1-16-2018


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SYSTEM NONLINEAR PERFORMANCE OF LOW-RISE BUILDINGS UNDER DATABASE-ASSISTED HURRICANE LOADS

A Dissertation

Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Doctor of Philosophy

in

The Department of Civil and Environmental Engineering

by

Jing He
B.S., Hebei University of Technology, 2009
M.S., Hebei University of Technology, 2012
May 2018
To my family
ACKNOWLEDGEMENTS

I would like to extend my sincere appreciation to those special people who have challenged, helped, and put their faith on me during this journey to pursuing my goal.

My first and deepest appreciation must go to my advisor, Professor Dr. Steve C.S. Cai, for his support, patience, and guidance throughout my Ph.D. study. He has always been so generous and inspiring with extensive personal and professional wisdom. He is a role model for me to follow in both scientific research and life.

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My gratitude also goes to the financial support of the Economic Development Assistantship offered by Louisiana State University and the National Science Foundation (NSF project No. CMMI-1233991).

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TABLE OF CONTENTS

ACKNOWLEDGEMENTS ............................................................................................................................ iii

ABSTRACT .................................................................................................................................................... vi

CHAPTER 1. INTRODUCTION .................................................................................................................. 1
  1.1. General Background .......................................................................................................................... 1
  1.2. Research Objective .......................................................................................................................... 3
  1.3. Dissertation Structure ......................................................................................................................... 3
  1.4. References ........................................................................................................................................... 6

CHAPTER 2. A REVIEW OF WOOD-FRAME LOW-RISE BUILDING PERFORMANCE STUDY UNDER HURRICANE WINDS ...................................................................................... 7
  2.1. Introduction ....................................................................................................................................... 7
  2.2. Hurricane Hazard Modeling .................................................................................................................. 10
  2.3. Wind-Structure Interactions ................................................................................................................... 12
  2.4. Building Representation (FE model) ................................................................................................... 20
  2.5. Development of Performance Criteria ................................................................................................. 28
  2.6. Research Challenges and Future Directions ......................................................................................... 31
  2.7. References .......................................................................................................................................... 34

CHAPTER 3. FINITE-ELEMENT MODELING FRAMEWORK FOR PREDICTING REALISTIC RESPONSES OF LIGHT-FRAME LOW-RISE BUILDINGS UNDER WIND LOADS ......................................................................................................................................... 45
  3.1. Introduction ....................................................................................................................................... 45
  3.2. Description of the Conducted Experiment ......................................................................................... 48
  3.3. Numerical Modeling of Wind Effects on the Low-rise Building ........................................................... 55
  3.4. Loading ............................................................................................................................................. 60
  3.5. Model Validation ................................................................................................................................. 62
  3.6. Effects of Modeling Methods .............................................................................................................. 69
  3.7. Conclusion ......................................................................................................................................... 74
  3.8. References .......................................................................................................................................... 75

CHAPTER 4. NEW DAD APPLICATION: PROGRESSIVE FAILURE OF LOW-RISE TIMBER BUILDINGS UNDER EXTREME WIND EVENTS .................................................................................. 79
  4.1. Introduction ....................................................................................................................................... 79
  4.2. Progressive Failure Analysis Methodology ........................................................................................... 82
  4.3. FIU Calibration Building ....................................................................................................................... 85
  4.4. LSU Building Model ............................................................................................................................... 92
  4.5. Conclusion ......................................................................................................................................... 96
  4.6. References .......................................................................................................................................... 96

CHAPTER 5. MODELING WIND LOAD PATHS AND SHARING IN A WOOD-FRAME BUILDING ................................................................................................................................. 100
  5.1. Introduction ....................................................................................................................................... 100
  5.2. Literature Review ............................................................................................................................... 101
5.3.  Modeling Methods and Loading Sources................................................................. 103
5.4.  Load Distribution Parameter Study....................................................................... 106
5.5.  Results: Total VM and Failure............................................................................... 121
5.6.  Conclusion............................................................................................................... 123
5.7.  References............................................................................................................... 124

CHAPTER 6. ASSESSMENT OF ASCE 7-10 FOR WIND EFFECTS ON LOW-RISE WOOD
FRAME BUILDINGS WITH DATABASE-ASSISTED DESIGN METHODOLOGY ....... 127
6.1.  Introduction............................................................................................................. 127
6.2.  LSU Aerodynamic Database.................................................................................. 130
6.3.  Comparison with ASCE 7-10 Provisions.............................................................. 133
6.4.  Discussion............................................................................................................... 134
6.5.  Conclusions............................................................................................................ 144
6.6.  References.............................................................................................................. 145

CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS...................................... 148
7.1.  Summary and Conclusion ..................................................................................... 148
7.2.  Recommendation for Future Study ..................................................................... 149

VITA................................................................................................................................. 151
ABSTRACT

Light-frame wood buildings account for over 95% of all residential structures in the U.S, of which the majority are designed as low-rise buildings. These low-rise residential buildings in the U.S. have performed unsatisfactorily and are the largest source of the damage and fatality during the past extreme wind events. To deepen the understanding and reduce the vulnerability of the infrastructures, the accurate prediction of the hurricane loss has been an urgent need, and the hurricane catastrophe models are developed in response. However, the current hurricane catastrophe models are focused on the economy loss estimation rather than investigating the root causes of structural failures that only little or empirical structural analysis is involved. Thus, these models cannot reveal the realistic load paths nor the stage-wise damage propagation. This dissertation aims to develop a validated finite-element (FE) modeling framework for predicting the system nonlinear performance of low-rise buildings under the spatiotemporally varying wind loads with the reasonable accuracy. This framework would serve for the successive damage prediction as a part of the risk assessment of low-rise buildings under extreme wind events.

To reach the final objective, a refined 3D modeling methodology is proposed first. This modeling methodology contributes to combine the strengths of each involved disciplines to achieve a desired resolution, i.e., the dynamic form of wind loads, the full-scale scope of modeling, and the extensive nonlinear representative of the critical components. It is validated by a large-scale wind test from the linear to the nonlinear range including the successive failure stages. This modeling methodology provides the foundation for the future research.

Secondly, a progressive failure prediction methodology aiming at finding the quantitative relationship between the wind intensity and the damage state of the building is well developed with an explicit explanation on the failure mode, the failure location, and the failure criteria. This methodology is also validated in the building scale and the individual connection scale by a corresponding destructive wind test with the agreement on the failure mode and sequence. Meanwhile, the database-assisted design (DAD) technique is extended from its original application on the linear prediction on the frames of the metal structure to the nonlinear modeling on the envelope of the wood structure in the current study.

This framework that consists of the building modeling and the failure prediction provides a guideline on the three crucial steps for a more accurate performance prediction: directly using the aerodynamic database derived from wind tests, applying the loads onto a refined building model considering the nonlinear behaviors of critical components, and conducting the analysis on the progressive failure process. Then, some attempts are made on the application of this framework, e.g., the effect of the geometric and loading forms on the load paths and structural failure is discussed, and the adequacy of the wind design by using the ASCE 7-10 wind provisions on residential buildings is evaluated.
CHAPTER 1. INTRODUCTION

1.1 General Background

In the United States, light-frame buildings account for over 95% of all residential structures, and the majority of these types are designed as low-rise buildings [1]. The definition of a low-rise building has been given in ASCE7 as a building with a mean roof height, \( h \), less than or equal to 60 ft (18 m) and at the same time does not exceed the least horizontal dimensions [2]. Wood frame structures are used in over 90% of these light-frame low-rise buildings [1]. A light-frame building is a complex structural system made of components and subassemblies (e.g., sheathing panels, shear walls, truss assemblies) with repetitive members connected by inter-component connections (e.g., nails, metal straps, foundation hold-downs, and anchors). In this way, a 3D significantly indeterminate structural system is formed, and the high redundancy leads to the difficulty in the determination of load paths. Even though the light-frame buildings are supposed to perform well as they can stay over a long time, when encountered extreme wind events, they have suffered extensive damage due to the little understanding on the load paths when constructed. Since around one-third of the U.S. population reside in the areas within 100 miles of hurricane-prone coastline, i.e., the Atlantic and Gulf coasts [3], their houses are in great danger.

As reported by the National Association of Insurance Commissioners, hurricane-induced catastrophes account for seven out of the top ten most costly insured property catastrophes in the U.S. [3]. The National Oceanic and Atmospheric Administration [5] estimated the damage caused by extreme wind disasters in 2015 Consumer Price Index (CPI) cost adjusted value. The economic loss inflicted by hurricanes, tornadoes, and severe storms were $55.4 billion from 1980 to 1989, $132.2 billion from 1990 to 1999, $370.7 billion from 2000 to 2009, and $165.5 billion from 2010 to 2014 (summarized by author). For example, in 1992, Hurricane Andrew resulted in $49.5 billion (converted to 2015 dollars) of economic losses which were the largest loss caused by a natural disaster that the United States had ever experienced at that time [6]. Most of these monetary losses came from the residential house damage, e.g., the house damage accounts for nearly 60% of the total insured losses from Hurricane Hugo [7]. As such, an accurate prediction of the low-rise building performance, especially the extent of failure during extreme wind events is desired in light of the compelling interest of insurance companies and the urgent need of effective mitigations of the windstorm hazards.

Observations of the reconnaissance trip on the wind damage event revealed that the main source of damage in houses was the lack of continuous uplift load path from the roof down to the foundation [9]. As opposed to the downward gravity effect that depends largely on the capacity of components such as the beams and studs, the uplift wind effects on the vertical load path make the inter-component connections the critical members, i.e., sheathing-to-truss connections (STTCs), roof-to-wall connections (RTWCs), and foundation connections, as shown in Fig. 1.1. Based on that, the vulnerability of residential houses is resulted from the following facts. First, most residential buildings in the U.S. are conventional, non-engineered (or called deemed-to-comply) construction where the construction techniques are based on traditional practice and experience rather than the engineering calculation, especially under wind loads for these critical members. Second, even constructed by building codes, the older house stock that was built before the improved building codes introduced in 1994 as a result of Hurricane Andrew and accounts for
more than 80% of homes in the U.S. by 2003 [10] is still more vulnerable. Third, the
misconstraction led by the poor inspections such as the missing nails and the material degradation
influenced by the environment over the years are the other reasons that result in the weakness in
the uplift load paths.

![Fig. 1.1. Hurricane damage and critical member along the uplift load paths](image)

Besides the load distribution within the building configuration, a deep understanding of
wind loads also determines the sufficiency in the design and evaluation of the low-rise wood
buildings. The evaluation of the temporal-spatial varying wind loading on the low-rise building is
a rather complicated issue. A large number of parameters influence the wind loads, including the
wind field created by surrounding areas such as surface roughness, shielding from neighbor
structures, and wind direction, etc. After the flow-structure interaction, turbulence changes in the
wake of its geometry such as roof slope, roof type, the presence of canopies and parapets, openings,
porosity in the wall, etc. Such complexity cannot be reflected in the building codes and standards
which have to be of generalizability, and the resolution of wind loads has to be sacrificed. Various
building design codes adopt different expressions and values of their parameters for the wind
pressure. For example, the directionality reduction factor defined in the ENV [11] is 1.0, while in
ASCE 7-10 and AS/NZS [12] the value is specified as 0.85. Researchers have been driven to
devote on the codification to validate and quantify these simplifications (e.g., [13]; [14]). To get a
better understanding of the low-rise building behavior under extreme wind events, the need for a
higher resolution wind loading, numerically and experimentally, becomes necessary.

The progress of the study on the low-rise building performance to winds is comparatively
slow. The advanced analytical methodologies that are widely applied to the high building or
seismic design have not been used in the low-rise building wind resistant design. For example, in
the majority of the seismic performance research, dynamic loads are applied on nonlinear models
with specially care on the nail connections where the material property is modeled by applying a
hysteretic curve. However, such resolution on both building model and loading is missing in the low-rise building wind analysis, and state-of-the-art technologies are expected to facilitate the performance-based design and drive the advances in the system vulnerability analysis under wind loading.

### 1.2 Research Objective

The general objective of this project is to develop a finite-element (FE) modeling framework for predicting the system nonlinear performance of low-rise buildings under database-assisted hurricane loads with reasonable accuracy. It is anticipated that this project will contribute to the damage prediction of low-rise buildings under extreme wind events, which is a critical part of the risk assessment. Serving for accomplishing this project, two wind tests are conducted, and two FE models are developed accordingly with a practical modeling methodology that is validated herein. During this process, the characteristic and the specific contribution are categorized as follow.

1) **FE modeling methodology:**
   - Detailed FE model: high resolution FE model with the detailed configuration regarding inter-component connections such as STTCs, RTWCs, and foundation connections.
   - Nonlinearity and envelope Analysis: load paths for envelope besides MWFRS in inelastic range.
   - Realistic wind loads: dynamic wind loading analysis on detailed low-rise building model.
   - Validated by large-scale wind tests.

2) **Progressive failure analysis methodology:**
   - Comprehensive failure modes and failure thresholds determined from experiment phenomenon are well investigated.
   - Realistic fluctuating wind loads.
   - Validated by large-scale destructive wind tests.

3) **Current wind loads resources:**
   - Evaluate the adequacy of ASCE 7-10 wind procedures on light-frame wood houses.
   - Extend the application of database-assisted design (DAD) from linear to nonlinear range and finally to the progressive failure stages with envelope behavior predictions.

### 1.3 Dissertation Structure

Specific tasks to accomplish the tentative contributions in the form of the dissertation chapters are shown in Table 1.1. The motivation of each chapter and how do they serve for the general objective are illustrated along with the approach and data needed.
This dissertation consists of seven chapters. Chapter 1 provides the background and the justification of the research. The overall objective and specific projects to accomplish it are identified.

In Chapter 2, the literature pertinent to the performance prediction framework is reviewed systematically, i.e., the hurricane hazard modeling, the wind-structure interactions, the numerical building representation, and the performance criteria. The challenges remain are identified, including the wind loading resolution, the resolution and scale of the building model, and the need for the full-scale or large-scale wind tests with the purpose of response measurements as well as the destructive tests.

To address these issues, the numerical modeling methodology with higher resolution that is capable of predicting the realistic response of the low-rise buildings under wind loads is explored in Chapter 3. A nonlinear modeling methodology calibrated by large-scale wind tests is proposed and believed to be suitable for the assessment on the performance of light-frame wood buildings under high wind speed events with adequate accuracy.

In Chapter 4, an analysis methodology for progressive failure of low-rise buildings under wind loads is proposed and validated by a large-scale destructive wind test. During this process, the modeling methodology presented in Chapter 3 is further validated in the failure range, and the application of database-assisted design (DAD) is extended beyond the linear range.

Based on the validated methodologies, applications are made on the parameter study in Chapter 5 and the assessment of building codes adequacy in Chapter 6. The parameters that affect the load paths are analyzed, including the building geometry variations and the wind loading resolution. The adequacy of the current design loading sources, i.e., ASCE 7, is evaluated in response level from the perspective of the methodology used in the development of it.

In the end, results and conclusions are briefly summarized in Chapter 7 and the recommendations for future work are specified.
Table 1.1. Dissertation framework

<table>
<thead>
<tr>
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<tbody>
<tr>
<td>Specific Objective</td>
<td>Review: Validate: Validate: Parameter Study: Assess:</td>
</tr>
<tr>
<td></td>
<td>the literature pertinent to the performance prediction framework</td>
</tr>
<tr>
<td></td>
<td>Collect wind loads and response data for the following research</td>
</tr>
<tr>
<td></td>
<td>the nonlinear FE modeling methodology;</td>
</tr>
<tr>
<td></td>
<td>the progressive failure prediction methodology;</td>
</tr>
<tr>
<td></td>
<td>Extend DAD application</td>
</tr>
<tr>
<td></td>
<td>on buildings with different constructions and wind loading resources</td>
</tr>
<tr>
<td></td>
<td>the adequacy of ASCE 7 to the wood-frame low-rise buildings</td>
</tr>
<tr>
<td>How Objective Relates to the Aim</td>
<td>Provide the background and motivation for the following chapters</td>
</tr>
<tr>
<td></td>
<td>Provide a nonlinear modeling methodology;</td>
</tr>
<tr>
<td></td>
<td>Serve as the theoretical basis for the following research</td>
</tr>
<tr>
<td></td>
<td>Investigate the progressive failure of the low-rise buildings</td>
</tr>
<tr>
<td></td>
<td>Analyze the parameters that affect the load paths</td>
</tr>
<tr>
<td></td>
<td>Evaluate the current design loading sources to the building performance prediction</td>
</tr>
<tr>
<td>Approach</td>
<td>Evaluate the progress of each of the categories that determines the accuracy the framework</td>
</tr>
<tr>
<td></td>
<td>Develop a building model with the proposed modeling method where the behavior of critical member is well modeled</td>
</tr>
<tr>
<td></td>
<td>Tracing the building behavior during the progressive failure process subjected to dynamic wind loads</td>
</tr>
<tr>
<td></td>
<td>Investigate the effect of geometry variation and loading resolution on the load paths with the validated modeling methodology</td>
</tr>
<tr>
<td></td>
<td>Examine the methodology used in the development of ASCE 7 by comparing the responses and location of the critical members to that subjected to dynamic winds</td>
</tr>
<tr>
<td>Data Required</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Wind loads and responses data from the FIU wind tests</td>
</tr>
<tr>
<td></td>
<td>Loads and responses from FIU wind test and wind loads from LSU database</td>
</tr>
<tr>
<td></td>
<td>Wind loads from LSU database and FIU wind test</td>
</tr>
<tr>
<td></td>
<td>ASCE 7-10 wind loads and LSU wind database</td>
</tr>
<tr>
<td>Chapter</td>
<td>2</td>
</tr>
</tbody>
</table>
1.4 References


CHAPTER 2. A REVIEW OF WOOD-FRAME LOW-RISE BUILDING PERFORMANCE STUDY UNDER HURRICANE WINDS

2.1 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 2.1 in the descending order of loss [1], 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 [1]. With this large number in mind, it is not surprising to see in Table 2.1 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.

Table 2.1. Ten most costly insured property U.S. catastrophes through 2014

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

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. 2.1. 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. 2.1. Without the subjective segregation of the damage distribution at each wind speed [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) [26]. 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. 2.1. Structural vulnerability assessment included as one component of catastrophe models after Hurricane Andrew 1992 and an illustration of typical vulnerability curve](image-url)
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.

![Probabilistic Building Performance Assessment](image)

Fig. 2.2. Streamflow diagram showing components contributing to probabilistic building performance assessment under hurricane wind loads
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. 2.2.

2.2 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.

2.2.1 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 m 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:

\[
\frac{u}{\partial r} - \frac{v^2}{r} - fv + \frac{v}{r} \frac{\partial u}{\partial \phi} + \frac{\partial p}{\partial r} - K \left( \nabla^2 u - \frac{u}{r^2} - \frac{2}{r^2} \frac{\partial u}{\partial \phi} \right) + F(c, u) = 0 = \frac{\partial u}{\partial t}
\] (1)
\[ u \left( \frac{\partial v}{\partial r} + \frac{v}{r} \right) + f u + v \frac{\partial v}{\partial \phi} - K \left( \nabla^2 v - \frac{v}{r^2} - \frac{2}{r^2} \frac{\partial u}{\partial \phi} \right) + F(\bar{c}, v) = 0 = \frac{\partial v}{\partial t} \] (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, \( \phi \) is the azimuthal coordinate, \( K \) is the eddy diffusion coefficient, and \( F(\bar{c}, u) \) and \( F(\bar{c}, v) \) 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.

### 2.2.2 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, \( W_{LF} \), 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, \( q_z = 0.613K_zK_tK_dV^2(N/m^2) \). 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.

2.3 Wind-Structure Interactions

2.3.1 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., setting 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., $Re \gg 50,000$ 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].

2.3.2 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 2.2 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.
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 (\(z_0 = 0.03\) m) were found to be about the same as those from suburban (\(z_0 = 0.3\) m) because the decrease in the mean roof height speed is offset by the increased turbulence level [53]. In Table 2.2, 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 2.2, 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 (\(z_0 = 0.03\) m by St. Pierre vs \(z_0 = 0.005\) m by Stathopoulos) for open terrains [53].

Table 2.2. Summary of parameter studies on external wind pressures on low-rise buildings

<table>
<thead>
<tr>
<th>Parameter</th>
<th>General Conclusion</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstream Terrain</td>
<td>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].</td>
<td>[62]</td>
</tr>
<tr>
<td>Roof Shape</td>
<td>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.</td>
<td>[58]</td>
</tr>
<tr>
<td>Roof Pitch</td>
<td>Roof pitch combined with roof shape affects both the magnitude and distribution of roof pressures by changing the flow separation and re-attachment pattern.</td>
<td>[58]</td>
</tr>
</tbody>
</table>

(Table 2.2. continued)
<table>
<thead>
<tr>
<th>Parameter</th>
<th>General Conclusion</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eave Shape</td>
<td>Roof with slope larger than 30° lacks extensive studies. In ASCE 7-10, no provision for hip roofs with slopes above 27° is provided.</td>
<td></td>
</tr>
<tr>
<td>Building Geometry (Height, overhang ratio, aspect ratios)</td>
<td>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].</td>
<td>[66]</td>
</tr>
<tr>
<td>Wind Direction</td>
<td>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].</td>
<td>[52]</td>
</tr>
<tr>
<td>Presence of canopy and parapet</td>
<td>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°</td>
<td>[55]</td>
</tr>
<tr>
<td>Presence of Surrounding Building</td>
<td>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.</td>
<td>[47]</td>
</tr>
<tr>
<td></td>
<td>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].</td>
<td>[62]</td>
</tr>
</tbody>
</table>

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.

Table 2.3. Field wind tests on low-rise buildings

<table>
<thead>
<tr>
<th>Period</th>
<th>Name</th>
<th>Description</th>
<th>Ref.</th>
<th>Validation Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>The 1970s</td>
<td>Aylesbury Experiment</td>
<td>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</td>
<td>[80]-[82]</td>
<td>[57]</td>
</tr>
</tbody>
</table>

(Table 2.3. continued)
<table>
<thead>
<tr>
<th>Period</th>
<th>Name</th>
<th>Description</th>
<th>Ref.</th>
<th>Validation Application</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>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].</td>
<td>[78] [82]</td>
<td>[78]</td>
<td></td>
</tr>
<tr>
<td>The 1980s</td>
<td>Jan Smuts Building Test in South Africa</td>
<td>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].</td>
<td>[68][67] [70]</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Silsoe Structures Building Test</td>
<td></td>
<td>[83]</td>
<td>[77]; [85]</td>
</tr>
<tr>
<td></td>
<td>TTU Building Test in the Texas Tech Wind Engineering Research Field Laboratory</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Florida Coastal Monitoring Program (FCMP)</td>
<td>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&amp;C loads in</td>
<td>[86]</td>
<td>[87]-[89]</td>
</tr>
</tbody>
</table>

(Table 2.3. continued)
<table>
<thead>
<tr>
<th>Period</th>
<th>Name</th>
<th>Description</th>
<th>Ref.</th>
<th>Validation Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>The 2000s</td>
<td>Florida Coastal Monitoring Program (FCMP)</td>
<td>wind tunnel studies with suburban exposure [90]. The turbulence parameters measured by FCMP are used as target parameters to develop WOW flow characteristics [88].</td>
<td>[86]</td>
<td>[87]-[89]</td>
</tr>
</tbody>
</table>

Note: Validation application refers to the studies including wind tests and numerical simulations that are validated by it.

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 2.3. 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]).

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

\[
\frac{\rho l_e V_0}{\gamma P_0 a} \dot{C}_{P_e} + \left( \frac{\rho V_0 U_H}{2 k \gamma P_0 a} \right)^2 \dot{C}_{P_i} = C_{P_i} = C_{P_e}
\]

where \( \rho \) = air density, \( l_e \) = effective length of the opening, \( V_0 \) = internal volume, \( \gamma \) = 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 \( \sqrt{a/V_0} \).

17
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].

2.3.3 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 Ojdrovic [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 \((G \dot{C}_p)\) wherein the gust-effect factor, \(G\), should not be separated. Basically, the actual \((G \dot{C}_p)\) 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 \((G \dot{C}_p)\) 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 \((G \dot{C}_p)\) 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.

**2.3.4 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 \(180^\circ\) range at \(5^\circ\) 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 \((\frac{1}{4}, \frac{1}{2}, 1:12, 1:12, \text{and} 3:12)\) 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%-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.

2.4 Building Representation (FE model)

2.4.1 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 2.4.2.

A handful of studies performed on entire building modeling under wind loads are listed chronologically in Table 2.4. 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. 2.3, 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. 2.3), 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. 2.3) 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.

Table 2.4. Finite element models of full low-rise wood buildings analyzed under wind loads

<table>
<thead>
<tr>
<th>Ref.</th>
<th>Analysis Method</th>
<th>Wind Loading Sources</th>
<th>Focus of Research (footprint/ FTFCs type/ STFCs type/ wind loads/ validation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[119]</td>
<td>SAP2000/linear</td>
<td>Wind tunnel (terrain between open country and suburban) and field measurements</td>
<td>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].</td>
</tr>
<tr>
<td>[15]</td>
<td>SAP2000/nonlinear</td>
<td>ASCE 7-05 MWFRS, Wind tunnel pressures (no exposure documented)</td>
<td>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].</td>
</tr>
<tr>
<td>[16]</td>
<td>SAP2000/linear</td>
<td>Uniform uplift pressure, wind tunnel uplift pressures ([129] with exposure category B), ASCE 7-05 C&amp;C</td>
<td>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.</td>
</tr>
<tr>
<td>[18]</td>
<td>SAP2000/linear</td>
<td>Field measurements; Wind tunnel pressures on open, light suburban, and heavy suburban terrain exposure</td>
<td>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 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</td>
</tr>
</tbody>
</table>

(Table 2.4 continued)
<table>
<thead>
<tr>
<th>Ref.</th>
<th>Analysis Method</th>
<th>Wind Loading Sources</th>
<th>Focus of Research (footprint/ FTFCs type/ STFCs type/ wind loads/ validation)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td>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.</td>
</tr>
<tr>
<td>[126]</td>
<td>SAP2000 /linear</td>
<td>ASCE 7-10 MWFRS</td>
<td>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.</td>
</tr>
<tr>
<td>[120]</td>
<td>ANSYS /nonlinear</td>
<td>Wind tunnel pressures (open terrain)</td>
<td>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.</td>
</tr>
<tr>
<td>[29]</td>
<td>SAP2000 /linear</td>
<td>Uniform uplift pressure, ASCE 7-05 MWFRS</td>
<td>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.</td>
</tr>
</tbody>
</table>

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 not 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 study. In Table 2.4, 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 2.4 are under static wind loads except Zisis and Stathopoulos [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.

2.4.2 Connections

2.4.2.1 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 the 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.

2.4.2.2 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. 2.4) as functions of specific gravity of the wood member, shank diameter, and the yielding mode. The direct comparisons in capacities of STFC withdrawal (Fig. 2.4a), STFC uplift and lateral shear (Fig. 2.4b), and FTFC uplift/shear (Fig. 2.4d) show that the design-code values are generally much lower than measurements. For example, the STFC withdrawal capacity (Fig. 2.4a) specified in NDS is about one third of the test measurements (in Fig. 2.4a, NDS 8d common/ DV (2008) 8d Common average = 0.234kN/0.69kN=34%). Other measurements for STFC withdrawal capacity as shown in Fig. 2.4(a) 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. 2.4(b). For the roof-to-wall connections (RTWC), the measured uplift capacity (shown in Fig. 2.4d) for a 3×12d twisted shank (TS) joint is 3.1 kN by “MG (2011)” while the measured shear capacity for a 1×12d
Note:
NDS 2015 – [141]; RR (1999) – [147];
8d C-Ave – 8d common nail averaged capacity; 3×12d TS-Uplift -- 3×12d Twisted Shank-Uplift.

Fig. 2.4. Constitutive models for (a) STFC withdrawal capacity, (b) STFC shear capacity, (c) STFC rotational capacity, and (d) RTWC uplift/shear capacity

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. 2.4(c). 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. 2.4.
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].

