PROBABLISTIC ASSESSMENT OF ROOF UPLIFT CAPACITIES IN LOW-RISE RESIDENTIAL CONSTRUCTION

Bagyalakshmi Shanmugam
Clemson University, sshanmu@clemson.edu

Follow this and additional works at: https://tigerprints.clemson.edu/all_dissertations

Part of the Civil Engineering Commons

Recommended Citation
Shanmugam, Bagyalakshmi, "PROBABLISTIC ASSESSMENT OF ROOF UPLIFT CAPACITIES IN LOW-RISE RESIDENTIAL CONSTRUCTION" (2011). All Dissertations. 741.
https://tigerprints.clemson.edu/all_dissertations/741

This Dissertation is brought to you for free and open access by the Dissertations at TigerPrints. It has been accepted for inclusion in All Dissertations by an authorized administrator of TigerPrints. For more information, please contact kokeefe@clemson.edu.
PROBABLISTIC ASSESSMENT OF ROOF UPLIFT CAPACITIES IN LOW-RISE RESIDENTIAL CONSTRUCTION

A Dissertation
Presented to
the Graduate School of
Clemson University

In Partial Fulfillment
of the Requirements for the Degree
Doctor of Philosophy
Civil Engineering

by
Bagyalakshmi Shanmugam
May 2011

Accepted by:
Dr. Bryant G. Nielson, Committee Chair
Dr. Scott D. Schiff
Dr. C. Hsein Juang
Dr. Nigel B. Kaye
ABSTRACT

Post hurricane damage investigations of light frame wood residential structures reveal that roof envelope failure induces considerable damage to the structure and its contents. Roof - to – wall (RTW) connection failures though not as common as roof sheathing failure also cause significant structural and material damage. Considerable changes have been made in the ASCE structural loads standard and International Building code (IBC) after hurricane Andrew in order to prevent RTW connection and sheathing failures. That includes not only a substantial increase in the design wind load in the past two decades but also a strict enforcement of tighter nailing schedules and stronger RTW connections (metal straps and hurricane ties). However a significant number of older buildings constructed with toenailed RTW connections exist and their safety and reliability needs to be investigated. Hence there is an apparent need to statistically understand the behavior of toenailed RTW connections in existing buildings. Fragility analysis of roofs of older buildings will provide an insight on the prevailing level of safety and help to identify the shortcomings and the associated ramifications. Experimental statistics and analytical models of the toenail behavior and sheathing fasteners will help to formulate accurate roof fragility estimations. Estimation of the effect of wind load spatial correlation on the fragility estimation and the sensitivity of fragility curves to various modeling assumptions will further enhance the credibility of roof system fragility analysis methodologies. Since hurricane ties have replaced toenailed RTW connections in modern residential construction and are used as a retrofit measure to complement existing
toenail connection capacities, understanding their behavior under high loads is essential. Experimental tests on hurricane ties subjected to uplift and combined (uplift and lateral) loads will not only provide an insight on the advantage of their usage in hurricane prone areas but also help in identifying the available design space when subjected to multi-axial loads. This information is crucial while developing statistical and analytical models for hurricane ties.

This research study evaluated the in-situ capacity of roof-to-wall connections and sheathing to rafter fasteners in light-framed wood construction. The outcome of this study was an analytical model designed to approximate the uplift behavior of toenail connections and to facilitate modeling of roof systems. In addition, the study experimentally examined three very common hurricane ties under uni-axial, bi-axial and tri-axial loads. After testing over 350 connections and performing detailed analyses, the currently used design equation for combined loads was found to be inefficient (least usable design space) and overly conservative. A new design space taking a 25% reduction on all allowable loads for hurricane ties when subjected to multi-axis load is proposed.

A finite element model of a light frame gable roof system was created using the developed analytical model of the RTW toenail connections and sheathing fasteners. Assessment of the overall impact of RTW and sheathing connector behavior on the wind-uplift fragility curves for the roof system was achieved using a Latin-hypercube based simulation strategy. It was found that the treatment of post ultimate connection behavior had a significant influence on the fragility assessment of the roof system. However
assigning variable and uniform stiffness for roof-to-wall connectors and sheathing fasteners had little to no effect on the distribution pattern of wind uplift load among connectors. Additionally, the effects of gable end supports, sheathing thickness, nailing schedule and wind pressure spatial correlation on the fragility estimation were explored. The results indicated that the fragility estimations of both roof to wall connections and sheathing panel systems are not sensitive to the spatial correlation of wind pressure for wind perpendicular to the ridge.
DEDICATION

This dissertation is dedicated to my grandmother, Sornambal Kuppusamy; my father Shanmugam Kuppusamy; my mother, Sasirekha Shanmugam; my husband, Vijai; and my brothers, Singaravelan and Magesh.
I wish to express my sincere thanks to my advisor Dr. Bryant G. Nielson, for his advice and guidance through my doctoral study at Clemson University. I would also like to acknowledge my committee members Dr. Scott D. Schiff, Dr. C. Hsein Juang and Dr. Nigel B. Kaye for their valuable input into this work.

I would like to thank Peter L. Datin, Paul Fama, Norman Moore III, Will Waterhouse, Jeremy Graham, Greg Roche and Andrew Dillenbeck for their assistance with the experimental work. Thanks are also due to Ranjith Shivarudrappa, Angelina Gleason, Masood Shirazi and Cole Edmonson for their help with the modeling efforts. I wish to express my sincere gratitude to Mr. Danny Metz and Mr. John D. Elsea for their assistance in the lab instrumentation. Special thanks are also due to Shubhada Gadkar, who has been a good friend and great support through graduate school.

I gratefully acknowledge Dr. Yue Li from Michigan Technological University and Dr. David O. Prevatt from University of Florida for their assistance in my graduate work.
## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TITLE PAGE</td>
<td>i</td>
</tr>
<tr>
<td>ABSTRACT</td>
<td>ii</td>
</tr>
<tr>
<td>DEDICATION</td>
<td>v</td>
</tr>
<tr>
<td>ACKNOWLEDGEMENTS</td>
<td>vi</td>
</tr>
</tbody>
</table>

1. INTRODUCTION ......................................................... 1  
   1.1 BACKGROUND ...................................................... 1  
   1.2 REFERENCES ...................................................... 4  

2. STATISTICAL AND ANALYTICAL MODELS FOR ROOF COMPONENTS IN EXISTING LIGHT-FRAMED WOOD STRUCTURES ... 6  
   2.1 ABSTRACT ......................................................... 6  
   2.2 INTRODUCTION ...................................................... 7  
   2.3 EXPERIMENTAL STUDY ............................................. 11  
   2.4 PLANK SHEATHING ROOF PANELS .................................. 25  
   2.5 STATISTICAL MODELS .............................................. 29  
   2.6 ANALYTICAL MODEL FOR ROOF-TO-TOP PLATE CONNECTIONS ......................................................... 33  
   2.7 CONCLUSIONS ...................................................... 37  
   2.8 ACKNOWLEDGEMENTS ................................................. 38  
   2.9 REFERENCES ...................................................... 38  

3. MULTI-AXIS TREATMENT OF TYPICAL LIGHT-FRAME WOOD ROOF-TO-WALL METAL CONNECTORS IN DESIGN .................. 42  
   3.1 ABSTRACT ......................................................... 42  
   3.2 INTRODUCTION ...................................................... 43  
   3.3 EXPERIMENTAL SETUP AND METHODOLOGY ......................... 49  
   3.4 RESULTS ........................................................... 57
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5</td>
<td>CONCLUSIONS/RECOMMENDATIONS</td>
<td>76</td>
</tr>
<tr>
<td>3.6</td>
<td>ACKNOWLEDGEMENTS</td>
<td>79</td>
</tr>
<tr>
<td>3.7</td>
<td>REFERENCES</td>
<td>79</td>
</tr>
<tr>
<td>4.</td>
<td>INFLUENCE OF TYPICAL MODELING PARAMETERS ON WIND BASED FRAGILITY ESTIMATES OF LIGHT FRAMED WOOD ROOF STRUCTURES</td>
<td>82</td>
</tr>
<tr>
<td>4.1</td>
<td>ABSTRACT</td>
<td>82</td>
</tr>
<tr>
<td>4.2</td>
<td>INTRODUCTION</td>
<td>83</td>
</tr>
<tr>
<td>4.3</td>
<td>FINITE ELEMENT MODEL</td>
<td>87</td>
</tr>
<tr>
<td>4.4</td>
<td>MODEL PARAMETER STATISTICS</td>
<td>92</td>
</tr>
<tr>
<td>4.5</td>
<td>FRAGILITY CURVE GENERATION</td>
<td>100</td>
</tr>
<tr>
<td>4.6</td>
<td>RESULTS AND DISCUSSION</td>
<td>107</td>
</tr>
<tr>
<td>4.7</td>
<td>CONCLUSION</td>
<td>125</td>
</tr>
<tr>
<td>4.8</td>
<td>REFERENCES</td>
<td>127</td>
</tr>
<tr>
<td>5.</td>
<td>EFFECT OF SPATIAL WIND LOAD CORRELATION ON THE FRAGILITY ASSESSMENT OF LIGHT FRAMED LOW RISE RESIDENTIAL ROOF SYSTEMS</td>
<td>134</td>
</tr>
<tr>
<td>5.1</td>
<td>ABSTRACT</td>
<td>134</td>
</tr>
<tr>
<td>5.2</td>
<td>INTRODUCTION</td>
<td>135</td>
</tr>
<tr>
<td>5.3</td>
<td>FINITE ELEMENT MODEL</td>
<td>139</td>
</tr>
<tr>
<td>5.4</td>
<td>WIND LOAD MODEL</td>
<td>142</td>
</tr>
<tr>
<td>5.5</td>
<td>SIMULATION PROCEDURE AND FRAGILITY CALCULATION.</td>
<td>148</td>
</tr>
<tr>
<td>5.6</td>
<td>RESULTS AND DISCUSSION</td>
<td>151</td>
</tr>
<tr>
<td>5.7</td>
<td>CONCLUSION</td>
<td>161</td>
</tr>
<tr>
<td>5.8</td>
<td>REFERENCES</td>
<td>163</td>
</tr>
<tr>
<td>6.</td>
<td>CONCLUSION AND RECOMMENDATIONS</td>
<td>167</td>
</tr>
<tr>
<td>6.1</td>
<td>CONCLUSIONS</td>
<td>167</td>
</tr>
<tr>
<td>6.2</td>
<td>RECOMMENDATION FOR FUTURE RESEARCH</td>
<td>169</td>
</tr>
<tr>
<td>7.</td>
<td>APPENDIX</td>
<td>171</td>
</tr>
</tbody>
</table>
## LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1. Comparative of uplift capacities</td>
<td>23</td>
</tr>
<tr>
<td>2.2. Proposed probability distributions for connection behavior parameters</td>
<td>34</td>
</tr>
<tr>
<td>3.1. Basic load case information</td>
<td>56</td>
</tr>
<tr>
<td>3.2. Uni-axis design load statistics</td>
<td>60</td>
</tr>
<tr>
<td>3.3. Correlation coefficients between design forces in each of the primary connector axes</td>
<td>67</td>
</tr>
<tr>
<td>3.4. Usable design space ratio for different design surfaces</td>
<td>70</td>
</tr>
<tr>
<td>3.5. Probability of actual design value based on tested specimens falling below the proposed design surfaces</td>
<td>74</td>
</tr>
<tr>
<td>3.6. Probability of actual connector strength based on tested specimens falling below the proposed design surfaces</td>
<td>75</td>
</tr>
<tr>
<td>4.1. Summary of load characteristics</td>
<td>96</td>
</tr>
<tr>
<td>4.2. Resistance of roof-to-wall connectors</td>
<td>99</td>
</tr>
<tr>
<td>4.3. Resistance statistics of sheathing fasteners</td>
<td>99</td>
</tr>
<tr>
<td>4.4. Case number assignment and explanation for various simulations runs</td>
<td>101</td>
</tr>
<tr>
<td>4.5. Comparison of Lognormal fragility parameters for a roof system to identify the sampling error</td>
<td>105</td>
</tr>
<tr>
<td>4.6. Lognormal fragility parameters for a light frame residential roof system using different cases of roof-to-wall and sheathing connection behavior</td>
<td>109</td>
</tr>
<tr>
<td>4.7. Lognormal fragility parameters for a roof system using two different types of roof-to-wall sheathing connection post ultimate behavior</td>
<td>114</td>
</tr>
</tbody>
</table>
4.8. Lognormal fragility parameters for RTWC System using two
different types of roof-to-wall connection post ultimate
behavior .....................................................................................................................115
4.9. Comparison of Lognormal fragility parameters for roof system
using different sheathing thickness ........................................................................117
4.10. Lognormal fragility parameters for a roof system considering different
additional framing Members .....................................................................................119
4.11. Roof sheathing fragilities with variable and uniform connection
parameters ................................................................................................................122
4.12. Comparison of roof sheathing fragility parameters .................................122
4.13. Values of roof sheathing fragility parameters for different nailing
schedules ....................................................................................................................124
5.1. Connection resistance statistics ........................................................................142
5.2. Summary of load statistics ................................................................................144
5.3. List of simulation cases .......................................................................................150
5.4. Lognormal fragility parameters for roof sheathing system when
subjected to the four wind load models .................................................................154
5.5. Lognormal fragility parameters for a RTWC system when subjected to
three different wind load models (Wind load models 1, 2 and 4) ......................158
5.6. Lognormal roof system fragility parameters for the four wind load models......160
LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1. Douthit Hills duplex residential structure</td>
<td>12</td>
</tr>
<tr>
<td>2.2. Typical roof framing plan of structure</td>
<td>12</td>
</tr>
<tr>
<td>2.3. Roof-to-wall connection detail</td>
<td>13</td>
</tr>
<tr>
<td>2.4. Experimental setup for uplift tests</td>
<td>15</td>
</tr>
<tr>
<td>2.5. Typical response of connections failing due to (a) Nail withdrawal (b) Combination</td>
<td>18</td>
</tr>
<tr>
<td>2.6. Failures by (a) nail withdrawal (b) wood split</td>
<td>18</td>
</tr>
<tr>
<td>2.7. Suspected failure mechanism of two and three nail connections</td>
<td>21</td>
</tr>
<tr>
<td>2.8. (a) Suction test apparatus for uplift test on roof sheathing planks (b) Typical failure of a sheathing plank unit</td>
<td>27</td>
</tr>
<tr>
<td>2.9. Comparison of theoretical and empirical CDFs for uplift capacities</td>
<td>31</td>
</tr>
<tr>
<td>2.10. Comparison of theoretical and empirical CDFs for initial stiffnesses</td>
<td>31</td>
</tr>
<tr>
<td>2.11. Comparison of theoretical and empirical CDFs for peak load Displacements</td>
<td>32</td>
</tr>
<tr>
<td>2.12. Proposed analytical model using a pinching4 material for capturing the uplift behavior of a roof-to-wall connection</td>
<td>35</td>
</tr>
<tr>
<td>2.13. Comparison of experimental and analytical connection behavior</td>
<td>36</td>
</tr>
<tr>
<td>3.1. Typical roof-to-wall connectors (a) Type 1 (b) Type 2 (c) Type 3</td>
<td>47</td>
</tr>
<tr>
<td>3.2. Tri-axial test frame</td>
<td>49</td>
</tr>
<tr>
<td>3.3. Generalized specimen schematic</td>
<td>50</td>
</tr>
<tr>
<td>3.4. Top-View of detail for load application to bottom-most plate</td>
<td>51</td>
</tr>
<tr>
<td>3.5. Specimen of Type 3 connector embedded in a triple bottom plate using Epoxy</td>
<td>53</td>
</tr>
</tbody>
</table>
3.6. Connector failure modes (a) strap tear (b) top plate split (c) nail withdrawal from top plate (d) buckling (e) nail withdrawal from rafter (f) combination ................................................................. 55

3.7. Example of Tri-Axis loading for a Type 1 Specimen
   (a) Force- Displacement (b) Force-Time ........................................ 63

3.8. Possible design spaces considered (a) Current – 1st Order (b) 2nd
    Order (c) Cuboid (d) Combination 1st Order - Cuboid ...................... 69

3.9. Experimental design values for all three connector types with
    respect to the combination design space shown in Fig. 3.8d.
    (a) Up-Out (b) Up-In (c) Out-In (d) Up-Out-In .................................. 72

4.1. Actual structure used for developing finite element model ............... 88

4.2. Rendering of the finite element model of the baseline roof system .... 89

4.3. Sheathing panel arrangement and nailing schedule for the roof.
    (The span dimension represents the length as measured along the
    incline.) ............................................................................................ 90

4.4. Force displacement behavior of (a) combin39 element that defines
    the uplift behavior of 2-16d toenails (b) combin39 element having the
    withdrawal behavior of 8d nail sheathing fasteners (c) contac12 element
    capturing the compressive behavior of RTWC and SF (d) combin39
    element that defines the shear behavior of RTWC and SF ................. 91

4.5. Components and cladding wind zones on the roof .......................... 94

4.6. One realization of the simulated wind pressures, in units of kPa on
    the roof system .............................................................................. 97

4.7. Histogram of 500 samples simulated using (a) crude Monte Carlo and
    (b) Latin hypercube sampling .......................................................... 103

4.8. Comparison of fragility curves to demonstrate the sampling error
    involved in using 500 samples .......................................................... 104

4.9. Estimated lognormal fragility curve for the 500 realizations .......... 108

4.10. Comparison of fragility curves obtained using different types of
     connection stiffness behaviors ....................................................... 110
4.11. Roof-to-wall and (b) sheathing connection uplift force-displacement behavior with no post ultimate stiffness ................................................................. 112

4.12. Roof system fragility using two different post ultimate connection behaviors for roof to wall and sheathing fasteners .................................................. 113

4.13. Roof-to-wall connection system fragilities using two different connection behaviors ............................................................................................................. 114


4.15. Fragility plots for roof system with and without gable end supports and stiffener near the rafter ends .............................................................................. 118

4.16. Fragility curves of roof sheathing using variable and same fastener Parameters.................................................................................................................. 120

4.17. Comparison of roof sheathing fragilities from two different studies and different sheathing fastener behavior ................................................................. 121

4.18. Roof sheathing fragility for different nailing schedule .......................... 124

5.1. Basic roof configuration ............................................................................. 140

5.2. Uplift force displacement behavior of (a) two 16d toenails and (b) 8d sheathing fasteners .......................................................................................................... 141

5.3. Roof plan showing wind pressure contour in kPa for
   (a) Wind load model -1 (b) Wind load model -2 (c) Wind load model -3
   (d) Wind load model -4 .................................................................................. 146

5.4. Roof plan showing (a) pressure tap locations as in the database
   (b) desired 30.5 cm x 30.5 cm (12 in x 12 in) pressure locations
   (c) correlation coefficient contour for the pressure tap marked in the figure
   (d) interpolated correlation coefficient contour for the sheathing element
   identified. .......................................................................................................... 148

5.5. Lognormal fragility curve estimated from 500 realizations ..................... 152

5.6. Roof sheathing system fragilities for four different wind load model ...... 155

5.7. Roof-to-wall connection system fragilities using three different wind load models (wind load models 1,2 and 4) ......................................................... 159
5.8. Roof system fragilities for the four wind load models considered.......................161
1. INTRODUCTION

1.1 BACKGROUND

Vulnerability of light frame roof structures in low rise buildings to extreme wind events is often a result of insufficient connection strength, for example, roof to wall connection and sheathing fasteners, in the wind uplift load path. The susceptibility of a roof component or system can be expressed using a fragility plot which conveys the probability of failure of the component or a system when subjected to a particular wind load. Toe nailed RTW connections, identified as one of the weakest links in the load transfer path may cause roof uplift failure during an extreme wind event [1]. Even though toenail connections have long been replaced by hurricane ties/metal straps as RTW connections, numerous old buildings with toenailed connections continue to be in service and their safety can be a concern. Numerous laboratory tests done in the past to formulate the withdrawal resistance statistics of toenailed connections failed to include the variability in the field construction practices and also precluded any system effect on the capacity (load sharing between neighboring connections) [2, 3]. Furthermore the estimated uplift resistance statistics were always guided by the ultimate capacities of the connections, thus overlooking any effect due to initial and post ultimate connection stiffness. The capacity-controlled resistance statistics were frequently used to analytically derive RTW component fragilities [4] and reliability indices [5]. But RTW connection system fragility has rarely been evaluated due to extensive instrumentation and setup costs. A finite element approach to modeling of the roof system would help to evaluate the system fragility if a proper wind load model, an efficient analysis
methodology, and an analytical model of the connector behavior are identified. In addition to the knowledge of toenailed connection behavior, an insight into roof to wall hurricane tie resistance capability would help to diversify the roof system fragility estimation to new and retrofitted buildings subjected to lateral and combined wind loads.

Sheathing nails fastening the roof sheathing panels to the framing members are identified as another critical link in the wind uplift resistance path. In many past studies, it has been assumed that the roof sheathing panel uplift capacity is governed by the weakest nail in the panel [6]. The above serial failure assumption was sometimes used to numerically evaluate the panel capacity and to evaluate roof sheathing panel component fragilities [6, 7]. However the effect of such an assumption on the fragility estimation was never studied. Additionally experimental sheathing panel uplift resistance statistics were obtained using uniformly applied wind uplift loads instead of spatially varying dynamic wind load [8]. The influence of the assumed wind load behavior on both the roof panel capacity and fragility estimation has rarely been investigated. An analytical model of the sheathing fastener defined by the peak withdrawal capacity, initial stiffness and post ultimate negative stiffness will help to numerically evaluate the roof panel capacity and to verify the credibility of the serial failure assumption. Finite element analysis of a roof system subjected to a spatially varying wind load will further help to evaluate the influence of wind model on the fragility and capacity estimation.

The primary motivation of the present study is to evaluate in-situ withdrawal capacities of toenail connections and to develop appropriate statistical and analytical models. The second motivation of the study is to evaluate the behavior of hurricane ties
subjected to multi axial loads and to identify a suitable design space. The third motivation is to formulate a finite element based fragility analysis methodology and to evaluate the sensitivity of roof system fragility estimations to various modeling conditions (both wind and resistance modeling conditions). The above motivations resulted in formulating specific objectives which are listed below for Chapters 2, 3, 4 and 5, which are stand alone journal manuscripts.

a) Development of probabilistic and analytical models of in-situ RTW toenail uplift connection behaviors. (Chapter 2)

b) Evaluation of roof to wall metal connector behavior under multi-axis loading and determination of a suitable and efficient design space. (Chapter 3)

c) Development of a finite element based fragility analysis methodology to identify the effect of various modeling conditions like, connection behavior, nail spacing, sheathing thickness on the roof fragility estimation. (Chapter 4)

d) Evaluation of the influence of connection behavior on the RTW and sheathing panel system fragility and roof sheathing panel capacity. (Chapter 4)

e) Estimation of the sensitivity of the roof system and RTW connection fragilities to spatially correlated wind load. (Chapter 5)

f) Assessment of the influence of spatial correlation on the roof sheathing panel capacity and fragility. (Chapter 5)

The dissertation is presented in the manuscript format with six chapters. The first chapter provides the outline of the research study and explains the organization of the dissertation along with demonstrating the motivation of the current study. Chapters 2, 3,
4 and 5 are presented as independent journal articles with abstract, introduction and background, experimental or analytical model, results and discussion and conclusion given within each of the chapters. Chapter 6 presents the overall conclusion and recommendations for future studies.

1.2 REFERENCES


2. STATISTICAL AND ANALYTICAL MODELS FOR ROOF COMPONENTS IN EXISTING LIGHT-FRAMED WOOD STRUCTURES*

2.1 ABSTRACT

Residential wood-framed construction failures account for the majority of economic losses following hurricanes. A common failure in these constructions during high wind events is loss of roof sheathing, especially in corner areas. Less common perhaps, but usually catastrophic, is the failure of the roof-to-wall connections in these structures. The main objective of the current research project is to evaluate the in-situ capacity of roof-to-wall connections and sheathing to rafter fasteners in light-framed wood construction. The unique opportunity provided by Clemson University to access four residential structures located within a residential complex enabled the collection of perishable yet statistically significant data on the strengths of existing residential structures. The uplift capacities of 100 roof-to-wall toenail connections and 34 plank sheathing units were evaluated from field and laboratory tests. Realizing the key role of probability distributions in developing fragility estimates and loss prediction models, distributions fits and parameters for these structural components are postulated. One conclusion drawn is that the uplift capacities of two and three nail connections are best described by

* This Chapter has been published in Engineering Structures Journal and following is the reference:
lognormal distribution. The initial stiffness and the vertical displacement at peak load of both two nail and three nail connections follow a normal and Weibull distribution respectively. The uplift capacity of plank sheathing follows a lognormal distribution. An analytical model designed to approximate the uplift behavior of toenail connections is developed to facilitate modeling of roof systems. These probabilistic and analytical models developed by this study allow for the performance of detailed reliability based studies on light-framed wood roof structures.

2.2 INTRODUCTION

The tremendous devastation caused by hurricanes mark them as one of the most significant natural hazards affecting the United States. The recent increase in the occurrence of hurricanes [1] and the continuing growth of construction activities along the shorelines has further increased the potential of hurricane damage [2]. The losses suffered by the insurance companies and governments and also the hardships faced by the general public have promoted research initiatives to focus on damage mitigation and loss prediction.

One significant area of research is looking at performance and damage mitigation of low-rise wood structures. Low – rise wood framed structures comprise the majority of residential structures (90%) and have shown appreciable vulnerability to high wind loads. For any structure to perform well, wind forces must be transferred from the roof and walls to the foundations through a complete and continuous vertical load path. Any discontinuity in this load path affects structural performance and subsequently reduces resistance to wind forces. Furthermore, load path discontinuities may result in damage
propagation to other structural components and increase the likelihood of complete failure of the structural system.

Two structural components within this vertical load path, which exhibit substantial vulnerabilities to extreme winds, are the roof sheathing to truss/rafter and roof-to-wall connections. According to the U.S. Census Bureau [3], the vast majority of residential structures (over 80%) in U.S. hurricane-prone regions were built before 1994 – the year building codes were upgraded due to Hurricane Andrew. The failures of pre-1994 structures were most often a result of an insufficient number of nails (nail schedule) in roof-to-wall and sheathing-to-rafter connections, resulting from inadequate or unenforced building codes at the time of construction. While these types of connections are simple to install they were never designed to resist significant uplift loads. As a result, these connections fail at relatively low wind speeds resulting in brittle failure of the structure. Over 90% of the existing inventory of light wood frame houses utilized these connections. Therefore, much effort has been devoted to understanding the uplift performance of these components, to quantify infrastructure vulnerability [4-6] and to develop mitigation solutions.

