, adjustments can be taken during the design to gain benefit from the systems. These systems are allowed to use the effective wind area, and not the tributary area, to select the external wind coefficient (which typically results in the designer being able to use a coefficient with smaller value and, therefore, the wind area may be taken as the effective width multiplied by the span length); in these instances, the effective width may be determined by taking 1/3 of the member span. It is not uncommon for the effective wind area for a C&C element to be different from the tributary area for the same element (see Figure 6-2). The effective wind area is applied to select the coefficient used to calculate the magnitude of the design wind pressure, while the tributary area is the area over which the calculated wind pressure is applied for that specific C&C-designed element.

It should be noted that the external gust/pressure coefficient is constant and maximum for effective wind areas less than 10 ft^2 and constant and minimum for effective wind areas greater than 500 ft^2. If the tributary area of a component element exceeds 700 ft^2, the design wind pressure acting on that component may be based on the main MWFRS provisions.

Once the appropriate MWFRS and C&C wind pressures are calculated for the safe room, they should be applied to the exterior wall and roof surfaces of the safe room to determine design wind loads for the structural and non-structural elements of the safe room. After these wind loads are identified, the designer should assemble the relevant load combinations for the safe room.

Finally, the designer should not reduce the calculated wind pressures or assume a lower potential for missile impacts on the exterior walls and roof surfaces of an internal safe room. Although a safe room inside a larger building, or otherwise shielded from the wind, is less likely to experience the full wind pressures and missile impacts, it should still be designed for the design wind pressures and potential missile impacts that would apply to a stand-alone safe room. This is required because it must be assumed that the structure surrounding the internal safe room may sustain substantial damage or collapse in extreme-wind events, offering no protection whatsoever. There is no conclusive research that can quantify allowable marginal reductions in design wind pressure for safe rooms within buildings or otherwise shielded from wind. Likewise, there is no conclusive research that can quantify the marginal increase in debris impact loads as a result of the progressive collapse of the structure surrounding the safe room. For this reason, the designers are cautioned to avoid building areas where an internal safe room may be exposed to an impact of a collapsing large or heavy building component that cannot be quantified and designed for as part of the design process.

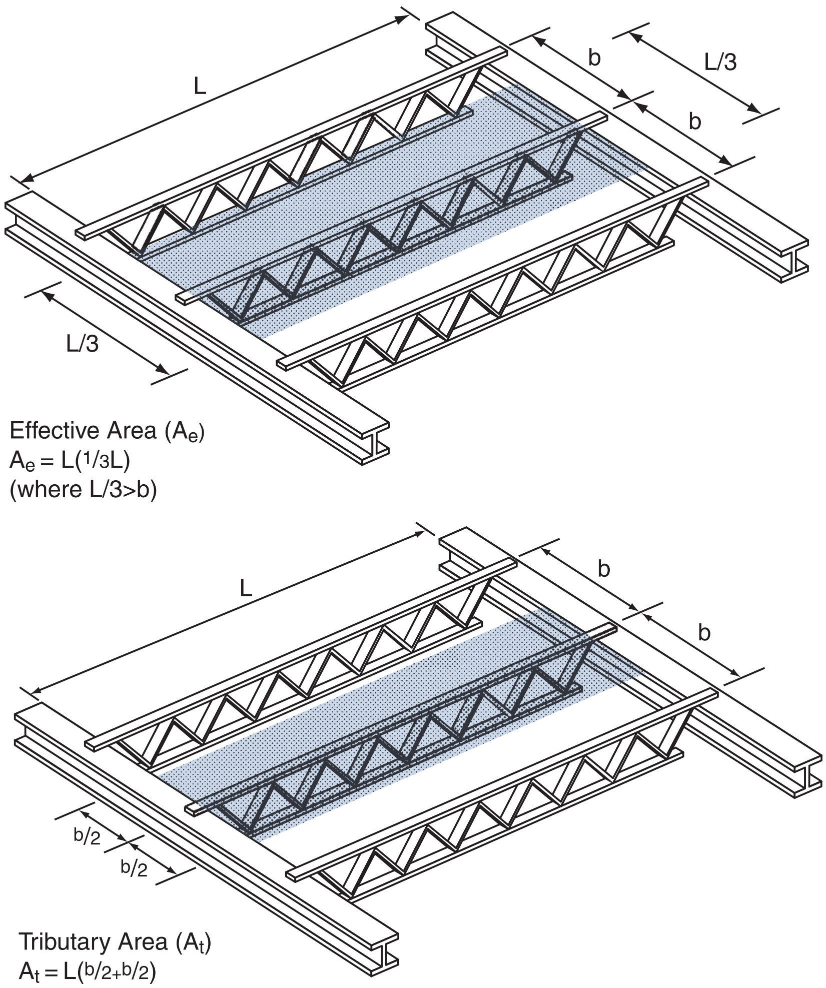

Figure 6-2. Comparison of tributary and effective wind areas for a roof supported by open-web steel joists

