Chapter 14
Wood-Frame Residential Buildings in Windstorms: Past Performance and New Directions

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Abstract Residential buildings in coastal areas are often at risk to hurricanes, which can result in both wind and storm surge damages, while tornadoes are one of the most devastating natural hazards that have occurred in all 50 states of the USA and can happen during any season of the year. This chapter focuses on summarizing some past studies on the performance of wood-frame residential buildings in recent major hurricanes and tornadoes. Damage data collected from hurricanes shows that in most hurricanes the damage to residential wood-frame buildings often comes from high winds, hurricane surge, flooding, and rainwater intrusion due to damage in the building envelope. Roof systems experienced extensive damage either directly from wind or due to failure of the flashing and coping. Hurricanes are often accompanied by heavy rain that results in substantial water intrusion through the breached area of the building, which in turn results in substantial financial loss to the structure and its contents. On the other hand, data collected from recent tornadoes in Tuscaloosa, Joplin, and Moore show that, for an EF-4 or EF-5 tornado, damage levels increase from the outer edges toward the centerline of a tornado track. Residential building damage in tornados is caused by high wind loading or debris impact, or both. A general procedure for performance-based wind engineering is proposed, and research needs for development of wood-frame performance-based wind engineering are also highlighted in this chapter.

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14.1 Introduction

14.1.1 Wood Building Performance in Hurricanes

Over 80 % of the total building stock in the USA, and well over 90 % of the residential stock in North America, are light-frame wood (wood-frame) construction. While wood-frame construction is the most prevalent type of building, it is also the most susceptible to wind damage. This was evident during the 2004 hurricane season, when four hurricanes made landfall in the USA and became even more publicized following Hurricane Katrina in 2005 (van de Lindt et al. 2007). Although several states along the US coastline have been struck by numerous hurricanes, a number of hurricanes making landfall affect every gulf coast and eastern seaboard state, as evidenced by Fig. 14.1.

Residential buildings in coastal areas are often at risk to hurricanes, which can result in both wind and storm surge damages. While the USA averaged $1.6 billion in normalized hurricane damage annually from 1950 to 1989, this figure rose dramatically between 1989 and 1995, to approximately $6 billion annually (Pielke and Pielke 1997). The average annual normalized hurricane loss in the USA was approximately $10 billion in recent years, with more than $150 billion in total damage in 2004 and 2005 (Pielke et al. 2008). In the past decade (2005–2015), the hurricanes that hit the gulf coast and eastern seaboard of the USA—Katrina, Rita, and Sandy—have been the most devastating, resulting in widespread damage

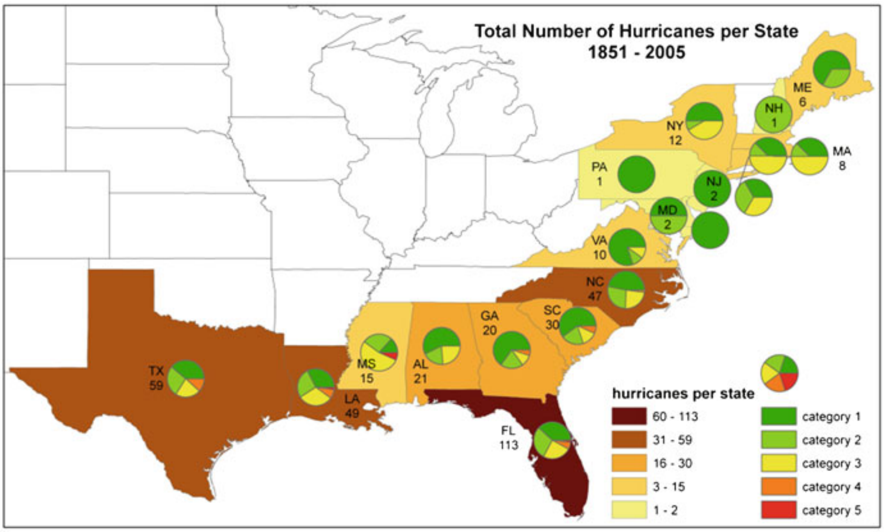

Fig. 14.1 Hurricanes making landfall in the USA (1851–2005) (excerpted from van de Lindt et al. 2007)