2.5 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 2.5. 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].
Table 2.5. Full-scale and large-scale wood-framed low-rise building test and analysis under wind and selected lateral loads (1990-now)

<table>
<thead>
<tr>
<th>Ref.</th>
<th>Load Form</th>
<th>Focus of Research</th>
</tr>
</thead>
<tbody>
<tr>
<td>[160]</td>
<td>Pointed lateral loads</td>
<td>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].</td>
</tr>
<tr>
<td>[162]</td>
<td>Pointed lateral loads (static, slow destructive cyclic)</td>
<td>A full-scale single-story, L shaped house. Roof diaphragm was found to be 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].</td>
</tr>
<tr>
<td>[161]</td>
<td>Environment wind and artificial distributed and pointed loads</td>
<td>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].</td>
</tr>
<tr>
<td>[164]</td>
<td>Uniform lateral loads</td>
<td>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.</td>
</tr>
<tr>
<td>[22]</td>
<td>Replicated wind pressure time histories on the roof based wind tunnel measurements</td>
<td>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].</td>
</tr>
<tr>
<td>[129]</td>
<td>Open-jet wind loads</td>
<td>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].</td>
</tr>
<tr>
<td>[18]</td>
<td>Environment wind loads</td>
<td>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].</td>
</tr>
</tbody>
</table>

(Table 2.5 continued)
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 2.5, since in the wind load scenario, shear walls resist horizontal thrusts on the windward face and suction on the leeward side.

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.

### 2.6 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 non-straight-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 conductions 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 was with poor experimental validation and
the latter was 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.

2.7 References


[39] ASCE 7-05. Minimum design loads for buildings and other structures. Reston (Virginia): American Society of Civil Engineers; 2005


[50] Leatherman SP, Gan Chowdhury A, Robertson CJ. Wall of wind full-scale destructive testing of coastal houses and hurricane damage mitigation. Journal of Coastal Research 2007; 23 (5); 1211–1217.


Liu Z. Field Measurements and Wind Tunnel Simulation of Hurricane Wind Loads on Single Family Dwellings, dissertation, the Graduate School of Clemson University. 2006.


Holmes JD. Mean and fluctuating internal pressure induced by wind. Proc., 5th Int. Conf. on Wind Engineering, Fort Collins, Colo. 1979; 435–450.


Paevere PJ, Foliente GC, Kasal B. Load-Sharing and Redistribution in a One-Story Woodframe Building. Journal Of Structural Engineering. 2003; 129 (9); 1275-84.


CHAPTER 3. FINITE-ELEMENT MODELING FRAMEWORK FOR PREDICTING REALISTIC RESPONSES OF LIGHT-FRAME LOW-RISE BUILDINGS UNDER WIND LOADS

3.1 Introduction

In the United States, light-frame wood buildings account for over 95% of all residential structures, and the majority of these buildings are designed as low-rise buildings [1]. These wood structures are not rigorously constructed, which follows the prescriptive requirements of the local building code rather than being fully analyzed and engineered [2]. As a result, poor performance of these non-engineered buildings has been witnessed during the past hurricane events (e.g., [3]-[6]). Low-rise buildings thus represent one of the most vulnerable structures under extreme wind events and are the largest source of the damage and fatality directly and indirectly inflicted by extreme weather events such as hurricanes, tornadoes, and storm surges [7]-[8]. As reported by the National Association of Insurance Commissioners, hurricane-induced catastrophes account for seven out of the top ten costly insured property catastrophes in the U.S. [9]. Compared to wind analysis and wind design of high-rise buildings, which are often informed by extensive wind tunnel studies, the work dedicated towards the low-rise building performance under winds, in stark contrast, falls behind. Meanwhile, the wind performance analysis of light-frame low buildings lags far behind the seismic analysis for such structures that employs finite-element (FE) modeling technique. This is partly due to the complexity of modeling the flow around low-rise buildings and the effect of oncoming turbulence due to the terrain roughness. Without detailed wind tunnel testing, which is mostly done for tall buildings (due to economic reasons), it is challenging to predict the realistic wind-induced effects on low-rise buildings which have large variation in geometries, including different shapes, roof types, and roof slopes that influence flow separation, reattachment, and vortex formations. Fluctuations in surface pressures and dynamic load transfer within the structural system are also important factors that can produce dynamic effect in terms of load attenuation for low-rise buildings, as confirmed by Zisis and Stathopoulos [10]. Overall, the estimation of realistic wind-induced effects (e.g., pressures, forces) and responses (e.g., deflections, strains) for low-rise buildings poses significant challenges due to the complex flow-structure interaction and strong temporal and spatial variations of wind loading. Thus, evaluation of realistic wind performance of and risks to low-rise buildings deserves more attention in light of the urgent need to reduce significant losses to these non-engineered structures during wind events, as will be discussed next.

The prediction of hurricane losses is of compelling interest to insurance companies besides the federal, state and local government who are responsible for enacting policies for reducing the vulnerability of infrastructures, e.g., DMA2K—Law 106–390 [11]. The insurance industry demands the assessment of structural performance under hurricanes in a quantitative way that is measured by damage ratio (repair/replacement costs) over a wind speed range, which involves a series of nonlinear deformations and progressive failures greatly influenced by building configurations. As such, some full-scale wind engineering laboratories are constructed, e.g., the Insurance Institute for Business & Home Safety (IBHS) Research Center, to provide more realistic predictions of the structure performance with realistic building configurations under high winds [12]. Direct experimental studies on low-rise wood buildings under winds are considered as one of the most reliable ways to analyze the structural behavior for more effective designs. However,
due to the high cost, these full-scale wind tests are performed for limited number of structures only, and their main objective is supposed to serve as the validation of numerical models, as indicated by the committee of National Research Council [13] after reviewing the need for the large-scale test facility. Furthermore, even the wind tunnel tests on scaled building models for specific designs are rare and only possible for big projects due to cost issues.

In light of the above discussions, it is important to reduce future reliance on physical model testing by developing numerical modeling approaches to predict performance of light-frame wood houses subjected to wind loads. This numerical modeling should be validated to be capable of predicting all of the critical locations, structural behaviors, and load paths over the entire failure process and apply simplifications without significantly affecting the results so as to reduce computational effort. The wind loads, applied to the surface of a structure determined by the eddy-structure interactions with building geometries, tend to follow the load paths governed by structural configurations through the relative stiffness and locations of individual components [14]. Trautner and Ojdrovic [15] found that the critical structural actions and locations of one building may not be the same for other buildings even with a single difference in bracing schemes. In light of this, more emphasis should be put on the modeling accuracy of the building configuration that determines how structures behave or deform, which further leads to a different sequence of failure modes. Observations from wind damage reconnaissance events revealed that the main source of damage in residential houses was the lack of continuous uplift load path from the roof down to the foundation [6]. Along the load path, the roof-to-wall connection (RTWC) [6]; 16; [17] and sheathing-to-truss connection (STTC) [3]; [8]; [17] are identified as the most critical weaknesses. Therefore, a higher resolution of numerical modeling on the building configuration that incorporates the behavior of inter-connections contributes to a more accurate prediction of structural responses and failure sequence along with the failure modes.

He et al. [18] reviewed and discussed the paucity of the available complete numerical low-rise building models under wind loads, and cited the research work by Zisis [19], Asiz et al. [20], Martin et al. [24], Zisis and Stathopoulo [10], Malone et al. [21], Pan et al. [22] and Pfretzschner et al. [23]. The building models of Martin et al. [24] and Pfretzschner et al. [23] were validated by comparing responses through three levels such as the deflections in 2D individual truss, load and deflection sharing in 3D truss assembly, the correlation of in-plane stiffness of the wall system and nail spacing in a 2D shear wall, and influence functions in 3D complete houses. In their models, sheathing gaps and sheathing nails were ignored by assuming the continuity of sheathing panels and incorporating the effects of sheathing nails into wall and roof by adjusting the shear modulus of the sheathing, respectively. Such modeling method was later adopted by Malone et al. [21]. However, the capability of their linear modeling methods that is sufficient for predicting lateral and vertical load paths is within the elastic range as concluded by Martin et al. [24], and the behavior of the critical location in the wind loading case on the inter-component connections such as sheathing nails cannot be captured.

Asiz et al. [20] and Pan et al. [22] both developed a 3D building model that is capable of capturing the sheathing nail behavior by adopting nonlinear elements simulating the translational deformation in three global directions. Asiz et al. [20]’s model considered all the inter-component connections including anchor bolts to the foundation, sheathing-to-frame nails, and framing-to-framing nails with special cares on the RTWCs by using nonlinear link elements. This numerical
model was calibrated by a full-scale wind test on a simple box-type light-frame wood structure where only three sides, i.e., two front wall and the whole roof surfaces, were applied with corning wind loads by using pressure loading actuators (PLAs) with load traces derived from wind tunnel tests. Despite the nonlinear capability, their model was only well validated at low loading level within the linear range rather than at high loading level, attributed to the exclusion of material nonlinearities of frame and sheathing members, as well as neglecting geometric nonlinearity induced by the large deformations that were observed before failures [20]. Pan et al. [22] incorporated the geometric nonlinearity into their modeling analysis and made one of the earliest attempts to address the progressive failure issue by utilizing one piece of sheathing panel on the roof and identifying seven failure modes. The failure modes include three capacities of STTCs regarding the withdrawal, pull-through, and load-slip, and four sheathing capacities on axial stress, shear stress, bending stress, and displacement thresholds. Pan et al. [22]’s modeling method with the geometric nonlinearity will be followed and validated in the current study.

Zisis and Stathopoulos [10] addressed the dynamic effect of the light-frame construction subjected to wind loads on the complete building level benefited from the NSERC Collaborative Research and Development (CRD) Project entailing the field monitoring with heavily instrumented full-scale facilities and special costly laboratory accommodations [19]. Taking advantage of the load cell mounted on the frame, roof, and foundation, the structural attenuation, a phenomenon incorporated in the National Building Code of Canada, was experimentally justified for the first time. To investigate the structural attenuation of the wind-induced internal force flow, a simple 3D linear elastic building model was created and applied with wind loading time series, of which the predicted forces were treated as a baseline and compared with the monitored values. The predicted uplift force at the RTWCs and foundation level in the form of correlation plots and reduction factors showed higher values. Based on that, they addressed that the estimation of building performance would tend to be conservative based on static analysis of structural systems inherent in the current wind design practices. This FE model was validated by comparing the distribution of uplift foundation forces under static analysis.

The current complete FE models subjected to wind loads remain validated within linear elastic range, and challenges exist with respect to the modeling technique and loading sources. The models involved significant simplification on the semi-rigid connection modeling with linear material property assumptions. Moreover, the spatiotemporally varying wind loading is usually simplified and applied in the form of static, slow increasing ramps or very basic cyclic loading. These issues have been greatly improved in the FE models at the roof structural level with advanced modeling techniques except specimen boundary conditions [2]; [25]-[27]. Jacklin et al. [2] modeled a roof structure to connected to top plates, which was witnessed to make the large contribution to the overall loss of building [4]; [6] while neglecting the wall system. They assumed that the wall beneath the RTWC had negligible effects on the deformation and reasoned that the wall members would experience little axial deformation under the magnitude of applied loading [2]. However, such an assumption of simplified boundary conditions of the roof structure precludes the prediction of failure modes related to the RTWCs, the critical components for low-rise residential buildings. So far, little-to-no studies exist to justify whether the valuable information derived from these advanced individual roof structure modeling can be applicable to the complete building structure. As stated above, the general challenges remaining in the computational modeling of the wind performance of low-rise buildings are to maintain both the high resolution
of a nonlinear model and the dynamic wind loading at whole building scales (i.e., modeling the entire structure). Additionally, a long-standing challenge is the lack of effective validation for the numerical modeling over the entire failure process based on results of large- or full-scale physical modeling in wind tunnels.

In order to address the current challenges in the numerical modeling, this paper presents a computational modeling methodology that can help determine the realistic load paths and load sharing of light-frame low-rise buildings under wind loadings throughout the linear to the nonlinear range. Realistic spatiotemporally varying wind pressures were measured on a large-scale (1:4) physical model of a prototype building using the NSF NHERI Wall of Wind (WOW) Experimental Facility (EF) at Florida International University. The WOW experiments also helped in obtaining deflections at key locations of the building system for the modeling validation from linear to nonlinear ranges until failure. A three-dimensional finite-element (FE) model of the whole building was developed using the same dimensions of the 1:4 physical model. The structural materials and connections, as used in the physical model (i.e., oriented strand board sheathing, 6-d and 8-d sheathing nails, framing-to-framing connections, roof-to-wall hurricane clips), were numerically modeled using mechanics-based load-deformation characteristics including non-linearity. Spatiotemporally varying pressure time histories, obtained based on the WOW physical model testing, were applied on the roof and wall surfaces of the numerical model as dynamic wind loading input. The numerical modeling was used to assess the performance of the structure, specifically the wind-induced responses of the most vulnerable components such as sheathing panels on the roof and connections such as RTWCs. The predicted structural responses of the computational framework showed reasonable agreement when compared to the experimental measurements in terms of the deflections at the roof sheathings and RTWCs. The validated numerical modeling framework is then used to analyze the effects of various modeling techniques, including the modeling of RTWCs and the foundation fasteners relevant to the boundary conditions in the simulations of the wind-induced responses of the roof assembly and the complete building, respectively. The effect of wall stud to building envelope connections is evaluated, and the rotational capacity of sheathing nail is discussed, which is critical to the failure of the roof sheathing that has been witnessed as one of the most common failures in light-frame wood houses during extreme wind events.

3.2 Description of the Conducted Experiment

An experiment on a 1:4 scaled model (in building overall geometry) of a gable roof prototype building was conducted at the Wall of Wind (WOW) facility at Florida International University (FIU) to serve for the validation of numerical modeling methods and explore the failure modes and progressive damages of residential houses under extreme wind events. Simultaneous measurements of wind loading and the responses it induces on envelopes, frame members and connections of the low-rise wood frame building model were conducted. The building model was exposed to both external and internal wind pressures, and sheathing uplift deflections and uplifts at RTWCs were measured. As will be described later, the experimental results helped in validating the numerical models, in which time histories of external and internal wind pressures were fed as input loading and the output responses were compared with those from the experimental responses under various wind speeds and wind directions.
The WOW (Fig. 3.1) is an open jet facility capable of producing hurricane strength wind speeds. It has been designated as one of the Experimental Facilities (EFs) under National Science Foundation’s (NSF’s) Natural Hazards Engineering Research Infrastructure (NHERI) program [43]. The NHERI WOW EF has 12 electric fans arranged in a two-row by six-column pattern, which produces a wind field of 6 m (19.7 ft) wide and 4.3 m (14.1 ft) high, allowing aerodynamic testing of large-scale models or full-scale portions of small buildings. Fig. 3.2 shows the mean wind speed profile (with target ABL of $\alpha = 1/6.5$) and turbulence intensity profile as used for the current testing.

Fig. 3.1. Wall of Wind (WOW) facility side at Florida International University (FIU): (a) Intake side and (b) Outlet side

Fig. 3.2. Simulation of open terrain ABL in the WOW: (a) mean wind speed profile (b) turbulence intensity profile

The tested structure was built to one of the generic low building models covered in the National Institute for Standards and Technology (NIST) aerodynamic database. This model is rectangular in plan, with a scaled footprint of 3.57 m (11.72 ft) in ridge direction by 2.29 m (7.50 ft).
ft) cross, and roof slope 14° for a scaled eave height 0.91 m (3.00 ft) (Fig. 3.3). A typical door was modeled on one of the longer side of the model to simulate an opening to evaluate the effect of internal pressurization during breach of the opening. Spruce-pin-fir #2 lumber was used for wall stud and roof truss members. Oriented strand board (OSB) of 11 mm (7/16 in.) thickness was used as wall and roof sheathing materials. Typical framing, sheathing and nailing details for the model walls and roof are shown in Fig. 3.4.

Fig. 3.3. Validation experiment on the 1:4 scale building model used in the current study at WOW, FIU

Fig. 3.4. Connections: (a) roof to wall connections using hurricane ties (b) gable end truss, ridge and roof sheathing connections (c) interior trusses to roof ridge member connections

Simultaneous measurements of external and internal time-histories of pressures, deflections and strains were conducted during the entire period of testing. The locations of key instruments are shown in Fig. 3.5. The test model was instrumented with 352 external pressure taps and 4 internal pressure taps (one on each wall and 2 near the roof deck) to measure time-varying aerodynamic pressures. Displacements of roof sheathings were measured using the linear variable differential transformer (LVDTs) (Fig. 3.6a). Upward displacements of roof to wall connections at all interior frames was measured using Celesco SP-2 string pots, placed on rigid pieces of lumber attached to the top plates as shown in Fig. 3.6(b).
Each test was conducted for a duration of 90 sec. Time-histories of wind pressures were collected at the sampling rate of 520 Hz using 6.35 mm ID polyurethane pressure tubes and a DSA4000, ZOC 33 Scanivalve data acquisition system. After the data were collected necessary corrections for pressure tubing length were performed by experimentally estimating the transfer function of the tubing system, and the data were low-pass filtered at a frequency of 100 Hz. Simultaneously, structural responses including the vertical deflections of the roof sheathings at the geometric center between trusses, and the vertical displacements at the RTWCs were measured by five LVDTs and six Celesco SP-2 string pots, respectively. The sampling frequency of all these structural response measurements was 100 Hz.

The building model without any opening was subjected to flows with the wind speed of 29.06 m/s (65 mph), 40.68 m/s (91 mph), and 46.94 m/s (105 mph) which falls into the category of tropical storm, Cat. 1 Saffir-Simpson hurricane scale (SSHS), and Cat. 2 SSHS, respectively over a range of 0° to 180° wind directions, considering symmetry, with 15° intervals. The model was also tested with a dominant opening to induce internal flows under the wind speed of 29.06 m/s (65 mph) and the same range of wind directions. These tests are marked as Test 1-Test 4 (see
Table 3.1). Sample results of mean deflections of the roof sheathing for various wind speeds, wind directions, and test cases (with the door in closed and open positions), as measured by the LVDTs, are shown in Fig. 3.7(a). The corresponding results of RTWCs, as measured by the string pots, are shown in Fig. 3.7(b). The results of pressures and responses were used to calibrate and validate the FE modeling framework (as described later).

Table 3.1. Testing plan

<table>
<thead>
<tr>
<th>TEST 1</th>
<th>Wind Directions</th>
<th>Opening</th>
<th>Wind Speed</th>
<th>Measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>0° to 180° (15°)</td>
<td>None</td>
<td>29.06 m/s (65 mph)</td>
<td>LVDT, SP, Cp</td>
<td></td>
</tr>
<tr>
<td>TEST 2</td>
<td>0° to 180° (15°)</td>
<td>Front Door</td>
<td>29.06 m/s (65 mph)</td>
<td>LVDT, SP, Cp</td>
</tr>
<tr>
<td>TEST 3</td>
<td>0° to 180° (15°)</td>
<td>None</td>
<td>40.68 m/s (91 mph)</td>
<td>LVDT, SP</td>
</tr>
<tr>
<td>TEST 4</td>
<td>0° to 75° (15°)</td>
<td>None</td>
<td>46.94 m/s (105 mph)</td>
<td>LVDT, SP</td>
</tr>
</tbody>
</table>

Fig. 3.7. (a) LVDT measuring sheathing displacement; (b) SP measuring displacement at RTWCs

(Fig. 3.7 continued)
(Fig. 3.7 continued)
Roof failure occurred under the wind direction of 75° and 46.94 m/s (105 mph) wind speed (under the closed condition). The roof sheathing started vibrating under the turbulent wind load, which created an opening between the wall and roof sheathings. Wind was able to leak inside through this opening and amplified the internal pressure leading to failure of the roof sheathings. Both nail pull-out and nail pull through type of failures were observed. The roof sheathing then tore up and became flying debris, landing more than 15m (50 ft) away from the model. The entire failure sequence was captured using video recording and is shown in Fig. 3.8.

Fig. 3.8. Sequence of the progressive failure
3.3 Numerical Modeling of Wind Effects on the Low-rise Building

The experiment building is numerically modeled by using the finite element program Mechanical APDL [29], and each component of the structure is represented directly by the ANSYS’ built-in elements. In the current study, the direct generation method is adopted to create the building model, a way that is easier to determine the location of every node and keep track of all the node numbers. Especially for the nail element that is developed from two coincident nodes, e.g., every sheathing nail element is composed of two nodes at the same location of which one node on the beam element and the other node on the shell element, the control over the node numbering and location guaranties the correct creation of nail elements. In this modeling method, the selection of mesh density and mesh algorithm involves several factors such as the adaptability to the triangle-shaped sheathing at the end truss and the discontinuity at doors, in which nail spacing and stud spacing are of the first priority. Given the complexity of the 3-D model and the time-consuming nature of the direct generation method, a program which is capable of creating a set of sheathing-unit nodes has been developed to adapt to versatile building geometries, simplify the tedious repetition work, and make it possible to be applied to general building models. As such, shell and beam elements have an approximate area of $6 \times 6$ in$^2$ and length of 6 in, respectively, in accordance with the nail schedule in the experiment model, and a total of 932 shell elements and 434 beam elements as well as 1753 spring elements are employed to model the experiment building model (Fig. 3.9).

Fig. 3.9. FE model of the experiment building with definition of global coordinate (a) meshing of framing (b) meshing of sheathing

The generally accepted modeling techniques are adopted by the current numerical model such as the selection of element to represent sheathing and beam, the real sheathing arrangement modeling, and the simplification assumption on the truss connections. In order to explore the general modeling method of each component in the structure valid in the nonlinear realm, the techniques that would yield a more accurate simulation suggested by the previous research are also adopted such as the geometric nonlinearity. A review of the generally accepted FE modeling of each component is referred to He et al. [18]. Other modeling methods not covered in the literature are also discussed herein including wall stud connections, STTCs, and RTWCs to explore the influence of the moment transferred at wall studs, the rotation capacity of STTCs, and the effect of lateral translations and rotations of RTWCs. The modeling methods discussed are listed in Table 3.2 with the validation models being Case 2 and Case 3 with the highest resolution, and the modeling of each involved member in the FE model is detailed in the following sections. Additionally, a model, named Case 0, is added to the validation with simplifications on both the
modeling and analysis method serving as a baseline, wherein all the components are rigid connected to each other, and the geometric nonlinearity is ignored. The following sections provide details of modeling various components and connections.

Table 3.2. Validation model cases (model classified by connectivity and analysis method)

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Wall Stud Connections</th>
<th>STTCS (^a)</th>
<th>Geometric Nonlinearity</th>
<th>RTWCs (^b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 0</td>
<td>Rigid</td>
<td>Rigid</td>
<td>No</td>
<td>Rigid</td>
</tr>
<tr>
<td>Case 1</td>
<td>Rigid</td>
<td>3 DOF</td>
<td>Yes</td>
<td>1 DOF</td>
</tr>
<tr>
<td>Case 2</td>
<td>Rigid</td>
<td>6 DOF</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Case 3</td>
<td>Pinned</td>
<td>6 DOF</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\(^a\)Roof is the vulnerable part, so the 6 DOF is only applied on sheathing connections on the roof to check the rotational effects.

\(^b\)Only the vertical translational nonlinear DOF is considered in the validation model; other modeling ways of the RTWCs are named as c1-c5 and discussed in later section.

3.3.1 Structural members

Sheathing panels, i.e., 7/16 in. Oriented Strand Board (OSB), are modeled by using shell elements to involve the system effects, and the sheathing arrangement (sheathing gap) of the test building model is incorporated with the discontinuity of shell elements between panels. The actual dimensions of 7/16 in. OSB sheathing panels are used in the FE model. A total of ten OSB sheathing panels are installed, of which two on each side of the roof, two on the side walls (front and back), respectively, and one on each of the gable wall. The roof covering such as the wood shakes or shingles was ignored which is also consistent with the experiment model. The sheathing element chosen to represent the sheathing panels with in-plane and out-of-plane behaviors is an 8-noded linear-elastic orthotropic quadrilateral shell with six degrees of freedom at each node. The thickness of the element is constant according to the dimension of selected sheathing available in the market. The mass of the element is given as the mass per unit area. The sheathing as presented here is capable of accommodating uneven settlement caused by the deformation of the frame under the wind loads.

All the frame members in the numerical model such as wall studs and trusses are represented by 3D linear isotropic beam elements which is 2-node with six degrees of freedom on each node, i.e., translation and rotation about the three mutually perpendicular axes, and good for large deflection analysis. 2 × 3 (1.5 in. × 2.5 in.) size SPF No.2 lumber is used for all the frames in the experiment, of which the lumber is doubled at the top plate of the wall and tripled as the wall stud on each corner of the building model. These multi-layers of plates in the experiment are also simplified into a single layer with a cross section equal to the sum of the individual cross sections of the framing members, such as the substitution of applying one 3 in. × 2.5 in. beam element for two 1.5 in. × 2.5 in. beam elements assuming that they are fully composite. The remaining framing members are selected as 1.5 in. × 2.5 in. beam elements with different directions on each face of the building. It is noteworthy that the actual size of wood productions is different from their nominal size since the lumber, a “rough cut” from the logs, shrinks after the wood dries.
from green and wet, and then would be smoothed on the surface and made to a uniform size. Thus, the size of the wood products determined by the nature of the wood material cannot be as accurate as steel products, resulting in the large variation in the wood member behaviors. Therefore, in the current study, the actual cross areas of frame members and the actual thickness of 7/16 OSB sheathing panels are adopted in the FE model as given in Table 3.3.

<table>
<thead>
<tr>
<th>Component</th>
<th>Nominal Size</th>
<th>Actual Size</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>2 × 3</td>
<td>1.5 × 2.5</td>
<td>38.1 × 63.5</td>
</tr>
<tr>
<td>Beam</td>
<td>Three 2 × 3</td>
<td>3 × 3</td>
<td>76.2 × 76.2</td>
</tr>
<tr>
<td>Beam</td>
<td>Double 2 × 3</td>
<td>2.5 × 3</td>
<td>63.5 × 76.2</td>
</tr>
<tr>
<td>Sheathing</td>
<td>7/16</td>
<td>0.5</td>
<td>12.7</td>
</tr>
</tbody>
</table>

The structure contains three interior fink style trusses spaced evenly on center with four web members connecting the top chord to the bottom chord of each truss, and two exterior trusses on the gable end with all five of the vertical webs. The truss members including top chords, bottom chords, ridge beams, and webs are connected by 16d common nails, and the modeling method of these connections follows that of Martin et al. [24] by using a rigid connection at the heel of the truss and pinned ones for the rest. Wall studs on the long side are spaced at an approximate distance of 430 mm (17 in) and around 457 mm (18 in) apart on the short side. They are end nailed to the top and bottom plates with two 8d common smooth nails, respectively, and considered to pinned or rigid at both ends in different validation cases to explore whether the moment should be released at the wall members based on the connection condition of the experiment as concerned by Li et al. [30]. This connection is rarely studied and has not yet come to a generally accepted modeling method since it is not as vulnerable as the roof structure. The top plate-to-plate connections at the corners of the wall of the building are pinned.

### 3.3.2 Inter-component connections and boundary conditions

Standard ANSYS zero mass nonlinear spring elements [29] are used to represent the inter-component connections of roof-to-wall and sheathing-to-frame and to track the behavior of the structure at each stage of failures. These spring elements with two coincident nodes account for the force-displacement relationship in each of the principal directions or rotations as shown in Fig. 3.10, where the two coincident nodes at the same location are separated for clarity. The relationship is characterized by piecewise multi-linear segments with symmetric or asymmetric behavior, rotational or translational degree of freedom, and capability of large displacement analysis, allowing for flexibility in modeling different mechanisms such as shear, withdrawal, and rotation.

