Design Guide 35

Steel-Framed Storm Shelters

Smarter.
Stronger.
Steel.

Design Guide 35

Steel-Framed Storm Shelters

Roger A. LaBoube, PE, PhD
James M. Fisher, PE, PhD

American Institute of Steel Construction

Printed in the United States of America

Authors

Roger A. LaBoube, Ph.D., P.E., is Curator's Distinguished Teaching Professor Emeritus of Civil Engineering and Director of the Wei-Wen Yu Center for Cold-Formed Steel Structures at the Missouri University of Science & Technology (formerly University of Missouri–Rolla). Dr. LaBoube holds B.S., M.S. and Ph.D. degrees in Civil Engineering from the University of Missouri–Rolla. Dr. LaBoube is a member of the American Iron and Steel Institute's Committee on Specifications for the North American Specification for the Design of Cold-Formed Steel Structural Members and chairperson of the AISI Committee on Framing Standards.

James M. Fisher, Ph.D., P.E., is Emeritus Vice President of CSD in Milwaukee, Wisconsin. He received a B.S. degree in Civil Engineering from the University of Wisconsin in 1962. After serving two years as a Lieutenant in the U.S. Army Corps of Engineers, he continued his formal education, and in 1965 and 1968, respectively, he received his M.S. and Ph.D. degrees in Structural Engineering from the University of Illinois. Dr. Fisher served as the Chairman of the AISC Committee on Specifications from 2003 until 2010. He continues to be active on several AISC Specification Task Committees. He is a member of the AISI Cold-Formed Specification Committee and ASCE 7. In addition, Dr. Fisher serves as the Consulting Engineer to the SJI.

Acknowledgments

The authors thank the American Institute of Steel Construction for funding this Design Guide. We also express our appreciation to Mr. Robert J. Neumann, R.A., for his assistance with the figures. We would also like to thank the review committee for their time and effort in reviewing this document:

Craig Archacki
Marc Barter
Charles Carter
Ken Charles
Bill Coulbourne
Brad Davis
Jay Harris
Mark Holland
Benjamin Kaan
Jay Larson
Bonnie Manley
Davis Parsons
Bob Paul
Don Scott
Victor Shneur
Lee Shoemaker
Tom Smith
Tom Sputo
Ron Yeager

AISC gratefully acknowledges the funding contributions made by the American Iron and Steel Institute (AISI), the Metal Building Manufacturer's Association (MBMA), the Steel Deck Institute (SDI), the Steel Framing Alliance (SFA), the Steel Joist Institute (SJI), and the Steel Tube Institute (STI).

Preface

This Design Guide summarizes the state-of-the-art in design requirements and design guidance to facilitate the incorporation of a steel-framed storm shelter or safe room utilizing typical industry standard products. Information is also presented to assist the design engineer in the selection of appropriate wall and roof assemblies to resist the impact of wind-borne debris.

Table of Contents

Chapter 1
Introduction

High wind events such as hurricanes and tornadoes have created a call for storm shelters or safe rooms to be provided in schools and other critical occupancy buildings. According to data collected by USTornadoes.com, an average of 1,224 tornadoes touch down each year in the United States. The top 10 states for the average number of tornadoes for the period 1991 to 2015 are as follows, in order from highest to lowest: Texas, Kansas, Oklahoma, Florida, Nebraska, Illinois, Colorado, Iowa, Alabama and Missouri. The purpose of this Design Guide is to summarize the state-of-the-art in design requirements and design guidance to facilitate the incorporation of a steel-framed storm shelter or safe room utilizing typical industry standard products.

The primary difference in a building's structural system when designed for use as a storm shelter or safe room, as compared to conventional construction, is the magnitude of the design wind forces and the need to resist wind-borne debris. Safe rooms and storm shelters are designed for greater wind speeds, which correspond to greater wind pressures, and for wind-borne missiles. It is important to understand that these two criteria are not concurrently occurring design events.

A storm shelter or safe room typically will be either an interior room within a building or a designated wing of a building. The concepts presented herein may be employed for the design of a stand-alone structure or the retrofit of an existing structure. Although current design guidelines address both community shelters and residential shelters, the primary focus of this design guide is a discussion of the design for community shelters.

1.1 STORM SHELTER VERSUS SAFE ROOM

Storm shelter and safe room are two terms that have been used interchangeably in engineering literature; however, there is a design difference. Storm shelters are structures designed to meet the criteria of *ICC/NSSA Standard for the Design and Construction of Storm Shelters,* ICC 500 (ICC, 2014), hereafter referred to as ICC 500. ICC 500 further defines residential storm shelters as serving occupants of dwelling units and having an occupant load not to exceed 16 persons and community storm shelters to be any storm shelter not defined as a residential storm shelter. Safe rooms are designed in accordance with *Safe Rooms for Tornados and Hurricanes: Guidance for Community and Residential Safe Rooms*, FEMA P-361 (FEMA, 2015a), hereafter referred to as FEMA P-361. FEMA P-361 refers to all structures constructed to meet its more restrictive criteria as "safe rooms." The FEMA P-361 and ICC 500 similarities and differences will be discussed herein. Also, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16 (ASCE, 2016), hereafter referred to as ASCE/SEI 7-16, design guidance will be reviewed.

All safe room design criteria of FEMA P-361 meets or exceeds the storm shelter requirements of ICC 500.

1.2 INDUSTRY DESIGN CODES, STANDARDS AND GUIDELINES

The 2015 *International Building Code* (ICC, 2015), Section 423, provides building code requirements for storm shelters. Specifically, for critical emergency operations in areas where the design wind speed for a tornado is 250 mph; 911 call stations; emergency operation centers; and fire, rescue, ambulance and police stations are to have a storm shelter constructed in accordance with ICC 500. Also, in areas where the tornado shelter design wind speed is 250 mph, all Educational Group E occupancies with an aggregate occupant load of 50 or more are required to have a storm shelter constructed in accordance with ICC 500. ICC 500 is a consensus standard for shelter design and construction and was first incorporated by reference into the 2009 *International Building Code* (ICC, 2009a) and the *International Residential Code for One- and Two-Family Dwellings* (ICC, 2009b).

The primary guidance for the design of safe rooms is provided by FEMA P-361. The FEMA guidelines must be adhered to when federal funding is involved. For example, schools, hospitals, or community buildings responsible for public safety may consider federal funding for construction of a safe room.

Neither ICC 500 nor FEMA mandates the design and construction of safe rooms or storm shelters within a jurisdiction. Rather, these documents provide guidance or requirements for regulating and enforcing the proper design and construction of safe rooms and shelters.

Both the building's structural system and envelope—that is, cladding system—have to be considered when designing a storm shelter or safe room. Wind loads are codified based on the local climate conditions and building configuration. However, the primary difference in a building's structural and cladding system designed for use as a storm shelter or safe room is the increased magnitude of the wind forces that the shelter or safe room must be designed to withstand. Additionally, for the building envelope or cladding system, the governing design criterion is typically the resistance to wind-borne debris. Both tornadoes and hurricanes produce wind-borne debris, or missiles, that may cause injuries and significant damage. Buildings located in wind-borne debris regions must have impact-resistant cladding or glazing systems, or added protection systems may be employed to protect the glazing.

Because of the critical nature of these structures, a peer review is required by ICC 500. A peer review is required to be conducted by an independent licensed design professional for compliance with the requirements of ICC 500, Chapters 3, 5, 6 and 7, for the following storm shelter types:

- Community shelters with an occupant load greater than 50.
- Storm shelters in elementary schools, secondary schools, and day care facilities with an occupant load greater than 16.
- Storm shelters in Risk Category IV (essential facilities) as defined in the 2015 *International Building Code*, Table 1604.5.

Chapter 2
Structural Design Load Criteria

A storm shelter or safe room must resist combinations of gravity, wind, flood and seismic loading. In addition, because wind forces may cause wind-borne debris, the building envelope for both storm shelters and safe rooms must be designed to withstand the impact of wind-borne debris, commonly referred to as missiles.

The hurricane wind speed map in *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-10 (ASCE, 2010), hereafter referred to as ASCE/SEI 7-10, is based on updated hurricane modeling methods. Determination of these loads is well understood for hurricanes; however, for tornadoes the determination of wind loads is not as well understood. The tornado wind speed map in ASCE/SEI 7-10 was developed from an analysis of historic tornadoes and represents a deterministic map of maximum tornadic wind speeds likely to occur in different regions of the country.

There is considerable uncertainty when it comes to the development and justification for tornado design wind loads. It is important to recognize the enhanced Fujita (EF) scale, used to classify a tornado, is essentially a damage scale based on a series of damage indicators (DI) and degrees of damage (DOD). The expert opinions that were used to establish the wind speeds associated with the DI and DOD were at least partially shaped by observations of damage in hurricanes where there was a higher degree of confidence in wind speeds than in tornadoes.

2.1 ICC 500 CRITERIA

The purpose of ICC 500 as defined by Section 101.1 is as follows:

> The purpose of this standard is to establish minimum requirements to safeguard the public health, safety and general welfare relative to the design, construction and installation of storm shelters constructed for protection from high winds associated with tornadoes and hurricanes. This standard is intended for adoption by government agencies and organizations for use in conjunction with model codes to achieve uniformity in the technical design and construction of storm shelters.

Section 101.2 goes on to state that the storm shelters may be areas within buildings or separate detached buildings or rooms.

Although the standard encompasses design criteria for all aspects of storm shelter design, the focus herein will be the structural design criteria as summarized in ICC 500, Chapter 3, which applies to both hurricane and tornado loading.

The current edition of ICC 500 references ASCE/SEI 7-10, therefore, load criteria will be used from the 2010 version of ASCE/SEI 7 rather than the 2016 version.

2.1.1 Load Combinations

Both load and resistance factor design (LRFD) and allowable strength design (ASD) are permitted. The respective load combinations as stipulated by ASCE/SEI 7-10, Sections 2.3 and 2.4, are to be used; however, Exception 1 to ASCE/SEI 7-10, Section 2.3.2, does not apply.

2.1.2 Rain Loads

Rain loads are to be determined in accordance with ASCE/SEI 7-10. For hurricane shelter roofs, additional rainfall must be considered. The hurricane rainfall rate is to be determined by adding 6 in. of rainfall per hour to the rainfall rate as defined by ICC 500, Figure 303.2.

2.1.3 Roof Live Loads

Storm shelter roofs are to be designed for the minimum roof live loads in accordance with ASCE/SEI 7-10, but not less than 100 psf for tornado shelters or 50 psf for hurricane shelters.

2.1.4 Hydrostatic Loads

Underground portions of storm shelters are to be designed for buoyancy forces and hydrostatic loads, assuming that the groundwater level is at the surface of the ground at the entrance to the storm shelter, unless adequate drainage is available to justify designing for a lower groundwater level.

2.1.5 Flood Loads

ASCE/SEI 7-10 is to be used to determine the design flood loads. ICC 500, Section 401, stipulates the definition for the design flood elevation.

2.1.6 Wind Loads

The basic wind load determination is accomplished using ASCE/SEI 7-10, Chapter 27, Part 1, or Chapter 28, Part 1. However, ICC 500, Section 304, provides additional wind load considerations.

According to ICC 500, the design wind speeds for tornado shelters and hurricane shelters differ and are given by Figures 2-1 and 2-2. When calculating the design wind pressure, the directionality factor, K_d, is taken as 1.0. For tornado shelters, the topographic factor, K_{zt}, need not exceed 1.0.

Exposure Category C is used for both tornadoes and hurricanes. An exception is permitted for hurricane shelters where wind loads for the main wind force-resisting system (MWFRS) only are permitted to be based upon Exposure Category B, where Exposure Category B exists for all wind directions and is likely to remain Exposure Category B after a hurricane.

Enclosure classifications for storm shelters are determined using ASCE/SEI 7-10, Section 26.2. For community shelters, the largest door or window on a wall that receives external pressure is to be considered as an opening. To reduce the design wind pressure for an opening, locating the opening outside of the corner wind pressure zones should be considered.

For tornado shelters classified as enclosed buildings, GC_{pi} is taken as ±0.18 when atmospheric pressure change (APC) venting of 1 ft^2 per 1,000 ft^3 of interior shelter volume is provided. APC venting consists of openings in the shelter roof having a pitch not greater than 10° from the horizontal or openings divided equally (within 10% of one another) on opposite walls. A combination of APC venting meeting the preceding requirements is permitted by ICC 500. For tornado shelters where APC venting is not provided, GC_{pi} = ±0.55 is used. Venting areas to relieve APC are not required for tornado shelters classified as partially enclosed buildings.

Storm shelters enclosed or partially enclosed within a host building or adjacent to other buildings not designed for the ICC 500 load requirements are to be designed considering the host building or adjacent building will be destroyed and the shelter will be fully exposed.

Where an element or component of the host building is connected to a storm shelter, the storm shelter is to be designed to resist the maximum force that could be transmitted to the shelter equal to the ultimate failure strength of the connection or element being connected, whichever is lower. This maximum force is concurrent with the other wind loads acting on the storm shelter as required by ICC 500.

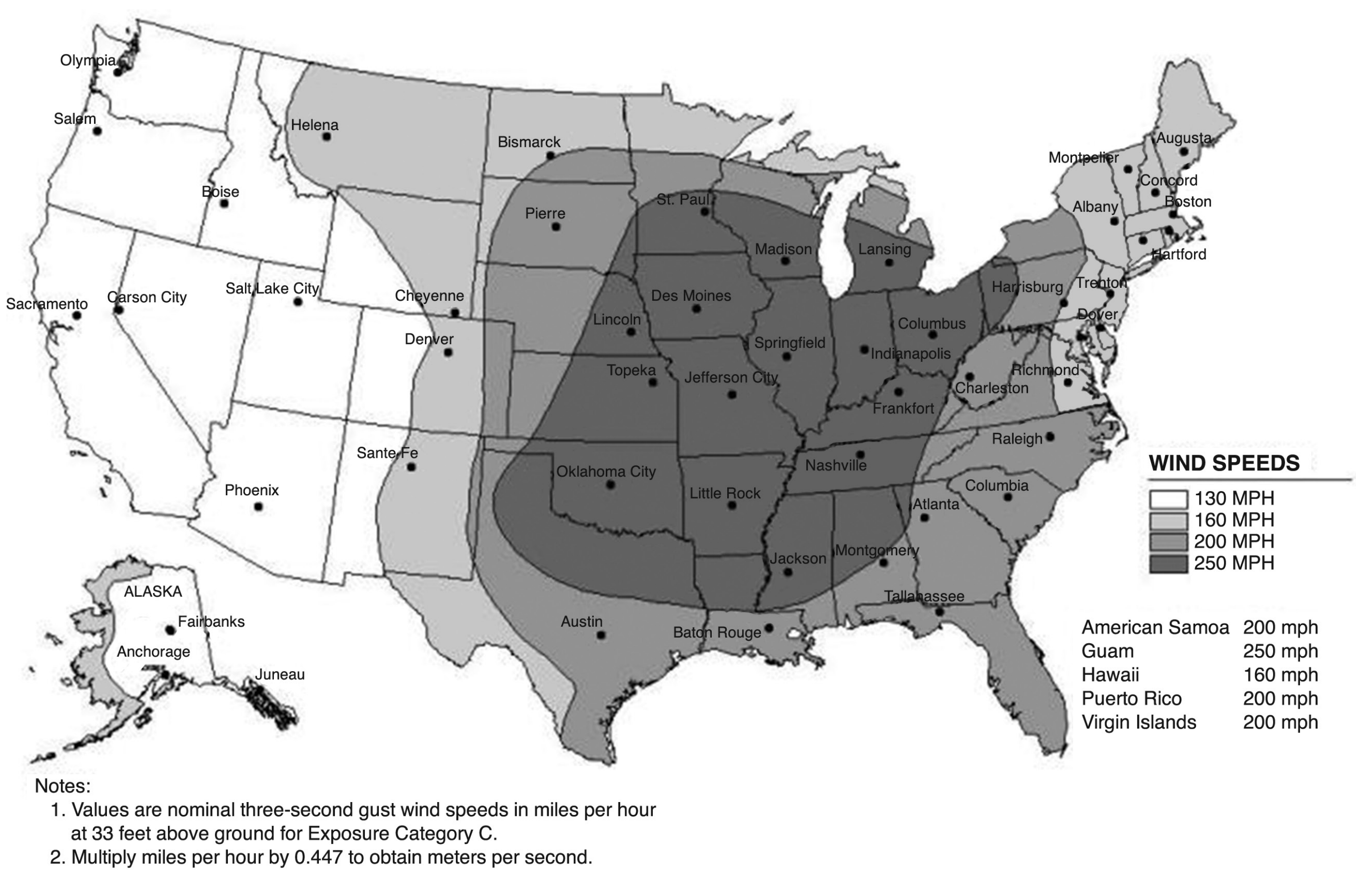

Fig. 2-1. Shelter design speeds for tornadoes.

Excerpted from the ICC 500: ICC/NSSA Standard for the Design and Construction of Storm Shelters; Copyright 2014. Washington, D.C.: International Code Council. Reproduced with permission. All rights reserved. www.ICCSAFE.org

2.1.7 Debris Hazards

In accordance with ICC 500, Section 305.1, all shelter envelope components are required to be designed for the impact of wind-borne debris as evaluated by the debris impact test. The debris impact test entails the impacting of a wall or roof assembly with a test missile following the procedures of *Standard Test Method for the Performance of Exterior Windows, Curtain Walls, Doors, and Impact Protective Systems Impacted by Missile(s) and Exposed to Cyclic Pressure Differentials*, ASTM E1886-05 (ASTM, 2005), hereafter referred to as ASTM E1886-05. The test missile criteria for wall and roof envelope components differ for a tornado shelter and a hurricane shelter.

For a tornado shelter, the test missile is nominally a 15-lb sawn lumber 2×4 that must travel at the designated speeds given by Table 2-1.

For hurricane shelters, the debris impact test missile for all components of the shelter envelope is a nominally 9-lb sawn lumber 2×4. The speed of the test missile when impacting the vertical shelter surface is a minimum of 0.50 multiplied by the shelter design wind speed. For test missiles impacting a horizontal surface, the missile impact speed is 0.10 multiplied by the design wind speed. Shelter envelope components meeting missile impact test requirements for tornado shelters are considered acceptable for hurricane shelters provided they meet structural design load requirements for hurricane shelters. For more details regarding missile testing requirements, see Chapter 3.