to residential buildings and infrastructure. Hurricane Katrina first made landfall near Buras, Louisiana, on August 29, 2005 (NIST 2006). The storm was classified at the upper end of Saffir–Simpson Category 3 by the National Hurricane Center (NHC), which estimated sustained maximum wind speeds of 125 mph, and storm surge heights up to 27 ft; less than 1 month later, Hurricane Rita hit land along the Texas–Louisiana border on September 24, 2005 (NIST 2006). This hurricane was also categorized as Category 3, with up to a 15 ft high storm surge when it made landfall. Those two hurricanes caused about 1400 fatalities and \$70–\$130 billion in damage, of which about \$45–\$65 billion was insured loss (NIST 2006). On October 22, 2012, a tropical depression formed in the southern Caribbean Sea off the coast of Nicaragua, which later became the Sandy hurricane. The hurricane then moved away from the Bahamas with winds of about 110 mph and made a turn to the northeast off the coast of Florida, having already caused about 70 or more fatalities in the Caribbean. On October 29, the hurricane made landfall at New Jersey as a huge storm, with winds covering almost a 1000 mile diameter and 11–12 ft of storm surge in some places. It affected more than 50 million people on the eastern seaboard and killed 73 people in the USA (FEMA 2015; National Geographic 2015).

Damage data collected from hurricanes (e.g., in van de Lindt et al. 2007) shows that in most hurricanes the damage to residential wood-frame buildings often comes from high winds, hurricane surge, flooding, and rainwater intrusion due to damage to the building envelope. Roof systems experienced extensive damage either directly from wind or due to failure of the flashing and coping. Roofing aggregate was responsible for the majority of wind-borne debris damage to windows (NIST 2006), which results in breach of the building envelope for structures hit with the debris. Hurricanes are often accompanied by heavy rain that results in substantial water intrusion through the breached area of the building, which in turn results in substantial financial loss to the structure and its contents (Dao and van de Lindt 2010). All of this type of damage was observed in hurricanes Katrina, Rita, and Sandy.

14.1.2 Wood Building Performance in Tornadoes

Tornadoes are found in all parts of the world, but the USA has by far the highest occurrence of tornadoes compared to any other countries. Tornadoes are one of the most devastating natural hazards that have occurred in all 50 states and can happen during any season of the year. The tornado record from 1950 to 2011, kept by the US National Oceanic and Atmospheric Administration (NOAA), documents 56,457 tornado events, of which 33,756 resulted in reported damage (Simmons et al. 2013). This means that, on average, there are about 925 documented US tornadoes annually, of which 553 cause reported damage. In particular, there were several years that saw major tornado outbreaks. In 1953, there were several outbreaks that resulted in 519 fatalities, over 500 injuries, and \$32.5 billion (2011 GDP normalized US dollars) in property and structural damage. In 1965, there were 301 people killed and $40.0 billion (2011 GDP normalized US dollars) in reported damage from tornadoes. In 1974, 348 people died, 6500 people suffered injuries, and $26.0 billion (2011 GDP normalized US dollars) in damage was done from tornadoes (Simmons et al. 2013). In 2011, there were two major tornadoes that occurred in Tuscaloosa, AL, and Joplin, MO. These tornadoes resulted in 219 fatalities with over 13,000 homes destroyed and an estimated $5 billion in damage (Prevatt et al. 2012). The losses from tornado damage continue to increase due to an increase in population and the development of the related infrastructure.

On May 20, 2013, an EF-5 tornado hit Moore, Oklahoma, and caused $1.5–$2.0 billion in damage and 24 fatalities, including ten children. This tornado was approximately 1 mile wide based on ground reports, and the damage path was very similar to the path observed after a tornado 14 years earlier (on May 3, 1999). Several schools and many residential structures were damaged or destroyed. The performance of residential structures along and across a tornado path provides a unique and useful research opportunity to document the progression of failure of wood-frame homes when exposed to tornadic winds. Comparison of the damage allows for a qualitative (and/or quantitative) analysis of the performance of residential structures under different failure progressions. A reconnaissance team of engineers from around the USA was formed and traveled to Moore 5 days after the tornado to collect and assess residential structural damage in the aftermath of the tornado. Many team members had also participated in the reconnaissance team in the aftermath of the Tuscaloosa, AL, and Joplin, MO, tornadoes in 2011.

Data collected from recent tornadoes in Tuscaloosa, Joplin, and Moore show a consistent pattern of damage to residential structures. For an EF-4 or EF-5 tornado, damage levels increase from the outer edges toward the centerline of a tornado track. In general, the progression of residential structure failure often depends on wind speed, wind direction, wind-borne debris impact, and the capacity of building components. Residential building damage in tornados is caused by high wind loading or debris impact, or both. Failure of residential buildings in tornado winds is often due to:

- Lack of continuous sheathing and fiberboard sheathing, lack of gable-end sheathing, and/or garages without anchor bolts.
- Lack of garage-wall sheathing and limited use of plywood, with most of the sheathing being fiberglass or hardboard siding.
- Observations from recent tornadoes Tuscaloosa, AL, April 27, 2011, (van de Lindt et al. 2012); Joplin, MO, May 22, 2011 (Prevatt et al. 2012) and Moore, OK, May 20, 2013, (Graettinger et al. 2014; LaFave et al. 2016) show that the structures often did not have continuous load paths from the foundation to the roof.