Numerous studies have been undertaken to understand the uplift behavior of toe nailed connections [7-9] and to investigate various retrofit strategies using both commercial metal connectors and adhesives [8-11]. Cost comparisons of various rafter tie installations indicated that the additional cost incurred by using metal connectors is negligible compared to the total cost of the structure [9]. This resulted in a noteworthy
conclusion stating that toenail connections should not be permitted in hurricane prone regions [7].

Recognizing that the apparent behavior of these connections can be influenced by other elements in the framed structure, Reed et al. [8] also conducted laboratory experiments on systems of connections. Even though the number of connections which could be tested was limited (less than 20), some basic statistical estimators (i.e. mean and variance) of the uplift capacity were obtained along with an estimate of an appropriate probability distribution – Normal [12]. This type of information becomes essential for conducting vulnerability [4, 13] and loss estimation [5] studies.

The loss of roof sheathing during a high-wind event significantly increases building damage as it readily permits water intrusion causing extensive damage to walls and interior contents [14]. Numerous experimental studies on uplift capacities for roof sheathing have been carried out. One such parametric study, estimated the uplift capacity of plywood sheathing for different types and spacing of nail fasteners [15]. Additionally, a functional relationship between individual fastener capacities and sheathing capacities has been proposed [16]. Past studies revealed that a single nail failure often resulted in the progressive failure of entire pieces of roof sheathing leading to complete loss and the uplift capacity can be conveniently described using a Normal distribution [16, 17]. In-service conditions also had a significant influence on the capacity [18].

The estimated probability models for roof component behaviors obtained by others [12, 16] have been utilized to develop loss prediction models and fragility estimates for roof-to-wall connections and roof sheathings. However the laboratory conditions under
which roof specimens are fabricated and tested can be a major source of uncertainty. This is because lab conditions fail to account for the variability due to actual construction practices which may significantly influence the resulting statistical parameters and probability distributions.

The current study seeks to add to the existing knowledge base on the performance of existing low-rise light framed wood structures exposed to high winds. Considering that there is a large portion of the existing inventory that has details similar to those contained in this study, the findings here will be relevant for evaluating risk and the need to retrofit. Furthermore, this performance data can be used to design appropriate retrofit schemes if and when necessary. To this end, this study looks to account for and quantify the variability in structural behavior of two key components, namely the RTW connection and roof sheathing, in their as-built condition. A significant number of actual component specimens were made available for testing due to the scheduled demolition of four residential structures located on the campus of Clemson University. One hundred as-built roof-to-wall toenail connections were tested to determine in-situ uplift capacities and find general connection behavior (i.e. force-displacement). Additionally, 34 as-built roof panels constructed with solid wood plank sheathing, which are typical of buildings constructed 50 -60 years before, were harvested and tested for uplift capacity. Relevant probability models are proposed using these relatively large data sets. An analytical model for RTW toenail connections is also developed and presented to better facilitate the modeling and vulnerability assessment of roof systems exposed to high winds. In addition retrofitted RTW connections were tested for their uplift capacity, the results of
which are given in Appendix (Chapter 7).

2.3 EXPERIMENTAL STUDY

The experimental tests were carried out on roof components found in four identical houses located in the Douthit Hills residential community on the campus of Clemson University, Clemson, South Carolina, USA. The houses are typical residential wooden structures constructed 50 – 60 years ago. These gable roofed duplex houses, scheduled for demolition, offered an excellent opportunity to study the in-situ uplift capacity of an appreciable number of toe nail connections and also to collect roof panel specimens for testing uplift capacity of sheathing in the laboratory. Fig. 2.1 shows a photo of one of the four houses having plan dimensions of 8.23m (27ft) wide by 20.73m (68ft) long. The roof frames were stick built using dimensional lumber and were made up of 38 x 140 mm (nominal 2x6 inch) or 38 x 89 mm (nominal 2x4 inch) horizontal ceiling joists and 38 x 140 mm (nominal 2x6) rafters. A layout of the structure and the roof framing is given in Fig. 2.2. Framing members are spaced at 0.41m (16 in) on center and every fourth rafter was reinforced using a collar tie. The rafters were placed at a 6:12 pitch and attached at their lower ends to the side of the ceiling joist by means of three 3.3 mm (0.131 inch) diameter, 63.5 mm (2.5 in) long smooth shank 8-d common nails. The ceiling joist was attached to the wall top plate using either two or three 4.1mm (0.161 in) diameter, 89mm (3 ½ in) long smooth shank 16-d common nails as illustrated in Fig. 2.3. The roof sheathing was made up of solid wooden planks of 19 mm thick by 140 mm wide (nominal 1 x 6 inches). Each plank was fastened using two 3.3 mm (0.131 inch) diameter, 63.5 mm (2.5 in) long smooth shank 8-d common nails to each rafter. Asphalt shingles
covered the sheathing planks and the building exterior was covered with brick veneer and vinyl siding. Visual inspection of the framing members revealed the wood type to be Southern Yellow Pine (SYP).

**Figure 2.1.** Douthit Hills Duplex residential structure.

**Figure 2.2.** Typical roof framing plan of structure.
2.3.1 **ROOF-TO-TOP PLATE TOENAIL CONNECTIONS**

**Experimental set up**

Previous studies carried out cyclic or monotonic uplift tests on either full scale or reduced scale roof-to-top plate connections modeled in the laboratory. Seldom was uplift tests carried out on in-situ roof to wall toenail connections. Even when conducted, in-situ tests did not control load rate or load sequence and displacements were not monitored. One must further recognize that when tested in a group, the behavior of in-situ connections is significantly influenced by the load redistribution and sharing by the neighboring connections. Also the redundancy of the roofing system allows for stiffer connections to take higher loads than weaker connections. Indeed three to four connections on either side of a given connection can participate in load sharing where the

![Diagram of roof-to-wall connection detail](image)

**Figure 2.3.** Roof-to-wall connection detail.
percentage of load shared is inversely proportional to the distance from the connection considered and directly proportional to the stiffness of the connections themselves [19, 20]. In the current study, the load redistribution effect on the perceived capacity of an individual connection is acknowledged by carrying out uplift tests on systems of four roof-to-top plate (ceiling joist to wall top plate) toenail connections. Furthermore, cyclic loading was applied in order to capture the hysteretic behavior of the connection at relatively low levels of deformation and thereby enable quantification of energy dissipation by the connection under an extreme wind load event. This result can be used to develop analytical models which mimic the behavior of toenail connections.

The weak link in the vertical load path of these structures is considered to be the ceiling joist to top plate connection (not rafter to joist). This is because of the framing scheme used in the given structures. The detail, as presented in Fig. 2.3, shows that three 8-d nails fasten the rafter to ceiling joist and act in single shear while the toenail connections that attach the ceiling joist to top plate act in withdrawal. Hence the ceiling joist to top plate connection is considered to be the weak link and also represents typical toenail connections in other structures.

The test set up has two automated screw jacks mounted on a reaction frame. The jacks carry a spreader beam which applies equal deflection on a system of four connections as shown in Fig. 2.4. Load cells attached to the top flange of the spreader beam both transfer and measure load going to each joist. The number of connections to be tested in a given system was limited by the capacities of the screw jacks and the size of the reaction frame. In order to exercise control over the influence from other
structural, as well as non-structural components, the system of four connections was segmented from the other structural components and crossing members. The whole system was allowed to act as a unit by applying cyclic displacements via the spreader beam.

Figure 2.4. Experimental setup for uplift tests.

Data acquisition

Computer controlled data acquisition devices were used to collect data from the four load cells and four LVDT’s. The load cells are compression/tension capable and have a
capacity of 22.2 kN (5 kips) each. The LVDT’s have a stroke length of \( \pm 50.8 \) mm (2 in) and a spring return armature for easy installation. The screw jacks, driven by micro stepping motors, each have a capacity of 22.2 kN (5 kips) giving a total load capability of 45 kN (10 kips).

**Experimental Test Procedure**

ASTM-D1761 [21] protocol was used as the testing guideline for this study. ASTM D 1761 presents a methodology for evaluating the direct withdrawal resistance of individual mechanical fasteners under monotonic loads. However, only limited guidance is provided to conduct tests on systems of connections for cyclic loading. In the absence of complete guidance, only the rate of withdrawal from monotonic loading test was adopted for the current study.

Three cycles with deflections corresponding to 1.6, 3.2 and 4.8 mm (0.0625, 0.125, 0.1875 in) were applied to the test segment via the spreader beam, at a recommended fastener withdrawal rate of 2.54 mm/min (0.10in/min) \( \pm 25 \% \), per ASTM-D1761. This displacement sequence was selected so as to adequately capture the hysteretic behavior of the connection in the range of low to moderate forces. One may expect that a roof connection may reasonably see uplift loads that are above the expected service loads but below the extreme loads multiple times during its lifetime. Therefore, an understanding of the cyclic behavior in this range is desired.

Once the 4.8 mm (0.1875 in) deflection cycle was completed, the load was increased at constant rate until failure (i.e. load peaked out). The dead load on each set of connections was estimated based on the recorded load after complete separation of the
ceiling joist and top plate occurred. The dead load is the self-weight of the ceiling joist, crossing members and framing system. The uplift capacity of each connection is taken as the maximum measured load at each joint minus this dead load. As the applied load and measurement locations were not concentric to the connection considered, adjustments were made to account for the actual placements of LVDTs and load cells using the relative moment arm.

2.3.2 Results

Twenty five specimens representing a total of 100 individual roof-to-wall connections were tested for the current study. Out of the 100 connections, 81 were constructed using two 16d nails and the remaining 19 used three 16d nails. Three types of failure mechanisms were observed which are 1) failure due to nail withdrawal from the wall top plate 2) failure due to splitting of wood in ceiling joist and 3) combination failure - withdrawal of one nail concurrent with the splitting of wood due to pull-through of the other nail. The latter two failure mechanisms are considered as brittle modes of failure even though there is an initial yielding of nails. This is because failure of the connection occurs mainly due to the splitting of wood which is brittle in nature. However nail withdrawal involves yielding of nail followed by pure withdrawal which exhibits a more ductile behavior.

Fig. 2.5a shows the typical load–displacement curve of a two nail connection that failed due to pure nail withdrawal. The initial response of the connection is characterized by hysteretic behavior capturing the yielding of nails and then followed by gradual off-loading as the nails withdraw. Fig. 2.5b shows the typical response of a connection that
experienced a combined mode of failure. The significant difference between the two behaviors is that the combined failure is characterized by load stepping in the load-displacement curve in the post ultimate load region. The sudden drop in load is due to the brittle nature of splitting wood. Images of the withdrawal and combination failure modes are given in Fig. 2.6.

**Figure 2.5.** Typical response of connections failing due to (a) Nail withdrawal (b) Combination.

**Figure 2.6.** Failures by (a) Nail withdrawal (b) Wood split.
The test data shows that 81 percent of the connections failed due to nail withdrawal, 16 percent due to the combined failure mechanism and the remaining 3 percent due to complete splitting of wood. The statistics clearly indicate that pure nail withdrawal is the dominant mode of failure for aged in-situ construction. The higher capacities of connections failing in one of the latter two failure modes indicate that they may be more preferred. Because the dominant failure mode may also be the least desirable, retrofitting the connections with metal straps is often employed to compensate.

In addition to the nail embedment length, the withdrawal capacity of the nail is a function of the angle of the nail, type and grade of lumber and the moisture content. In order to account for the effect of moisture on the capacity, the in-situ moisture content of each connection was estimated using a two prong moisture meter. After adjustments for type of lumber, the average moisture content was found to be 8.5 percent with a standard deviation of 0.63 percent. The estimate of the correlation between the moisture content and ultimate uplift capacity was also examined and found to be 0.11. This indicated that over the range of moisture contents recorded (7.5% - 9.5%), these two parameters were only slightly correlated and hence can reasonably be ignored for reliability studies in which these connections are involved.

**Uplift capacity**

Uplift capacity is the maximum load sustained by the connection minus the dead load and is defined as the ultimate strength of the connection ($F_{ult}$). The average uplift capacity of the two-nail connections in this study is 1.51 kN (341 lbs) with a coefficient of variation (COV) of 0.36. The three-nail connections have, on average, a 30 percent
larger uplift capacity than their two-nail counterpart with a mean of 1.97 kN (442 lbs) and a COV of 0.38. This is an interesting result in that one would have suspected approximately a 50 percent increase in capacity since there was a 50 percent increase in nail embedment length. One possibility for this discrepancy is that with the two-nail connection, the nails are driven at opposing angles, one on each side of the ceiling joist, and both must yield for the nails to withdraw. However, in the three nail connection, two nails angle in from one side while the third nail is driven at an opposing angle from the other side. This imbalance in the resistance causes the single nail to yield before the double nails. A small lateral shift occurs in the connection as one nail yields and the other two primarily avoid yielding while only experiencing direct withdrawal. Fig. 2.7 pictorially describes this phenomenon.

The wind pressure that could be safely withstood by the connections is evaluated from the uplift capacities. This was calculated to be 1.05 kPa (22 psf) for two nail connections and 1.34 kPa (28 psf) for three nail connections. Generally a factor of safety (FOS) from 2 to 5 [7, 8, 22] is used to estimate the design capacity of the connections. After applying a FOS of 2 the ultimate capacity of connection obtained in terms of pressure was 0.53 kPa (11 psf) and 0.67 kPa (14 psf) for two and three nail connections respectively. Although this does not account for the help given by the dead load, this is considerably lower than the wind pressures that would act on roofs at times of wind storms. Therefore it is clear that toenail connections are not structurally safe against extreme wind loads that occur in hurricane prone regions.
Table 2.1 presents the uplift capacity statistics for the two types of toenail connections considered in this study and compares them with the findings from previous research studies. These studies applied monotonic uplift loading on roof-to-wall connections as opposed to the cyclic loading applied in the current study. The comparison table presented herein assumes that there is no strength degradation in the present case due to repetitive loading. The results from several other studies [10, 23] are not presented for comparison as their complete statistics and sample sizes are unknown. The uplift capacity of in-situ connections was found to be less than the uplift capacity of lab-tested toenail connections. The lower failure capacities observed in this study may be due to the deterioration in the joint strength with age as the connections under
consideration were constructed 50-60 years before. The deterioration may be due to decline in the wood quality, wood shrinkage or joint fatigue. Cyclic loading may also be a possible cause for the reduced capacity. The most notable change observed for the in-situ condition is the appreciable increase in the COV of the capacity. The larger mean values and smaller COVs for the laboratory tests are likely due to the controlled manner in which the test specimens were constructed. In-situ as-built testing has the ability to capture the variability in connection behavior due to actual construction practices. As seen in Table 2.1, the COV estimates resulting from laboratory tests can underestimate actual COVs by as much as 50 percent. Failure to capture this uncertainty may significantly affect the reliability assessment of these connections under wind loads. Sensitivity analyses may help to quantify the effect of this increased COV on reliability assessments.

The type of failure mechanism generally has a considerable influence on the capacity of the connection. But due to insufficient number of samples in two of the failure mechanisms from the present study, statistically significant inferences cannot be made from the estimates. However, the mean estimate is provided herein for the sake of comparison. The results are for the two-nail connections. Out of 81 two-nail connections, 65 failed due to nail withdrawal, 3 due to wood split and 13 connections failed in combined failure mode. The mean ultimate uplift capacity of the two-nail connections is 1.43 kN (322 lbs), 2.26 kN (508 lbs) and 1.76 kN (395 lbs) for withdrawal, wood split and combined failure modes respectively. The above statistics represent an average increase in connection capacities of 25 to 55 percent over pure withdrawal,
which reinforces the assertion that wood splitting, while more brittle, appears to be a preferred mode of failure.

**Table 2.1** Comparative table of uplift capacities.

<table>
<thead>
<tr>
<th>Type of connection</th>
<th>No. of specimen</th>
<th>Average ultimate capacity kN (lbs)</th>
<th>COV</th>
<th>Study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toenail (SP)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) 2 – 16d</td>
<td>81</td>
<td>1.51 (341)</td>
<td>0.36</td>
<td>Present</td>
</tr>
<tr>
<td>(b) 3 – 16d</td>
<td>19</td>
<td>1.97 (442)</td>
<td>0.38</td>
<td>study</td>
</tr>
<tr>
<td>Toenail 2-16d box nail</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) Spruce Pine Fir (SPF)</td>
<td>16</td>
<td>1.56 (350)</td>
<td>0.164</td>
<td></td>
</tr>
<tr>
<td>(b) Douglas Fir (DF)</td>
<td>14</td>
<td>2.59 (584)</td>
<td>0.212</td>
<td>[7]</td>
</tr>
<tr>
<td>(c) Southern Yellow Pine (SP)</td>
<td>14</td>
<td>2.69 (605)</td>
<td>0.155</td>
<td></td>
</tr>
<tr>
<td>Toenail 3 -8d nail (SP/SPF)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) Single</td>
<td>16</td>
<td>1.92 (430)</td>
<td>0.23</td>
<td></td>
</tr>
<tr>
<td>(b) Repetitive (System of 7 connections)</td>
<td>2</td>
<td>2.99 (670)</td>
<td>na¹</td>
<td>[8]</td>
</tr>
</tbody>
</table>

¹ not available due to small sample size

**Stiffness**

Knowledge of relative initial stiffness \(k_o\) of the roof-to-wall connection is critical in understanding cyclic behavior and in developing analytical models. Studies to estimate the stiffness of toenail connections have seldom been carried out in the past. As such, one significant contribution of this study is the explicit treatment of connection stiffness. Due to the nonlinearity of the connection behavior the secant stiffness is taken
as a representative initial stiffness. Three displacement values were considered as candidates for calculation of the secant stiffness – 0.254 mm, 1.6 mm, 3.2 mm (0.01, 0.0625, 0.125 in). The estimate of secant stiffness using the deflection at 0.254 mm was found to be an unreasonable indicator because it is very sensitive to minor fluctuations in the recorded data. As such, this estimate is unstable and a poor indicator of generalized behavior. The secant stiffness at 3.2 mm was also found to be misleading, as the response of the connection is mostly nonlinear at this range. Hence, the secant stiffness taken at a 1.6 mm displacement is assumed to be the most appropriate because of the numerical stability of the load at this displacement and the overall response linearity.

The average secant stiffness of the two-nail connections is 0.37 kN/mm (2126 lbs/in) with a COV of 0.36. The three nail connection has an average stiffness of 0.47 kN/mm (2696 lbs/in) with a COV of 0.42. The average stiffness of three-nail connections is 27% greater than their two-nail counterpart. The higher COV for 3-16d nails in comparison to 2-16d nails may be due to a smaller sample size. In order to check the influence of the failure mechanism on the stiffness of the connection, an estimate on the average stiffness for each failure mode is obtained. The average stiffness of the 2-nail connection that failed due to withdrawal was found to be 0.360 kN/mm (2055 lbs/in), for those which failed due to splitting it was 0.492 kN/mm (2807 lbs/in) and for those connections that failed in a combined mode it was 0.407 kN/mm (2329 lbs/in). Since only three connections failed due to splitting of wood, the mean stiffness reported herein is only for comparison purpose and no statistical inference is made. One is able to observe the same
relationship between stiffness and failure mechanism as was seen between uplift capacity and failure mechanism.

**Displacement at peak load**

In an effort to help describe the non-linear behavior of roof-to-wall toenail connections, the vertical displacement coinciding with ultimate uplift capacity is tracked ($\delta_{PL}$). For the 2-nail connection the mean displacement is found to be 11.2 mm (0.44 in.) with a COV of 0.54. The 3-nail connection gives mean and COV values of 11.9 mm (0.47 in) and 0.51 respectively. One readily made observation is that the COV values for this parameter are much higher than for the stiffness and capacity. The displacement value at which peak load occurs should be sensitive to the embedment length which in turn is dependent on the angle at which the nails are driven into the connection. Considering that the tested connections are constructed in the field under real circumstances (i.e. not having lab type control), one would indeed expect that a great deal of variability exists.

2.4 **PLANK SHEATHING ROOF PANELS**

Even though current construction practices include the use of plywood or OSB panels as the roof sheathing, there is still a large inventory of existing buildings that have been constructed with plank sheathing. Quantification of the uplift capacity of plank sheathings in existing structures will facilitate developing appropriate retrofits, if necessary, that would protect them from high wind loads. The present study aims to identify the uplift capacity of plank sheathing and compares it with the uplift capacity of
plywood/OSB sheathing. The influence of failure modes and nailing patterns is discussed

2.4.1 Experimental setup

Cyclic testing of roof sheathing is the preferred testing method as it considers the fatigue loss of uplift strength of roof sheathing. Cyclic pressures corresponding to the actual pressure on roof sheathings are applied while displacements are recorded. Unfortunately, BRERWULF (Building Research Establishment Real-time Uniform Load Follower), a testing apparatus generally used to test the uplift capacity of roof sheathing under cyclic and monotonic loadings is only capable of developing pressures up to 10 kPa (200 psf). A preliminary test of the roof panels indicated that panel capacities would likely exceed this limit. Therefore, a suction chamber capable of developing the requisite pressures was utilized but it was only capable of applying them in a monotonic fashion.

The roof panel specimens had dimensions averaging 1295 mm by 1650 mm (51 x 65 inches). The size of the specimens was driven by the size of the suction chamber and also by feasibility of removal from the roof. Each panel specimen contained four rafters spaced at 410 mm (16 inches) o.c. Planks which were 19 mm x 140 mm (nominal 1 x 6 inch) were attached to the rafters by two 8d smooth shank hand driven nails spaced on average at 76 mm (3 inches). From visual observation it was noted that the framing members were Southern Yellow Pine (SYP). The specimen was placed inside the suction chamber, sheathing side down with rafters spanning onto chamber walls and sealed using plastic sheathing and duct tape (Fig. 2.8a). It was ensured that no leakage of air occurred and then a negative pressure was applied uniformly over the entire roof unit. Suction
pressure was applied at a constant rate until failure – defined as the separation of at least one plank. Since failure of a wooden plank was followed by the failure of entire roof sheathing unit, application of negative pressure was stopped when the failure of the first plank was observed. Fig. 2.8b shows a typical failure of one such plank sheathing specimen. Positive pressure acting on roof plank sheathing was not considered for the present study. Using an electronic data acquisition system the pressure inside the suction chamber was recorded. The uplift capacity of the plank sheathing unit is taken as the maximum pressure withstood by the unit prior to first failure.

![Image](image_url)

**Figure 2.8.** (a) Suction test apparatus for uplift test on roof sheathing planks (b) Typical failure of a sheathing plank unit.

### 2.4.2 Results

A total of 34 plank sheathing units were tested in the suction chamber under increasing monotonic negative pressure and the average uplift capacity was 11.54 kPa (241 psf) with a COV of 0.15. The average uplift pressures needed to pull off plank sheathing is significantly larger (about 11 times) than that required to fail the roof-to-wall
connections described in the previous section. This indicates that under an extreme wind event, roof-to-wall connections in buildings of similar construction are more likely to fail prior to loss of the sheathing. The above failure sequence is catastrophic because when the roof system is lifted off; walls lose their lateral support and may subsequently fail. The result from the present study is compared with the previous results from lab uplift tests on OSB/plywood sheathing.

To justify this comparison, one must look at the failure modes common to both types of sheathing (plank and OSB/plywood). In the present study, failure of plank sheathing was exclusively due to pull out of nails from the framing element. OSB/plywood sheathings from previous studies have shown that failure can occur from either nail pull out, nail pull through or a combination thereof [15, 16, 24-26]. The type of failure was affected by the type of load (uplift or uplift/lateral), sheathing thickness and nailing schedule. The studies on OSB/plywood showed that the majority of the sheathing subjected to uplift failed due to nail pull out. As such, a cursory comparison between the two types is appropriate as they both predominantly failed due to nail pull out. In one laboratory test [17], 30 specimens of oriented strand board (OSB) sheathing panels – 1.22m x 2.44 m (4ft x 8 ft) – were constructed and tested using BRERWULF. The OSB sheets were 11.9 mm (15/32 in) thick and were attached to the southern pine framing system spaced at 0.61 m (24 in) using 8d nails at a spacing of 152 mm (6 in). The mean uplift capacity was estimated to be 6.3 kPa (131 psf) with a COV of 0.14. Though the results from the above study are not directly comparable with the present result, it indicates that the capacity of OSB sheathing is significantly less than that of plank
sheathings. The mean capacity of plank sheathing from the current study is almost
double (183\%) the estimated capacity of OSB sheathings.

Laboratory test of 10 specimens of 1.22 m x 2.44 m (4 ft x 8 ft), 11.9 mm (15/32 in)
 thick plywood sheathing attached to a Spruce Pine Fir framing system spaced at 0.61 m
(24 in) using 8d nails at a spacing of 152 mm (6 in) / 304 mm (12 in) estimated the
capacity to be 2.87 kPa (60 psf) with a COV of 0.20 [4, 15, 16]. The capacity of 4
specimens using 6-d nails, for the same sheathing and framing system as above was
estimated to be 1.2 kPa (25 psf) with a COV of 0.15. Failure of sheathing in the above
two tests were primarily due to nail pull out. In all cases, the capacity of plank sheathing
from the present study was greater than the plywood sheathing but the COVs were
comparable. This higher capacity is understandable when one recognizes that total
number of nails required for attaching plank sheathing is almost double the number
required for panel sheathing.

2.5 STATISTICAL MODELS

2.5.1 Roof-to-top plate toenail connections

To make statistical inferences and to carry out reliability studies it is essential to
identify appropriate statistical models for describing the connection behavior parameters
(i.e. uplift capacity \(F_{ulh}\)), initial stiffness \(k_o\) and vertical displacement at peak load
\(\delta_{PL}\). Goodness-of-fit (GOF) tests are used to ascertain the most plausible probability
distributions that would describe the collected set of observations. The Anderson Darling
GOF test, sensitive in the tails of the distribution, is used to check the plausibility of the
distribution and the distribution parameters are calculated using the maximum likelihood
method. Since a 5% level of significance is traditionally used by experimenters, the same is used for the current study. Various distribution types including normal, lognormal, extreme value and Weibull distributions are considered. The GOF tests are evaluated by the $p$-value where if the value of $p$ is greater than the considered level of significance i.e., 0.05, then the assumed distribution is considered to be plausible. The larger the $p$-value, the stronger this statement becomes.