For both tornado and hurricane shelters, vertical surfaces

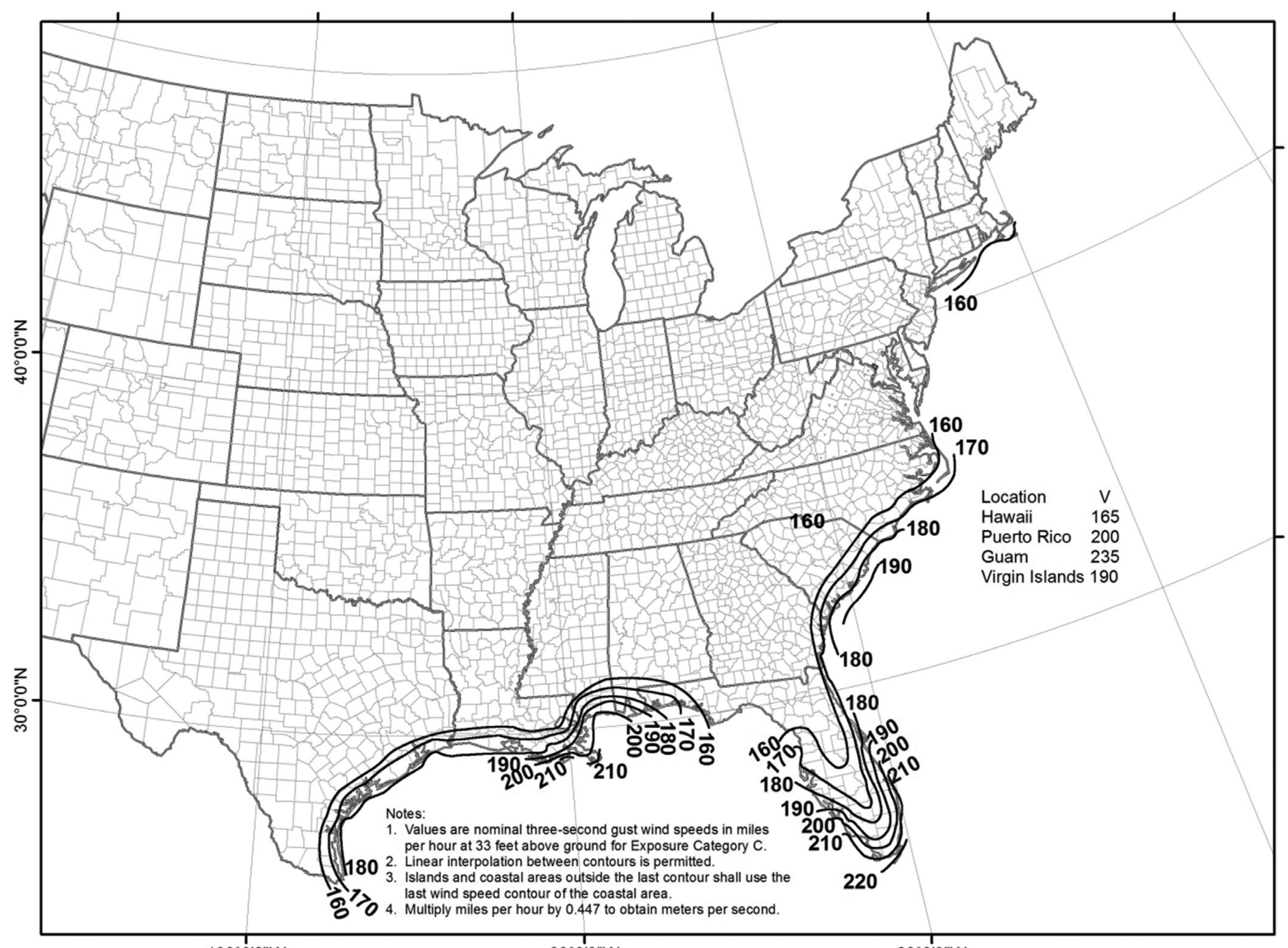

Fig. 2-2. Shelter design speeds for hurricanes.

Excerpted from the ICC 500: ICC/NSSA Standard for the Design and Construction of Storm Shelters; Copyright 2014. Washington, D.C.: International Code Council. Reproduced with permission. All rights reserved. www.ICCSAFE.org

Table 2-1. Speeds for Tornado Shelter Missile	
Design Wind Speed, mph	**Missile Speed and Safe Room Impact Surface**
130	80 mph vertical surfaces
	53 mph horizontal surfaces
160	84 mph vertical surfaces
	56 mph horizontal surfaces
200	90 mph vertical surfaces
	60 mph horizontal surfaces
250	100 mph vertical surfaces
	67 mph horizontal surfaces

of the shelter envelope are defined as surfaces inclined 30° or more from the horizontal. Surfaces inclined less than 30°, i.e., a roof pitch of 6.92 on 12, from the horizontal are to be treated as horizontal surfaces.

Soil-covered surfaces having less than 12 in. of soil cover protecting the shelter horizontal surfaces or with less than 36 in. of soil cover protecting the shelter vertical surfaces are to be tested for resistance to missile perforation as though the surfaces are exposed. To qualify for shielding from soil cover, the soil surfaces must slope away from the entrance walls or other near-grade enclosure surfaces of underground shelters at a slope of not more than 2 in./ft for a horizontal distance of not less than 3 ft from the exposed portions of the shelter or unexposed portions deemed to be protected by soil cover.

ICC 500, Section 804.10, summarizes the pass/fail criteria for a missile impact test as follows:

- Any perforation of the interior surface of the tested component of the shelter envelope by the missile shall constitute failure.
- Specimens and load-bearing fasteners shall not become disengaged or dislodged during the test so as to endanger occupants. The pass criterion is defined as specimens or fasteners failing to penetrate a witness screen comprised of #70 unbleached kraft paper located within 5 in. of the interior surface of the shelter component.
- Excessive spalling shall not occur. Excessive spalling is defined as that which perforates a #70 unbleached kraft paper witness screen located within 5 in. of the interior of the test specimen.
- Permanent deformation of an interior surface of the test specimen shall not exceed 3 in.

2.2 FEMA P-361 CRITERIA

Safe rooms designed and constructed in accordance with the guidance presented in FEMA P-361 (FEMA, 2015a) are intended to provide "near-absolute protection" from extreme-wind events. Near-absolute protection means that, based on the current knowledge of tornadoes and hurricanes, the occupants of a safe room built according to this guidance will have a very high probability of being protected from injury or death. The knowledge of tornadoes and hurricanes is based on substantial meteorological records as well as extensive investigations of damage from extreme winds. To date, a wind event exceeding the maximum design criteria stipulated by FEMA P-361 has not been observed. For this reason, the protection provided by these safe rooms is called "near-absolute" rather than absolute.

FEMA P-361 defines a community safe room as a shelter that is designed and constructed to protect a large number of people from a natural hazard event. The number of persons taking refuge in the safe room will typically be more than 16 and could be up to several hundred or more. These numbers exceed the maximum occupancy of small, in-residence safe rooms recommended in *Taking Shelter from the Storm: Building a Safe Room for Your House or Small Business*, FEMA P-320 (FEMA, 2015b).

2.2.1 Load Combinations

FEMA P-361 load combinations are the same as prescribed by ICC 500. Both LRFD and ASD are permitted. The respective load combinations as stipulated by ASCE/SEI 7-10, Sections 2.3 and 2.4, must be used; however, Exception 1 to ASCE/SEI 7, Section 2.3.2, does not apply.

2.2.2 Rain Loads

FEMA P-361 rain loads are the same as ICC 500. Rain loads are to be determined in accordance with ASCE/SEI 7-10. For hurricane safe room roofs, additional rainfall must be considered. The hurricane rainfall rate is determined by adding 6 in. of rainfall per hour to the rainfall rate as defined by ICC 500, Figure 303.2.

2.2.3 Roof Live Loads

FEMA P-361 roof live loads are the same as ICC 500. Safe room roofs are to be designed for the minimum roof live loads in accordance with ASCE/SEI 7-10, but not less than 100 psf for tornado shelters or 50 psf for hurricane safe rooms.

2.2.4 Hydrostatic Loads

FEMA P-361 loading is the same as ICC 500. Underground portions of safe rooms are to be designed for buoyancy forces and hydrostatic loads assuming that the groundwater level is at the surface of the ground at the entrance to the safe room, unless adequate drainage is available to justify designing for a lower groundwater level.

2.2.5 Flood Loads

FEMA P-361 recommends that safe rooms not be located in high-risk flood hazard areas (see Chapter 5, Section 5.1.2, for a discussion of siting recommendations). Safe rooms subject to flooding, including any foundation or building component supporting the safe room, should be designed in accordance with the provisions of ASCE/SEI 7-10 and *Flood Resistant Design and Construction*, ASCE 24-05 (ASCE, 2005), hereafter referred to as ASCE 24-05.

2.2.6 Wind Loads

FEMA P-361 stipulates the same wind loading as ICC 500 for community safe rooms. However, FEMA P-361, Section B3.1, provides for additional wind load considerations for residential safe rooms.

2.2.7 Debris Hazards

FEMA P-361 design guidance requires that all safe room envelope components be designed for the impact of windborne debris as evaluated by the debris impact test. FEMA P-361 references ICC 500, Section 305, for design requirements. Thus, the debris impact test entails impacting a wall or roof assembly with a test missile following the procedures of ASTM E1886-05.

The following pass/fail criteria, identical to ICC 500, are used to evaluate the missile impact test performance:

- Any perforation of the interior surface of the safe room envelope by the missile constitutes a failure.
- Disengagement or dislodgement of fasteners that endangers occupants.
- Excessive spalling from the interior surface.
- Permanent deformation of an interior surface greater than 3 in.

2.3 ASCE/SEI 7-16 CRITERIA

Although ICC 500 and the discussion thereof in this Design Guide are based on ASCE/SEI 7-10, considerable guidance regarding tornado loads has been incorporated into the Commentary of ASCE/SEI 7-16.

2.3.1 Tornado Loading

Tornado load requirements are not part of the mandatory ASCE/SEI 7-16 standard. ASCE/SEI 7-16, Section 26.14, states "Tornadoes have not been considered in the wind load provisions." But, ASCE/SEI 7-16 Commentary, Section C26.14, provides information and design guidance to enable design for reduced property damage or increased occupant protection in regions where buildings may be affected by a tornado. It is important to note that the Commentary is not mandatory.

ASCE/SEI 7-16 Commentary, Section C26.14, indicates that tornadoes have not been considered in the wind load provisions because of their very low probability of occurrence. Contained in the Commentary is a discussion of (1) tornado wind speeds and probabilities, (2) wind pressures induced by tornadoes versus other windstorms, (3) designing for occupant protection, (4) designing to minimize building damage, (5) designing to maintain continuity of building operations, and (6) designing trussed communications towers for windborne debris. The reader is referred to the Commentary for the complete discussion.

The National Weather Service rates tornado severity according to the six levels of observed damage in the EF scale. Table 2-2 presents the EF number and associated wind speed. For information pertaining to the EF scale and the assessment for an EF rating, refer to McDonald and Mehta (2006).

Tornadoes that have EF2 and EF3 ratings produce wind pressures that are comparable to ASCE/SEI 7-16 wind pressures for hurricane-prone regions. EF4 and EF5 rated tornadoes may produce wind pressures that are in excess of pressures for the design wind speeds for hurricane-prone regions. For EF3-, EF4- and EF5-rated tornado shelters and safe rooms, the ASCE/SEI 7-16 Commentary recommends using the provisions of either ICC 500 or FEMA P-361.

The ASCE/SEI 7-16 Commentary wind pressure calculations are consistent with the wind pressure calculations of ICC 500 and FEMA P-361; however, ASCE/SEI 7-16 Commentary, Section C26.14.4, provides two wind pressure calculation methods—the Extended Method and the Simplified Method.

The Extended Method uses wind pressure calculation parameters as outlined in ASCE/SEI 7-16, Chapter 27. Note that several of the parameters are modified for tornado wind loading. The Extended Method is based on an ultimate wind

Table 2-2. EF Scale	
EF Number	**Wind Speed, mph**
EF0	65–85
EF1	86–110
EF2	111–135
EF3	136–165
EF4	166–200
EF5	> 200

speed event, and therefore the wind speed should be taken as either the maximum wind speed for the target design EF scale or taken from the wind speed map of ICC 500 or FEMA P-361. Exposure C should be used in all cases unless the design exposure is D, in which case D should be used.

In ASCE/SEI 7-16, Chapter 26, the directionality factor, K_d, is taken as 0.85 for buildings. This accounts for the reduced probability of maximum winds coming from any given direction and the reduced probability of the maximum pressure coefficient occurring for that same wind direction. However, because of the rotational winds in a tornado and the potential that at least one building corner or window will experience the worst GC_p direction in conjunction with a maximum wind speed, ASCE/SEI 7-16 Commentary, Section C26.14.4, recommends that K_d be taken as 1.0.

Because the topographical effects on tornado wind speeds are not well documented, it is recommended that the topographic factor, K_{zt}, be taken as 1.0 when using the Extended Method. Furthermore, the gust-effect factor, G, is taken as 0.90 for rigid buildings.

Because of the limited research on wind pressures in tornado simulators, a 10% reduction on GC_p is recommended when computing component and cladding loads. ASCE/SEI 7-16, Chapter 30, presents the GC_p values for the various building zones. Anticipating a breach of the building envelope, which will significantly increase the internal wind pressure, GC_{pi} is taken as ±0.55, as recommended in ASCE/SEI 7-16 Commentary, Section C26.14.4.

The velocity pressure equation, $q = 0.00256K_dK_zK_{zt}K_eV^2$, as given in ASCE/SEI 7-16, Equation C26.14-1, is simplified for tornado pressure to $q = 0.00256K_zV^2$. It is recommended that q be determined at the mean roof height and q_h be used throughout the pressure calculations as the value of velocity pressure, q. The design wind pressure, p, can then be calculated using ASCE/SEI 7-16, Equation C26.14-2 or Equation C26.14-3.

For MWFRS pressure:

$$p = q_h\,[GC_p - (\pm 0.55)]$$

(ASCE/SEI 7-16, Eq. C26.14-2)

where

C_p = external pressure coefficient from Chapter 27

G = 0.90 or higher

For components and cladding (C&C) pressure:

$$p = q_h\,[0.9(GC_p) - (\pm 0.55)]$$

(ASCE/SEI 7-16, Eq. C26.14-3)

where

GC_p = external pressure coefficient for C&C found from Chapter 27

Because the tornado design wind speed is an ultimate wind speed, when evaluating load combinations, the load factor for strength design is taken as 1.0.

The Simplified Method combines the changed wind parameters of the Extended Method into one single multiplier and is intended to provide a simple method of accounting for various tornado related design considerations. The tornado factor (TF), summarized in Table 2-3 for Exposures B, C or D, can be applied to either ASCE/SEI 7-16 LRFD or to ASD design pressures or loads calculated for the building to determine the design tornado pressures or loads.

Tornado factors used to increase the design loads on elements of enclosed buildings are based on the effects of high internal pressures. High internal pressures have a much greater effect on elements that typically receive less wind, so the net effect of these increase factors is typically much higher than would result if the building were designed for the specific tornado loads or if the tornado factors were used with partially enclosed building designs.

Using the TF, the design wind pressure, p, is determined using ASCE/SEI 7-16, Equation C26.14-4:

$$p = q_i(GC_p - GC_{pi})TF$$

(ASCE/SEI 7-16, Eq. C26.14-4)

where

GC_p = product of external pressure coefficient and gust-effect factor to be used in determination of wind loads for buildings

Table 2-3. ASCE/SEI 7-16, Tornado Factors, *TF*			
Enclosure Classification	**Loading**	**Exposure B**	**Exposure C or D**
Partially enclosed	MWFRS	1.7	1.25
	C&C	1.45	1.05
Enclosed	MWFRS	2.5	1.75
	C&C	1.85	1.4

GC_{pi} = product of internal pressure coefficient and gust-effect factor to be used in determination of wind loads for buildings

TF = tornado factor

The numerical values for $q_i(GC_p - GC_{pi})$ are determined using the ASCE/SEI 7-16, Chapter 27, directional method for enclosed buildings. The wind speed used when determining q is the wind speed for tornado design taken from the target design EF scale or the wind speed map of ICC 500, FEMA P-320 or FEMA P-361.

Alternatively, the wind pressures can be calculated by scaling the wind speeds from ASCE/SEI 7-16, Figures 26.5-1A, B or C as follows:

$$p = q_i\left(GC_p - GC_{pi}\right)\left(\frac{V_{tornado}}{V_{design}}\right)^2 TF \tag{2-1}$$

where

V_{design} = ASCE/SEI 7-16 mapped wind speed for the location, mph

$V_{tornado}$ = selected wind speed to be used for this tornado design, mph

$V_{tornado}$ is taken as the wind speed from the target design EF scale or the wind speed map of ICC 500, FEMA P-320 or FEMA P-361.

2.3.2 Hurricane Loading

ASCE/SEI 7-16 wind pressure calculations use wind speeds that reflect hurricane velocities. In hurricane-prone regions, wind speeds derived from simulation techniques are only to be used in lieu of the basic wind speeds given in ASCE/SEI 7-16, Figures 26.5-1A, B or C, when approved simulation and extreme value statistical analysis procedures are used. The ASCE/SEI 7-16 maps are based on a new and more complete analysis of hurricane characteristics. The maps removed inconsistencies in the use of importance factors that actually should vary with location between hurricane-prone and nonhurricane-prone regions and acknowledge that the demarcation between hurricane and nonhurricane winds change with the recurrence interval.

Hurricane-prone regions as defined by ASCE/SEI 7-16 are areas vulnerable to hurricanes. In the United States and its territories these regions are

1. The U.S. Atlantic Ocean and Gulf of Mexico coasts where the basic wind speed for Risk Category II buildings is greater than 115 mph, and
2. Hawaii, Puerto Rico, Guam, the Virgin Islands, and American Samoa.

For applications of serviceability, design using maximum likely events, or other applications, it may be desired to use wind speeds associated with mean recurrence intervals rather than those given in ASCE/SEI 7-16, Figures 26.5-1A to 26.5-1C. ASCE/SEI 7-16 Commentary, Appendix C, presents maps of peak gust wind speeds at 33 ft above ground in Exposure C conditions for return periods of 10, 25, 50 and 100 years.

Because of the nature of hurricane winds and exposure to debris hazards, glazing of buildings sited in wind-borne debris regions has a vulnerability to breakage from wind-borne missiles. Thus, ASCE/SEI 7-16 requires impact protection for glazed openings in Risk Category II, III or IV. Wind-borne debris regions are defined by ASCE/SEI 7-16 as:

1. Within 1 mile of the coastal mean high water line where the basic wind speed is equal to or greater than 130 mph, or
2. In areas where the basic wind speed is equal to or greater than 140 mph.

For Risk Category II and III buildings and other structures, except health care facilities, the wind-borne debris region is to be based on ASCE/SEI 7-16, Figures 26.5-1B and 26.5-2B. For Risk Category III health care facilities, the wind-borne debris region is to be based on Figures 26.5-1C and 26.5-2C. For Risk Category IV buildings and other structures, the wind-borne debris region is to be based on Figures 26.5-1D and 26.5-2D. Risk Categories are to be determined in accordance with ASCE/SEI 7-16, Section 1.5; however, care should be taken when selecting the category because of the life safety implications, even though the occupancy level might be low.

