A Review of Wood-Frame Low-Rise Building Performance Study under Hurricane Winds

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Abstract:

The unsatisfying performance of light-frame low-rise buildings under hurricane winds is a long-standing issue. Most past research effort has been made in creating a wind pressure database using advanced testing facilities, studying “cutout” subassemblies (i.e., roof subassemblies/sheathed shear walls) without considering the imparted influences of the whole system, analyzing load transfer mechanisms within full building models in the linear range, and quantifying building performance for the purpose of estimating economic losses. What still remains unclear is the contribution of some key factors, such as the simplified spatial-temporal varying wind loads, the resolution of numerical models, and the building component capacities, with large uncertainties, to the ultimate building performance. This paper aims to review the state-of-the-art research of four related disciplines that may contribute to a rational building performance assessment from an engineering perspective. Hurricane hazards, public wind pressure databases, and design wind loads are reviewed to indicate the importance of parameters other than wind speed to the building performance. The evolution of the low-rise building FE modeling is summarized with an emphasis on the connection modeling as well as their comparisons with experimental measurements. The quantification of the building performance is discussed by using stochastic finite element modeling techniques. At last, areas requiring future studies are identified to fill knowledge gaps.

Keywords: Low-rise; Buildings; Light frame; Wind loads; Hurricane

1. Introduction

Low-rise residential buildings have performed unsatisfactorily during past hurricane events. Seven out of the ten most costly catastrophes in terms of insured U.S. properties through 2014 were hurricane-induced, as summarized in Table 1 in the descending order of loss **Error! Reference source not found.**, which excludes flood damage covered by the federally administered National Flood Insurance Program. In general, low-rise residential buildings have not received rigorous engineering design like tall buildings have. In ASCE 7 [2], low-rise residential buildings are defined as having a mean roof height, h , less than or equal to 18 m (60 ft.) and a height not exceeding the least horizontal dimension. Traditional dwelling construction practices generally deliver reliable building performance under gravity loads. However, the inadequate considerations on the structural performance under non-gravity loads, such as wind/wave uplifts and horizontal earthquake shaking forces, result in large differences in structural integrity. These defects put the ever-growing building stock along the hurricane-prone coastlines vulnerable to abrupt climate changes. In 2013, a total of 61,678,940 countrywide homeowner package policies were written to cover residential building damage and contents loss according to NAIC **Error! Reference source not found.** With this large number in mind, it is not surprising to see in Table 1 that the insured property loss caused by Hurricane Katrina in 2005 was twice as much as that caused by 911 terrorist attacks on World Trade Center in 2001.

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Table 1: Ten most costly insured property U.S. catastrophes through 2014

Rank	Date	Peril	Dollars when occurred (millions)	In 2014 Dollars (millions)
1	Aug.2005	Hurricane Katrina	41,100	48,383
2	Sep.2001	Terrorist Attacks on World Trade Center and Pentagon	18,779	24,279
3	Aug.1992	Hurricane Andrew	15,500	23,785
4	Oct.2012	Hurricane Sandy	18,750	19,307
5	Jan.1994	Earthquake in Northridge, CA	12,500	18,345
6	Sep.2008	Hurricane Ike	12,500	13,639
7	Oct.2005	Hurricane Wilma	10,300	12,125
8	Aug.2004	Hurricane Charley	7,475	9,083
9	Sep.2004	Hurricane Ivan	7,110	8,639
10	Apr.2011	Tornado in Tuscaloosa, AL	7,300	7,652

The studies performed in the past on low-rise building performance under hurricanes fall into four major categories: (1) hurricane catastrophe models using little or empirical structural analysis for economic loss prediction purposes [3]; (2) deterministic finite element analysis of different modeling scopes, including the component level, the subassembly level [6], and the whole building level [15]; (3) probabilistic building performance assessment at the component level, i.e., a piece of roof sheathing [20]; and (4) direct building tests under natural wind [18] and under wind pressures replicated from wind tunnel measurements [23]. Until now, no study has been done on the probabilistic building performance assessment at the system level to account for both uncertainties in wind loads and structural resistances, which would provide the most comprehensive information on how to improve building performance. Fortunately, this goal could be achieved by combining the strengths of each involved discipline, as briefed in the following sections.

Modern hurricane catastrophe models contribute to integrate the structural damage assessment into a probabilistic framework by using vulnerability curves to quantify the extent of structural damage as illustrated in Fig. 1. After Hurricane Andrew in 1992, the structural performance assessment was included to help lay down a foundation for the loss estimation over the high wind speed range where the loss rate escalates, but the claim data is usually not sufficient for reliable regressions without large errors. In catastrophe models, the structural performance under hurricane winds is measured by the damage ratio, i.e., the replacement cost divided by the property cost, over a wind speed range. The vulnerability curve is generated using the mean percent damage value from the damage distribution at each wind speed [25] as illustrated in Fig. 1. Without the subjective segregation of the damage level as used in the qualitative damage matrix, i.e., “no damage,” “moderate damage,” and “severe damage” [8], vulnerability curves provide sufficient resolution to quantify damage or to evaluate structural degradation and beneficial structural upgrades. However, the structural representations of low-rise buildings by catastrophe models are focused on reflecting the structural damages as sufficiently as possible for the economy loss estimation rather than investigating the root causes of structural failures. For example, the building system is empirically simplified into several 2D super elements, i.e., a piece of shear wall with predefined constant load sharing in the Hazus®-MH and the Florida Public Hurricane Loss Model (FPHLM) **Error! Reference source not found.** This approach reflects failure modes consistent with post-disaster investigations, but it is limited because it cannot reveal the weakest load path nor the damage propagation.

Fig. 1. Structural vulnerability assessment included as one component of catastrophe models after Hurricane Andrew 1992 and an illustration of typical vulnerability curve

Available deterministic static analysis of a truss assembly by Cramer et al. [13] found that a system does impart influence into a single representative 2D truss, which indicates that the aforementioned assumed fixed load sharing might not be appropriate even under static loads. Alternatively, the load transfer mechanisms within low-rise residential buildings could be captured rigorously by finite element modeling techniques based on mechanical principles. The building structure will be represented at a selected resolution varying in complexity and accuracy, especially for the inter-component connections that govern the ultimate building performance in a nonlinear manner. Finite element models yield verifiable structural responses, such as the peak global responses, which were compared with experiments set up in a controlled manner in the past [18]. Also, the influence of the uncertainties inherent in hurricane-induced loads and structure capacities on the building performance, which is difficult to investigate by the full-scale testing, could be evaluated affordably by using the probabilistic finite element analysis.

Meanwhile, the concurrent development of experimental and computational aspects opened a new avenue for analytical tools that directly uses the pressure time histories measured through a large number of pressure taps in wind tunnels. Currently, the design tools assisted by the wind tunnel database, i.e., WILDE-LRS [30], the NIST wind PRESSURE [31], and DEDM-LR [32], have been developed to evaluate wind-induced responses of the main wind force resisting system (MWFRS) using influence coefficients. In the near future, this direct access to wind pressure time histories will facilitate the numerical investigations on structural stiffness degradation under fluctuating wind loads for building envelope whose poor performance leads to consequent contents damage as well as windborne debris formation. The manifested structural damage reported during the extended duration can be studied quantitatively by using wind loading time histories. For example, Hurricane Wilma lasted two days and caused at least four times greater losses than Hurricane Emily which lasted six hours despite the fact that both of them were Category 4 hurricanes that landed and decayed along similar paths [33]. So far, no damage accumulation due to extended hurricane duration is considered by current catastrophe models or design guidelines.

The purpose of this paper is to review the state-of-the-art research from multiple disciplines that will benefit a probabilistic building performance assessment from an engineering perspective, including (1) hurricane hazard modeling (hurricane tracks and intensity modeling), (2) wind-structure interaction, (3) building representation and validation, and (4) building performance assessment by structural response criteria. The sequences of this multi-disciplinary effort along with the associated outcome examples from each step of the study are illustrated in a streamflow diagram shown in Fig. 2.

Fig. 2. Streamflow Diagram Showing Components Contributing to Probabilistic Building Performance Assessment under Hurricane Wind Loads

2. Hurricane Hazard Modeling

The hurricane hazard modeling is reviewed first to understand the hurricane life cycle and the key parameters that the building performance is sensitive to in the upstream meteorology aspect. Currently, most hurricane hazard models are developed based on the historical records for tropical cyclones and hurricanes in the Atlantic tropical cyclone basin, known as HURDAT, which dates as far back as 1851. Due to the inadequacy of records, physical flow motion models have been developed since the 1970's [33] to model hurricane hazards in conjunction with the Probabilistic Distribution Functions (PDFs) of key atmospheric parameters derived from the HURDAT database. The atmospheric component in the Florida Public Hurricane Loss Model (FPHLM) is briefed here as a representative of similar versions developed in the past. The wind hazard maps associated with certain return periods specified in ASCE 7-10 [2] as the only basic wind design parameter to guide new designs are also reviewed.