6.4 Commentary on Hurricane Community Safe Room Design Criteria

The design methodology used for hurricane community safe rooms is the same as for tornado community safe rooms. The design premise is to provide a building that affords the safe room occupant “near-absolute protection.” The only difference in the design procedure is that hurricane safe room design wind speeds are obtained from the wind speed contours shown in Figure 3-2. These wind speeds represent ultimate design wind speeds.

Site exposure for hurricane safe rooms should be a C exposure unless a B exposure exists on all four sides of the safe room. Since the wind can come in any direction, and the wind is rotating during a hurricane, it is believed if any part of the site exposure is open terrain (Exposure C), that exposure should govern the design. The urban, suburban exposure would need to be present on all four sides in order to ensure that the exposure could effectively reduce the velocity pressure experienced by the safe room area.

The flood design criteria for hurricane safe rooms stipulate that the safe room should be located outside the floodway, the 500-year floodplain, and the 100-year floodplains in coastal areas designated as velocity (V) zones. The safe room should not be located so that access to the safe room can be cut off by flooding. The lowest floor of the safe room area should either be 2 feet above the 100-year flood elevation or at the 500-year flood elevation, whichever is higher. Should the safe room be located in an area subject to storm surge, the safe room must be elevated so that the floor of the protected area is located at or above the storm surge inundation elevation predicted for a Category 5 storm surge.

It is extremely important to conduct a thorough flood hazard analysis on the proposed site or building for a hurricane community safe room. The possibility of flooding from a hurricane event is very high, and since occupants might be in the safe room for some extended period of time (perhaps longer than 24 hours), the scenario of saving people from death or injury caused by extreme winds only to cause them harm by flooding or drowning must be avoided.

The minimum space requirements for hurricane community safe room occupants were shown in Table 3-3 of this publication.

The determination of usable space for any particular building might not be straightforward because of the configuration of the interior. The calculation methodology below is intended to provide guidance on how to determine the usable space. For almost all spaces, the usable space is less than the building footprint because of interior walls or partitions; bathroom or kitchen fixtures; permanently mounted desks, chairs, or tables; or the storage area required to store portable desks, chairs, and tables. The maximum usable space is 85 percent of the footprint as calculated using the parameters below.

Calculation of Usable Floor Area. The usable safe room floor area should be determined by subtracting the floor area of excluded spaces, partitions and walls, columns, fixed or movable objects, furniture, equipment, or other features that under probable conditions can not be removed or stored during use as a safe room from the gross floor area.

An alternative method for determining the usable safe room floor area is to use the following percentages:

- Reducing the gross floor area of safe room areas with concentrated furnishings or fixed seating by a minimum of 50 percent.

- Reducing the gross floor area of safe room areas without concentrated furnishings or fixed seating by a minimum of 35 percent.
- Reducing the gross floor area of safe room areas with open plan furnishings and without fixed seating by a minimum of 15 percent.

6.5 Commentary on Residential Safe Room Design Criteria

The design methodology used for residential safe rooms is the same as for tornado community safe rooms. The design premise is to provide a building that affords the safe room occupant "near- absolute protection." The only difference in the design procedure is for hurricane safe room design; wind speeds are obtained from the wind speed contours shown in Figure 3-2. These wind speeds represent ultimate design wind speeds.

Site exposure for residential safe rooms should be a C exposure in all cases. The safe room should also be designed as a partially enclosed building in all cases. While conservative, it follows the design premise of providing "near-absolute protection."

It is extremely important to conduct a thorough flood hazard analysis on the proposed site or building for a residential safe room. The possibility of flooding from a hurricane event is very high, and since occupants might be in the safe room for some extended period of time (perhaps longer than 24 hours), the scenario of saving people from death or injury caused by extreme winds only to cause them harm by flooding or drowning must be avoided.