When a house is hit by wind-borne debris, the internal pressure changes causing progressive damage of the structure. The failure progression depends on where the debris impacts occurred, the relative location of the house to the tornado track, and the orientation of the house with respect to the tornado path.

In order to document the damage due to tornadic winds, the assessment team collected damage data in a neighborhood at the western edge of Moore, as indicated by the circle on Fig. 14.2. This fairly new neighborhood was located on Kyle Dr. and SW 151st Street and built around 2006. The buildings in this neighborhood were believed to be less affected by wind-borne debris because the tornado traveled from west to east, and the area west of this neighborhood is flat open fields with only a few residential structures that were not damaged. Therefore, it was believed that the damage to the houses in this neighborhood mostly came from tornado wind loading rather than debris impact. The houses in this area had similar structural configuration, which made it easier to identify the wind field from the damage patterns as compared to other areas.

Figure 14.3a shows the damage to residential structures on SW 151st Street looking west. This figure shows houses on both sides of the street, and one can see that all homes on the left side still have garage doors and sustained little or no damage, while homes on the right side of the street lost garage doors that blew inward. Since all the garage doors had similar design and were installed on rails inside the door opening, the lateral reinforcement bars were placed inside the door (Fig. 14.3b), and these garage doors had higher capacity when experiencing negative pressure (outward load) as compared to lower capacity with positive pressure (inward load). This is because in the outward load case the load is transferred to the wall and rails through compression, while in the inward load case, the rails failed by bending into the garage (see Fig. 14.3b). With this reasoning, it can be seen from Fig. 14.3a that the residential houses in this picture experienced mainly one wind direction as the tornado passed this street, and the garage doors on the left were on the leeward walls (negative wind pressure), while those on the right side of the street were on the windward walls (positive pressure). This behavior can also be seen on streets where the direction of the street changes, and the garage doors change from leeward to windward as shown in Fig. 14.3c. This means that when the tornado passed this area, these houses mainly experienced horizontal winds (the failure pattern was similar to straight line winds seen during hurricanes). For those houses having garage doors on the windward side in straight wind, the garage doors often fail before roof sheathing panels.

Homes on Kyle Dr., a north-south street shown in the circled area on Fig. 14.1, indicated an increase in damage when located closer to the centerline of the tornado track. This damage pattern is similar to what was observed in the aftermath of the Tuscaloosa and Joplin tornadoes. Figure 14.4 shows the damage pattern of houses on Kyle Dr., in which the camera was pointed to the south looking away from the centerline of the tornado. It should be noted that the tornado vortex was counterclockwise, and this neighborhood was on the south side of the tornado centerline. Therefore, the houses on the south edge of the damage area experienced west-to-east winds, which is right to left in Fig. 14.4. In Fig. 14.4, the houses on the right (west) side near the middle of the street had the roof fail before the garage door failed. Failure of the roof happened on the leeward side, and the roof was intact on the windward side. The garage doors, on the leeward side, were not damaged on these homes. Similar damage was observed on the roofs of the houses on the left