Glazing in buildings requiring protection is to be protected with an impact-protective system or is required to be impact-resistant glazing. Impact-protective systems and impact-resistant glazing are subject to missile impact tests and cyclic pressure differential tests in accordance with *Standard Specification for Performance of Exterior Windows, Curtain Walls, Doors, and Impact Protective Systems Impacted by Windborne Debris in Hurricanes*, ASTM E1996-12 (ASTM, 2012), hereafter referred to as ASTM E1996-12. Testing to demonstrate compliance with ASTM E1996-12 is conducted in accordance with ASTM E1886-05. Impact-resistant glazing and impact-protective systems comply with the pass/fail criteria of ASTM E1996-12, Section 7, based on the missile required by ASTM E1996-12, Table 2. Note that when using ASTM E1996-12, the wind zones specified for use in determining the applicable missile size have to be adjusted for use with the wind speed maps of ASCE/SEI 7-16 and the corresponding wind-borne debris regions.

Chapter 3
Building Envelope Considerations

In accordance with ICC 500, Section 305.1 (ICC, 2014), all shelter envelope components are required to be designed for the impact of wind-borne debris as evaluated by the debris impact test.

In lieu of providing rated glazing systems, the designer may consider using standard glazing along with a missile impact-rated door or screen to act as a storm shutter. In any case, the exterior cladding, walls and roofs, must resist the design wind pressures and stop a wind-borne missile. Today, the ability to resist the impact of a wind-borne missile can only be determined by test. ICC 500 and FEMA P-361 (FEMA, 2015a) rely on the same test protocol. However, the ASCE/SEI 7-16 test protocol guidelines are different.

The following discussion presents an overview of the various test protocols. Prior to engaging in a test program, the respective test protocols should be carefully reviewed for complete testing details.

3.1 ICC 500 AND FEMA TEST PROTOCOL

ICC 500, Chapter 8, provides the test requirements for both impact and pressure testing. Test specimens are to replicate the in-place construction using the same materials, details, methods of construction, and methods of attachment. The testing of components such as wall, roof, door or window assemblies is addressed.

Where both pressure and impact tests are to be performed, a single test specimen subjected separately to each load effect may be used. Alternately, two specimens where each specimen is subject to either pressure or impact may be used. The test specimens are to be conditioned at an ambient temperature in the range of 59°F to 95°F for a minimum of two hours prior to a test.

Doors, windows, and impact-protective systems are to be tested for both the maximum and minimum size to be used. Operative doors and windows are to be tested for the conditions of swing and latching as specified for the product's use. When it is not possible to install a door or window frame to replicate in-place conditions, then the unit or assembly is to be mounted in a test frame to replicate in-place conditions.

3.1.1 Missile Impact Testing

The minimum test assembly is 4 ft wide by 4 ft high unless the dimensions of the actual assembly are less than these dimensions.

The test apparatus and procedure prescribed by ICC 500 and FEMA P-361 for a missile test are defined by ASTM E1886-05 (ASTM, 2005).

The test missile is a 2×4 and can be any common softwood lumber species as defined by the *American Softwood Lumber Standard*, PS 20-10 (DOC, 2010). The lumber must be grade stamped No. 2 or better and be free of splits, checks, wane, or other significant defects. The bow or warp of the missile must be such that stretching a string or wire on the side of the board from end to end is within 0.5 in. of the 2×4's surface over its entire length.

For tornado and hurricane impact tests, the test missile is to be conditioned at ambient temperature in the range of 59°F to 95°F, for a minimum of two hours. For a tornado impact test, the wood density, including moisture content, must be such that the 13.5 ft ± 6 in. missile weighs 15 ± 0.25 lb. For a hurricane impact test, the wood density, including moisture content, must be such that the 8 ft ± 4 in. missile weighs 9 ± 0.25 lb. These respective weights are to be verified within two hours of their use.

For both tornado and hurricane safe rooms, vertical surfaces of the shelter envelope are defined as surfaces inclined 30° or more from the horizontal. Surfaces inclined less than 30° from the horizontal are to be treated as horizontal surfaces.

The missile speed, as summarized in Chapter 2, has a tolerance of 4 mph above and 0 mph below the prescribed speed. The angle of missile impact is to be within 5° of the normal to the plane of the test specimen—for example, vertical or horizontal planes.

Required impact locations for wall and roof construction vary with the test assembly configuration as illustrated in Figures 3-1 to 3-3.

For roof and wall construction, no more than three impacts are to be made on any one test specimen. Where more than three impacts are required, multiple identical test specimens are to be used.

For windows and other glazed openings, Figure 3-4 illustrates the impact locations. No more than two impacts are to be made on any one test specimen. Where more than two impacts are required, multiple identical test specimens are to be used.

Doors and other entry/egress systems are to be impacted as illustrated in Figure 3-5. If the door contains a glazed opening that is less than or equal to 12 in. × 12 in., an additional sample is to be impacted in the center of the glazing. If the glazed opening dimension is greater than 12 in., the glazed opening is to be treated as a window and tested accordingly.

ICC 500, Chapter 8, contains additional criteria for missile testing of impact-protective systems and alcove or baffled entry systems that require the use of a #70 unbleached

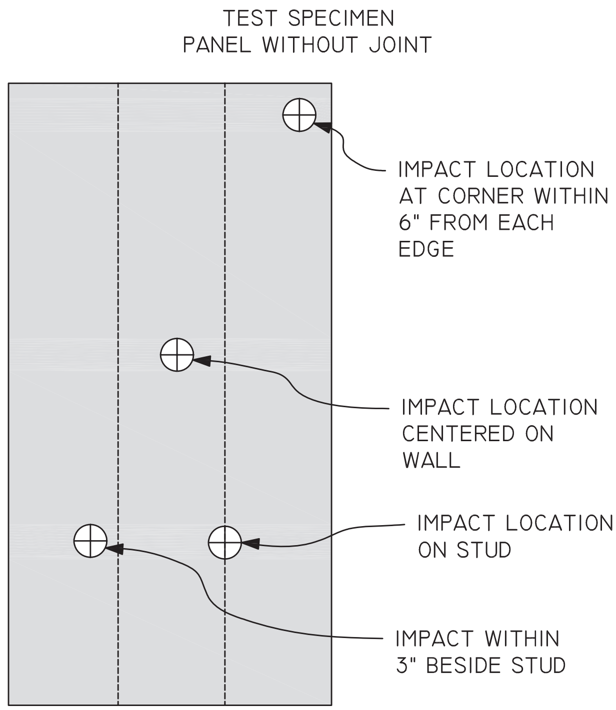

Fig. 3-1. Panel, framed or solid wall or roof construction.

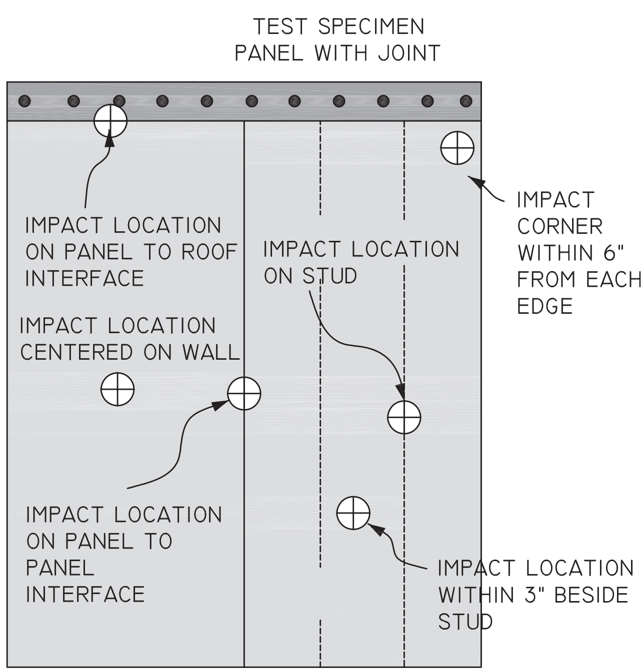

Fig. 3-2. Panel, framed or solid wall or roof construction with panel joint.

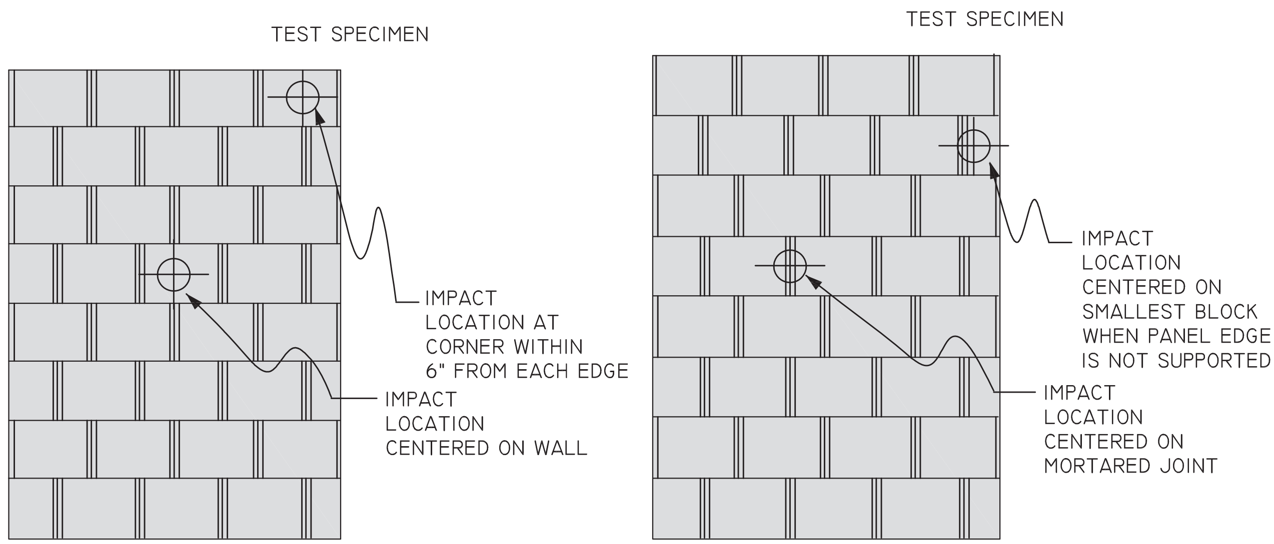

Fig. 3-3. Masonry unit wall or roof construction.

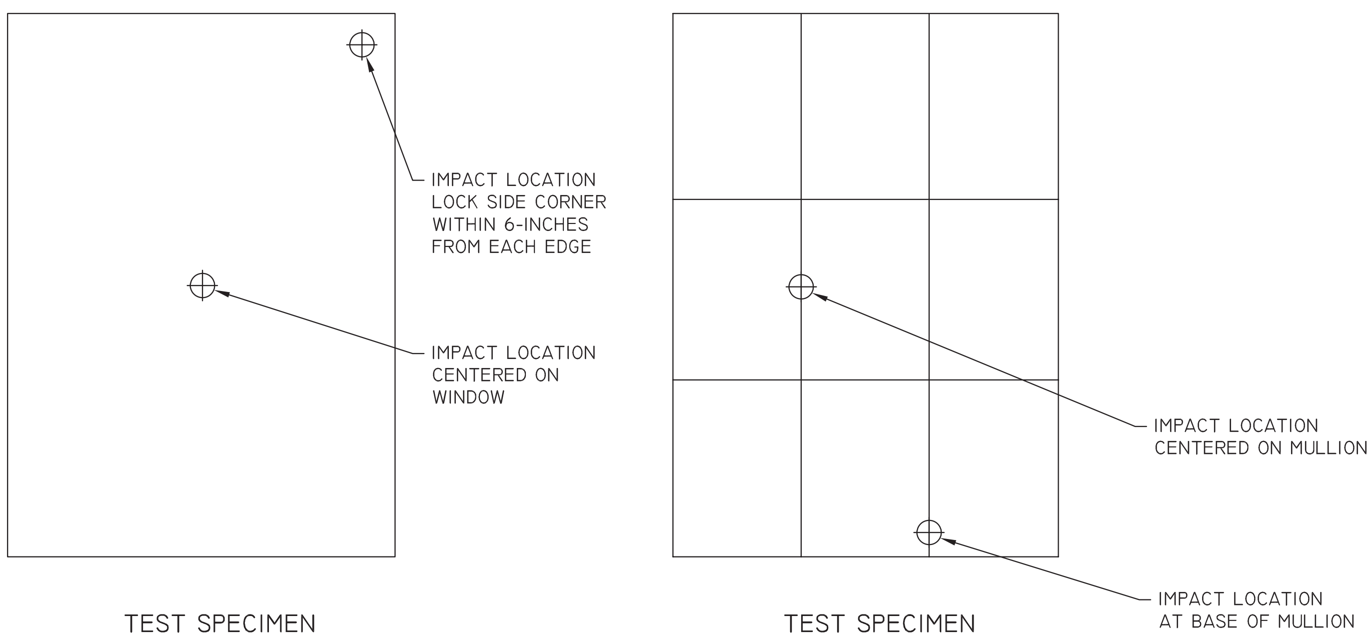

Fig. 3-4. Windows and other glazed openings.

kraft paper witness screen with its surface secured in place on a rigid frame installed within 5 in. of the interior surface of the shelter component.

3.1.2 Pressure Testing

For wall and roof sections subject to pressure testing, the minimum test assembly is 4 ft wide × the full length of the span of the wall section from support to support.

The test loading sequence is defined by either *Standard Test Method for Structural Performance of Exterior Windows, Doors, Skylights and Curtain Walls for Uniform Static Air Pressure Difference*, ASTM E330-02 (ASTM, 2002), hereafter referred to as ASTM E330-02, for a static pressure test or ASTM 1886-05 for a cyclic pressure test.

For wall and roof assemblies, when using ASTM E330-02 the assembly is subjected to a pressure equal to or higher than 1.2 times the design wind pressure.

Door assembly testing varies depending upon the event, tornado or hurricane, and the presence of glazing to include sidelights or transoms. Doors for tornado shelters with or without glazing are to be static pressure tested to a pressure of 1.2 times the design wind pressure. The pressure test is permitted to be performed separately from the missile impact tests. For hurricane shelter applications, doors without glazing are tested to 1.2 times the design wind pressure and then subjected to the missile impact test followed by cyclic pressure testing in accordance with ASTM E1886-05. If the door is subjected to 1.5 times the design wind pressure before the impact test then the cyclic test is not required. Door assemblies with glazing are tested to 1.2 times the design wind pressure and then subjected to the missile impact test followed by cyclic pressure testing in accordance with ASTM E1886-05.

For both tornado and hurricane shelters, window assembly testing consists of the application of a static pressure of 1.2 times the design wind pressure per ASTM E330-02. Additionally, for hurricane shelters, a window assembly is subsequently subjected to the missile impact test and the cyclic pressure test.

ICC 500, Chapter 8, contains additional criteria for impact-protective systems and alcove or baffled entry systems.

3.2 ASCE/SEI 7-16 TEST PROTOCOL

For hurricane shelters, ASCE/SEI 7-16 requires that impact-protective systems and impact-resistant glazing be subject to missile and cyclic pressure differential tests in accordance with ASTM E1996-12 (ASTM, 2012). Testing to demonstrate compliance with ASTM E1996-12 is conducted in accordance with ASTM E1886-05. Impact-resistant glazing and impact-protective systems comply with the pass/fail criteria of ASTM E1996-12, Section 7, based on the missile required by ASTM E1996-12, Table 2.

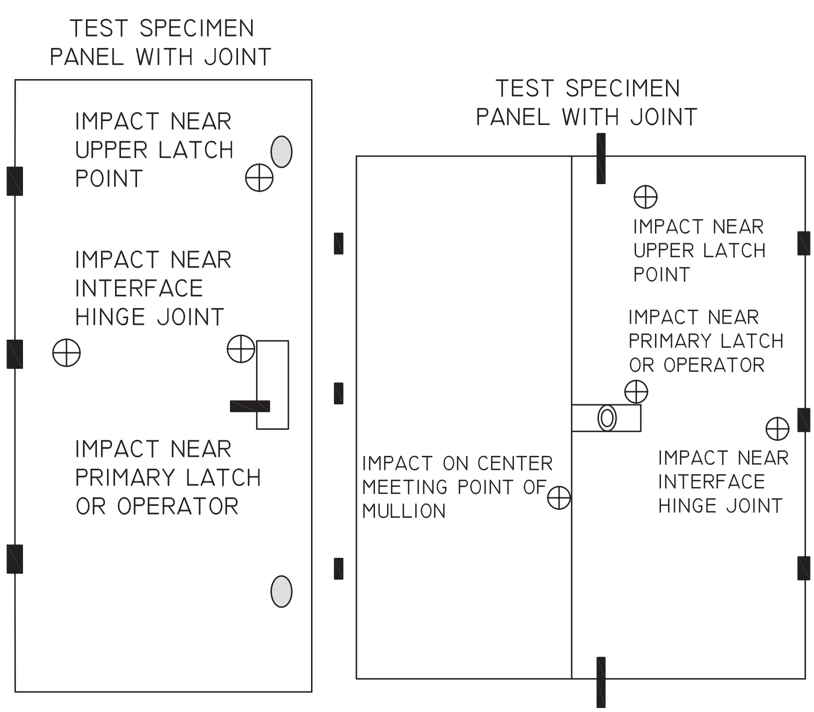

Fig. 3-5. Door and other egress systems.

To minimize breaching of exterior glazing by EF0-, EF1- or EF2-rated tornadoes, ASCE/SEI 7-16 Commentary, Section C26.14.4, suggests that glazing systems be tested in accordance with ASTM E1886-05 using ASTM E1996-12 test missile D, or preferably E. To avoid breaching of exterior glazing by EF3-, EF4- or EF5-rated tornadoes, ASCE/SEI 7-16 Commentary, Section C26.14.4, suggests that glazing systems be tested in accordance with the American Architectural Manufacturers Association (AAMA) *Voluntary Specifications for Tornado Hazard Mitigation Fenestration Products*, AAMA 512-11 (AAMA, 2011), using test missiles stipulated by ICC 500 or FEMA P-361.

3.3 ASTM E1886-05

ASTM E1886-05 is a test method that covers the performance of exterior windows, curtain walls, doors, and impact-protective systems impacted by missile(s) and subsequently subjected to cyclic static pressure differentials. A missile propulsion device, an air pressure system, and a test chamber are used to model conditions that may be representative of wind-borne debris and pressures in a windstorm environment. This test method is applicable to the design of entire fenestration or impact-protection system assemblies and their installation. The performance determined by this test method relates to the ability of elements of the building envelope to remain unbreached during a wind event. Exterior garage doors and rolling doors are governed by the Door & Access Systems Manufacturers Association, International (DASMA), *Standard Method for Testing Sectional Garage Doors and Rolling Doors: Determination of Structural Performance Under Missile Impact and Cyclic Wind Pressure*, ANSI/DASMA 115-12 (ANSI/DASMA, 2012), and are beyond the scope of this test method.

3.4 ASTM E1996-12

ASTM E1996-12 provides the information required to conduct the ASTM E1886 protocol. This standard covers exterior windows, glazed curtain walls, doors, and impact-protective systems used in buildings located in geographic regions that are prone to hurricanes. Exterior garage doors and rolling doors are governed by ANSI/DASMA 115-12 and are beyond the scope of this specification.