2.1 Atmospheric Component in FPHLM

The atmospheric component in FPHLM is mainly briefed based on the work of Powell et al. [35]. At the beginning of the life cycle of hurricanes, the expected number of storms that form or first appear within a certain geographic domain is determined for a given year, using the PDF of annual occurrence rate derived from the HURDAT data. Then, for each expected tropical cyclone, the genesis time and location are computed based on the empirically fitted historical seasonal genesis frequency. In parallel, the discrete PDFs of the changes in each atmospheric quantity of interest (translation speed, direction, central pressure) are developed for each 0.5° latitude/longitude box region at a given current status. For example, the PDF of the change in the heading direction is developed for a given location and a given heading direction. Following the storm genesis, the subsequent motion and intensity evolution of a storm are determined by the repeated sampling via Monte Carlo simulations from those PDFs of changes in atmospheric quantities. Particularly, the intensity is first expressed as the gradient of pressure, the difference between the central minimum sea level pressure and an outer peripheral pressure, when the storm is initiated, and later is defined according to the maximum surface wind speed at a 10 m height. To do so, a mean atmospheric boundary layer (ABL) motion field at 500 m height, or vertically averaged over the height of the ABL of 1000 meter thickness, is set up to represent a steady circular flow balanced by the inward-directed pressure gradient forces and the outward Coriolis and centripetal accelerations. Mathematically, the wind field is modeled by two-dimensional time-dependent governing equations below:

$$u\partial u\partial r - v^2/r - fv + vr\partial u\partial\phi + \partial p\partial r - K\nabla^2 u - ur^2 - 2r^2\partial u\partial\phi + F_c, u=0 = \partial u\partial t \quad ((1))$$

$$u(\partial v \partial r + v r) + f u + v r \partial v \partial \phi - K \nabla^2 v - v r^2 - 2 r^2 \partial u \partial \phi + F_{c,u} v = 0 = \partial v \partial t \quad ((2))$$

where u and v are the respective radial and tangential wind components relative to the moving storm, p is the sea-level pressure which varies with radius (r), f is the Coriolis parameter which varies with latitude, ϕ is the azimuthal coordinate, K is the eddy diffusion coefficient, and $F_{c,u}$ and $F_{c,v}$ are frictional drag terms. Again, all terms are assumed to be representative of means through ABL. Once the mean ABL motion field is determined, the tangential and radial wind speeds will be scaled down from the mean height of ABL to the surface wind for marine exposure at 10 m height, and further converted to open terrain over land considering the fetch-dependent roughness. Since this review is focused on the structural engineering aspects, the many other components of the hurricane hazard model, such as genesis model, tracking model, intensity model, r_{max} model, and filling model are not given here for brevity.

2.2 ASCE 7-10 Wind Hazard Map

The wind hazard in ASCE 7-10 [2] is expressed by basic design wind speeds, i.e., 3-second gust speeds at 10 m (33 ft) above the ground for exposure category C (open terrain with scattered obstructions having heights less than 30 ft or 9.1 m). The basic design wind speed maps specified for non-hurricane prone regions were prepared from peak gust data collected at 485 weather stations where at least five years of data were available. While for hurricane prone regions, wind speed maps are based on the results of a Monte Carlo simulation model developed and updated by Vickery et al. [36]. The dividing wind speed between two regions is 54 m/s (115 mph) for Risk Category II buildings along the Atlantic Ocean and Gulf of Mexico coasts. The hurricane risk estimation techniques used to form the wind speed maps are similar to the approach used by the FPHLM briefed above. The model was updated by incorporating new hurricane data measured from 2000 to 2009, which led to lower wind speeds given in ASCE 7-10 [2] than in ASCE 7-05 [39]. It was found that about 70% of the overall wind speed modeling uncertainty is controlled by the modeling Holland B parameter and central pressure [37].

The wind speed maps specified in ASCE 7-10 [2] correspond to a wind load factor, WLF , of 1.0, which departs from prior editions that used a single map with an importance factor and a load factor of 1.6 and thus brings wind specification in line with seismic cases by eliminating the use of a load factor for strength design. Consequently, the associated return periods with ASCE 7-10 [2] wind speed maps are determined in accordance with ASCE 7-05 [39] and earlier editions, wherein the wind pressures were determined using wind speed maps, importance factors, and wind load factors to achieve appropriate pressures for strength design. As a result, the mean return intervals for Risk Category I, II, III, and IV buildings in ASCE 7-10 [2] are 300 years, 700 years, 1700, and 1700 years, respectively.

There are several factors that serve as multipliers to determine the velocity pressure, q_z , for design purpose to cover issues not represented by wind speed maps. For example, in SI unit, $q_z = 0.613 K_z K_{zt} K_d V^2 (N/m^2)$. Since the wind may come from any horizontal direction, the wind directionality factor, K_d , (i.e., taken as 0.85 for buildings) is used to account for the reduced probability of maximum winds coming from any given direction and the reduced probability of the maximum pressure coefficient occurring for any given wind direction. Another factor, the topographic factor K_{zt} , is used to consider the effects of significant topographic features, such as escarpments, ridges, or hills, that are not reflected in the basic design wind speed maps because those maps were developed based on data measured at open-country exposure, i.e., airports and similar. The velocity pressure exposure coefficient, K_z , accounts for the variation in the wind speed with the variation in the height for a specific exposure. To determine wind loads, the velocity pressure, q_z , needs to be further multiplied with the pressure coefficient and the gust effect factor, G , which accounts for the loading effects in the along-wind direction due to wind turbulence-structure interaction as well as possible dynamic amplification for flexible structures.

3. Wind-Structure Interactions

3.1 ABL Simulation Methods

Wind speeds are turbulent (or gusty) by nature where the highly turbulent part is located at the lowest part of ABL where low-rise buildings are immersed. The resulting wind pressures are also temporally fluctuating by reason of not only those fluctuations in the upwind turbulent velocity but also the local vortex shedding in separated flow regions (i.e., sudden geometry changes near sharp corners, roof ridges and eaves) [40]. The quantification of

surface wind pressures requires the ABL reproduction that is currently simulated by boundary layer wind tunnels (BLWTs), computational fluid dynamics (CFD), or open-jet facilities.

The advent of wind tunnel technologies dates back to 1871 with the pioneering work of Wenham as summarized by Baker [41]. Wind tunnels were not extensively used to measure pressures on low-rise buildings until the 1950s when Jensen [42] fully established the similarities of using a turbulent boundary layer to obtain pressure coefficients in agreement with full-scale values [40]. Besides Jensen number (H/z_0 , the non-dimensional ratio of building height to roughness length), other criteria used to justify the BLWT simulation to the desired type of terrain include the mean velocity profile, the turbulence intensity profile, and the velocity spectrum profile. Those agreements could be achieved through a series of wind tunnel setups, including the straightener (i.e., settling room and honeycomb), the accelerator (i.e., contraction part), the full turbulence developer (i.e., long test section), and the flow conditioner (i.e., trip boards, spires, and roughness elements) [44].

To avoid a matchbox-sized scaled low-rise building model, when the entire ABL is reproduced in the wind tunnel (about 1,000 m deep), only the surface layer (about 100 m) is typically simulated to allow for the use of large scale models for low-rise buildings, for example, at least 1:50 order as suggested by Tieleman [44]. However, this solution, in turn, creates a mismatched smaller integral length scale at the wind tunnel with the full-scale test due to the missing of large size eddies associated with low frequency in the velocity spectrum. The consequent influences on the pressure coefficient reproduced in wind tunnels was found insignificant if the relaxation of the integral length scale can be controlled within 20% of the target value [43]. Regarding the issue of a lower Reynolds number (Re) due to scaling effects, a consensus has been achieved that the Reynolds number similarity can be relaxed at large values with certain thresholds given in the literature, i.e., $Re \geq 50,000$ proposed by Tieleman [44] and $Re > 10,000$ as suggested by ASCE 7 (1999). Under those conditions, the distortions of the flow field and separation points produced on the sharp edge building model are regarded as independent of the Reynolds number.

Comparatively, CFD is not limited by the incompatible similarity issues due to its ability to perform full-scale simulations, and it is able to provide data on every grid point of the meshed computational domain (“the whole-flow field data”) with better visualization results [45]. Parametric studies could be performed in the CFD to evaluate the effects of different building configurations to the flow field in a convenient way [46]. However, challenges remain in the simulation of the high turbulence, the high Reynolds number, the 3D flow field, and bluff bodies with the associated flow separation and vortex shedding [47], and it is computationally costly to derive peak pressure values [48]. The results from the CFD simulations are very sensitive to a wide range of parameters that have to be set by users, such as the approximate form of the governing equations, the turbulence model, the computational domain, the computational mesh, the boundary conditions, and the convergence criteria [45]. Thus, additional parallel studies are needed to bolster the credibility of its results [72].