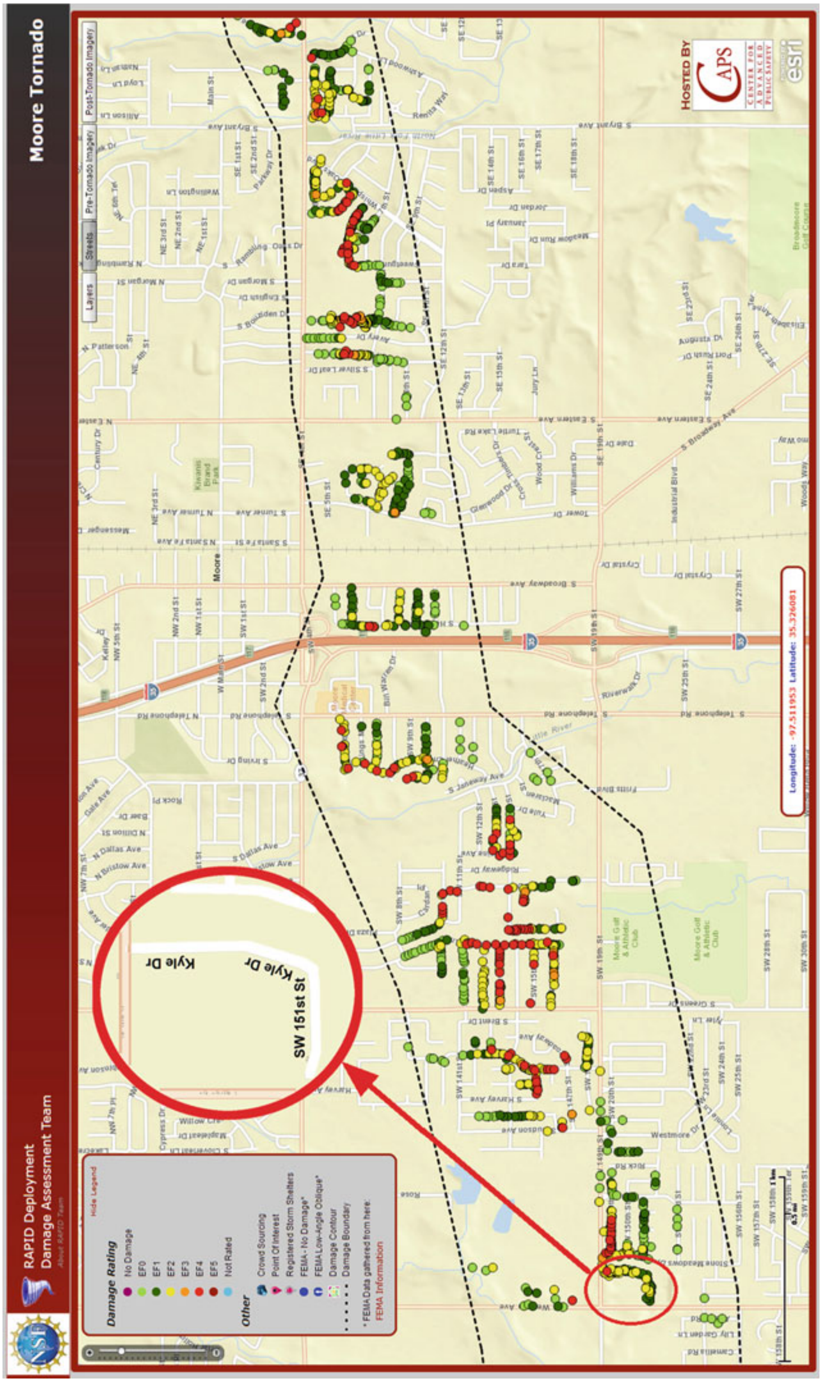

Fig. 14.2 Overview map of the Moore tornado path, with *circle* showing the neighborhood used for the detailed damage investigation (Dao et al. 2014)

Fig. 14.3 Different garage door damage levels on the same street. (**a**) Leeward (*left*) and windward (*right*) sides of SW 151st Street. (**b**) Garage door supports and configuration. (**c**) Change of garage door damage level when home orientation changes from leeward to windward direction

Fig. 14.4 Damage pattern on Kyle Dr., Moore, Oklahoma (camera pointed to the south)

(east) side of the street (failure happened on the leeward roof, but the windward roof was intact), while the garage doors on the left side of the street all failed. This is because roof sheathing panels are weaker in uplift and stronger in compression due to nail withdrawal capacity. This failure pattern indicates that these houses experienced higher wind speed than at the edge of the damage area, and mainly in one direction of horizontal wind. It can also be seen in Fig. 14.4 that garage doors and roofs failed on both sides of Kyle Dr. to the north, where the photo was taken, and several walls were collapsed in the house on the northeast end of Kyle Dr. (far left in Fig. 14.4).

Fig. 14.5 Failure of roofs and garage doors on different sides of Kyle Dr. (camera pointed to the north)

A closer inspection near the northern segment of Kyle Dr. indicated that the homes in this area experienced wind in one direction, as shown in Fig. 14.5. Again, the house on the left had roof failure and the garage door survived, while the house on the right had the garage door blown in, and only a minimal amount of sheathing panels failed.

14.1.3 Current ASCE-7 Wind Load and History of Wind Engineering

Wind-engineering studies appear to be recorded first in 1759 when Smeaton (1759–1760) attached in a small-scale windmill model to the end of a rotating arm to do research on windmill sails. Since then, wind engineering has become an integral part of hydraulic and aerospace engineering, but the first US National Conference on Wind Engineering (Roshko 1970) only occurred less than 50 years ago. It took more than a century for wind engineering to emerge as a new engineering discipline because during the period there was a lack of a reliable methodology for quantification of wind characteristics and wind effects on structures. This barrier had been removed when the application of physical modeling to wind-engineering problems was invented by Cermak (1975) when he published his 1974 ASME Freeman Lecture “Application of Fluid Mechanics to Wind Engineering.” The publication indicated the application of fluid mechanics in the development of boundary-layer wind tunnel (BLWT) which could be used to model wind loads on structures and wind flow in the boundary layer. This is the first time a reliable methodology for quantification of wind characteristics was introduced in wind engineering to study wind effects on structures. Since the invention of BLWT, the research on wind effects on structures came to a new era and led to many publications, codes, and standards; these includes wind load effects on bridges (e.g., Bienkiewicz et al. 1981; Cermak et al. 1981), buildings (e.g., Cermak and Peterka

1984; Kopp et al. 2008; ASCE-7 (American Society of Civil Engineers) 2010), and special structures (e.g., ASCE-7 2010).