For the two nail connection, uplift capacity is most strongly a lognormal distribution with a $p$-value of 0.627. The initial stiffness is best described by a normal distribution ($p$-value = 0.454) while the displacement at peak load is only plausibly described by a 3-parameter Weibull distribution ($p$-value = 0.085). For the three-nail connection the distribution fits for uplift capacity, stiffness and peak displacement are taken as for the two-nail connections with respective $p$-values of 0.346, 0.097 and 0.101. A visual comparison of theoretical and empirical CDFs (cumulative distributive function) for the uplift capacity of both the 3-nail and 2-nail connections is given in Fig. 2.9. As expected, larger deviations between the CDFs appear in the 3-nail connection data than appear in the 2-nail connection data. This is because of the appreciably smaller sample size for the former. In short, one may see that the CDFs reinforce the findings of the GOF test and that the larger deviations result in lower $p$-values. Similar trends are seen in Figs. 2.10 and 2.11 where the CDFs are given for stiffness and displacement at peak load respectively. Estimated parameter values for all selected distributions are presented in Table 2.2.
Figure 2.9. Comparison of theoretical and empirical CDFs for uplift capacities.

Figure 2.10. Comparison of theoretical and empirical CDFs for initial stiffnesses.
Since three parameters of connection behavior are being tracked, a measure of the statistical dependence between the three is imperative. This measure is given through the correlation coefficient – a term used for quantifying statistical dependence in many reliability based studies using a tool like the Nataf transformation [27]. The results of this study indicate that uplift capacity and stiffness are significantly correlated having correlation coefficients of 0.62 and 0.77 ($\rho_{F_{\text{ult}}, k_o}$) for the two and three-nail connections respectively. Correlation coefficients between $\delta_{\text{PL}}$ and the other two parameters tend to be appreciably lower with the following values – 2-nail $\rho_{k_o, \delta_{\text{PL}}} = 0.096$ and $\rho_{F_{\text{ult}}, \delta_{\text{PL}}} = 0.393$ – 3-nail $\rho_{k_o, \delta_{\text{PL}}} = 0.194$ and $\rho_{F_{\text{ult}}, \delta_{\text{PL}}} = 0.409$. This low level of correlation

Figure 2.11. Comparison of theoretical and empirical CDFs for peak load displacements.
associated with $\delta_{PL}$, when considered in conjunction with the high variability, further illustrates its sensitivity to nail placement.

### 2.5.2 Plank sheathing

A-D GOF tests were carried out to estimate the best fit that describes the uplift capacity of the plank sheathing. The two parameter lognormal distribution having parameter values of $\lambda = 2.434 \ln(kN/m^2) \ [5.47 \ln(lb/ft^2)]$ and $\zeta = 0.15$ is found to be the most plausible. The associated p-value is 0.284.

### 2.6 ANALYTICAL MODEL FOR ROOF-TO-TOP PLATE CONNECTIONS

To better facilitate evaluation of roof system responses in both design and reliability based studies, an analytical model approximating the response of roof-to-wall toenail connections is developed. Though there are three failure mechanisms that describe the failure of the toenail connection, the analytical model presented herein represents the dominant mode of failure – failure by nail withdrawal. In the past these connections were generally modeled as pinned connections having a specified uplift capacity. This specific assumption fails to simulate the actual nonlinear response, hysteretic behavior and subsequent failure of such connections when exposed to fluctuating extreme wind loads.

Considering that this analytical model is likely to be used in research based studies the open source finite element package, OpenSees [28], is selected for model development. The connection is modeled using a zero length element in conjunction with a Pinching4 material. This material model facilitates multi-linear behavior with an ability to capture both strength and stiffness degradation. Furthermore, this material provides for a loss of strength beyond a user specified limit ($u_3$).
Table 2.2 Proposed probability distributions for connection behavior parameters.

<table>
<thead>
<tr>
<th>Connection Type</th>
<th>Uplift capacity ($F_{ult}$) kN (lbs)</th>
<th>Initial stiffness ($k_o$) kN/mm (lbs/in)</th>
<th>Displacement at peak load ($d_{PL}$) mm (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dist</td>
<td>$\lambda$</td>
<td>$\zeta$</td>
</tr>
<tr>
<td>2 – 16d</td>
<td>LN</td>
<td>0.356 (5.771)</td>
<td>0.35</td>
</tr>
<tr>
<td>3 – 16d</td>
<td>LN</td>
<td>0.613 (6.028)</td>
<td>0.36</td>
</tr>
</tbody>
</table>

* $k = $ shape factor, $u = $ scale factor, $e = $ threshold

This feature is important in that it can model the failure of a connection which is an essential part of roof system modeling. Complete documentation of this material may be found on the OpenSees website [28].

The pinching4 material requires the definition of 28 parameters. These parameters are used to define the backbone and ensuing degradation rules of the material. However, many of these quantities are set to zero for the proposed model. Fig. 2.12 presents a schematic of the model backbone behavior and any nonzero parameters required. This model requires the user to provide three inputs – ultimate uplift capacity ($F_{ult}$), initial secant stiffness ($k_o$) and displacement at peak load ($d_{PL}$). The ultimate displacement ($u_3$) at which all strength is lost is taken to be 40 mm (1.57 in.) which is an
approximate displacement value for complete withdrawal of 16d toenails. The only type of cyclic degradation used in this model is unloading stiffness degradation (all $g_K$ – see OpenSees documentation) taken to be -0.5. The reloading force ratio ($r_{\text{ForceP}}$) is taken to be 0.6. Though it is not possible nor warranted to simulate exactly the response of a connection, a generalized agreement of behavior including initial stiffness, ultimate capacity and nonlinear action is sought.

![Graph](image)

**Figure 2.12.** Proposed analytical model using a pinching4 material for capturing the uplift behavior of a roof-to-wall connection.

Fig. 2.13 gives a comparison of the experimental and analytical models for a set of four connections. Fig. 2.13a (Connection A21) was previously presented in Fig. 2.5a. The plots indeed demonstrate the ability of the proposed model to capture the desired behavior. To further validate the appropriateness of the proposed model, the energy dissipated by the analytical model, when subjected to the same displacement sequence as the experiment, is compared with experimental results. The errors or differences in the energy dissipation are given in Fig. 2.13. Errors for the illustrated connections range
from 6.7 percent to 0.7 percent – well within acceptable limits. One may note that connection A22 (Fig. 2.13b) exhibits the largest discrepancy. The high initial stiffness and long plateau region are some of the likely reasons for this difference. Fortunately few connections displayed these characteristics. In the case of connection A24 (Fig. 2.13d) a very strong agreement is achieved including hysteresis loops and off-loading stiffness.

Figure 2.13. Comparison of experimental and analytical connection behavior.
2.7 CONCLUSIONS

The susceptibility of roof-to-wall connections and roof sheathing in light frame residential structures to damage from high winds has always been a major concern. New building codes have mandated using connections such as hurricane ties and metal straps for roof-to-wall connection that do not rely on the limited strength of toenail connections to minimize the damage. However, in existing construction the use of toenail roof-to-wall connections and wood structural planks is prevalent in hurricane prone regions. Thus, an evaluation and subsequent modeling of their efficacy is necessary. This study evaluated the component behavior of roofs under uplift loads in a statistical fashion and it also developed an analytical model of the structural behavior of roof-to-wall connections. This analytical model can be used to better quantify the redistribution of forces to connections which are part of a roof system. With the ability of the analytical model to capture the failure of individual connections – the sequence of roof failure during high wind events can be more closely examined.

Three connection parameters (ultimate uplift capacity ($F_{ult}$), initial stiffness ($k_o$) and vertical displacement at peak load ($\delta_{PL}$)) are used to define the analytical model. As such relevant statistics and probability distributions are proposed for these three parameters. This study proposes the use of the lognormal distribution to model uplift capacities for both two and three nail connections. The normal distribution and three parameter Weibull distribution are proposed for $k_o$ and $\delta_{PL}$ respectively. These distributions and parameters can be used to evaluate the component level reliability of toenail connections and roof sheathing. With a well-defined limit state, the results can be used to assess the
system reliability and thus can contribute significantly to performance based evaluation of residential structures in hurricane prone areas. The probability models for connection parameters can also be used to formulate damage prediction and loss calculation models.

2.8 ACKNOWLEDGEMENTS

This study has been supported by the National Science Foundation (NSF) under award number CMS-0642455. The authors would also like to gratefully acknowledge the advice and direction provided by Dr. Scott D. Schiff for this project. The views expressed in this paper are solely that of the authors and do not reflect the opinion of NSF.

2.9 REFERENCES


3. MULTI-AXIS TREATMENT OF TYPICAL LIGHT-FRAME
WOOD ROOF-TO-WALL METAL CONNECTORS IN DESIGN*

3.1 ABSTRACT

Proper selection of metal roof-to-wall connectors is needed to provide a cost-effective load path to transfer uplift loads on a roof system down to the supporting walls and transfer lateral loads into and out of the roof diaphragm of light frame wood structures. Structural engineers, architects and builders rely upon published design values in catalogues, software, and websites provided by individual manufacturers to aid in the appropriate selection of connectors once the determination has been made for the required capacities of the connector. To date the state-of-the-practice for dealing with multi-axis loads in these connectors is to use a linear unity equation based on uni-axis design values. However, no significant validation of this practice is to be found in the literature. This study experimentally examines three very common connector types under both bi-axis and tri-axis loads and helps to understand the behavior of such connectors under multi-axis loads. After testing over 350 connections and performing detailed analyses, the currently used design equation is found to be inefficient (least usable design space) and overly conservative. Based on the criteria of efficiency, performance and safety, a design space using either the linear unity equation or simply take a 25%

* This Chapter has been accepted for publication in Engineering Structures Journal and following is the reference:
reduction on all allowable loads is proposed. The proposed design space for the three types of connectors is shown to have a high level of safety and adequate performance while providing up to 2.5 times the usable design space as compared with the current practice.

3.2 INTRODUCTION

Metal connectors are extensively used to establish roof-to-wall and wall-to-foundation connections in light frame wood structures. Metal connectors replace or complement traditional nail fasteners and can be effective in withstanding extreme seismic and wind events. If properly installed, the connectors serve to establish a continuous load path in both residential and commercial structures. A disconnected load path is seen as one of the leading causes for roof uplift failure during intense windstorms and hurricanes. If metal connectors are to provide efficient yet effective solutions for establishing robust load paths, it is important that one understands their behavior in both uni-axis and multi-axis conditions. Appropriate yet straightforward design guidelines can then be developed to accommodate this behavior. Current practice is to obtain the design capacity of the connector and the proper method of installation in manuals provided by each manufacturer. Manufacturers apply AC13 – acceptance criteria [1] approved in 2006 by the International Code Council Evaluation Services (ICC-ES) for joist hangers and similar devices to determine design capacities. These criteria are specifically defined to develop allowable vertical capacities of a connector where the allowable design capacity is the smallest value taken from:
a) Lowest vertical ultimate load divided by three if only three tests are conducted and each load does not vary by more than 20% from the average vertical load. Alternately, if six or more tests are conducted, the design capacity may be taken as the average of the ultimate vertical load divided by three. The same procedure is also currently used to calculate the capacity of roof-to-wall connectors when subjected to lateral loads even though lateral capacities are not explicitly addressed in AC13.

b) The average load corresponding to a 3.2 mm (0.125 inch) vertical displacement.

c) The NDS (National Design Specification) [2] prescribed allowable design load for the connected wood members or the nail fasteners.

Although AC13 was originally intended to determine design values for hangers subjected to gravity loads, these design values are used with possible adjustments for wind and seismic loads. The test protocol only subjects the specimen to a slowly increasing monotonic load where degradation due to cyclic loading, especially from a seismic event, is not accounted for in the test protocol.

Multitudes of research studies are being carried out in the form of full-scale model studies and wind tunnel experiments in an effort to better comprehend the fluctuating wind and seismic demands on light frame wood components and systems. In contrast, few research studies [3-9] (disregarding manufacturer studies as they are generally not accessible) are available to recognize and understand the ultimate resistance of structural components like roof-to-wall metal connectors. Furthermore, except for a few studies [8, 10, 11] which are discussed later, none have addressed the multi-axis load interaction
effect on the resistance capability of a connector. When wind flows over a typical low-rise building with a low angle roof it generally causes uplift loads on the roof, suction on the side and leeward walls and positive pressure on the windward wall resulting in concurrent loads in all three primary axes of the roof-to-wall connectors – uplift, parallel to top-plate and perpendicular to top-plate. Common practice for dealing with these concurrent loads is to use a simple linear interaction equation for determining acceptability where the design values for each direction are determined based on uni-axis tests. This linear combination can be expressed as given in Equation 3.1:

\[
\frac{\text{Req'd uplift}}{\text{Allowable uplift}} + \frac{\text{Req'd parallel to plate}}{\text{Allowable parallel to plate}} + \frac{\text{Req'd perpendicular to plate}}{\text{Allowable perpendicular to plate}} \leq 1.0 \quad (3.1)
\]

where a permitted design is any combination of orthogonal loads which will keep the resulting value below 1.0. This design equation, loosely termed an interaction equation, has been selected for simplicity and is believed to be a conservative representation of the true load interaction. However, limited experimental data is available for verifying the above assumption. Furthermore, the “three specimen” criterion, discussed above, for arriving at the uni-axis design capacities may or may not be appropriate for metal ties under multi-axis loading.

Indeed, the appropriateness of using the “three specimen” criterion for metal connectors has been scrutinized and its shortcomings have been presented elsewhere [9]. Rosowsky et al. explicitly stated that three tests do not provide a significant sample size from which to make decisions. This is particularly true if there is a lot of uncertainty in
the testing procedure and/or specimen materials. Furthermore, a factor of safety (FOS) of 3 may be overly conservative if tearing of the metal strap is expected.

Under multi-axis loading ambiguity exists in how and where to measure the deflection in order to check the 3.2 mm (0.125 in) deflection criterion. Combined loading causes metal connectors to deform and displace in all three orthogonal directions. This raises the question: Should the 3.2 mm (0.125 in) deflection criterion be applied to the resultant displacement or the displacement in each of the primary axes? No guidance on this issue is provided in current testing protocol [1].

The third criterion limits the capacity of the metal ties to the NDS allowed capacities of connected wood members or the capacity of nails attaching the metal connectors. The nails that fasten the metal ties are subjected to combined loads resulting in combined failure modes. However, lack of guidance from governing bodies like ASTM (American Society for Testing and Materials) in setting up the experiment for combined loading complicates the scenario.

Only a few research studies in the past tested the multi-axis capacity of metal connectors. From these studies it is apparent that metal ties enhanced both the lateral and uplift capacity of roof-to-wall connection [5, 8] as compared with toenails. Uplift capacities of connections which are part of a system tend to be higher than capacities of individual connectors [7]. When roof and wall (masonry and wood) assemblies, fastened with metal connectors, were tested under combined uplift, in-plane (parallel-to-plate) and out-of-plane loads (perpendicular-to-plate) it was discovered that out-of-plane loads applied as uniform constant pressure on the wall section did not significantly alter the
resistance capability of connector in the other directions [10]. As a result, a vector form of the interaction equation was suggested as an alternate to a linear interaction equation [10]. In another study the combined effect of simultaneously applied in-plane and uplift cyclic and monotonic loads on the capacity of connectors in a roof and wall assembly was investigated [11]. The interaction plot indicated that the combined cyclic loading did reduce the capacities to some degree [11]. However, the metal connectors are still much stronger than toenail connections – even when subjected to multi-axis loads [8]. Therefore understanding the combined load effect on metal connectors is key to effective performance of roof system [12].

The primary objective for the present experimental study is twofold – 1) to verify the perceived notion that the capacity of the connector is reduced when loaded in more than one direction and that the linear interaction equation is conservative in acknowledging this combined load effect 2) if appropriate, to propose an alternate efficient design strategy for roof-to-wall connectors.

![Figure 3.1](image)

**Figure 3.1** Typical roof – to - wall Connectors (a) Type 1 (b) Type 2 (c) Type 3.
In order to achieve the above two objectives in a generalized manner, the current study makes a concerted effort to study three classes of typical metal roof-to-wall connectors under uni-axis, bi-axis and tri-axis loading conditions. The three connectors are selected based on their characteristics and their ability to represent general classes of connectors. The evaluated connectors, which are shown in Fig. 3.1, all exhibit an uplift stiffness much higher than their shear stiffnesses. The connector types and their basic characteristics are given as follows:

a) **Type 1** – Simpson Strong-Tie’s H10 metal connector. This represents flat plate like connectors that have significant differences in the in-plane and out-of-plane stiffnesses. This connector exhibits a significantly larger stiffness in the in-plane direction. It is fabricated from 18 gauge ASTM A653 GR 33 steel which has the following properties: minimum yield strength = 227 MPa (33 ksi), minimum ultimate strength = 310 MPa (45 ksi) and modulus of elasticity = 207 GPa (30,000 ksi).

b) **Type 2** – Simpson Strong-Tie’s H2.5A metal connector. This represents twisted ties that exhibit near equal stiffness in both the in-plane and out-of-plane lateral directions. It is fabricated out of 18 gauge ASTM A653 GR 33 steel.

c) **Type 3** – Simpson Strong-Tie’s META20 straps. This represents embedded straps that have a moderately higher stiffness in the out-of-plane direction as compared to the in-plane stiffness. It is fabricated out of 20 gauge ASTM A653 GR 33 steel.
3.3 EXPERIMENTAL SETUP AND METHODOLOGY

To facilitate the bi-axis and tri-axis testing of roof-to-wall connections, a unique test fixture was designed and built. This fixture, which is seen in Fig. 3.2, consists of a three dimensional reaction frame that holds the test specimen, loading mechanism and data acquisition devices like load cells and LVDT’s (Linear variable displacement transducer). The rigid frame is designed such that it reacts against itself and does not require any special foundation. It is equipped with two 22 kN (5 kip) screw jacks – one to apply an increasing vertical uplift force and the other to apply an increasing horizontal force component onto the specimen. The specimen is located between two horizontal steel plates. The bottom plate is capable of motion in two orthogonal horizontal

![Figure 3.2. Tri-axial test frame.](image)
directions by using two sets of low friction roller bearings. The upper plate is capable of vertical motion that is guided by four steel rods and low-friction roller bearings. The specimen is then bolted to the bottom plate and then attached to the top plate using a steel saddle that prevents the specimen from rotating when it is being loaded (Fig. 3.3).

Uplift load is applied to the top plate through a vertical steel rod. An 89 kN (20 kip) load cell, placed in-line with the steel rod for measuring uplift force, is connected to a loading lever at the top. This lever is a rectangular steel tube that is pivoted at one end and is supported on the other end by the vertical screw jack. The lever doubles the amount of load the screw jack provides bringing up the total capable load to 44 kN (10 kips) vertically. A counter weight is provided to offset the weight of the steel plate itself, to ensure that the measured uplift force is a direct measure of load in the specimen and to provide for ease of installation and removal of specimens.

Figure 3.3. Generalized specimen schematic.
Horizontal load to the bottom plate is applied by a system similar to that which applies the vertical load. As illustrated in Fig. 3.4, a screw jack and lever system with an inline load cell apply the load. A secondary plate on transverse rails sits atop the bottom plate. Load is not directly applied in this transverse direction and any desired movement this direction is restrained by a load cell to measure the resulting force acting in the transverse direction. Great care was taken in the design of the test structure to eliminate alternate load paths. This is because alternate load paths (i.e. not passing through a load cell) can result in an overestimation of actual connector capacities.

**Figure 3.4.** Top-view of detail for load application to bottom-most plate.

Components of in-plane and out-of-plane loading are accomplished by changing the orientation of the specimen relative to the bottom plate. Thus application of a
simultaneous bi-axis horizontal load is made possible by orienting the specimen at an angle to the applied horizontal load. Uplift load, when applied in conjunction with the horizontal load, produces a simultaneously occurring tri-axis load.

The lateral displacements of the connectors in the two mutually perpendicular directions are measured using two LVDTs and the vertical separation between the top plate and the rafter/ joist is measured using a string pot. The rate of application of load, applied in accordance with the ASTM D 1761 protocol [13], is controlled by adjusting the speed of the screw jacks such that the resultant speed is equal to 2.54 mm/min. (0.1 inch/min.). By altering the speed ratio between the horizontal and vertical screw jacks, various percentages of uplift and lateral loads are applied to the specimen. The data from the load cells, LVDTs and string pot are recorded at a sampling rate of 5 Hz.

The Type 1 and Type 2 connectors are seismic and hurricane ties that are used to fasten rafters/ ceiling joists or trusses to wood top plates. The specimens for testing Types 1 and 2 connectors were fabricated using double 38.1 mm x 139.7 mm (nominal 2 in. x 6 in.) top plates and 38.1mm x 139.7 mm (nominal 2 in. x 6 in.) rafters cut from 4.9 m (16 ft.) long No. 2 or better graded Southern Yellow Pine (SYP) lumber. Both the top plates and the rafter specimens were cut to be 305 mm (24 in.) long. Since the emphasis of the study is on the response of the metal connector, toenails were not used along with metal ties to attach the rafter/ceiling joist to the top plate. The Type 1 and Type 2 connectors were fastened using special galvanized 8d short nails that are 38.1 mm (1.5 in.) long, as given in the manufacturer specification.
The Type 3 connectors are embedded straps with staggered holes and are used to anchor roof trusses to concrete and masonry walls. Type 3 specimens for the current study were made up of triple layered wood top plates and a 38.1 mm x 184.2 mm (nominal 2 in. x 8 in.) bottom truss chord. The straps were connected to the bottom chord using seven - 10d short nails which are 38.1 mm (1.5 in.) long. Generally in the field, Type 3 straps are bent around the top chord of the truss and a minimum of seven nails are used to secure the strap. In the present study, the strap was cut above the 7th hole in order to facilitate the setup of the specimen inside the clamping saddle on the test frame and to represent the realistic installation of the strap. Simpson Strong-Tie High Strength Epoxy Tie (SET) anchoring adhesive was used in lieu of concrete and is used to fill the inside of a deep slot made at the center of the top plate. The strap was inserted into the adhesive
up to the appropriate embedment length (102 mm (4 in.)), as shown in Fig. 3.5, and allowed to cure for at least 24 hours. The strap was nailed to the bottom chord section. The top plate and rafter sections for all connector specimens were made from SYP. The epoxy and wood combination was used in lieu of concrete to facilitate the fabrication of a large number of specimens (> 100) and to expedite the curing time required prior to testing. This decision was based on previous manufacturer tests which indicated that failure in the concrete was not an observed mode of failure for this connector [14].

The test setup ensured that there was no load path other than through the metal connector. However, the setup deviates from an in-field scenario in four main ways – 1) rotation of the rafter is not permitted 2) there is no dead load acting on the connector 3) no toenails exist and 4) no system effect exists (i.e. no load sharing). These deviations are justified since the emphasis of this study is on the capacity of metal connectors under combined loading. Moreover, except for prevention of rotation, all the other deviations are conservative and would represent the use of the connector in a worst-case scenario.

The connectors were tested under pure uplift, pure in-plane (parallel to the top plate), pure out-of-plane (perpendicular to the top plate), combined uplift and in-plane, combined uplift and out-of-plane, combined out-of-plane and in-plane and also a combination of all three loading directions. According to the ASTM D 1761 protocol [13], monotonic loads at a rate of 2.54 mm/min (0.1 inch/min) were applied for unidirectional load cases. For multidirectional loading situations the resultant rate of displacement was also kept at 2.54 mm/min (0.1 inch/min). Different load ratios were achieved by altering the orthogonal displacement rates and also the orientation of the
specimen with respect to the displacement vector. As can be observed in Fig. 3.6, the load directions were chosen such that the nails would be in withdrawal as opposed to the wood members pushing into the connector. This was done in recognition that this would produce the most conservative results. Loads were applied continuously until the
specimen failed – which is defined as the loss of load carrying capacity in any one of the directions.

Table 3.1. Basic load case information.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Type 1</th>
<th></th>
<th>Type 2</th>
<th></th>
<th>Type 3</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sample size</td>
<td>No. of Load Scenarios</td>
<td>Sample size</td>
<td>No. of Load Scenarios</td>
<td>Sample size</td>
<td>No. of Load Scenarios</td>
</tr>
<tr>
<td>Uplift</td>
<td>10</td>
<td>1</td>
<td>10</td>
<td>1</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>Out-of-Plane</td>
<td>10</td>
<td>1</td>
<td>10</td>
<td>1</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>In-Plane</td>
<td>10</td>
<td>1</td>
<td>8</td>
<td>1</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>Up - Out</td>
<td>19</td>
<td>2</td>
<td>7</td>
<td>1</td>
<td>17</td>
<td>2</td>
</tr>
<tr>
<td>Up – In</td>
<td>20</td>
<td>2</td>
<td>20</td>
<td>2</td>
<td>20</td>
<td>5</td>
</tr>
<tr>
<td>Out - In</td>
<td>19</td>
<td>2</td>
<td>20</td>
<td>2</td>
<td>20</td>
<td>2</td>
</tr>
<tr>
<td>Up-Out-In</td>
<td>35</td>
<td>10</td>
<td>41</td>
<td>13</td>
<td>30</td>
<td>9</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>123</strong></td>
<td></td>
<td><strong>116</strong></td>
<td></td>
<td><strong>117</strong></td>
<td></td>
</tr>
</tbody>
</table>

For the sake of brevity, a full description of the specimen orientations and displacement rates in the three primary axes is omitted here but can be found in [15]. However, a basic presentation of the number of samples and the load cases considered is given in Table 3.1. For a given load case (e.g. uplift – out-of-plane abbreviated as Up-Out), a number of different load rates were considered. For example, in the Up-Out case of the Type 1 connector, two different displacement scenarios were explored. The first scenario imposed a 0.76 mm/min (0.030 in/min) in the Up direction while imposing a 2.42 mm/in (0.095 in/min) in the Out direction. The second scenario imposed a 0.36
mm/min (0.014 in/min) in the Up direction while imposing a 2.51 mm/in (0.099 in/min) in the Out direction. These displacement rates were based on relative stiffnesses between the directions being tested – identified using preliminary test results. Simple observation would indicate that the Type 1 stiffness in the up direction is much higher than in the out-of-plane direction – hence the significant differences in the displacement rates. Table 3.1 simply presents the number of scenarios considered for each load case. Ten specimens per scenario was the target for uni-axis and bi-axis load cases. This dropped to approximately three specimens per scenario for the tri-axis case so that more of the interaction space could be explored.