The open-jet facility is a relatively new way to perform wind tests, as an alternative to traditional wind tunnel, with advantages including that the blockage effects are minimized, the destructive testing under strong winds is allowed, and the Reynolds number is greatly improved. As a promising ABL simulator, the open-jet concept has been built in recent decades to perform wind driven rain full-scale tests, such as the testing done by Hangan [49]. Two examples of those open-jet facilities are the Wall of Wind (WOW) located at the International Hurricane Research Center (IHRC) at Florida International University (FIU) [50] and the one located at the Insurance Institute for Business and Home Safety (IBHS) Research Center in South Carolina [51]. The limitations in the ABL created by open-jets (such as the lack of large-scale turbulence similar to the wind tunnel, the overabundance of small-scale turbulence without flow conditioners, and the short test section that is not long enough for turbulence to grow [48]) could be mitigated by using planks as passive devices and active controls designed based on wind data obtained through Florida Coastal Monitoring Program (FCMP) [171].

3.2 Wind Load Characterization

The wind characteristics of external and internal pressures are determined by the flow-structure interaction, where the vast majority of past research efforts has been focused on. The building geometry and the surrounding configurations are found to be the two major factors influencing surface pressures.

The key parameters that affect the magnitude and distribution of external wind pressures are categorized in Table 2 along with related studies, as well as their study outcomes. Roof shapes and terrains are the most influential variables to external wind pressures acting on the roof structures. For a given storm, higher pressures are generated on the gable roof rather than on the hip roof with the other parameters staying the same [52]. The effect of terrain roughness includes two folds: (1) the mean roof height reference velocity is reduced in a rougher terrain; and (2) the turbulence intensity will increase in a rougher terrain. The magnitude of the pressure coefficient, especially the peak pressure coefficient, is higher in a rougher terrain. However, it does not necessarily imply that the peak wind loads

themselves are larger than those for the smoother terrain [101]. Experimentally, it was found that the higher local wind loads on a sheathing panel (averaged by square/rectangular-shaped area, say 4×4 taps) are produced in a smoother open terrain, due to the increased peak wind speeds, rather than a suburban terrain [52]. The overall peak vertical uplifts and horizontal thrusts that St. Pierre et al. measured from open country ($z_0=0.03m$) were found to be about the same as those from suburban ($z_0=0.3m$) because the decrease in the mean roof height speed is offset by the increased turbulence level [53]. In Table 2, consensus has been achieved on the effects of most of these parameters such as the roof shape, the roof pitch, and the building geometry, while different opinions exist on whether surrounding structures cause enhancement or shielding effects due to the cancellation of various factors. Each of the affecting parameters has been studied including the ratio of building height to mean spacing (e.g., [54]), the interference factor (IF) (e.g., [55]), and the building area density (e.g., [56]). Gavanski et al. [52] categorically evaluated such combined actions by comparing measured external pressure coefficient contours from 87 building configurations and concluded that the effect of surrounding structures for design can be neglected. In other words, roof loads of a single, isolated building are adequate and effective. Of the literature listed in Table 2, the pressure contours on the roof measured by Holmes [57] were frequently compared for the calibration of the wind tunnel measurements [52]. The external pressure coefficients and velocity profiles measured at the University of Western Ontario (UWO) [59] was compared by St. Pierre et al. [53] and Case and Isyumov et al. [61] to ascertain the effectiveness of their wind tunnel database, where the overall agreement was achieved while the higher peak wind loads than those in Davenport et al. [59] and Stathopoulos [60] were found caused mainly by the higher turbulence intensity profile ($z_0=0.03m$ by St. Pierre vs $z_0=0.005m$ by Stathopoulos) for open terrains [53].

Table 2 Summary of Parameter Studies on External Wind Pressures on Low-Rise Buildings

Parameter	General Conclusion	Reference
Upstream Terrain	Wind loads are higher in the smoother terrain due to the increased peak wind speeds; A 10% higher in the standard deviation of pressures from the open and suburban terrains was measured by Ho et al. [62]. The ASCE 7-02 [64] underestimates the peak coefficients in the suburban terrain according to wind tunnel testing [63].	Ho et al. [62]; Case and Isyumov [61]; Gavanski et al. [52]
Roof Shape	A hip roof performs better than a gable roof, which was concluded by comparing the magnitude and distribution of wind loads on these two roof types with the same 4:12 pitch [65]. The studies on the hip roofs are very limited compared with that on gable roofs.	Meecham et al. [65]; Xu and Reardon [58]; Gavanski et al. [52]
Roof Pitch	Roof pitch combined with roof shape affects both the magnitude and distribution of roof pressures by changing the flow separation and re-attachment pattern. Roof with slope larger than 30° lacks extensive studies. In ASCE 7-10, no provision for hip roofs with slopes above 27° is provided.	Stathopoulos [60]; Xu and Reardon [58]; Gavanski et al. [52]
Eave Shape	Curved eaves change the generation of separated flow regions and the vortex generation on the sharp eaves, as demonstrated with flow visualization by Hoxey et al. [66]. Reynolds number effects were found to be absent for curved eave models by the comparison of wind tunnel and full-scale data [70].	Hoxey et al. [66]; Franchini et al. [69]; Richardson and Blackmore [70]
Building Geometry (Height, overhang ratio, aspect ratios)	The uplift wind loads are higher for higher eave heights for both roof slopes (i.e. hip and gable) in both exposures (i.e. open and suburban terrains). The pressure coefficient in North American codes does not increase with eave height. Thus these codes usually underestimate the peak uplift forces, by 40% in the case of St. Pierre et al. [53].	Holmes [71]; Ahmad and Kumar [55]; Gavanski et al. [52]; St. Pierre et al. [53]
Wind Direction	The critical wind angle of attack (AOA) resulting in the conical vortex on the roof falls into the range of 15°~75° depending on the building geometry and roof shape, with the special case of symmetrical building being normally close to 45°	Ahmad and Kumar [55]; Franchini et al. [69]

Presence of canopy and parapet	They can effectively change the wind loads on both flat and curved roofs by changing the location and the type of the corner vortex. In the study of Franchini et al. [69], cantilever parapet has a reduction load factor up to 2.4 on the flat roof.	Stathopoulos et al. [47]; UWO series represented by Kopp et al. [73]; Franchini et al. [69]
Presence of Surrounding Building	The effect is controversial due to the combined action of various factors such as geometry and arrangement of these structures as well as the wind angle of incidence and upstream terrain conditions [74].	Ho et al. [62]; Holmes [71]; Surry et al. [75]; Kim et al. [56]; Khanduri et al. [74]

Note: The overhang ratio refers to the ratio of overhang width to eave height, and aspect ratios are the fraction of eave height over base width.

The field wind pressure measurements have served for the validation of wind tunnel tests in the form of pressure coefficients (e.g., TTU field test data adopted by Ho et al. [63]) and wind profiles (e.g., TTU field test data is used to calibrate LSU wind field by Pan et al. [76]). They have also been used to explore the resolution of the wind tunnel tests by comparing their results with the field test data such as the comparison by Cochran and Cermak [77] based on the TTU building as listed in Table 3. The comparison shows that most of the mean and standard deviation of external pressure coefficients measured in wind tunnel tests compared well with the corresponding in-situ experiment values. However, distinct discrepancies exist on the peak pressure coefficients, especially around the roof edge and corner, attributed to the missing large eddies in wind tunnels (e.g., [78]).

Table 3 Field Wind Tests on Low-Rise Buildings

Period	Name	Description	Reference	Validation Application
The 1970s	Aylesbury Experiment	The field test Aylesbury model is a 13.3×7.0×5.0 m gable roof building equipped 72 taps to measure the external pressure with changeable roof pitch from 5° to 45°. The Aylesbury Comparative Experiment (ACE) based on the field tests consisted of seventeen labs around the world sponsored by the International Association of Wind Engineering (IAWE). The comparison of results from these labs showed significant tunnel-to-tunnel differences in pressure measurements on low-rise buildings and the significant inconsistency might result from the tubing system, model blockage, and other factors as summarized by Sill et al. [82].	Eaton and Mayne [80]; Marshall [81]; Sill et al. [82]	Holmes [57]
The 1980s	Jan Smuts Building Test in South Africa	Boosted by the advances in the wind tunnel testing instrumentation techniques, three full-scale building tests were developed including the 100×150×30 m Jan Smuts building with 50 pressure taps, the 24×12.9×5.3 m Silsoe structure with 77 pressure taps, and the 13.7×9.1×4.0 m TTU building. Of the three, the TTU building equipped with more than 100 pressure taps has been extensively adopted as benchmark tests to validate wind tunnel simulation and the well-documented similarities and differences between the wind tunnel and full-scale TTU building tests can be found from Cochran [83].	Milford et al. [78]	Milford et al. [78]
	Silsoe Structures Building Test		Richardson et al. [68]; Hoxey et al. [67]	BLWT at Building Research Establishment (BRE) and the University of Western Ontario (UWO) (e.g., Richardson and Blackmore [70])
	TTU Building Test in the Texas Tech Wind Engineering Research Field Laboratory		Levitan et al. [84]; Cochran [83]	Cochran and Cermak [77]; Lin et al. [85]; Holmes [40]
The 2000s	Florida Coastal Monitoring Program (FCMP)	Collect in-situ full-scale wind pressure data under hurricanes rather than the previously tested extra-tropical storms. Wind pressures were collected on a variety of single-family homes including gable, hip, and complex roof line shapes during 2004 and 2005 hurricane seasons [86]. Comparison between these measured data and the data measured in wind tunnel test at Clemson University indicates the underrepresented local peak C & C loads in wind tunnel studies with suburban exposure [90]. The turbulence parameters measured by FCMP are used	Balderrama et al. [86]; Liu et al. [90]; Yu et al. [88]	House FL-27 [87]; Yu et al. [88]; Li et al. [89]

		as target parameters to develop WOW flow characteristics [88].		
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Note: Validation application refers to the studies including wind tests and numerical simulations that are validated by it.