Currently, the procedures for wind load calculation specified by ASCE-7 provide wind pressure and forces for the design of main wind force-resisting system (MWFRS) and for the design of components and cladding (C&C) for buildings and other structures. The procedures include the determination of wind velocity pressure, gust factor, wind directionality, and pressure or force coefficient. For the selection of pressure or force coefficient, it is often based on the results from research using BLWT. The wind load is then combined with other loads such as live load, dead load, earthquake load, snow load, etc., for use in structural design. Therefore the wind load is used in design for the purpose of life safety only, and the design procedure does not include other expectation of building performance. Also, the ASCE-7 wind load calculation procedures imply for general wind, and application to east coastal area where it was heavily affected by hurricane wind is accounted through assigning higher wind speed. And tornado wind loads are not considered in ASCE-7 wind load procedures.

14.2 Proposals for Performance-Based Wind Engineering

Performance-based design (PBD)has been defined many ways over the last decade with perhaps the most general definition being provided by Ellingwood (1998) as “an engineering approach that is based on (1) specific performance objectives and safety goals of building occupants, owners, and the public, (2) probabilistic or deterministic evaluation of hazards, and (3) quantitative evaluation of design alternatives against performance objectives; but does not prescribe specific technical solutions.”

In the USA, performance-based design has been focused primarily on seismic, fire, and manufacturing engineering. PBD is, by and large, felt by most to be a system-level philosophy that allows inclusion of system-level behavior including the improvement in performance as a result of this assertion. However, in wind engineering most failures are understood to be at the component and subassembly level. A recent paper by Ellingwood et al. (2006) highlights the current status and future challenges for PBD for wood including performance-based wind engineering (PBWE). In that paper it was stated that guidelines for PBWE do not currently exist in the USA. It was also stated that extreme winds (with the exception of tornadic winds) are not viewed as a life safety issue in force-based design primarily because of the opportunity for prior warning, which is not true for earthquakes and fire. Thus, the parallel with these other hazards stops at the life safety performance expectation to some degree. Finally, it was articulated in Ellingwood et al. (2006) that models are needed which model both load and non-load-bearing walls as an integrated system.

14.2.1 Proposed Performance-Based Wind-Engineering Procedure for Hurricane Wind

The proposed PBWE procedure is an extension of the fragility studies outlined earlier (Rosowsky and Ellingwood 2002; Ellingwood et al. 2004). However, there are two distinct differences, namely, in the present study the focus is spread over four of the five performance descriptors listed in Table 14.1 and a system-level finite element model is used to more accurately address structural instability issues related to collapse. Returning to Table 14.1, five performance descriptors are proposed (van de Lindt et al. 2007). To date, only two of these have been addressed in previous studies: continued occupancy and life safety. Continued occupancy is assumed herein to correspond to loss of the first sheathing panel which is consistent with previous studies. Figure 14.6 shows a photograph of what, at first inspection, looks like a moderate gable opening during Hurricane Katrina. The owner was not able to remain in the structure following the hurricane, and according to the owner the

Table 14.1 Performance expectations and related model damage parameters for PBWE of wood

Performance expectation	Performance description	Model damage parameter	Study addressing issue
Occupant comfort	Little or no reduction in living/inhabitant comfort	Almost a durability issue; no damage or water entry limited to moisture, i.e., no pooling	Present study
Continued occupancy	Up to moderate reduction in comfort but no threat to safety or injury. Electrical, plumbing, and egress still present	Loss of first gable or roof sheathing panel	Lee and Rosowsky (2005); Ellingwood et al. (2004); present study
Life safety	Structural integrity is questionable; significant risk of serious injury might occur; safety normally provided is not present	Roof truss-to-wall connection failure; supporting column/post failure	Ellingwood et al. (2004); present study
Structural integrity	Visible signs of structural distress, i.e., permanent deformation and structure not safe	Collapse of roof; loss of lateral capacity	Present study
Manageable loss	Cost to repair structure is below a selected percentage of reconstruction/replacement value. This is dependent on numerous factors and is often the result of rainwater intrusion	Loss fragility based on the assembly of damaged components	Not addressed, but would likely require assembly-based vulnerability or other method

Fig. 14.6 Gable-end damage during Hurricane Katrina

2005 insurance estimate was equal to the 1998 purchase price of the home (van de Lindt et al. 2007). If this structure had not been insured for wind (and subsequent water) damage, this would most likely exceed what can be called "manageable loss" for most homeowners. Manageable loss is beyond the scope of the work presented herein, but can best be explained as the upper limit of the cost that a homeowner can (or is willing) to pay (whether borrowed or out of pocket) to be able to live in the structure comfortably. In Table 14.1 this is indicated as some percentage of the reconstruction/replacement value for repair. The concept of continued occupancy refers to the owner's ability to inhabit the dwelling following the event.