3.4 RESULTS

Since the objective of this paper is to provide a discussion concerning the multi-axis behavior of metal connectors from a design perspective, the results are presented in the form of design based capacities. The design capacity for a given connector was estimated based on the first two conditions of the AC13 acceptance criteria [1]. In order to ensure that the third condition of the acceptance criteria was not the controlling factor, approximate NDS [2] recommended values for nails attaching the metal ties were calculated. Since the NDS prescribed values are for a single failure mode, the capacity of the fasteners attaching the hurricane ties in both withdrawal and shear failure modes were calculated. The nails attaching the Type 1 and Type 2 hurricane ties to ceiling joists/rafters and top plates are in different planes. As such, when one set of nails is subjected to a withdrawal force the other set of nails is subjected to a shear force. This provides justification for treating each failure mode individually. A detailed analysis
concluded that the approximate NDS values obtained for fasteners attaching the Type 1, Type 2 and Type 3 metal connectors were not controlling – indicating that the experimentally determined values should be used for design capacities. The criteria used for determining allowable capacities for all connectors subject to both uni-axis and multi-axis loading in this study are:

a) The loads associated with the lowest ultimate load that the connection can withstand in any of the orthogonal directions divided by an appropriate FOS for the given failure mode. (force controlled).

b) The force value corresponding to a 3.2 mm (0.125 in) resultant displacement of the connector (displacement controlled).

The FOS for unidirectional loading is controlled by the mode of failure. For failure of connecting wood members or nail fasteners, a FOS of 3 is used as per the acceptance criteria [1]. Nail withdrawal, top plate split, and rafter split are a few examples of such a type of failure. A FOS of 2 is used when the failure mode is exclusively tearing of a flat strap. This type of failure has a lower degree of variability which is the rationale for using the lower FOS. For example, all Type 3 specimens subjected to uni-axis uplift loading exhibited a flat strap tear failure mode. This was the only metal connector and load case to show such behavior and as such is the only case that used a FOS of 2. All other cases used the FOS of 3.

One should note that a connection subjected to combined loading may be controlled by displacement criteria in one direction and force criteria in another direction. Furthermore, the connection may not reach a 3.2 mm (0.125 in) displacement or its peak
capacity in both directions at the same time. In such situations, the direction in which any one of the criteria is first met is considered to be the controlling direction.

3.4.1 Connector – Type 1

The Type 1 hurricane ties are galvanized steel rectangular plates with a slot at the top edge that holds the rafter/joist or truss. The metal ties need 8-8d nails to attach to the rafter/ joist and 8-8d nails to the top and bottom plates. The 8d nails are special short length nails and are used in place of common 8d nails since regular 8d nails are too long for the rafter. The ties exhibit significant difference in stiffness under parallel-to-plate (in-plane) and perpendicular-to-plate (out-of-plane) loadings.

A total of 123 specimens were tested for the Type 1 connector. Ten specimens were used for each of the primary axes. The allowable uplift value is 3.97 kN (893 lbs) with a coefficient of variation (COV) of 0.13. This value is force controlled with observed modes of failure including steel tear (Fig. 3.6a) and splitting in the top wood plate. Summary statistics for the uni-axis loadings are given in Table 3.2. The table indicates the controlling factor for the presented design values – force or displacement. The largest uni-axis design value is seen in the uplift direction where force controls. The smallest design force – 0.94 kN (211 lbs) – occurs in the out-of-plane direction where displacement controls. For all three connector types, relatively small COVs (i.e < 0.15) are observed for the force controlled values while a significant increase in variability (i.e. 0.28 ≤ COV ≤ 0.53) is observed for displacement controlled values. This phenomenon is likely due to the fact that displacement values are more sensitive to construction variabilities – including nail seating.
The NDS values for this connector are presented here as evidence that they do not control the design value. The uplift capacity of the fasteners connecting the Type 1 hurricane ties using the NDS equations is 5.88 kN (1321 lbs). The in-plane NDS capacity of nails in the top plate is 5.88 kN (1321 lbs) and the out-of-plane capacity of nails in the top plate is 3.35 kN (752 lbs) (withdrawal mode). These values are clearly well above the values given in Table 3.2 and as such do not control as per AC13.

**Table 3.2.** Uni-axial design load statistics.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Type 1</th>
<th></th>
<th>Type 2</th>
<th></th>
<th>Type 3</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean kN (lbs)</td>
<td>COV</td>
<td>Mean kN (lbs)</td>
<td>COV</td>
<td>Mean kN (lbs)</td>
<td>COV</td>
</tr>
<tr>
<td>Uplift</td>
<td>3.97 (893)**</td>
<td>0.13</td>
<td>2.31 (520)**</td>
<td>0.11</td>
<td>6.84 (1538)**</td>
<td>0.01</td>
</tr>
<tr>
<td>Out-of-Plane</td>
<td>0.94 (211)*</td>
<td>0.28</td>
<td>0.62 (139)*</td>
<td>0.35</td>
<td>1.96 (440)**</td>
<td>0.11</td>
</tr>
<tr>
<td>In-Plane</td>
<td>2.42 (543)**</td>
<td>0.14</td>
<td>0.77 (173)*</td>
<td>0.39</td>
<td>0.95 (213)*</td>
<td>0.53</td>
</tr>
</tbody>
</table>

* Displacement controlled values  
** Force controlled values

An example of the resulting force-displacement curves for one of the tri-axis load cases is presented in Fig. 3.7a. One may observe that an off-loading occurs in all three axes but they occur at different rates and at different times. The initial stiffness of the Type 1 connector in the vertical direction is very high as is seen in the near vertical section between 0 and 3.6 kN (800 lbs). However, as steel tearing or wood plate splitting commences, the stiffness begins to drop until the connector reaches its ultimate load. This high initial stiffness is not exhibited in the horizontal directions since the out-of-
plane direction the connector is behaving as a flat plate subject to bending while the in-plane direction is prone to localized buckling (Fig. 3.6d). One should also note that the ultimate load for each orthogonal direction is not reached at the same displacement (Fig. 3.7a) or the same time (Fig. 3.7b).

Fig. 3.7b helps to illustrate how design values for a given specimen were determined. For example, when the connection underwent a resultant displacement of 3.2 mm (0.125 in), the forces in each of the three orthogonal directions were identified. Next, the ultimate force values were taken at the instant in time when force shedding is observed in any one of the primary axes. In this case, the time associated with the peak load in the parallel-to-plate direction is used for defining the ultimate load. The ultimate loads are then divided by three and compared with the values associated with displacement. The smallest set of values for each direction is selected as the design values. For the specimen indicated, the design forces are considered to be force controlled because for all three directions the ultimate force divided by three is less than the displacement-based forces. (e.g. 7.70 kN/3.0 = 2.57 kN < 3.45 kN  ➔ force controlled)

A number of different failure modes were observed throughout the various load cases. Failure modes for this connector include steel tearing, nail withdrawal, wood splitting, buckling or a combination thereof. The types of failure modes seen depend on the type of load being applied. For example, the failure modes in Figs. 3.6(a) and 3.6(b) occurred under an uplift load. The nail withdrawal and buckling occurred under a load parallel to the top plate while the combined failure mode occurred for loading in both horizontal directions.
Presenting summary statistics for all load cases in all orthogonal directions for all three connectors is quite lengthy. Therefore, for the sake of brevity, these statistics are not presented herein for the multi-axis load cases. Rather a treatment of the multi-axis behavior of these connectors is presented later in this paper when considering appropriate design spaces.

3.4.2 Connector – Type 2

Type 2 metal connectors are twisted straps with two legs and are used to attach rafter/ceiling joists or trusses to top plates. Type 2 ties require five-8d nails to fasten to the top plates (three nails into upper plate and two nails into lower plate) and five-8d nails to attach to the rafter/joist. The 8d nails are special and are same as the ones used for Type 1 ties. The legs of the Type 2 are orthogonal to one another and theoretically possess the same stiffness. The uplift capacity of the fasteners connecting the Type 2 hurricane ties using the NDS equations is computed as 7.45 kN (1674 lbs). The NDS withdrawal load (in-plane/out-of-plane) of five nails is computed as 2.10 kN (471 lbs) while the same nails considered in shear have an NDS value of 3.72 kN (837 lbs). The summary statistics presented in Table 3.2 indicate that the design values for the Type 2 are not governed by the NDS values. The uplift capacity is force controlled with a value of 2.31 kN (530 lbs) and a COV of 0.11. The horizontal capacities are less than a third of the uplift and are controlled by their displacements having COVs greater than or equal to 0.35. The multi-axis behavior is discussed later in this paper.

Due to space limitations, force displacement plots for this connector are not presented. However, their generalized behavior is similar to those presented in Fig. 3.7
except that they do not exhibit the same type of high initial vertical stiffness as is seen for the Type 1. This is because vertical deformation of the Type 2 connector is achieved through twisting of the strap (illustrated in Figs. 3.6(b) and 3.6(c)) unlike the predominant axial action expected in the Type 1 connector. This connector exhibited great ductility while undergoing significant deformation. The failure modes identified for this connector include strap tear, nail withdrawal, wood splitting and a combination thereof.

3.4.3 Connector – Type 3

The Type 3 is an embedded metal strap connector that is used to attach rafters or trusses to masonry/concrete walls. The strap is bent at one of its ends to facilitate anchorage inside the concrete or masonry. It is connected to the rafter using 10d short length nails. The depth of embedment of the strap into the base material is specified as 102 mm (4 in) by the manufacturer. The specimens were constructed using epoxy instead of concrete to expedite the manufacturing of the specimens. A previous study
found that pullout from the concrete was not a failure mode that needed to be considered because the anchorage strength far exceeded the tensile strength of the strap [14]. Although, these straps were embedded in epoxy instead of concrete for this study, this was not seen to affect the uplift capacity since strap tear was always found to be the mode of failure – not pullout. In the present study, the strap was cut above the 7th nail hole so that it would not interfere with the clamping saddle. Since the 7th nail (upper-most) as seen in Fig. 3.6f never showed signs of distress during any of the tests, cutting of the straps did not cause premature failure of the specimen.

Since all the Type 3 specimens subjected to uni-directional uplift load failed due to strap tear, the theoretical capacity is obtained from the rupture strength of the strap. Using an ultimate strength of 65 ksi, the tensile rupture strength is found to be 6.45 kN (1450 lbs) -- practically the same as found through experimentation. Applying the NDS procedure, the in-plane and out-of-plane capacities of the fasteners are found to be 6.37 kN (1431 lbs) and 3.31 kN (745 lbs) respectively. Experiments demonstrated, for the same loading directions, the design capacities of 0.95 kN (213 lbs) and 1.96 kN (440 lbs) respectively illustrating once again that NDS values do not control for this connector. The COV for the force-controlled values is around 0.14 while the displacement-controlled values have a COV of 0.28.

Unlike the other two connectors, the Type 3 connector did not maintain any load capacity once it reached its ultimate uplift load. This is because the strap would tear at this peak load as a result of its simple cross-sectional configuration. Whereas, even if the other straps tore, it was not before significant twisting, buckling or shear deformation.
occurred. The failure modes identified for this connector include strap tear, nail withdrawal, buckling and a combination thereof. Figs. 3.6e and 3.6f illustrate a few of these failure modes including a combination mode. When the load was applied parallel to the plate, nail withdrawal was the primary mode of failure. This strap could sustain large displacements in this direction because as the nails withdrew, the strap would switch from providing resistance through shear and bending to providing resistance through axial action. It is clear that allowable displacements are the limiting factor for this loading direction.

3.4.4 Multi-Axis Interaction/Design

One of the objectives of this study is to better understand the interaction that occurs between the primary orthogonal directions of the connectors under multi-axis loads. To facilitate this analysis and permit comparison between the connector types all of the computed design forces were normalized by their related uni-axis design values given in Table 3.2. For example, all of the uplift forces associated with the Type 1 connector were divided by 3.97 kN (893 lbs). Thus a value greater than 1.0 indicates that the actual design capacity for that loading scenario exceeds the uni-axis design value.

A sense of the interaction between design loads in each loading direction can be obtained by examining the correlation coefficient for each load case. One must keep in mind that a correlation coefficient is a measure of the linear dependence between two variables. So while a low coefficient may indicate a lack of interaction, this is not necessarily always the case as it may just be an indication that the interaction is not a linear function. Therefore, the correlation coefficient is only used in this paper as an
indication of trend indicating whether there is a positive relationship or a negative relationship between design values. Table 3.3 gives these correlation values for all three connector types. There is no clear pattern in the values of the correlation coefficient. However, there is an interesting phenomenon that must be pointed out. For the load cases where the design values are determined by the displacement criterion it was observed that a positive correlation exists. This means that when a connector’s displacement controlled capacity increases in a given direction, a capacity increase is also seen in the other direction. The opposite is found to be true for those load cases whose design values are controlled by the force criterion. Indeed, for these cases a negative correlation exists indicating that increased capacities in one direction are accompanied by a decrease in the other direction.

**Design Space**

For devising an appropriate design equation for multi-axis loads, one would ideally fit an equation or interaction surface to the experimental data. However, due to the lack of consistent patterns in the data and also the need to consider practical application of the design equation to different connector classes, generic types of interaction/design spaces are explored and evaluated. This evaluation utilizes the normalized experimental design data to assess appropriateness of these design spaces and to make recommendations for practical implementation.

A generic interaction surface for tri-axis loads can generally be expressed in the form of Equation 3.2.
where, Design load$_{1}$ or 2 or 3 – Design load in direction 1, 2 and 3 respectively

Allowable load$_{1}$ or 2 or 3 – Allowable load in direction 1, 2 and 3 respectively

Factor$_{1}$ or 2 or 3, Power$_{1}$ or 2 or 3 – define the relationship between the load ratios.

### Table 3.3. Correlation coefficients between design forces in each of the primary connector axes.

<table>
<thead>
<tr>
<th>Directions – Bi-axis</th>
<th>Correlation coefficient for design forces</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type 1</td>
</tr>
<tr>
<td>Up – Out-of-Plane</td>
<td>0.47</td>
</tr>
<tr>
<td>Up – In-Plane</td>
<td>-0.73</td>
</tr>
<tr>
<td>Out-of-Plane – In-Plane</td>
<td>-0.75</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Directions – Tri-axis</th>
<th>Correlation coefficient for design forces</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type 1</td>
</tr>
<tr>
<td>Up – Out-of-Plane</td>
<td>0.52</td>
</tr>
<tr>
<td>Up – In-Plane</td>
<td>-0.35</td>
</tr>
<tr>
<td>Out-of-Plane – In-Plane</td>
<td>-0.67</td>
</tr>
</tbody>
</table>

The NDS accounts for combined loading on timber beams and columns (short) by a linear interaction equation of normalized stress values. The premise for this linear interaction equation is that the resultant stresses due to combined loads act in one single direction and the ensuing effect is simply the addition or subtraction of the stresses. The same proposition, however, is in question when applied to combined loading on metal fasteners and is investigated as part of this study since the linear interaction of Equation
3.1 (i.e. $Power_i = 1$ and $Factor_i = 1$) represents the state-of-the-practice. Indeed, there has existed an initial sense that this interaction equation is overly conservative but no sufficient research data has been available to confirm this notion. The present study is an effort to verify or improve upon this assumption of linearity. An improvement would come in the form of an interaction description that would result in an increase of the allowable design space of the connector.

Four types of interaction or design surfaces, most using the form of Equation 3.2, were examined for both bi-axis and tri-axis loading scenarios. The first type explored was the currently used linear (1st Order) relationship as given in Equation 3.1 and graphically represented in Fig. 3.8a. The second interaction surface is a nonlinear (2nd Order) surface where $Power_i = 2$ and $Factor_i = 1$ as given in Equation 3.2 (Fig. 3.8b). The third interaction surface looks like a cuboid and simply applies a reduction factor to the allowable loads when multi-axis loads are experienced (Fig. 3.8c). This cuboid is described by Equation 3.3

\[
\frac{\text{Req'd uplift}}{\text{Allowable uplift}} \leq RF
\]
\[
\frac{\text{Req'd parallel to plate}}{\text{Allowable parallel to plate}} \leq RF
\]
\[
\frac{\text{Req'd perpendicular to plate}}{\text{Allowable perpendicular to plate}} \leq RF
\]

where, RF is a reduction factor imposed upon the allowable loads to account for any interaction effects. A reduction factor of 0.60, 0.70, 0.75, 0.80 and 0.90 were explored
for this study but only the findings for the values between 0.70 and 0.80 are presented for the sake of brevity. A fourth design space scenario is explored which would permit the use of either the cuboid or the 1st Order unity equation (left to the discretion of the designer). This would effectively create a design space as shown in Fig. 3.8d. Since all design scenarios have the same ease of application by a designer, the question that needs to be addressed is which of the investigated spaces would provide for the most efficient (i.e. most usable design space) yet safe approach.

**Figure 3.8.** Possible Design Spaces Considered (a) Current – 1st Order (b) 2nd Order (c) Cuboid (d) Combination 1st Order – Cuboid.
To aid in the identification of an appropriate design equation, the design space ratio for each scenario is computed. The design space ratio represents the amount of design space allowed compared with the full design space denoted in Fig. 3.8 as the full design boundary (i.e. no interaction and no reduction of uni-axis design values). For example, the full design space is calculated as the volume of a cube having leg lengths of 1.0. This volume is computed as \((1.0)(1.0)(1.0) = 1.0\). If a reduction factor of 0.70 is selected (i.e. a cuboid) then the design space is \((0.7)(0.7)(0.7) = 0.34\) and the design space ratio is calculated as \(0.34/1.0 = 0.34\). All of the investigated design spaces are compared with the full design boundary to get the ratios presented in Table 3.4 keeping in mind that higher ratios are indicative of more efficient design spaces. The most efficient scenario is the 2\(^{nd}\) Order having ratios for the bi-axis and tri-axis cases being 0.79 and 0.52 respectively. The next most efficient design space is the cuboid with a reduction factor of 0.8 followed closely by the composite space of Fig. 3.8d. The least efficient and hence least desired space is the currently used 1\(^{st}\) Order scenario of Equation 3.1.

Table 3.4. Usable design space ratio for different design surfaces.

<table>
<thead>
<tr>
<th>Design Scenario</th>
<th>Bi-axis</th>
<th>Tri-axis</th>
</tr>
</thead>
<tbody>
<tr>
<td>70 %</td>
<td>0.49</td>
<td>0.34</td>
</tr>
<tr>
<td>75 %</td>
<td>0.56</td>
<td>0.42</td>
</tr>
<tr>
<td>80 %</td>
<td>0.64</td>
<td>0.51</td>
</tr>
<tr>
<td>75 % &amp; 1(^{st}) Order</td>
<td>0.63</td>
<td>0.43</td>
</tr>
<tr>
<td>1(^{st}) Order</td>
<td>0.50</td>
<td>0.17</td>
</tr>
<tr>
<td>2(^{nd}) Order</td>
<td>0.79</td>
<td>0.52</td>
</tr>
</tbody>
</table>
Fig. 3.9 shows the actual normalized data collected from this study and compares it to the composite design space. The data from all three connectors are represented on the plot where only bi-axis load cases are given in Figs. 3.9(a-c) and only the tri-axis load cases are given in Fig. 3.9d. The markers indicate how the plotted design values were derived – whether by force, by displacement or a combination thereof. Seeing that most of the experimental data falls outside of this design space would lend one to consider this as an acceptable design space. However, a more rigorous evaluation of the data is carried out which is discussed hereafter.

To further evaluate the appropriateness of the selected design spaces an investigation of several other criteria was conducted. The first criterion is a measure of whether the design equation would provide a design level consistent with the principle demonstrated by AC13. This principle states that if six or more tests are conducted that the design value should be taken as the mean of the design values for each individual test. This means that if the test data is symmetrically distributed (e.g. normally distributed) about the mean then 50 percent of the test data falls below the selected design value and 50 percent lies above. If it is not symmetrically distributed, as was seen for some of the uni-axis tests in this study, then one may see anywhere between 40 and 60 percent of the test data falling below the selected design value. Since this is an acceptable level of performance as per AC13 [1] it is utilized in this study. Specifically, the above criterion is measured as the probability that an actual measured design capacity for a connector is less than the design value proposed by the design surface where an acceptable level of performance is around 50 to 60 percent.
Figure 3.9. Experimental design values for all three connector types with respect to the combination design space shown in Fig. 3.8d. (a) Up-Out  (b) Up-In (c) Out-In (d) Up-Out-In.

The second criterion is one concerned with safety and not with performance. For this criterion only strength capacities are examined. This looks at the probability that the actual measured strength capacity of a connector is lower than the design strength proposed by the design equation. When safety is a concern only low probabilities can be tolerated. The acceptance threshold for this criterion is taken as a probability of $1.3(10)^{-3}$. 
This probability level corresponds to a reliability index of 3.0. However, one must recognize that this evaluation of capacity is only a consideration of half of the reliability problem and that the selected threshold is within acceptable limits [16]. When one considers load in the formulation then the reliability index will increase (probability decrease). For this reason this threshold of 1.3 \((10)^{3}\) is deemed appropriate for use in this evaluation. Furthermore, the approach for selecting design values in this study is rooted in the concepts of the allowable stress design (ASD) philosophy. The probability estimates provided herein are simply performed to demonstrate AC13 equivalence and adequate safety – not to establish a different design methodology.

Not enough data was collected to be able to estimate the probabilities needed for the evaluation of the above-mentioned criteria directly. Therefore, the collected data was fit to jointly normal distributions thus allowing for calculation of the requisite probabilities. When necessary, the data was transformed using either a logarithmic or Johnson transformation prior to fitting the distribution. This ensured that a jointly normal distribution was an appropriate model for the data. The quality of the distribution fits where evaluated using the Anderson-Darling goodness-of-fit test [17]. Of the 72 probability distributions that were estimated, all but four were deemed as plausible at the 5% acceptance level and the remaining four were deemed plausible at the 1% level. Overall the distribution fits are seen as appropriate and are used to calculate the necessary probabilities using a Monte Carlo simulation approach. The simulations utilized a sample size of \(1(10)^{6}\) which makes the precision of the estimate to be \(1(10)^{-6}\).
The first criterion looking at the evaluation of the design space was carried out and the calculated probabilities are given in Table 3.5. Recalling that the acceptance threshold is set around 50-60 percent, one clearly sees that the 2nd Order design scenario is unacceptable having probabilities as high as 77 and 95 percent for the Type 1 and Type 3 connectors respectively. The 75%-1st Order and the 80% scenarios are considered
acceptable, even for the Type 1 connector, which has probabilities as high as 62 and 63 percent respectively for the in-plane – out-of-plane loading. This assessment is because of the perceived low probability associated with the real-world existence of this load case in any practical structure subjected to wind loads. It is interesting to note however that the Type 2 behavior falls well within the acceptance criteria for all proposed scenarios.

Table 3.6. Probability of actual connector strength based on tested specimens falling below the proposed design surfaces.

<table>
<thead>
<tr>
<th>Connector</th>
<th>70 %</th>
<th>75 %</th>
<th>80 %</th>
<th>75% - 1st Order</th>
<th>1st Order</th>
<th>2nd Order</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Uplift – Out-of-Plane</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type 1</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>Type 2</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>Type 3</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td><strong>Uplift – In-Plane</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type 1</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>Type 2</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>Type 3</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td><strong>Out-of-Plane – In-Plane</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type 1</td>
<td>3.80e-5</td>
<td>9.80e-5</td>
<td>1.95e-4</td>
<td>7.00e-6</td>
<td>2.38e-4</td>
<td>9.80e-5</td>
</tr>
<tr>
<td>Type 2</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>Type 3</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td><strong>Tri-Axis</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type 1</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>Type 2</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>Type 3</td>
<td>2.56e-4</td>
<td>5.47e-4</td>
<td>1.02e-3</td>
<td>5.47e-4</td>
<td>4.00e-6</td>
<td>3.01e-4</td>
</tr>
</tbody>
</table>

** Probability below 1e-6
The second criterion, which is an evaluation of safety, is examined through the probabilities given in Table 3.6. The only borderline behavior was found for the Type 3 under tri-axis loading for the 80% scenario with a probability of $1.02(10)^{-3}$. Recall that this probability states the likelihood that the actual strength of a connector falls below the design strength assumed by the 80% cuboid. Further note that the large majority of probabilities are below the value of $1(10)^{-6}$ indicating a high level of safety. This is particularly true for the Type 2 where all probabilities fall at or below this level.