As witnessed by post-storm inspections, internal pressures rise up when at least one windward opening is broken (e.g., [91]). The overshoot of internal pressures could reach the magnitude comparable to external pressures if “Helmholtz resonance” occurs. This resonance was first presented by Holmes [92] through using a differential equation of the transient response with an assumption that a slug of air oscillates at the opening. The air inside the structure was treated as a pneumatic spring and the energy lost in the discharge was modeled as damping so that

$$\rho l_e V_0 \gamma P_0 a C_{Pi} + \rho V_0 U_H^2 k \gamma P_0 a^2 C_{Pi} C_{Pe} + C_{Pi} = C_{Pe} \quad (3)$$

where ρ = air density, l_e = effective length of the opening, V_0 = internal volume, γ = polytropic gas constant, P_0 = atmospheric (static) pressure, k = orifice discharge coefficient, a = opening area, and U_H = mean wind speed at the average roof height. The first item is the inertial term of the air slug, and the second item represents the damping effects with respect to the energy loss when the flow passes through openings. More focus of interest has been given to the amendment of the coefficient in the Helmholtz resonator equation from wind tests or numerical simulations (e.g., [93]).

Besides the improvement in theory, recent studies of internal pressures mainly focus on parameters, such as buildings with leeward openings or the openings parallel to the oncoming wind, background leakage, and larger volume. These factors can be traced back to Eq. (3), and each of them in Eq. (3) is a good indication of the required conditions for resonant amplification. Two of them are critical, namely, the length scale formed by the ratio of internal volume to opening area, V_0/a and the Helmholtz frequency, f_H , which is approximately proportional to a/V_0 .

The research is developed to target more specific building configurations including the background leakage, various building volumes, compartmentalization (horizontal and vertical), wind flow directions (normal and others), and building envelope flexibility etc., as summarized by Oh et al. [95], as well as other factors such as location and numbers of openings (e.g., [76]). The general conclusion is that wind loading provisions underestimate the peak internal pressures including ASCE 7 in the cases of the partially enclosed and the enclosed building [76], buildings with a dominant opening [95], and large façade openings for all configurations tested by Karava and Stathopoulos [96]. Similar observations are also made for other provisions such as NBCC, AS/NZS, and Eurocode for buildings with a dominant opening [95]. The transient effect is another future work direction [99]. For more reviews regarding the wind load on low-rise buildings, please see Holmes [57], Stathopoulos [72], Krishna [102], Uematsu and Isyumov [103], and Surry [104].

3.3 ASCE 7 Pressure Coefficient

In ASCE 7-10, the design wind pressure is categorized for the Main Wind Force-Resisting System (MWFRS) (i.e., cross-bracing, shear walls, roof trusses, and roof diaphragms that assist in transferring overall loads) and Components and Cladding (C&C) that receives wind loads directly. It is noteworthy that some C&C can be part of the MWFRS when they act as shear walls or roof diaphragms. In other words, those members need to be designed for both types of loadings. For example, long-span roof trusses should be designed for the loads associated with MWFRS, and individual members of trusses should also be designed for C&C loads [105].

ASCE 7-10 allows the design of the MWFRS for low-rise buildings by using either the directional procedure per Chapter 27 or the envelope procedure per Chapter 28 (informally referred as the all heights and low-rise procedures, respectively). The formulation of design pressure in the two procedures is fundamentally different as stated in the Commentary of the two chapters. The all-heights procedure aims to represent the envelope of the measured pressures on each surface of the building as a function of the wind incidence angle (e.g., either perpendicular to or parallel to the ridge). In comparison, the low-rise procedure does not represent the actual surface pressure on buildings but instead a set of pseudo-loading conditions that will yield the envelope of a specific set of structural actions. Rather than developing spatially averaged pressures, in the low-rise procedure the pseudo loading conditions were developed to match the maximum values of five induced structural responses (i.e., total uplift, total

horizontal shear, and bending moments at the knee and ridge) of single-story moment-resisting frames based on the work of Davenport et al. [59]. It was reported that the all-heights loads create significantly higher bending moments (both negative and positive) than the low-rise loads for the roof of a facility consisting of 28 steel truss frames by Trautner and Ojdovic [106]. Their work also indicates that the extension of pseudo-pressure coefficients may not cover the worst scenario to other types of structures because the structural actions not considered in the development of the low-rise loads may be critical to the survival of a building under realistic wind loads.

The pressure coefficients for C&C were developed by spatial averaging and time averaging of the point pressures over the effective area transmitting loads to a specific location on the building through 360°. The directionality of the wind and influence of exposure have been removed, and the surfaces of the building have been “zoned” to reflect an envelope of the peak pressures possible for design purpose. The external pressure coefficients for C&C are combined values in the form of (GC_p) wherein the gust-effect factor, G , should not be separated. Basically, the actual (GC_p) values associated with the Exposure B terrain (suburban) would be higher than those for the Exposure C terrain (open) because of the reduced velocity pressure in the Exposure B terrain (ASCE 7-10, page 569 for C&C Chapter C30). The (GC_p) values currently given in ASCE 7-10 are associated with Exposure C terrain as obtained in the wind tunnel testings by Davenport in 1978 [59]. This set of (GC_p) values is also used for Exposure B terrain in conjunction with the velocity pressure exposure coefficients K_z that is defined (ASCE 7-10 Table 30.3-1) to envelop the influence of exposure. The ASCE 7 specifies constant internal pressure coefficients for both partially enclosed and enclosed buildings.

In ASCE 7, low-rise buildings are considered rigid, as the fundamental frequency is greater than or equal to 1 Hz. Therefore, static pressures are applied to evaluate the residential structure’s capacity, whereas the dynamic and duration effect of wind loads are ignored. In real world conditions, wind loads on low-rise building structures are dynamic and vary spatially as that on high-rise buildings. After Kasperski [107] questioned that the decision to neglect dynamic response of low-rise buildings was based on experience rather than on fundamental studies, Hill et al. [108] did comparisons on the nail fastened roof panels between the static testing according to the ASTM E330 method [109], and the dynamic testing using pressure traces developed from wind tunnel tests. It was found that the static pressures case overestimated the failure capacity of the panels, specifically by 20% for the case studied, which was confirmed by Dixon and Prevatt [110] who carried out similar comparisons but using more samples, i.e., expanding 5 tests per group used by Hill et al. [108] to 15 tests per group. Similarly, Habte et al. [111] found that for the standing seam metal roof, the static testing employing the ASTM E1592 testing protocol produced less deflections than the WOW tests simulating aerodynamic pressures. What’s more, the failure mode in the static ASTM tests, which is the clip slippage, was different from the clip rupture in dynamic WOW tests attributed to the vibration in the WOW tests. Therefore, there is a need to examine the dynamic characteristics of low-rise buildings to understand the whole progressive failure process.

3.4 Wind Tunnel Database

The existing design pressure coefficients are based on tests done over 30 years ago, using wind tunnel technology far less advanced than what is available nowadays, which includes issues with flow simulated with lower turbulence intensities, larger increment in wind directions in 45 degrees instead of 5 degrees used currently, and lower pressure tap density. As a result, the structural responses predicted by pressure coefficient defined in ASCE 7 are about 30% lower than the counterparts developed from the pressure measured at the University of Western Ontario (UWO) using up-to-date techniques [63].

Due to current advanced information storage and computational capabilities, Dr. Emil Simiu from National Institute of Standards and Technology (NIST) initiated a testing program as part of the “NIST/TTU Cooperative Agreement—Windstorm Mitigation Initiative” to create a large database of wind pressure time histories for public access [63]. This idea was specified as the database-assisted design (DAD) methodology and summarized by Simiu and Stathopoulos [112] and Whalen et al. [30] to simulate structural responses for improving the building design under wind loads. The Boundary Layer Wind Tunnel Laboratory at UWO contributed 95 model data sets to the NIST aerodynamic database of wind forces, and each data set was measured at 37 wind angles over an 180° range at 5° increments [53]. The tested models were gable roof buildings on a scale of 1:100 or 1:200, which were rectangular in plan, with scaled footprints of 13.72-76.20 m (45-250 ft) in ridge direction by 9.14-48.77 m (30-160 ft) across, and different roof slopes (14:12, 12:12, 1:12, and 3:12) for a range of scaled eave heights between

3.66 m (12 ft) and 12.19 m (40 ft). Full records can be accessed through the NIST windPRESSURE, an offline DAD software for rigid, gable roof buildings.