Life safety is perhaps the most difficult to define, but is summarized here as being a condition in which the safety normally afforded by a structure is no longer present. For wind damage, this can be characterized as failure of the roof-to-wall connection or supporting column/post failure. Figure 14.7 shows the collapse of a porch overhang as a result of poor (or no) anchorage during Hurricane Katrina. The life safety issue in this case arises from the fact that the joists frame back into the ceiling of the first level and failure then occurred within the living portion of the structure.

Two additional performance expectations that have not been explicitly addressed to date are as follows. Structural integrity, which can be summarized as the state at which the structure shows significant signs of distress, may include the collapse of the roof or the loss of lateral capacity either locally or globally. Although the general consensus is that complete loss of lateral capacity from wind load is rare, it is possible as evidenced by Fig. 14.8. This is a convenience store in

Fig. 14.7 Loss of a porch overhang due to lack of support post anchorage

Fig. 14.8 Collapse of a wood-frame (metal clad) building

Mississippi that was literally blown over in Hurricane Katrina. van de Lindt et al. (2007) describe this failure with the following sequence: The roof uplifted and there was a loss of roof sheathing. The front glass window “blew out” and the roof trusses collapsed. The trusses were tied to the walls with hurricane clips, but without roof sheathing they did not provide lateral stability for the trusses alone. The structural instability performance expectation includes life safety, meaning none of the performance expectations are necessarily mutually exclusive. For example, if the structure collapses, i.e., does not meet the expectation of structural integrity, clearly all of the other expectations were not met, albeit they may be tied to different hazard intensities. The performance expectation of life safety may not be met even when there is no local or global collapse.

The other performance expectation which has not been addressed is occupant comfort. This is intended to mean following the event since it is not anticipated that the homeowner would necessarily be present during a hurricane. In the present study, it is proposed to model this as water penetration at roof sheathing edges resulting in potential mold and other issues related to moisture. A detailed finite element model, which utilizes a new 6-DOF fastener model developed by Dao and van de Lindt (2008), is used to detect/model sheathing uplift and help develop fragilities as a function of edge uplift.

14.2.1.1 Performance Expectations

Consider Fig. 14.9 whose concept is adopted from performance-based seismic design. Current force-based design utilizes a single peak 3-second gust and designs with some level of safety or with both a load and resistance factor (e.g., ASCE 16 1996). In Fig. 14.9 the leftmost line corresponds most closely to current force-based design values. However, it is important to note that simply by defining multiple performance objectives the design philosophy is no longer the same. For this leftmost line, returning to the performance expectations and damage parameters in Table 14.1, and for a peak gust of 90 mph (145 km/h), an owner would expect no damage and no water intrusion. For a well-designed and well-constructed residential structure, this is typically the case provided wood is in a non-decaying state and fasteners are spaced appropriately. For the same leftmost line (squares in Fig. 14.9), one would expect to provide life safety at 170 mph (270 km/h) meaning no loss of truss-to-wall connections or supporting post failures. Although this is the performance expectation described here, the method described below is probabilistic, and thus there is always some probability of exceeding such an expectation, as examined by Ellingwood et al (2004). Therefore some level of exceedance probability must be selected, which for the present study is set at 50 % for illustrative purposes.

The concept of PBWE can be further explained by again returning to Fig. 14.9. Now, focusing on the rightmost line with triangles, one can see that no water intrusion or damage would be expected at 130 mph (210 km/h), life safety expected at 210 mph (335 km/h), and structural integrity expected at 250 mph (400 km/h).