3.5 CONCLUSIONS/RECOMMENDATIONS

Until now the response of metal connectors subjected to multi-directional loading has been addressed using a linear interaction equation. Assumed to be conservative, with no research studies to support or contradict, this linear interaction equation is representative of the state-of-the-practice. The present study is an effort to validate the current interaction surface and provide guidance for a new and more efficient failure surface. Under the present research study three types of metal connectors, each having distinctly different characteristics, were subjected to uni-axis, bi-axis and tri-axis loads. The normalized design capacities and strength capacities were evaluated against different proposed design surfaces. These design surfaces include a 1$^{st}$ Order surface, a 2$^{nd}$ Order surface, several cuboid surfaces and a composite surface of a 1$^{st}$ Order and a cuboid. Using a Monte Carlo simulation approach, probabilities for design (performance) and safety (strength) were evaluated against principles extrapolated from the AC13 document [1]. From this evaluation the following conclusions and recommendations are given.
3.5.1 Conclusions

a) There appears to be no readily parameterized interaction equation which describes the actual experimental data collected (Fig. 3.9)

b) A generalized design/interaction surface appears to be most appropriate for application to a wide range of metal connector types

c) The unity equation (Equation 3.1) currently used in practice is considered to be inefficient and overly conservative based on the analyses of this study.

d) This study only examined three connector types. However, these connectors were selected with great care to ensure they are representative of general classes of connectors. As such, the findings of this study are believed to be appropriate for all connectors of similar types although admittedly this hypothesis can only be verified through additional research.

e) If the allowable design loads of a connector are controlled by the NDS values and not the experimental values then the proposed design surface will be even more conservative. This is because the NDS values are essentially imposing a reduction beyond what is indicated by the experimental data and thus the probabilities that the actual test data falls below this reduced value drop. As such, the proposed design surface is seen as appropriate for use with NDS controlled design values.
3.5.2 **Recommendations**

a) The composite design space given in Fig. 3.9 is proposed for all three connector types based on both efficiency and performance. The 75 percent – 1st Order threshold is considered to be valid for both bi-axis and tri-axis load cases for all three connector types. Although one load case for the Type 1 connector warranted a closer look from a performance perspective (62%) it is well within the safety based acceptance criteria (0.055%). Furthermore, since no real guidance exists regarding acceptable displacements for performance measurement of multi-axis loads, this study assumed the most severe criterion of 3.2 mm (0.125 in) resultant displacement. In addition to the fact that this one load case is not very probable under in-field conditions, the levels conservatism in the performance and in the safety lead one to consider that the 62% is acceptable.

b) The proposed design space may be communicated to the designer using text similar to the following: “As an alternate to the linear interaction equation, allowable simultaneous loads in more than one direction for Type 1, Type 2 and Type 3 connectors may be evaluated as follows: For each of the simultaneous load directions, the Design Load in the direction being evaluated shall be no more than 75% of the Allowable Load in that direction.”

c) Throughout this study a set of acceptance criteria rooted in the concepts of allowable stress design, which was developed for floor joist connectors, has been used. However, it has become clear that a set of acceptance criteria, including specimen design, load application and displacement measurement, should be
developed for metal roof-to-wall connectors. Specifically, this set of criteria should address cyclic loading, multi-axis loads, field installation and behavior of the connector, and should be developed around the concepts of a reliability-based design.

The findings of the present study are crucial as they help to properly understand the resistance characteristics of metal connectors subjected to combined loading. Realizing that the capacity of the metal connector is reduced when subjected to multi-directional loading will lead to safe design of structures while not submitting to over-conservatism. The proposed failure surface is easy to implement as it involves no tedious calculation and is easily communicated to designers. Furthermore, the recommended design space is considered valid for all three classes of connectors.

3.6 ACKNOWLEDGEMENTS

The authors would like to thank Simpson Strong-Tie for their financial support of this study. In particular we appreciate the guidance of Mr. Randy Shackelford in helping to define the parameters of this work and providing previous test data. However, the views expressed in this paper do not necessarily represent the views of Simpson Strong-Tie.

3.7 REFERENCES


4. INFLUENCE OF TYPICAL MODELING PARAMETERS ON WIND BASED FRAGILITY ESTIMATES OF LIGHT FRAMED WOOD ROOF STRUCTURES

4.1 ABSTRACT

Roof-to-wall connections and sheathing fasteners for light frame residential structures are typically designed based on a conservative assumption that they function independent of each other. However load distribution and redistribution among connectors is widely contemplated by researchers. Also speculated is that the relative stiffness between the connectors influences the degree of distribution/ redistribution of loads. Understanding the above phenomenon is important in predicting the failure of the roof system when subjected to wind uplift loads. Furthermore, estimation of wind based fragility curves for residential roof system may be significantly influenced by the effect of relative stiffness on load distribution/ redistribution among connectors.

Using a finite element and simulation based approach, this study looks to identify the influence that individual connector stiffnesses have on overall roof system performance. Through the use of nonlinear springs, the influence of connector modeling parameters can be explored in a probabilistic fashion. Specifically, the governing parameters of the nonlinear spring element are the initial stiffness, peak withdrawal capacity and displacement at peak withdrawal capacity of the connectors. Assessment of the overall impact of such parameters is achieved by developing wind-uplift fragility curves for the roof system using a Latin-hypercube based simulation strategy and then performing direct comparisons.
The effects of roof to wall connector and sheathing fastener initial stiffness and post ultimate negative stiffness on the roof fragility estimation is investigated. In addition the effect of sheathing thickness, nail spacing and gable end supports on the fragility estimate is also explored. Assigning same and different stiffness for roof-to-wall connectors and sheathing fasteners had little to no effect on the distribution pattern of wind uplift load among connectors. However failure to consider the off-loading capability of the connectors had a significant impact on the resulting failure of the roof system.

4.2 INTRODUCTION

The millions of dollars spent in rebuilding low-rise wood structures in the aftermath of past hurricanes has clearly identified the need for performance based design (PBD) of light framed wood residential structures (LFWRS). PBD is a design methodology that identifies various limit states like life safety, comfort level and structural integrity and designs the buildings according to the specified need. A thorough understanding of the nature of demand (wind and dead load) and resistance (behavior of components like roof sheathing, connections and framing members) on the structure is fundamental for an efficient implementation of the PBD philosophy [1].

In an effort to understand the demand on LFWRS during an extreme wind event, wind tunnel studies on models of light frame wood residential homes have been carried out by a number of researchers [2, 3]. Various statistical models were proposed to describe the wind pressure distribution over a low rise structures including probability distribution parameters governing various wind load factors [4, 5]. Additionally, the force displacement behaviors of various components of LFWRS have also been studied.
to better comprehend the resistant nature of the structure [6-11]. Others have used these
derived load and resistance statistics to examine the fragility and reliability of wood roof
systems subjected to high winds [12-22]. Such fragility and reliability based
performance assessments can become an integral part of performance based design of
LFWRS.

Roof damage has always been a major cause of failure of LFWRS [23]. In particular,
connections within the roof system have been shown to be the weakest link [24] and have
been the ultimate cause of observed failures. Because of their significant role in roof
performance components like roof to wall connectors (RTWC) and sheathing fasteners
(SF) are often the focus of analytically based roof system wind uplift fragility estimates
and the results from many experimental studies have been used to study the uplift
capacity including studies on the withdrawal behavior of nailed connections. The
influence of various factors on the peak withdrawal capacity was also studied. [6, 10, 25-
32]. Component fragility curves for various RTWC like toenails, hurricane straps and
metal ties have been derived using the resistance statistics obtained from multiple lab
tests [13, 14, 33, 34]. A few researchers have also investigated a group of toenail
connections to explore the system effect along with identifying the proper load
application mechanism [8].

In dealing with the system effect of a framed roof, one study considered a serial type
of failure to appropriately describe the failure sequence of a system of RTWCs when
system fragilities are analytically developed [19]. This assumption assumes that a single
roof to wall (RTW) connection failure will induce failure in the rest of the connections in
the system, causing the entire roof to detach from the walls. However, this argument fails to acknowledge the degree of redistribution of load after a connection fails and thus offers only a conservative estimate of the roof system fragility. Furthermore, typical analytical modeling neglects the effects of composite action due to the presence of sheathing panels and framing members.

Sheathing panel uplift failure is another major failure mode of light frame roof systems [23]. It has been shown that the loss of a few roof sheathing panels is enough to cause considerable rain induced damages to a given building. One instance showed that the cost of repair nearly equaled 80 percent of the total cost of the building [35]. Sheathing panel capacities have either been experimentally determined from pressure tests or analytically calculated using individual sheathing fastener withdrawal test data [7, 9, 36, 37]. Failure of sheathing was sometimes assumed to be initiated by the weakest nail in a roof panel which causes a zipper-like effect leading to entire panel uplift failure. Static and dynamic pressures on sheathing panels were used to determine the panel capacity, failure pattern and to identify critical nails [7, 36]. Even though two types of sheathing nail failures- Pull out and pull through- have been observed nail pull-out was considered as the main cause of failure of sheathing panel [36]. This pull out (withdrawal) behavior can be analytically modeled using a non-linear or linear spring with force displacement behavior closely approximating laboratory test data. In a recent study, finite element model of a sheathing panel utilizing the nail spring model was used to analytically determine the panel capacity [38]. In another study, resistance statistics obtained from lab tests have been used in finite element based reliability model to
calculate reliability indices of sheathing panels located at various zones within a roof [22]. While the first study closely captures the behavior of an individual sheathing panel subjected to wind uplift, it fails to consider the sheathing as a part of a roof system. The second study, on the other hand, estimates the roof system reliability but using a crude approximation of nail withdrawal behavior. In another significant study, a nail element model that has coupled withdrawal-moment behavior was used to estimate the roof sheathing fragility under various performance expectations [39, 40]. Though coupling withdrawal and moment behavior of a nail element helps in accurately projecting the sheathing displacement, it is of less significance when the capacity of the sheathing panel is the main concern. The reason is only edge nails are prone to rotation and the critical nails that govern the panel capacity are interior sheathing nails, i.e., not edge nails. [7].

The present study is a finite element based effort to provide better understanding of various modeling issues which may arise in the wind fragility estimation process of a roof system. This work looks to validate various assumptions made on the RTWC and SF behaviors used during previous studies. It also evaluates the sensitivity of the fragility estimates to various modeling issues. The credibility of the serial failure assumption for systems of roof to wall connection and roof sheathing are investigated. The sensitivity of fragility curves to two different nail models - one with an offloading behavior after reaching the peak capacity and the other with no negative stiffness after the peak capacity- is also investigated along with the effects of relative connection stiffnesses. This study not only improves understanding of the wind fragility of wood roof systems
but it also provides a more solid understanding of wood roof behavior which is needed for effective implementation of the PBD philosophy.

4.3 **FINITE ELEMENT MODEL**

To facilitate the current study, the geometry and construction details for a typical baseline structure were taken from a single story residential duplex apartment located in Clemson, South Carolina (Fig. 4.1). The structure was slated for demolition and as such presented a prime opportunity for collecting much needed in-situ RTWC behavior information. A field investigation of systems of roof to wall connections was conducted and an analytical model for the force-displacement behavior of the toenail roof to wall fastenings has been proposed. In addition probability distributions governing the uplift capacity, stiffness and displacement at peak withdrawal capacity were recommended [10]. This wealth of information along with the simplicity of the roof system made this selected structural configuration ideal for the current sensitivity study.

A finite element model (Fig. 4.2) of part of the roof system of the reference structure was developed using ANSYS [41]. The modeled gable ended roof system has an overall width of 7.32 m (288 in.) and a span of 9.27 m (365 in.). It is stick built using 38 x 140 mm (2 x 6 in. - nominal) rafters and 38 x 89 mm (2 x 4 in. nominal) ceiling joists. Thirteen pairs of rafters with a pitch of 5:12 and spaced at 0.64 m (24 in.) on center (o.c.) constituted the entire roof system. The ceiling joist acts as a bottom tie and resists the outward thrust at the ends of the rafter. The rafters were tied together by a 38 x 184 mm (2 x 8 in. - nominal) ridge board at the center of the roof and were attached to the wall top plates, on either side of the roof, by two 4.1 mm (0.161 in.) diameter by 89 mm (3.5 in.)
long smooth shank 16-d common nails toenails driven one on each side of the ceiling joist. Plywood panels of 11.9 mm (15/32 in.) thickness and of size 1.22 x 2.44 m (4 x 8 ft.) were used to cover the roof. The sheathing panels were nailed to the rafters using 3.3 mm (0.131 in.) diameter by 63.5 mm (2.5 in.) long smooth shank 8-d common nails spaced at 15.2 cm (6 in) at the edge and 30.5 cm (12 in.) in the field, over the entire roof as shown in Fig 4.3. All the roof framing members were modeled assuming the material properties for Southern Yellow Pine.

In the finite element replica of the roof system, ceiling joists and rafters were modeled using the linear-elastic Beam4 elements. Beam 44 elements were used at the rafter ends to facilitate the release of end moments, imitating the actual rafter end connections in the roof system.
Figure 4.2. Rendering of the finite element model of the baseline roof system.

The plywood sheathing panels were modeled using the linear-elastic shell63 elements. Shell63 elements are capable of out-of-plane bending and membrane elongations (large deformation capability). Even though plywood sheathing is made up of different strands of wood, the sheathing elements were modeled as a single layered homogeneous isotropic element with a deterministic longitudinal E value in order to simplify the analysis process. This assumption is valid as reduction of E to half its value causes the panel capacity to reduce only by 2% and any error associated with the E value is therefore negligible [38].
Both the RTWC and SF were modeled in ANSYS using the nonlinear *combin39* and *contac12* zero-length elements. These elements, as shown in Fig 4.4, are used in parallel to achieve the required behavior in all three model dimensions. For the present scenario the *contac12* elements were specified such that they have a high compressive strength and zero tensile and shear stiffnesses. *Combin39* is a nonlinear spring element having longitudinal or torsional capability in one, two or three dimensions. For the current research study, one dimensional longitudinal capability with no rotational restraint was
Figure 4.4. Force displacement behavior of (a) *combin39* element that defines the uplift behavior of 2-16d toenails (b) *combin39* element having the withdrawal behavior of 8d nail sheathing fasteners (c) *contac12* element capturing the compressive behavior of RTWC and SF (d) *combin39* element that defines the shear behavior of RTWC and SF enabled for both the RTWC and SF *combin39* nail elements. Both the edge and field nails of the sheathing panel use the same nail model irrespective of the fact that the edge nail withdrawal behavior is affected by the bending of sheathing and therefore would more appropriately be modeled using a coupled withdrawal-moment behavior. The present use of the same nail models for field and edge nails is recognized as a slightly conservative approach as no rotational restraint is offered by the edge nails resulting in a slight overestimation of displacement. Moreover, the primary focus of this study is on
sensitivity of fragility curves to various modeling assumptions rather than on estimation of the sheathing panel capacity.

Each toenail connection was modeled using one contac12 element (to mimic the nearly incompressible nature of the wood connection) and two combin39 elements (one element to resist the uplift load and the other to counteract any out-of-plane load). The contac12 element and the combin39 element that resists the uplift load are connected as in a parallel system. For a RTWC, the combin39 uplift behavior is defined by four crucial points in the force displacement curve as shown in Fig.4.4a. Simulated correlated random values of peak capacity, displacement at peak capacity and initial stiffness were used to identify Point (F2, D2) and Point (F3, D3). The effective nail length is used to identify Point (F4, D4).

One contac12 element and three combin39 elements were used to model the sheathing fasteners. One of the combin39 elements resists uplift load while the other two resist out-of-pane and in-plane load respectively. The force displacement curve has three significant points as shown in Fig. 4.4b. As in RTWC, the correlated random values of peak capacity and displacement at peak capacity corresponding to point (F2, D2) in the plot were simulated and assigned to sheathing fasteners.

4.4 MODEL PARAMETER STATISTICS

4.4.1 Dead Load statistics

Dead load on the sheathing was calculated using the respective sheathing thickness and wood density which followed a normal distribution (Table 4.1). Dead weight from mechanical and electrical equipment along with the false ceiling dead weight was applied
as a point load on top of the RTWC and was calculated assuming a mean value of 69 kPa (10 lb/ft²). The point load was assumed to follow a normal distribution (Table 4.1). The probability distribution type and COV values were taken from Rosowsky et al., [19]

4.4.2 Wind load statistics

According to ASCE7-10, [42] the members of a low rise building can be either considered as components and cladding (C&C) or as part of the main wind force resisting system (MWFRS). Members like sheathing panels that are subjected to direct wind loads fall under the C&C category. These members are often subjected to localized increased wind pressure due to the spatio-temporal variation of the wind (dynamic wind load) acting on the roof. On the other hand, members of the MWFRS transfer wind load from the C&C system to other components or to the foundation. Wind tunnel tests have shown that there is a definite correlation between various pressure coefficients recorded using pressure taps at different locations on a roof [3, 43]. This correlation evens out localized maximum and minimum wind pressures enabling to design a MWFRS for lower wind loads compared to C&C. In addition, the redundant nature of the load path (load distribution and redistribution among components) supports the reduced load on MWFRS concept. In the present work, wind uplift pressures on the roof are calculated using correlated pressure coefficients on various wind zones defined using C&C. Using this approach facilitated applying high uplift pressure in the roof zones where turbulent wind conditions exist, while still smoothing out the overall wind uplift load acting on the RTWC (MWFRS).
ASCE 7-10 identifies three definite wind zones – Zones 1, 2 and Zone 3 for a C&C system marked as shown in Fig 4.5 and provides external pressure coefficients for these areas. These codified wind pressure coefficients are conservative static approximation of their dynamic counterparts. Using these coefficients, wind pressure on a roof zone can be calculated as given in equation 4.1 [42].

\[
p = 0.613 \, K_z \, K_{zt} \, K_d \, V^2 \, I \, [(GC_p) - (GC_{pi})] \quad (N/m^2)
\] (4.1)
where

\( K_z \) – Velocity pressure exposure coefficient

\( K_d \) – Wind directionality factor

\( I \) – Importance factor

\( K_{zt} \) – Topographic factor

\( G_{Cp} \) – External pressure coefficient

\( G_{Cpi} \) – Internal pressure coefficient

\( V \) – Wind speed in m/s

In this study, instead of using the codified values, the above variables were simulated using the respective governing probability distribution functions and parameters obtained using a Delphi study [4, 15]. A Correlation coefficient matrix (CCM) generated from a National Institute of Standards and Technology (NIST) database for rigid gable roofed structures was used to generate the correlated random external pressure coefficients [44]. The generation of the pressure coefficient correlation matrix was performed by Yin et al., [45]. The initial correlation matrix was developed using time series wind pressure coefficient values in the pressure taps located on various parts of the roof. Since the pressure tap locations did not coincide with the center of sheathing elements, a two-dimensional spatial interpolation method called krigging was employed to derive the final CCM. Using this final CCM, wind pressure coefficients varying over every 30.5 cm x 30.5 cm (12 in. x 12 in.) sheathing area was simulated. Fig. 4.6 shows the wind pressure contour used in one of the simulations. The generated random values ensured that uncertainty in the wind load is captured including spatial variability. Since the modeled
Table 4.1. Summary of load characteristics.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Parameters</th>
<th>Mean</th>
<th>COV</th>
<th>CDF</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Density wood (used for calculating dead load of sheathing and applied as</td>
<td>534 kg/m³</td>
<td>0.1</td>
<td>Normal</td>
<td>Assumed</td>
</tr>
<tr>
<td></td>
<td>pressure)</td>
<td>(0.0193 lb/in³)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Mechanical, electrical and false ceiling load (applied as point load onto</td>
<td>1.33 kN</td>
<td>0.1</td>
<td>Normal</td>
<td>Assumed</td>
</tr>
<tr>
<td></td>
<td>the RTWC)</td>
<td>(300 lb)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>GCₚ - External pressure coefficient for components and claddings system</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Zone 1</td>
<td>0.86</td>
<td>0.12</td>
<td>Normal</td>
<td>[2, 13]</td>
</tr>
<tr>
<td></td>
<td>Zone 2</td>
<td>-1.62</td>
<td>0.12</td>
<td>Normal</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Zone 3</td>
<td>-2.47</td>
<td>0.12</td>
<td>Normal</td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>GCₚᵰ - Internal pressure coefficient</td>
<td>0.46</td>
<td>0.33</td>
<td>Normal</td>
<td>[4, 15]</td>
</tr>
<tr>
<td>5.</td>
<td>Kₜ - Wind directionality factor</td>
<td>0.89</td>
<td>0.13</td>
<td>Normal</td>
<td>[4, 15]</td>
</tr>
<tr>
<td>6.</td>
<td>Kₓ - Velocity pressure exposure coefficient</td>
<td>0.82</td>
<td>0.14</td>
<td>Normal</td>
<td>[4, 15]</td>
</tr>
<tr>
<td>8.</td>
<td>I - Importance factor</td>
<td>Deterministic</td>
<td></td>
<td></td>
<td>[4, 15]</td>
</tr>
<tr>
<td>9.</td>
<td>V - Wind speed</td>
<td>Lower limit</td>
<td>Upper limit</td>
<td>Distribution</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>25 m/s</td>
<td>89 m/s</td>
<td>Uniform</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(55 mph)</td>
<td>(200 mph)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
roof configuration is different than the reference roof configuration used in the Delphi study, adjustments to the mean values were made. The structure was assumed to be partially enclosed and subjected to exposure C. The distribution type and parameters for the wind pressure variables are given in (Table 4.1).

![Figure 4.6](image.png)

**Figure 4.6.** One realization of the simulated wind pressures, in units of kPa on the roof system.

### 4.4.3 Resistance statistics

The rafters are attached to the ceiling using 2-16d nails at each of the rafter ends. Following the recommendations of Shanmugam et al.[10] the present study used an analytical model which is a function of initial stiffness, peak capacity and displacement at peak capacity of the toenails to define the force displacement behavior. The statistics for these three parameters are given in (Table 4.2).
The 8d sheathing fastener was defined using a bi-linear analytical model with a post-ultimate negative stiffness. The performance of the fastener was expressed by means of peak capacity and displacement at peak capacity whose values were simulated using the statistics specified in Table 4.3. The test data for the uplift behavior of SF was obtained from Dao et al. [39] and was analyzed to derive the governing distribution parameters.

The out-of-plane capacity of the RTWC and the in-plane and out-of-plane capacities of SF were considered as deterministic and all three behaviors were modeled using a bi-linear element as shown in Fig. 4.4d. Results from lab experiments conducted by Clemson University graduate students were used to determine the deterministic values of shear capacity of RTWC and SF. The mean out-of-plane and in-plane capacities of the 8d sheathing fastener nails was evaluated as 1.4 kN (307 lb) and the mean out-of-plane capacity of toenail connection was estimated as 4.2 kN (950 lb).
Table 4.2. Resistance of roof-to-wall connectors.

<table>
<thead>
<tr>
<th>Connection type</th>
<th>Uplift capacity (F3) kN (lbs)</th>
<th>Initial stiffness (F2/ D2) kN/mm (lbs/in)</th>
<th>Displacement at peak load (D3) mm (in)</th>
<th>Ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dist. $\lambda$</td>
<td>$\zeta$</td>
<td>Dist. $\mu$</td>
<td>$\sigma$</td>
</tr>
<tr>
<td>2 – 16d toenails</td>
<td>Lognormal 0.356 (5.771)</td>
<td>0.35</td>
<td>Normal 0.372 (2126)</td>
<td>0.134 (768)</td>
</tr>
</tbody>
</table>

Ref = Reference

Table 4.3. Resistance statistics of sheathing fasteners.

<table>
<thead>
<tr>
<th>Connection type</th>
<th>Uplift capacity (F2) kN (lbs)</th>
<th>Displacement at peak load (D2) mm (in)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dist. $\lambda$</td>
<td>$\zeta$</td>
<td>Dist. $\mu$</td>
</tr>
<tr>
<td>8d nail</td>
<td>Lognormal 0.239 (5.654)</td>
<td>0.185</td>
<td>Lognormal -1.251 (-2.183)</td>
</tr>
</tbody>
</table>
4.5 FRAGILITY CURVE GENERATION

4.5.1 Simulation Procedure

Fragility curve generation for the subject roof system is performed using a simulation procedure requiring 500 samples per scenario. This requires that random sampling of many different correlated and uncorrelated variables be conducted. The roof-to-wall connection behavior was determined using the initial stiffness, peak capacity and displacement at peak capacity. Results from the experiments on 2-16d toenails revealed that the above three elements followed different distributions (Table 4.2) but that they are correlated. Hence correlated random values have to be simulated to effectively capture the connection characteristics. Since non-normal multivariate simulation is problematic, an approximate method, the Nataf transformation, was employed to change this into a normal multivariate simulation [46]. The transformation used modified correlation coefficients (correlation coefficient converted to normal space) to generate correlated standard normal random values. Each RTWC required 500 samples to be generated. One should remember that each sample contains three correlated values including capacity and displacement.

Each simulation scenario explored mean wind speed values which were uniformly distributed between 25 m/s and 90 m/s (55 mph and 200 mph). A total of 500 realizations of wind uplift pressure matrix were generated including uncertainty in the values of $K_z$, $K_d$, $GC_{pi}$, $GC_p$ and CCM between pressure coefficients. The various simulation cases considered for the present study is given in (Table 4.4).
Table 4.4. Case number assignment and explanation for various simulations runs.

| Parameter          | Case No. | Case 1 | Case 2 | Case 3 | Case 4 | Case 5 | Case 6 | Case 7 | Case 8 | Case 9 | Case 10 | Case 11 | Case 12 | Case 13 | Case 14 | Case 15 | Case 16 | Case 17 | Case 18 | Case 19 | Case 20 |
|--------------------|----------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|
|                    |          | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    | Yes    |
| RTWC capacity      |          | Non uniform | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   |
|                    |          | High | x   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
|                    |          | Median | x   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
| SF capacity        |          | Non uniform | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   |
|                    |          | High | x   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
|                    |          | Median | x   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
| RTWC stiffness     |          | Non uniform | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   |
|                    |          | Uniform | x   | x   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
|                    |          | Median | x   | x   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
| SF stiffness       |          | Non uniform | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   |
|                    |          | Uniform | x   | x   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
|                    |          | Median | x   | x   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
| RTWC post ultimate stiffness | Yes | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   |
|                    | No |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   | x   |   |   |
| SF post ultimate stiffness | Yes | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   | x   |
|                    | No |   |   |   |   |   |   |   |   |   |   | x   |   |   |   |   |   |   |   | x   |   |

101
Table 4.4. Continued

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
<th>Case 5</th>
<th>Case 6</th>
<th>Case 7</th>
<th>Case 8</th>
<th>Case 9</th>
<th>Case 10</th>
<th>Case 11</th>
<th>Case 12</th>
<th>Case 13</th>
<th>Case 14</th>
<th>Case 15</th>
<th>Case 16</th>
<th>Case 17</th>
<th>Case 18</th>
<th>Case 19</th>
<th>Case 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>SF spacing</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>152 mm/ 305 mm</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>152 mm entire roof</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>152 mm Zone 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sheathing thickness</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11.9 mm</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>9.5 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11.1 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15.9 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gable end support</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yes</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stiffener near rafter ends</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yes</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
</tbody>
</table>
Figure 4.7. Histogram of 500 samples simulated using (a) crude Monte Carlo and (b) Latin hypercube sampling.