Tokyo Polytechnic University (TPU) [113] has released another aerodynamic database for a variety of building configurations including flat, hip, and gable roofs with a set model geometry scale of 1:100. This database for low-rise buildings took into account detailed influence parameters such as eave and surrounding environment and was categorized into three domains, namely the isolated low-rise buildings without eaves, isolated low-rise buildings with eaves, and non-isolated low-rise buildings (<http://wind.arch.t-kougei.ac.jp/system/contents/code/tpu>). Similar to the NIST windPRESSURE developed based on the NIST database, a database-enabled design module for low-rise buildings (DEDM-LR) was developed to host the TPU database [32]. It was found that the peak bending moments at the knee of the middle frame yielded by TPU database were 30% to 86% higher than those by ASCE 7-10, and the difference increased with the increase of the roof angle [32]. The agreement in the general trends of the bending moments calculated by DEDM-LR and NIST windPRESSURE was also reported using linear adjustments of scale model dimensions as well as tap locations due to the different model geometries used by the two databases.

4 Building Representation (FE model)

4.1 Building Configurations

As building geometries affect the wind-structure interactions, the accompanying structural configurations determine load paths through the relative stiffness and locations of individual components. Under a given wind loading condition, building configurations govern where particular failure modes will be expected when specific structural responses exceed the corresponding capacities. The identification of these weakest links along load paths is crucial to the improvement of building performance but remains challenging because the difficulties in structural representation are greatly accentuated at the building level. Large variations in building configurations make structural representation even more complicated. As indicated by Trautner and Ojdrovic [106], structural actions and locations considered as critical in the analysis of one building may not be critical for other buildings with different structural types or even for buildings with the same structural type but different bracing schemes. In other words, building configurations influence how structures behave or deform, which further steers to a different sequence of failure modes.

Low-rise buildings, represented in the two public hurricane loss models, Hazus®-MH and Florida Public Hurricane Loss Model (FPHLM), employ the combination of 2D subsystem models (or super elements) to simplify three-dimensional static analysis of full houses. This empirical engineering approach breaks down low-rise buildings into a series of super elements with the rigid diaphragm assumption and ignores the intercomponent connections among diaphragms. Specifically, the super elements include windows, doors, wall cladding, roof cladding, roof cover, shear walls, roof-to-wall connections, and foundation anchors. Although this simplification reflects the failure modes consistent with post-disaster investigations, the prescribed load paths, i.e., constant load sharing predefined among those isolated super elements, could not fully represent the interplay among structural members as a whole structure when subjected to time varying multidimensional loads.

Finite element models of low-rise buildings have been developed in the past decades to investigate the root causes of structural failures at different levels of modeling scope, structural redundancy, material nonlinearity, and load history dependency. The three-dimensional modeling dedicated to the analysis of wood framed houses under strong winds falls behind the advances in Finite Element modeling software, the FE models developed for building frames made of steel or reinforced concrete [28], or even the current capabilities of load measurements in wind tunnels [104]. There was no three-dimensional model of full wood framed houses until 1983 [28]. Full house models that are capable of performing a dynamic analysis of earthquakes and wind loads were not seen until 1997 [28] and 2012 [18], respectively. Until now, no dynamic analysis has been done to address the building envelope breach which allows water intrusion and results in interior damage that could be significantly magnified up to Category 4 hurricanes [114]. In the past, building modeling, structural components, including frames, sheathings, frame-to-frame connectors (FTFCs, especially roof-to-wall connectors or RTWCs), sheathing-to-frame connectors (STFCs), and building hold-downs, were mostly selected and studied for 2D shear walls under lateral earthquakes, wind uplift on 3D roof assemblies, and 3D box-shaped buildings (i.e., rectangular footprint and several roof types) under both earthquakes and wind loads. Less work was done on the influence of compartmentalization diaphragms, buildings with reentrant corners, and the contribution of nonstructural members (e.g., masonry veneer walls).

In terms of structural component modeling methods, broad agreements have been achieved on timber frames that are generally modeled as the linear isotropic beam elements [115], especially for the truss system of which the linear relationship between deflections to loads was verified by several studies [116]. Different modeling

methods were used in sheathing representations that range from sheathing beam with assumed discontinuity points and selected tributary width as shown in Figure 3 [115], to shell elements using in-plane stiffness as a substitute of a specific STFCs' nailing schedule [16], and to shell elements with realistic gaps and connection boundaries modeled [15]. The material properties of sheathing panels are mostly assumed as linear orthotropic [16] with few exceptions using isotropic material [119]. Limitations of the sheathing beam method lie in the fact that (1) the selection of the tributary width of the sheathing beam, which was represented by a row of sheathing panel and played an important role in the load distribution, is based on the engineering judgment; (2) the effect of sheathing fasteners cannot be captured in such a sheathing-nail-integrated modeling method; (3) the gaps between the sheets of sheathing are not modeled, which may overestimate the diaphragm stiffness. The modeling methods used on connections in the past will be discussed separately in Section 4.2.

Fig. 3. Modeling methods used for structural components in past FE building models

A handful of studies performed on entire building modeling under wind loads are listed chronologically in Table 4. Martin [5] made one of the earliest attempts to model full low-rise wooden structures including the joint behavior under realistic wind loads by using a simple linear model with the SAP 2000 program built-in elements. The truss members were pinned at chords and rigid at heels as shown in Fig. 3, which follows the modeling method used by Gupta and Limkatanyoo [14]. The plates and studs in the wall were connected as pinned as shown in Figure 3, leaving the lateral stiffness provided by sheathing. The foundation hold-down, such as Simpson Strong-Tie HDU2 used by Martin [5] (see Fig. 3), was modeled as one spring element in the vertical direction since the design of wood structures established practices [121] identified that the hold-down devices only carried vertical loads. Meanwhile, each of the anchor bolts that connected bottom plate to the foundation (see Fig. 3) was modeled with three spring elements representing translational degrees of freedom in three directions that carry all of the shear forces as well as resists wind uplift loads. Martin's [5] model was verified with experiments in the literature by comparing the deflections in 2D individual truss [116], load sharing and deflections in 3D truss assembly [12], the correlation of in-plane stiffness of the wall system and the nail spacing for a 2D shear wall [123], and influence functions at 3D entire building level [125]. He concluded that linear modeling methods were sufficient for predicting lateral and vertical load paths within the elastic range, and particularly, the uplift load path was highly dependent on the truss orientation. Martin's [5] modeling method was further expanded to complex building plans by Pfretzschner [17]. In both studies, the sheathing panels were considered as continuous in their models ignoring sheathing gap, and the effect of

sheathing nails was incorporated into the wall and roof panels by adjusting the sheathing's shear modulus. Their modeling methods were later used by Malone et al. [126] to compare the structural load path of a timber frame (TF) and a light-frame (LF) structure. It was found that TF outperformed the LF in both resisting uplift and story drift because continuous posts resisted out-of-plane wind loading more effectively than platform-framed exterior walls did, and the structural insulated panels used in the TF had greater stiffness compared with the LF shear walls.

Asiz et al. [15] developed a 3D building model that is capable of capturing the behavior of sheathing nails as well as framing-to-framing nail connections by using nonlinear link elements to simulate both of the translational and rotational deformations of each framing connector, which required load-displacement data to define the joints' characteristics. By comparing the reactions of RTWCs, total base reactions, and deformations at four corners on top of shear walls from test measurements and the predictions from the FE model, satisfactory agreements were found at a low loading level within the linear range but no at a high loading level. The mismatch at high loading levels was attributed to the exclusion of material nonlinearities of frame and sheathing members, as well as the geometric nonlinearity induced by the large deformations that were observed before failures [15]. The geometric nonlinearity was considered by another model developed by Pan et al. [120]. The difference between Pan et al.'s [120] model and Asiz et al.'s [15] is mainly on the frame connections where Pan et al. [120] adopted the generally accepted pinned connections at RTWCs and truss assemblies, and rigid connections for the rest parts. Seven failure modes were considered to identify the corresponding first failure wind speeds based on the withdrawal, pull-through, and load-slip capacities of the STFCs and the sheathing capacities controlled by the axial stress, shear stress, bending stress, and displacement thresholds [120]. Uncertainties in oncoming wind speeds, STFCs' stiffness, and sheathing boundary conditions were included to perform the vulnerability assessment, which indicated that the variations in STFCs were as important as those in wind pressures in causing structural damage based on their sensitivity studies [19]. No degradation of the nail stiffness was considered in Pan et al.'s [120] study. In Table 4, the sources of wind loadings are also listed along with the available numerical models developed for full low-rise buildings. The wind loading sources include the measurements from wind tunnels or fields, pressure coefficients defined in ASCE 7, and uniform uplift pressures on the roof. It is noteworthy that all of the analyses performed by the building models as listed in Table 4 are under static wind loads except Zisis and Stathopoulou [18], who applied wind loading time series to their linear elastic building model. Until now, no nonlinear dynamic analysis has been performed on the entire house level subjected to winds.