The H3-18 Gauge hurricane clip as shown in Fig. 3.10(b) composed of 6d common nails is used as the RTWC connecting the top chord of the truss to the top plate of the wall. Based on the previous research as discussed above, one nonlinear spring element in the vertical direction is applied in the FE model for each RTWC to account for only the axial uplift capacity rather than the pinned or rigid connection. Then, the effect of the nonlinear lateral and rotational capacities of the RTWCs are discussed to explore whether they can be simplified later. For the sheathing
fastener, 6d common nails are used in the experiment model building to connect the sheathing to the frame with a conventional nailing schedule of 6 in. /12 in. along the exterior panel edge (edge nailing) and the intermediate supports (field nailing). Then, the roof edge nailing along the side walls are intentionally eliminated to weaken the load path and sharing to observe failure. Each nail connecting the wall sheathing panels to the wall is modeled by three nonlinear spring elements with translational load-displacement relationship in each of the global direction; while the sheathing nail on the roof (STTCs), the critical component in the vertical load path, is simulated by either three or six nonlinear spring elements in different validation models to verify if the rotation DOF can be neglected.

![Diagram of nail connection](image)

**Fig. 3.10. NL-spring element of nail connection: (a) a demonstration of connections (b) a photo of the RTWC of FIU model**

The test building model is connected to the foundation with 16 anchor bolts of which 4 bolts on each side of the building are evenly distributed throughout the length of the sole plates. Rigid assumption is made on these connections, and no deformation was witnessed during the experiments. The uplift wind loads are smaller than or around the structure’s self-weight throughout the tests as shown in Table 3.7 later.

### 3.3.3 Material property of linear element

Beam elements, which represent the wall and truss members, are modeled using elastic isotropic material properties (Table 3.4); shell elements representing wall and roof sheathing panels are modeled by using elastic orthotropic material properties (Table 3.5).

**Table 3.4. Elastic isotropic material properties—frame members**

<table>
<thead>
<tr>
<th>Species /Size</th>
<th>Modulus of Elasticity (^a) (10^6) psi</th>
<th>Shear Modulus (^b) (10^6) psi</th>
<th>Poisson’s Ratio (^c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPF, 2 in ×3 in</td>
<td>1.2</td>
<td>0.4286</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td>8,274</td>
<td>2,955</td>
<td></td>
</tr>
</tbody>
</table>

\(^a\)Modulus of elasticity value from NDS code [32]  
\(^b\)Shear modulus value computed from function: \(G = \frac{E}{2(1+\mu)}\)  
\(^c\)Poisson’s ratio from Wood Handbook [33]
Table 3.5. Elastic orthotropic material properties—sheathing panels

<table>
<thead>
<tr>
<th>Species/Size</th>
<th>Modulus of Elasticity(^a), (10^5) psi (MPa)</th>
<th>Shear Modulus(^a), (10^5) psi (MPa)</th>
<th>Poisson’s Ratio(^b)</th>
<th>Density(^a), kg/m(^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(E^1) (E^2) (E^3) (G_{12}) (G_{13}) (G_{23}) (\mu_{12}) (\mu_{13}) (\mu_{23})</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7/16 OSB</td>
<td>7.4 (5,100) 2.3 (1,590) 2.3 (1,590) 1.1 (790) 1.1 (790) 1.1 (790) 0.08 0.08 0.08</td>
<td></td>
<td></td>
<td>600</td>
</tr>
</tbody>
</table>

\(^a\)Modulus of elasticity, shear modulus, and density values from Doudak [34]  
\(^b\)Poisson’s ratio from Kasal [35]

3.3.4 Material property of nonlinear element

Components simulated with nonlinear properties include RTWCs and sheathing nails as illustrated in Table 3.6. A multi-linear force-deflection relationship of hurricane clip with an initial stiffness of 2.7 kN/mm is applied to the current numerical model to simulate the behavior of the RTWC as shown in Fig. 3.11. The positive (tension) part of this connection property is adopted from the experiment work presented by Riley and Sadek [38] in NISTIR 6938 on the uplift strength of the same type of hurricane ties, H3. However, their hurricane ties were mounted with 8-8d common nails and two additional toe-nails were used for erection, which may result in higher strength than the hurricane tie with only 8-6d common nails used in the current study. Due to the paucity of the material property of the hurricane ties in the literature compared with amounts of experiments conducted on the toe-nail type RTWCs, their results are still used herein with the similar configuration, and the predictions of the RTWC deformation are expected to be larger than the measurements in the wind test. This high-value relationship is also used for the in-plane, out-of-plane strength to explore the appropriate modeling method with an accuracy in balance with simplicity. A high stiffness value (106.8 kN/mm) derived from Winkel [39] is assigned to the negative (compression) part representing the truss bearing on the top plate of the wall.

Table 3.6. Nonlinear material properties

<table>
<thead>
<tr>
<th>Item</th>
<th>Type</th>
<th>Translation</th>
<th>Rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Axial</td>
<td>Lateral</td>
</tr>
<tr>
<td>RTWC(^a)</td>
<td>Simpson Strong-Tie H3 with 8-6d Common Nail</td>
<td>Fig. 3.11</td>
<td>N/A</td>
</tr>
<tr>
<td>STTC(^b)</td>
<td>6d Common Smooth Shank Nail</td>
<td>Fig. 3.12 (a)</td>
<td>Fig. 3.12 (b)</td>
</tr>
</tbody>
</table>

\(^a\)Load-displacement relationship of RTWCs in three directions from Riley and Sadek [38]  
\(^b\)Load-displacement relationship of STTCs in three directions from Dao and van, d. L. [31] and Kent et al. [40] also applied to the sheathing nails on the wall.
The withdrawal and lateral load-displacement relationships of sheathing nails are determined using the study of Dao and van de Lindt [31] and Kent et al. [40], respectively (Fig. 3.12a); the rotational load-displacement relationships of STTCs are derived from the experiments conducted by Dao and van, de Lindt [31] (Fig. 3.12b). It is pointed out that Dao and van, de Lindt’s [31] experiments condition is different in the nail size (8d-box nails) from the current study (6d common nails). However, since the withdrawal strength is determined by the specific gravity of the member holding the nail point and the contact area in terms of shank diameter as indicated by NDS [32], it is still reasonable to adopt the strength of 8d-box nail as that of 6d common nail due to their very close diameter of 0.113 in.

The time-history pressures measured from FIU wind tests are of desired resolution attributed to the large-scale test model. Since the loading resolution largely depends on the number of available pressure taps based on the equivalent area principle, the current model is equipped with high density pressure taps, i.e., 352 external pressure taps. The loading grid is determined directly by the number of pressure taps. Each pressure time history is used to represent each of the equivalent areas without further area averaging so as to reflect all the fluctuations as measured by
the pressure taps. Totally, pressure time histories recorded by the 356 taps including 4 internal pressure taps are applied to the FE model.

Two types of load distributions are selected to validate the numerical model including the time-averaged mean value of each wind loading cases and the time-history loading under the wind direction normal to the roof ridge, i.e., $90^\circ$. The time-history variations of the sheathing deformation are also presented. Building weight and inertia response are considered in the numerical model to yield more accurate predictions at higher wind levels, where the nonlinear behavior of the structure has occurred, and the responses such as the deflection of roof sheathing would experience a change of direction after the wind loads are applied to reflect the real process of failure. The total mean uplift forces acting on the building model under all the test conditions are illustrated in Table 3.7. Considering the symmetry of the structure, only the global uplift forces under wind directions of $0^\circ$–$90^\circ$ are presented. As can be seen, the global uplift force acting on the current structure is negative for most wind loading cases showing that the weight of this building, approximately 5.34 kN, is larger than the uplift force induced by the wind loads and the weight of the structure cannot be ignored in the analysis to reflect the compression condition of the anchors.

Table 3.7. Mean global uplift force (Unit: kN)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Wind Speed</th>
<th>Opening</th>
<th>Wind Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wind Speed</td>
<td></td>
<td>0°</td>
</tr>
<tr>
<td>1</td>
<td>29 m/s (65 mph)</td>
<td>w/o</td>
<td>-4.62</td>
</tr>
<tr>
<td>2</td>
<td>29 m/s (65 mph)</td>
<td>With</td>
<td>-4.49</td>
</tr>
<tr>
<td>3</td>
<td>41 m/s (91 mph)</td>
<td>w/o</td>
<td>-3.93</td>
</tr>
<tr>
<td>4</td>
<td>47 m/s (105 mph)</td>
<td>w/o</td>
<td>-3.46</td>
</tr>
</tbody>
</table>

The mean pressure distribution for wind direction $75^\circ$ is selected to display the pressure distribution with and without opening, as shown in Fig. 3.13 and Fig. 3.14, respectively. The pressure displayed includes internal and external pressures, where the positive value represents the loads acting out of the surface, and the negative value means that the loading direction is on to the

![Fig. 3.13. Mean pressure distribution on the building w/o opening (wind speed=65mph, wind direction=75°) (a) total pressure (Maximum=631.61 Pa, Minimum=-297.97 Pa) (b) external pressure only (Maximum=618.91 Pa, Minimum=-310.67 Pa)](image-url)
surface. In general, while the internal flows induced by the opening on the windward wall show small effects on the pressure distribution pattern on the building, they change the magnitude of not only the internal but also the external pressures. As revealed in Fig. 3.13 (b) and Fig. 3.14 (b) where only the external pressures are plotted, the opening magnifies the uplift forces on the other sides of the building, i.e., approximately 1.5% increase (from 618.91 to 628.48 Pa) in the peak value and weakens the wind force on the windward wall, i.e., around 12.3% reduction (from 310.67 to 272.50 Pa) of the maximum value. The flow rolling up onto the leading edge of the roof where only the external pressures are plotted, the opening magnifies the uplift forces on the other sides of the building, i.e., approximately 1.5% increase (from 618.91 to 628.48 Pa) in the peak value and weakens the wind force on the windward wall, i.e., around 12.3% reduction (from 310.67 to 272.50 Pa) of the maximum value. The flow rolling up onto the leading edge of the roof evolves with more vortices caused by the separation of the flow with the opening resulting in larger and more variability in the values of pressures on the roof; the decreased pressures also caused by the separation of the flow around the opening together with the increased internal pressure greatly reduced the force on the front wall, making the front wall safer with the occurrence of opening.

3.5 Model Validation

3.5.1 Comparison of time-averaged response

The time-averaged deformations of roof sheathing panels and RTWCs under the four test conditions illustrated in Table 3.7 are measured by using the LVDTs (Fig. 3.15-Fig. 3.16) and SPs (Fig. 3.17-Fig. 3.19), respectively. Due to the space limitation, not all the loading cases and results are presented here. In each figure, three or four results are compared. They are the experimental measurements providing the realistic structural behavior, the Case 0 values serving as the baseline and predicted from the simplest model of which all the elements are linear and rigidly connected to each other, and the Case 2 and Case 3 values derived from the FE models built on the proposed modeling methodology with the only difference in wall stud connection modeling.

For the sheathing flexible deformation, the magnitudes on the side of the roof (i.e., LVDT 1 and LVDT 3) is presented where is regarded as the most vulnerable position based on the high-
pressure coefficient (Fig. 3.15-Fig. 3.16). There is a strong agreement between the measurements and predictions in the trend of the time-averaged deflection of roof sheathing under each wind direction. For the LVDT 3 on the corner of the front roof (Fig. 3.15a and Fig. 3.16), both the FEA predictions and the experiment measurements decrease when the wind direction changes from 0° with wind directions parallel to the roof ridge to a small angle of about 30° and slowly increase when the angle changes to 90° with wind perpendicular to the roof ridge. After that, with leeward wind pressure, the LVDT 3 values decrease until 180°. Compared with LVDT 3, the deflections of LVDT 1 (Fig. 3.15a) on the corner of the back roof show more variations caused by separations at the ridge or gable corner for oblique winds. Both the numerical results and experiment measurements show that the roof sheathing at LVDT 1 experiences less deflection under wind direction 15° than other directions closed by, and the deflection slowly goes down when the wind direction changes from 30° to 90°, then after a local maximum value at 105°, the deflection continue to decrease until 180°.

For the baseline Case 0 predictions with linear assumptions, they are reasonably smaller than the experiment measurements and yield the lowest values. Under the lower wind pressure condition, i.e., wind speed of 65 mph or leeward side, the experiment measurements are closer to the Case 0 predictions, indicating that the behavior of the low-rise building captured by the linear numerical model works very well under the low wind speed in terms of the sheathing deformation (Fig. 3.15a and Fig. 3.15b). When the model subjected to a higher wind speed at which the failure occurred, i.e., 105 mph, the extent of the underestimation produced by Case 0 greatly increases due to the nonlinear behavior of the structure subjected to higher wind loading, i.e., from 3% under 65 mph to 36% under 105 mph wind from 0° for LVDT 3, as shown in Fig. 3.15(a) and Fig. 3.16(b). Therefore, under the extreme wind events such as hurricanes, the linear behavior regarding the sheathing displacement predicted by Case 0 is too conservative to guarantee the safety of occupants and property.

For the nonlinear predictions, the roof sheathing deflections yielded by Case 2 and Case 3 are almost identical, indicating that the way the wall studs connect to the top and bottom plate has little effect on the deflection of the roof sheathing panel for the building configuration discussed here. Under the same wind speed over all the wind directions, larger discrepancies to the measurements occurred when the LVDT1 and LVDT 3 are on the windward with the wind blowing from 0° to 90°. These differences tend to decrease when the two pieces of sheathing are on the leeward since the magnitude of the sheathing displacement is much smaller under the lower pressure induced by the wind from direction 90° to 180°. Over the different wind speeds, the extent of the discrepancies between the nonlinear model predictions and experiment measurements changes significantly. In looking at the low wind speed results, Fig. 3.15(a), it can be seen that the nonlinear predictions overestimate the sheathing deformation, which is likely led by the lower initial stiffness in the constitutive model of the sheathing nail adopted from the literature than the value in the current experiment. Since the capacity of nail is of high COV due to numerous uncertainties such as the specific gravity of the wood member related to the humidity and the direction in driving the nails, it is predictable to have discrepancies between the constitutive models from the actual and the literature even with the same nail type and wood type. In comparison, when subjected to higher wind loads, the accuracy of the nonlinear predictions has increased nearly 30% for the same location and same load pattern with minor overestimation, i.e., from 74% (2.3 mm vs. 4.0 mm) under 65 mph to 53% (6.7 mm vs. 10.3 mm) under 105 mph wind
for LVDT 3 under the wind direction of $0^\circ$ where worst prediction occurs, as shown in Fig. 3.16 (b). It is noted that if the value of LVDT1 instead of LVDT3 is used, the accuracy is improved to 20% (8.6 vs 10.3 mm) since these two measurements should be close due to their symmetric location. Thus, the roof sheathing deformation predicted by the validation models shows very strong agreement with the experimental data taking advantage of the nonlinearity.

Fig. 3.15. Deflection of roof sheathings at the LVDTs on test 1 (65 mph, w/o opening) (a) LVDT 1; (b) LVDT 3

It is noteworthy that the variation of the accuracy predicted by the numerical model might result from the variation in the validity of the experiment measurements. Since all the tests are conducted on the same building model, after a long duration of testing, there is a possibility of the occurrence of nail slip triggering the load redistribution before the failure was recorded, while this redistribution cannot be captured by the validation model without updating the configuration. Thus, such an experiment error is unavoidable and influences the validity of the data measured.

Fig. 3.17-Fig. 3.19 show the time-averaged deflection of RTWCs of Case 2 and Case 3 obtained under each of the wind directions over the different wind speeds and opening conditions. The predicted SP value distributions are as expected under the given arrangement of the sensors, the structure configuration, and the range of wind directions. For example, the SP1 and SP4 measurements under the wind direction of $0^\circ$ should be close to the SP3 and SP6 measurements with the opposite wind direction. For the entire variation with wind directions as illustrated in Fig.
3.5, the curve of SP1 and SP4 should be similar to that of SP3 and SP6 in a reverse way about 90°. Also, the variation of SP2 and SP5 should be symmetric about the wind direction of 90° since they are on the center line of the building model, and under the wind perpendicular to the roof ridge, the values derived from corresponding SPs on the two side of the house, i.e., SP4 vs. SP6 and SP1 vs. SP3, should be similar. Based on these similarities, only SP3, SP 4, and SP5 are presented here for brevity.

![Graph](image)

**Fig. 3.16. Deflection of roof sheathings at the LVDT 3 on (a) test 2 (65 mph, with opening); (b) test 4 (105 mph, w/o opening)**

Over all the cases as illustrated in Fig. 3.17-Fig. 3.19, the predictions from the two numerical models, one with wall studs rigid connected to the top and bottom plate and the other with wall studs pinned to the plates, are nearly identical indicating that the wall stud connection does not affect the behavior of RTWCs much either. For the trend of the deflection subjected to winds of different angles, strong agreements are shown between the FE based estimates and experiment measurements. The FEA well captured the phenomenon observed in the experiments that the displacement of RTWCs on the windward corner, i.e., SP4, experienced a change of trend under the wind with an incident angle of approximately 15° to the ridge line direction.
Fig. 3.17. Deflection of RTWCs on test 1 (65 mph, w/o opening)

Regarding the magnitude of prediction, the numerical model tends to greatly underestimate the vertical displacement at the RTWCs when compared to the experiment measurements, especially under the low wind speed, i.e., two orders of magnitude. As shown in figures, the values on the primary (left) axis denote the displacement of the RTWCs measured from experiments, which are much larger than the corresponding values under each of the wind directions predicted from numerical models, i.e., Case 2 and Case 3, on the secondary (right) axis. What is noteworthy is that the load-displacement relationship assigned to the RTWCs is derived from the connection
composed of a hurricane tie mounted with 8-8d common nails and two additional toe-nails that is much stronger than the connection used in the experiment which is only a hurricane tie with 8-6d common nails. Thus, such gap in values just verifies the developed models to some extent. Additionally, the string pot accuracy might be another reason to account for such discrepancies between the experiment measurements and predictions, since even the largest deformation of RTWCs measured over all the loading cases are very small with a value of around 1mm which is just within the SP 2 string pot’s measurement accuracy.

Fig. 3.18. Deflection of RTWCs on test 2 (65 mph, with opening)
Fig. 3.19. Deflection of RTWCs on test 3 (91 mph, w/o opening)

3.5.2 Validation in time-history domain

Fig. 3.20 shows the sheathing deflection variation obtained under Test 1 loading case, i.e., 65 mph and structure without opening. The numerical prediction matches very well with the experiment measurement in terms of the maximum magnitude. On average, there is no more than 3% difference for the duration as shown in Fig. 3.20 between the two sets of values where the average deflection in the experiment and FE model is 1.81 mm and 1.77 mm, respectively. The
maximum from the experiment is 2.96 mm which has only 5.7% difference from the 3.13 mm calculated by the numerical model, and the minimum difference is 13.0% between 0.69 mm from experiment and 0.78 mm from prediction. Also, there is a strong agreement in the fluctuation pattern of the roof sheathing deflection. The small shift is due to the sampling rate difference between the LVDT, i.e., 100 Hz, and the numerical model, i.e., 120 Hz that is decided to accommodate both the pressure acquisition frequency 520 Hz and the LVDT sampling frequency 100 Hz. For example, the experiment captured a local peak deflection of 2.58 mm at 0.65 s, while that time point is not used in the FEA modeling and thus is missed by the prediction. The closest numerical time points are 0.658 s and 0.642 s with the deflections of 2.4 mm and 1.16 mm, respectively.

Fig. 3.20. Time-history deflection of LVDT 1 (wind direction of 90°, test 1)

3.6 Effects of Modeling Methods

3.6.1 Effect of frame connection modeling on envelope load sharing

To have a sense of magnitude and highlight the differences between the modeling methods of RTWC in terms of the different treatment of the 6 DOFs, the von Mises stress distributions on the envelope of the structure for seven cases under the same wind loading are compared in Fig. 3.21-Fig. 3.23. As illustrated in Table 3.8, these seven cases are intended to cover all the possible ways of RTWC modeling which are categorized by the DOF treatment, i.e., realistic nonlinear behavior or the simplified fixed or free connection. For the cases defined in Table 3.8, the wall connections and STTCs are the same as that defined in Table 3.2, while the RTWCs are varied. Of them, the modeling case numbered Case 2_c4 is the validation model. Case 3_c4 with the same RTWCs modeling is also included in these figures to determine how the application of different wall stud connections affect the stress distributed on the envelope of this structure. The general stress distribution is examined under the wind direction of 75° on three rings of points evenly spaced across the sheathing panels as shown in Fig. 3.21-Fig. 3.23, including the horizontal ring (H ring), the vertical ring perpendicular to the roof ridge (VP ring), and the vertical ring longitudinal to the roof ridge (VL ring). The numbering scheme is also shown in these figures, e.g., the points H1-H5 on the gable wall and the points H19-H27 on the side wall.
As shown in these figures, roughly two sets of results are yielded by the RTWC modeling cases considered herein, where the results of the Case 0 models and the Case 2 models approximately overlap each other, respectively. A key difference between these two groups is that whether the sheathing nail behavior is considered. Under the oblique winds \(75^\circ\), the models without sheathing nail simulation tend to underestimate stress distribution on almost the entire building envelope except for the area on the lower corner of the windward roof sheathing. This kind of models ignoring all the connection behaviors is typically used to set or evaluate the wind loads provision for buildings such as ASCE 7-10 [28] and therefore would induce unconservative design of the structures with respect to the load sharing on the envelope. On the other hand, the similar stress distribution produced by Case 2 models with various ways of RTWC modeling reveals negligible effect of RTWCs to the load sharing on the building envelope, which is adopted by the large amount of research in the literature taken on the roof assembly scale to simplify the boundary conditions on the RTWCs.

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Nonlinear DOF</th>
<th>Fixed DOF</th>
<th>Released DOF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 0</td>
<td>N/A</td>
<td>Rotx, Roty, Rotz &amp; Ux, Uy, Uz</td>
<td>N/A</td>
</tr>
<tr>
<td>Case 0_uy</td>
<td>Uy</td>
<td>Rotx, Roty, Rotz &amp; Ux, Uz</td>
<td>N/A</td>
</tr>
<tr>
<td>Case 2_c1</td>
<td>Ux, Uy, Uz</td>
<td>N/A</td>
<td>Rotx, Roty, Rotz</td>
</tr>
<tr>
<td>Case 2_c2</td>
<td>Ux, Uy, Uz</td>
<td>Rotx, Roty, Rotz</td>
<td>N/A</td>
</tr>
<tr>
<td>Case 2_c3</td>
<td>Uy</td>
<td>Ux, Uz</td>
<td>Rotx, Roty, Rotz</td>
</tr>
<tr>
<td>Case 2_c4</td>
<td>Uy</td>
<td>Rotx, Roty, Rotz &amp; Ux, Uz</td>
<td>N/A</td>
</tr>
<tr>
<td>Case 2_c5</td>
<td>N/A</td>
<td>Rotx, Roty, Rotz &amp; Ux, Uy, Uz</td>
<td>N/A</td>
</tr>
<tr>
<td>Case 3_c4</td>
<td>Uy</td>
<td>Rotx, Roty, Rotz &amp; Ux, Uz</td>
<td>N/A</td>
</tr>
</tbody>
</table>

The internal pressure induced by the opening changes both the magnitude and the distribution of the stress on the envelope: with opening, the von Mises stress develop significantly on the roof, while has either strengthening or weakening effects on the wall depending on the relative location to the winds. It is also noteworthy that the prediction on the VL ring No.2 is comparatively larger than the points nearby, which agrees well with the observed failure that started from the left lower corner of the front roof sheathing panel on the right. These agreements indicate that the proposed nonlinear numerical modeling method is capable of capturing detailed local damage besides the particular responses measured in the experiment. By comparing the von Mises stress distribution of Case 2_c4 and Case 3_c4 with different wall stud connections, the negligible difference is shown on the VL and VP rings which cover the windward sheathing panels on the roof, one of the most vulnerable parts of the building under extreme winds. Also, for the sheathing stress on the walls, which are more durable and thus has less influence on the performance of the building during high wind speed events, the predictions of the two FE models are almost the same for most points as illustrated by the results on the H ring. This indicates that wall stud connection as little effect on the envelope stress distributions.
3.6.2 Effect of roof sheathing fastener capacity on sheathing failure

The real rotation behavior of sheathing nail is nonlinear semi-rigid as tested by Hunt and Bryant [42] and Dao and van de Lindt [31]. It falls into the realm somewhere between the pure hinge and the pure rigid modeled using fixed connections. While the former connection would overestimate the displacement of roof sheathing under uplift force, the latter one would underestimate the sheathing displacement. Dao and van de Lint [31] confirmed the effect of different nail modeling by comparing the sheathing displacements on a panel structure using only an axial nail model and a coupled axial-bending nail model at each nail connection. It is expected
to see smaller sheathing displacements of the model (Case 2) considering the rotation capacity of STTCs than the model without (Case 1).

The roof sheathings on the corner with peak pressures are selected to evaluate the rotational capacity effect of sheathing nails, specifically the deflections on the geometric center of the two roof sheathings on the corner, in accordance with the location of LVDT 3 and LVDT 1 as shown in Fig. 3.24 (a) and Fig. 3.24 (b), respectively. A general trend is apparent from the figure: the effect of rotational capacities of sheathing nail to restrain the deformation of the sheathing panels is more significant under higher wind loads. This effect is particularly pronounced when the building model is subjected to the wind blowing directly to the gable wall adjacent to them, i.e., a wind direction of 0°, where these sheathing panels receive the highest wind loads under the same wind speed. In that condition, the difference in the deflections predicted by the two models achieves its maximum value, and the corresponding percent differences of these values increase with higher wind speeds from 8% under 65 mph wind to 13% under the wind with speed of 105 mph. With the change of wind direction from 0° to 180°, the wind load does not drop as quickly on the back roof as that on the front roof, thus the behavior of the back corner sheathing panel is influenced by the rotational capacities of sheathing nails under more wind incidence angles, i.e., 0° to 45° opposed to from 0° to 15° on the front roof based on the current model. In summary, the predicted performance of the light-frame wood building without considering the rotational capacity of the sheathing nail would overestimate the sheathing response. Although this overestimation is limited under low-speed winds, when subjected to high wind speed events such as hurricanes, the effect of the rotational capacity will become significant and cannot be neglected especially to the failure prediction.

Fig. 3.24. Displacement at corner sheathing on the (a) front roof and (b) back roof

3.6.3 Effect of foundation connections

In order to evaluate how the foundation connections affect the structural behavior in design range, the building response with two arrangements of foundation connections are predicted and compared under a wind direction of 75°, the critical failure direction observed in the wind tunnel test. Three wind speeds are applied to the building structure without opening, including 65 mph, 91 mph, and 105 mph. The first numerical model consists of 16 bolts anchoring the building to the foundation with 4 bolts on each side evenly distributed throughout the length of the sole plates,
and the other building model is connected to the foundation at each end of the beam elements that simulate the sole plates by 76 bolts in total.

The predicted mean values of the sheathing deflection and the RTWC displacement at the places in accordance with the response measuring points in the experiment from the two types of numerical models are compared to each other in Table 3.9 and Table 3.10, respectively. Based on the cases discussed, as the foundation connection gets denser, the behavior of building envelope and connections in the Main Wind Force-Resisting System (MWFRS) in terms of deformation does not change much. The maxima difference for the sheathing deformation is well below 3% even under the highest wind speed, and most of the percent differences for the RTWC deformation is around 1% with several exceptions reaching 25.5% but with very low values around 8.4E-04 mm.