In order to keep the runtime for each simulation scenario reasonable (i.e. use 500 simulation) while reducing sampling variability the Latin hypercube sampling technique was used [47]. The advantage of using Latin hypercube sampling over the crude Monte Carlo sampling method is visually demonstrated by means of Fig. 4.7. Fig. 4.7a is a
combined histogram and CDF plot of 500 lognormally distributed samples of RTWC uplift capacity generated using Latin hypercube technique whereas Fig. 4.7b is an identical plot produced employing Monte Carlo simulation to generate values for the same variable. In order to check the adequacy of the total number of simulations (500 simulations) Case 1 was run three times i.e., three independent sets of 500 realizations. Case 1, 2 and 3 are the three simulation cases, each case with the same modeling conditions as the other and with 500 realizations. The fragility curves (Fig 4.8) from the three sets of simulation almost overlap each other and the medians of the lognormal fragility curves fall within a 1 m/s (3 mph) interval (Table 4.5). The uncertainties associated with the fragility curves from Case 1, 2 and 3 are of the same order of magnitude and suggest that 500 is an appropriate sample size.

![Comparison of fragility curves](image)

**Figure 4.8.** Comparison of fragility curves to demonstrate the sampling error involved in using 500 samples.
4.5.2 System failure

The failure to converge (solution non-convergence) is taken as an indication of system failure. This is reasonable as non-convergence was the result of physical separation of either a part or whole of the roof system from the supports. Sanity checks of the model revealed that the non-convergence was not caused due to failure of framing or sheathing members but instead by large displacement at nodes that form the connectors/fasteners. Furthermore effort was taken to ensure that the analysis method used was reliable and is not the cause of non-convergence. When a sufficient number of sheathing nails (combin39 spring) reach their ultimate capacity and start offloading, a point is reached where the excess load distributed to the neighboring connections can no longer be carried without actually causing further redistribution i.e. more nails start offloading. At this stage, the sheathing panel separates from the framing member resulting in an unstable or failed structure as evidenced numerically by non-convergence.

Table 4.5. Comparison of lognormal fragility parameters for a roof system to identify the sampling error.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
<th>Wind speed in mph</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>lambda (λ)</td>
<td>zeta (ζ)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.</td>
<td>Case 1</td>
<td>2.027 (4.535)</td>
<td>0.096</td>
<td>41.9 (93.6)</td>
<td>4.0 (9.0)</td>
<td>41.7 (93.2)</td>
</tr>
<tr>
<td>2.</td>
<td>Case 2</td>
<td>2.029 (4.538)</td>
<td>0.122</td>
<td>42.1 (94.2)</td>
<td>5.2 (11.5)</td>
<td>41.8 (93.5)</td>
</tr>
<tr>
<td>3.</td>
<td>Case 3</td>
<td>2.019 (4.516)</td>
<td>0.095</td>
<td>41.1 (91.9)</td>
<td>3.9 (8.8)</td>
<td>40.9 (91.5)</td>
</tr>
</tbody>
</table>
Thus, failure of a single sheathing panel is reasonably assumed as one of the relevant limit states used throughout this study. Failure of a sufficient number of RTW connections to allow for separation of the entire roof from the base support was considered as another relevant and identifiable limit state. The number of RTWCs that actually cause separation is variable and is a function of the capacity of the neighboring connections and the shed load.

4.5.3 *Fragility curve parameter estimation*

Based on the findings of previous studies [17, 48], a lognormal cumulative distribution function (CDF) was used to model the fragility curves of the roof system. The two parameters (median and the log-standard deviation) of the lognormal fragility curve were obtained using the maximum likelihood method as explained by Shinozuka [49]. The likelihood equation can be written as

\[
L = \prod_{i=1}^{N} [F(w_i)]^{x_i} [1 - F(w_i)]^{(1-x_i)}
\] (4.2)

where \(F(w_i)\) is the lognormal CDF for the \(i^{th}\) roof system failure. \(x_i\) is an outcome of a Bernoulli trial and can take values either 0 or 1. For the present study \(x_i\) is 1 when the roof system has failed (solution non-convergence) at a particular wind speed \(w_i\) and 0 when it did not fail. \(N\) is the number of trials which in the current case is the number of houses tested. The lognormal fragility curve \(F(w)\) for roof system failure can then be expressed as

\[
F(w) = \Phi \left[ \ln(w) - \lambda \right] / \zeta \] (4.3)
where \( w \) is the wind speed acting on the roof, \( \Phi \) is standard normal cumulative distributive function, \( \lambda \) ( \text{in-median} \) and \( \zeta \) ( \text{log-standard deviation} \) are the two lognormal fragility parameters. The two parameters, \( \lambda \) and \( \zeta \) can be evaluated by maximizing the likelihood function employing a straightforward optimization technique [49]. Fig. 4.9 shows the 500 realizations and fitted lognormal fragility curve whose parameters were evaluated using the optimization technique.

4.6 RESULTS AND DISCUSSION

4.6.1 Effect of relative initial stiffness of connectors

It is common knowledge that trusses within a LFWRS attract load in proportion to their stiffness [50, 51]. However, the level of influence that the connection stiffnesses have on this load distribution and redistribution is currently unclear. Since the information on the stiffness influence is pertinent for fragility studies the knowledge may help in formulating appropriate analytical models for future studies. The assumptions on connector stiffness may influence the load path and the fragility estimates of a roof system. In order to ascertain the effect of connection stiffness on fragility modeling six scenarios (Cases 1, 4, 5, 6, 7 and 8) of RTWC and SF stiffness were considered (Table 4.4). The resulting fragilities are compared in Fig 4.10 and their lognormal fragility parameters are given in Table 4.6. At first glance, all fragility curves in Fig 4.10 appear to have almost the same median and uncertainty but on close observation, it is evident that assigning the same stiffness to all the sheathing connectors influences the uncertainty of the roof system fragility.
As one would expect the overall uncertainty in the system decreases and the failure to account for uncertainty in the SF is realized. However, this difference does not appear to be significant since all log-standard deviations are in the range of 0.087 – 0.127. From Fig 4.10 it is clear that having same stiffnesses and different capacity for RTWC has little to no effect on the fragility estimate. This suggests that the RTWC, irrespective of their stiffness have similar influence areas and their failure is dependent more on the variability of wind pressure in their tributary area. Analysis of the results from Cases 1, 4, 5 and 8 also revealed that fragility estimate is more sensitive to peak capacity of the RTWC than to their initial stiffness. Allowing all RTWC to have same stiffness and capacity (median values-Case 8) has clearly shifted the fragility curve to the left side (more fragile) by 5%. The reason for such behavior is since all the connections have same stiffness and capacity there is no reserve capacity available when one of the
connections (especially one in the midsection of roof) has reached its ultimate capacity. When one connection at the roof mid-section reaches its ultimate capacity the other connections are very close to their peak capacities so that they all fail simultaneously. The end RTW connections receive very little load from the rafters as the gable end rafters are supported at regular intervals. These intermediate supports transfer the load on gable end rafters to the foundation through wall studs.

Table 4.6. Lognormal fragility parameters for a light frame residential roof system using different cases of roof-to-wall and sheathing connection behavior.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
<th>lambda (λ)</th>
<th>zeta (ζ)</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Case 1</td>
<td>2.027/0.096/41.9/4.0/41.7</td>
<td>4.535/0.096/41.9/4.0/41.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Case 4</td>
<td>2.015/0.127/40.9/5.2/40.6</td>
<td>4.508/0.127/40.9/5.2/40.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Case 5</td>
<td>2.000/0.081/39.3/3.2/39.2</td>
<td>4.473/0.081/39.3/3.2/39.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>Case 6</td>
<td>2.013/0.095/40.5/3.9/40.3</td>
<td>4.503/0.095/40.5/3.9/40.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.</td>
<td>Case 7</td>
<td>2.008/0.101/40.1/4.1/39.9</td>
<td>4.492/0.101/40.1/4.1/39.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.</td>
<td>Case 8</td>
<td>1.994/0.112/38.9/4.4/38.7</td>
<td>4.460/0.112/38.9/4.4/38.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
As these rigid end rafters attract most of the load applied onto the end zone only little load is transferred to penultimate rafters and RTWC. However as we move far away from the gable end, the rafters in the middle receive approximately equal loads. Since their capacities are equal and they receive roughly equal load, after one connection attains its peak capacity, the rest of the adjacent connections (within the roof mid-section) fails causing the system to fail. But when the connection capacities are random, sometimes connections near the end fail because of their low capacities in spite receiving relatively low load compared to the middle connections.

Another outcome of this study is, for the applied load (wind load variables from Delphi study and correlated pressure applied on C&C zones) roof configuration (gable roof with 22° slope) and connection details (2-16d toenails as RTWC and 8d nails at 15.32cm /30.5 cm (6in/12in) spacing as sheathing fasteners) failure of the roof system is
due to failure of the roof to wall connections. This conclusion is reached after comparing the fragility plots for Cases 1 and 4. Even though Case 1 depicts the roof system fragility and Case 4 captures the roof-to-wall connector system fragility (no failure of SF was allowed), both the fragility plots are identical and almost overlap each other.

The following points summarize the findings:

a) Variability within the withdrawal peak capacity of RTWC has more influence on the fragility estimates than initial stiffness uncertainty.

b) Assuming uniform stiffness for all the sheathing connectors influences the wind load transfer to RTWC and marginally reduces the uncertainty associated with fragility of the roof system.

c) When the gable end rafters are properly connected to the wall supports, the end and penultimate RTWC have very low probability of failure compared to the connectors located in the middle roof.

d) Assigning the same stiffness and capacity forces to all RTWCs, lead to the middle connections initiating the failure.

e) Failure of the considered roof system is mainly due to failure of RTW connections.

4.6.2 Sensitivity of fragility estimate to different post ultimate connection behavior

Until now analytical fragility curves for a system of roof to wall toenail connections and sheathing fasteners were obtained by considering only the peak withdrawal capacity of the nails. Withdrawal resistance statistics of fasteners were used along with load statistics to capture the fragility of the connections. Nail failure was defined simply as
exceedance of peak withdrawal capacity. This indicates that the studies did not acknowledge the negative stiffness offered by the nails after reaching the ultimate capacity. Furthermore, the load redistribution model (after one connection failure) used in some of these studies assumed that as soon as one of the connection reaches its peak capacity, it can no longer carry any load and the neighboring connections share the load carried by the so called failed connection at the time of failure. In order to verify the validity of these assumption two post ultimate behavior models for roof to wall toenail connection and sheathing fasteners were investigated (Cases 1 and 9). When one of the nail models has a post ultimate negative stiffness (realistic representation of nail behavior), the other model behaves like a brittle spring and breaks as soon it reaches the peak capacity (Fig 4.11). Fig. 4.12 shows the difference in fragility estimates when two different post ultimate connection behaviors were used. The median value of the lognormal fragility curve reduced from 41.9 m/s to 38.8 m/s (Table 4.7) when the

**Figure 4.11.** (a) Roof-to-wall and (b) Sheathing connection uplift force-displacement behavior with no post ultimate stiffness.
connection negative stiffness is ignored. This change in fragility is due to the fact that the fasteners will still continue to carry load well beyond their peak capacity which will not require such instantaneous redistribution of loads to adjacent connectors. Failure to consider the post ultimate stiffness overestimates the fragility of the roof system (i.e. lower median wind speed). The brittle spring behavior assumption overloads the adjacent connections, thus initiating the “zipper-like” effect and speeding up the failure process.

In order to find the effect of these two post ultimate connection behaviors on RTW connection systems the sheathing nails were assigned very high stiffness and capacities to prevent SF failure. Fig. 4.13 and Table 4.8 demonstrate the change in fragility of RTWC
system between Case 4 and Case 10 showing a 12 percent difference between median wind speed estimates. Similar to the fragility of the entire roof system,

Table 4.7 Lognormal fragility parameters for a roof system using two different types of roof-to-wall sheathing connection post ultimate behavior.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
<th>lambda (λ)</th>
<th>zeta (ζ)</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Case 1</td>
<td>2.027 (4.535)</td>
<td>0.096</td>
<td>41.9 (93.6)</td>
<td>4.0 (9.0)</td>
<td>41.7 (93.2)</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Case 9</td>
<td>1.991 (4.455)</td>
<td>0.125</td>
<td>38.8 (86.7)</td>
<td>4.9 (10.9)</td>
<td>38.5 (86.0)</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.13. Roof-to-wall connection system fragilities using two different connection behaviors.

RTWC fragility is overestimated when the brittle spring model is used. Thus, by looking at this data the following can be concluded
a) Neglecting post ultimate stiffness of fasteners overestimates the roof system fragility medians by as much as 10 to 15 percent.

b) Post ultimate stiffness reduces the rate at which load is transferred to the neighboring connection.

c) Brittle spring behavior initiates a serial type failure in a roof to wall connection system.

d) Post ultimate negative stiffness model refutes the serial type failure behavior assumption for RTWC. At the same time a definite limit state (i.e. number of failed connections) for the failure of RTWC system cannot be established.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
<th>lambda (λ)</th>
<th>zeta (ζ)</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Case 4</td>
<td>2.015 (4.508)</td>
<td>0.127</td>
<td>40.9 (91.5)</td>
<td>5.2 (11.7)</td>
<td>40.6 (90.8)</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Case 10</td>
<td>1.958 (4.379)</td>
<td>0.098</td>
<td>35.8 (80.2)</td>
<td>3.5 (7.9)</td>
<td>35.7 (79.8)</td>
<td></td>
</tr>
</tbody>
</table>

### Table 4.8. Lognormal fragility parameters for RTWC system using two different types of roof-to-wall connection post ultimate behavior.

#### 4.6.2 Effect of Sheathing thickness on roof-to-wall connection system fragility

In order to estimate the impact of sheathing thickness (i.e. stiffness) on the estimated fragility of a roof system, four different sheathing thicknesses were considered. Fragility plots for Case 1, 11, 12 and 13 are given in Fig 4.14 while the fragility parameter estimates for the four cases are listed in Table 4.9. The post ultimate stiffness (F3, D3) of
the sheathing fasteners are different for the four cases, as the effective nail embedment length is reduced with increasing sheathing thickness. From comparing the force in each sheathing fasteners for the four different cases, it was evident that a thick

![Figure 4.14. Roof system fragility using different sheathing stiffness-thickness.](image)

sheathing transfers load to the field fasteners in proportion to their stiffness and a thin sheathing transmits approximately equal load to all the field fasteners which is more tributary based. Thus when sheathing panel of thickness 0.95 cm (0.375 in.) (a flexible sheathing) was used, approximately equal loads were transferred to the roof to wall connections thus reducing the uncertainty in their fragility estimation. However the influence of sheathing thickness on the median values is rather inconclusive. There seems to be no pattern associated with the sheathing thickness and fragility estimation. The reason may be due to the fact that change in sheathing thickness is not an
independent factor but is associated with change in the sheathing fastener behavior (nail effective length is reduced).

Table 4.9. Comparison of lognormal fragility parameters for roof system using different sheathing thickness

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
<th>Lambda (λ)</th>
<th>Zeta (ζ)</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Case 1</td>
<td>2.027 (4.535)</td>
<td>0.096</td>
<td>41.9 (93.6)</td>
<td>4.0 (9.0)</td>
<td>41.7 (93.2)</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Case 11</td>
<td>2.003 (4.482)</td>
<td>0.086</td>
<td>39.7 (88.7)</td>
<td>3.4 (7.7)</td>
<td>39.5 (88.4)</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Case 12</td>
<td>1.995 (4.463)</td>
<td>0.125</td>
<td>39.1 (87.4)</td>
<td>4.9 (11.0)</td>
<td>38.8 (86.7)</td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>Case 13</td>
<td>2.013 (4.502)</td>
<td>0.110</td>
<td>40.6 (90.8)</td>
<td>4.5 (10.0)</td>
<td>40.3 (90.2)</td>
<td></td>
</tr>
</tbody>
</table>

4.6.3 Influence of additional framing members on the fragility estimates.

Post hurricane investigations have revealed that improper gable end connections have resulted in damage to gable end walls and roof failure. The end rafters when properly secured will transmit a significant portion of the wind load acting on the roof end zones to the foundation through the wall studs. The presence of gable end supports reduces the failure probability of the end and penultimate RTWCs. When gable end bracings are not connected properly to the wall beneath, end zone wind pressures are transmitted to the foundation through the RTWC instead of gable end supports. This increases the chances of failure of RTWC. The fact that gable end supports force the RTWCs which are far away from the end supports to fail more often than the end and penultimate connectors must be kept in mind while deriving component fragilities for RTWC systems. Fig 4.15
depicts the change in roof fragility with the presence and absence of gable end support (Cases 1 and 14). Table 4.10 indicates that there is 9 percent increase in the roof fragility when the gable ends were not supported. From Fig 4.15 it is obvious that the presence of a stiffener (e.g. fascia board) near the rafter ends doesn’t significantly affect the fragility estimation since there is only a 1.2 m/s difference between the median wind speeds (Cases 1 and 15).

![Fragility plots for roof system with and without gable end supports and stiffener near the rafter ends.](figure)

**Figure 4.15.** Fragility plots for roof system with and without gable end supports and stiffener near the rafter ends.

### 4.6.4 Roof sheathing system fragility

It is assumed that failure of a single nail (exceeding the ultimate withdrawal capacity) results in progressive failure of the entire sheathing. The basis of this assumption is failure of the weakest nail in a roof panel onsets the failure of adjacent nails leading to complete panel and rafter separation. Since single panel uplift is enough
Table 4.10  Lognormal fragility parameters for a roof system considering different additional framing members.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
<th>lambda (λ)</th>
<th>zeta (ζ)</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Case 1</td>
<td>2.027 (4.535)</td>
<td>0.096</td>
<td>41.9 (93.6)</td>
<td>4.0 (9.0)</td>
<td>41.7 (93.2)</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Case 14</td>
<td>1.979 (4.426)</td>
<td>0.125</td>
<td>37.7 (84.3)</td>
<td>4.7 (10.6)</td>
<td>37.4 (83.6)</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Case 15</td>
<td>2.015 (4.507)</td>
<td>0.126</td>
<td>40.8 (91.4)</td>
<td>5.2 (11.6)</td>
<td>40.5 (90.6)</td>
<td></td>
</tr>
</tbody>
</table>

To cause severe rain induced damage, one panel failure is considered as the limit state for the roof sheathing system fragility. Sometimes the panel capacity has been estimated from an individual nail withdrawal capacity by considering a factor to reflect the influence of adjacent nails [9, 37]. However this assumption has yet to be verified. So until recently the redistribution of load after a connection reaches its peak capacity and the post ultimate negative stiffness was either completely ignored or indirectly considered in roof sheathing fragility estimation.

The present study uses a finite element model of sheathing panels over an entire roof to estimate the roof sheathing system fragility. Single sheathing panel uplift was considered as the limit state. In order to estimate the influence of sheathing fastener stiffness on the roof sheathing system fragility two cases were considered. The first case had different stiffness and capacity for each of the sheathing fasteners while the other case had the same stiffness and capacity (median values) for all of the sheathing fasteners. Fig 4.16 shows the fragility curves for the two cases (Case 16 & 17) considered. The lognormal fragility parameters for both the cases are presented in Table
4.11. Assigning median capacity had shifted the fragility curve to the right indicating an increase in the panel uplift capacity. The reason for this change in fragility may be that having same stiffness and capacity for all the fasteners resulted in equal load distribution to all the field nails which in turn caused a serial type failure, i.e., when one connection reaches the ultimate capacity, the adjacent connections (field nails) reach their ultimate capacity almost simultaneously and the roof sheathing fails. In the case where different capacities and stiffnesses were used, even though the weakest nail capacity doesn’t govern the sheathing panel capacity it still is the source of failure. Since the assigned median uplift capacity value is greater than the weakest nail capacity in a sheathing panel, the panel uplift capacity has increased for Case 17. On investigating the failed panels from the two different cases the average panel uplift capacity for Case 16 was found to be 4.5 kPa (94 psf) while the same was 4.7 kPa (98 psf) for Case 17. Thus,

![Figure 4.16. Fragility curves of roof sheathing using variable (Case16) and same fastener parameters (Case 17).](image-url)
attributing the same stiffness to sheathing connectors has an impact on load distribution to RTWC but no effect on the sheathing fragility estimation.

To investigate the effect of load redistribution and post ultimate negative stiffness of fasteners two cases, one using the nail model depicted in Fig. 4.4b and the other using the fastener behavior given in Fig. 4.11b were considered. The fragility estimates for Cases 16 and 18 are given in Table 4.12 along with fragility parameters obtained from a similar study by Li [14]. Fig. 4.17 demonstrates the change in fragility for the two post ultimate nail behaviors. The difference is highly significant as considering the post ultimate negative stiffness decreased the roof sheathing fragility by 39 percent. Close

![Figure 4.17](image)

**Figure 4.17.** Comparison of roof sheathing fragilities from two different studies and different sheathing fastener behavior.
Table 4.11 Roof sheathing fragilities with variable and uniform connection parameters.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
<th>lambda (λ)</th>
<th>zeta (ζ)</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Case 16</td>
<td>2.282 (5.105)</td>
<td>0.104</td>
<td>74.1 (165.8)</td>
<td>7.7 (17.3)</td>
<td>73.7 (164.9)</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Case 17</td>
<td>2.308 (5.163)</td>
<td>0.095</td>
<td>78.4 (175.5)</td>
<td>7.5 (16.7)</td>
<td>78.1 (174.7)</td>
<td></td>
</tr>
</tbody>
</table>

observation of the fragility plot reveals that the fragility estimate from Case 18 almost matches with the results from Li [16]. However one has to be remember that the sheathing statistics from Li [16] were obtained from Schiff et al.[9] which in turn derived the panel capacity using the weakest nail failure assumption. Therefore ignoring the post ultimate stiffness suggests that the weakest nail (single nail) governs the panel capacity and a serial type failure occurs as soon as the weakest nail reaches its ultimate capacity. Conversely acknowledging the existence of negative stiffness after the peak capacity advocates that one or more than one connection failure is needed to initiate the zip-type failure and the limit state is a function of withdrawal capacities of the nails adjacent to the failed fastener and the negative stiffness of the failed connection.

Table 4.12. Comparison of roof sheathing fragility parameters.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
<th>lambda (λ)</th>
<th>zeta (ζ)</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Case 16</td>
<td>2.282 (5.105)</td>
<td>0.104</td>
<td>74.1 (165.8)</td>
<td>7.7 (17.3)</td>
<td>73.7 (164.9)</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Case 18</td>
<td>2.134 (4.774)</td>
<td>0.122</td>
<td>53.3 (119.3)</td>
<td>6.5 (14.6)</td>
<td>52.9 (118.4)</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Li [16]</td>
<td>2.145 (4.798)</td>
<td>0.168</td>
<td>55.0 (122.9)</td>
<td>9.3 (20.7)</td>
<td>54.2 (121.2)</td>
<td></td>
</tr>
</tbody>
</table>

122
At the time of failure, when the brittle spring model was employed, the average maximum pressure on the sheathing panel was 2.9 kPa (60 psf). However when the negative stiffness nail model was considered, the average panel capacity increased to 4.5 kPa (94 psf). This 57 percent increase in the panel capacity emphasizes the importance of considering the post ultimate reserve capacity available in the fasteners while estimating the roof sheathing fragility.

4.6.5 Influence of nailing schedule on the roof sheathing fragility estimation

The American Plywood Association (APA) suggests that for low-rise structures located in areas where the basic wind speed (3-sec gust wind) is 40 m/s (90 mph) the sheathing nails should be fastened at every 152 mm (6 in.) at the edges and at every 305 mm (12 in) in the field [52]. APA recommends 152 mm (6 in.) spacing at the gable end supports and in Zone 3 (Fig. 4.5). However it is a common construction practice to use 152 mm (6 in) in the edges and 305 mm (12 in) in the field for the entire roof. The above two nailing schedules were considered for the present study. In addition nailing schedule with the fasteners spaced at every 152 mm (6 in.) over the entire roof was also investigated. The roof sheathing fragilities for the three considered nailing schedules are tabulated in Table 4.13 and shown in Fig. 4.18. It is evident from the plot that the fragility of the roof decreased when closer nail spacing was used either for the entire roof or for Zone 3 alone. Closer nail spacing in Zone 3 alone reduces the fragility by 19 percent whereas 152 mm (6 in) nail spacing throughout the roof reduces the fragility by 27 percent. Investigation of the failed roof panels distinctly revealed that the panels located in the edges are prone to fail more frequently than the panels in the inner area of the roof.
When closer nail spacing was used in Zone 3, the susceptibility of the edge panels was reduced.

![Image](image_url)

**Figure 4.18.** Roof sheathing fragility for different nailing schedule.

**Table 4.13.** Values of roof sheathing fragility parameters for different nailing schedules.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
<th>lambda ((\lambda))</th>
<th>zeta ((\zeta))</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Case 16</td>
<td>2.282 (5.105)</td>
<td>0.104</td>
<td>74.1 (165.8)</td>
<td>7.7 (17.3)</td>
<td>73.7 (164.9)</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Case 19</td>
<td>2.387 (5.339)</td>
<td>0.118</td>
<td>93.7 (209.7)</td>
<td>11.1 (24.9)</td>
<td>93.1 (208.2)</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Case 20</td>
<td>2.358 (5.275)</td>
<td>0.121</td>
<td>87.9 (196.7)</td>
<td>10.7 (23.9)</td>
<td>87.3 (195.3)</td>
<td></td>
</tr>
</tbody>
</table>

4.6.6 Additional discussion

When comparing Fig. 4.10 and Fig. 4.16 it is apparent that, when a roof system is constructed with 2-16d toenails as roof to wall connection along with the 8d nails at 15.2
cm/30.4 cm (edge nail spacing/field nail spacing) spacing as sheathing fastener, failure of the roof system is caused by failure in the RTWCs. Even though the sheathing fasteners can withstand a wind speed of 74.1 m/s (165.8 mph) – median value, the roof system fails at 41.9 m/s (93.6 mph) – median value, well below the sheathing panel capacity. This in fact is not an economical design of the roof system. In order to have a balanced design either the RTWC should be a metal strap/hurricane tie (decrease the fragility of RTWC) or the nailing schedule for sheathing panels could be relaxed (increase the roof sheathing panel fragility).