Table 4 Finite Element Models of Full Low-Rise Wood Buildings Analyzed under Wind loads

Reference	Analysis Method	Wind Loading Sources	Focus of Research (footprint/ FTFCs type/STFCs type/ wind loads/ validation)
Zisis [119]	SAP2000 / linear	Wind tunnel (terrain between open country and suburban) and field measurements	Rectangular building model with rigid connections at all joints applied with static wind loads using statistical mean and peak values. Validated on 2D individual frame level by comparing the sum of reactions, and on entire building level by comparing mean force coefficients and the percentages of force distribution along shear walls with the measurements from load cells installed on each shear wall [119].
Asiz et al. [15]	SAP2000 / nonlinear	ASCE 7-05 MWFRS, Wind tunnel pressures (no exposure documented)	Rectangular building model with pinned interior trusses. All STFCs and FTFCs were modeled by nonlinear link elements. Anchor bolts were modeled by linear elements. Extensive experimental tests were conducted to provide the load-slip curves of connections. Nonlinear static analysis was performed using load-controlled (as opposed to the displacement controlled e.g., 0.9 mm/min used by Ahmed et al. [128]) method to investigate failure behavior of the test structure). Validated in full-scale level by comparing base reaction forces and deformation at the four corners of on top of shear walls at low load level [15].
Martin et al. [16]	SAP2000 / linear	Uniform uplift pressure, wind tunnel uplift pressures (Datin and Prevatt [129] with exposure category B), ASCE 7-05 C&C	Rectangular building with pinned or rigid FTFCs. Anchor bolts and foundation hold-downs were modeled by linear spring elements. Validated in 2D individual truss, 3D truss assembly, 2D shear wall, and 3D full-scale house levels by comparing within elastic range.
Zisis and Stathopoulou [18]	SAP2000 / linear	Field measurements ; Wind tunnel pressures on open,	Rectangular building model with rigid link elements modeling STFCs. Only in-plane behavior of sheathing shell was considered while neglecting the out-of-plane bending. Applied wind pressure time

		light suburban, and heavy suburban terrain exposure	histories and analyzed linear dynamic responses such as reactions forces at foundation and RTWCs. Validated in 3D full building level by comparing the distribution of uplift foundation forces under static analysis in his dissertation [130]. Investigated structural attenuation of wind-induced internal force flow by comparing the FE model predicted reaction forces with field monitoring values at the truss-wall interfaces and at the foundation level in terms of correlation plots and reduction factors. Addressed the conservative estimation of building performance induced by static analysis of structural systems.
Malone et al. [126]	SAP2000 / linear	ASCE 7-10 MWFRS	Rectangular building model with pinned FTFCs. Compared structural load paths and system behavior of an LF structure with a TF structure and concluded the TF out performs the LF.
Pan et al. [120]	ANSYS / nonlinear	Wind tunnel pressures (open terrain)	Rectangular building model with pinned and rigid FTFCs. Nonlinear spring elements modeling the STFCs. Examined the first failure wind speeds associated with seven failure mechanisms of building envelope under statistical mean wind pressures measured from LSU wind tunnel.
Pfretzschner et al. [29]	SAP2000 / linear	Uniform uplift pressure, ASCE 7-05 MWFRS	L-shape building model with pinned or rigid FTFCs. Linear spring elements were used to model the anchor bolts and foundation hold-downs. Validated by deflections in 2D individual truss [116], load sharing and deflection sharing in 3D truss assembly [12], 2D shear wall [127], and 3D full-scale house [162] levels. Concluded the modeling method of Martin et al. [16] is applicable to irregular configuration wood structures and investigated the effects of reentrant corners, wall openings, and gable-end retrofits on system behavior within elastic range.

4.2 Connections

4.2.1 Connection Modeling

As discussed earlier, the accuracy of an FE model largely depends on the resolution of the modeling of joints that are primarily the starting point of building damage. The actual behavior of joints monitored under wind loads was found as nonlinear semi-rigid, which falls into the range of somewhere between the pure pinned and pure rigid [161]. The connection modeling method has evolved from complex, to simple, and to case-specific rational for particular applications.

One of the earliest efforts on joint modeling was done by Mtenga [131], who adopted nonlinear connections to model all the joints in a two-dimensional truss model. Even though only a 2D truss was modeled, he noticed that the model was complicated in the way the joints were modeled and also cast doubts on the necessity of such complexity. Thus, more simplified approaches using pinned, rigid, or spring elements (semi-rigid) were proposed in the following years. Li et al. [115] and Dung [132] employed the linear spring elements in ETABS and SAP2000, respectively, to model the heel joints and bottom-chord-splice joints of metal-plate-connected (MPC) fink trusses to account for the semi-rigidity while with the rest connections were pinned or rigid. The simplified approach received satisfactory results under uniform loading by comparing the predicted results with the experimental load sharing on 2D and 3D truss levels. Limkatanyoo [133] proposed an even simpler truss model by assuming all of the connections were pinned or rigid. To verify this method, their simulated responses of 2D and 3D models under uniform vertical loads were compared with and turned out to agree with, the numerical simulations from a more complicated, semi-rigid model by using an industry program called VIEW. However, this simplified joint model was only valid in service load range where connectors behave linearly as it failed to simulate the subsequent non-linear failure process when exposed to high wind loadings.

Recently, there has been active research that has developed more complex nonlinear semi-rigid connection models to capture the shift of load paths due to the change of connection stiffness caused by different connection configurations (i.e., nail spacing and diameters) or the connection plastic deformation. Wolfe and LaBissionere [134] found that the joint-slip, related to the withdrawal capacity of STFCs on the wood-frame roofs, controlled the load sharing through partial composite action. Here, the “composite action” refers to the interaction between the roof sheathing and the top chord of the truss to increase member stiffness and reduce member deflection by load sharing. The “partial” related to the incomplete shear transfer between the two members caused by the nonlinearity of the connections has later been widely accepted by researchers such as Doudak [21] used on the RTWCs and sheathing to truss connections. Taking into account the shear capacity of sheathing nail (STFCs), researchers such as Shivarudrappa and Nielson [135] and Pan [19] modeled each sheathing nail with three nonlinear spring elements to simulate the withdrawal and shear capacities simultaneously. Given the existence of rotational capacity at the

connections, additional rotational springs may be required. Collins et al. [136] investigated this by comparing the whole building model with and without rotational springs at the connections (FTFCs) under seismic loadings and tentatively concluded that the rotational capacity of FTFCs is negligible. Dao and van de Lindt [138] reached a different conclusion in wind load applications by adopting a new nonlinear roof sheathing fastener model with three moment-rotation DOFs as well as the one translational DOF in the withdrawal direction. Their study found a significant reduction of resistance capacity in the model that considered only the nail withdrawal capacity compared with the model consisting of rotational springs. In their study, the shear capacities in the two translational directions, other than the withdrawal direction, were neglected.

4.2.2 Constitutive Models of Connections

Since the displacement-to-force and moment-to-rotation relationships of connections are phenomenological by nature, monotonic and cyclic loading experiments have been conducted in the past to obtain the parameters needed to formulate connection constitutive models. In the available literature, the quantification of connection stiffness can be found in three sources: the recommended values from engineering design specifications, such as the National Design Specifications (NDS), the experimentally measured load-deformation relationships (e.g., [138]) and theoretical models (e.g., [139]).

National Design Specification for Wood Construction (NDS) [141] provides deterministic values to define the withdrawal and lateral shear strength (in blue lines as shown in Fig. 4) as functions of specific gravity of the wood member, shank diameter, and the yielding mode. The direct comparisons in capacities of STFC withdrawal (Fig. 4a), STFC uplift and lateral shear (Fig. 4b), and FTFC uplift/shear (Fig. 4d) show that the design-code values are generally much lower than measurements. For example, the STFC withdrawal capacity (Fig. 4a) specified in NDS is about one third of the test measurements (in Fig. 4 (a), NDS 8d common/ DV (2008) 8d Common average = 0.234kN/0.69kN=34%). Other measurements for STFC withdrawal capacity as shown in Fig. 4a are 1 kN for 8d common nail by “RWB (2001)”, 0.663 kN for 8d common nail by “RR (1999)”, and 2.6 kN for #8 screw by “RWB (2001)”, which shows that screw nail has significant higher withdrawal capacity than common nails. The capacities of STFC on the lateral shear direction of common nails range from 1.27 kN for 6d nail by “KLRM(2005)”, 1.57 kN for 6d nail by “CNJ(1998)”, and 1.78 kN for 8d nail by “RR (1999)” as shown in Fig. 4(b). For the roof-to-wall connections (RTWC), the measured uplift capacity (shown in Fig. 4d) for a 3×12d twisted shank (TS) joint is 3.1 kN by “MG (2001)” while the measured shear capacity for a 1×12d common nail joint is 1.45 kN by “CNJ (1998)”. Although roof-to-wall connections are most studied in the past under monotonic and cyclic uplifts, lateral loads, and combined [142], the moment-to-rotation curves are not available till now. The rotational capacity of sheathing-to-frame connection was tested as about 6.8 N×m as shown in Fig. 4c. It is also noteworthy that the conclusions drawn from past studies were focused on a certain range of nails (e.g., smaller than 12d), so the NDS [141] may be unconservative for large-diameter nails that are seldom tested [143]. The overestimation of current practices rises from two sources. First, the allowable loads of the connection are tested under uniaxial loads, while the lateral force may reduce the capacity [144]. Second, the implicit assumption in the practice is that the capacity of the connection is proportional to the number of mechanical fasteners. For roof-to-wall connections (RTWC), mechanical fasteners could be hurricane ties [128] or nails for toe-nail connections [146]. However, different failure modes were observed in the tested joints consisting of different numbers of fasteners per joint [146], which breaks the assumed proportional relationship of strength to nail number. Hysteretic connection models under lateral cyclic loads can be found more in the study under earthquake loads, which is not included in Fig. 4.