It is also important to remember that FEMA does not support placing safe rooms offering protection against extreme-wind events where floodwaters have the potential to endanger occupants within the safe room. Although the ICC-500 allows the placement of residential shelters in areas subject to flooding, FEMA safe room design criteria for residential safe rooms significantly limits the placement of safe rooms in Special Flood Hazard Areas (SFHAs). A residential safe room may only be sited in a mapped SFHA where no wave action or high-velocity water flow is anticipated. Therefore, the installation of a safe room in a home supported by piles, piers, or columns should be scrutinized for its location with respect to flood hazards. With building connectors commercially available, it is extremely difficult to economically and structurally separate the safe room from the elevated floor framing and ensure that the safe room will withstand the forces of extreme winds.

If your safe room is located where coastal or riverine flooding may occur during hurricanes, it should not be occupied during a hurricane. Further, a residential safe room should not be located in an area subject to storm surge inundation. Although occupying such a safe room during a tornado may be acceptable provided that the safe room is located where it will not be flooded by rains associated with other storm and tornado events, it should not be used during a hurricane. A residential safe room sited in the SFHA should meet the flood-specific FEMA safe room design criteria presented in Section 3.6. Consult your local building official or local NFIP representative to determine whether your home or small business, or a proposed stand-alone safe room site, is susceptible to coastal or riverine flooding. In any case, the installation of any safe room in a hurricane-prone area should be coordinated with local emergency management and law enforcement to ensure that its use during extreme-wind events is not a violation of any local or state evacuation plan.

6.6 Commentary on Continuous Load Path Concepts

Structural systems that provide a continuous load path are those that support all loads acting on a building: laterally and vertically (inward and outward, upward and downward). Many buildings have structural systems capable of providing a continuous load path for gravity (downward) loads, but they are unable to provide a continuous load path for the lateral and uplift forces generated by tornadic and hurricane winds.

A continuous load path can be thought of as a “chain” running through a building. The “links” of the chain are structural members, connections between members, and any fasteners used in the connections (e.g., nails, screws, bolts, welds, or reinforcing steel). To be effective, each “link” in the continuous load path must be strong enough to transfer loads without permanently deforming or breaking. Because all applied loads (e.g., gravity, dead, live, uplift, lateral) must be transferred into the ground, the load path must continue unbroken from the uppermost building element through the foundation and into the ground.

In general, the continuous load path that carries wind forces acting on a building’s exterior starts with the non-load-bearing walls, roof covering and decks, and windows or doors. These items are classified as C&C in ASCE 7-05. Roof loads transfer to the supporting roof deck or sheathing and then to the roof structure made up of rafters, joists, beams, trusses, and girders. The structural members and elements of the roof must be adequately connected to each other and to the walls or columns that support them. The walls and columns must be continuous and connected properly to the foundation, which, in turn, must be capable of transferring the loads to the ground.

Figure 6-3 illustrates typical connections important to continuous load paths in masonry, concrete, or metal-frame buildings (e.g., residential multi-family or non-residential buildings); Figure 6-4 illustrates a continuous load path in a typical commercial building. Figure 6-3 also illustrates the lateral and uplift wind forces that act on the structural members and connections. A deficiency in any of the connections depicted in these figures may lead to structural damage or collapse.

In a tornado or hurricane safe room, this continuous load path is essential and must be present for the safe room to resist wind forces. The designers of safe rooms must be careful to ensure that all connections within the load path have been checked for adequate capacity. Again, designers should refer to ASCE 7-05 and the design wind speed and parameters specified in this manual when determining the loads on the building elements and ensure that the proper pressures are being used for either MWFRS or C&C building elements.