3.5 INDUSTRY MISSILE IMPACT ASSEMBLY TESTS

In 2016, missile impact tests that utilized standard steel construction methods and materials were performed at the National Wind Institute (NWI) Debris Impact Facility at Texas Tech University (NWI, 2017). The test protocol was based on ICC 500 and FEMA P-361 test criteria.

The goal of the test program was to assess the performance of alternative, more cost-effective systems than has been previously tested utilizing standard steel construction methods. The test series comprised the following construction materials:

- 18- and 20-gage 1.5-in.-wide rib steel decking (commonly referred to as Type B)
- 12K5 open web steel joists
- HSS6×3×⅛
- Nailbase insulation consisting of 3-in. polyisocyanurate + 1-in. spacers + ⅝-in. CDX plywood

Five test series were performed with Series 1, 2 and 3 being preliminary, or exploratory, tests and Series 4 and 5 being the final tests:

- Series 1—20-gage decking supported on open web steel joist
- Series 2—18-gage decking supported on open web steel joist
- Series 3—18-gage decking supported on HSS
- Series 4—18-gage decking supported on open web steel joist
- Series 5—18-gage decking supported on HSS

The test specimen configuration as illustrated by Figure 3-6, and the deck sidelap detail, shown in Figure 3-7, were unchanged for all test series.

Series 1, 2 and 3 were preliminary tests to assess the impact of varying parameters. Series 4 and 5 were the certification tests:

- Series 1 failed the 67 mph impact with the single layer of plywood. An additional layer of plywood was determined to be necessary to achieve satisfactory performance.
- Series 2 and 3 passed both the 67 mph and 100 mph impacts with a single layer of plywood.
- Series 4 passed both the 67 mph and 100 mph impacts with a single layer of plywood.
- Series 5 passed the 67 mph impact but failed the 100 mph impact. The 100 mph impact resulted in an ejection of the insulation board screw into the safe compartment, which is a violation of the ICC 500 test standard. The screw is deemed to be a projectile that may injure an occupant.

Based on the tested performance, Series 4 and 5 assemblies were deemed to be adequate for horizontal applications (roof assembly with slope 30° or less) for design velocities up to 250 mph. The Series 4 assembly was deemed acceptable for a vertical application (wall or roof with slope over 30°) at a design velocity of 250 mph. However, the Series 5 assembly was also deemed acceptable for a vertical application (wall or roof with slope over 30°) if ⅝-in.-thick gypsum board, which would typically be desired as an interior finish and may be required to achieve membrane fire protection,

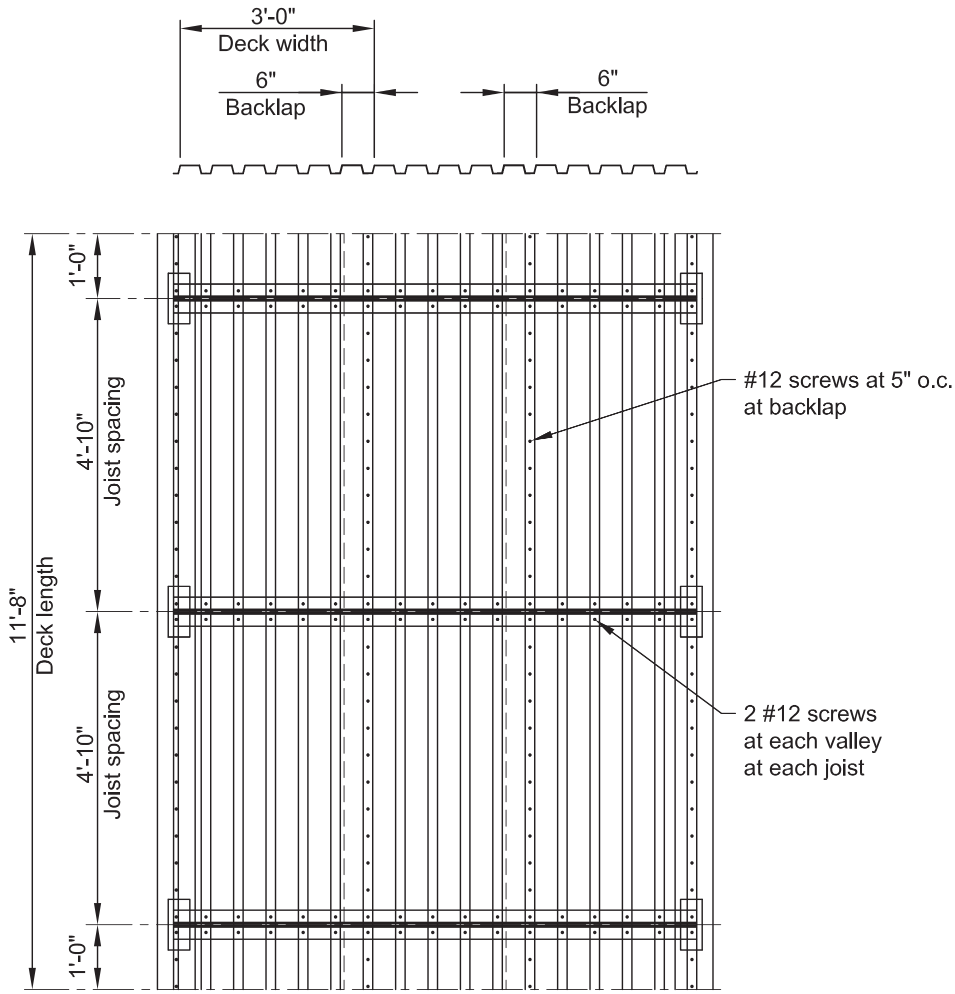

Fig. 3-6. Test specimen configuration.

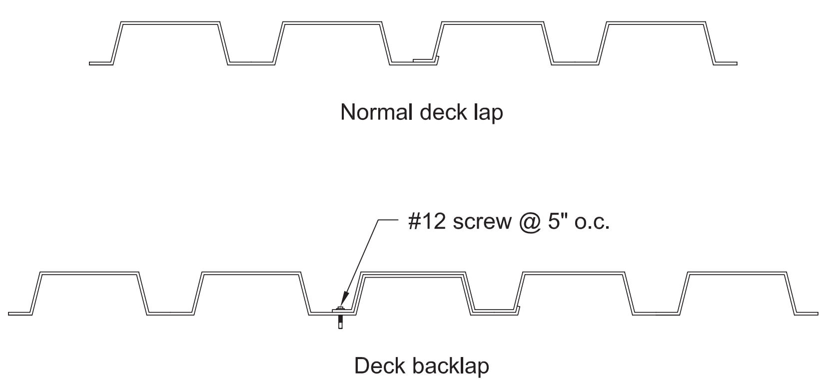

Fig. 3-7. Backlap sidelap detail.

was employed. The gypsum board is considered to be of adequate strength to capture the screw that was dislodged during the test.

3.6 PREVIOUS BUILDING ENVELOPE SYSTEM TESTS

Clemson University (Roper, 2015) performed missile impact tests on industry standard steel decks. Both 16 and 18 gage 1.5WR and 3DR deck profiles attached to steel frame supports were tested. Wide-flange steel members were chosen for the frame supports, and the frame was designed to remain elastic throughout testing so that only a negligible portion of the impact energy was absorbed by the steel frame. The frame can be considered to provide the worst-case scenario for missile impact testing because the energy from the missile would be absorbed by the deck and not through the supporting frame members. The wide-flange frame was then supported by a rigid frame connected to a concrete foundation.

The test program considered the performance of bare deck only. That is, the test specimen did not incorporate overlay material such as insulation or plywood. All tests were performed in accordance with FEMA P-361.

Recommendations for the 3DR deck profile:

- Span length: 3DR-16 deck should span not more than 8 ft between supporting members. 3DR-18 deck should not span more than 6 ft between supporting members.
- Bearing length: A minimum 3-in. bearing length is recommended at ends of deck panels.
- Lap connection: The lap connection should be reinforced using an 18-gage cover plate connected with at least No. 12 screws spaced 12 in. on center with top and bottom rows staggered. Unreinforced lap connections consisting only of screws should not be used.
- Deck thickness: The thicker 16-gage deck is recommended; however, with proper detailing, the 18-gage deck is also acceptable in wind zones of 200 mph or less.
- Connection at bearings: At least two screws are recommended in each rib where decks are attached to supporting members. At the end of the deck, the screws should be spaced to maximize the edge distance. This can be accomplished by orienting the screws parallel to the framing members. The minimum edge distance for the screws should be 1.5 in.
- Screws: Self-tapping No. 12 screws or larger are recommended.

Where FEMA P-361 design wind speeds are 250 mph or less, the 3DR-16 deck was deemed to be adequate. Where the FEMA P-361 design wind speeds are 200 mph or less, the 3DR-18 deck is acceptable.

Recommendations for the 1.5WR deck profile:

- Span length: 1.5WR-16 and 1.5WR-18 deck should span not more than 4 ft between supporting members.
- Bearing length: A minimum 3-in. bearing length is recommended at ends of deck panels.
- Lap connection (with reinforcing cover plate): The lap connection should be reinforced using an 18-gage cover plate connected with at least #12 screws spaced 12 in. on center with top and bottom rows staggered. Lap connections consisting only of screws should not be used.
- Lap connection (with overlap of one flute at lap joint): Lap connections using overlap of one flute should be attached using at least two #12 screws set 3 in. apart and spaced 10 in. on center.
- Deck thickness: The thicker 16-gage deck is recommended; however, with proper detailing, the 18-gage deck is also acceptable in wind zones of 160 mph or less.
- Connection at bearings: At least two screws are recommended in each rib where decks are attached to supporting members. At the end of the deck, the screws should be spaced to maximize the edge distance. This can be accomplished by orienting the screws parallel to the framing members. The minimum edge distance for the screws should be 1.5 in.
- Screws: Self-tapping #12 screws or larger are recommended.

Where FEMA P-361 design wind speeds are 250 mph or less, the 1.5DR-16 deck was deemed to be adequate. The 1.5WR-18 deck is only deemed to be acceptable in regions where the FEMA P-361 design wind speeds are 160 mph or less.

In 2005, the Wind Science and Engineering Research Center at Texas Tech University (Kiesling and Tanner, 2005) performed two series of tests for Nuconsteel:

- Series 1 was a 4-ft by 8-ft sandwich panel consisting of two sheets of ¾-in.-thick 48/24 plywood with a 14-gage cold-rolled steel sheet between two cold-formed steel studs and the plywood. The assembly was attached to an 18-gage cold-formed steel stud assembly using #10 screws located at 12 in. on center. The stud assembly was composed of double C-channels, nominally 2 in. by 4 in., with the studs located at 16 in. on center. The studs were horizontally braced with double studs on 32-in. vertical centers and located between each stud space.
- Series 2 replicated Series 1 with the exception that 20-gage steel was substituted for 18 gage.

For both test series, satisfactory performance was achieved when subjected to a 100 mph missile impact.

FEMA P-361, Appendix E, lists a variety of other wall sections that have passed missile impact tests. Information is provided for each wall section and contains a description of the wall construction (e.g., stud wall with plywood and/or metal sheathing, stud wall with concrete infill, reinforced concrete masonry unit (CMU) wall, insulating concrete form wall, etc.), cross-section illustration, test missile speed, and description of damage. It is important to note that the inclusion of a wall section in Appendix E does not signify that the section will necessarily pass the current missile impact standard tests of ICC 500 and FEMA P-361. However, these wall sections have passed tests held to previous standards that, in some cases, may have been more stringent than current standards. Thus, the information in Appendix E may be used merely as a method of determining which wall sections might be considered for use in a shelter or safe room application.

Texas Tech University has a reputation for wind engineering research conducted through their Institute for Disaster Research and the Wind Science and Engineering Research Center. One aspect of the wind engineering research is the effect of wind-borne debris on structures. The Research Center has performed hundreds of tests in their Debris Impact Test Facility. Test results on products that are not proprietary are reported in *A Summary Report on Debris Impact Testing at Texas Tech University* (WSERC, 2003). The data is presented in a tabular format to facilitate comparison of missile type, weight, speed, momentum, energy, and description of damage for the various types of targets. While the report is not complete in terms of considering all types of debris or all types of impacted surfaces, it does provide a significant contribution to a database for wind-borne debris resistance by the most commonly used shelter or safe room components. Thus, the information may be used as a method of determining which wall sections might be considered for use in a shelter or safe room application.

Chapter 4
Framing Systems Design Considerations

Wind forces must be transferred with appropriate load paths between all building elements. The most important load paths indicated by damage observations are the roof-to-wall connections, the wall-to-foundation connections, and the connections between the exterior walls at the corners. In addition to the exterior walls, shear walls in the interior of the building may reduce the tendency for the building to rack and/or overturn. Adequate shear wall design requires proper anchorage at the ends of the shear walls and sometimes at the ends of shear wall segments.

It is a performance objective to avoid collapse of interior walls and ceilings in the event that exterior glazing is breached. Resistance to failure may require additional or strengthened connections between the top of interior walls and the roof or ceiling assemblies and connections between the bottom of interior walls and the floor or foundation. ASCE/SEI 7-16 Commentary recommends that 80% of the exterior wall design loads be used for design of shelter interior walls. As noted earlier, community shelters are designed assuming the largest door or window on a wall that receives external pressure is an opening.

The main objective of this chapter is to discuss the most viable structural systems for gravity, wind and projectiles. Roof, wall, and lateral load-resisting systems, along with typical connection details, are presented.

Several roof and wall systems were investigated relative to their applicability for shelters. Gravity loads are of concern; however, roof systems that can best resist large uplift forces and missile projectiles and wall systems that can resist the large lateral loads and missile projectiles are of great importance.

4.1 ROOF SYSTEMS

4.1.1 Steel Deck

Poured concrete on steel deck has been deemed to provide reliable strength to resist missile projectiles, gravity loads, wind and tornado shear, and uplift forces. However, recent tests as described in Section 3.6 utilizing steel deck with a nail-base insulation consisting of 3-in. polyisocyanurate + 1-in spacers + ⅝-in. CDX plywood may provide a more economical roof system solution. Both composite and noncomposite steel decks provide adequate strength to resist these loads. For design information on steel deck, refer to the Steel Deck Institute *Floor Deck Design Manual* (SDI, 2014).

The reader is referred to manufacturers' recommendations for the design and installation requirements for steel deck with nail-base insulation and plywood. In addition, requirements can be found in the following references:

- *Standard Specification for Faced Rigid Cellular Polyisocyanurate Thermal Insulation Board,* ASTM C1289-16a (ASTM, 2016a), Type V
- *Standard for Fire Test of Roof Deck Constructions,* UL Standard 1256 (UL, 2002), Classification—Construction No. 120, 123 and 458
- *Standard for Standard Test Methods for Fire Tests of Roof Coverings*, UL Standard 790 (UL, 2004), for use with Class A, B or C shingles, metal or tile roof coverings
- *Standard Test Methods for Fire Tests of Roof Coverings,* ASTM E108-17 (ASTM, 2017), for use with Class A, B or C shingles, metal or tile roof coverings
- *Standard for Fire Tests of Building Construction and Materials,* UL Standard 263 (UL, 2011), Fire Resistance Classification
- *Standard Test Methods for Fire Tests of Building Construction and Materials,* ASTM E119-16 (ASTM, 2016b)
- *Approval Standard for Class 1 Insulated Steel Deck Roofs,* Approval Standard Class No. 4450 (FM, 1989)
- *Approval Standard for Single-Ply, Polymer-Modified Bitumen Sheet, Built-Up Roof (BUR) and Liquid Applied Roof Assemblies for use in Class 1 and Noncombustible Roof Deck Construction*, Approval Standard Class No. 4470 (FM, 2012)
- IBC (ICC, 2015), Chapter 26

Design recommendations for composite deck:

1. Use composite decks for the support of the gravity loads.
2. Use normal weight concrete with 7-in. to 8-in. slab thicknesses to maximize dead load.
3. Provide negative reinforcement near the top of the slab. No known research exists to determine the concrete bond between noncomposite or composite decks for uplift forces that exceed the dead load of the concrete slab. In the absence of such research, it is suggested that reinforcing steel be placed in the concrete along with steel headed stud anchors to transfer uplift forces from the slab to its supporting element.
4. Design the steel headed stud anchors for uplift and the concrete for punch out using the *PCI Design*

Handbook (PCI, 2010) and *Building Code Requirements for Structural Concrete and Commentary,* ACI 318 (ACI, 2014).

4.1.2 Purlin Systems

Several purlin systems can be economically used to support the steel deck and to transfer forces to the primary structural members. For the systems indicated below, the choices are presented in no particular order:

1. Cold-formed steel purlins and trusses
2. Wide-flange beams
3. Open-web steel joists
4. Hollow structural sections (HSS)

Cold-Formed Steel Trusses and Purlins

Cold-formed steel trusses are ideally suited for architectural schemes that require significant roof slopes; however, slopes are limited based on the ability to pour the concrete on a slope (approximate slope of 1.5:1) (ACI, 2005). Also roof systems that are not relatively flat cause an increase in the lateral loads due to wind. Due to the light weight and the nature of cold-formed steel trusses and purlins, a significant amount of bridging or other lateral bracing will likely be required for stability of the bottom chord under wind uplift loads. The reader is referred to the *North American Specification for the Design of Cold-Formed Steel Structural Members*, AISI S100-16 (AISI, 2016), for purlin design and the *North American Standard for Cold-Formed Steel Structural Framing,* AISI S240-15 (AISI, 2015), for truss design information.

Wide-Flange Beams

Wide-flange beams provide a good solution for purlins. If steel headed stud anchors are provided for uplift loads as described in the previous section, they can serve a dual purpose as steel headed stud anchors for the design of composite wide-flange beams. For composite beams, spans up to 60 ft can be economical. In some cases, the flanges will require lateral bracing for wind uplift. For design information for structural steel shapes, refer to the AISC *Specification for Structural Steel Buildings* (AISC, 2016). Wide-flange beams used for secondary members can be designed to brace the primary beams will little to no extra cost.

Open-Web Steel Joists

Open web steel joists are well suited for the purlin systems (SJI, 2016). They are often the logical choice for spans greater than 30 ft. An increased number of bridging lines may be required to prevent the bottom chord from buckling under uplift loading as compared to typical construction. The EOR must carefully coordinate the loading requirements and the connection detail between the joist and the supporting steel with the joist manufacturer. The reader is referred to the *Design of Steel Joist Roofs to Resist Uplift Loads*, SJI Technical Digest 6 (SJI, 2012), for information on uplift connections with joists. Also, as mentioned relative to wide-flange beams, consider using composite joists (SJI, 2007) if steel headed stud anchors are required for the slab-to-deck connection.

HSS

Purlin systems using hollow structural sections (HSS) are also possible. The advantage of HSS is that they have superior resistance to lateral buckling and will probably not require flange bracing (AISC, 2016). Economical spans are up to 30 ft.