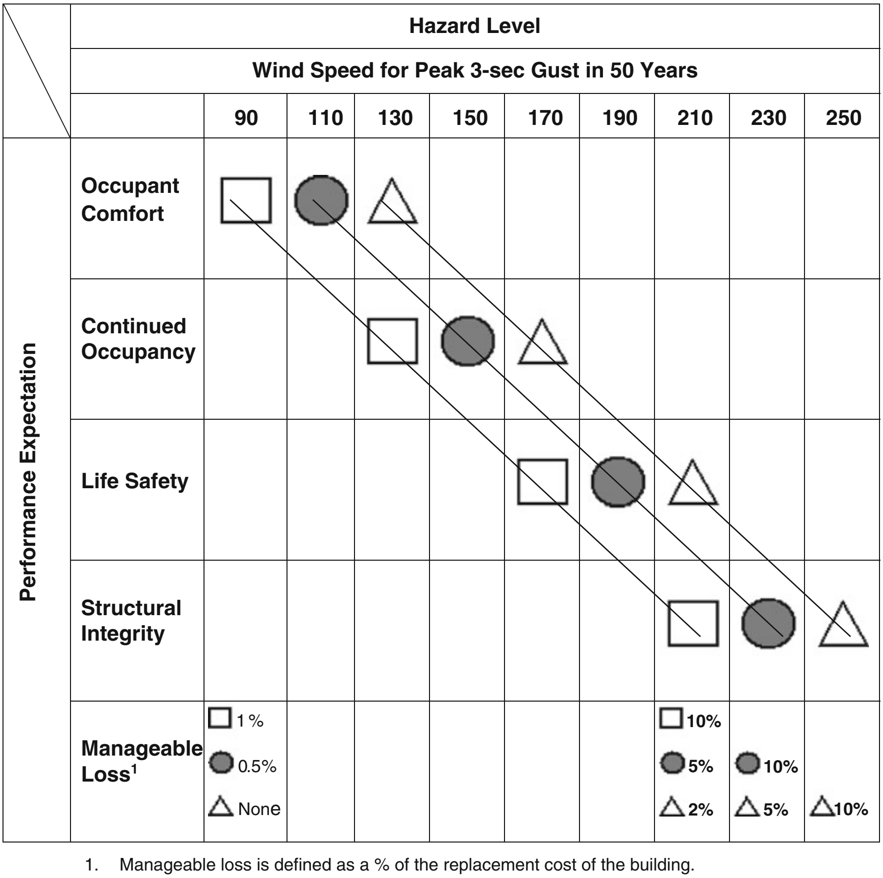

Fig. 14.9 Example of various levels of building performance as a function of hazard level. [1]Manageable loss is defined as a percentage of the replacement cost of the building

Of course, it should be noted that at wind speeds this high, debris acting as airborne missiles will ultimately have to be considered in performance-based design but is not here. The force exerted by the debris is understood, but unfortunately little beyond speculation is available for occurrence modeling since it is related to many things beyond the engineer's control, e.g., equipment left out in the open. Several studies have examined this concept with some recent work being completed by Lin and Vanmarcke (2008). Finally, recall that the pressure varies as the square of the wind velocity, so although the various performance expectations are linear when expressed as pressures and subsequently in terms of strength requirements, the force exhibited by these wind speeds increases substantially from occupant comfort to even continued occupancy.

Perhaps the most important aspect of PBWE will eventually be addressing manageable loss through modeling and detailed comparison of structural performance to estimated losses during high-wind events. For example, in Fig. 14.9 how would one ensure that at 210 mph the performance expectation level for the leftmost line (indicated by squares) only has losses not to exceed 10 % of the replacement value of the structure? Further, for the "triangle" performance level, these would not be expected to exceed 5 % at 210 mph. As mentioned earlier, this is not quantitatively addressed in this chapter. To accurately assess the damage in terms of dollars would require the full inclusion of damage due to wind-borne debris and a mechanism to assess volume and affect of rainwater entry. This would likely include an approach such as assembly-based vulnerability (Porter et al. 2001; Pei and van de Lindt 2009; Dao and van de Lindt 2012; van de Lindt and Dao 2012).

14.2.1.2 Additional Considerations

Articulating, or quantifying, the performance expectations of a peak 3-second gust in 50 years does not address other "failure" mechanisms that may occur during a hurricane as a result of the duration (sometimes in excess of 8 h). Figure 14.10 shows a photograph of a hurricane clip that lasted almost 4 h during Hurricane Katrina and finally failed (van de Lindt et al. 2007). Another factor is roof coverings and siding, which are not designed to carry wind load, but are envisioned to protect the structural components such as paneling from direct water exposure during storms, thus helping to maintain the integrity of the building envelope. In this study, the nonstructural

Fig. 14.10 Photo of missing fasteners in a hurricane clip. The roof lifted off after several hours of uplift pressures

siding and roof coverings are assumed to have been removed by the wind prior to the analysis performed on the wood components and assemblies. Finally, although considered beyond the scope of the current study, it is again stressed that it is imperative that PBWE ultimately considers projectiles and breakage of windows for an accurate assessment of risk caused by wind events.