### Table 3.9. Differences and percent differences in LVDT prediction (unit: mm)

<table>
<thead>
<tr>
<th>Wind speed</th>
<th>Model</th>
<th>LVDT 1</th>
<th>LVDT 2</th>
<th>LVDT 3</th>
<th>LVDT 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>65 mph</td>
<td>Case 0</td>
<td>0.01 (0.7%)</td>
<td>0.03 (1.9%)</td>
<td>0.01 (1.1%)</td>
<td>0.03 (2.0%)</td>
</tr>
<tr>
<td></td>
<td>Case 2</td>
<td>0.01 (0.7%)</td>
<td>0.06 (3.0%)</td>
<td>0.02 (1.3%)</td>
<td>0.07 (2.5%)</td>
</tr>
<tr>
<td></td>
<td>Case 3</td>
<td>0.01 (0.7%)</td>
<td>0.06 (2.9%)</td>
<td>0.02 (1.1%)</td>
<td>0.06 (2.4%)</td>
</tr>
<tr>
<td>91 mph</td>
<td>Case 0</td>
<td>0.00 (0.1%)</td>
<td>0.05 (1.4%)</td>
<td>0.02 (0.7%)</td>
<td>0.06 (1.3%)</td>
</tr>
<tr>
<td></td>
<td>Case 2</td>
<td>0.03 (0.7%)</td>
<td>0.12 (2.9%)</td>
<td>0.04 (1.2%)</td>
<td>0.13 (2.5%)</td>
</tr>
<tr>
<td></td>
<td>Case 3</td>
<td>0.03 (0.7%)</td>
<td>0.12 (2.9%)</td>
<td>0.04 (1.1%)</td>
<td>0.13 (1.5%)</td>
</tr>
<tr>
<td>105 mph</td>
<td>Case 0</td>
<td>0.02 (0.6%)</td>
<td>0.05 (1.6%)</td>
<td>0.03 (1.1%)</td>
<td>0.08 (2.0%)</td>
</tr>
<tr>
<td></td>
<td>Case 2</td>
<td>0.03 (0.7%)</td>
<td>0.16 (2.9%)</td>
<td>0.05 (1.0%)</td>
<td>0.19 (2.7%)</td>
</tr>
<tr>
<td></td>
<td>Case 3</td>
<td>0.04 (0.7%)</td>
<td>0.16 (2.9%)</td>
<td>0.05 (1.0%)</td>
<td>0.18 (2.6%)</td>
</tr>
<tr>
<td>Absolute Average</td>
<td>0.02 (0.6%)</td>
<td>0.09 (2.4%)</td>
<td>0.03 (1.0%)</td>
<td>0.10 (2.2%)</td>
<td></td>
</tr>
</tbody>
</table>

### Table 3.10. Differences and percent differences in SP prediction (unit: mm)

<table>
<thead>
<tr>
<th>RTWC No.</th>
<th>65 mph</th>
<th>91 mph</th>
<th>105 mph</th>
<th>Absolute Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP 1</td>
<td>5.2E-06</td>
<td>2.9E-06</td>
<td>1.5E-03</td>
<td>1.6E-03</td>
</tr>
<tr>
<td></td>
<td>0.2%</td>
<td>0.1%</td>
<td>0.2%</td>
<td>0.2%</td>
</tr>
<tr>
<td>SP 2</td>
<td>2.2E-05</td>
<td>1.8E-06</td>
<td>3.1E-03</td>
<td>1.6E-03</td>
</tr>
<tr>
<td></td>
<td>1.2%</td>
<td>0.1%</td>
<td>5.4%</td>
<td>2.7%</td>
</tr>
<tr>
<td>SP 3</td>
<td>4.4E-06</td>
<td>2.3E-06</td>
<td>2.9E-05</td>
<td>-1.1E-04</td>
</tr>
<tr>
<td></td>
<td>0.2%</td>
<td>0.1%</td>
<td>0.0%</td>
<td>-0.1%</td>
</tr>
<tr>
<td>SP 4</td>
<td>-9.0E-07</td>
<td>1.3E-05</td>
<td>-2.1E-03</td>
<td>-9.9E-04</td>
</tr>
<tr>
<td></td>
<td>0.0%</td>
<td>0.6%</td>
<td>-2.4%</td>
<td>-1.2%</td>
</tr>
<tr>
<td>SP 5</td>
<td>-8.4E-04</td>
<td>1.7E-04</td>
<td>-3.6E-03</td>
<td>-2.1E-03</td>
</tr>
<tr>
<td></td>
<td>-25.5%</td>
<td>6.7%</td>
<td>-4.5%</td>
<td>-2.7%</td>
</tr>
<tr>
<td>SP 6</td>
<td>-3.4E-07</td>
<td>8.1E-06</td>
<td>-1.7E-03</td>
<td>-1.1E-03</td>
</tr>
<tr>
<td></td>
<td>0.0%</td>
<td>0.4%</td>
<td>-2.0%</td>
<td>-1.3%</td>
</tr>
</tbody>
</table>
3.7 Conclusion

In this chapter, to investigate a general modeling methodology of low-rise buildings under wind loads, a FE model is developed and validated by a large-scale building model experiment conducted under the Wall of Wind (WOW) Experimental Facility (EF) at Florida International University. The modeling methodology applied is capable of simulating the nonlinear behavior of connections as well as the entire structure and has advantages of more accurate performance predictions over linear-based counterparts. It also shows the feasibility of the direct use of FE software built-in elements as a practical modeling technique. The validation of the numerical model is carried out by comparing the deflection of roof sheathing panels and RTWCs obtained numerically and experimentally under realistic wind loads that spatially-temporally vary. The comparison between the FE predictions and experiment measurements shows reasonable agreement in both the magnitude and trend over different wind incident angles and time durations. The modeling methodology is believed to be suitable for the assessment on the performance of light-frame wood building under high wind speed events.

Based on the validated model, investigations on the effect of modeling on some particular parts of the structure is performed, including RTWCs, foundation fasteners, wall stud connections, and STTCs. The following conclusions are formed on the basis of building models described and load cases conducted herein. The extension of these conclusions is expected in the future to building models with realistic configurations of typical residential houses pertaining dimension and compartment, etc.

- In this study, since all the material properties are derived directly from the literature without conducting actual test as we do in most applications such as design, the accuracy of the validation would be greatly limited, as demonstrated, due to the lack of exact constitutive information especially on the nonlinear constitutive load-force relationships in the connection models.

- The internal flows induced by the opening on the windward wall show slight effect on the pressure distribution pattern on the building, while they change the magnitude of not only the internal but also the external pressures.

- On the roof sheathing deformation, the linear model Case 0, typically used to set or evaluate the wind loads provision for buildings such as ASCE 7 which is force-based, predicts well under low wind speed while would induce underestimations of responses in high wind speed. This indicates the adequacy of this simplest model in the linear range, but not suitable for damage prediction.

- By incorporating connection models into the whole structure, the displacement and the stress distribution on the validation model are changed so does the critical points to the failure.

- The comparison of the envelope stress distribution between different RTWCs models reveals the negligible effect of them before failure and validates to some extent a large
amount of the past research taken only on the roof assembly with the simplified assumption on the boundary conditions corresponding to RTWCs.

- The structure is insensitive to the rotational capacity of the wall stud connections, especially on the most vulnerable parts such as RTWCs and the roof sheathings in terms of deflection, and thus any modeling of this connection, pinned or rigid connected to the plate, is valid.

- The sheathing nail modeling is the predominant parameter to the load paths and load sharing in the structure compared with other components such as the RTWCs and foundation connections. The rotational capacity of the sheathing nail can be ignored under low winds to some extent, while should be incorporated under high wind speed events to avoid the overestimation of the sheathing deformation.

- The foundation connection density influences slightly on load paths with the entire building before the failure of these connections.

One limitation of the current study is that the validation wind tests did not include the mechanical testing on the structural components (e.g., sheathing panels and frame members) and joints (e.g., sheathing nails and hurricane clips). Thus, the material property of wood members and load-displacement curves of joints from the literature are adopted. However, such adoption may alter the load paths due to the discrepancy in the high varied material property and in the end, affect the validation results. Regardless, it is demonstrated that using the material properties in the literature can reasonably predict structural behavior. In addition, the lack of the hysteresis curves of the connectors under highly fluctuating wind loads is unable to reflect the incremental removal of nails over handful of peak loads under realistic wind loads. These limitations related to the material property can be overcome by carrying out wind tests on the structural components and connections.

Future work should examine and fully quantify the mechanical property of structural components, especially the connections. The critical bottleneck restricting the modeling of connections in actual applications has often been the lack of a comprehensive pool of the nonlinear material properties of connections with various wood types and contact conditions. Paying more attention to the fastener or connection ductility may lead to the more effective use of the strength of the wood material components such as frame and sheathing without pushing it out of the linear range.

3.8 References


ASCE. Minimum design loads for buildings and other structures. ASCE/SEI 7-10, Reston, VA 2010.


CHAPTER 4 . NEW DAD APPLICATION: PROGRESSIVE FAILURE OF LOW-RISE TIMBER BUILDINGS UNDER EXTREME WIND EVENTS

4.1 Introduction

Building design standards play an important role in the performance of low-rise buildings inflicted by hurricanes. For the state of Florida, before the adoption of the Florida Building Code (FBC) in 2002 with higher design wind pressures, the 1997 Edition of the Standard Building Code (SBC) was administered. As observed in Hurricane Charley, compared with newly built houses designed and constructed to the FBC with little to no damage, the older buildings suffered pervasive damage to the structure systems due to the insufficiency of the old building code, the SBC [91]. Such post damage observation highlights the significance of building codes and standards including both the state-wide building codes, e.g., FBC and SBC, and the national building codes, e.g., the International Residential Code [2], the International Building Code [3], and the Wood Frame Construction Manual [4]. Those codes are developed based on the ASCE 7 [5] for the reference as wind-related requirements. Thus, for a safer building design, database-assisted design (DAD) is initiated to increase the accuracy of wind loads by replacing the application of the tabular pressure coefficients specified in the ASCE 7 with the direct use of pressure time histories obtained from comprehensive wind tunnel tests, i.e., the tests equipped with denser external pressure taps and tested in finer wind direction intervals.

The DAD (also referred to as DED, the database-enabled design, by Tokyo Polytechnic University, TPU) approach is implemented by using an aerodynamic database hosted by software such as the windPRESSURE for the National Institute of Standards and Technology (NIST) database and the DEDM-LR for TPU database. These design modules perform the response analysis to the time domain-based wind pressures from the database and search for the peak structural responses such as bending moment at the knee of multiple locations and wind directions with the influence coefficients for the structural response of interest. Such analyzing procedure that allows for the turbulence levels and other test factors such as the tap density is proved to be superior to the provisions of the ASCE 7 standard that only provides constant wind loads obtained from cruder wind direction intervals.

Comparisons by St. Pierre et al. [6] on a series of peak responses (i.e., the moments, the vertical uplift, and the horizontal thrust) calculated directly using pressure time series of the NIST database with those predicted by the ASCE 7 showed that the general underestimations were given by the ASCE 7 and the difference generally increased with the increase of the building height. Then, Coffman et al. [7] confirmed St. Pierre et al. [6]’s result by comparing loading effects on the peak bending moments at the knee of frames, of which moments are considered generally have the largest magnitude of wind-induced response at the Main Wind Force-Resisting System (MWFRS). These peak bending moments based on the NIST aerodynamic database calculated by the DAD technique were generally larger by 10%-30%, and the discrepancies increased with the larger roof slope and the higher eave height. Mensah et al. [8] confirmed the suitability of the DAD approach for predicting the structural reactions in light-frame wood buildings with a good agreement on the reactions at the roof-to-wall and wall-to-foundation between the direct measurements and the DAD predictions. They also examined the adequacy of the ASCE 7 by comparing the predicted peak reactions at the roof-to-wall with that obtained through the DAD
method and came to the same conclusion that both the results predicted by following both the MWFRS and C&C approaches of the ASCE 7 were highly non-conservative and can result in risk-inconsistencies after comparing to the DAD-predicted structural response. After the release of TPU database, Hagos et al. [9] performed a comparison between the TPU database and the NIST database and reported that the two databases were reasonably equivalent for practical engineering purpose with tolerable differences, i.e., the absolute value of differences in peak pressure coefficients are within 15% for 71% cases discussed, increasing the confidence in both datasets. Kwon et al. [10] presented the DEDM-LR analysis based on the TPU database with efficacy and validity by comparing the calculated peak bending moments with that from the NIST windPRESSURE. Then, the DEDM-LR analysis was used to evaluate the ASCE 7 MWFRS method, including both the directional procedure and the envelope procedure. The result of the four cases selected in terms of roof angles of gable roof buildings again revealed the non-conservative nature of the ASCE 7: the peak bending moments at the right knee of the middle frame predicted by the DEDM-LR analysis were about 30-90% higher than that of the ASCE 7 and this ratio increased with the increase of the roof angle.

However, the DAD concept is originally conceived for and its current application is restricted to the building design by predicting the peak responses on the MWFRS of the low-rise metal frame structure, which are consistent with the development of the ASCE 7 procedures on the same building type and responses [10] [11]. Such a building type along with its critical demands apparently cannot represent the typical low-rise buildings in North America, which are the light-frame wood constructions and account for the majority of wind induced monetary loss [12]. In addition, the most common failures on the structures repeatedly observed in Hurricanes, i.e., the failures of the roof sheathing and connections along the uplift load paths, cannot be reflected. The challenge of the current DAD application on low-rise buildings is that it has no connection modeling and only performs static, linear analysis concentrating on frames rather than the more vulnerable envelope system of the structure. Fortunately, the essence of DAD is to take advantage of the aerodynamic database and make a dynamic analysis which is not restricted by the building model employed and should not be restricted to be a supplementary of codes for the life-safety design purpose. Thinking out of the box and expanding the DAD application onto the wood buildings with nonlinear connections is necessary for a more accurate prediction of the structural response subjected to the varying load-carrying capacities. The new application allows for the changing load-resisting mechanisms in the progressive failure of the entire building.

The concept of the progressive failure analysis has been defined in many ways over the last decade. By and large, it is felt by most to be a successive-stage damage philosophy that allows for the occurrence of the failure on one structural member after another until the whole building system loses its function. Past applications of it have been concentrated on the high-rise buildings of reinforced concrete or steel structures, especially after the 911 tragedy in 2001, e.g., [13]-[15]. It is referred to as the terminology “progressive collapse” with the accurate definition given by the ASCE 7 as “the spread of an initial local failure from element to element, resulting eventually in the collapse of an entire structure or a disproportionately large part of it.” A threat independent methodology, the alternative path method (APM), is widely employed by it to analyze the structural behavior after the failure of some critical structural members. Such analysis is generally performed in the context of a “missing column” scenario without any load-structure interactions.
and concentrates on the potential of a building for the progressive collapse in its ability to absorb the loss of a critical member while regardless of how the triggering event happened.

For low-rise buildings, the research on the progressive failure is still in its infancy, especially for the light-frame timber structures in the criteria of wind engineering. Post-storm reports contribute greatly to the identification of the critical weakness in the constructions by picking up the first-hand damage information after the attack of hurricanes. However, the failure process remains unknown, and the mechanism of the failure is hardly traceable from the massive wreckage. A reliable reproduction on the building performance that goes beyond the first failure is imperative for the significant advances in deepening the understanding of what kind of, where and in which sequence does the damage would occur when a large-scale destruction takes place in a significant storm. Such reproduction serves for the dual-objective design that simultaneously guarantees the life safety governed by the potential for the catastrophic building collapse while reducing the structural damage and economic loss. Serving for this purpose, the progressive failure analysis on low-rise timber structures must not only allow for the triggering event, i.e., the wind-structure interaction, but also allow for it in a continuous way until being fully destroyed. However, due to the complexity of the timber building configurations with highly redundant non-structural components, little-to-no-studies exist so far towards the simulation on the progressive failure of full-scale 3D building models subjected to wind loads. Notable exceptions include Thampi et al. [16] and Kumar et al. [17] applied with tornado wind loads in a quasi-static manner with only one failure mode (pull out) for sheathing nails represented by only 3 translational DOF, and Pan et al. [120] proceeded only to the first failure analysis. So far, no progressive failure analysis method has been validated by destructive wind tunnel tests due to the high cost.

This paper explores the stress distribution, the failure modes of structural members, and the locations and order of these failures on low-rise wood buildings in the interactions with significant straight-line winds. The proposed failure analysis methodology is validated to provide more accurate damage predictions on low-rise buildings during the stage-wise wind-structure interactions for a more cost-effective design. This methodology also helps with the assessment on the wind intensity from post damage observations. In support of this mission, two deterministic finite-element (FE) models that incorporate various failure modes of structural components were developed in a full-scale level using ANSYS. The first model with the geometry covered in the National Institute for Standards and Technology (NIST) aerodynamic database is used to extend the effectiveness of a past validated nonlinear modeling methodology to the failure range. It also serves for validating the progressive failure analysis methodology with the calibration from the phenomena captured in a corresponding destructive wind test at Florida International University (FIU). The second model is built to the South/Key CBG type of the Florida Public Hurricane Loss Model (FPHLM) with the intention to simulate and study the successive stages of failure on a U.S. representative residential house subjected to extreme wind loads in a more general way. Pressure time histories derived from Louisiana State University (LSU) aerodynamic database are applied with the purpose of expanding the application of the DAD from the linear to nonlinear range and finally to the progressive failure stages. Such new applications of DAD would open up an avenue for the vulnerability assessment and could be used as a strong tool for the damage and loss predictions to serve for the insurance industry.
4.2 Progressive Failure Analysis Methodology

4.2.1 Methodology

With the wind speed increases, if any failure mode is observed, the corresponding component was considered as failed and removed. This removal is accomplished by deactivating the elements that comprise the failure component in ANSYS through explicitly setting their stiffness to around zero for the FE analysis at the next time step. Although the deactivated element remains in the FE model, it contributes a near-zero stiffness to the overall stiffness matrix and nothing to the overall mass matrix, and thus, its contribution to the responses are set to zero. The loads applied to the deactivated elements do not generate a load vector and thus, these loads are zeroed out of the entire load vector. Such methodology has been applied to failure analysis on the low-rise building under wind loads for some cases [16]-[18]. The validation of such applications is in need and will be carried out in the current study with more comprehensive failure modes.

One noticeable limitation of the progressive failure mimicking has been the application of loads. As the failure proceeds, both of the internal and external pressures change significantly due to the updated fluid-structure interaction. However, such changing wind loading information is not available due the high cost of equipment consumptions in the destructive wind tests. In the current study, the wind pressure time history are assumed to be the same under each of the successive failure stages.

4.2.2 Failure Mode and Failure Criteria

Failure modes are dependent on the loading condition of the case discussed, including the type and the direction of external loads applied. For example, the failure mechanism induced by the wind and earthquake are different in that the wind loads are proportionally applied to the exposed surface of the building, whereas earthquake loads exert inertial forces resulted from the deformation produced by the earthquake motion and lateral resistance of the structures [15]. The structural failures subjected to the uplift force representing the main effect of wind loads occur on the vertical load paths including the roof sheathing nails, the roof-to-wall connections (RTWCs), and the foundation hold-downs and anchor bolts. As opposed to that, when inflicted by lateral forces mainly referring to earthquake loads, failures concentrate along the lateral load paths with typical modes such as the nail failure on the wall sheathings or the buckling of wall sheathings.

In the current study, failure modes considered are consistent with the wind force effects, including the withdrawal, the pull through, and the slip for the sheathing nails, the uplift for RTWCs, and sheathing panel failure caused by the exceedance of its axial, shear, or bending stress capacity or the large displacement as shown in Table 4.1. Attention must be given to the failure criteria of connections including the sheathing-to-truss connections (STTCs) and RTWCs as they are the typical critical components in wood structures under hurricanes and the primary reflections of the structural nonlinearity. Additionally, the first two modes in the table cannot be applied at the same time.
### Table 4.1. Failure modes and criteria

<table>
<thead>
<tr>
<th>Structural Component</th>
<th>Failure Mode</th>
<th>Force Type</th>
<th>Failure Threshold</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>STTC</strong></td>
<td>Withdrawal</td>
<td>Axial force (nail yielding)</td>
<td>680 N [26]</td>
</tr>
<tr>
<td></td>
<td>Pull through</td>
<td>Axial force (wood brittle)</td>
<td>1070 N [27]</td>
</tr>
<tr>
<td></td>
<td>Slip</td>
<td>Shear force (nail yielding and/or wood crushing)</td>
<td>1250 N [28][148]</td>
</tr>
<tr>
<td><strong>RTWC</strong></td>
<td>Uplift (strap tear &amp; nail pullout of rafter)</td>
<td>Axial and Shear</td>
<td>4400 N [29]</td>
</tr>
<tr>
<td></td>
<td>Axial</td>
<td>Tension/Compression in plane of plies</td>
<td>6.76/6.69 MPa(^a) [30]</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Shear through the thickness/Rolling Shear</td>
<td>1.07/0.43 MPa(^b) [30]</td>
</tr>
<tr>
<td><strong>Sheathing Panel</strong></td>
<td>Bending Stress</td>
<td>Extreme fiber stress in bending</td>
<td>9.86 MPa [30]</td>
</tr>
<tr>
<td></td>
<td>Displacement</td>
<td></td>
<td>0.09 m (FIU destructive test)</td>
</tr>
</tbody>
</table>

\(^a\) Sheathing panel failure under the axial force induced by both the tension and compression are considered in the current study.

\(^b\) Shear force induced sheathing panel failures including both the shear through the thickness and rolling shear are considered in the current study.

For STTCs, only one of the first two failure modes could occur when the sheathing nail subjected to a load applied along its axis: the nail shank withdrawal from the lumber or the nail head pull through off the sheathing panel (Fig. 4.1). The first mode refers to the nail being completely removed from the frame member while still being attached to the sheathing panel; the latter mode means that the nail remains to be attached to the frame while the sheathing splits with it. These failure modes are different in nature. The nail withdrawal involves the friction to the lumber and the yielding of the nail, and its capacity is dependent on factors such as the embedment and hammered angle of the nail, the type and grade of the lumber, and the moisture condition, etc. The process of the pure withdrawal can be reflected by the experiment phenomenon and the load-displacement curve (Fig. 4.2). Initially, the nail sustains the entire loads when it is pulled, and the curve goes up. But as the load increases, when it is larger than the friction between the nail and lumber, the nail starts to be withdrawn from the wood resulting in the reduction in the embedment length and the related friction, making the nail easier to be pulled out. This stage is shown in the reduction part of the curve. As opposed to the more ductile withdrawal failure mode that involves nail yielding, the nail pull through is considered as a brittle mode since this failure is caused by the wood splitting that is brittle in nature. This mode is generally ignored by timber design codes, e.g., [20]-[22]) with the assumption that the head of the fastener is large enough to prevent pull through [23]. However, as the increase in the withdraw strength in the wood design, this failure mode should be put into consideration.
The general observation on in-situ constructions is that the nail withdrawal is the predominant failure mode due to its lowest capacity, which has been verified by Shanmugam et al. [24] through the statistical analysis on a series of experiments. Morrison and Kopp [25] came to the same conclusion and furthered its application from the ramping to the temporal-spatial variation wind loads. Both the modes should be considered in a vulnerability analysis. However, for a specific case, whether the nail will be withdrawn or pulled through is unknown due to the high COV in the nail capacity. Decisions must be made for a deterministic analysis to approach a failure process with a larger possibility, and thus, the predominant withdrawal failure mode is chosen over the pull through.

Failure can mean different things to a structural component under different considerations, such as the capacity, the safety, or from a financial perspective, e.g., when a sheathing panel experiences a large deformation, it can be considered as a failure since it allows the water intrusion causing extensive damages. In this scenario, the sheathing panel failed to prevent the happening of such a main source of financial loss in a hurricane. Since the main objective of the current study is to reproduce the real stage-wise failure under hurricanes, the safety or the financial concerns would induce the premature failure and the underestimation of the structural resistance. Therefore, the failure threshold is defined only from a purely structural perspective that whether the component is still contributing to the structural resistance.
The failure of the STTC and RTWC is defined by the maximum force capacity with the purpose to account for the time accumulation effect. Recent research has found that the failure of both the roof sheathing (e.g., [31]) and toe-nailed RTWCs (e.g., [25]; [31]; [32]) is caused by the incremental damage accumulation over a handful of peak loads under fluctuating winds. Thus, the longer the loading duration goes, the more peak loads would occur, leading to more damages to the structure. Such progressive damage of connections is identified by the post-storm damage investigations (e.g., [33]; [34]). Since the failure capacity under realistic, fluctuating wind loading is “remarkably similar” to that of the ramp loading as indicated by Morrison and Kopp [25], the failure threshold for the connection is defined as the force reaching its capacity with the assumption that the whole process of the incremental removal of connectors over time has been incorporated. And the connectors are expected to fail at a slightly lower wind speed with longer duration. The sheathing panel failure is evaluated by the stress capacities in three categories, i.e., axial, shear, and bending stresses. It is also evaluated by the large displacement for the case of sheathing turnover induced by the lack of sufficient connections after losing the sheathing nails. In this condition, the entire piece of sheathing is removed.

4.3 FIU calibration building

4.3.1 Building model chosen for validation

A one-story light-frame timber house was tested at the Wall of Wind (WOW) facility at FIU for the validation of both the numerical modeling framework and the failure analysis methodology for predicting the realistic responses of the low-rise buildings under wind loads (Fig. 4.3a). This structure was built following one of the generic low-rise building models covered in the NIST aerodynamic database with a rectangular plan of dimensions 3.57 m (11.72 ft) × 2.29 m (7.50 ft). It has an eave height of 0.91 m (3.00 ft) and a gable roof of 14º slope.

Fig. 4.3. Destructive wind tests at WOW, FIU: (a) a photo of the FIU calibration building model (the end wall sheathing was removed, and the door was left open for demonstration); (b) the layout of the sheathing panel with the definition of wind angles and roof sheathing panel numbers.

Targeting at the calibration, this light-frame building simply consists of the lumber frames, the board sheathing, and the connections including nails and metal traps that are critical to the performance under extreme wind events. The non-structural components that contribute little to
maintain the continuous uplift load paths are ignored such as the wood shakes or shingles. One special feature in this building is that the individual roof sheathing panels are modeled to allow for the failure component as well as the trajectories followed to be determined at a more specific location. Such valuable information can be used to investigate the load paths in the building system and examine the potential of secondary damage for the structures downstream caused by the windborne debris, respectively. A total of ten oriented strand board (OSB) of 11 mm thickness (known colloquially as 7/16) are used as sheathing panels, of which four on the roof, four on the side walls, two on the gable walls, and one at the door, shown in Fig. 4.3 (b). The sheathing is nailed to the frame members by 6d common nails with a conventional nailing schedule of 6 in. /12 in. along the exterior panel edge (edge nailing) and the intermediate supports (field nailing) per the Section 2304.9.1 Note b in International Building Code [3]. Then, the roof edge nailing along the side walls are intentionally eliminated to weaken the load paths to observe failures.

Spruce-pin-fir (SPF) #2 lumber of dimension 38.1 mm × 63.5 mm (known colloquially as 2 × 3) is consistently used for all the frame members in the structure, i.e., trusses, beams, and studs. For the truss assembly, three interior trusses made of the fink style are evenly spaced on the center, each of which has four oblique webs connecting the top chord to the bottom chord. Gable truss with five vertical webs is used as the exterior truss at each roof end. The five trusses with members connected with 16d common nails are linked at the peak with the roof ridge beams by 8d common nails and connected at corners to the double head plates by H3-18 Gauge hurricane clip that is composed of 8-6d common nails. Wall studs are spaced approximate 430 mm (17 in.) on the side wall and around 457 mm (18 in.) apart on the end wall, of which the one at the intersection of two walls (corner of the building model) consists three lumbers. A typical door opening with header

Fig. 4.4. Connections: (a) gable end truss and ridge connections; (b) web and top chord connections; (c) web and bottom chord connections; (d) RTWCs using hurricane tie at exterior truss; (e) RTWCs using hurricane tie at interior truss; (f) wall stud to bottom or top plate connections
supported by jack and king studs is modeled at the center of one side wall for the evaluating the effect of the internal pressurization during breach of the opening. All the wall studs are end nailed to the head and sole plates with two 8d common smooth nails, respectively, at each end. In the end, the building is anchored to the WOW turntable at the sole plate by four bolts evenly distributed on each side, i.e., a total of 16 bolts. As such, continuous load paths are formed to transfer all the wind loads from the roof and wall to the foundation. The installation of connections between frame members are shown in Fig. 4.4.