4.1.3 Primary Members

For the systems indicated the choices are presented in no particular order:

1. Wide-flange beams
2. Built-up sections
3. Joist girders
4. Fabricated steel trusses

Wide-Flange Beams

Wide-flange beams are often used for the primary members because connections to secondary members are simple and, thus, economical. Also for uplift loads, uplift bracing is easily provided by inserting “kickers” from the compression flange to the secondary members. Vertical deflection criteria can also be easily met.

Built-Up Sections

Built-up members have the same advantages as W-shapes. When incorporated into moment frames for lateral load resistance, metal building rigid frames are often economical.

Joist Girders

Joist girders provide economical primary members particularly when spans exceed 30 ft. Connection details to joist girders are simple when the purlin system consists of open-web steel joists. For other purlin systems connection details become more expensive. As with W-shapes and built-up members, uplift bracing is easily provided by inserting kickers from the compression flange to the secondary members. As indicated in SJI Technical Digest 6, uplift strengths for K-series joists are limited to 10.5 kips (LRFD) or 7.0 kips (ASD) using standard bolted-seat details. For greater uplift forces, special detailing is required or LH joists can be specified. Uplift strengths up to 106 kips (LRFD) or 70.7 kips (ASD) can be accommodated with bolted-seat connections with LH joists, DLH joists and joist girders.

Fabricated Steel Trusses

Fabricated steel trusses have the same advantages and similar design considerations as joist girders.

4.2 COLUMNS AND INTERIOR LOAD-BEARING WALL SYSTEMS

The logical choices for columns include wide-flange shapes, either rolled shapes or built-up members, and HSS. HSS columns are often the economical choice in single-story buildings for gravity loads when the eave height is greater than 28 ft. This is because of their low slenderness ratios about both axes. For uplift loads their advantage over W-shapes is lessened. Columns are used for perimeter framing when the wall system is non-load-bearing. In some cases, interior walls are used rather than columns depending on architectural requirements.

4.3 EXTERIOR WALL SYSTEMS

Due to the high wind loads and projectile protection, hard wall systems are generally required. They include:

1. Reinforced masonry
2. Tilt-up concrete
3. Precast concrete
4. Steel deck with nail-base insulation and plywood as discussed in Section 3.5
5. Nuconsteel composite plywood-steel sheet assembly as discussed in Section 3.6

The wall systems listed can provide adequate vertical and horizontal strengths. The advantage of one system over another is generally based on location in the country and the general contractor's preference.

4.4 LATERAL FORCE-RESISTING SYSTEMS AND DIAPHRAGMS

Lateral force-resisting systems include:

1. Shear walls
2. Moment frames
3. Braced frames

Each system listed can be designed to adequately resist the shear forces from hurricane and tornado loads. Careful detailing and design for hold-down forces and force transfer into and out of each system is extremely important. Each of the systems is typically used at the perimeter of the building; however, depending upon force requirements, interior lateral force-resisting systems may be required. In the authors' opinion, the aspect ratio for nonhurricane diaphragms used to transfer the lateral loads to shear walls, moment frames and braced frames is limited to span-to-depth ratios of approximately 5:1. For the higher force requirements of hurricanes and tornadoes, the aspect ratios should be reduced. This not only reduces the shear requirements, but also helps reduce uplift forces.

Building aspect ratios also dictate whether shear walls or braced frames should be used in lieu of moment frames. For relatively square buildings, the most economical structure is one using braced frames or shear walls. For long narrow buildings, moment frames may be the optimum framing scheme if interior shear wall or interior bracing is not permitted.

4.5 CONNECTION DETAILS

Representative connection details are shown in this section. By no means are all of the possible details presented; however, the choices shown represent details that can be used for high wind uplift loads. The details are not typical for structures with moderate to low wind uplift. Illustrated in Figures 4-2, 4-5, 4-13 and 4-17 are single-course bond beams. In many situations, double-course bond beams or poured concrete bond beams are required. Fastener types and spacing shown in the figures are examples; other fastener types and spacing are permitted.

As in all construction, the load path through the connections to the supporting element(s) is of paramount importance. Loads must be transmitted to the foundation system, which in turn must be designed for all load combinations. Forces and connection details must be properly specified and shown on the contract documents.

Purlin, Girder and Column Connections

Purlin, girder and column connection details are shown in Figures 4-1 through 4-16. Deck edge angles and their support connections are not shown in the details for clarity.

Shear Wall Connections

Shear wall connections are shown in Figures 4-17 and 4-18.

Moment Frame Connection

Details are not presented for rigid frame construction because they are adequately covered in many textbooks and other technical literature. A typical moment connection of a joist girder to a W-shape column is shown in Figure 4-19.

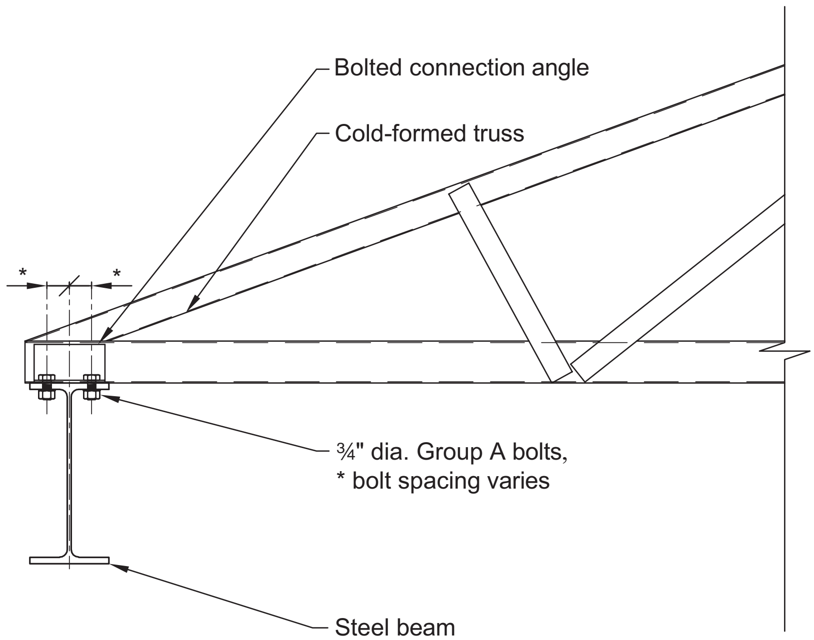

Fig. 4-1. Cold-formed truss-to-steel beam connection.

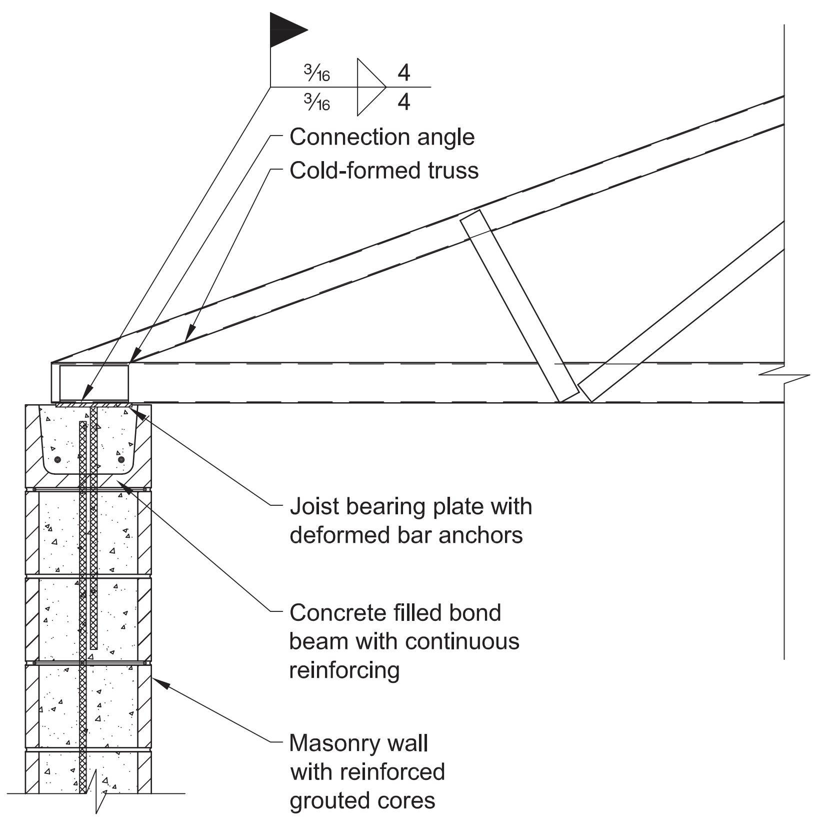

Fig. 4-2. Cold-formed truss-to-masonry wall connection.

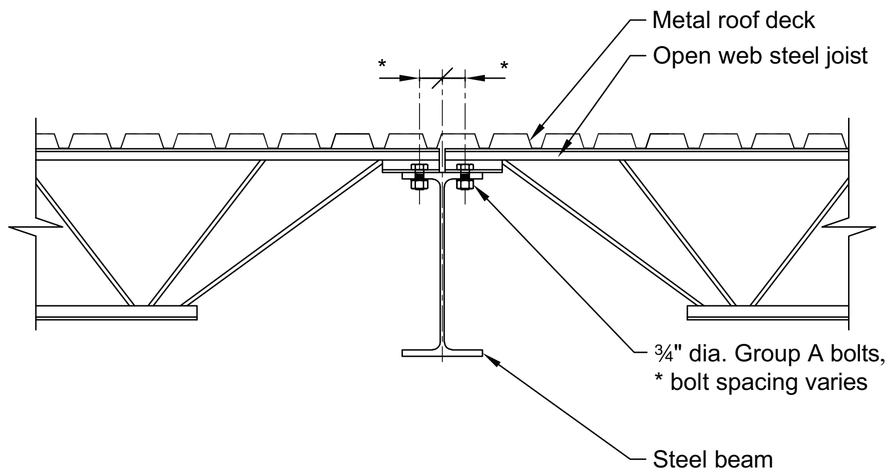

Fig. 4-3. Open-web steel joist-to-steel beam connection.

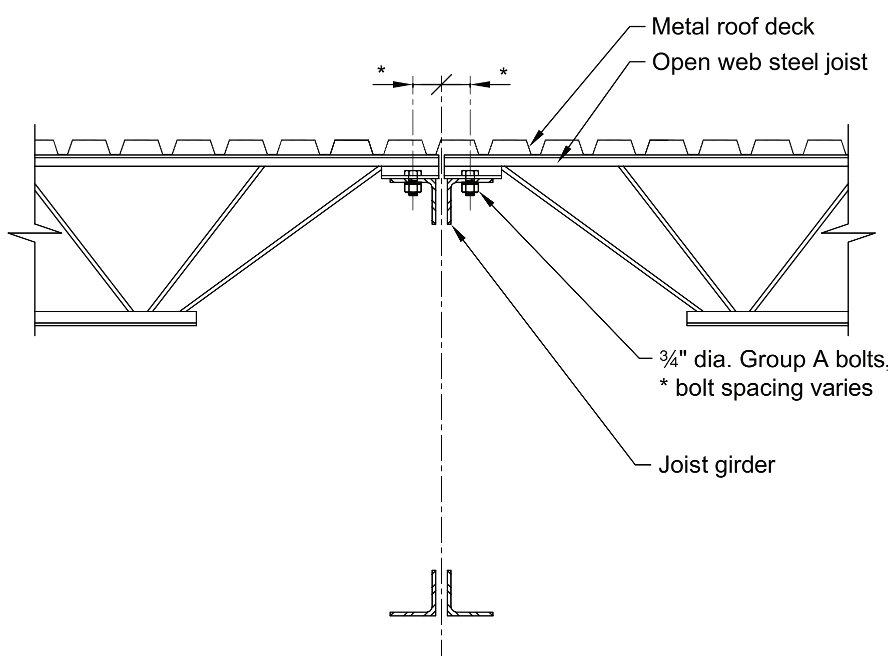

Fig. 4-4. Open-web steel joist-to-joist girder connection.

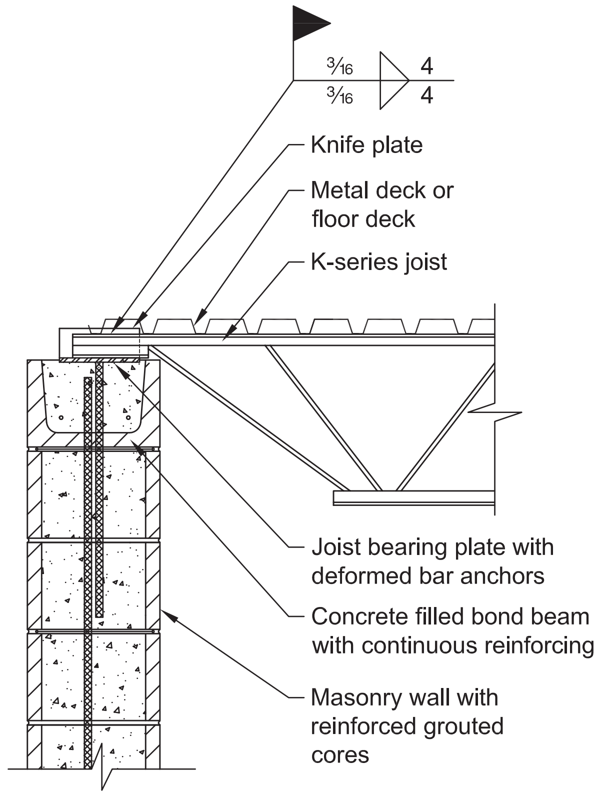

Fig. 4-5. Open-web steel joist-to-masonry wall connection. Contact SJI steel joist manufacturers for knife-plate details.

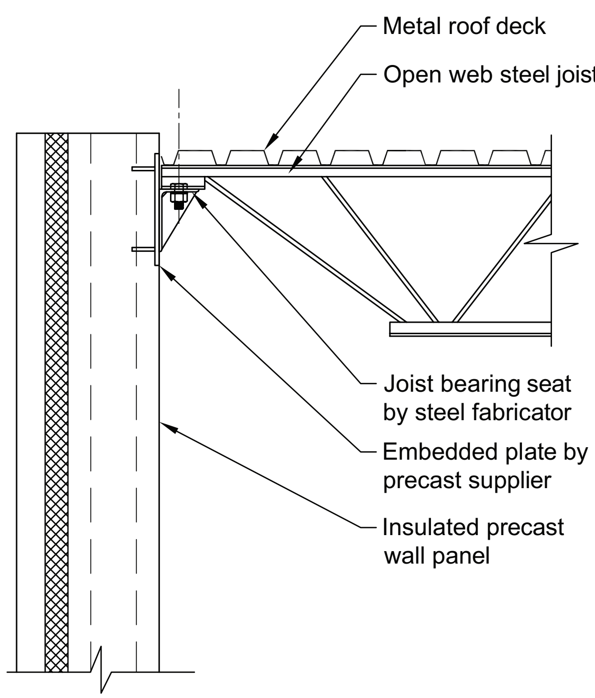

Fig. 4-6. Open-web steel joist-to-precast or tilt-up wall connection.

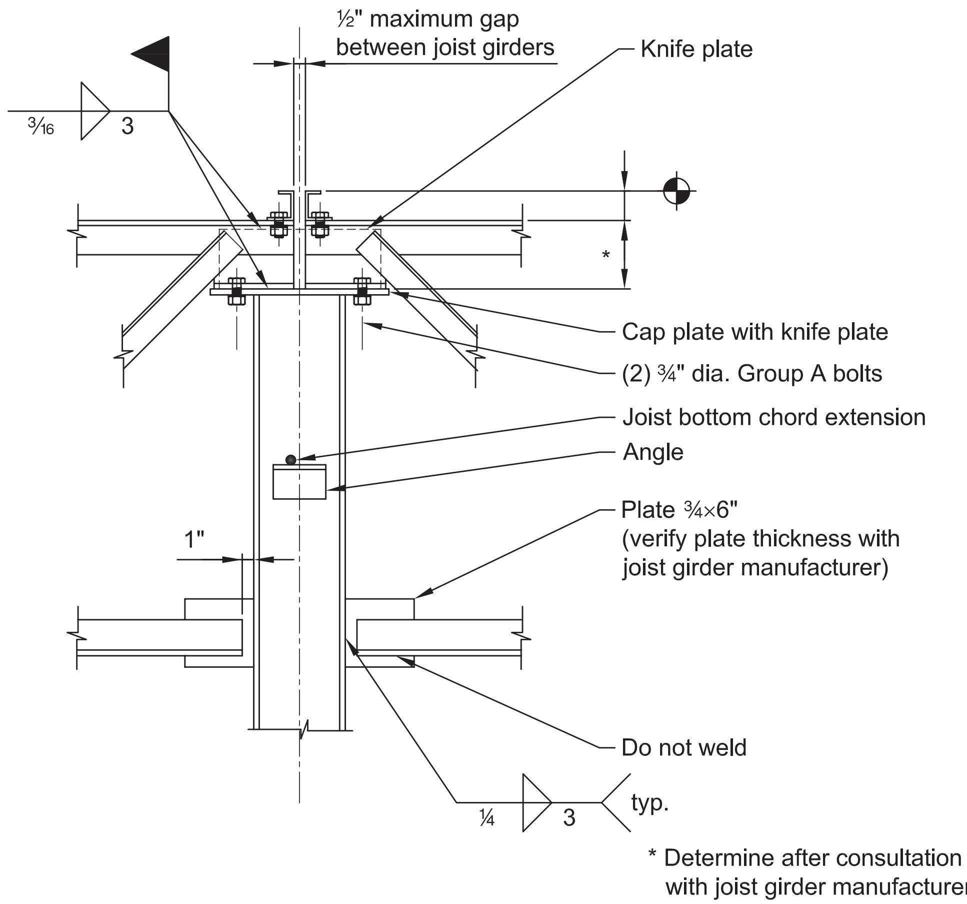

Fig. 4-7. Joist girder-to-W-shape column connection.