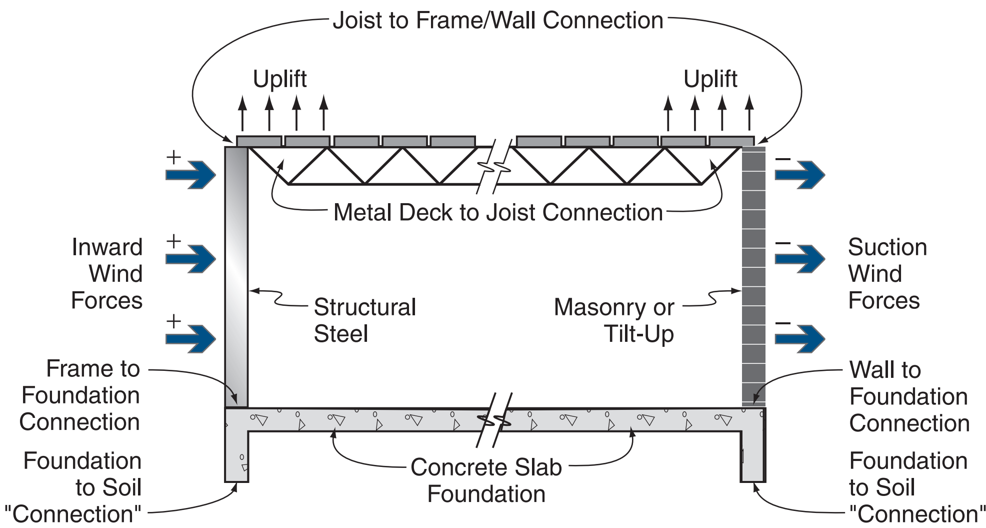

Figure 6-3. Critical connections important for providing a continuous load path in a typical masonry, concrete, or metal-frame building wall. (For clarity, concrete roof deck is not shown.)

6.7 Anchorages and Connections

A common failure of buildings during extreme-wind events is the failure of connections between building elements. This failure is often initiated by a breach in the building envelope, such as broken doors and windows or partial roof failure, which allows internal pressures within the building to increase rapidly. This phenomenon is discussed in Chapter 3 and the schematic in Figure 4-4 illustrates the forces acting on buildings when a breach occurs.

Anchorage and connection failures can lead to the failure of the entire safe room and loss of life. Therefore, the design of all anchorages and connections should be based on the C&C loads calculated from ASCE 7-05 and on the specified design assumption stated in Section 6.7.2. All effects of shear and bending loads at the connections should be considered.

6.7.1 Roof Connections and Roof-to-Wall Connections

Adequate connections must be provided between the roof sheathing and roof structural support, steel joists, and other structural roofing members and walls or structural columns. These are the connections at the top of the continuous load path and are required to keep the roof system attached to the safe room.

Reinforcing steel, bolts, steel studs, welds, screws, and nails are used to connect roof decking to supporting members. The size and number of these connections required for a safe room depend on the wind pressures that act on the roof systems. Examples of connection details that have been designed for some of these conditions may be found in Appendices C and D for cast-in-place and pre-cast concrete safe room designs.