Note: DV (2008) -- Dao and van de Lindt [138]; RWB (2001) -- Rammer et al. [143];
NDS 2015 -- National design specification for wood construction 2015 [141]; RR (1999) -- Rosowsky and Reinhold [147];
KLRM (2005) -- Kent et al. [148]; CNJ (1998) -- Chui et al. [149]; MG (2001) -- Morrison and Kopp [24];
8d C-Ave -- 8d common nail averaged capacity; 3x12d TS-Uplift -- 3x12d Twisted Shank-Uplift.

Fig. 4. Constitutive models for (a) STFC withdrawal capacity, (b) STFC shear capacity, (c) STFC rotational capacity, and (d) RTWC uplift/shear capacity

Nail fastened joints have large variations in their behaviors. Affected by numerous factors as classified into four groups (namely nail, wood, joint characteristics, and loading) [150], the capacity of nails is of high COV, such as around 30% for withdrawal capacity with ASTM D1761 test protocol, and an even higher COV that can be up to 91% by using in-situ nail test procedure [151]. This, in turn, creates substantially large variations on the capacity of connections in the literature that was examined by Khan [152] to be around 30% COV failure capacity of toe-nails. Thus, the results of the numerical analysis that adopted the nail tested data as deterministic input data might have large discrepancies when compared with the experimental results, which requires a reliability analysis. Another thing that needs to be confirmed is whether static test results can be applied in dynamic analysis. Due to the limitation of experiment facilities and cost, previous RTWC tests have mostly been done as static testing or as cyclic but at a low displacement rate [146], which is different from the realistic wind loads condition. In the last decade, benefited from the new apparatus PLA, the researchers from UWO and the University of Florida (UF) did a series of tests on the roof and connections under realistic wind loads (e.g., [24]). Through the comparison with static test results, it was found that failure capacities of toe-nails under the two kinds of loading were similar and comparable to the capacity recorded in the literature. It was also concluded that the mean failure capacity of toe-nails is independent of loading rate under both the ramp loads and realistic wind loads [24], [147]. These findings indicate that the toe-nail capacities in the literature are still valid and can be used in the FE model. Last but not least, capacities of the toe-nails are not proportional to the embedment length/the number of them, which conflicts with the potential assumption in design code. This is due to the imbalance in the resistance of the three-nail case: two nails driven at opposite angles would yield together to withdraw, while for the 3 nail case, the single nail on one side yields before the two nails on the other side probably causing the two nails to avoid yielding [146]. Thus, the toe-nail capacity listed in the design code is not accurate, and the interpolation from experimental results by the number of nails is not applicable to the toe-nail capacity [128]. In lieu of toe-nails in old constructions, metal straps are the common RTWCs in the current buildings, and thus in recent years, there has been active research on the new forms of RTWC such as hurricane clip and the connectors with new materials, e.g., fiber reinforced polymer or FRP [155].

5 Development of Performance Criteria

The judgement of building performance under hurricane winds needs a hierarchy of criteria that are usually in descriptive format stated as “minor damage,” “moderate damage,” “severe damage,” and “destruction,” which were quantified by the percentage of loss of roof cover, window/door, roof deck, missile impacts on walls, roof structure failure, and wall structure failure [8]. When deciding whether or not a specific performance level is achieved, it needs mapping between those qualitatively stated criteria and a response quantity/limit state (measuring force or deformation) that can be checked using principles of structural analysis and mechanics [20]. In other words, those mapped performance criteria will describe structural behaviors from the linear range to the first failure where the design limit states are set for, and throughout the progressive failure in the nonlinear range. It is noteworthy that

such a mapping invariably requires that the behavior of the building structural system be considered as a whole [20]. For the building performance under seismic loads, FEMA Report 356 related the immediate occupancy, life safety, and collapse prevention performance levels for vertical structural elements in light-frame wood construction subjected to seismic effects to transient lateral drift ratios of 0.01, 0.02, and 0.03 [156]. No such mapped performance criteria have been established for the building performance evaluations under hurricane winds until now.

One challenge in setting up the mapping relationship is how to equate performance-based criteria to deformation-based criteria, internal force-based criteria, or the mixed ones. Post-disaster investigations only provide the damage state, such as the percentage of building envelope damage and the failure modes, but are limited to provide neither the wind pressure fields that cause that damage nor the structural responses that can be used for FE model validations. Fortunately, recent full-scale and large-scale building tests, as summarized later in this section, were done in a controlled manner to collect valuable information on wind loading and structural responses simultaneously. That measured data could be grouped rationally into categories to better reflect the damage states for different hurricane intensities from an engineering perspective. Another issue is, unlike the seismic case, the structural responses under wind loads are three dimensional, which cannot be simplified in the lateral drift as a single parameter. A combination of key deformations of representative building constructions may be selected as indicators to construct vulnerability curves based on the observed building damage in post events and experimental tests.

The full building tests that measure the structural response, the load distribution, as well as the wind loads, carried out in the past released valuable information for FE model validation for the building performance under extreme wind events. The early full wood house tests were performed to analyze the load path as well as the resistance to lateral cyclic loads provided by the shear wall and roof diaphragm, which serves the purpose for seismic performance evaluation (e.g., [157]). Even though the failure mechanisms are different (i.e., uplift force induced by wind loads, lateral loads generated by earthquakes), this valuable information can still be used to examine the wall behavior under low wind force, validate analytical models to some extent, and deepen the understanding of the load distribution and load-sharing mechanisms. Due to the high cost, few full building tests have been performed under either hurricane winds or lateral seismic loads, as summarized chronologically in Table 5. Only two of them were conducted on nonsymmetrical L-shaped buildings, representing typical North American single-story houses other than the rectangular plan, which made possible the investigation on the effect of reentrant corners. The one built by North Carolina University (NCU) has been widely compared by numerical simulations for both seismic and wind cases [159].

General observations on the behavior of full wood structure under lateral loads include: (1) the roof diaphragm acts rigid compared with shear walls, which is actually contradicting the assumption that the horizontal diaphragms are “flexible” compared to the shear walls [159]; (2) the load sharing capability of the in-plane shear wall is dependent on the relative location and stiffness of the wall, which can transfer loads approximately from 20% to 80% to the rest of components [160]; (3) transverse shear walls carry few loads. It is noteworthy that some of the suggestions on the methods and elements to be used in the FE model are based on the lateral loads tests that induce different failure mechanisms and might not be valid for wind scenario, such as the ignorance of sheathing gaps or sheathing fasteners ideally modeled as pin connections [125].

Only two major full building tests have been done under wind loads so far. One is through the consortium of Canadian Universities awarded by the NSERC Collaborative Research and Development (CRD) Project. Both the wind pressure and internal force data are monitored and collected from three light-frame wood buildings located in Saint Soy of Quebec City [161], in Fredericton of New Brunswick [18], and in Winnipeg of Manitoba, respectively. The other is the “Three Little Pigs” Project (3LP) at the UWO Insurance Research Lab for Better Homes (IRLBH). This project was done to replicate full spectrum pressure fluctuations by PLA with the input pressure traces derived from wind tunnel tests and to monitor the failure process of roof-to-wall connections [153]. It is worth noting that with the restriction of PLAs, the current 3LP experiment applies wind loads only on the roof surface instead of the whole structure, and this leads to the ignorance of the effects of the lateral wind force acting on the wall. Until now, most full-scale and large-scale building tests were focused on the measurements of structural responses within linear range or before the global structural failures. Destructive testing would be a future direction to provide a full picture of the building performance. Selected building tests under lateral loads are also included in Table 5, since in the wind load scenario, shear walls resist horizontal thrusts on the windward face and suction on the leeward side.