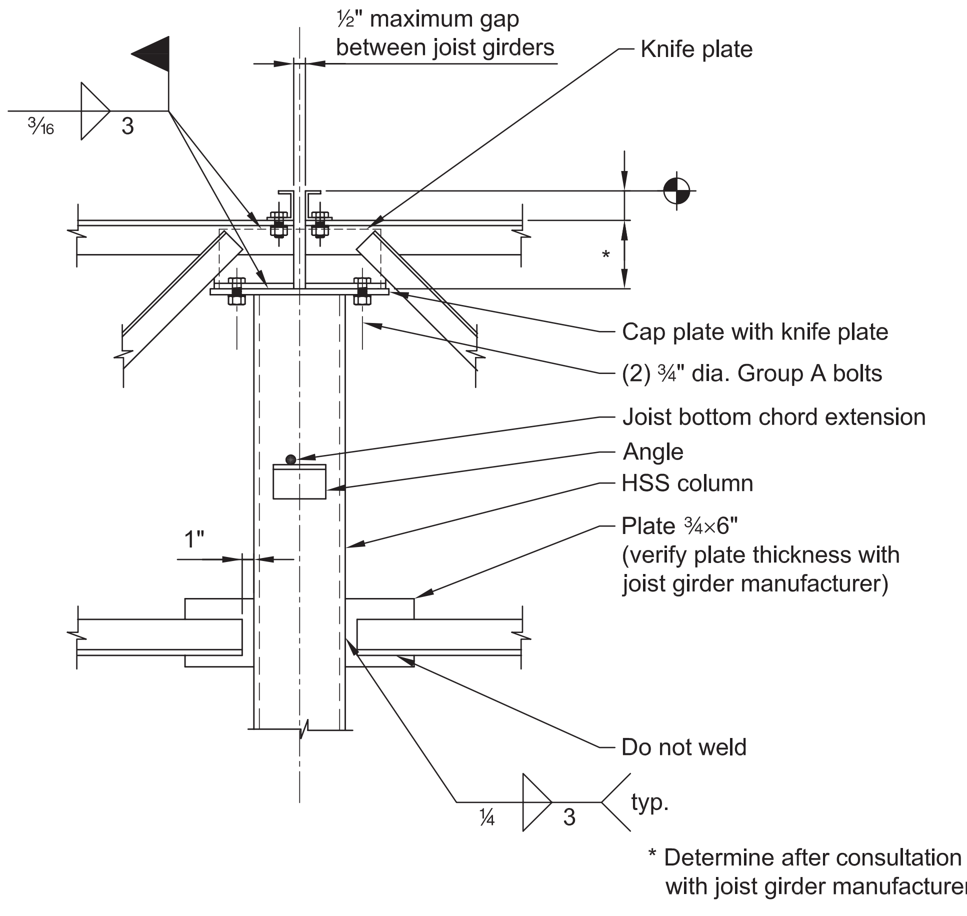

Fig. 4-8. Joist girder-to-HSS column connection.

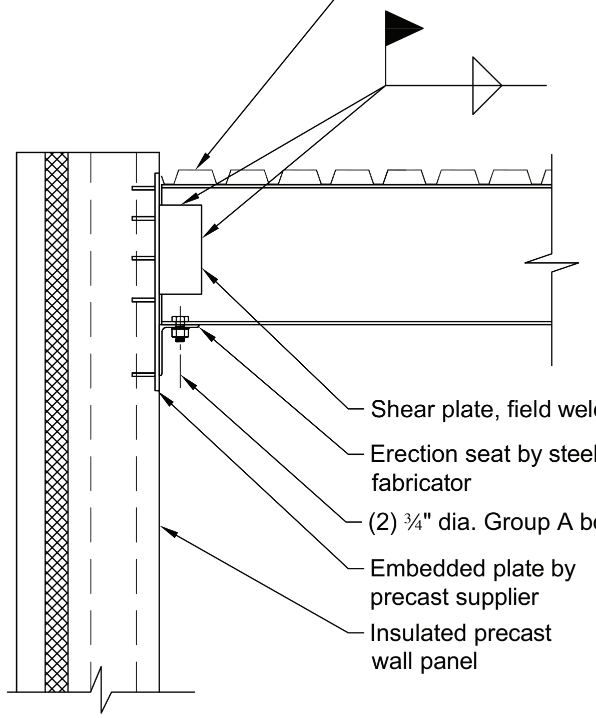

Fig. 4-9. W-shape beam-to-precast or tilt-up wall connection.

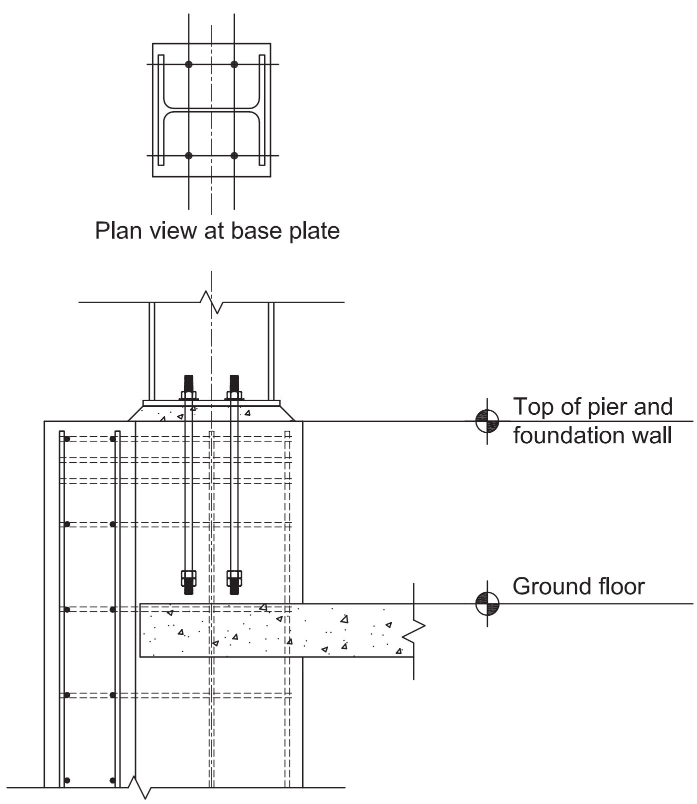

Fig. 4-10. W-shape column-to-concrete pier or exterior wall footing connection.

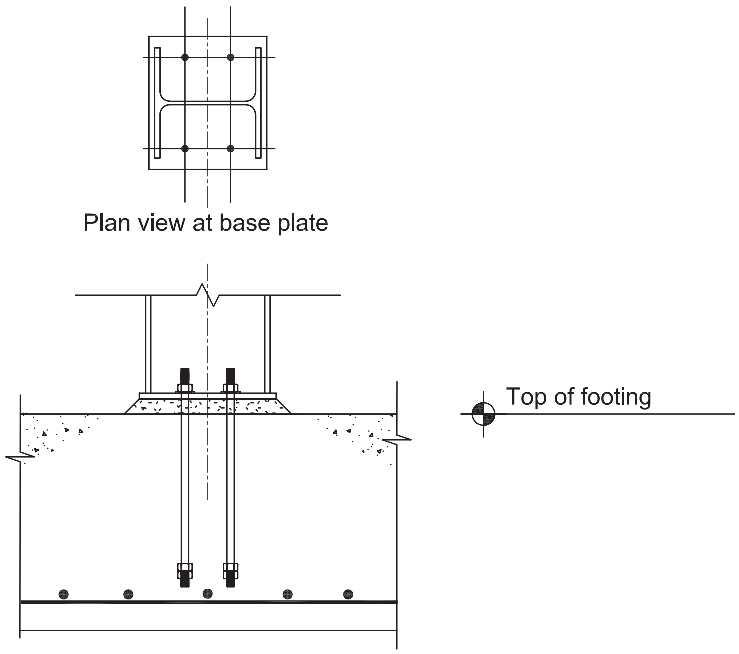

Fig. 4-11. W-shape column-to-concrete footing connection.

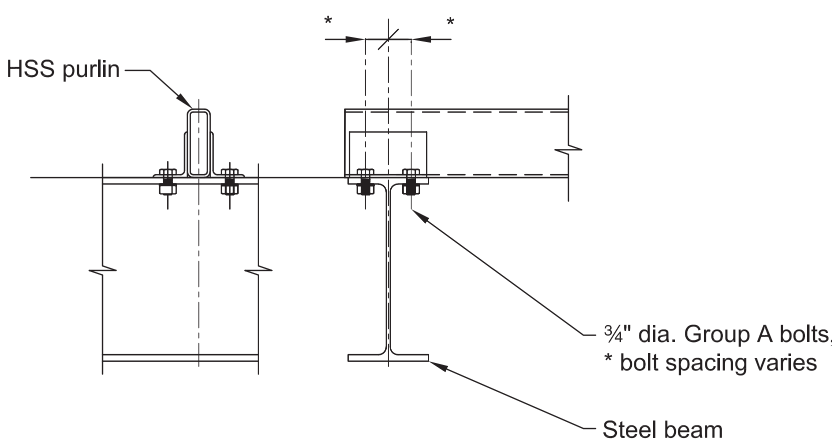

Fig. 4-12. HSS purlin-to-steel beam connection.

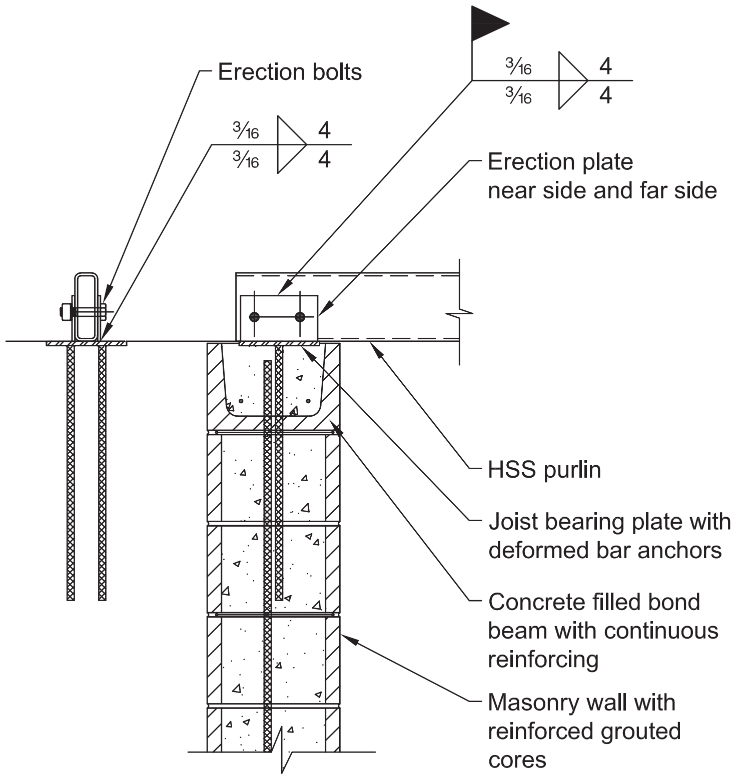

Fig. 4-13. HSS purlin-to-masonry wall connection.

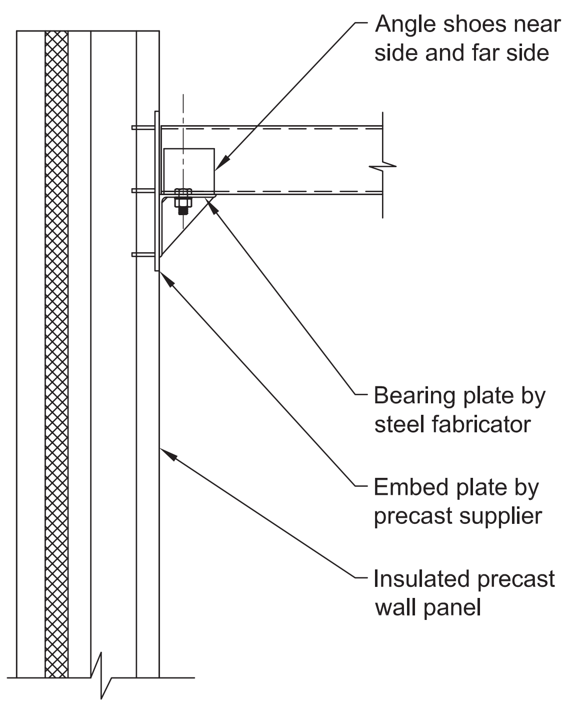

Fig. 4-14. HSS purlin-to-precast or tilt-up wall connection.

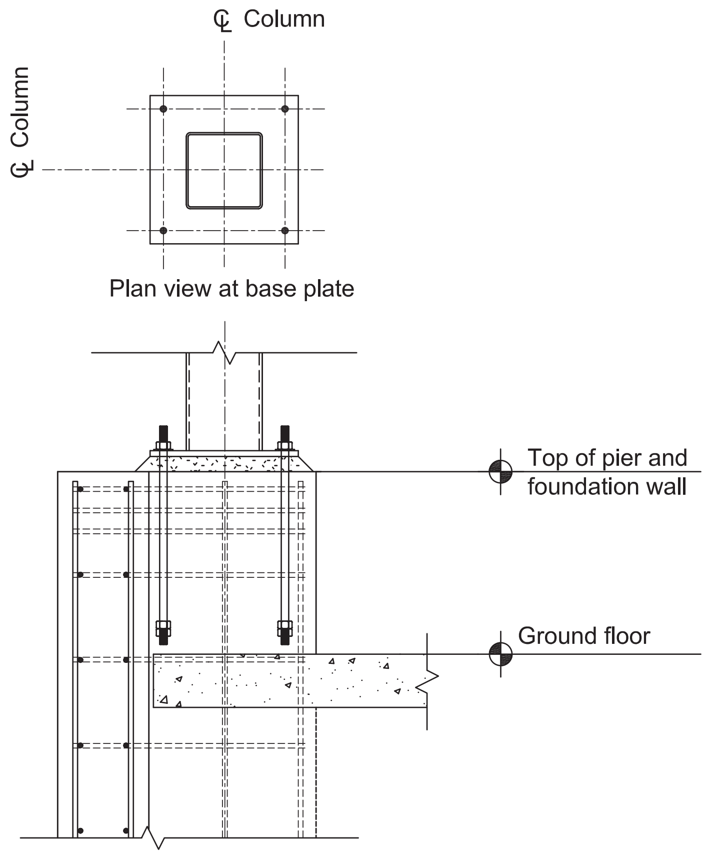

Fig. 4-15. HSS column-to-concrete pier or exterior wall footing connection.

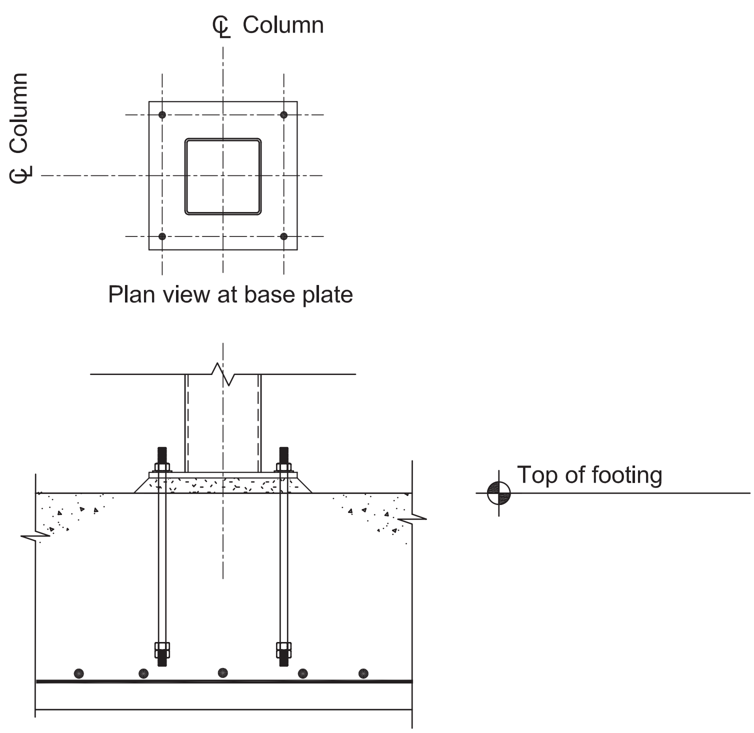

Fig. 4-16. HSS column-to-concrete footing connection.

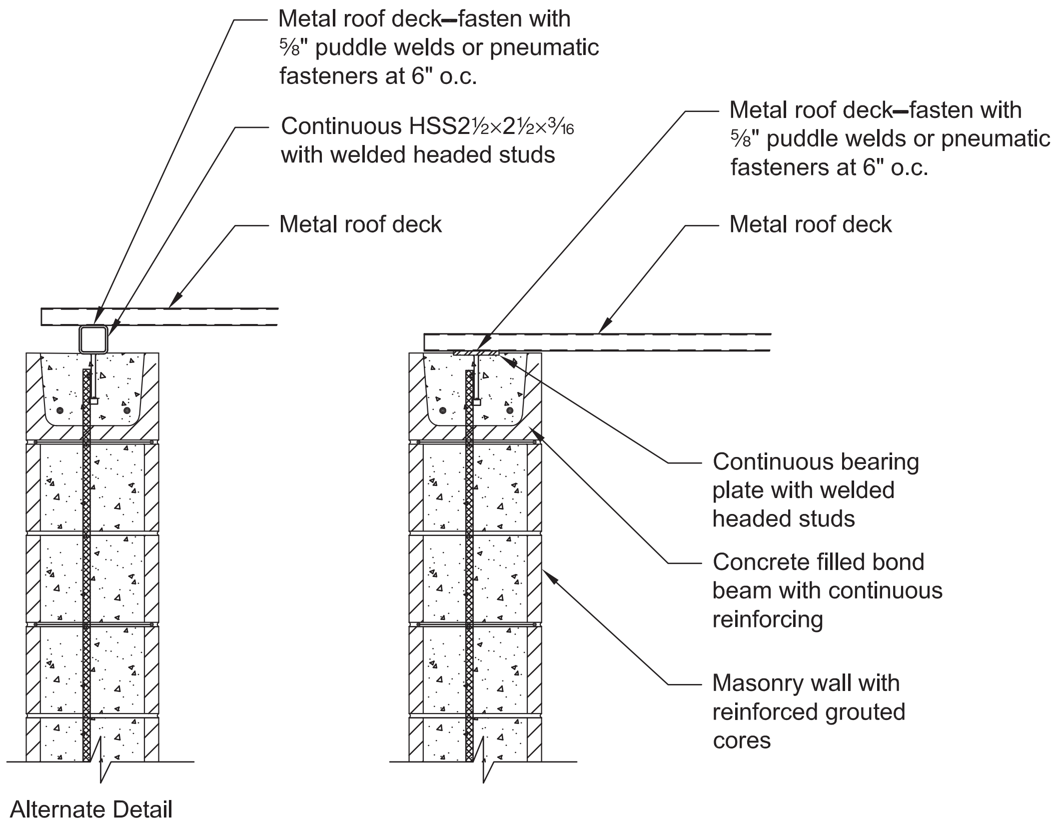

Fig. 4-17. Metal deck-to-masonry wall connection.

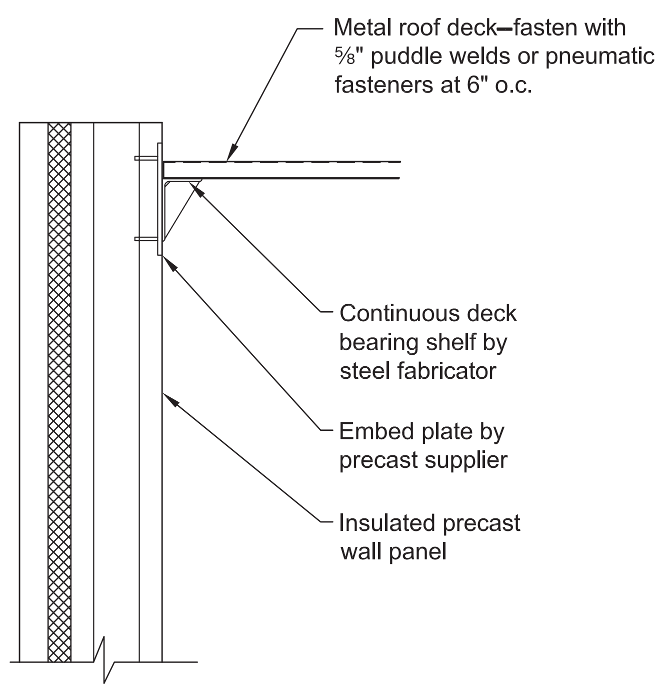

Fig. 4-18. Metal deck-to-precast or tilt-up wall connection.