14.2.2 Needs for PBWE of Wood-Frame Buildings

Many recent studies have focused on a framework for PBWE (e.g., van de Lindt and Dao 2009; Spence et al. 2015; McCullough and Kareem 2011; Griffis et al. 2012; Muthukumar et al. 2012; Unnikrishnan et al. 2012), but none have yet to fully develop and propose a methodology that leads close enough to code adoption. In most of the frameworks proposed, the fragility methodology was used to measure the performance levels either at the component or system levels of a building subjected to high wind. The construction of component or system fragility was often based on either reliability theory or Monte Carlo simulation for the loads and system resistance. A statistical approach is suitable for the frameworks with current available technologies in wind engineering and can be used for insurance policy decisions. However, for engineering practice it is likely that PBWE for wood-frame buildings will utilize a semi-prescriptive approach with certain design decisions achieving some level of statistical performance level.

To make PBWE for wood-frame buildings a viable option in engineering practice and effectively move beyond merely the applications of statistics and reliability theory, understanding building performance in high-wind events is essential. The data collected from high-wind events gives an overview of building performance for these, often vulnerable buildings to wind load, and the damage patterns in high-wind events allow forensic investigation into the failure mechanisms. However, the local wind speeds, quality of building construction, specific design of each individual building are often neglected in the collected data; in fact, they are often unknown once the damage has occurred. This is because the large amount of data needs to be collected in a short period before the sites are cleaned to begin the recovery process for a community. This is the reason most current wind load standards or codes are often based on experimental results from wind tunnel experiments using rigid-body models. Currently, wind load simulation is based on a wind tunnel test either for small- or large-scale structures. For small-scale wind tunnel tests, the wind load is often measured on a scaled solid model based on similitude laws in aerodynamics and fluid mechanics. A small model allows researchers to simulate the effects of terrain roughness and aerodynamic behavior around the tested structures for a relatively low cost compared with that for large-scale tests. But because the model is solid, the building performance under the wind load cannot be properly evaluated. For these reasons, several large facilities for wind load simulation of large- or full-scale structures have been recently built. Even though the development of large-scale wind load simulation helps to improve the understanding the performance of buildings under the load, it has some drawbacks:

(1) The construction of large-scale wind load simulation laboratories is expensive.
(2) The cost of experiments is often very high due to the cost of constructing large-scale structural models as well as the cost of operating the tests.
(3) It is difficult and sometimes impossible to model the effects of aerodynamic behavior around the prototype from adjacent structures in a large-scale wind load simulation laboratory; therefore the effects of wind turbulence due to the vortex in the built environment cannot be modeled properly, thereby making it difficult to evaluate wood-frame building performance in high-wind events with the effects from surrounding buildings in a community (including wind-borne debris impact).

14.3 Research Needs for Development of Wood-Frame PBWE

In order to develop PBWE for wood-frame building, the knowledge of how these types of buildings perform in high-wind events needs to be investigated further. This includes understanding of the wind loading and the building response in high wind, including both structural and nonstructural components such as the building envelope, finishing products, and building contents. As mentioned, atmospheric boundary-layer wind tunnels can be used to model the wind environment in the boundary layer properly, but cannot model the building performance accurately. This is because they do not account for the nonlinear behaviors of building materials and structures. To solve these drawbacks mentioned earlier, it will be necessary for the wind-engineering wood-frame building research community to develop a new testing method that includes both the wind environment and building behaviors in high wind.

In order to be able to estimate the performance of building structures within an extreme wind field, the mechanics behavior of the interaction between the high-wind field and structures needs to be explored numerically and then validated with these experimental studies. Although there has been a significant volume of studies focusing on numerical fluid–structure interaction (FSI) problems over the last several decades, a comprehensive study of such problems remains a challenge due to the strong nonlinearity of the problem and multidisciplinary approach required for a solution (Hou et al. 2012). For this reason, purely analytical solutions to the model equations for the interaction between high wind and structures are likely impossible to obtain. Recently, some limited laboratory studies (Haan et al. 2010) have focused on the pressure coefficients on a rigid structural model in which the interaction between the extreme wind field and structural model was not included. Thus, to investigate the fundamental physics involved in the complex interaction between fluids and structures, numerical methods should also be employed (Hou et al. 2012).

This may require an algorithm to model the interaction between the extreme wind field and structural dynamics.

After the numerical modeling and experimental tools are available, these can be applied into one of the PBWE frameworks that have been developed recently. This process may take several generations of PBWE before it can comprehensively be applied in engineering practice.

14.4 Closure

PBWE for wood-frame buildings is a relatively new topic in both research and design and owes its impetus to damaging hurricanes over the last 15 years and supercell tornado outbreaks across the Central and Southern USA. It is clear that PBWE will not take the same format as design for heavier and stiffer buildings made of steel frames and/or concrete walls/frames. Those types of buildings typically keep their structural system intact but lose windows, covering, and curtain walls, whereas wood-frame buildings suffer loss of components and cladding and their main wind force resisting system. So, while the underlying hybrid research required will be robust, it will be necessary to use a semi-prescriptive approach to truly implement PBWE in wood-frame design practice.

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