4.3.2 Large-scale destructive testing

Testing the low-rise building to failure by winds has most often been “an act of God” [37] due to the paucity of the destructive wind test. The reproduction of the realistic wind loading with strong temporal and spatial variations at large scale is very difficult to realize in a scientific controlled fashion. Mimicking the detailed behavior and failure of all the components in the complete system at large- or full-scale is even more challenging and expensive. However, the destructive wind test is irreplaceable as a reliable approach to capture the 3D performance of the structure under complex wind pressure patterns until the ultimate failure. As summarized by He et al. [36], some attempts that have been made to analyze the complete building behavior at full-scale under realistic wind loads mainly come from two sources. One is through the consortium of Canadian Universities awarded by the NSERC Collaborative Research and Development (CRD) Project on existing buildings under natural winds that cannot be controlled. The other is the “Three Little Pigs” Project (3LP) at the UWO Insurance Research Lab for Better Homes (IRLBH) applied with pressures controlled by pressure loading actuator (PLA) only on the roof. Until now, most

Fig. 4.5. Sequence of the progressive failure recorded by video [35]
full-scale or large-scale building wind tests were focused on the measurements of structural responses within linear range or before the global structural failures, while the full picture of the building performance is in short. Reflecting this fact, a small step was taken in that direction by performing a large-scale destructive wind test at WOW, FIU in a controlled manner with comprehensive outcomes such as the failure criteria and failure sequence.

This wind test was carried out on the FIU building model at three wind speeds starting from 29.06 m/s (65 mph) under the wind angles from 0° to 180° with 15° intervals, considering the building symmetry. The failure occurred at the wind speed of 46.94 m/s (105 mph) with 75° wind angle, and the whole process of the failure was recorded by video with images showing the characteristic motion of components (Fig. 4.5). Initially, the withdrawal of nails was observed at the field nailing on the sheathing RP4 resulting in an air gap between the roof sheathing and the wall for the wind blowing in. This, in turn, created higher internal pressures acting on the roof sheathings to lift them up, leading to more nails withdrawal at the field nailing on sheathing RP3 and the additional air gap allowing more flows in. In wake of the larger internal flow pushing the roof, the RP3 sheathing panel started to be unzipped from the corner and finally was broken leaving a piece of crack on the corner at the top of the gable end. This unzip process along the edges happened in a moment which only lasted for 0.2 s corresponding to the figures (d)-(g). As a result, a big hole created on the roof for the outlet of the internal flow attributed to reduce the uplift internal force as witness by the smaller displacement on the edge of sheathing RP4. Due to the drop of the net pressure, no further noticeable damage was observed, and the destructive experiment stopped.

![Damage details: (a) nail withdrawal on RP4; (b) nail pull through on RP3; (c) nail withdrawal on RP3; and (d) two front roof sheathings with demonstration of the crack location, failure mode, and failure location](image)

For the whole process, it was observed that instead of instantaneous failure, the roof sheathing would bounce at the leading edge indicating that the peak loading has already overcome the resistance of the sheathing nails at the field nailing, but did not last long enough to fail all these
connections. This phenomenon is consistent with that illustrated by Surry et al. [37] in a small-scale destructive wind tunnel tests.

Damage studies were performed after the destructive wind test, where the failure components and failure modes along with the locations are identified and documented by a series of photographs as shown in Fig. 4.6. Besides the breaking of sheathing panel, the nail withdrawal and pull through are the predominant failure modes. Clear nail withdrawals were observed at the field nailing on the sheathing RP4. For the sheathing RP3 that experienced turnover during the test, both nail pull through and withdrawal were spotted on the sheathing nails attached. After examining the deformation of all the sheathing nails, no obvious nail slip was observed. Also, no clear vertical displacement was observed at the RTWCs during the entire process. Especially with the occurrence of the opening on the windward roof, both the external and internal pressures acting on the roof sheathing panels dropped, leading to the decrease in the uplift force of RTWCs. The damage information is summarized and demonstrated by the sketch in Fig. 4.6 (d).

### 4.3.3 FE building model progressive failure

Extensive work has been performed on the numerical modeling of the FIU calibration building using the FE program Mechanical APDL (ANSYS) with its built-in elements (Fig. 4.7). To be able to provide more accurate performance predictions, the critical connections such as the sheathing nails and RTWCs are modeled by nonlinear spring elements to allow for the contribution of both their translational and rotational capacities to the load sharing in the structure. Sheathings are represented by the 8-noded linear-elastic orthotropic quadrilateral shell elements with six degrees of freedom at each node to reflect the in-plane and out-of-plane behaviors. All the frame members are simulated by 3D linear isotropic beam elements that have six degrees of freedom on each node and are good for the large deflection analysis. Geometric nonlinearity is considered in the performance prediction. This modeling methodology has been validated previously in the intact building form by comparing the deflection of roof sheathings and RTWCs obtained numerically and experimentally under realistic wind loads [35].

![Fig. 4.7. FE model of FIU calibration building: (a) sheathing; (b) framing; (c) RTWC load-deflection relationship [35]](image)

In the present study, the prediction of the failure process was performed under the wind speed of 155 mph when the first failure occurred in the form of the nail withdrawal at the field
nailing. The higher failure speed in the simulation is likely led by the discrepancy in the material properties adopted from the literature due to the lack of material testing. Also, before the destructive test, the experiment building model that had been subjected to winds for a long time may have already had some of its components partially failed, e.g. nail withdrawal, and thus entered the failure stage beforehand. The sheathing panel RP4 experienced complete field nailing withdrawal one by one from the edge to the ridge. Then, the field nailing in the sheathing panel RP3 started to be pullout as well until the RP3 lost its entire field nails, as shown in Fig. 4.8 (a). This process compares well with the destructive wind test in both the failure mode and failure sequence. After that, with less connections, the sheathing panel RP3 experienced excessive displacement and was removed from the model, as shown in Fig. 4.8 (b). This final stage signaled with the failure component is also consistent with that of the wind tunnel test. During this process, the real pressure pattern had been changing dramatically with the opening condition especially when the air gaps occurred and after the edge nailing unzipped. The failure modes had greatly varied accordingly. However, these pressures were not measured in the wind test and thus, cannot be updated in the numerical model. It is unpredictable that how does the nail unzip and where does the sheathing fracture. Subjected to the pressure patterns under the intact building form, the predicted failure should be conservative, leading to a higher failure wind speed and a lower quantity of the failure components, which conforms to the current failure modeling.

![Image](image_url)

(a) (b)

Fig. 4.8. Stress distribution under the: (a) damage state with sheathing nail withdrawal and (b) final damage state with the sheathing RP3 removed. The shell elements connected to the failure sheathing nails are not displayed to demonstrate the location of these nails.

### 4.3.4 Individual connection incremental failure

The detailed numerical modeling enables us to track the progressive failure of critical connections under fluctuating wind loads which is an important indicator to reflect the accuracy and adequacy of both the modeling and failure analysis methodologies. One example of this is taken on the withdrawal behavior of the RTWC at the middle truss on the front that connects both the sheathings RP3 and RP4. Fig. 4.9 depicts the simulated displacement time-history and the predicted force versus displacement for this RTWC under the time-history wind loads of the eight wind speeds with the assumption of the intact envelope and the constant internal pressure. Positive
displacement represents the roof lifting away from the truss stretching the connection to trigger the withdrawal failure.

Fig. 4.9. (a) Displacement time series and (b) the load-displacement behavior of a RTWC for the time history wind loads

The predicted displacement time histories under the different wind speeds (Fig. 4.9a) demonstrate that the withdrawal of the RTWC accumulates over a handful of peak loads under the fluctuating wind loads. When the peak loading acting on the RTWC is within the elastic range, the curve would slightly rise and go back to its original level forming the “^” shape; while when the peak loading is large enough to cause the sheathing nail withdrawal, the curve would “jump” to a higher level and form a “J” shape. Such a “jump” pattern (sheathing nail withdrawal) led by the peak loading \( P = \frac{1}{2} \rho V^2 C_p \) would be amplified by the increasing wind speed for the same pressure coefficient data, reflected in the cases with the wind speed over 170 mph. This incremental removal well reflects the characteristic failure process of connections observed in the wind tests [25] and [32].

The permanent removal of connections under each damaging peak is also illustrated in the load-displacement relationship (Fig. 4.9b). Under the lower wind loading when the first damage peak has not occurred, the relationship is nearly linear. However, after the first damage peak as the case of 170 mph at around 1.7 kN, the RTWC is permanently withdrawn by around 0.15 mm. Following the subsequent damaging peaks, more permanent displacements are induced and accumulate towards the complete failure. For the wind speeds discussed, the complete withdrawal has not occurred yet. This is consistent with the input RTWC load-deflection relationship in Fig. 4.7(c) in which the linear range ends around 1.7 kN, and the failure force (around 4.5 kN) is higher than the maximum force (less than 3.5 kN) in the current discussion. The predicted response also agrees well with another phenomenon found from the previous experiment in [25] that the stiffness of the connector remains unchanged in spite of the partial removal as the load-displacement behavior remains similar to the undamaged case between the damaging peaks. Therefore, the proposed modeling methodology shows the ability to capture the progressive withdrawal behavior.
4.4 LSU building model

4.4.1 Building model and wind loading

To demonstrate the possible failure process of the U.S. low-rise buildings during extreme wind events, the validated progressive failure analysis methodology was applied on to a building model with more realistic configurations, i.e., building geometries, components arrangement, and opening layout, etc. This process also unveiled the capability of the progressive failure analysis methodology to be applied onto a more complex building model. The South/Key CBG type of the Florida Public Hurricane Loss Model (FPHLM) was chosen as the prototype and the wind loading was derived from the LSU aerodynamic database [19]-[39] on a 1:50 scale model of the same prototype.

This building model representing a one-story 5:12 pitched gable roof residential house with a rectangular footprint of 18.3 m by 13.4 m (60 ft by 44 ft) and overhang height of 3.0 m (9.8 ft) was developed following the same nonlinear modeling methodology that has been validated by He et al. [35] and that has been discussed earlier. The accuracy of the FE model is attributed largely to the relatively uncompromising modeling in all the representative configurations. For example, all the nonlinear behaviors of the STTCs including the translational and the rotational are modeled by nonlinear spring elements. The same constructive practices as Pan et al.’s [120] are adopted by this building such as the arrangement of studs and staggered sheathing panels as well as the installation of sheathing nails (6 in. /12 in.). Finally, this building model is built up with 12,811 beam elements, 39,505 shell elements, and 50,664 nonlinear spring elements in total.

The failure process is carried out under the 3-s gust wind blowing perpendicular to the side wall. The pressure coefficient collected from the LSU database are converted to the storm condition used by the ASCE 7-10, i.e., 3-s, 33 ft (10 m), open terrain with roughness length $z_0=0.03m$, through the equations specified by He et al. [39]. Analyses were performed for the sealed building condition assuming the openings on the walls are well protected. Another assumption was made on the wind loading that the pressure coefficients would not change under the successive stages of failure due to the lack of the corresponding data. For example, the internal pressure coefficient measured in the wind test so far or defined in the ASCE 7 standard is for the opening on the wall, i.e., door or window, while the opening on the roof has not been explored due to the large uncertainty of the occurrence.

4.4.2 Progressive failure analysis

To reach the complete failure state, the building model was subjected to the mean wind loads with the increasing speed from the 75 mph (33.5 m/s) with 5 mph (2.2 m/s) increment. For each wind speed, the model was applied with only one load step without considering the storm duration. As such, the damage of the structure under hurricane wind loads from the Category (Cat.) 1-5 in Saffir-Simpson hurricane wind scale (SSHWS) is observed. Fig. 4.10 shows the damage on the building envelope and the sheathing nail in a quantitative perspective under the varying wind speed.
Fig. 4.10. Failure condition under the increasing wind speed: (a) the ratio of damaged sheathing area to the whole sheathing area of the building; (b) quantity of the failed nails

Under the Cat. 1 hurricane wind (75 mph), a slight amount of the sheathing nails (i.e., 4 nails) lost their functions while the sheathing panels stayed intact. As the wind getting stronger, more nails failed, and the building started losing its envelope. Such a procedure continued, and the quantity of the damaged components steadily climbed until the leeward roof lost most of its sheathing panels corresponding to the destructive test condition. In the end, the area of the damaged sheathing panels rose as high as 35% of the entire building surface area under the wind speed of 360 mph. The final failure state with the von Mises distribution is shown in Fig. 4.11. Besides the sheathing on the roof, the gable wall also experienced the sheathing pulled off close to the windward side. Overall, the progressive failure analysis methodology exhibits its full capability to be applied on the complex building model.

Fig. 4.11. Final damage state for the increasing wind speed (windows and doors are not displayed intentionally to demonstrate their locations)
### 4.4.3 Failure due to the duration effect under the same wind speed

To assess the accurate intensity of the hurricane from the observed damage state of the building, it is necessary to determine that if the wind of lower intensity could cause the same amount of damage considering the storm duration effect. The analysis was performed under the time-history wind load with the speed kept 150 mph (Cat. 4) for a duration of 10 s by applying a period of 2-s, 10 Hz pressure coefficient time history repetitively as shown in Fig. 4.12. The duration effect regarding the number of the peak loading would be reflected by the increase of the damage with the increase of time and the difference in the wind speed that resulted in the same amount of damage as that with the ignorance of the loading duration. Then, the same analysis was performed under the lower wind speeds, i.e., 10 mph (tropical depression) and 100 mph (Cat. 2), to demonstrate the influence of the magnitude of the peak loading to the duration effect.

![Fig. 4.12. A sample of the pressure coefficient time history](image)

![Fig. 4.13. Failure condition under 150 mph wind speed for a duration of 10 s: (a) damaged surface area; (b) quantity of the failed nails](image)

From the successive failure under 150 mph displayed in Fig. 4.13, the structure had almost no damage under the starting point, which is comparable to that in the increasing wind speed condition in Fig. 4.10. However, as the time went by, the structure experienced the sheathing nail
failure with the sheathing panel failure followed. The damage steadily climbed throughout the remainder of the time and into the last moment examined, i.e., the time of 10 s, eventually peaking an 8% for the damaged surface of the building and a quantity around 350 for the sheathing nails that lost their functions. Fig. 4.14 shows the von Mises distribution for the final failure state. The extent of these two kinds of damage after a duration with the wind speed keeping at 150 mph are comparable to the damage created at the wind speed of 225 mph and 305 mph, respectively, when the duration effects were ignored as it is the case in Fig. 4.10. As such, the building vulnerability to the wind increased by over 50% due to the duration effects in terms of the wind speed. This explains why that even some storm which is not big enough to be a hurricane can still create significant loss as it happened in hurricane Allison (2001) in the southern United States and hurricane Matthew (2010) around Central America causing $9.0 and $2.6 billion in property damage, respectively.

Fig. 4.14. Final damage state under the wind speed of 150 mph

For the occurrence of failure, the loss had always proceeded at some particular loading points indicating the occurrence of the peak pressures. For example, the quantity of damage on the sheathing and nail stretched a lot at the 13th loading point of the sample for each time segment, corresponding to the time of 1.3 s, 3.3 s, 5.3 s, 7.3 s, and 9.3 s. From the observation on the trend shown in Fig. 4.13, the peak loading still had the potential to cause more components to be destroyed if the loading duration could last any longer.

The duration effect also can be observed under the lower wind speed as it is the case of 100 mph in Fig. 4.13. Even though the damage area ratio stayed at 0% throughout the failure analysis representing the building did not lose any of its sheathing panel, the number of the failure sheathing nails increased with the loading time. On the other hand, the duration effect under the lower magnitude of the peak loading in the case of 100 mph is not significant comparatively. The growth of the failure on the sheathing nails stopped at the time of 5.3 s with a total quantity of 43 which is much fewer than the 345 in the case of 150 mph. Under the even lower wind speed, i.e., 10 mph, no failure was observed as expected when the peak loading remained in the elastic range. Clearly, the storm duration effect is not fully analyzed in the current study since only the limited number
of the peak loadings is considered. Thus, more damage would be induced by the duration effects than the current prediction if taking into account of the magnitude of the peak loading as well as the material deterioration.

4.5 Conclusion

Two proposed methodologies are calibrated by a destructive wind test performed on a large-scale timber building model at the WOW facility at FIU. One is a previously proposed nonlinear FE modeling methodology on the failure stage of the building system. The other one is a progressive failure analysis methodology for the light-frame timber house under the extreme wind events.

The failure analysis methodology is well developed here with the explicit explanation on the failure mode, the failure location, and the failure criteria. The comparison of the modeling and wind tests results in terms of the failure mode and sequence shows the failure behavior in either the building scale or the individual connection scale is estimated reasonably well by the two methodologies. After that, the proposed analysis methodology is applied onto a U.S. representative residential building model which reveals its full capability to deal with the more complex models. The storm duration effects on the whole building failure was investigated by the failure analysis method, and it was found that the building vulnerability to the wind increased by over 50% in terms of the wind speed due to the duration effects for the case studied. This methodology is expected to assist in predicting the progressive failure of the low-rise building resulted from the wind-structure interaction for a cost-effective design and help with evaluating the wind intensity from the observed damage state of buildings.

Throughout the research, the DAD technique is adopted in helping with yielding more accurate predictions, wherein its application is extended from the linear prediction on the MWFRS of the metal frame structure to the nonlinear modeling on the envelope of the wood structure. Such new application of DAD would open up an avenue for the vulnerability assessment and could be used as a strong tool for the damage and loss predictions to serve for the insurance industry.

A reliable reproduction of the progressive failure process is determined by many factors such as the updated wind loading to the changing building configuration and the accurate material property of the critical connections and wood components. In this regard, the current study is preliminary, and the wind pressure data under the successive failure and the real material property should be measured and be incorporated into the modeling for a more accurate prediction. However, the current study forms a basis for the further quantitative analysis on the loss or the risk for the low-rise light-frame timber structures.

4.6 References


CHAPTER 5 . MODELING WIND LOAD PATHS AND SHARING IN A WOOD-FRAME BUILDING

5.1 Introduction

The light-frame wood buildings in the U.S. account for over 95% of all the residential structures most of which are designed as low-rise buildings [1]. For the U.S. population, around one-third resides within 100 miles of hurricane-prone coastline by 2007, i.e., the Atlantic and Gulf coasts [2], and the population in coastline areas grow steadily, i.e., from 47 million in 1960 to 87 million in 2008 [3], putting their residential house in great danger. As a result, the residential light-frame wood houses become the major source of the monetary losses caused by the extreme wind events, e.g., approximately 60% of the total insured losses for Hurricane Hugo [4].

Observations from the reconnaissance trips on the wind damage events revealed that the main source of damage in houses was the lack of continuous uplift load path from the roof down to the foundation to resist uplift winds [5], where the most common failure is concentrated on the roof sheathing and connections [5]; [6]. Such poor performance is likely the result of some factors. First, most residential buildings in the U.S. are conventional, non-engineered (or called deemed-to-comply) construction where the construction techniques are based on tradition and experience with little solid engineering input, especially under wind loads for these critical members. Second, the older house stock typically suffered more damage due to the insufficient building codes in terms of the anchor spacing and wind loads, etc. For example, the older homes in Florida built to the old code SBC experienced more damage than the buildings constructed since the adoption of 2001 FBC in Hurricane Charley [7]. Third, the misconstruction due to the poor inspections such as the missing nails and the degradation of the building component and connections resulted from the material deterioration and termite infestation, etc. lead to the weaknesses in the uplift load paths.

As stated above, one interesting phenomenon that has been repeatedly documented is that the newly built houses perform relatively well during hurricanes with little to no damage to the

Fig. 5.1. Newer building with little damage and older building in the same neighborhood with extensive structural damages after Hurricane Katrina (van de Lindt et al. 2007)
structural system, while for the older buildings, damage observations are pervasive due to the insufficient design and construction of old codes as shown in Fig. 5.1 (e.g., [5]; [7]; [8]). Such vulnerability and potential damage in the large portion of existing old building stock aroused our attention, and there is an urgent need to investigate how the load shares and distributes in the configuration that tends to induce failure in both qualitative and quantitative ways.

When the wind blows onto a building, uplift pressures could be applied onto the roof surface due to the flow separation at the leading edges, i.e., the top of the windward wall and the roof ridge. These loads will be distributed on the sheathing panels which then send the loads to the truss assemblies through the sheathing-to-truss connections (STTCs) such as sheathing nails. These loads on the truss assemblies will be further transferred to the walls via the toe-nails or metal traps, generally referred to as the roof-to-wall connections (RTWCs) that link the truss top chord to the top plate of the wall. In the end, the loads flow along the wall studs and reach the foundation through the connections such as foundation hold-downs or anchor bolts. The current consensus on the load path and load sharing is twofold: structural loads tend to follow the path of greatest resistance in terms of the stiffness [9], which thus carries a greater share of loads; the load sharing increases the capacity of individual member by distributing loads to adjacent members [10]. However, due to the high redundancy of the indeterminate light-frame wood structure that is made of repetitive frame members, the understanding on how the loads share and distribute through numerous possible load paths in different configurations under the complex wind loading condition is still limited.

In light of this, a comprehensive study on the system behavior of and the load paths in the light-frame wood structures is imperative and is the objective of the current study to improve their performance and mitigate their failure to strong winds. The scope of the current study covers a wide range of parameters that can alter the damage of houses as indicated by FEMA [7], including the gable end sheathing continuity, the gable end truss stiffness, the STTC schedule, the opening condition, and the sheathing thickness. The effects of these configurations on the building performance are analyzed with the aid of a validated 3D nonlinear finite-element (FE) model and are directly evaluated in the failure stage that goes from the linear to the nonlinear range by the first failure wind speed. The resolution of wind loads provided by the wind tunnel tests on small-scale building models is also discussed by comparing the wind effects to that of the loads derived from large-scale wind tests. The results of these investigations serve a better estimation on the performance of the existing building stock under high winds, enable the application of proper mitigation techniques, and instruct the future constructions.

5.2 Literature Review

Martin et al. [11] developed a full 3D linear rectangular building model where connections at the foundation were modeled as linear spring elements to account for the load-displacement behavior. Other connections such as the sheathing nails and RTWCs were simplified as rigidly connected, and the effect of nail spacing was incorporated by adjusting the shear modulus of sheathing. Two geometric scenarios were investigated by this model to evaluate system effects and explore the load paths under uniform uplift pressures. The edge nailing (2, 3, 4, 6, 12-in spacing considered) was revealed to affect the distribution of loads from roof to foundation, especially on the gable wall where the denser the edge nailing gets, the more evenly the loads
distribute to the foundation. For the wall opening effects, it was found the load carrying capacity for the entire wall would drop due to the occurrence of opening, and the wall opposite to the opening can also be influenced dependent on the orientation of the related trusses. However, the effects of the considered scenarios were limited to be checked on the foundation level since the foundation hold-downs and anchor bolts were the only connections explicitly represented with finite elements where the loads carried can be examined more accurately. In addition, this linear model cannot reflect load redistribution due to the nonlinear behavior of the critical components such as sheathing nails and RTWCs of a light-frame wood building under uplift loads.

This simplified linear modeling methods developed by Martin et al. [11] was later adopted by Pfretzschner et al. [12] and Malone et al. [13]. Pfretzschner et al. [12] expanded its application to a more complex L-shaped wood house to investigate the effects of reentrant corners, wall openings, and gable-end retrofits on load paths. The effect of adding the reentrant corner or the opening was found to be largely dependent on the orientation of trusses with respect to the walls. The large torsion induced by the reentrant corners might be reduced by balancing the stiffness of the walls. Openings in the wall parallel to the trusses had the least effect on the uplift reactions in the remaining walls. Effects of the retrofit were examined and showed no signs of additional torsion by modeling C-shaped retrofit at each of the gable-end stud. Malone et al. [13] took the perspective of highlighting the difference in the load paths between the timber frame (TF) and the light-frame (LF) structure. The TF was found superior to the LF in resisting both uplift and story drift because continuous posts resisted the out-of-plane wind loadings 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. However, based on the same modeling methods, Pfretzschner et al. [12] and Malone et al. [13]’s investigation on the load paths were also limited to elastic range as concluded by Martin et al. [11], and the behavior of the critical members such as the sheathing nails cannot be captured.

The modeling resolution has generally been improved at the assembly and the component level, e.g., roof structure and RTWCs. Shivarudrappa and Nielson [14] developed a roof structure model where STTCs and RTWCs are explicitly modeled with nonlinear spring elements. Sensitivity studies were performed to investigate the uplift load paths in both linear and nonlinear range by load influence coefficient contours under point loads on parameters such as the connection stiffness, sheathing stiffness, framing type, and nonlinear behavior. For the RTWCs, the load paths were found more sensitive to the overall stiffness, and their relative stiffness began to have larger impacts when they entered the nonlinear range with decreased stiffness. Compared with sheathing connections, sheathing stiffness itself had a notable impact on the load distribution. However, such a modeling on the roof assemblies only reflects a part of the entire load paths from the roof to foundation, and these load paths may shift with the load redistribution due to the neglect of the interaction with the wall system, causing the discrepancies with real case. Targeting on the modeling resolution of the RTWCs, Satheeskumar et al. [15] developed a solid model of the roof-to-wall triple grip connection that consisted of five separate parts: triple grip, nail, membrane, truss, and top plate. This model accounts for the large deformation and the contact between the nail and timber in linear and nonlinear phases up to failure. Load paths on this scale were found to be significantly affected by the nails located near the center line of the loading action in that the responses of these nails dominated the uplift capacity and failure types of the RTWCs. The verification of this model against test results showed the predictions given by the FE model were
acceptable in terms of the deformation and the failure mode. The force-displacement relationship obtained from this model could be used as a substitution of experiment measurements. The challenge with such a model is how to incorporate it into a 3D full building model.

Besides the effect of FE model techniques used, load paths and sharing in the wood house under extreme wind events are also dependent on the resolution of loading, which currently has the forms of the uplift uniform pressures, the wind codes defined values (e.g., [16]), discretized static pressures, and database-assisted design (DAD) time-history pressures [17]. The uplift uniform pressure is the most simplified version, which qualitatively represents the characteristics of wind loads that induce suctions (uplift force) on the roof (e.g., [11]; [12]; [14]). This form is easy to apply and convenient to do sensitivity studies but cannot reflect the true wind distribution. As oppose to that, other forms are quantitatively based utilizing the wind pressures measured from wind tunnels or other tools, where the wind provisions are of the lowest resolution providing pressure coefficients in prescribed zones with peak values derived from wind tunnel tests (e.g., [13]; [18]; [19]). To improve the accuracy of loads from provisions, many research adopted the similar procedure used in the development of the wind provisions to process the measured data but with finer area-averaged discretization (e.g., [20]-[23]). The modern experimental and computational techniques make possible to use the pressure time histories directly by the DAD method. Utilizing the pressures with spatial and temporal variations enable engineers to do transient dynamic analysis and to manipulate data into any target forms, such as peak values or mean values (e.g., [24]; [25]). However, the existing DAD databases are developed upon the wind tunnel tests on small-scale models and the descrepancies due to the load resolution in the load paths under the wind tunnel pressure measurements with the field test loading is unknown. Thus, explorations on the effect of the wind loading resolution on the building performance are still needed.