4.7 CONCLUSION

In the past studies, it is common to use laboratory tests on sheathing panels and roof to wall connections to formulate the fragility of the entire roof. Thus system fragilities were generally expressed as a function of component fragilities, precluding any effect due to composite action of individual components. Single component failure was the chosen system limit state in such studies. However this simplified system fragility calculation needs to be validated for any practical use. The current study primarily investigated the credibility of simplified system fragility calculation by employing a finite element model of an entire roof and deriving its fragility.

The secondary motivation for the current study is to check the effect of various modeling assumptions on the fragility calculation. Advancements in computational power and tools spurred ways to model and analyze parts or entire roof system subject to wind loads and to estimate the system fragility. However certain modeling assumptions may overestimate or underestimate the fragility assessments and needs to be verified.
The present study evaluated the sensitivity of fragility curves to various modeling approaches. Treatment of fastener behavior, composite action of framing members and sheathing panels, effect of gable end supports, sheathing thickness and nailing schedule were some of the modeling aspects that were explored. The treatment of post ultimate connection behavior has a huge influence on the fragility assessment of the roof system. Neglecting the negative stiffness (reserve capacity) of the connector after its peak capacity has been exceeded, as often done in simplified fragility analysis, not only underestimates the uplift strength of the roof system but also results in incorrect understanding of the roof behavior as a system.

Based on the failed sheathing panels, the sheathing panel capacity was numerically evaluated as 4.5 kPa (94 psf) using connections with post ultimate stiffness while using fasteners with no post-ultimate capacity produced a sheathing capacity estimate of 2.9 kPa (60 psf). There is a 28 percent shift in the median values of the fragility curves. A similar trend is observed in the roof to wall connection fragility, when the above two types of connection behavior were used. Influence on the fragility due to the absence of proper gable end connections and to the addition of stiffener elements such as fascia boards was evaluated. Presence of gable end supports and tighter nailing schedules reduced the roof system fragility considerably whereas the additional stiffener element did not significantly alter the roof system fragility estimation.

The results from the current study indicate that while estimating the fragility of roof system or component it is essential to include the post ultimate connection (both RTWC and SF) behavior. Therefore any fragility analysis methodology that calculates roof
system or component using only the peak uplift capacities should be modified to account for the post ultimate negative stiffness. Since the initial stiffness did not alter the fragility estimate significantly, uniform stiffness can safely be assumed for all the connectors. Non-inclusion of members like fascia board or ratrun in the finite element model of a roof system, will not affect the fragility estimation. If roof to wall system fragility is desired it is essential to account the gable end supports and its influence on penultimate connections.

4.8 REFERENCES


5. EFFECT OF SPATIAL WIND LOAD CORRELATION ON 
THE FRAGILITY ASSESSMENT OF LIGHT FRAMED 
LOW - RISE RESIDENTIAL ROOF SYSTEMS 

5.1 ABSTRACT 

Wind pressures on low rise residential roof systems not only vary temporally but also vary spatially. These pressures can be correlated, the extent of which is dependent on the wind direction, orientation of the building, roof configuration and roof zone. The spatial correlation is often neglected while experimentally determining the roof sheathing panel (RSP) capacity and roof to wall connection (RTWC) capacities. The influence of the spatially correlated wind pressures on fragility estimates of RTWC and sheathing panel systems is unknown and needs to be investigated. The present study utilizes a correlation coefficient matrix (CCM) derived from a National Institute of Standards and Technology (NIST) database for wind external pressure coefficients to account for the spatial correlation on a low rise rigid roof system. Probability distribution parameters from a Delphi study was used to identify the probability distribution functions (PDF) and parameters for the other wind load factors. The calculated wind pressure was applied to a finite element model of a roof structure and fragility plots for the roof system, RTWC system and RSP were derived. The results from this spatially correlated wind load model was compared with three other wind load models: 1) considers no uncertainty in any of the wind load factors (deterministic) and uses mean values obtained from the Delphi study to calculate the wind pressure, 2) each of the three roof zones use simulated wind
pressure values calculated using the PDF’s from the Delphi study. The uplift pressure values can be considered as fully correlated within a zone but no correlation exists between the zones and 3) Uncorrelated wind pressure over the entire roof. The results from the study indicated that the fragility estimations of both RTWC and sheathing panel systems are not sensitive to the spatial correlation of wind pressure when wind flows perpendicular to the ridge. In addition using uncorrelated wind pressures instead of a spatially correlated wind pressures on a RSP doesn’t make a substantial difference in estimated panel capacities. Furthermore the study explores ways to improve the wind model used in the present study in order to realistically estimate the fragility of the roof system.

5.2 INTRODUCTION

The uncertain and fluctuating nature of wind loads can pose significant challenges for a structural engineer when involved in structural assessment activities. Comprehending the wind behavior is essential to being able to adequately numerically model the wind load and its effects on a building. The temporal and spatial variation of wind loads on low rise residential roof structures is implicitly accounted for by ASCE 7-10 by using an equivalent static uniform wind load over different roof zones [1]. The codified static equivalent load is supposed to be a conservative estimate over the time and space varying wind loads. The different zones over the roof identified by ASCE 7-10 - field, edge and corner areas - are demarcated roof areas whose mean wind pressures are significantly different (spatial variation) from one another. In addition, experimental studies have revealed that correlation exists between pressure coefficients at different parts of the roof.
[2-4]. However a clear consensus has not been reached on whether the assumed fully correlated static uniform wind loads over different zones effectively capture the extreme wind load case.

Past and ongoing studies on wind tunnel scaled models and full size structures are an effort to find the influence of the varying nature of wind load on the realized sheathing panel capacities, roof failure modes, roof to wall connection system capacities and wind effects (bending moment and internal force). These types of studies also help to ascertain the conservatism in using the static uniform load over different roof zones. In one of the earliest studies, a correlation coefficient matrix for wind pressure coefficients at different parts of the roof zone of a full scale building were obtained for cornering wind loads [2]. The recorded pressure coefficients were averaged over time and space and then compared with the prevailing codified values (ASCE 7-02) at that time. Not only was it revealed that the spatio-temporal averaging was not conservative at some places, but high correlation of pressure coefficients was found to exist in the conical vortex region. [2]. However a portion of a standing seam metal roof subjected to a dynamic spatially varying cornering wind load had about 50% conservatism (50% lower wind load than ASCE 7-02) over a uniformly applied static pressure values defined by ASCE 7-02 and required by ASTM E1592 tests [5]. This result was further supported by a wind tunnel study on a scaled model of the standing seam metal roof. [5]. Area averaged peak pressure coefficients from a wind tunnel study was found to be conservative when compared to effective peak pressure coefficients from the Australian code, except in one wind direction [6]. Similar results were observed for wind load effects when equivalent static
wind pressure coefficients derived using a covariance integration method and a load–response-correlation method was used on the scaled model [6]. Furthermore, one study reported a change in failure mode for mechanically attached roofs when static loads were replaced by dynamic wind loads [7] – a change from a fastener pullout failure mode to a membrane shear failure mode.

Uniformly distributed suction loads applied in steps on an individual roof sheathing panel inside a pressure chamber is the generally accepted experimental procedure to determine the roof sheathing uplift capacity. The earliest known studies used the above experimental method to statistically determine the roof sheathing panel capacities for different nail types, nail spacing, panel types (Oriented strand board or plywood) and sheathing thickness [8-10]. The results from these studies have been widely used to determine roof sheathing fragility and reliability indices [11-14]. Until now little effort was taken to study the influence of spatio-temporal variation of wind loads on the RSP capacity. A recent wind tunnel study on a scaled model of a light framed wood residential system (LFWRS) identified dynamic pressure traces to be used in experimental tests on RSPs along with developing CCMs for spatially varying wind pressure coefficients for different wind directions [4]. A follow up study using the developed dynamic pressure traces showed that temporally varying wind pressure reduced the perceived panel capacities by five percent [15]. A finite element based reliability study on roof sheathing panels using a 3-minute wind pressure time series from Hurricane Ivan concluded that there is reduction in reliability indices when dynamic wind loads were considered [14].
Cope et. al.[3] developed correlation coefficients for wind pressures recorded at pressure taps located within the same row (rows are parallel to the ridge) for different wind directions. Different levels of correlation between wind pressure coefficients were assumed to investigate the effect of spatial correlation on panel failure. Correlated wind pressure coefficients increased the probability of failure of a sheathing panel near the ridge for wind flowing parallel to the ridge [3]. Furthermore, high correlation and non-Gaussian pressure characteristics were observed for a sheathing panel located near the ridge when the wind direction was perpendicular to the gable end [3]. The same conclusion was drawn by Gleason [16] for RTWC system fragilities, when an assumed correlation matrix for wind pressure coefficients was used in a finite element model of the roof system. However, numerical evaluation of a sheathing panel capacity using a finite element model of a single RSP showed that incorporating a spatially varying wind pressure model does not alter the panel capacity obtained using the uniform wind pressure model [17].

The effect of spatially varying wind loads on roof to wall connections was also investigated on a full scale residential structure [18, 19]. Temporally varying and spatially fluctuating wind pressure traces obtained from a wind tunnel test on a scaled model were applied to the structure using pressure loading actuators and airbags of different sizes. In a follow up study, realistic wind loads were applied to individual RTWCs and their capacities were compared with those obtained using ramp loads. Fluctuating wind loads caused the RTWC to fail at a lower load when compared to ramp loads [19].
The present study in particular is concerned about the influence of spatial correlation of the wind load on fragility estimates of low-rise wood roof systems with emphasis on the RTWC system and roof sheathing panels for a particular wind direction. Impact of spatially varying wind load on RSP capacity is also analyzed. Additionally various options available to improve the current wind load models are explored.

5.3 FINITE ELEMENT MODEL

A finite element model of the roof system of a typical yet simple residential structure located in South Carolina was developed using the general finite element package ANSYS [20]. Prior to its demolition the single story gable roofed structure was tested to evaluate in-situ RTW connection capacities. Several reasons for selecting this roof system as a baseline for this study include - 1) simple rectangular roof configuration typical of many low rise residential structures 2) availability of RTWC analytical model for the considered structure 3) straight forward development of CCM using the NIST database [21] 4) code defined wind zones available for a rectangular roof system 5) comparison with previous studies made easy because of similar roof configuration.

The gable roofed system had a 5:12 pitch with the 38 x 89 mm (2 x 4 in. nominal) ceiling joists spanning over a length of 9.27 m (30.42 ft.) (Fig. 5.1). Thirteen pairs of stick built roof rafters spaced at 0.64 m (24 in.) with the upper rafter ends connected to a ridge board at the roof center and the lower end tied to the double top-plate by two 4.1 mm (0.161 in.) diameter by 89 mm (3.5 in.) long smooth shank 16-d common nails toenails formed the entire roof system. The 11.9 mm (15/32 in) thick and 1.22 x 2.44 m
(4 x 8 ft.) sized plywood roof sheathing panels are attached to the rafters using 3.3 mm (0.131 in.)
diameter by 63.5 mm (2.5 in.) long smooth shank 8-d common nails spaced at 152 mm (6 in) at the edge and 305 mm (12 in.) in the field.

*Beam4* and *beam44* elements were used to model the roof framing members. The plywood sheathing panels were modeled using a single layered *shell63* element with isotropic properties. A zero length nonlinear *combin39* and a *contac12* element were arranged in parallel to model both the RTWC and sheathing fastener (SF) uplift behaviors. In addition, two *combin39* elements were used to model both the in-plane and out-of plane behavior of the SF. The out of plane force - displacement behavior of the RTWC was modeled using one *combin39*. Detailed description of the fastener models is
given in the previous chapter. The RTWC spring element captures the nonlinear uplift behavior and offloading characteristics of two 16d toenails as was experimentally determined in a previous study [22]. The contact12 element offers very high compressive stiffness which simulates the rafter bearing on a wall system. Fig. 5.2a illustrates the basic backbone behavior of the RTWC. The SF is also defined in the same way as the RTWC with the only exception being the bilinear force-displacement behavior as seen in Fig. 5.2b. The resistance statistics used to develop the connection uplift behaviors are given in Table 5.1.

![Figure 5.2](image)

**Figure 5.2.** Uplift force displacement behavior of (a) two 16d toenails and (b) 8d sheathing fasteners
Table 5.1. Connection resistance statistics.

<table>
<thead>
<tr>
<th>Connection type</th>
<th>Uplift capacity in kN (lbs)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dist</td>
<td>λ</td>
</tr>
<tr>
<td>8d nail</td>
<td>Lognormal</td>
<td>0.239 (5.654)</td>
</tr>
<tr>
<td>2 – 16d toenails</td>
<td>Lognormal</td>
<td>0.356 (5.771)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Connection type</th>
<th>Initial stiffness in kN/mm (lbs/in)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dist</td>
<td>μ</td>
</tr>
<tr>
<td>2 – 16d toenails</td>
<td>Normal</td>
<td>0.372 (2126)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Connection type</th>
<th>Displacement at peak load in mm (in)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dist</td>
<td>μ</td>
</tr>
<tr>
<td>8d nail</td>
<td>Lognormal</td>
<td>-1.251 (-2.183)</td>
</tr>
</tbody>
</table>

|                 | Dist | κ | u | ε |          |
| 2 – 16d toenails | Weibull | 1.299 (0.336) | 8.52 (0.130) | 3.308 (0.130) | [24] |

5.4 WIND LOAD MODEL

The edge, corner and field zones of the roof structure were defined using the ASCE 7-10 components and cladding system (C&C). The choice of the C&C system for the current study is to ensure that the sheathing elements are subjected to localized peak pressure while still transferring an averaged out load to the RTWC. The building was considered to be partially enclosed and located in an exposure category C. The wind was assumed to flow perpendicular to the ridge. The governing probability distributions for various factors in the wind pressure equation of ASCE 7-10 were identified using a
Delphi based study [23]. As per ASCE 7-10 the wind pressure (p) on a low rise building is given by equation (5.1)

\[ p = 0.613 \times K_z \times K_{zt} \times K_d \times V^2 \times I \times [(G_{C_p}) - (G_{C_{pi}})] \]  
\[(N/m^2)\] \hspace{1cm} (5.1)

The various factors in the above equation, namely internal pressure coefficient (GC_{pi}), zonal external pressure coefficients (GC_p), wind directionality factor (K_d), velocity pressure exposure coefficient (K_z)- were simulated using the guidance of the Delphi study. The Delphi statistics are listed in Table 5.2. Both the topographic factor (K_{zt}) and importance factor (I) were treated as deterministic values. In equation (5.1) V is the basic wind speed for the region where the reference structure is located. For the current study however, wind fragilities are being generated which are probabilistic statements of failure conditioned upon the wind speed. Thus, this generation procedure requires that a broad range of wind speeds be considered. Therefore, V is the mean wind speed generated using a uniform distribution with the upper and lower limits as given in Table 5.2. The use of Delphi based statistics in the present study is justified by its use in many of the reliability and fragility studies [12, 13, 24] to date. In fact Cheng et al., [25] who developed wind speed composite statistics using wind speed data from three coastal cities utilized the Delphi based statistics to find the mean to nominal ratio of wind pressure and coefficient of variation (COV).
Following are the four wind load models that were considered for this research to explore the impact of various wind modeling assumptions on resulting wind fragility estimates for wood roof systems.

**Table 5.2. Summary of load statistics.**

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Parameters</th>
<th>Mean</th>
<th>COV</th>
<th>CDF</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>GC&lt;sub&gt;p&lt;/sub&gt; - External pressure coefficient for components and claddings system</td>
<td>-0.86&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.12</td>
<td>Normal</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Zone 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Zone 2</td>
<td>-1.62&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.12</td>
<td>Normal</td>
<td>[23, 35]</td>
</tr>
<tr>
<td></td>
<td>Zone 3</td>
<td>-2.47&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.12</td>
<td>Normal</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>GC&lt;sub&gt;pi&lt;/sub&gt; - Internal pressure coefficient</td>
<td>0.46</td>
<td>0.33</td>
<td>Normal</td>
<td>[23, 35]</td>
</tr>
<tr>
<td>3.</td>
<td>K&lt;sub&gt;d&lt;/sub&gt; - Wind directionality factor</td>
<td>0.89</td>
<td>0.13</td>
<td>Normal</td>
<td>[23, 35]</td>
</tr>
<tr>
<td>4.</td>
<td>K&lt;sub&gt;z&lt;/sub&gt; - Velocity pressure exposure coefficient</td>
<td>0.82</td>
<td>0.14</td>
<td>Normal</td>
<td>[23, 35]</td>
</tr>
<tr>
<td>5.</td>
<td>K&lt;sub&gt;zt&lt;/sub&gt; - topographic factor</td>
<td>Deterministic</td>
<td></td>
<td></td>
<td>[23, 35]</td>
</tr>
<tr>
<td>6.</td>
<td>I - Importance factor</td>
<td>Deterministic</td>
<td></td>
<td></td>
<td>[23, 35]</td>
</tr>
<tr>
<td>7.</td>
<td>V - Wind speed</td>
<td>25 m/s (55 mph)</td>
<td>89 m/s (200 mph)</td>
<td>Uniform</td>
<td></td>
</tr>
</tbody>
</table>

<sup>a</sup> – the mean values were modified to account for the deviation of the roof configuration from the reference roof dimension

a) Model 1: Correlated external pressure coefficients for zones 1, 2 and 3 were obtained using a CCM derived from the NIST database [21] and the Delphi study [23]. The
factors $GC_{pi}$, $K_z$, $K_d$ all follow a normal distribution with the distribution parameters obtained from the Delphi study. The method employed to develop the CCM is explained in detail later in this paper. The simulated values were used in equation (5.1) to obtain the wind uplift pressure values on the roof. Fig. 5.3a shows a characteristic roof pressure realization contour obtained using this model.

b) Model 2: Mean values of $GC_p$, $GC_{pi}$, $K_z$, $K_d$ taken from the Delphi study were used to derive the wind uplift pressure values in zones 1, 2 and 3 (All the wind load factors are deterministic except for the different wind speed which had values between 25 m/s-90 m/s (55 - 200 mph). The roof pressure contour simulated using this wind load model is shown in Fig. 5.3b. In summary, no uncertainty is considered in the wind but uncertainty in the structural characteristics is still present.

c) Model 3: The values of $GC_p$, $GC_{pi}$, $K_z$, $K_d$ were simulated using the distribution parameters from the Delphi study. The external pressure coefficients are same within a given zone and can be assumed as fully correlated inside a zone. However no correlation exists between the zonal uplift pressures – i.e. no correlation between the pressures in zones 1, 2 and 3. Fig. 5.3c shows one of the realizations of roof pressure generated using this wind load model.

d) Model 4: Uncorrelated external pressure coefficients were generated for every 305 mm x 305 mm (12 in x 12 in) sheathing area within each zone using the respective zonal distribution parameters. Simulated values of $GC_{pi}$, $K_z$, $K_d$ were then used to derive the wind uplift pressure over each 305 mm x 305 mm (12 in x 12 in) sheathing
area. Fig. 5.3d shows a representative roof pressure contour obtained using this wind load model.

**Figure 5.3.** Roof plan showing wind pressure contours in kPa for (a) Wind load model -1 (b) Wind load model -2 (c) Wind load model -3 (d) Wind load model -4.
windPRESSURE is database assisted design (DAD) software that provides wind pressure time series, as taken from many wind tunnel studies for rigid gable roofed buildings, in order to compute various peak wind load effects [21, 26]. The software uses an interpolation scheme for buildings whose dimensions do not match with the reference structures to generate wind pressure time series. With pressure coefficient time series being provided at various locations on the roof, a correlation matrix can then be developed based on actual data. The current study utilizes a CCM developed by Yin et al., [27] for the baseline structure with pressure taps located as shown in Fig. 5.4a using the wind pressure time series data from the NIST archives [21]. Since the location of pressure taps did not exactly matchup with the discretized sheathing panel elements (center point of sheathing elements shown in Fig. 5.4b) of the finite element model, a two dimensional spatial interpolation method called krigging was employed to modify the CCM. The updated CCM provides the correlation between wind pressure coefficients between every 305 mm x 305 mm (12 in x 12 in) sheathing element. To check the veracity of the method employed a contour plot of the correlation coefficients for a tap located at the roof coordinates 3.66 m, 2.13 m (12 ft., 7 ft.) (Fig. 5.4c) was compared with the corresponding CCM contour plot of a sheathing element located close to the tap as indicated in Fig. 5.4d. The plots are near identical and have the coefficient values in the same range. The reader must keep in mind that using this spatial interpolation technique may sometimes create a matrix that is not positive definite. A correction method may need to be employed in that case to make the matrix positive definite so that simulation using this matrix may be performed.
Figure 5.4 Roof plan showing (a) pressure tap locations as in the database (b) desired 305 mm x 305 mm (12 in x 12 in) pressure locations (c) correlation coefficient contour for the pressure tap marked in the figure (d) interpolated correlation coefficient contour for the sheathing element identified.

5.5 SIMULATION PROCEDURE AND FRAGILITY CALCULATION

The cases considered for the present research study are defined in Table 5.3. The listed cases were selected in order to study the sensitivity of fragility estimates of sheathing panels, RTWC system and the entire roof system (i.e. both sheathing and
RTWC combined) to wind pressure spatial correlation. A few cases were included to see the impact of connection stiffness on the fragility estimates using a particular wind load model. For each scenario 500 realizations of wind load were generated using the Latin hypercube sampling [28] technique. The RTWC and SF parameters were also simulated from their respective probabilistic distribution for each of the 500 realizations. Non-linear analysis of the finite element model results in non-convergence when there is a physical separation of part or whole of the roof from either the base support (RTWC failure) or framing members (SF failure). The solution converged when the applied uplift pressure was safely sustained by the roof system. A failure was recorded as one and a survival was noted as a zero. The above information along with the corresponding wind speed data was used to obtain the fragility plot as described below.
Table 5.3. List of simulation cases.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Parameters</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
<th>Case 5</th>
<th>Case 6</th>
<th>Case 7</th>
<th>Case 8</th>
<th>Case 9</th>
<th>Case 10</th>
<th>Case 11</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Values of $GC_{pi}$, $K_d$, $K_z$</td>
<td>Probabilistic</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>Deterministic</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>2.</td>
<td>Values of $GC_p$</td>
<td>Probabilistic</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>Deterministic</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>Fully Correlated</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>Uncorrelated</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>Partially correlated (CCM)</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>3.</td>
<td>RTWC uplift capacity</td>
<td>Very High</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>Actual</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>4.</td>
<td>SF uplift capacity</td>
<td>Very High</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>Actual</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>5.</td>
<td>RTWC initial stiffness</td>
<td>Uniform</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>Non uniform</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>6.</td>
<td>SF initial stiffness</td>
<td>Uniform</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>Non uniform</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
</tbody>
</table>
A lognormal distribution has been used to model sheathing and RTWC fragilities in past studies [29, 30]. The present study therefore assumes that the lognormal cumulative distribution function (CDF) is appropriate to model the roof fragility curves subjected to wind loads. The lognormal parameters are estimated using the maximum likelihood method (MLE) where the likelihood equation is expressed as given in equation 5.2.

\[ L = \prod_{i=1}^{N} [F(w_i)]^{x_i} [1-F(w_i)]^{(1-x_i)} \]  

(5.2)

where \( F(w_i) \) is the lognormal CDF evaluated for the \( i^{th} \) realization, \( w_i \) is the considered wind speed for the \( i^{th} \) simulation and \( x_i \) is the solution convergence indicator (0-survived, 1-failed). A straightforward optimization technique was used to obtain \( \lambda \) (\( \ln(\text{median}) \)) and \( \zeta \) (logarithmic-standard deviation) – the two lognormal fragility parameters. A detailed description of the fragility parameter estimation procedure is given by Shinozuka et al.,[31]. An example of the estimated lognormal fragility curve using 500 realizations is shown in Fig. 5.5.

5.6 RESULTS AND DISCUSSION

The definition of different roof zones by ASCE 7-10 is to account for the spatial variation of the wind load near the edges and corner where the flow separates. Roof sheathing fastening schedules for wind uplift loads account for this spatially fluctuating wind pressure by having closer nail spacing near the gable ends and roof corners than the
rest of the roof [32]. However the impact of the wind pressure spatial correlation on the RSP capacity has not been accounted for while evaluating the roof sheathing fragility estimates. The sensitivity of a RTWC system fragility and a roof system fragility to spatially fluctuating wind pressure have also not been investigated so far. The results obtained from the current finite element based simulation approach using four different wind load models for wind flowing perpendicular to the ridge may shed some light on the sensitivity of fragility estimates to wind pressure spatial correlation.

5.6.1 Effect of spatial correlation on the fragility of roof sheathing panel and sheathing capacity

In order to obtain the sheathing system fragility the finite element model of the roof system was created such that all of the RTWCs have very high capacities making it impossible for them to fail for the applied wind pressure thus ensuring that if the model fails it will be because of sheathing failure. Four types of wind load models which were
explained earlier in the paper were considered to find the effect of spatial correlation on the fragility estimates. The first simulation set (Case 1) used correlated uplift pressure values over the entire roof (Wind load model 1). This case considered partial correlation (realistic) between wind pressures on a roof when wind flowed perpendicular to the ridge. Careful investigation of the CCM indicated that high correlations between pressure coefficients existed over a short distance and then decreased in a nonlinear fashion as this distance increased. Low correlation generally exists between locations on the windward side and points on the leeward side of the roof. These characteristics are consistent with the CCMs obtained from other studies. The second wind load model used for Case 2 is representative of a deterministic wind load. The mean values of the governing wind load factors like $-G_{C_p}$, $G_{C_{pi}}$, $K_z$, $K_d$ – were obtained from the Delphi study and represent the reduced values of the codified nominal values from ASCE 7-10. The reduced nominal values were used in order to compare with the other three load models. Since the mean values were obtained from the code defined nominal values using a reduction factor, the shape of the fragility curve will actually represent the shape of the fragility plot if codified nominal values were used. The only difference would have been a shift of the curve to the left of the plot with the shift proportional to the mean to nominal ratio. The third wind load model used for Case 3 represents the fully correlated wind pressure model. The simulated wind pressure values within a zone are fully correlated (uniform pressure values in each zone) but no correlation is considered between the different zones. This correlation model is based on the proposition that full correlation is possible within a short distance but not over the entire roof (no correlation between the zones).
However the chance of this condition to exist in reality is very unlikely but all the past experimental procedures to determine the RSP capacity indirectly assert this condition. The fourth wind load model, used for simulation Case 4, considered no correlation to exist between the wind pressure values at any part of the roof. Again the chance of this condition to exist in reality is very unlikely.