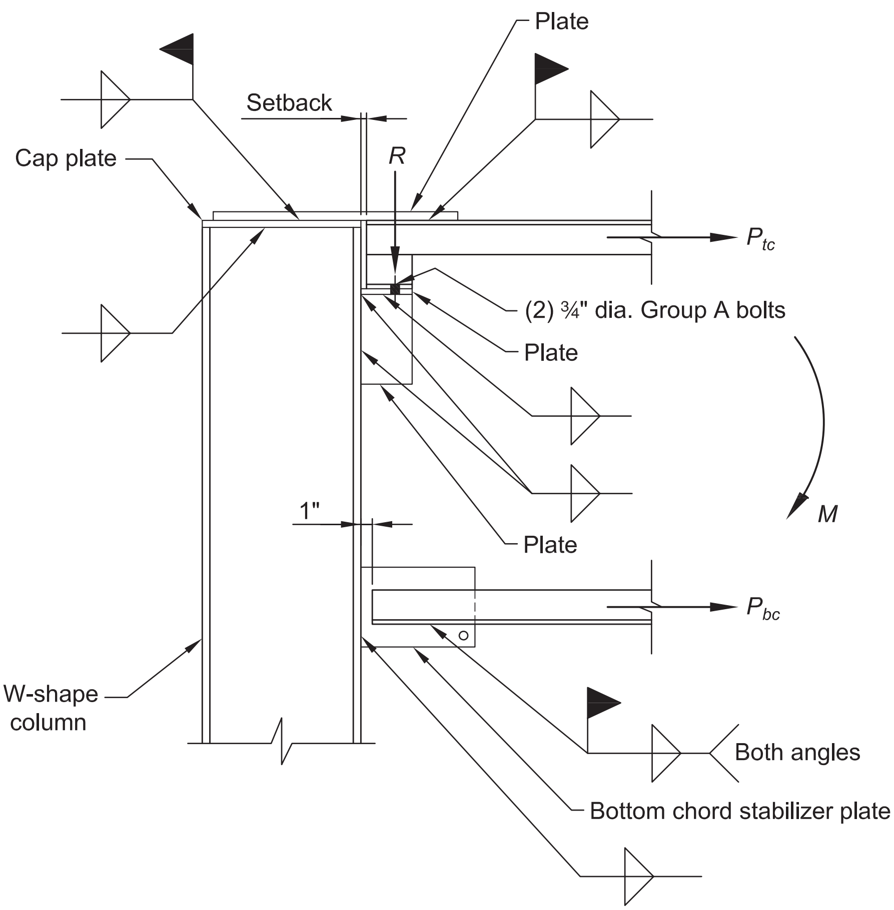

Fig. 4-19. Moment connection—joist girder-to-wide-flange column.

Chapter 5
Other Design Considerations

Designing a building to ensure it will remain operational if struck by an EF4- or EF5-rated tornado can be expensive. However, for facilities such as emergency operation centers and hospitals where it is necessary to avoid interrupted operations, refer to *Mitigation Assessment Team Report—Spring 2011 Tornadoes: April 25–28 and May 22; Building Performance Observations, Recommendations, and Technical Guidance*, FEMA P-908 (FEMA, 2012), for design guidance.

For hurricane construction, the *Coastal Construction Manual*, FEMA P-55 (FEMA, 2011), is a two-volume publication that provides a comprehensive approach to planning, siting, designing, constructing and maintaining homes in the coastal environment. Volume I provides information regarding hazard identification, siting decisions, regulatory requirements, economic implications and risk management. The primary audience for Volume I is design professionals, officials, and those involved in the decision-making process. Volume II contains in-depth descriptions of design, construction and maintenance practices that, when followed, will increase the durability of buildings in the harsh coastal environment and reduce economic losses associated with coastal natural disasters. The primary audience for Volume II is the design professional who is familiar with building codes and standards and has a basic understanding of engineering principles.

5.1 SITING FOR SHELTERS

If possible, safe rooms should be located outside known flood-prone areas and away from any potential large debris sources, such as trees, poles, towers, antennas, satellite dishes, and roof-mounted mechanical equipment that could topple or become airborne during a tornado or hurricane.

5.1.1 ICC 500

ICC 500 (ICC, 2014) requires that community shelters are located outside of the following high-risk flood hazard areas:

1. Flood hazard areas subject to high-velocity wave action (V zones)
2. Floodways

Exception: Community shelters are to be permitted in flood hazard areas subject to high-velocity wave action (V zones) where permitted by the Board of Appeals in accordance with the provisions of the International Building Code (ICC, 2015).

The lowest floor used for a community shelter is to be the highest of the following elevations:

1. Flood elevation, including coastal wave effects, having a 0.2% annual chance of being equaled or exceeded in any given year
2. Flood elevation corresponding to the highest recorded flood elevation if a flood hazard study has not been conducted for the area
3. Maximum flood elevation associated with any modeled hurricane category including coastal wave effects
4. Minimum elevation of the lowest floor required by the authority having jurisdiction (AHJ) for the location
5. 2 ft above the flood elevation

Items 1 and 3 do not apply to shelters designated as tornado safe rooms only.

5.1.2 FEMA P-361

ICC 500, Chapter 4, is used as the basic requirements for the siting of community shelters. However, FEMA P-361 (FEMA, 2015a) imposes a few more conservative requirements.

Community shelters are to be located outside of the following high-risk flood hazard areas:

1. Flood hazard areas subject to high-velocity wave action (V Zones) and Coastal A Zones
2. Floodways

Exception: Community safe rooms are permitted in flood hazard areas subject to high-velocity wave action (V Zones) and Coastal A Zones where permitted by the Board of Appeals in accordance with the provisions of the International Building Code and after completing the eight-step decision process for Executive Order (EO) 11988, as amended, and as provided by Title 44 of the Code of Federal Regulations Part 9.6, Decision-Making Process (Federal Register, 2018). Coastal A Zones are defined as the area landward of Zone V or landward of an open coast without mapped Zone V. The inland limit of the Coastal A Zone is the limit of moderate wave action if delineated on a flood insurance rate map or designated by the AHJ.

The lowest floor used for safe room space and/or safe room support areas should be elevated to the higher of the following elevations, which should be used as the design flood elevation (DFE) for flood load calculations:

1. Flood elevation, including coastal wave effects, having a 0.2% annual chance of being equaled or exceeded in any given year
2. Flood elevation corresponding to the highest recorded

flood elevation if a flood hazard study has not been conducted for the area
3. Maximum flood elevation associated with any modeled hurricane category including coastal wave effects
4. Minimum elevation of the lowest floor required by the AHJ for the location
5. 2 ft above the flood elevation

Item 3 does not apply to shelters designated and used only as tornado safe rooms.

Safe rooms subject to flooding, including any foundation or building component supporting the safe room, should be designed with consideration given to the provisions of ASCE 24-05.

5.2 OCCUPANCY, MEANS OF EGRESS, AND ACCESS

5.2.1 ICC 500

The number of standing, seated, wheelchair, or bed-ridden spaces is to be determined by the applicable AHJ and the designer. ICC 500, Chapter 5, stipulates minimum usable shelter floor areas.

Means-of-egress doors are to be determined based on the occupancy load in accordance with the applicable building code. If the local building code requires only one door, an emergency escape opening is to be provided. The emergency escape opening is an additional door or an opening having a minimum of 5.7 ft^2 in area. See ICC 500 for details regarding escape opening requirements.

5.2.2 FEMA P-361

Safe room occupancy, means of egress, access, and accessibility should be designed and constructed in accordance with the provisions of ICC 500, Chapter 5. FEMA P-361 does not recommend any additional criteria.

From a design and construction standpoint, there is no limitation on the maximum population that a safe room may be designed to protect. However, there are size limitations for safe rooms funded by FEMA grants. Refer to FEMA P-361, Section A1.1, for guidance and criteria related to the maximum allowable population. Additionally, the latest guidance regarding the design and construction for a community safe room may be obtained from the FEMA regional office.

5.3 SIGNAGE

5.3.1 ICC 500

Signage is required within a facility to direct occupants to the shelter areas. In addition, at every entrance to a shelter, signage indicating "Tornado Shelter" or "Hurricane Shelter" is required as shown in Figure 5-1. In lieu of a sign, an appropriate symbol may be used. ICC 500, Section A117.1, provides applicable requirements. In addition to signs at the shelter locations, identifying signs posted 60 in. above the finished floor to the centerline of the sign are required, depicting the general location of shelters and access ways at the following locations:

- Adjacent to access doors on the inside of the shelter
- Office of the facility manager
- In the designated shelter manager's area within the shelter

5.3.2 FEMA P-361

All safe rooms should have a visible and legible sign outside or inside the safe room. Signs should include the following information:

- Name of the manufacturer or builder of the safe room
- Its purpose (i.e., the storm type: tornado or hurricane)
- The design wind speed

In addition, community safe rooms are also required to have additional signage, as follows:

- An entrance sign at every entrance to the safe room, indicating "Tornado Safe Room" or "Hurricane Safe Room."
- An identifying sign, different from the entrance sign, depicting the general location of the safe room(s) and access ways. Identifying signs should be posted in prominent locations 60 in. above the finished floor to the centerline of the sign. An identifying sign is required in each of the following locations:
  (a) Adjacent to access doors on the inside of the safe room
  (b) The office of the facility manager, if present
  (c) In the designated safe room manager's area within the safe room, if present

5.4 FIRE SAFETY

5.4.1 ICC 500

For community shelters, fire barriers and horizontal assemblies separating spaces or areas designed as shelters from other building areas are to have a minimum two-hour fire rating and are to be constructed in accordance with the applicable building code. A fire extinguisher meeting the requirements of *Portable Fire Extinguishers*, NFPA 10-13 (NFPA, 2013), is required within the shelter. Placement of the fire extinguisher must not compromise the structural or missile impact performance of the exterior shelter envelope.

5.4.2 FEMA P-361

FEMA P-361 refers to ICC 500 for fire criteria. FEMA does not stipulate additional requirements.

5.5 OPERATING A SHELTER OR SAFE ROOM

5.5.1 ICC 500

In addition to the storm shelter's structural performance requirements, the following operational, maintenance, and human factor criteria must be considered:

- Standby power (e.g., generator)
- Protection of critical support systems such as a generator
- Occupancy duration
- Ventilation
- Minimum square footage per occupant
- Egress
- Distance and travel time for occupants traveling to the safe room
- Access for disabled occupants
- Special needs requirements
- Lighting
- Emergency provisions (food, water, sanitation management, emergency supplies, communication equipment)
- Operations and maintenance plans for the safe room

Each of these items is further elaborated on in ICC 500 and FEMA P-361.

5.5.2 FEMA P-361

Ventilation, sanitation, power, and other recommendations for tornado community safe rooms should be incorporated into the design of the safe room in accordance with ICC 500. In addition, the safe room should be equipped with an electrical system having an emergency power backup system for lighting and other needs as stipulated in accordance with ICC 500.

5.6 EXISTING BUILDINGS

Buildings that do not have areas designed to serve as a shelter or safe room should be assessed to determine the best available refuge area. For example, interior areas with short-span

Fig. 5-1. Signage examples.

roof systems, such as restrooms and corridors, may serve as refuge areas. *Tornado Protection: Selecting Refuge Areas in Buildings*, FEMA P-431 (FEMA, 2009), and the checklist in FEMA P-361, Appendix C, provides guidelines for selecting a refuge area.

5.6.1 Retrofit of Existing Buildings

When retrofitting an existing area, the previously discussed ICC 500 design requirements and FEMA P-361 guidance must be achieved. Thus, retrofitting an existing refuge area (e.g., hallways/corridors, bathrooms, workrooms, laboratory areas, kitchens and mechanical rooms) to serve as a safe room can be both technically challenging and expensive. FEMA P-361 suggests consideration of the following:

- **Roof system (roof deck and structural supporting members).** Are the roof deck and structural supporting members over the proposed refuge area structurally independent of the remainder of the building? If not, is it possible to strengthen the existing roof to resist the expected wind and debris loads? Can the openings in the roof system for mechanical equipment or lighting be protected during an extreme-wind event? It may not be reasonable to retrofit the rest of the proposed safe room if the roof system is part of a building-wide system that was not designed for ultimate-wind load requirements.
- **Wall system.** Are the wall systems accessible so that they can be retrofitted for improved resistance to wind pressure and missile impact? It may not be reasonable to retrofit a proposed safe room area to protect the roof or the openings if the wall systems (load-bearing or non-load-bearing) cannot withstand wind pressures or cannot be retrofitted in a reasonable manner to withstand wind pressures and missile impacts.
- **Openings.** Windows and doors are extremely vulnerable to wind pressures and debris impact. Shutter systems and doors rated to meet debris impact criteria may be used to protect windows for tornado and hurricane missile impacts. There is often only minimal warning time before a tornado; therefore, a shelter design that relies on manually installed shutters may be impractical. Automated shutter systems may be considered, but they would require a protected backup power system to ensure that the shutters are closed before an event. Doors should be constructed of impact-resistant materials (e.g., steel) and secured with six points of connection (typically three hinges and three latching mechanisms); regardless of the number of hinges and latches, all doors should be tested to meet the debris impact testing requirements of ICC 500, Chapter 8. Door frames should be constructed of at least 16-gage metal and adequately secured to the walls to prevent the complete failure of the door/frame assemblies.
- **Existing functions and conditions in the refuge area.** For example, bathrooms have been used as refuge areas during tornadoes and hurricanes because they often have minimal numbers of openings to protect. However, emergency managers may find it difficult to persuade people to sit on the floor of a bathroom when the sanitary condition of the floor cannot be guaranteed. Also, mechanical rooms that are noisy and may contain hot or dangerous machinery should be avoided as refuge areas whenever possible. The permanent fixtures and furnishings in a proposed safe room area (e.g., permanent tables, cabinets, sinks and large furniture) occupy some of the available space within the safe room, and they may make the safe room uncomfortable for its occupants or pose a hazard to the occupants. These types of safe room areas should be used only when a better option is not available.

5.7 ENGINEERING DESIGN DOCUMENTS

Engineers are encouraged to add the following information and qualifiers to their contract documents:

- The identified area should be considered by building owners as only a "best available area of refuge" and occupants could still be injured or killed
- Missile impact tests performed
- Total number of occupants the area can hold
- The approximate maximum safe wind speed for the best available refuge area
- The timeframe before which the area should be reevaluated
- An outline of potential modifications that could be made to the structure to improve its performance in high-wind events
- Changes to the building may make the refuge area no longer the best available refuge area

Agreement between the client and the design professional on these points may ease liability concerns. Administrators and facilities managers for buildings with large occupancies should also review FEMA P-431 (FEMA, 2009) and the refuge area evaluation checklists presented in FEMA P-361, Appendix C.

Chapter 6
Design Examples

The following examples are representative of the wind load calculations and the magnitude of design wind load that may be required when designing a storm shelter or safe room.

Example 6.1—ICC 500 and FEMA P-361 Tornado Wind Load Calculation

Given:

Determine the main wind force-resisting system (MWFRS) design tornado wind pressure, p, on the windward wall of a building (Exposure C) having a mean roof height of 35 ft located in Joplin, MO. All wind pressures will be affected by the ICC 500 and FEMA P-361 modifications; however, only windward wall pressure is being illustrated when using ASCE/SEI 7-10, Chapter 27, Part 1.

Solution:

The following modifications are made according to ICC 500 and FEMA P-361 requirements:

Exposure C
$GC_{pi} = \pm 0.55$
$K_d = 1.0$
$K_{zt} = 1.0$
$V = 250$ mph read from Figure 2-1

Although the openings must be protected, ICC 500 requires that the largest opening be considered open; therefore, the calculated design wind pressure is computed for a partially enclosed building.

The design wind pressure, p, is calculated using ASCE/SEI 7-10, Equation 27.4-1:

$$p = qGC_p - q_i(GC_{pi}) \qquad \text{(ASCE/SEI 7-10, Eq. 27.4-1)}$$

From ASCE/SEI 7-10:

$C_p = 0.8$
$G = 0.85$
$K_z = 1.01$ for a mean roof height of 35 ft

The velocity pressure, q, is taken as q_z for windward walls evaluated at height z above the ground and is calculated using ASCE/SEI 7-10, Equation 27.3-1:

$$\begin{aligned} q_z &= 0.00256K_zK_{zt}K_dV^2 && \text{(ASCE/SEI 7-10, Eq. 27.3-1)} \\ &= 0.00256(1.01)(1.0)(1.0)(250\text{ mph})^2 \\ &= 162\text{ psf} \end{aligned}$$

The velocity pressure, q_i, may conservatively be evaluated at height h when calculating positive internal pressure in partially enclosed buildings, in which case, $q_i = q_h = q_z$. The MWFRS design wind pressure can then be calculated using ASCE/SEI 7-10, Equation 27.4-1:

$$\begin{aligned} p &= q_zGC_p - q_z(GC_{pi}) && \text{(from ASCE/SEI 7-10, Eq. 27.4-1)} \\ &= (162\text{ psf})(0.85)(0.8) - (162\text{ psf})(\pm 0.55) \\ &= 199\text{ psf} \end{aligned}$$

Example 6.2—ASCE/SEI 7-16 Commentary Tornado Wind Load Calculation—Extended Method

Given:

Determine the MWFRS design tornado wind pressure, p, on the windward wall of a building (Exposure C) having a mean roof height of 35 ft located in Joplin, MO. Use the Extended Method discussed in ASCE/SEI 7-16 Commentary, Chapter 26, and ASCE/SEI 7-16, Chapter 27, Part 1.

Solution:

The following values are obtained using ASCE/SEI 7, Chapter 27, Part 1, with modifications provided in Commentary Chapter 26:

C_p = 0.8
G = 0.9
GC_{pi} = ±0.55
K_d = 1.0
K_z = velocity pressure exposure coefficient evaluated at mean roof height for Exposure C
= 1.01
K_{zt} = 1.0
V = upper end of wind speed range for target EF scale or the speed from ICC 500 or FEMA P-361
= 250 mph

Although the openings must be protected, ICC 500 requires that the largest opening be considered open; therefore, calculate the design wind pressure for a partially enclosed building.