Table 5 Full-Scale and Large-Scale Wood-Framed Low-Rise Building Test and Analysis under Wind and Selected Lateral Loads (1990-now)

Reference	Load Form	Focus of Research
Phillips et al. [160]	Pointed lateral loads	A full-scale single-story, rectangular gable roof house. Investigated how horizontal diaphragm distributes lateral load into shear walls. Different sheathing stiffness was analyzed by varying fastener spacing. The roof diaphragm was found to be rigid and affected the lateral load distribution among shear wall while transverse walls did not. The load sharing was determined by the stiffness of the loaded wall, and the configuration and stiffness of the surrounding structure. Used for FE model validation by Kasal [159].
Paevere et al. [162]	Pointed lateral loads (static, slow destructive cyclic)	A full-scale single-story, L shaped house. Roof diaphragm was found or built stiffer than shear walls; in-plane wall (parallel to lateral loads) are sharing 20-80% loads with other walls depending on location and stiffness of surrounding walls. Measurements were used for FE model validation under earthquake loads [163] and wind loads [29].
Doudak et al. [161]	Environment wind and artificial distributed and pointed loads	A full-scale single-story, rectangular, flat-roof house. This belongs to CRD monitoring project on an existing industrial shed located in Quebec City, Canada and owned by Forintek Canada Corp. Only deflections were measured. The composite action and load sharing mechanism should be considered in the structure analysis. Used for FE model validation by Doudak [21].
Songlai et al. [164]	Uniform lateral loads	A full-scale single-story, L-shaped house. The gypsum sheathing significantly affected the strength and stiffness of shear walls. The floor diaphragm behaved as a semi-rigid diaphragm in load distribution. A 1/400 inter-story drift restriction of shear wall for lateral wind loads in serviceability limit states design was recommended.
Morrison [22]	Replicated wind pressure time histories on the roof based wind tunnel measurements	A full-scale two-story, rectangular gable roof house. Belongs to the 3LP project that applied realistic wind load on realistic low-rise building. The loads applied only on roof. Performance of toe-nailed RTWCs was examined. The failure of toe-nail RTWCs is incremental at peaks of fluctuating wind loads. The maximum capacity of the RTWCs was not affected by the loading form (ramp or fluctuating). Used by the validation of Jacklin et al. [118].
Datin et al. [129]	Open-jet wind loads	A 1/3-scale single-story, rectangular gable roof house. FE models validated by it include Datin [125]; Martin et al. [16]; Shivarudrappa and Nielson [135].
Zisis and Stathopoulos [18]	Environment wind loads	A full-scale single-story, rectangular gable roof house. This belongs to CRD monitoring project in Fredericton, New Brunswick. The field studies verified the wind tunnel and finite element analysis. The significant attenuation of wind loads was identified. Two side walls (parallel to the ridge) contributed much more to sustain the wind uplift force than end walls (normal to the ridge). Used for FE model validation by Zisis and Stathopoulos [18].
Doudak et al. [165]	Pointed lateral loads and vertical artificial gravity loads	A full-scale single-story, rectangular gable roof house. This belongs to CRD monitoring project in New Brunswick, Canada. Localized external forces have effects through entire system. Building superstructure was rigid enough under lateral loads.

It is promising to quantify the building performance using vulnerability curves/fragility curves of the key structure responses, which requires a validated deterministic FE full building model, the distributions of capacities of structural members/joints, and the full information of surface pressure more than peak or mean values. Such an approach, supported by the stochastic finite element method (SFEM), extends the classical deterministic FE approach to stochastic framework [166]. The first step in SFEM is to set up a refined analytical model built with detailed structural components, such as FTFC and STFC, which helps reflect more accurate structural responses, especially in the nonlinear range. The second step is to validate the deterministic model and then to propagate uncertainties throughout the building system for the parametric sensitivity analysis and the assessment of overall building performance. In such a way, the information collected from the full/large-scale building testing will be leveraged through a validated FE model to predict the influences of large uncertainties imbedded in both wind loads and structure capacities.

6 Research Challenges and Future Directions

A comprehensive review of the progress and state-of-the-art research on the theme of low-rise building performance under wind loads is presented, mostly from the viewpoint of the structural engineering side. To tackle this topic completely, the meteorological components, such as modeling of hurricane genesis, track, intensity, radius

of maximum speed, and filling rate, are also important to understand and improve for the accuracy of damage prediction. To improve the performance of low-rise buildings under strong winds, the following areas related to wind engineering and structural engineering need more attentions in the near future.

- ABL Simulations:

The current well adopted ABL simulation methods including BLWTs, open-jet, and CFD have their own pros and cons. For example, the BLWT is able to generate a very smooth flow that can be managed for a target profile, while having difficulty in simulating large-scale, low-frequency turbulent fluctuations. The open-jet experiments greatly improve the Reynolds number problem and allow for destructive tests. However, besides having a similar challenge with BLWTs, the short test section of the open-jet and the lack of flow conditioners such as the settling room and the honeycomb in the wind tunnel lead to the immature and high-frequency turbulence. The CFD is a powerful tool, which allows simulating any object with a proper flow modeling. However, this tool requires powerful computation ability and validation from parallel studies. Thus, there is a significant need to develop more realistic ABLs and or nontraditional ABLs using new techniques.

- Wind Pressure Characteristics:

Although the wind pressure coefficient has always been the focus of research on low-rise buildings under extreme wind loads, there are still some aspects that need more studies. First, the external pressure coefficients on the hip roof structure and the low-rise building with roof slopes larger than 30° are rarely tested. Second, no consensus on the interference effect of surrounding structures calls for more studies to determine the condition of enhancement or shielding effects and to quantify them, which serves for the simplifications of codes and standards on wind loads. Currently, only one study by Zhang et al. [167] in the literature was carried out on the interference effect under tornado-induced wind loads and flow patterns, which indicates the need for nonstraight-line winds research. Third, the volume scaling should be considered to produce the correct internal fluctuation and resonance, including the transient condition due to the sudden opening, and the dynamic similarity should be ensured.

- Scaled Model Wind Tests:

First, real residential buildings with different types of trusses and irregular truss shapes, etc. are usually much more complex than the simple rectangular building plan that is usually tested. Since the system behavior is largely affected by the geometry of the trusses rather than the material properties, current results may not include all of the system effects that may exist in more complex roof geometries. Thus, more realistic house tests are needed. Second, most wind tests are undertaken under extra-tropical storms rather than hurricanes [87]. The characteristics of the two kinds of storms are different in that hurricanes are gustier, more turbulent, and produce higher peak pressures [168]. Thus, extreme wind speeds are needed to induce the wind tests, rebuild the current database, and modify the wind load design code, which is currently based on the low wind speed tests.

- Full-Scale and Large-Scale Wind Tests:

At the present time, the full-scale or large-scale wind tests are limited mainly due to the existence of a social, economic, and institutional barrier to the deployment of those tests as concluded by the committee of National Research Council [169] after reviewing the need for a large-scale test facility. Furthermore, although the full-scale test that is able to measure the response is a big step from the classic type, such as TTU and Silsoe field tests that only measure wind pressures, it still has a long way to go to become truly practical for designers and engineers. For example, destructive tests are needed for the progressive failure study of residential houses towards Performance Based Hurricane Engineering with a significant challenge to relate wind loads to specific failures of the whole structure. Currently, the prediction of failure wind speed has made its way to some scaled model wind tunnel tests [170]. These tests were carried out on the structural-component level, such as roof, which led to the discrepancy with the real three-dimensional case and brought about significant assumptions, such as the implicit failure mode.

Therefore, systematic full-scale and large-scale tests are needed to build a comprehensive database with uniform formats for the public domain that includes pressure time series along with response time histories, especially at different stages of failure. Although such a huge project would demand plenty of manpower, materials and financial resources, which may require the collaborative research between organizations around the world, it is promising in the long run. Benefited from this kind of work, unnecessary expense on the unsystematic and repetition conduction of the wind test can be avoided and the time spent on processing the data with different format from different database could be saved.

- Large Database of Connection Behavior:

A reliable FE model requires the correct material property inputs. The current available load-displacement relationships of connections are scarce due to the following reasons. First, since interpolation by the number of fasteners is not applicable to the capacity of either the toe-nail connection [146] or the hurricane tie [128], every condition needs to be tested. Second, since the load-displacement behavior of individual toe-nails is of high variations (high COV) [152], the results of numerical analysis adopting the toe-nail tested data might have discrepancies with the experimental results. As such, each condition needs to be tested in a large amount to get the statistical parameters. Therefore, extensive mechanical experiments on a variety of joints are needed for numerical model analysis. Similar to the experiences of seismic analysis, it would be useful to perform experiments to obtain hysteresis curves of connectors under highly fluctuating wind loads as the nail strength is very sensitive to loading with respect to the duration and types, so that more accurate FEA results can be obtained.

- Full Numerical Model:

There are several challenges associated with the simulation in regards with pursuing higher resolution results of the numerical analysis of full building models subjected to wind loads. First, due to the large scale and complexity of the model, convergence becomes an issue in nonlinear analyses. Therefore, sacrifice and simplification have been made on the material property in all the validated numerical models, and those models are only applicable in the linear range. Second, static analysis still dominates the FEA on the full building scale, which results in the failure to capture the damage accumulation of the structure induced by nails under fluctuated wind loads. Third, analysis on the failure process is very rare. Asiz et al. [15] and Pan et al. [120] tried to address the first failure system behavior but the former with poor experiment validation and the latter without validation. Last but not least, most previous validations associated with the 3D full-scale level are based on the global response data alone, which may lead to false confidence in the accuracy and even result in misleading conclusions on the assessment of system behavior. Thus, validations in a wide range from the 2D subassemblies to the 3D whole model are needed. Another problem with the validation is its limitation on the static and linear range, which results in questionable results when the model is used for the analysis of the post-linear phase to failure.

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