5.3 Modeling Methods and Loading Sources

5.3.1 Model description and modeling methods

A nonlinear numerical building model at entire building scope developed by a validated modeling methodology is used in the current study to explore the factors that affect the vulnerability of the light-frame wood buildings under extreme wind events. The footprint dimensions of the numerical model are 2.19 m (7.2 ft) wide by 3.42 m (11.25 ft) long for an eave height of 0.79 m (2.6 ft), as shown in Fig. 5.2 Fig. 3.9. It is a one-story gable roof wood house with a roof slope of 14°. The modeling methodology adopted is practical for users by directly employing the built-in features of the FE software, Mechanical APDL (ANSYS), i.e., the beam members in the truss and wall are modeled by using beam elements, and the sheathings on the wall and roof are represented by shell elements.
Fig. 5.2. Building model: (a) FE model of framing; (b) FE model of sheathing; (c) demonstration of inter-component connections

This building including both the main frame and building envelope systems was modeled in great detail. Reflecting the performance of the most vulnerable components in the wood structure under uplift wind loads, all the inter-component connections including the sheathing nails, the RTWCs, and the foundation hold-downs and anchor bolts are modeled by nonlinear spring elements. Each spring element composed of two coincident nodes at the same location accounts for the behavior of the connection in each DOF in nodal directions. The multi-linear force-displacement relationship for each DOF is applied to the corresponding spring element that is consistent with the value used by He et al. [23]. In order to accurately determine the location of each spring element and keep track of the node numbers of all the spring elements, the direct generation method is adopted for this modeling method. To capture the complex structural responses while maintaining the simplicity of the modeling technique, some assumptions are made. The material properties of the beam and sheathing wood members are assumed to be elastic isotropic and elastic orthotropic, respectively; the truss assembly is composed entirely of pinned connections; no internal compartment is considered, etc. Specific modeling features and detailed geometry of the building can be found in He et al. [23].

5.3.2 Model validation and FIU open-jet wind test datasets

The currently discussed building model was tested at the Wall of Wind (WOW) Experimental Facility (EF) at Florida International University (FIU), shown in Fig. 5.3. It was carried out to validate the nonlinear modeling methodology adopted and explore the failure modes as well as the progressive damages of residential houses under extreme wind events. The wind loads including the external and internal pressures were collected under a wind speed of 29.06 m/s (65 mph) under the wind directions varying from 0° to 180° with 15° intervals, considering symmetry of the building, for both the building model with and without door opening. Then, the wind speed was increased to 40.68 m/s (91 mph) and 46.94 m/s (105 mph) with the same incident angles without door opening. The failure occurred under the 46.94 m/s (105 mph) speed wind with the direction of 75°. This direction of vulnerability is used throughout the current study. To maintain a higher resolution of the applied pressures, the loading grid is determined directly by the number of pressure taps. That is to say, the loading is discretized into 352 areas on the building surface corresponding to the 352 external pressure taps in total, and each pressure trace is used onto its equivalent areas without further area averaging so as to reflect all the fluctuations as
measured by the pressure taps. Additional details can be found in He et al. [23]. These wind load datasets with realistic pressure distributions are applied for the analysis of load paths and sharing.

Fig. 5.3. FIU open-jet wind tests: (a) WOW set-up; (b) the validation experiment with building model

5.3.3 NIST wind tunnel database

A wind load dataset on a 1/100 scaled building model from NIST aerodynamic database contributed by the Boundary Layer Wind Tunnel Laboratory at the University of Western Ontario (UWO) is applied as to verify the effect of scale relaxation and the resolution of atmospheric boundary layer (ABL) simulated by scaled wind tunnel tests. Full-scale wind test is irreplaceable due to the incompatible similarity issues associated with the ABL physical modeling in wind tunnels such as the duplication of the Reynolds number, raising studies on the subject of the smallest model scaling of a low-rise building in wind tunnels, i.e., 1:50 recommended by Tielemans [27]. The Reynolds number effects also lead to the discrepancies on the peak pressure coefficients between the wind tunnel simulation and full-scale wind tests. Specifically, the peak pressure is determined by the turbulence intensity and the power spectrum density of the free stream where the small-scale turbulence is important for the roll-up of the separated shear layer, and the large-scale turbulence is responsible for the vortices to reach full maternity [26]–[29]. However, full-scale or large-scale wind tests are still in short. It is practical to utilize the available large amount of wind tunnel data sets, especially the aerodynamic databases such as NIST and Tokyo Polytechnic University (TPU) database that are serving for the database assisted design (DAD). Another look at the resolution of the wind tunnel data is taken from the perspective of structural response that is directly related to the response of structure rather than wind pressures. The average form of wind loads instead of the peak values is used for the current study due to its high correlation between wind tunnel simulation and full-scale wind loads.

To be consistent with FIU test, the selected data set is for the low-rise, gable roof building in open terrain (roughness length, $z_0=0.03$ m). The geometries of the two building models are similar in the roof slope ($14^\circ$) and the aspects ratio (length: width: roof height), i.e., $38.1\,\text{m} \times 24.4\,\text{m} \times 9.75\,\text{m}$ (125 ft. $\times$ 80 ft. $\times$ 32 ft.) of NIST model and $5.3\,\text{m} \times 2.3\,\text{m} \times 0.91\,\text{m}$ (11.72 ft. $\times$ 7.5 ft. $\times$ 3.0 ft.) of FIU model, with variation only in size, which is approximately 10.7 to 1. As indicated by Ho et al. [29], the mean pressure distributions on the buildings with identical aspect ratio can reach quite well agreements with slightly lower values on building with lower eave heights due to the difference in the characteristics of the turbulence at different height. Hence, the mean values of pressure coefficient data measured from total 625 taps on the model in UWO wind tunnel
experiment are adopted and the real test data on the current building model are expected to be very similar with limited decreases. The configuration and tap layout of the selected model building with a demonstration of a period of pressure coefficient (Cp) time history measured at a tap are shown in the Fig. 5.4(a). It is noteworthy that the pressure distribution measurement has taken account of the internal pressure due to the distributed leakage. A demonstration of the derived mean Cp contour and its distribution on the FE model after discretization under the wind parallel to the ridge are shown in Fig. 5.4(c) and (d), respectively.

![Fig. 5.4. NIST dataset: (a) 80 ft *125 ft *32 ft model and its tap layout with direction instruction and a sample of Cp time history; (b) mean Cp contour (θ=180º); (c) discretized mean Cp applied on the FE model (θ=180º)](image)

5.4 **Load Distribution Parameter Study**

5.4.1 **Geometric and loading scenarios**

For the load paths investigation, the Case 1 building model is set as the control case that is made up of conventional configurations as listed in Table 5.1. Then, this model is altered systematically in configurations and loading forms to perform parameter studies including the gable end sheathing continuity, gable end truss stiffness, STTC schedule, opening condition, sheathing thickness, and the loading resolution. A detailed description of parameters used in each case can be found in Table 5.1.

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Cases 1 and 2 investigate the effect of gable end sheathing continuity on the load paths and building integrity. In Case 2 model, the two-piece gable end sheathings are connected to the frame with the same conventional nailing schedule as adopted by Case 1, i.e., 6 in. /12 in. along the exterior panel edge (edge nailing) and the intermediate supports (field nailing). Three conventional gable end truss types are discussed by Cases 1, 3, and 4 including the fink, gable, and queen type that have the similar web numbers but vary in web configurations. Cases 1, 5, and 6 vary in the STTC schedule, of which the 6/6 and 6/12 correspond to the new and old building code, respectively. The schedule 6(36)/12 is consistent with that of the FIU wind test model with the STTCs along the side walls intentionally eliminated to 36 in. to weaken the load path. The changing in the stiffness of walls and the load paths resulted from adding opening is discussed by Cases 1, 7, and 8, in which the model has closed door, open door, and no door, respectively. The sheathing thicknesses considered correspond to the common OSB sheathing panels in the market, as shown in Cases 1, 9, and 10. The wind loading resolutions from different sources are examined by Cases 1, 11, and 12.

The effects of these parameters on the load paths under 100 mph wind are displayed by von Mises (VM) stress. It is an equivalent stress combining the stresses in all three directions into a single index that gives an appreciation of the overall magnitude of the tensor, and it is often used as an indicator of the failure by ductile tearing. The structural response at the critical members shown in Fig. 5.5 including the RTWCs of the building model and the STTCs of the front roof sheathing panel on the right, provide another way of looking at the load distribution under the systematically changing parameters.

![Fig. 5.5. Plan view of roof sheathing panels with connected frames showing the locations of RTWCs (blue square symbols) and STTCs (green circle symbols). (F=front, B=back, L=left, R=right, M=middle)](image)

### 5.4.2 Failure threshold

The first failure wind speed and location are chosen as the final indices to reflect the effect of parameters discussed. The wind speed increases at an interval of 1 mph until failure. The STTC withdrawal failure is the only threshold considered as it is the dominant failure mode as witnessed in the past due to the relative lower capacity than the other connections, and therefore, it is efficient
to use it to reflect the first failure condition. Allowing for the time accumulation effects, the failure criteria of the sheathing nail are chosen as the withdrawal force reaching its capacity (680N) rather than the nail exceeding a relative displacement representing the complete pullout.

5.4.3 Effect of gable end sheathing continuity

The loss of sheathing on the gable end walls is a common failure observed by the past reconnaissance of wind damage, as shown in Fig. 5.6. The failure of the vulnerable gable end wall often leads to the pressurization and the complete collapse of the side of the structure. The purpose of this section is to investigate the effect of the gable end sheathing continuity on the sheathing behavior and the structure integrity as well as the load sharing on the critical connections. As stated above, since the Cases 1 and 2 models are installed with the one-piece and two-piece gable end sheathing, respectively, with the same nailing schedule, the difference between the two models is twofold: Case 2 model has one more piece of sheathing and one more line of edge nailing.

Fig. 5.6. Gable end wall failure: (a) Hurricane Charley [7]; (b) Hurricane Katrina [8]

The general difference caused by the gable end sheathing continuity is displayed by VM stresses of wall sheathings on the gable and the building corner in Fig. 5.7. Case 1 model with one-piece gable walls shows a stress concentration right beneath the roof ridge on both the gable end sheathings in Fig. 5.7(a) and (b). As opposed to that, the load is more evenly distributed by the two-piece gable sheathing model, i.e., the Case 2 model, with the smaller nailing tributary areas under the same loading condition. For the area below the eave height, the Case 1 model experiences higher reactions on the sheathing of the windward wall (right end wall), with the maximum increased by 19.48% occurring around the field nailing at lower height than that of the Case 2 due to the direct interactions with wind on the triangle areas. However, the stresses on the leeward wall (left end wall), subjected to suction wind loads are slightly higher in Case 2 model with maximum increased by 1.98% (from 599418 N/m² to 611559 N/m²). In looking at the front wall on the corner in Fig. 5.7(c), the stress concentration and deformation are higher in Case 2 in response to the sacrifice of the structural integrity by breaking up the load path on the gable wall. However, since the front wall is not as vulnerable as the gable end walls or the roof assemblies, this weakness caused by the sheathing discontinuity is not significant.
Fig. 5.7. VM comparison of gable wall sheathing continuity: (a) left wall (deformation scale=100); (b) right wall (deformation scale=100); (c) windward corner (deformation scale=40)

Fig. 5.8 shows the withdrawal forces of gable end sheathing nails at the bottom chord where the sheathing discontinuity is discussed. As the nailing gets denser, the wall would become stiffer and capable of distributing the wind loads more evenly through its sheathing nails. As expected, with the fewer nails, each sheathing nail in Case 1 carries more loads than the corresponding edge nailing in Case 2 on each of the two sheathing panels under the same wind loading. The Case 1 maximum force reaches 30.19 N on nail #6 of the left wall. The summation forces in absolute values carried by the sheathing nails at eave height in Case 2 are also smaller than that in Case 1 and have smaller variance, e.g., Case 1(105.58) vs. Case 2_Down (23.96) and Case2_Up (2.17) vs. Case 2_Sum (22.16) for the right wall. The practical implication of this finding is that even
with the sacrifice of the sheathing continuity, installing more sheathing nails may increase the structural resistance to winds.

Fig. 5.8. Withdrawal force of gable sheathing nails at the bottom chord: (a) left wall; (b) right wall

As the sheathing discontinuity breaks up the original load paths in Case 1, the loads redistribute and lead to the change in the direction of forces acting on the sheathing nails in Case 2. On the leeward side in Fig. 5.8(a), the Case 1 sheathing nails at eave height are subjected to the suction force; while in Case 2, the sheathing nails at the same height under the same wind condition are loaded with compression on both upper and lower piece. A similar trend is observed on the windward side in Fig. 5.8(b), where all the nails in Case 1 are under compression, but the corresponding sheathing nails in Case 2 on the upper side are in tension.

The withdrawal forces in sheathing nails at the top chord are shown in Fig. 5.9. Unlike those on the eave height, the sheathing nails along the roof edge in both the Cases 1 and 2 generally sustain compression on the left wall and tension on the right wall. The overall absolute forces are larger in Case 2 for the left wall and in Case 1 for the right wall, which is consistent with the stress concentration results discussed above. Exceptions are on the #1 sheathing nails on both the end walls that have larger absolute forces in Case 1. The negative force representing the nail enduring compression does not induce neither the nail shank withdrawal from the lumber nor the nail head pull-through of the sheathing panel, and thus can be ignored when considering failure. For the positive values, the sheathing nail receives its maximum of 5.54 N on the #6 nail of the right wall which is still way smaller than the peak value on the eave height, indicating that Case 1 model is more vulnerable on the gable end based on sheathing nails discussed.
Fig. 5.9. Withdrawal force of left gable sheathing nails at top chord: (a) left wall; (b) right wall.

The plots in Fig. 5.10 provide another way of looking at the structural stability influenced by the sheathing continuity at the gable ends subjected to wind loads. The uplift load distribution on RTWCs including the five connectors on the back roof marked as B and the five connectors on the front roof denoted as F are considered. In Case 1, the uplift forces transferred to the connectors at the end trusses, i.e., B1, B5, F1, F5, are less than their counterparts in Case 2. Close investigation of this phenomenon reveals that this difference is induced by the contribution of the shear force provided by the sheathing nails at the gable end in Case 1 model. As the gable end sheathing continuously past the RTWCs and connected to the wall below, the shear force of the nails on the sheathing panels provides additional uplift connections between the roof assemblies and the walls. Thus, the load share taken by the RTWCs would decrease accordingly. The RTWCs in Case 1 take approximately 5% (1742.1N vs. 1822.9N) less total uplift force than that in Case 2. Compared with RTWCs on the gable end, the ones connected to the interior trusses sustain higher uplift loads under the loading condition discussed. The maximum result in Case 1 occurs on the F2 RTWC with 309.9 N that is higher than the peak values in Case 2. This reflects that breaking up the continuity on the gable end sheathing as well as the integrity of the structure will not increase the vulnerability of RTWCs under uplift wind loads.
5.4.4 Effect of gable end truss stiffness

The introduction of the metal plate to connect wood trusses in the roofs of residential light-frame buildings in the mid-1950s significantly simplified the complicated system that was consisted of lumber rafters and board sheathing constructions before [30]. This change on the truss makes possible of its design to virtually any imaginable configuration and profile. This is the case, especially for the example of the gable end trusses, where they are most often built above the end wall saving the contractor the time and expense of field framing the end wall to match the roof slope [31]. However, it is imperative to remember these gable end trusses are parts of and must be incorporated into the design of the end wall to function integrally.

The increased gable end stiffness is confirmed to attract more load and lead to the overloading of the STTCs on the roof as well as the removal of the roof sheathing due to this increased demand [118]. However, the effect of it has yet to be studied in the analysis of the roof sheathing failure in terms of the withdrawal of the STTCs. Confusions also exist as which of these existing web configurations functions better. Reflecting these issues, three common truss shapes with the similar material quantity as shown in Table 5.2 are selected as the gable end trusses in this section to verify the effect of different truss types to the structure performance. The analysis on the withdrawal failure of the STTCs is also completed and will be presented later.

The building resistance to lateral loads is influenced by the gable end truss and provided by the sheathing, nail, and bracing, of which the first is the same for all the three types discussed whereas the rest two are different. The gable truss type is used for the two exterior trusses in Case 1 model with five vertical webs and one sheathing connector on the web in the middle. Case 3 model is built on the fink style of end truss where four oblique webs are included without any sheathing connectors on the web, but the sheathings are connected on the top and bottom chords. Compared with Case 3, Case 4 model with the same component quantity has one less web and one more sheathing connector in the queen style end truss adopted, where two oblique webs are symmetric about a vertical web in the middle with one sheathing connector. The effect of the truss shape related gable end stiffness is illustrated through the withdrawal forces on the sheathing nails that are suspicious to fail, i.e., the ones at the bottom and top chord on the end wall.

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<td>Gable Web</td>
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<td>Web sheathing nail</td>
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</table>

In Fig. 5.11, the effect of the gable end truss type on the sheathing panel in terms of the force distribution can be seen. There is a strong similarity in the pattern of the stress contours between Case 3 and Case 4. The stresses in Case 3, i.e., from 12649 to 574868 N/m² and 16388 to 445461 N/m² for the left and right wall, respectively, are close to the stress distribution in Case 4,
The difference in the corresponding upper and lower limit of the two sets of stress ranges for Case 3 and Case 4 is as low as 1.3%. Opposed to that, the sheathing panels in Case 1 experience much lower force, which is evidenced by its stress range of 10156 to 342154 N/m² for the left side and 5927 to 29938 N/m² for the right side with the least absolute difference to Case 4 being 27%. One may conclude that the gable truss type with more webs and sheathing nails is stiffer than the fink and queen type.

The withdrawal forces on the sheathing nails cross the side walls demonstrate a similar variation along the same distance. Whenever a sheathing connector is lined up with the wall stud, e.g., nail #1, #4, and #7 in Fig. 5.12, the ability of the gable end sheathing panel to transfer loads to this connector is limited. For the connector does not line up with the wall stud, as the case of nail #2 and #3 in Fig. 5.12, etc., the force transferred to withdraw the connector is notably larger, a similar observation noted by Jacklin et al. [118] when investigating the load transfer on RTWCs by using the influence coefficient contours. This phenomenon is unable to detect in a simplified numerical model where either a single sheathing panel is modeled along the entire span [30][11], or beam elements are used to model the sheathing.

Of all the sheathing nails discussed, the ones at bottom chord on the left wall experience the highest withdrawal forces as shown in Fig. 5.12(a). Both the fink (Case 3) and queen (Case 4) trusses result in a similar load distribution over the gable end sheathing nails, differing by only 1% on average. Limited variance between the two cases occurs at the nails right beneath the roof ridge, i.e., nail #8 and #9, where the fink truss is higher by about 13%, indicating the sheathing nail is more effective in strengthening the stability of the gable end wall than the web when applied with the same amount. The maximum value in the fink shape truss is almost double that of the gable shape truss. As expected, with more components, the stiffer gable shape truss experienced the lowest demand on the sheathing nails attached, and thus can withstand much more forces.
Fig. 5.12. Withdrawal force of left gable sheathing nails at bottom chord: (a) left wall; (b) right wall

Fig. 5.13. Withdrawal force of left gable sheathing nails at top chord: (a) left wall; (b) right wall

Fig. 5.14 takes another perspective of the uplift capacity of roof structure on the critical locations including the STTCs and the RTWCs to examine the effect of gable end truss stiffness to the vulnerability of the structure. It is again found that the fink and queen truss shape models demonstrate very similar results on these critical locations and are higher than that in the gable truss shape model. This increased demand in the already weaker trusses would result in the overloading of the critical points for the light wood structures under wind loads followed by the removal of roof sheathing panels. The difference in the uplift forces of the STTCs on the front right roof sheathing panel is not that significant especially on the edge nailing. However, for the uplift forces on the RTWCs, the maximum in the fink and queen truss shape models are over twice as large as that of the gable truss shape model.
In the truss industry, the gable end frame is classified as the non-structural gable end frame having continuous support along the entire span or the structural gable end frame with bearing at specific locations [33]. The former type is called so in that it is not designed to transfer the load between bearing walls along the span, and thus, the web members are oriented vertically and function as load carrying members only in vertical direction. As opposed to that, the latter is designed to carry loads over openings in the end wall containing both diagonal and vertical web members. As the capability of gable end frame to transfer loads from one bearing wall to another across the span could enhance the building stability, more analysis should be completed from this perspective by comparing the structural responses between constructions built with non-structural and structural gable end frames.

5.4.5 Effect of STTC schedule

The roof sheathing failure was observed as the most common failure for wood-frame buildings under winds resulted from inadequate connections to the underlying roof frame leading to discontinuous load path [5]; [34]. Even a single nail failure could often trigger the progressive failure of an entire roof. In other words, a proper installation of sheathing nails determines the performance of the roof structure as well as the entire structure, and a good command of the effect of nailing schedule is critical to the accuracy of the building performance predication. Therefore, this section targets at the effects of the STTC schedule on the uplift capacity of the roof structure by the comparisons between Cases 1, 5, and 6.

According to the US Census Bureau [35], over 80% of the United States’ residential structures in hurricane-prone areas were built before 1994, the year in which the code was upgraded due to Hurricane Andrew. This vast proportion of building stock was built on the old building provisions that specified the STTCs to be 6d smooth shank nails spaced at 6 in. / 12 in., e.g., the Florida Building Code [36]. This nailing schedule is represented by Case 1 model. In the building provision after 1994, the minimum requirement for the STTCs has updated to be spaced at 6 in. / 6 in. (e.g., [37]), and this schedule is analyzed by Case 5 model. Additionally, other STTC schedules would apply due to the construction defects such as missing nails or the different requirement in the building codes from different geographic regions and are accounted for by Case 6 model which also adopted in the FIU wind test model.
The VM stress distributions on the roof sheathing panels for Case 1, Case 5, and Case 6 are shown in Fig. 5.15. As the field nailing gets denser, the sheathing panels would become stiffer and are able to distribute the wind loads more evenly [11]. By comparing the first two cases, this effect of nailing density is not significant for the current building configuration, where the maximum stress decreased by only 0.6% (1.6E6 N/m² of Case 1 and 3.73E6 N/m² of Case 5) under Case 5 nailing schedule at the same location. By comparing Case 1 and Case 6, one may note that without the edge nailing on the side walls, Case 6 model with fewest nailing experienced the highest stress of 3.73E6 N/m² with an increment as large as 133.1% to that of the Case 1. The location of the maximum stress changed to the nail with the lowest nailing density.

![Fig. 5.15. VM comparison between roof sheathing (deformation scale=50)](image)

Fig. 5.15. VM comparison between roof sheathing (deformation scale=50)

![Fig. 5.16. Uplift force on: (a) STTCs; (b) RTWCs](image)

Fig. 5.16. Uplift force on: (a) STTCs; (b) RTWCs

Fig. 5.16 presents the effect of the roof sheathing nail schedule on the demand of the critical connections in the wood frame structures under the winds including the STTCs and the RTWCs. In Fig. 5.16(a), take Case 5 as a benchmark, one can see that no matter whether the sheathing nails are missing in the field nailing such as Case 1, or with more nails missing in the edge nailing such as Case 6, the field sheathing nails are more sensitive to these changes than the nails on the edge. Also, the nail with less nailing density due to the nail missing around is as expected to have higher uplift force, e.g., L1 and R1. A similar trend is also observed in the uplift force on the RTWCs,
where the one lines up with the field nailing is more sensitive to the roof sheathing nail schedule but with much smaller variations than that of the STTCs. One can conclude that using the sheathing nailing of higher density helps increase the capacity of both the roof sheathing panel and the RTWCs.

5.4.6 Effect of opening

The pressurization caused by the internal pressure from the broken window or door is found to be an important factor in the structural failure. As shown in Fig. 5.17 that was taken after Hurricane Charley [7], one condominium without shutters lost most of its upper floor framing on the top unit; while the other one located two buildings away with the similar configuration but protected by shutters survived the storm relatively unscathed. It was the shutters that protected window and doors from debris, keeping the condo “enclosed” and preventing the generation of internal pressure pushing the roof to fail.

Fig. 5.17. Same type of building nearby after Hurricane Charley (FEMA 2005)

Reflecting on this dramatic failure of the roof structure contributed by the internal pressure, there has been active research on the quantification of internal pressures with various influencing parameters [92]-[76]. In contrast, the study is very rare focusing directly on the effect of the internal pressure on the building response or the structural failure, especially the decrease in the failure wind speed resulted from the occurrence of significant internal pressures. Furthermore, this information that relates the wind speed to the building damage is important for the damage and loss prediction for the insurance company. Thus, the effect of opening on the building performance is discussed in this section, and the failure part will be illustrated in a separate section later.

Generally speaking, the internal pressure would increase significantly due to the occurrence of the opening, together with the uplift external pressure pushing up the roof, leading to the higher stress intensity on the roof sheathing. Therefore, the discussion here focuses on the stress on wall with opening instead of the roof as shown in Fig. 5.18. An obvious point to be made is that the presence of the opening will greatly reduce the probability that the front wall will be broken under extreme wind events. The peak pressure value of the part of the front wall presented has decreased by 72.9 % from the 622557 N/m² in Case 1 to the 168412 N/m² in Case 7. As for the modeling method for the opening, whether the door is modeled in a separate sheathing panel does not have much effect to the stress prediction that the peak stress of Case 1 and Case 8 occurs...
at the similar location with the similar value. Fig. 5.19 shows the uplift forces on the critical connections, i.e., the STTCs and the RTWCs, under the different opening conditions. As expected, the connections of either kind carry more loads than the uplift forces suffered by the connections without the opening. The peak forces on the STTCs and the RTWCs in Case 7 are more than doubled the peak values for Case 1. Again, the modeling method of the opening also has little effect on the capacity of the critical point.

One limitation of the current discussion is that to be consistent with the FIU wind test model that had both the internal and external pressure data measured, the building model has only one equivalent opening, i.e., a door positioned in the center of the side wall. Thus, it is not representative of the typical residential structure. More analysis on building models with various opening conditions should be completed on this topic.

Fig. 5.18. VM comparison on the opening effect (deformation scale=350)

Fig. 5.19. Uplift force on: (a) STTCs; (b) RTWCs
5.4.7 Effect of sheathing thickness

The effect of sheathing thickness, i.e., stiffness, is demonstrated in Fig. 5.20 and Fig. 5.21 by comparing the structural responses from models different in the sheathing thickness. As the thickness increased by 64.3% and 100% from the control case, the maximum VM stress of Cases 9 and 10 decreased by 31% and 78%, respectively. One may conclude that the sheathing capacity can be effectively increased by simply changing the thickness.

Fig. 5.20. VM comparison of the sheathing thickness (deformation scale=60)

Fig. 5.21. Uplift force on: (a) STTCs; (b) RTWCs

The capacity of the critical connectors is also influenced by the sheathing since it is the mechanism by which the loads are distributed. The sheathing panel facilitates the force distributions among the subcomponents of a structure, which has been found to be influenced by the relative stiffness of the sheathing and frame members to some extent [41]. In Fig. 5.21(a), the uplift force distributed to the STTCs on the edge nailing is barely affected by the sheathing stiffness, while for the field nailing, the force is more evenly distributed to the STTCs with stiffer sheathing but not to a large degree. For the RTWCs, Shivarudrappa and Nielson [14] concluded that the low sheathing stiffness requiring a single RTWC to carry a higher share of the load applied directly over it. This applies especially on the RTWCs in the middle where the sheathing panel gaps are.
Overall, the thicker roof sheathing panel would decrease the chance of both the sheathing panel itself and the critical connections to failure.

### 5.4.8 Effect of wind loading sources applied

The purpose of this section is to compare the effect of wind loading sources with different resolution on the building performance to gain further insight into the load sharing. The wind loading for Cases 1 and 11 models are provided by a 1/4 large scale and 1/100 small sale building model measurements, respectively. Case 12 model is applied with equivalent uniform pressures that match the realistic wind pressure distribution of Case 1 in terms of the global uplift force.

Fig. 5.22 shows the stress distribution on the roof sheathing panels subjected to each of the three wind loading sources discussed. One may observe that the wind loadings derived from small-scale model result in a similar stress distribution pattern with that under the loads from large-scale model but with discrepancies on the magnitude especially near the roof leading edges, roof corner, and roof ridges. This indicates that the small-scale wind tunnel tests cannot reproduce the peak pressures on the roof regions under conical vortices or separation bubbles attributed to the missing of large eddies, a characteristic has been noted elsewhere (e.g., [65]; [66]) by comparing the pressure coefficients from the full-scale and wind tunnel test measurements. For the uniform pressure results, the stress on the roof sheathing achieve the peak value around the field nailing implying that the capacity of the sheathing is influenced by the nailing density.