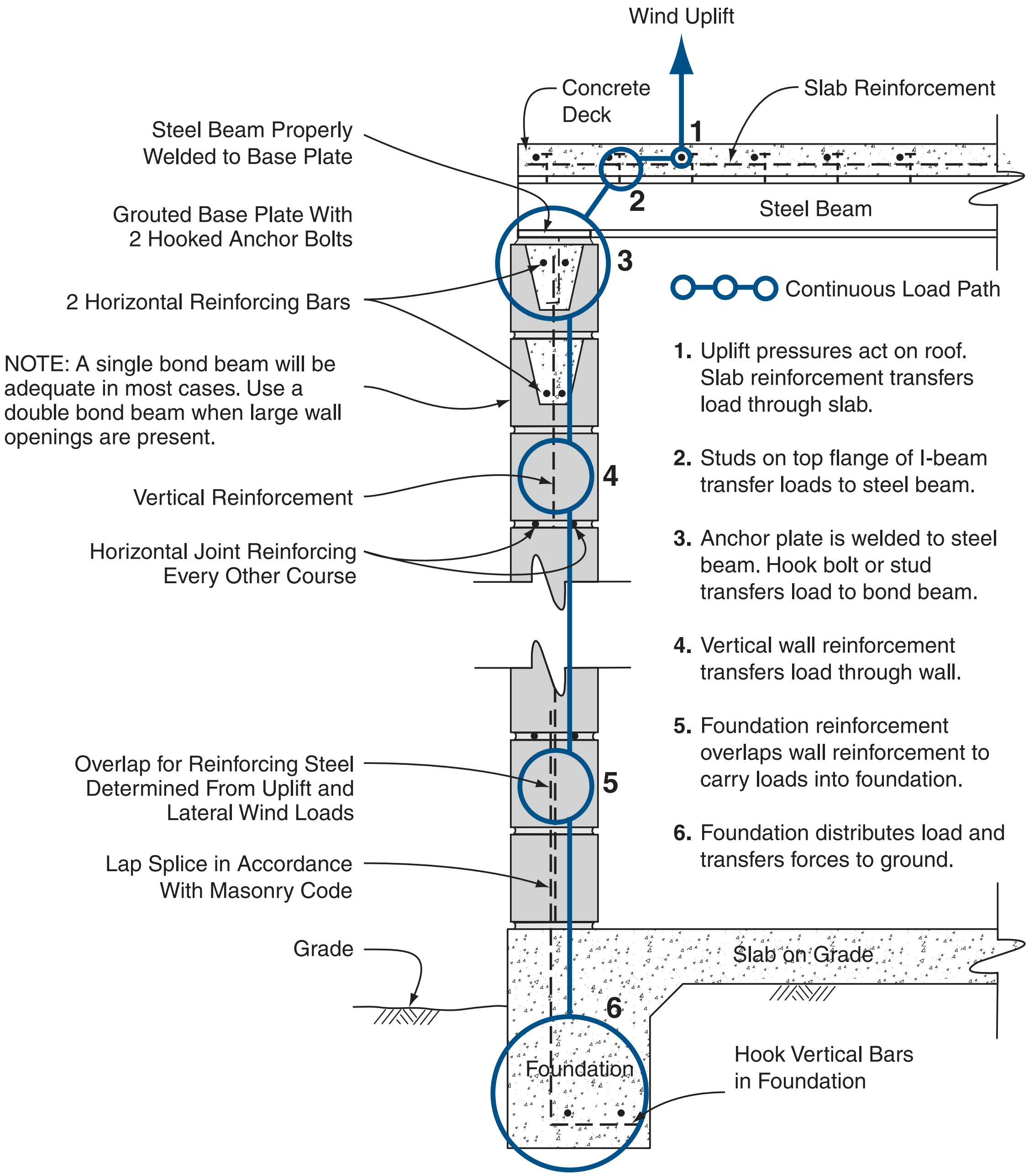

Figure 6-4. Continuous load path in a reinforced masonry building with a concrete roof deck

Figure 6-5 shows damage to a school in Oklahoma that was struck by a tornado. The school used a combination of construction types: steel frame with masonry infill walls and load-bearing unreinforced masonry walls. Both structural systems support open-web steel joists with a lightweight roof system composed of light steel decking, insulation, and a built-up roof covering with aggregate ballast.

The figure highlights a connection failure between a bond beam and its supporting unreinforced masonry wall as well as the separation of the bond beam from roof bar joists. See Figure 6-4 for an illustration of connections in a reinforced masonry wall that are likely to resist wind forces from a tornado or hurricane. Note that four connection points – between the roof decking and joists, the joist and the bond beam, the bond beam and the wall, and the wall to the foundation – are critical to a sound continuous load path.

Figure 6-5. Failure in this load path occurred between the bond beam and the top of the unreinforced masonry wall. This school building was in the path of an F4 tornado vortex.

6.7.2 Foundation-to-Wall Connections and Connections Within Wall Systems

Anchor bolts, reinforcing steel, imbedded plate systems properly welded together, and nailed mechanical fasteners for wood construction are typical connection methods used to establish a load path from foundation systems into wall systems. These connections are the last connections in the load path that bring the forces acting on the building into the foundation and, ultimately, into the ground. The designer should check the ability of both the connector and the material into which the connector is anchored to withstand the design forces.

Figure 6-6 shows two columns from a building that collapsed when it was struck by the vortex of a weak tornado. Numerous failures at the connection between the columns and the foundation were observed. Anchor bolt failures were observed to be either ductile material failures or, when ductile failure did not occur, embedment failures.

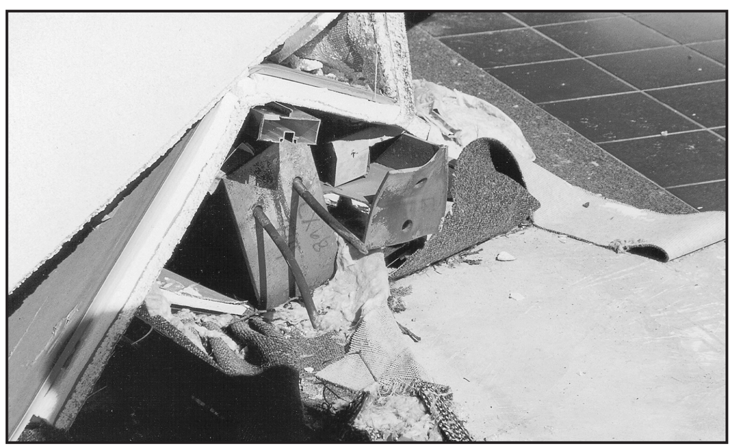

Figure 6-6. These two steel columns failed at their connection to the foundation. The anchor bolts that secured the column released from the concrete (embedment failure) while the anchor bolts that secured the column on the right experienced a ductile failure.