Commentary Chapter 26 allows the velocity pressure equation, $q_h = 0.00256K_zK_{zt}K_dV^2$, to be simplified for tornado wind pressure to:

$$
\begin{aligned}
q_h &= 0.00256K_zV^2 \\
&= 0.00256(1.01)(250\text{ mph})^2 \\
&= 162\text{ psf}
\end{aligned}
$$

Because of the nature of the wind profile in a tornado, ASCE/SEI 7-16 Commentary, Section C26.14.4, recommends that the velocity pressure, q, be determined at the mean roof height, h, and that q_h be used throughout the pressure calculations as the value for q. The MWFRS design wind pressure can then be calculated as:

$$
\begin{aligned}
p &= q_h[GC_p - (\pm 0.55)] \qquad \text{(ASCE/SEI 7-16, Eq. C26.14-2)} \\
&= (162\text{ psf})[(0.9)(0.8) - (\pm 0.55)] \\
&= 206\text{ psf}
\end{aligned}
$$

Example 6.3—ASCE/SEI 7-16 Commentary Tornado Wind Load Calculation—Simplified Method

Given:

Determine the MWFRS design tornado wind pressure, p, on the windward wall of a building (Exposure C) having a mean roof height of 35 ft located in Joplin, MO. Use the Simplified Method discussed in ASCE/SEI 7-16 Commentary, Chapter 26, and ASCE/SEI 7-16, Chapter 27, Part 1.

Solution:

The following values are obtained using ASCE/SEI 7, Chapter 27:

C_p = 0.8
G = 0.85
GC_{pi} = ±0.55

K_d = 0.85
K_e = 1.0
K_z = velocity pressure exposure coefficient evaluated at mean roof height for Exposure C
= 1.01
K_{zt} = 1.0
TF = 1.25 from Table 2-3
V_{design} = design wind speed from ASCE/SEI 7-16 wind map
= 120 mph
$V_{tornado}$ = selected tornado wind speed
= 250 mph for the top end of the EF2 range, which is appropriate for Joplin, MO

Although the openings must be protected, ICC 500 requires that the largest opening be considered open; therfore, calculate the design wind pressure for a partially enclosed building.

Using the Simplified Method, the design wind pressure equation becomes:

$$p = q_i\left(GC_p - GC_{pi}\right)\left(\frac{V_{tornado}}{V_{design}}\right)^2 TF \tag{2-1}$$

The velocity pressure, q_i, may conservatively be evaluated at height h when calculating positive internal pressure in partially enclosed buildings, in which case, $q_i = q_h = q_z$.

$$\begin{aligned} q_z &= 0.00256 K_z K_{zt} K_d K_e V^2 \qquad \text{(ASCE/SEI 7-16, Eq. 26.10-1)} \\ &= 0.00256(1.1)(1.0)(0.85)(1.0)(120\text{ mph})^2 \\ &= 34.5\text{ psf} \end{aligned}$$

The MWFRS design wind pressure can then be calculated as:

$$\begin{aligned} p &= q_z\left(GC_p - GC_{pi}\right)\left(\frac{V_{tornado}}{V_{design}}\right)^2 TF \qquad \text{(from Eq. 2-1)} \\ &= (34.5\text{ psf})\left[(0.85)(0.8) - (\pm 0.55)\right]\left(\frac{250\text{ mph}}{120\text{ mph}}\right)^2 (1.25) \\ &= 230\text{ psf} \end{aligned}$$

Example 6.4—ICC 500 and FEMA P-361 Hurricane Wind Load Calculation

Given:

Determine the MWFRS design hurricane wind pressure on the windward wall of a building (Exposure C) having a mean roof height of 35 ft located in New Orleans, LA. Although all wind pressures will be affected by the ICC 500 and FEMA P-361 modifications, only windward pressure is being illustrated when using ASCE/SEI 7-10, Chapter 27, Part 1.

Solution:

The following modifications are made according to ICC 500 requirements:

Exposure C
$GC_{pi} = \pm 0.55$
K_d = 1.0
K_{zt} = 1.0
V = 210 mph, from Table 2-2

Although the openings must be protected, ICC 500 requires that the largest opening be considered open; therefore, calculate the design wind pressure for a partially enclosed building.

The design wind pressure, p, is calculated using ASCE/SEI 7-10, Equation 27.4-1:

$$p = q_h G C_p - q_i (GC_{pi}) \qquad \text{(ASCE/SEI 7-10, Eq. 27.4-1)}$$

From ASCE/SEI 7-10:

$C_p = 0.8$
$G = 0.85$
$K_z = 1.01$ for a mean roof height of 35 ft

$$\begin{aligned} q_z &= 0.00256 K_z K_{zt} K_d V^2 && \text{(ASCE/SEI 7-10, Eq. 27.3-1)} \\ &= 0.00256(1.01)(1.0)(1.0)(210 \text{ mph})^2 \\ &= 114 \text{ psf} \end{aligned}$$

The velocity pressure, q_i, may conservatively be evaluated at height h when calculating positive internal pressure in partially enclosed buildings, in which case, $q_i = q_h = q_z$. The MWFRS design wind pressure can then be calculated using ASCE/SEI 7-10, Equation 27.4-1:

$$\begin{aligned} p &= q_h G C_p - q_i (GC_{pi}) && \text{(ASCE/SEI 7-10, Eq. 27.4-1)} \\ &= (114 \text{ psf})(0.85)(0.8) - (114 \text{ psf})(\pm 0.55) \\ &= 140 \text{ psf} \end{aligned}$$

Example 6.5—ASCE/SEI 7-10 Hurricane Wind Load Calculation

Given:

Determine the MWFRS design hurricane wind pressure, p, on the windward wall of a building (Exposure C) having a mean roof height of 35 ft in New Orleans, LA. Use ASCE/SEI 7-10, Chapter 27, Part 1.

Solution:

The following values are obtained using Chapter 27:

$C_p = 0.8$
$G = 0.85$
$GC_{pi} = \pm 0.55$
$K_d = 0.85$
K_z = velocity pressure exposure coefficient evaluated at mean roof height for Exposure C
$= 1.01$
$K_{zt} = 1.0$
V = 180 mph, from ASCE/SEI 7-10, Figure 26.5-1B

Although the openings must be protected, ICC 500 requires that the largest opening be considered open; therefore, calculate the design wind pressure for a partially enclosed building.

The design wind pressure, p, is calculated using ASCE/SEI 7-10, Equation 27.4-1:

$$p = q_h G C_p - q_i (GC_{pi}) \qquad \text{(from ASCE/SEI 7-10, Eq. 27.4-1)}$$

The velocity pressure, q_i, may conservatively be evaluated at height h when calculating positive internal pressure in partially enclosed buildings, in which case, $q_i = q_h = q_z$.

$$q_h = 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(ASCE/SEI 7-10, Eq. 27.3-1)}$$

$$= 0.00256(1.01)(1.0)(0.85)(180\text{ mph})^2$$
$$= 71.2\text{ psf}$$

The MWFRS design wind pressure can then be calculated using ASCE/SEI 7-10, Equation 27.4-1:

$$p = q_h GC_p - q_i(GC_{pi}) \quad \text{(from ASCE/SEI 7-10, Eq. 27.4-1)}$$
$$= (71.2\text{ psf})(0.85)(0.8) - (71.2\text{ psf})(\pm 0.55)$$
$$= 87.6\text{ psf}$$

Example 6.6—ICC 500 and FEMA P-361 Tornado C&C Wind Load Calculation

Given:

Evaluate the components and cladding (C&C) loading on a roof deck connector for a storm shelter in Joplin, MO. Determine the C&C tornado wind pressure on the corner zone, edge, and field of roof (Exposure C). The ICC 500 and FEMA P-361 wind loading criteria are being illustrated when using ASCE/SEI 7-10, Chapter 30, Part 1. The structure has a gable roof slope less than 7° and a height less than 60 ft. The tributary area for the joist is 25 ft^2.

Solution:

The following modifications are made according to ICC 500 requirements:

Exposure C
$GC_{pi} = \pm 0.55$
$K_d = 1.0$
$K_{zt} = 1.0$
$V = 250$ mph, from Table 2-2

From ASCE/SEI 7-10 and using Figure 30.4-2A:

$GC_p = -0.95$ (roof field zone 1)
$GC_p = -1.55$ (roof edge zone 2)
$GC_p = -2.1$ (roof corner zone 3)
$K_z = 0.85$

Although the openings must be protected, ICC 500 requires that the largest opening be considered open; therefore, the calculated design wind pressure is computed for a partially enclosed building.

The design wind pressure, p, is calculated using ASCE/SEI 7-10, Equation 30.4-1:

$$p = q_h[(GC_p) - (GC_{pi})] \quad \text{(ASCE/SEI 7-10, Eq. 30.4-1)}$$

The velocity pressure, q_h, is calculated using ASCE/SEI 7-10, Equation 30.3-1, for all three roof zones:

$$q_h = 0.00256K_zK_{zt}K_dV^2 \quad \text{(ASCE/SEI 7-10, Eq. 30.3-1)}$$
$$= 0.00256(0.85)(1.0)(1.0)(250\text{ mph})^2$$
$$= 136\text{ psf}$$

The maximum C&C design wind pressure can then be calculated using ASCE/SEI 7-10, Equation 30.4-1.

Roof field zone:

$$p = q_h[(GC_p) - (GC_{pi})] \quad \text{(ASCE/SEI 7-10, Eq. 30.4-1)}$$
$$= (136\text{ psf})[(-0.95) - (\pm 0.55)]$$
$$= -204\text{ psf}$$

Roof edge zone:

$$p = q_h[(GC_p) - (GC_{pi})] \qquad \text{(ASCE/SEI 7-10, Eq. 30.4-1)}$$
$$= (136 \text{ psf})[(-1.55) - (\pm 0.55)]$$
$$= -286 \text{ psf}$$

Roof corner zone:

$$p = q_h[(GC_p) - (GC_{pi})] \qquad \text{(ASCE/SEI 7-10, Eq. 30.4-1)}$$
$$= (136 \text{ psf})[(-2.1) - (\pm 0.55)]$$
$$= -360 \text{ psf}$$

Symbols

C_p External pressure coefficient

G Gust-effect factor

GC_p Product of external pressure coefficient and gust-effect factor to be used in determination of wind loads for buildings

GC_{pi} Product of internal pressure coefficient and gust-effect factor to be used in determination of wind loads for buildings

K_d Wind directionality factor

K_z Velocity pressure exposure coefficient

K_{zt} Topographic factor

TF Tornado factor

V Basic wind speed, mph

V_{design} ASCE/SEI 7 mapped wind speed for the location, mph

$V_{tornado}$ Selected wind speed to be used for this design, mph

p Design pressure to be used in determination of wind loads for buildings, psf

q Velocity pressure, psf

q_h Velocity pressure calculated at mean roof height h, psf

q_i Velocity pressure for internal pressure determination, psf

q_z Velocity pressure evaluated at height z above the ground, psf

Acronyms and Abbreviations

AAMA	American Architectural Manufacturers Association
ACI	American Concrete Institute
AHJ	authority having jurisdiction
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
APC	atmospheric pressure change
ASCE	American Society of Civil Engineers
ASD	allowable strength design
CMU	concrete masonry unit
DI	damage indicator
DOD	degree of damage
DASMA	Door and Access Systems Manufacturers Association
DFE	design flood elevation
EF	enhanced Fujita
EO	executive order
FEMA	Federal Emergency Management Agency
HSS	hollow structural section
ICC	International Code Council
LRFD	load and resistance factor design
MBMA	Metal Building Manufacturer's Association
MWFRS	main wind force-resisting system
NFPA	National Fire Protection Association
NWI	National Wind Institute
PCI	Precast Concrete Institute
SDI	Steel Deck Institute
SFA	Steel Framing Alliance
SJI	Steel Joist Institute
STI	Steel Tube Institute

References

AAMA (2011), *Voluntary Specifications for Tornado Hazard Mitigation Fenestration Products*, AAMA 512-11, American Architectural Manufacturers Association, Schaumburg, IL.

ACI (2005), *Guide for the Consolidation of Concrete*, ACI 309R-05, American Concrete Institute, Farmington Hills, MI.

ACI (2014), *Building Code Requirements for Structural Concrete and Commentary*, ACI 318, American Concrete Institute, Farmington Hills, MI.

AISC (2016), *Specification for Structural Steel Buildings*, ANSI/AISC 360-16, American Institute of Steel Construction, Chicago, IL.

AISI (2015), *North American Standard for Cold-Formed Steel Structural Framing*, AISI S240-15, American Iron and Steel Institute, Washington, DC.

AISI (2016), *North American Specification for the Design of Cold-Formed Steel Structural Members*, AISI S100-16, American Iron and Steel Institute, Washington, DC.

ANSI/DASMA (2012), *Standard Method for Testing Sectional Garage Doors and Rolling Doors: Determination of Structural Performance under Missile Impact and Cyclic Wind Pressure,* ANSI/DASMA 115-12, Door & Access Systems Manufacturers Association International, Cleveland, OH.

ASCE (2005), *Flood Resistant Design and Construction*, ASCE 24-05, American Society of Civil Engineers, Reston, VA.

ASCE (2010), *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-10, American Society of Civil Engineers, Reston, VA.

ASCE (2016), *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16, American Society of Civil Engineers, Reston, VA.

ASTM (2002), *Standard Test Method for Structural Performance of Exterior Windows, Doors, Skylights and Curtain Walls for Uniform Static Air Pressure Difference*, ASTM E330-02, ASTM International, West Conshohocken, PA.

ASTM (2005), *Standard Test Method for Performance of Exterior Windows, Curtain Walls, Doors and Impact Protective Systems Impacted by Missile(s) and Exposed to Cyclic Pressure Differentials*, ASTM E1886-05, ASTM International, West Conshohocken, PA.

ASTM (2012), *Standard Specification for Performance of Exterior Windows, Curtain Walls, Doors, and Impact Protective Systems Impacted by Windborne Debris in Hurricanes*, ASTM E1996-12, ASTM International, West Conshohocken, PA.

ASTM (2016a), *Standard Specification for Faced Rigid Cellular Polyisocyanurate Thermal Insulation Board*, ASTM C1289-16a, ASTM International, West Conshohocken, PA.

ASTM (2016b), *Standard Test Methods for Fire Tests of Building Construction and Materials*, ASTM E119-16, ASTM International, West Conshohocken, PA.

ASTM (2017), *Standard Test Methods for Fire Tests of Roof Coverings*, ASTM E108-17, ASTM International, West Conshohocken, PA.

DOC (2010), *American Softwood Lumber Standard*, PS 20-10, U.S. Department of Commerce, Gaithersburg, MD.

Federal Register (2018), *Code of Federal Regulations*, Title 44, Part 9.6, Executive Order 11988, Federal Register, Washington, DC.

FEMA (2009), *Tornado Protection: Selecting Refuge Areas in Buildings,* FEMA P-431, Federal Emergency Management Agency, Washington, DC.

FEMA (2011), *Coastal Construction Manual: Principles and Practices of Planning, Siting, Designing, Constructing, and Maintaining Residential Buildings in Coastal Areas*, 4th Ed., FEMA P-55, Federal Emergency Management Agency, Washington, DC.

FEMA (2012), *Mitigation Assessment Team Report—Spring 2011 Tornadoes: April 25–28 and May 22; Building Performance Observations, Recommendations, and Technical Guidance,* FEMA P-908, Federal Emergency Management Agency, Washington, DC.

FEMA (2015a), *Safe Rooms for Tornados and Hurricanes: Guidance for Community and Residential Safe Rooms*, FEMA P-361, 3rd Ed., Federal Emergency Management Agency, Washington, DC.

FEMA (2015b), *Taking Shelter from the Storm: Building a Safe Room for Your Home or Business*, FEMA P-320, Federal Emergency Management Agency, Washington, DC.

FM (1989), *Approval Standard for Class 1 Insulated Steel Deck Roofs*, Approval Standard Class No. 4450, FM Global, Norwood, MA.

FM (2012), *Approval Standard for Single-Ply, Polymer-Modified Bitumen Sheet, Built-Up Roof (BUR) and Liquid Applied Roof Assemblies for Use in Class 1 and Non-combustible Roof Deck Construction*, Approval Standard Class No. 4470, FM Global, Norwood, MA.

ICC (2009a), *International Building Code*, International Code Council, Washington, DC.

ICC (2009b), *International Residential Code for One- and Two-Family Dwellings*, International Code Council, Washington, DC.

ICC (2014), *ICC/NSSA Standard for the Design and Construction of Storm Shelters*, ICC 500, International Code Council, Washington, DC.

ICC (2015), *International Building Code*, International Code Council, Washington, DC.

Kiesling, E.W. and Tanner, L.J. (2005), *Investigation of the Wind Projectile Resistance of the Nusafe Storm Shelter Panel System*, Wind Science and Engineering Research Center, Texas Tech University, Lubbock, TX.

McDonald, J.R. and Mehta, K.C. (2006), *A Recommendation for an Enhanced Fujita Scale (EF Scale)*, Wind Science and Engineering Research Center, Texas Tech University, Lubbock, TX.

NFPA (2013), *Portable Fire Extinguishers*, NFPA 10-13, National Fire Protection Association, Quincy, MA.

NWI (2017), *AISC Research Project Report*, Report No. 20160422A, National Wind Institute Debris Impact Facility, Texas Tech University, Lubbock, TX.

PCI (2010), *PCI Design Handbook*, 7th Ed., Prestressed Concrete Institute, Chicago, IL.

Roper, R. (2015), "Combined Numerical and Experimental Assessment of Steel Roof Decks Subjected to Windborne Debris," Thesis Paper 2073, Clemson University, Clemson, SC.

SDI (2014), *Floor Deck Design Manual*, Steel Deck Institute, Fox River Grove, IL.

SJI (2007), *Standard Specifications for Composite Steel Joists, Weight Tables and Bridging Tables*, 44th Ed., Steel Joist Institute, Florence, SC.

SJI (2012), *Design of Steel Joist Roofs to Resist Uplift Loads*, Technical Digest 6, Steel Joist Institute, Florence, SC.

SJI (2016), *Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders*, 44th Ed., Steel Joist Institute, Florence, SC.

UL (2002), *Standard for Fire Test of Roof Deck Constructions*, UL Standard 1256, Underwriters Laboratories, Northbrook, IL.

UL (2004), *Standard for Standard Test Methods for Fire Tests of Roof Coverings*, UL Standard 790, Underwriters Laboratories, Northbrook, IL.

UL (2011), *Standard for Fire Tests of Building Construction and Materials*, UL Standard 263, Underwriters Laboratories, Northbrook, IL.

WSERC (2003), *A Summary Report on Debris Impact Testing at Texas Tech University,* Wind Science and Engineering Research Center, Texas Tech University, Lubbock, TX.

Produced in association with:

American Iron and Steel Institute

MBMA
METAL BUILDING MANUFACTURERS ASSOCIATION®

STEEL DECK INSTITUTE sdi®

Steel Framing Alliance™

STEEL JOIST INSTITUTE SJI

Steel Tube Institute OF NORTH AMERICA

Smarter. Stronger. Steel.
American Institute of Steel Construction
312.670.2400 | www.aisc.org

D835-19