![Fig. 5.22. VM comparison of load resolution (deformation scale=50)](image)

Fig. 5.23 presents the comparisons of the uplift force on the STTCs and the RTWCs under the three loading sources. Based on the connectors examined, the wind pressures from the small-scale model underestimated the maximum uplift forces, and thus, the loading sharing under such wind loads is not sufficient to analyze the structural behavior. It is noteworthy that the extent of the underestimation due to the scaling effect may not be as large as it is shown in the figure, since even measured in wind tunnel on the model with the same scale, the pressures can be different from laboratory to laboratory, e.g., the international round-robin set of wind tunnel tests of a low-
rise structure conducted at six reputable laboratories [44]. For the equivalent uniform pressure, it is interesting to find that even without the consideration of the wind incident direction, from laboratory to laboratory, e.g., the international round-robin set of wind tunnel tests of a low-rise structure conducted at six reputable laboratories [44]. For the equivalent uniform pressure, it is interesting to find that even without the consideration of the wind incident direction, the uplift forces on the critical connectors examined exhibit similar results. Additionally, the peak values are higher than that of the Case 1 results, indicating the equivalent uniform pressure can be sufficient to be create a similar behavior to the realistic pressure distribution under certain circumstances.

![Fig. 5.23. Uplift force on: (a) STTCs; (b) RTWCs](image)

### 5.5 Results: total VM & failure

To give a whole picture of the influence from all the parameters studied, the maximum VM stress for the entire building surface of each case is summarized in Fig. 5.24. Besides, further analysis is conducted on the first failure wind speed and location of critical connections in Fig. 5.25, two indicators that sheds lights directly on the question of the influence of the geometric parameters and loading resolution on the building performance. This connection results together with the sheathing response provides a better understanding of the load sharing in the light-frame wood house in overall and localized scale.

By comparing the results of Case 1 and Case 2, it can be seen that breaking up the sheathing continuity at the gable end has little effect with limited decrease in the sheathing demand and increase in the connection demand in terms of the maximum stress by 1% and the failure wind speed by 4%, respectively. The effect of gable end truss stiffness determined by the truss shape is apparent on the sheathing behavior with the maximum stress increased by over 90% in both Cases 3 and 4 to the gable type Case 1 model. However, there is essentially no notable difference between the demand on the critical connections under the different truss shapes. The differences between Cases 1, 5, and 6 are significant, which are conducted to compare the effect of the sheathing nail schedule to the building vulnerability. With more field nails, although the resulted peak sheathing stress is almost equal, there is a pronounced increment by as large as 40% in the highest wind speed that the building can take. Opposed to that, with missing nails on the roof edge, the sheathing panels are under higher risk of failure, while the capacity of the sheathing nails barely changes. The effect of opening is significant as demonstrated in Case 7 that the sheathing is subjected to
over 100% higher forces, and the highest wind speed plummets 19% to 164 mph suggesting that the building resistance to the winds has been greatly weakened. As for the way that the door is modeled, it has little to no effect on both the sheathing and nail response as shown in the results of Case 8. And one may reasonably conclude that there is no need to incorporate the specific modeling of openings such as the door or the window for the enclosed condition in the analysis of the building performance under winds. By comparing the results of Cases 1, 9, and 10, the benefit of having the thicker sheathing is obvious in enhancing the capacities of both the sheathing and critical connection.

For the wind loading of three different resolutions, the one measured from small-scale building model leads to a higher peak stress on the sheathing panel by 33% and an increase of 19% in the first failure wind speed compared with the results under the wind loads derived from large-scale model. This suggests that using the wind pressure data from small-scale wind tunnel tests such as the NIST database is conservative for the sheathing design but unconservative for the design of critical connections which govern the vulnerability of building in extreme wind events. From the results of Case 12 model which is subjected to the uniform loading, the predictions based on it underestimate the maximum force on both the sheathing panel and the STTCs.

Fig. 5.24. Maximum VM stress of each case (with percentage difference to Case 1)

Fig. 5.25. First failure wind speed (with percentage difference to Case 1) and location of each case
The location of the first failure STTC partially reflects the load paths and distribution in the structure. Most of the cases, i.e., Cases 2-9, and 11, fail at the same place with the control case, Case 1 as shown in Fig. 5.25, indicating the corresponding parameters discussed does not change the failure sequence to some extent. Exceptions exist on Cases 6, 10, and 12. The first STTC to fail in Case 6 that has missing nails on the roof edges is on the front roof sheathing, which is different from the control case, emphasizing the significant influence of the nails schedule especially on the edge nailing to the load paths of the structure. It is also noted that failure beginning from the front right roof sheathing in Case 6 is consistent with the phenomenon observed from the FIU destructive wind test, which further verified the modeling methodology in the failure stage. From the different failure locations of Case 10 to the control case, it is interesting to find that the load sharing is more sensitive to the sheathing thickness than many other building configurations by changing the relative stiffness of sheathing to framing and connectors. As for Case 12, the first failure location subjected to the uniform loading is reasonably different from that of the control case under realistic wind loading.

5.6 Conclusion

This study aims to enhance the understanding of the effect of parameters that have great influence on the load paths and especially are critical to the failure of older buildings. This is done by conducting a parameter study on a 3D FE building model subjected to various geometric and loading scenarios such as the gable end sheathing continuity, the gable end truss stiffness, the STTC schedule, and the opening condition, etc. The results of these investigations can serve a better estimation on the performance of the existing building stock under high winds, enable the application of proper mitigation techniques, and instruct the future constructions:

- Breaking up the sheathing continuity on the gable end changes the load sharing and even the direction of the way that loads are distributed, but it does not much weaken the structure. The structural integrity is compensated due to the higher nailing density by adding extra sheathing nails at the breakup according to the same nailing schedule. This higher nail density at gable end increases the capacity of the RTWCs at the interior truss and the sheathing nails on the gable walls.

- The truss shape that has more webs and nails at gable end greatly increases the capacity of the sheathing and RTWCs but has little effect to the failure of STTCs. The fink and queen shape trusses with the same components quantity exhibit similar performance.

- The roof nailing schedule strongly influences the resistance of the building especially the roof field nails to winds. Missing nails at the roof edge will change the load sharing and lead to a different progressive failure of the house.

- The occurrence of opening on the wall decreases the load carried by that wall and increases the building vulnerability to winds by 20% in terms of first failure wind speed. No specific modeling is needed on the door or window for the enclosed condition in the analysis of the building performance to the wind.
• Overall, one of the most efficient way to mitigate the failure of the light-frame wood structure to wind loading is to be installing extra sheathing nails, especially on the field nailing. And then, choosing a thicker sheathing panel also helps in building a stronger house. Of all the geometric parameters examined, missing nail leads to the worst case and should be avoid in the construction.

• Unconservative building design can be induced by using the wind loads from the small-scale wind tunnel tests. Uniform loading is not sufficient to reflect the load sharing and building behavior under wind load distribution.

5.7 References


He, J., Pan, F., Cai, C.S. Assessment of ASCE 7-10 for wind effects on low-rise wood frame buildings with database-assisted design methodology. 2017a, In Process


[38] Holmes, J.D. Mean and fluctuating internal pressure induced by wind. Proc., 5th Int. Conf. on Wind Engineering, Fort Collins, Colo. 1979; 435–450.


CHAPTER 6. ASSESSMENT OF ASCE 7-10 FOR WIND EFFECTS ON LOW-RISE WOOD FRAME BUILDINGS WITH DATABASE-ASSISTED DESIGN METHODOLOGY

6.1 Introduction

In North America, above 90% of residential buildings are designed as light-frame wood constructions [1]. As reported by Pielke and Landsea [2], the United States has at least a one-in-six chance of suffering hurricane-induced damage of at least $10 billion (in normalized 1996 dollars) each year. The vast majority of these damages are the result of the failure of wood-frame residential houses [3]. Specifically, in 1992, Hurricane Andrew resulted in $26.5 billion economic loss, which marked the largest loss caused by a natural disaster that the United States had ever experienced at that time. The inadequate performance of houses during Hurricane Andrew prompted improvements in detection capability for storms and the upgrade of building codes and standards [4]. Although the buildings constructed with the upgraded code had a “clearly superior performance” as indicated by Reinhold [5], the code of practice is still insufficient as observed in 2005 Hurricane Katrina that resulted in $108 billion of damage and broke the most destructive and costliest storm record in the history of the United States. Substantial improvements and strengthening are to be made on the basis for and the process of the codification of wind loads. However, for serving as a design standard that must be practical for engineers and construction workers, simplifications that would induce the underestimation of the wind effects are inevitable to avoid bulky documents with overly complex provisions. Thus, this study takes another perspective to reinvestigate the adequacy of the ASCE 7-10 [6] in the structural response prediction on light-frame wood houses, which requires an appreciation of the methodologies that lead to the current provisions.

The ASCE 7 allows for the design wind loads on different components of a low-rise building that are categorized into the Main Wind Force-Resisting System (MWFRS) and the components and cladding (C&C). The wind loads for MWFRS can be determined by using either the Envelope Procedure or the Directional Procedure on the basis of the ASCE 7, also referred to as low-rise procedure and all heights procedure, respectively. The pressure coefficients developed within the framework of the Envelope Procedure are the “pseudo” loading conditions that envelop the desired critical wind effects, i.e., the peak bending moment at the knee of the two-hinged frame and three-hinged frame, bending moment at the ridge of the two-hinged frame, the total uplift, and the total horizontal shear, based on the work at the University of Western Ontario (UWO) by Davenport et al. [7].

As opposed to the Envelope Procedure, a more general envelope approach is adopted in the Directional Procedure, where the pressure coefficients reflect the actual peak loading on each surface of the building as a function of the wind direction. As such, it can be expected that the Envelope Procedure that is derived directly from the structural actions would predict more accurate reactions at the postulated critical members than the Directional Procedure when the configuration considered is consistent with what the provisions are developed upon. Isyumov and Case [8] extended the application of the pseudo-pressure coefficient to another structural system, i.e., a single-story shear wall structure with a truss roof by comparing the the response of it to that of a moment frames structure as used in the development of ASCE 7. Five more postulated critical
structural actions are considered in accordance with the new structure type. They are the maximum shear in the north-south walls, the maximum shear in the east-west walls, the maximum uplift at truss reaction, the maximum positive member force for truss, and the maximum negative member force for the truss. It is noted that for other structures, especially for other different types of structural systems, as suggested by Trautner and Ojdrovic [9], there is no guarantee that the structural actions selected for the development of wind loads under the Envelope Procedure will also be critical for the design, as the structural behaviors are governed by building configurations.

The C&C consists of components (i.e., fasteners, studs, and roof trusses) and cladding (i.e., wall coverings, roof coverings, exterior windows, and door) that receive wind loads directly or from each other. The pressure coefficients for the C&C, with an attempt to address the “worst case” loading scenario on a particular member during the wind event [10], are developed by using an envelope approach that is different from the method followed by the Directional Procedure. It involves spatial and temporal averaging of point pressures over an effective area through 360° wind angles to account for the small effective area of a particular component. As such, for the pressure coefficients given in the C&C chapter, the directionality of wind has been removed, and the surfaces of the building have been “zoned” to reflect the envelope of the peak pressures in the horizontal direction besides the vertical direction considered in the directional method of the MWFRS. The influence of exposure has also been removed since the design wind pressures for the C&C are intended to be based on the exposure category resulting in the highest wind loads for any wind direction at the site. The larger wind effects of the C&C than the MWFRS wind loads on the structural system are found by Martin et al. [11] by applying both of them on a numerical model. This result is not surprising in that the spatial coherence of the pressures is greater between pressures acting over small than over large surfaces. From wind engineering perspective, the larger area that covers the MWFRS contains more vortices, each of which can be considered by its resultant force, and some of the vortices would cancel out each other, resulting in a limited resultant force. In contrast, for the C&C typically dealing with small areas, some vortices would cover the entire element, leading to the resultant forces larger than the “canceled out” values of the MWFRS.

Studies on the ASCE 7 evaluation is numerous in the literature and can be categorized into two levels by the code performance indicator: peak pressure coefficient or peak structural response that is consistent with the methodology of the Directional Procedure and the Envelope Procedure of the MWFRS, respectively. The significant underestimations based on the code procedures have repeatedly been pointed out on the pressure level and the degree of discrepancy in the temporal and spatial averaged pressure coefficients depends on factors such as roof zone, building shape, size of the effective area, etc. (e.g., [12]-[14]). Taking advantage of the development of database-assisted design (DAD) technique, the response level approach is adopted more in recent years, and the highly non-conservative wind effects of the ASCE 7 are also recognized in the Envelope Procedure. Such risk-inconsistency is found to increase with the building height by St. Pierre et al. [15] and Coffman et al. [16] with the NIST database, and it also increases with the increase of the roof angle as stated by Kwon et al. [17] with the TPU database implemented by DEDM-LR. To be consistent with the ASCE 7 Envelope Procedure, these evaluations only compare the reactions of the MWFRS and are based on industrial pre-engineered metal buildings with single-story moment resisting steel frames as shown in Fig. 6.1(a). The selected structural reactions in terms of influence coefficients at postulated critical members are also consistent with the critical demands in the development of the ASCE 7 as code performance indicators. However, in the case
of residential structures of which over 95% are light-frame wood buildings in the U.S. [25], such indicators are no more rational.

Fig. 6.1. Typical structural system for (a) steel frame [16]; (b) light-frame low-rise wood building

For the light wood construction (Fig. 6.1b), the predominant damage is not a structural failure, but a failure of the building envelope, such as doors, windows, and the roof systems [18]. Once the envelope is breached, the rains accompanying a hurricane can intrude to the building resulting in major interior damage. Meanwhile, the internal pressure increases rapidly leading to a significant overloading on both the MWFRS and C&C that they are probably not designed to handle. Thus, except for the strength of each structural component, the integrity of the entire building relies heavily on the adequacy of the connections between components to properly transfer the forces. The critical demands in the configuration of wood buildings under wind loads are the uplift forces along vertical load paths, particularly at roof-to-wall connection (RTWC) and the sheathing-to-truss connection (STTC) [19]-[21], rather than the bending moment at the knee and ridge, etc., as considered in the development process of the ASCE 7. As such, the design of residential buildings with the ASCE 7 provisions which fail to incorporate the critical structural responses of this type of configuration would cause the wrong estimation of wind loads and result in the unexpected vulnerability of the structure during extreme wind events. To the authors’ knowledge, the evaluation of the applicability of the ASCE7-10 on the wood frame residential building in terms of responses is still missing. The reason behind is partly due to the lack of having a significantly detailed and validated finite-element (FE) model. This model should be able to reflect the actual performance of wood houses under wind loads by modeling all the connections which determine the load paths and even the initial collapse, as summarized by He et al. [22]. According to the discussions made above, a few notes are ready for the present study:

1. The purpose of this study is to evaluate the adequacy of the ASCE 7 code-specified procedures for wind design of the residential structures that typically consist of the light-frame wood building. The wind load effect in terms of the peak pressures that are dependent on the building exterior geometry is not discussed here since they are not influenced by the load paths and therefore, cannot reflect the adequacy of the ASCE 7 on to a different type of structures. Instead, a validated numerical model with detailed component simulation such as nails is used to evaluate the ASCE 7 in the response level, i.e. the peak response.
2. Both the MWFRS (including two procedures) and the C&C methods in the ASCE 7 are applied to compare the wind effects of DAD methodology based on the pressure time histories from Louisiana State University (LSU) aerodynamic database, simply called DAD approach in some cases later on.

3. The critical demands such as the displacement /uplift force at connections and roof for the light-frame wood structure are considered as code performance indicators, rather than the bending moments for industrial buildings.

4. Some situations are identified that each of the ASCE 7 procedures yielding unconservative wind loads on a typical low-rise residential building.

6.2 LSU Aerodynamic Database

6.2.1 Building model

A 1:50 scale building model is selected from the LSU database with the prototype being a one-story 5:12 pitched gable roof residential house with timber construction and a rectangular plan of 18.3 m by 13.4 m (60 ft by 44 ft) and overhang height of 3.0 m (9.8 ft). This configuration is designed in accordance with the South/Key CBG type that is defined in the Florida Public Hurricane Loss Model (FPHLM) and intends to be the representative of the United States residential buildings. This typical building model mainly consists of four parts: lumber frames, roof and wall sheathings, connections between sheathing and frame, frame and frame, and foundation hold-downs as they act as the critical load bearing components. Especially, unlike the past models used to evaluate the ASCE 7 standard, openings along with the induced internal pressures measured from wind tunnel tests are incorporated for the comparison of the wind effect between the ASCE 7 standard estimate and the time-history wind loading design. There are 17 openings in total, i.e., windows and doors, distributed on the walls to capture the behavior of the building subjected to internal fluctuations that lead to the over-pressurization along with the failure of the structure. More information pertaining to the opening layout and the geometric configuration such as the size of beams and sheathings and the arrangement of trusses of the building model can be obtained from Pan et al. [23]. This scaled model is mounted with 192 pressure taps (188 external taps and 3 internal taps) and connected to Scanivalve DSA3217/16Px (Serial#2100), a pressure acquisition system at a sampling rate of 500 Hz for 1 hour in full scale as shown in Fig. 6.2(a).

In the current study, a nonlinear numerical model is developed by using a modeling methodology for a light-frame wood structure that is validated by He et al. [27]. In this model, sheathing panels are represented by shell elements which are 8-noded quadrilateral with six DOF at each node to involve system effects and are built to the realistic arrangement, i.e., roof panels are staggered, and the discontinuity between the panels are taken into consideration. Frame members are modeled by 3D linear isotropic beam elements that are 2-noded with six DOF on each node, of which the trusses are assumed to be pinned, and the rest are considered rigid connected. The arrangement of these frame members is illustrated in Fig. 6.1(b). Both the sheathing and beam members are assumed to have elastic material properties. For the most critical member that usually initiates the failure of a low-rise building under wind events, the nonlinear behavior of sheathing nails is considered in this model and used as “code performance indicators”
to determine how applicable of the ASCE 7 pressure coefficients to this kind of building. Each sheathing nail on the wall is modeled by three nonlinear spring elements with the force-displacement relationship in each direction to reflect the translational capacities. The rotational capacities are considered for STTCs and each of these nails is represented by six nonlinear spring elements. In this study, the roof and the sole plate are rigidly connected to the wall and the foundation, respectively. As such, continuous load paths are formed to transfer all the wind loads from the roof and wall to the foundation. More modeling details related to the element selection and connection refer to He et al. [27]. Totally, the model is created by 12,901 beam elements, 39,505 shell elements, and 50,664 nonlinear spring elements.

### 6.2.2 Wind loading

The Boundary Layer Wind Tunnel at LSU is an open return wind tunnel with a test section of 2.44 m (8 ft) in length, 1.32 m (4.3 ft) in width, and 0.99 m (3.2 ft) in height and it is powered by a 2.4 m (7.9 ft) diameter fan that is capable of producing a free stream velocity of up to 12 m/s [26]. An open terrain atmospheric boundary layer with a roughness length $z_0$ of 0.0142 m is simulated by setting roughness elements such as carpet on the floor, spires at the entrance, and saw tooth trip in the downstream from spires. The external pressure datasets for the LSU aerodynamic database are collected under three angles, namely $0^\circ$, $45^\circ$, and $90^\circ$ which roughly covers the entire angle range due to the symmetric building geometry. For the internal pressure measurements, the volume scaling is considered by adding an internal volume chamber to the building model. These internal pressure datasets are tested under eight wind angles over a $360^\circ$ range at $45^\circ$ increments with different opening cases, some of which have been published by Pan et al. [23].

For the comparison of wind effects with the ASCE 7 provisions, four loading cases based on the DAD methodology were carried out in the current study that employs the pressure datasets from the LSU aerodynamic database, as illustrated in Table 6.1. All these cases apply the external dynamic pressures from LSU database with a duration of 2s. The internal pressure datasets from LSU are used for the first case subjected to a wind angle of $90^\circ$ to evaluate the internal pressure effects as opposed to the previous ASCE 7 evaluation research with the DAD method that focused only on comparing the external pressure effects and used the standard defined internal pressures such as Coffman et al. [16]. The ASCE 7 defined internal pressure coefficients are used for the
rest cases covering all three angle cases of external pressure in the LSU database for consistency. Finally, the DAD responses used for the comparisons with the ASCE 7 are the peak values, positive and negative, of all the four DAD cases. The comparisons among these four DAD cases are also discussed and detailed later to investigate the wind directional effects and code-based internal pressure coefficients effects.

Table 6.1. DAD loading cases

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Wind Angle</th>
<th>External Pressure</th>
<th>Internal Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>DAD1</td>
<td>90°</td>
<td>LSU</td>
<td>LSU</td>
</tr>
<tr>
<td>DAD2</td>
<td>0°</td>
<td>LSU</td>
<td>ASCE 7-10</td>
</tr>
<tr>
<td>DAD3</td>
<td>45°</td>
<td>LSU</td>
<td>ASCE 7-10</td>
</tr>
<tr>
<td>DAD4</td>
<td>90°</td>
<td>LSU</td>
<td>ASCE 7-10</td>
</tr>
</tbody>
</table>

In order to compare with the wind effect of the ASCE 7-10 provisions, the pressure coefficient measures from the LSU wind tunnel referenced to the mean roof height pressure are re-normalized to the storm condition specified in the ASCE 7-10 of a 3-s gust wind speed at 33 ft (10 m) in an open terrain (roughness length $z_0$=0.03 m, Table C26.7-2, ASCE 7-10).

$$C_{p,3s,10m,z_0=0.03m} = C_{p,ref} \times \left( \frac{q_{ref}}{q_{3s,10m,z_0=0.03m}} \right) = C_{p,ref} \times \left( \frac{V_{ref}}{V_{3s,10m,z_0=0.03m}} \right)^2 \quad (1)$$

where

$$C_{p,ref} = \frac{p}{q_{ref}} = \frac{p_i-p_0}{q_{ref}} \quad (2)$$

$$\left( \frac{V_{ref}}{V_{3s,10m,z_0=0.03m}} \right)^2 = \left( \frac{V_{1h,10m,z_0=0.0142m}}{V_{3s,10m,z_0=0.0142m}} \right)^2 \times \left( \frac{V_{3s,10m,z_0=0.0142m}}{V_{3s,10m,z_0=0.03m}} \right)^2 \quad (3)$$

In Eq. (1), $C_{p,3s,10m,z_0=0.03m}$ is the normalized wind pressure coefficient; $C_{p,ref}$ is the pressure coefficient at the reference height, testing terrain, and testing wind speed; $q_{ref}$ is the dynamic pressure at the upper level reference height in the wind tunnel measured by the pitot tube; and $q_{3s,10m,z_0=0.03m}$ is the dynamic pressure at the storm condition (i.e., 3-s gust wind, 10 m reference height and terrain roughness length $z_0$ of 0.03 m) that is consistent with that defined in the ASCE 7-10. In Eq. (2), $p$ represents the net tap pressure and is expressed by the difference between the model surface pressure measured by the pressure taps $p_i$ and the reference level static pressure $p_0$ simultaneously derived from the pitot tube. The first term in Eq. (3) represents the adjustment for height and is obtained from the velocity profile measured by Pan et al. [23]; the second term adjusts for the average time taken from the “Durst Curve” in Fig. C26.5-1, ASCE 7-10; and the last ratio adjusting for terrain is obtained from the Engineering Science Data Unit [24] model.

The pressure coefficients measured on the pressure taps are then discretized to be applied to their tributary area on the refined finite-element (FE) model as illustrated in Fig. 6.3, and the applied wind pressures is calculated by Eq. (4) for a 44.7 m/s -64.8 m/s (115 mph-145 mph) basic wind speed used in the ASCE 7 that belongs to Category (Cat.) 2-4 in the Saffir-Simpson hurricane
wind scale (SSHWS). Pressures in the form of time histories from the LSU database are used here as a DAD method to evaluate the ASCE 7 provision.

\[
P = \frac{1}{2} \rho V^2_{35,10m,z_0=0.03m} \left[ C_{pe,3s,10m,z_0=0.03m} - C_{pl,3s,10m,z_0=0.03m} \right]
\]  

(4)

Fig. 6.3. LSU FE model: (a) complete elements; (b) demonstration of loading discretization under wind direction of 45°

6.3 Comparison with ASCE 7-10 Provisions

The critical demands corresponding to the ASCE 7-10 are calculated following both the analytical methods for the MWFRS (both envelope and directional procedure) and C&C since the critical members of a low-rise building subjected to wind loads can belong to either of the two systems based on the definitions given in the standard. The ASCE 7-10 states that components can be part of the MWFRS when they act as shear walls or roof diaphragms that work together to transfer wind loads acting on the entire structure to the ground. For the roof truss system, the long-span trusses should be designed based on MWFRS method, and the individual members of trusses should be designed for loads associated with the C&C method [28]. Morrison [29] suggested the toe-nail represented the RTWCs in his building model should be treated in both ways of the MWFRS and C&C as they are unclear which category should they belong to according to the definition given by the ASCE 7-10. Mensah et al. [30] and Roueche et al. [31] also applied both the MWFRS and C&C methods to an entire building for the design of the RTWCs and wall-to-foundation connections (WTFCs) to provide a more direct comparison of the wind effects between the ASCE 7-10 and DAD. It is noteworthy that the loads specified in the C&C method are not intended for the use when considering the interaction of loads from multiple surfaces and typically should not be used for an entire building. As such, one can expect the larger structural response on the component or cladding when calculated following the C&C procedure. However, in the present analysis, both procedures, i.e., ASCE7-10 sect.27.4.1 and ASCE7-10 sect. 28.4.1 on the one hand and ASCE7-10 sect.30.4.1 on the other are applied, and the results of them would provide a range in which ASCE7-10 varies and a more direct comparison to quantify the differences.

According to the ASCE 7, the current building model is regarded as enclosed by its opening arrangement, and the internal pressure coefficient, \((GC_{pl})\), is \(\pm 0.18\) for this enclosure classification. The basic parameters such as wind directionality factor \((K_d)\) and the topographic factor \((K_h)\) are
0.85 and 1.0, respectively. All the possible loading scenarios of the three procedures in ASCE 7, namely the Directional Procedure, the Envelope Procedure, and the C&C method are listed in Table 6.2 considering the symmetry of the building.

<table>
<thead>
<tr>
<th>Combination</th>
<th>Wind Direction</th>
<th>Internal Pressure</th>
<th>Condition</th>
</tr>
</thead>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D1</td>
<td>N</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td>D2</td>
<td>N</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>D3</td>
<td>N</td>
<td>+</td>
<td>1</td>
</tr>
<tr>
<td>D4</td>
<td>N</td>
<td>+</td>
<td>2</td>
</tr>
<tr>
<td>D5</td>
<td>P</td>
<td>-</td>
<td>N/A</td>
</tr>
<tr>
<td>D6</td>
<td>P</td>
<td>+</td>
<td></td>
</tr>
<tr>
<td>Envelope</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>E1</td>
<td>A</td>
<td>-</td>
<td>N/A</td>
</tr>
<tr>
<td>E2</td>
<td>A</td>
<td>+</td>
<td></td>
</tr>
<tr>
<td>E3</td>
<td>B</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>E4</td>
<td>B</td>
<td>+</td>
<td></td>
</tr>
<tr>
<td>C&amp;C</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>N</td>
<td>-</td>
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</tr>
<tr>
<td>C6</td>
<td>P</td>
<td>+</td>
<td></td>
</tr>
</tbody>
</table>

\(a\) \(N = \) normal to roof ridge and \(P = \) parallel to roof ridge for Directional Procedure (Fig. 27.4-1 in ASCE 7-10); A and B for Envelope Procedure refer to Load Case A (45\(^\circ\)-90\(^\circ\)) and Load Case B (0\(^\circ\)-45\(^\circ\)) (Fig. 28.4-1 in ASCE 7-10); P, O, and N denotes parallel, oblique, and normal to roof ridge for C&C procedure.

\(b\) Internal pressure corresponds to the enclosed enclosure classification with pressure coefficient \((GC_{pi}) = \pm 0.18\), where the plus and minus signs signify pressures acting toward and away from the surfaces, respectively.

\(c\) Conditions 1 and 2 refer to the two values of external pressure coefficient for the windward roof in ASCE 7-10 (Fig. 27.4-1).

### 6.4 Discussion

#### 6.4.1 Critical demands comparison

Table 6.3-Table 6.5 list the positive and negative peak responses obtained through various ASCE 7 procedures and wind tunnel loadings for the low-rise wood frame building stated under the hurricane of Cat. 2-4, i.e., 115 mph, 130 mph, and 145 mph ASCE 7 basic wind speed. The responses include the peak sheathing displacement, a representative indicator of the failure of wood houses on the roof, and the peak uplift forces at all the critical connections including the
STTCs, RTWCs, and WTFCs. The positive and negative peak values of the sheathing displacement represent the displacement perpendicular to the roof surface out of and into the building, respectively. The positive uplift force at all the connections represents the vertical forces to stretch the connections that would cause failure. The negative value means the forces to push the sheathing to the truss, the roof assemblies to the wall, and the entire building to the foundation for the STTCs, RTWCs, and WTFCs, respectively. Thus, the negative value is not our primary concern. These selected structural responses are taken as the code performance indicators to explore how applicable of the ASCE 7 procedures to the light-frame wood house, a different structural system from industrial buildings. The peak values of these code indicators calculated with the DAD methodology are compared to the predictions based on all the procedures provided in the ASCE 7. The critical demand for metal frame buildings such as peak bending moment at the knee considered in the ASCE 7 codification is also incorporated here to compare the responses based on the ASCE 7 loads to the DAD predictions on the same wood structure representative building model. Due to the structural difference, the wall stud bending moment at the eave height in the current model with the corresponding location to the knee is used to represent the knee bending moment for the purpose of discussions.

As it is shown in these tables, the design based on the ASCE 7 wind loads are not always conservative based on the positive results under the wind speeds discussed. The maximum uplift forces on the RTWCs based on the Envelope procedure and C&C method are larger than the DAD induced values, the difference being less than 13% and 16%. For the uplift force on the WTFCs, the maximum result of the C&C method is larger than the DAD prediction by 22%. However, for all the other code performance indicators, the DAD maximum reactions are higher including the uplift force on the STTCs and the displacement on the sheathing, which are more influenced by the local pressures. Of the three critical connections, the discrepancy in the STTCs between the DAD and ASCE 7 maximum predictions of all cases (i.e., D, E, and C&C) varies from 1.1 to 2.8 that are larger than the differences in the RTWCs and WTFCs, with the factors both being 0.8-1.6. Since the DAD prediction employs the actual measured wind loading, this large discrepancy on the responses of the STTCs between the ASCE 7 design and the DAD predictions indicates the insufficiency of the ASCE 7 design, especially on the roof envelope. Such design procedure makes the roof sheathing nail the most vulnerable component for the winds encountered. This vulnerability is consistent with most common damage being the roof coverings blown off reported by the past reconnaissance such as the Mitigation Assessment Team deployed by the Federal Emergency Management Agency’s (FEMA’s) Mitigation Division (e.g., [18]). In looking at Table 6.3-Table 6.5, for a wind direction that results in the peak uplift force at the STTCs, all peak responses were observed in the range of 45°~90°, i.e., 90° for the directional procedure, Load Case A (45°-90°) for the envelope procedure, and 45° for DAD method. This emphasizes the importance of the wind directional effects on the roof where the pressure distribution is greatly determined by the separation of flow at the windward edges and the secondary flow separation at the ridge in accordance with the wind direction.

For the negative peak values, ASCE 7 predictions are larger than the DAD results and are conservative for most critical demands only except the STTC uplift force. However, in every case, these negative peak values are of considerably less magnitude than their positive counterparts. Therefore, the efficient capacities and schedules of sheathing nails and frame connections that are
sufficient for the positive peak responses should also be sufficient for these smaller negative peak values.

For the bending moment that is typically considered in the design of steel portal structures, the maximum absolute value is from D2 for the Directional Procedure, from E4 for the Envelope Procedure, and from C2 for C&C method in the current discussed wood frame model. These critical cases for the peak bending moments are not necessarily the same for the other structural responses that correspond to the critical demands for wood structures under wind loads. For example, the STTC experience the positive peak value based on E2 for the Envelope Procedure, instead of E4. Therefore, the comparison and evaluation of the ASCE 7 procedures based on the metal building critical demands are not applicable to wood structures. For the critical wind direction as shown later in Fig. 8 from the DAD prediction, the largest predictions at all the key members for low-rise wood frame buildings are obtained at the wind direction of 45°, while the prediction at the knee bending moment that represents the critical demand for the metal buildings reaches its highest value under wind direction of 90°. This difference in the most disadvantaged incident wind angle also illustrates the inadequacy of the ASCE 7 to the light-frame wood buildings to some extent.

The peak values (positive and negative) of all the cases for each ASCE 7 procedure are compared with the corresponding DAD results in Fig. 6.4 and Fig. 6.5. Generally speaking, the absolute maximum responses based on Envelope Procedure are larger than the Directional Procedure results and closer to the results based on the DAD methodology. The Envelope Procedure developed by enveloping the critical demands such as the moments is also better at predicting the critical structural actions such as the uplift force at connections with closer results to the DAD predictions than the Directional Procedure for the light frame wood buildings. For the critical structural actions mainly subjected to local pressures such as the peak uplift force on the STTCs, the two MWFRS procedures predict similar values. The results based on the C&C method is larger than the MWFRS methods at all the critical demands as expected. However, even though conceptually using the C&C method should have overestimated the system effect of the structure, the responses such as the STTC uplift force induced by this method is still smaller than the DAD results indicating the non-conservatism aspect of the design provision.

![Graph showing uplift forces for STTC and RTWC](image-url)
Fig. 6.4. Critical demands for residential structures based on ASCE 7 and DAD procedures at sheathing panels, STTCs, RTWCs, and WTFCs (115mph)
Table 6.3. Peak response in critical members discussed under Cat. 2 SSHWS: 115 mph ASCE 7 wind speed

<table>
<thead>
<tr>
<th>Case</th>
<th>Peak Type</th>
<th>Sheathing Displacement (m)</th>
<th>STTC Uplift (N)</th>
<th>RTWC Uplift (N)</th>
<th>WFTC Uplift (N)</th>
<th>Knee Bending Moment (N·m)</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>DAD</td>
<td>ASCE</td>
<td>Ratio</td>
<td>DAD</td>
<td>ASCE</td>
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<td>D1</td>
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<tr>
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<td>3.4E-02</td>
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<td>N/A</td>
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<tr>
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<td>2.9E-02</td>
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Note: The highlighted values are the positive and negative peak values for each ASCE 7 procedure, i.e., D, E, and C. Ratio= DAD/ASCE. The Ratios less than 1 are highlighted by squares and represent the ASCE 7 design is conservative.
Table 6.4. Peak response in critical members discussed under Cat. 2 SSHWS: 130 mph ASCE 7 wind speed

<table>
<thead>
<tr>
<th>Case</th>
<th>Peak Type</th>
<th>Sheathing Displacement (m)</th>
<th>STTC Uplift (N)</th>
<th>RTWC Uplift (N)</th>
<th>WFTC Uplift (N)</th>
<th>Knee Bending Moment (N·m)</th>
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<td>ASCE</td>
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<td>ASCE</td>
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<td>5.1E-02</td>
<td>1.1E-02</td>
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<td>1330.8</td>
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<td>-</td>
<td>1.4E-02</td>
<td>3.0E-01</td>
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<td>D2</td>
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<td>-</td>
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<td>N/A</td>
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<td>547.8</td>
<td>2.4</td>
<td>3777.2</td>
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<td>-</td>
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<td>-267.3</td>
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<td>2.9E-03</td>
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<td>209.0</td>
<td>6.4</td>
<td>6860.0</td>
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<td>-</td>
<td>1.6E-03</td>
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<td>+</td>
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<td>231.2</td>
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<td>1566.6</td>
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<td></td>
<td>-</td>
<td>2.6E-03</td>
<td>5.3</td>
<td>-199.8</td>
<td>4.1</td>
<td>N/A</td>
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</table>

| Mean | +         | 6.2 | 4.1 | 3.6 | 2.2 | 3.6 |
|      | -         | 4.5 | 3.6 | N/A | 1.3 | 2.5 |
| E1   | +         | 1.4E-02 | 3.7 | 369.2 | 3.6 | 3186.5 | 1.8 | 4730.0 | 2.3 | 175.6 |
|      | -         | 4.5E-04 | 20.8 | -179.9 | 4.2 | N/A  | N/A  | -298.7 | 1.9 | 304.2 |
| E2   | +         | 2.3E-02 | 0.6 | 647.1 | 2.1 | 5347.0 | 1.1 | 8335.4 | 1.3 | 308.5 |
|      | -         | 4.5E-04 | 30.0 | -318.3 | 2.4 | N/A  | N/A  | -502.0 | 1.1 | 308.5 |
| E3   | +         | 2.2E-02 | 2.3 | 327.7 | 4.1 | 4534.9 | 1.3 | 7435.2 | 1.5 | 191.6 |
|      | -         | 3.2E-03 | 4.3 | -403.3 | 1.9 | N/A  | N/A  | -3233.2 | 1.4 | -493.1 |
| E4   | +         | 3.1E-02 | 1.6 | 605.6 | 2.2 | 6724.3 | 0.9 | 10612.1 | 1.0 | 319.5 |
|      | -         | 3.2E-03 | 4.3 | -471.6 | 1.6 | N/A  | N/A  | -3024.8 | 1.5 | -652.4 |

| Mean | +         | 1.8 | 3.0 | 1.3 | 1.6 | 3.2 |
|      | -         | 17.4 | 2.6 | N/A | 1.4 | 1.3 |
| C1   | +         | 3.6E-02 | 1.4 | 906.4 | 1.5 | 5254.7 | 1.1 | 10715.9 | 1.0 | 427.4 |
|      | -         | N/A  | N/A  | -542.5 | 1.4 | N/A  | N/A  | -547.6 | 1.0 | 427.4 |
| C2   | +         | 4.3E-02 | 1.2 | 1131.1 | 1.1 | 6941.0 | 0.8 | 14290.4 | 0.8 | 541.8 |
|      | -         | 8.0E-05 | 172.4 | -625.6 | 1.2 | N/A  | N/A  | -707.6 | 0.8 |
| C3   | +         | 3.7E-02 | 1.4 | 812.4 | 1.6 | 4853.9 | 1.2 | 11067.3 | 1.0 | 366.9 |
|      | -         | 1.3E-02 | 1.1 | -536.9 | 1.4 | N/A  | N/A  | -6475.1 | 0.7 | -555.6 |
| C4   | +         | 4.4E-02 | 1.2 | 1091.1 | 1.2 | 6017.3 | 1.0 | 14070.8 | 0.8 | 481.2 |
|      | -         | 4.9E-02 | 2.8 | -608.7 | 1.3 | N/A  | N/A  | -5735.5 | 0.8 | -508.2 |
| C5   | +         | 3.7E-02 | 1.4 | 590.9 | 2.3 | 4078.8 | 1.4 | 10424.7 | 1.1 | 356.8 |
|      | -         | 1.4E-02 | 1.0 | -607.0 | 1.3 | N/A  | N/A  | -8177.2 | 0.5 | -342.4 |
| C6   | +         | 4.3E-02 | 1.2 | 857.6 | 1.6 | 5780.5 | 1.0 | 11527.5 | 1.0 | 447.7 |
|      | -         | 8.4E-03 | 1.5 | -682.8 | 1.1 | N/A  | N/A  | -8243.0 | 0.5 | -298.6 |

| Mean | +         | 1.3 | 1.5 | 1.1 | 0.9 | 1.7 |
|      | -         | 28.8 | 1.3 | N/A  | 0.6 | 1.3 |

Note: The bold values are the positive and negative peak values for each ASCE 7 procedure, i.e., D, E, and C. Ratio= DAD/ASCE. The Ratios less than 1 are highlighted by squares and represent the ASCE 7 design is conservative.
Table 6.5. Peak response in critical members discussed under Cat. 3 SSHWS: 145 mph ASCE 7 wind speed

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<th>Case</th>
<th>Peak type</th>
<th>Sheathing displacement (m)</th>
<th>STTC uplift (N)</th>
<th>RTWC uplift (N)</th>
<th>WFTC Uplift (N)</th>
<th>Knee Bending Moment (N-m)</th>
</tr>
</thead>
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<tr>
<td>D1</td>
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<td>6.8E-02</td>
<td>1.4E-02</td>
<td>4.7</td>
<td>1055.8</td>
<td>346.5 4.8</td>
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<td>-</td>
<td>-1.7E-02</td>
<td>-3.8E-03</td>
<td>-1.38</td>
<td>-967.4</td>
<td>-200.8 4.8</td>
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<td>+</td>
<td>4.5</td>
<td>4.5</td>
<td>4.5</td>
<td>582.1</td>
<td>2.8 1617.5 4.5</td>
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<td>-</td>
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<td>N/A -4416.6 -472.1 0.9</td>
</tr>
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<td>N/A -8749.5 1.6</td>
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</table>

Mean + 6.2 4.1 3.8 2.2 6.6 3.6 1.0 2.5
Mean - 4.5 3.6 N/A 1.0

E1   | +         | 1.7E-02                    | 3.7             | 3.6             | 459.4           | 3.6 390.3 1.8       |
|      | -         | -5.6E-04                   | -350.3          | 4.3             | -233.8          | N/A N/A             |
| E2   | +         | 2.8E-02                    | 2.2             | 2.2             | 906.9           | 2.1 6652.1 1.1      |
|      | -         | -5.5E-04                   | -350.3          | 4.3             | -233.8          | N/A N/A             |
| E3   | +         | 2.7E-02                    | 2.3             | 2.3             | 407.7           | 4.1 5652.9 1.3      |
|      | -         | -4.0E-04                   | -501.8          | 1.9             | N/A             | N/A -4022.4 1.1     |
| E4   | +         | 3.9E-02                    | 1.6             | 1.6             | 753.4           | 2.2 8365.6 0.9      |
|      | -         | -4.0E-03                   | -501.8          | 1.9             | N/A             | N/A -4022.4 1.1     |

Mean + 2.5 3.0 1.3 1.6 2.5 2.6 1.1 1.3
Mean - 17.4 2.6 N/A 1.1

C1   | +         | 4.5E-02                    | 1.4             | 1.4             | 1127.3          | 1.5 661.9 1.1       |
|      | -         | -8.9E-05                   | -778.6          | 1.2             | N/A             | N/A -880.3 0.8      |
| C2   | +         | 5.3E-02                    | 1.2             | 1.2             | 1474.8          | 1.1 8635.7 0.8      |
|      | -         | -9.9E-05                   | -778.6          | 1.2             | N/A             | N/A -880.3 0.8      |
| C3   | +         | 4.6E-02                    | 1.4             | 1.4             | 1010.9          | 1.6 6031.9 1.2      |
|      | -         | -1.6E-02                   | -888.0          | 1.4             | N/A             | N/A -803.7 0.5      |
| C4   | +         | 5.5E-02                    | 1.2             | 1.2             | 1357.8          | 1.2 7468.9 1.0      |
|      | -         | -6.1E-02                   | -755.7          | 1.3             | N/A             | N/A -7153.5 0.6     |
| C5   | +         | 4.5E-02                    | 1.4             | 1.4             | 735.2           | 2.3 5068.3 1.4      |
|      | -         | -1.7E-02                   | -755.4          | 1.3             | N/A             | N/A -10485.6 0.4    |
| C6   | +         | 5.4E-02                    | 1.2             | 1.2             | 1066.5          | 1.6 7121.4 1.0      |
|      | -         | -1.0E-02                   | -849.2          | 1.1             | N/A             | N/A -10373.5 0.4    |

Mean + 1.3 1.5 1.1 0.9 29.8 1.3 N/A 0.5
Mean - 29.8 1.3 N/A 0.5

Note: The highlighted values are the positive and negative peak values for each ASCE 7 procedure, i.e., D, E, and C. Ratio= DAD/ASCE. The Ratios less than 1 are highlighted by squares and represent the ASCE 7 design is conservative.
For an industrial building, the bending moment at the knee where the roof is connected to the wall is expected to be similar between the two MWFRS procedures design values and DAD predictions since this demand is considered in developing the Envelope Procedure. It is interesting to see that the bending moments calculated from the Envelope Procedure on the current wood house is also similar to that based on the Directional Procedure, which is similar to the relationship in metal houses. However, the peak bending moments at the knee based on the DAD methodology are larger by 134%-146% than their counterparts based on the MWFRS procedures. As opposed to the metal frame structure where the frames are continuous from the truss of the roof to the column of the wall, the truss system in the wood structure is connected to the horizontal top plate of the wall via RTWCs. As such, even for the bending moment considered in the original development of the Envelope Procedure, significant change will be expected due to the different building configurations and this demand may be no longer critical for the residential houses.

Fig. 6.5. Critical demands for knee bending moment base on ASCE 7 and DAD procedures (115mph)

6.4.2 Critical location/ Critical member comparison

Fig. 6.6 and Fig. 6.7 demonstrate the locations of the critical members that are determined by the load distribution for all the peak actions discussed here. For the sheathing displacement as shown in Fig. 6.6(a), the DP makes a same prediction on the critical member as the DAD (LSU) prediction with the location by the middle of the ridge on the leeward roof. The C&C method prediction is around the similar area, but on the opposite side of the roof; the EP predicts the largest sheathing displacement happening on the same side as the DAD prediction while on the head of the ridge. Fig. 6.6(b) shows that for the sheathing nail experiencing the largest uplift force, all the three procedures of the ASCE 7 point to the same critical member that is on the leading edge near the ridge. In comparison, the LSU critical point is on the other end edge that is symmetric about the center line of the building in the gable wall direction.

In Fig. 6.6(c), the critical RTWCs under the ASCE7 procedures are on the same truss but different ends: the members based on the EP and C&C method are on the wind ward end and the DP prediction is on the other end. The DAD prediction is on the leeward side but closer to the gable end, which makes the DP a closer prediction on the same side of the roof. Being the last
structural member along the load path, the critical WTFC predicted by all the methods are on the wall studs by the large openings as shown in Fig. 6.6(d), i.e., front and back door, which is partly accounted for by the fact that there are WTFCs right under every stud in this current model. Overall, the Directional Procedure is found to yield closer predictions on the critical members to the DAD predictions, indicating the load distribution defined in the Directional Procedure is closer to the actual one.

Fig. 6.6. Locations of the critical demands for residential structures (115mph): (a) sheathing panels; (b) STTCs; (c) RTWCs; and (d) WTFCs. (DP: Directional Procedure; EP: Envelope Procedure; C&C: C&C method; LSU: DAD method based on the LSU database)

Fig. 6.7 shows the location of the critical member for the bending moment of low-rise buildings. In this case, the discrepancy between the ASCE 7 predictions and the DAD prediction is of the largest level among the five demands discussed based on the location of the critical member. This discrepancy indicates the significant influence of the building configuration on the structural behaviors and thus, the current building codes developed based on the metal building is completely not suitable for the wood building.
Fig. 6.7. Locations of the critical demands for industrial structures: knees of the frame

6.4.3 Wind directional effects and code-based internal pressure coefficients

The structural responses under the loading cases predicted by the DAD are shown in Fig. 6.8, where the DAD loading cases have been detailed in Table 6.1.

The DAD1 predictions using wind tunnel measured internal pressures under 90° winds are plotted in the same figure as the rest DAD predictions that employ the ASCE 7 internal pressure values to illustrate the discrepancy induced by using different internal pressure coefficients. This item also serves as a reference to deduce the peak realistic structural responses under other wind directions. The peak responses predicted by DAD1 loading cases have increased, compared with their counterparts based on DAD4, by 120%, 122%, 11%, and 91% for the STTC uplift force, RTWC uplift force, WTFC uplift force, and sheathing displacement, respectively. These larger discrepancies are induced by the larger internal pressures measured. This indicates that the underprediction of the ASCE 7 illustrated earlier can be even more significant based on realist internal wind loads.
Fig. 6.8. Critical demands predicted by DAD loading cases (115 mph)

Among the loading cases using the ASCE 7 internal pressures, i.e., DAD2-DAD4 as shown in Fig. 6.8(a)-(d), the peak values of all the demands for residential houses considered here are obtained under oblique incident winds, i.e., 45°. For the bending moment at the knee, the peak value is led by the wind normal to the ridge, i.e., 90°. As such, in terms of the critical wind direction, the ASCE 7 provision is not applicable to the residential house design.

6.5 Conclusions

This study evaluated the adequacy of wind design by using the ASCE 7-10 wind provisions on residential buildings. A review of the methodologies behind the ASCE 7-10 procedures showed a great gap between the industrial building type which the code is developed upon and the residential constructions with different configurations, namely material property and inter-connections. Based on the discussions of the present study, the following can be concluded and recommended.

- The adequacy of the ASCE 7 methods, including the Directional Procedure and Envelope Procedure for MWFRS and the C&C method, to the design of low-rise wood buildings in
terms of matching critical structural actions calculated from the actual wind tunnel loading is not a clear cut. The Directional Procedure is found to be consistently unconservative; the Envelope Procedure may over predict or predict close critical actions at the RTWCs and WTFCs while not sufficient for the responses governed by local pressures such as the uplift force on the STTCs. The C&C method that has incorporated the system effects is conservative for most actions as expected except the STTC uplift indicating the even larger underestimations of the ASCE 7 on these demands. As stated above, the design of ASCE 7 is unconservative especially on the vulnerable STTCs. Therefore, for the structural actions determined more by local wind pressures, the design that follows the ASCE 7 provisions has a great chance of being unconservative.

- Between the two procedures for the MWFRS, the Directional Procedure represents envelopes of the real wind loads acting on the building as opposed to the Envelope Procedure that uses the fictitious load fitted from the values of postulated critical reactions. Therefore, the wind effect followed by the Directional Procedure will not be influenced by the load paths and sharing of a specific configuration, and less difference is expected from that of DAD. However, the Envelope Procedure is found to result in higher response and be generally better at predicting the critical demands for wood houses compared with the DAD approach than the Directional Procedure in the sense of magnitude. Regarding the location of the critical member, the Directional Procedure does a better job.

- For the loading scenarios considered, the critical wind angle that may trigger the first failure of wood framed buildings on the vulnerable members including the sheathing panels, the STTCs, the RTWCs and the WTFCs is found to be oblique, i.e., 45º. However, for the bending moment that is considered in the development of ASCE 7, the critical wind angle is 90º. This illustrates the inapplicability of the ASCE 7 to the residential building to some extent.

- To allow the ASCE 7 provisions to be confidently used on wood residential houses, research should be undertaken for a more comprehensive set of comparisons in the future for buildings with various geometries and wider potential critical actions. The methodology to envelop the critical reactions is superior in targeting directly at the structural response, but it is limited in exhausting all the constantly evolving structural configurations. A new methodology is expected to revise the ASCE 7 wind loads code that can inherit the merits of both the current procedures to eliminate the underestimation of wind effects and reflect the critical member under wind loads.

### 6.6 References


146


CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary and Conclusion

A comprehensive review of the progress and state-of-the-art research on the theme of low-rise building performance under wind loads identified the need to combine the strengths of each involved disciplines, e.g., the dynamic form of wind loads and the full-scale scope of modeling. Therefore, in this dissertation, a numerical modeling framework that fully utilizes the currently available testing data, advanced modeling techniques, and up-to-date engineering analysis theory was developed to predict the performance of low-rise buildings under extreme wind events. This framework provides a guideline on the three crucial steps for the accurate performance prediction: directly using the aerodynamic database derived from wind tests, applying the loads onto a refined building model that is capable of reflecting the nonlinear behaviors of critical components, and conducting the analysis on the progressive failure process.

A refined 3D modeling methodology applicable to nonlinear range was proposed and validated by a large-scale building model experiment conducted under the Wall of Wind (WOW) Experimental Facility (EF) at Florida International University. The comprehensive numerical model developed by this methodology can accommodate various materials and structural connections with mechanics-based load-deformation characteristics such as sheathing nails and framing-to-framing connections, so as to be capable of predicting the performance of the components and connections that are difficult to model in general but are the most vulnerable parts of a low-rise structure as witnessed during past hurricanes. Serving for the loss prediction and vulnerability assessment, a progressive failure analysis methodology was proposed based on the current available techniques and experiment phenomena. A destructive wind test was carried out at WOW on the same building models to validate the modeling methodology and demonstrated the capability of the proposed failure analysis methodology. Since the whole proposed framework has been well entailed and verified, this promising platform could be adopted with confidence for the future building performance related research.

Some attempts on the application of this framework have been performed onto a more typical U.S. residential configuration revealing its full capability to deal with the more complex models. The storm duration effects considering the number of peak loading was investigated on the whole building failure by the failure analysis method. The building vulnerability to the wind increased by over 50% in terms of the wind speed due to the duration effects for the case studied.

Additionally, taking advantage of the framework with validated methodology, the explorations are made on the effect of modeling techniques, the building configurations, and the wind loading resolution. For example, the linear model with components either pinned or rigidly connected showed the adequacy in linear range in that it predicted well under low wind speed while would induce underestimations when subjecting to high wind speed. The modeling of the rotational capacity of the wall stud connections was found to have little effect to the behavior of critical components such as the RTWCs and the roof sheathing in terms of deflection, while that of the sheathing nail was critical to the load paths and load sharing in the structure especially under high winds and thus, cannot be ignored. The truss shape with more webs and nails at gable end could greatly increase the capacity of the sheathing and RTWCs but had little effect to the failure
of STTCs. Adding more sheathing nails, especially on the field nailing was found to be significantly effective in failure mitigation of the timber structures. On the contrary, missing nail would lead to the worst case and should be avoided in the construction. Applying the wind loads from the small-scale wind tunnel tests would induce unconservative building design.

Last but not least, the adequacy of ASCE 7 code-specified procedures for the wind design of light-frame wood buildings in terms of structural responses was evaluated. Both the Main Wind Force-Resisting System (MWFRS) including two procedures and the Components and Cladding (C&C) in the ASCE 7 were applied on to a detailed model based on the proposed nonlinear modeling methodology. After comparing the wind effects such as the displacement and the uplift force at connections and roof with that induced by the DAD methodology, the design of low-rise wood buildings based on the ASCE 7 code provisions was found questionable.

7.2 Recommendation for Future Study

The following issues are believed to deserve further research:

- Extensive mechanical experiments should be carried out to examine and fully quantify the mechanical property of structural components, especially the connections. The critical bottleneck restricting the modeling of connections in actual applications has often been the lack of a comprehensive pool of the nonlinear material properties of connections with various wood types and contact conditions. Especially, 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.

- Systematic full-scale and large-scale tests including destructive tests are needed to build a comprehensive database with uniform formats for the public domain that includes wind pressure time series along with structural response time histories, especially at different stages of failures. 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 conductions of the wind test can be avoided and the time spent on processing the data with different formats from different database could be saved.

- The failure analysis methodology proposed by the current project serves as a potential framework for the progressive failure prediction and can be improved in many ways to receive better accuracy. For example, more information such as the internal pressure and external pressure measured at the moment of the failure could be implemented into the analysis. Additionally, to approach to the real physics of the structure during failure, this methodology could be used in other software packages with more appropriate environment.

- Essential for the destructive and progressive failure simulations, internal pressures under various opening conditions on the roof should be collected and analyzed.
• Both the proposed methods are expected to be applied on various building configurations with various failure scenarios, e.g., different wind directions and different sequences of the broken opening caused by debris.

• A new methodology is expected to revise the ASCE 7 wind loads code that can inherit the pros of both the current procedures, i.e., the Main Wind Force-Resisting System (MWFRS) and the Components and Cladding (C&C), to eliminate the underestimation of wind effect and reflect the critical member performance under wind loads.

• The performance analysis framework is well developed and has made its applications to the deterministic numerical model at the complete building level at current stage. It is expected that in the next stage, the framework could extend its application to the vulnerability assessment by including the probabilistic nature of wind loading and material properties, etc.
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