The roof sheathing fragility curves for the different wind load models are given in Fig. 5.6 and the corresponding fragility estimates are given in Table 5.4. From the fragility plots for Case 1, 3 and 4 it is evident that the level of correlation has only a little impact on the fragility estimates of the roof sheathing system. Close investigation revealed that irrespective of the level of correlation, the total pressures acting on a sheathing panel for the three load cases were the same. This may explain why the roof sheathing fragility is insensitive to the spatial correlation.

**Table 5.4.** Lognormal fragility parameters for roof sheathing system when subjected to the four wind load models.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>lambda ($\lambda$)</td>
</tr>
<tr>
<td>-------</td>
<td>------</td>
<td>---------------------</td>
</tr>
<tr>
<td>1.</td>
<td>Case 1</td>
<td>2.282 (5.105)</td>
</tr>
<tr>
<td>2.</td>
<td>Case 2</td>
<td>2.278 (5.096)</td>
</tr>
<tr>
<td>3.</td>
<td>Case 3</td>
<td>2.285 (5.111)</td>
</tr>
<tr>
<td>4.</td>
<td>Case 4</td>
<td>2.284 (5.109)</td>
</tr>
</tbody>
</table>
Figure 5.6. Roof sheathing system fragilities for four different wind load model.

The only notable change in fragility curves are that when the wind pressures are fully correlated (Case 3), the uncertainty involved is greater than that of Case 1 and Case 4. The same conclusion was reached after the mean uplift capacities of the sheathing panels for the three loading cases were calculated. The difference between mean RSP capacities for the three cases was less than 95 Pa (2 psf). Therefore it can be safely concluded that the level of correlation between the pressure coefficients for wind perpendicular to the roof ridge is insignificant while estimating the fragility and capacity of RSP. The above conclusion may seem to contradict the results from Cope et al.[3] but upon further consideration such a comparison is inappropriate for the following reasons:

a) In the study by Cope et al., 2005, the wind direction was either parallel to the ridge or at an oblique angle to the ridge, which is not the case in the present study.
b) The governing probability distribution parameters of wind pressure external coefficients used in the current study are different from those used in Cope et al., 2005.

As expected, using a deterministic wind load (Case 3) reduced the uncertainty significantly in the fragility estimates of RSP system. If codified nominal values had been used instead of the reduced code defined nominal wind loads, one would expect the fragility curve of Case 3 to shift entirely to the left of the three other fragility plots indicating an inherent safety margin for design. This is because the code defined static uniform wind loads are expected to envelope the extreme wind load fluctuations. The RSP designed for the code based design load should be able to sustain the wind load fluctuations without failure.

The reader is reminded that the external pressure coefficients (for Case 1, 3 and 4) in the present study were simulated using the mean and COV values from a Delphi study and not using the statistics from the pressure taps. The roof zones classified according to the ASCE 7-10 C&C system could have been different since the CCM was obtained with wind flowing perpendicular to the ridge. Because of the above two reasons, the fragility plots presented herein could be used only to demonstrate the sensitivity of fragility curves to spatial correlation but should not be used to represent the actual fragility of the roof sheathing system.

Following is the summarized conclusion from the roof sheathing system fragility study:
a) Spatial correlation does not appear to significantly affect the estimation of roof sheathing system fragility when the wind direction is perpendicular to the roof ridge. However when the wind pressures are fully correlated within a zone more uncertainty in the fragility estimates are observed.

b) The mean roof sheathing panel capacity is insensitive to the wind pressure spatial correlation.

c) Deterministic wind load reduces the uncertainty of the fragility estimates significantly.

d) The current study does not represent actual fragility of the sheathing panels but is a sensitivity study of wind pressure spatial correlation effect on the fragility estimation.

5.6.2 Impact of spatial correlation on the roof to wall connection system fragility

Roof to wall connection system fragility plots were obtained by assigning very high capacity to the sheathing fasteners preventing their failure so that if any failure occurred in the roof system that it would be due to failure of the RTWCs. The effect of wind load models 1, 2 and 4 on RTWC fragility was studied using the simulation cases 5, 6 and 7. As in the previous simulation case (RSP fragility) both partially correlated and uncorrelated wind pressure values had little influence on the RTWC fragility estimation. On the other hand assigning deterministic wind pressure loads significantly reduced the uncertainty in the fragility estimates. The fragility plots for the three cases are given in Fig. 5.7 and the calculated fragility curve parameters are given in Table 5.5.
### Table 5.5. Lognormal fragility parameters for a RTWC system when subjected to three different wind load models. (Wind load models 1, 2 and 4)

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
<th>lambda (λ)</th>
<th>zeta (ξ)</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Case 5</td>
<td>2.015 (4.508)</td>
<td>0.127</td>
<td>40.9 (91.5)</td>
<td>5.2 (11.7)</td>
<td>40.6 (90.8)</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Case 6</td>
<td>2.005 (4.486)</td>
<td>0.023</td>
<td>39.7 (88.8)</td>
<td>0.9 (2.0)</td>
<td>39.7 (88.7)</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Case 7</td>
<td>2.019 (4.517)</td>
<td>0.126</td>
<td>41.2 (92.3)</td>
<td>5.2 (11.7)</td>
<td>40.9 (91.5)</td>
<td></td>
</tr>
</tbody>
</table>

In the absence of temporal variation one would expect to use Main Wind Force Resisting System (MWFRS) pressure coefficients as defined by ASCE 7-10 [33] to calculate wind loads instead of the C&C pressure coefficients for the RTWC fragility estimation. However, since the current investigation is a sensitivity study to spatial correlation and involves multiple components like RTWC and SF, the use of C&C is justified. If an accurate estimation of the RTWC fragility was warranted, the actual probability distributions from the wind pressure time series recorded in the pressure taps must be used along with CCM to simulate roof pressure values.

### 5.6.3 Influence of spatial correlation on the overall roof fragility

The fragility of the roof system subjected to four different wind load models was investigated by assigning the actual connection stiffness and capacity to both the SFs and
Figure 5.7. Roof to wall connection system fragilities using three different wind load models. (wind load models 1, 2 and 4)

the RTWCs. Cases 8, 9 10 and 11 used wind load models 1, 2, 3 and 4 respectively. The results from the four studies corroborated the conclusion from the previous two simulation sets (RTWC and SF fragility). Spatial correlation is insignificant to roof system fragility estimation when wind flows perpendicular to the ridge. The roof system fragility curves using four different levels of spatial correlation are presented in Fig 5.8. The lognormal fragility parameters for the four cases (Cases 8,9,10 and 11) are listed in Table 5.6.

5.6.4 Improvements in the current wind load model

The current study is a promising step towards realistic wind load modeling and its effect on fragility estimates. The developed roof system fragility analysis methodology could be expanded by having correlation coefficient matrices derived for various wind directions. In addition the influence of building dimension and roof configuration
Table 5.6. Lognormal roof system fragility parameters for the four wind load models.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Case</th>
<th>Wind speed in m/s (mph)</th>
<th>lambda (λ)</th>
<th>zeta (ζ)</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Case 8</td>
<td>2.027 (4.535)</td>
<td>0.096</td>
<td>41.9 (93.6)</td>
<td>4.0 (9.0)</td>
<td>41.7 (93.2)</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Case 9</td>
<td>2.008 (4.492)</td>
<td>0.023</td>
<td>39.9 (89.3)</td>
<td>0.9 (2.1)</td>
<td>39.9 (89.3)</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Case 10</td>
<td>2.012 (4.500)</td>
<td>0.117</td>
<td>40.5 (90.6)</td>
<td>4.7 (10.6)</td>
<td>40.2 (90.0)</td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>Case 11</td>
<td>2.014 (4.505)</td>
<td>0.107</td>
<td>40.7 (91.0)</td>
<td>4.3 (9.7)</td>
<td>40.5 (90.5)</td>
<td></td>
</tr>
</tbody>
</table>

on the CCM could be evaluated. Furthermore actual pressure time series from the wind pressure taps could be used to simulate wind pressure external coefficients instead of the values from Delphi study. Finally dynamic pressure traces can be used to study the effect of time varying wind pressure load on the fragility estimates of the roof system, sheathing panel and RTWC system. The change in the building enclosure type (fully enclosed to partially enclosed) after a SF or RTWC has failed also needs to be studied. The suggested improvements will complete the current roof system fragility methodology and will help in verifying the conservatism involved in using the codified static uniform wind loads.
5.7 CONCLUSION

The effect of spatial variation of the wind load on the fragility estimates of roof to wall connection systems and sheathing panel is so far undetermined. Uniformly distributed wind pressure is usually used to calculate the sheathing panel capacity and to obtain component fragility curves for RTWC and sheathing panels. The calculated fragilities may not be a realistic representation of the actual fragilities due to the assumed wind load model. NIST has a database for wind tunnel tests on scaled models of low rise building equipped with pressure taps. The database characterizes the spatially varying dynamic wind pressure values for different wind directions and roof configurations. An interpolation scheme was employed to derive the wind pressure coefficients for any other roof configuration that is not in the system. The present study used the NIST database to calculate the CCM between external pressure coefficients on a reference roof system for wind flowing perpendicular to the ridge. The computed CCM was used to simulate
correlated wind pressure coefficients which in turn were used to calculate spatially correlated wind pressures on the roof. Sheathing panel and RTWC system fragilities were obtained by applying simulated spatially varying wind loads on a finite element model of the roof system. The estimated fragilities were compared with fragility curves obtained using three other wind load models employing varying levels of correlation and uncertainty. It was concluded that the spatial correlation between pressure coefficients for wind flowing perpendicular to the ridge did not significantly affect either the capacity or fragility of roof sheathing panel. Similar conclusion was drawn for roof system fragility and RTWC system fragility.

Since the study considered only one wind direction, the conclusion cannot be extended for a roof system subjected to different wind directions. Also the effect of roof pitch and building dimension on the CCM is unknown. The use of a Delphi study based mean and COV values for external pressure coefficients could have misrepresented the zonal pressure values used for the fragility estimation. For all the above reasons, the present report should therefore be considered as a sensitivity study of fragility curves for different levels of wind pressure spatial correlation and not as an effort to evaluate the actual roof fragility itself. However the methodology adopted herein to incorporate the spatial correlation effect on fragility estimation can be extended to include the temporal variation of wind along with investigating the effect of roof pitch, building dimension and wind direction on the CCM.
5.8 REFERENCES


6. CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

This study identifies the effects of various modeling assumptions – from both the demand and resistance side - on the wind fragility estimation of a gable roof system. First, as part of the investigation a statistical and analytical model was developed for toenailed roof to wall connections. Second, the resistance behavior of metal roof to wall connectors when subjected to combined loads was studied and an efficient design space was identified. Third, the developed analytical model for toenailed connection was used in finite element based simulation to evaluate the influence of connection behavior on roof fragility estimation. The influence of various modeling conditions like nail spacing, sheathing thickness and gable end supports on roof system fragility estimation was explored. Also, the sensitivities of the RTW connection system fragility, roof sheathing system fragility and roof sheathing panel capacity to connection behavior were explored. Finally the effect of spatially correlated wind loads on roof system fragility and sheathing panel system fragility estimates was investigated along with evaluating the sensitivity of roof panel capacity to various levels of wind pressure correlation.

The specific conclusions from Chapter 2 which investigated the in-situ capacity of RTW toenail connections are:

a) The lognormal distribution can be used to model uplift capacities of in-situ toenailed roof to wall connections. The normal distribution and three parameter Weibull distribution are proposed for initial stiffness and displacement at peak load respectively.
b) Three connection parameters (ultimate uplift capacity, initial stiffness and vertical displacement at peak load) can be used to appropriately define the analytical model of the connection uplift behavior.

The specific conclusions presented in Chapter 3 for metal RTW connectors subjected to multi-axial loads are:

a) The currently used design equation for metal connectors subjected to multi-axial loads was found to be inefficient and overly conservative.

b) Based on the criteria of efficiency, performance and safety, a new design space for the three types of metal connectors was proposed. The new design surface is shown to have a high level of safety and adequate performance while providing up to 2.5 times the usable design space as compared with the current practice.

Chapter 4 presents the following conclusions for the sensitivity study on fragility curves due to various modeling conditions:

a) The uncertainty in initial stiffness of the connectors has no significant influence on the fragility estimation. However variability in withdrawal peak capacity of roof to wall connectors has significant influence on the fragility estimates.

b) Neglecting post ultimate stiffness of sheathing fasteners overestimates the roof system fragility medians by as much as 10 to 15 percent and underestimates the roof panel capacity.

c) Ignoring the post ultimate stiffness of roof to wall connectors overestimates the roof system by as much as 7 to 8 percent.
d) Closer nail spacing in corner zones reduces the roof sheathing system fragility significantly.

e) When the gable end rafters are properly connected to the wall supports, the end and penultimate roof to wall connectors have very low probability of failure compared to the connectors located in the middle roof. Presence of gable end supports was found to reduce the roof system fragility.

Chapter 5 investigated the effect of spatially correlated wind pressure values on roof system fragility and its conclusions are given as below:

a) The spatial correlation between pressure coefficients for wind flowing perpendicular to the ridge did not significantly affect either the capacity or fragility of the roof sheathing panels.

b) The spatial correlation has no significant influence on the roof system fragility and roof to wall connection system fragility estimation.

6.2 RECOMMENDATIONS

Based on the outcome of Chapters 2 and 3, the following recommendations are made

a) Development of resistance statistics and analytical model for roof to wall metal connectors subjected to wind uplift loads.

b) Estimation of roof to wall metal connector system fragility.

Given below are recommendations based on the outcome of Chapters 4 and 5

a) Evaluation of realistic roof system fragility estimation using actual material properties and realistic wind loading.
b) Estimation of the influence of wind direction, building dimension and roof configuration on the pressure coefficient correlation matrix and thereby to identify their influence on roof system fragility estimation.

c) Numerical evaluation of realistic roof panel capacity for dynamic wind loads.

d) Extension the fragility study to include lateral wind loads.
APPENDIX
7. PROBABILISTIC EVALUATION OF RETROFITTED ROOF – TO – WALL CONNECTIONS IN LIGHT FRAME WOOD RESIDENTIAL STRUCTURES

7.1 ABSTRACT

Post storm investigations exposed the vulnerability of toenail connections to extreme wind loads and advocated the use of hurricane ties as retrofits for roof – to – wall connections. Improper installation of hurricane ties leads to reduced capacity and must be taken care of. The present study involved structural evaluation of retrofitted roof – to – wall connections and existing metal clip connections. Six existing structures were investigated for their adequacy against extreme wind loads. Thirty two connections retrofitted with single H2.5 Simpson strong tie and sixteen connections using H1.0 hurricane tie were tested on site and their mean uplift resistance was estimated. Thirty two connections utilizing double H2.5 as retrofits were field tested to evaluate the mean uplift capacity. Sixty five existing metal clip connections were lab tested and their uplift capacity was calculated. Probability distribution fits and parameters describing the capacity each of the above mentioned retrofits and metal clip connections were identified. The initial effective stiffness of H2.5 and Trip – L – Grip metal clip connections was assessed and their probability distribution fits were identified. An analytical model that captures the response of H2.5 connection was developed. The detrimental effect of improper installation of hurricane ties was emphasized and the relative benefit of proper installation was highlighted.
7.2 INTRODUCTION

Following Hurricane Andrew building codes were revised in 1994. Post hurricane investigations after Hurricane Charley and Ivan (2004, 2005) revealed that structures built according to the revised code sustained little or no damage. However the country still hosts an enormous building inventory in hurricane prone regions that were built prior to 1994 and were not upgraded in accordance with the new code. These structures not only pose a serious threat to themselves but also to neighboring structures by being a source for debris. These conventional (non-engineered or partially engineered and not built according to code) structures have to be effectively strengthened in order to transfer appropriate wind force acting on them to the foundation. It is always faster, easier and economical to retrofit an existing structure that to rebuild it for high wind loads. Improving the nailing schedule for the roof sheathing fasteners, using storm resistant shutters for doors and windows, installing strong hurricane ties for roof – to – wall connections and using metal straps for wall to foundation connection are few of the retrofit options available for existing homes. The above mentioned retrofits would ensure that the entire structure is properly tied as a unit against wind load.

A potential weak link that is decisive to maintain the structural integrity and is present in the wind load resistant path of a non-engineered house is the roof – to – wall connection. The age old practice of using two or three toenail to secure the roof system to wall has often proved to be detrimental, the reason being that these connections (toenail connections) were not designed for high wind loads. Failure of the roof – to – wall connection leads to discontinuity in the structural resistance path and fuels damage
propagation. Retrofitting such untenable roof – to – wall toenail connections is necessitated by state and city building codes and insurance companies. The insurance companies in order to limit their post hurricane losses switched to percentage deductible instead of the conventional dollar deductible. This amounts to the homeowners paying for a percentage, say 2 or 5%, of the damage suffered, from their pocket before the insurance company chips in. The damage incurred by existing houses not retrofitted is considerably higher than the retrofitted and engineered residential structures resulting in higher percentage deductible. The homeowners are further burdened with higher premiums if their houses are not protected against wind storms. Some state governments encourage homeowners to adapt to new codes by offering discounts for insurance, if their homes were upgraded using retrofits and necessary damage mitigation measures were taken.

In order to illustrate the significance of retrofits various studies were undertaken. The strengths of retrofit connections fabricated in the laboratory were evaluated for both cyclic and monotonic loading [1]. The capacities of various retrofits such as hurricane ties, adhesives and metal clips were compared and contrasted [2]. The probability distribution fit for the H2.5 metal clip connection was identified to be normal. The strength and stiffness degradation of the roof – to – wall retrofit connection when switched from pure uplift load to biaxial load (including in-plane shear) and tri-axial load (including in-plane and out-of-plane shear) was categorically examined. The reduction in the effective stiffness of the retrofit connection when subjected to cyclic loading instead of monotonic loading was discussed. It is imperative that the hurricane ties are installed
correctly to utilize their capacity to the fullest. The manufacturer’s guidelines specify the number of nails required to fasten the ties and the procedure for correct installation. Lack of nails may result in failure of roof system as evidenced during hurricane Katrina. Wrong installation procedure often leads to reduced capacity[3]. The need for proper installation of hurricane tie was emphasized by studying the effect of installing the hurricane ties inside the wall, as against the manufacturer’s specification, on the uplift capacity.

The present study focuses on obtaining the in-situ uplift capacity of retrofitted connection in existing buildings constructed 50 – 60 years before. The study is an extension of previous study from the same authors that evaluated the in-situ capacity of toenail connections in existing buildings. The study further lab tested roof – to – wall metal clip connections, obtained from residential structures constructed 50 – 60 ago. Clemson University provided access to six residential structures constructed between 1947 and 1960 which were to be demolished. This provided a unique opportunity to collect valuable perishable data on the strength of existing (Trip – L – Grip metal clip connection) and retrofitted (H2.5 and H1.0) residential structures. Thirty two toenail roof – to – wall connections were retrofitted with single H2.5 hurricane tie and were field tested. H1.0 was used to retrofit 16 toenail connections and their insitu capacities were assessed. 32 more connections were retrofitted using double H2.5 connection and their onsite uplift strengths were evaluated. The mean stiffness of H2.5 hurricane tie was estimated and an analytical model to simulate the response of H2.5 connection against uplift load was developed. In addition, sixty five roof – to – wall metal clip (Trip – L –
Grip) were lab tested and their uplift resistance and initial effective stiffness were evaluated. The Trip – L – Grip metal clip connections lacked four nails that attached the metal clip to the side of the top plate. The influence of this improper installation was studied and discussed in brief. Probability distribution fits and parameters were identified for the capacity of both retrofit and existing metal clip connections. The distribution fit and parameter for initial stiffnesses of H2.5 and Trip – L – Grip metal clip were also identified.

By using connections from existing structures, the study reduced the chances for epistemic uncertainty, accounting for the variability involved in workmanship in field construction. The developed analytical model will promote enhanced design and analysis of light frame wood residential structures. Proposed probability distribution fits and parameters would positively contribute to loss prediction and damage mitigation studies.

7.3 EXPERIMENTAL STUDY

Two types of baseline light frame wood residential structures were considered for the present study. Both the types of structures were located inside the Clemson University campus. The first category of structures is single storey; duplex houses located in the Douthill housing community (Fig. 7.1).
Figure 7.1. Douthit Hills duplex residential structure

The second kind is similar to the first in type (single story and duplex) and plan (rectangular) and is situated in Thornhill Village (Fig. 7.2). Both the type of buildings have been continuously occupied and maintained up to the time they were handled over for testing purposes.

The Douthill residences located in Clemson University campus was constructed approximately 60 years ago. The construction is representative of the 1950s, consisting of wood stud wall with veneer cladding and a sloped roof (6:12). The stick built roof system is made up of 38 x 140 mm (nominal 2 x 6 inch) or 38 x 89 mm (nominal 2 x 4 inch) ceiling joist and 38 x 40 mm rafters. Fig. 7.3 is the schematic of the roof framing plan.

The roof sheathing consists of 1 x 6 wood planks fastened using two 8d nails per rafter. The roof exterior was covered with asphalt shingles. Each Rafter was secured firmly at their lower end to the side of the ceiling joist by means of three 3.3 mm (0.131 inch) diameter, 63.5 mm (2.5 in) long smooth shank 8-d common nails as illustrated in Fig. 7.4. Roof to wall connections (ceiling joist to wall top plate) were made using two
or three 4.1mm (0.161 in) diameter, 89mm ( 3 ½ in) long smooth shank 16-d common nails as depicted in Fig. 7.4. In the present study, single H2.5, H1.0 and double H2.5 were used as retrofits for toenail roof to wall connections.

**Figure 7.2.** Thornhill duplex residential structure.

**Figure 7.3.** Typical roof framing plan of Douthill residential structures.

The Thornhill residential structures were student housing units located on the east side of the Clemson University campus. Fig. 7.5 is a schematic representation roof framing plan. The roof system was made up prefabricated truss having a 3:12 pitch and
covered with 1x8 plank sheathing. The trusses were spaced at 0.41m (16 inch) and the top and bottom chords were 38 x 89 mm in size (nominal 2 x 4 inch). The web members of the truss were 38 x 140 mm (nominal 2 x 6 inch).

**Figure 7.4.** Roof-to-wall connection detail.
The heel and ridge joints of the truss were constructed using 19 mm (½ inch) diameter bolts. The walls were made up of hollow blocks and were covered in the exterior by vinyl sidings. The wall top plate was wood and was anchored to the masonry block by means of 19 mm (½ inch) diameter bolts at 0.81 m (32 inch) on center. The trusses are attached to the top plate at each joint by a single Teco type Trip-L-Grip metal clip anchor (Fig. 7.6). From visual observation it was evident that, both, Douthill and Thornhill residential structures were constructed using Southern Yellow Pine (SYP).

### 7.4 INSITU TEST

#### 7.4.1 Experimental set up

The setup is similar to the one designed previously by the same authors to evaluate the in-field capacity of system of toe nail roof to wall connections. Similar experimental
studies used various test set ups to evaluate the uplift capacity. Hydraulic crane on the site and load tree or hydraulic jack on the lab are some of the few loading devices to mention. The type of loading, Cyclic or Monotonic and the number of connections tested simultaneously i.e., system or individual test, also varied from test to test.

The present setup was designed to test a system of four connections simultaneously. This system test is an effort to capture the load sharing and redistribution effect on the uplift capacity of the connections. Sharing of load among the connections is influenced significantly by the stiffness of each connection and distance of the connections from the point of application of load. Thus due to load sharing effect the uplift capacity of a connection tested individually is never the same as the capacity when tested in a group of connections. The present study therefore accommodates the load sharing effect by carrying out uplift tests on a system of four connections simultaneously.

The size of the test set up and the capacity of the Screw jack controlled the number of connection (four, in the present study) to be tested simultaneously. Cyclic displacements were applied in order to describe the hysteretic nature of the connection and to quantify the energy dissipated under uplift load. This information is necessary while developing the analytical model of the connection.

At first, the four connections were segmented from the rest of the roofing system by cutting out the crossing members on either side of the segment. Part of the drywall and roof ceiling was removed to fix the hurricane ties that connected the top plate with the ceiling joist. Three types of hurricane ties were considered for the current study. Single H2.5, double H2.5 and H1.0 were the retrofits tested for the present study. Two
automated screw jacks each having capacity of 22.2 kN (5 kips) carried the spreader beam that applied cyclic displacements to the system of connections. Four load cells each having a capacity of 22.2 kN (5 kips) and attached to the top flange of the spreader beam applied uplift load to the corresponding ceiling joists. Four LVDT’s attached to wall studs measured the relative displacements between the top plate and the bottom of the ceiling joist. (Fig. 7. 7) Automation of the screw jack and the data acquisition from the load cell and LVDT were controlled using Labview 8.0.

Figure 7.6. Trip – L – Grip metal clip connector.
7.4.1 Experimental procedure

ASTM D1761 is the general test procedure for testing individual mechanical fasteners under monotonic loading. The loading rate from the above testing protocol was extended for the present study on system of fasteners under cyclic loading. Cyclic deflections were applied at three predetermined deflections -1.6, 3.2 and 4.8 mm (0.0625, 0.125, 0.1875 in) at a controlled rate of 2.54 mm/min (0.10in/min) ± 25 %, as per ASTM-D1761. For each cyclic stage uplift load was applied till the prearranged displacement is reached, after which, load was brought down to zero. Once the displacement has cycled at 4.8
mm, load was applied till the failure of the connection. This loading sequence was selected in order to mimic the low loads experienced by the connection prior to the extreme wind event.

Dead load on each of the connection is recorded from the load cell, after the failure of the connection. The dead load is the load from ceiling joist and other roof framing members. The uplift capacity is the maximum load withstood by the connection minus the dead load. Corrections were applied to the uplift capacity and displacement of each connection to accommodate for the eccentric placements of load cell and LVDT along the longitudinal axis.

7.5 LABORATORY TEST

7.5.1 Experimental set up

A new test set up was devised to test the capacity of metal clip (Trip- L-Grip metal clip) connections that attached the trusses to wall top plates. These metal clip connections are either attached to the rafters or to the bottom chords of the truss member. The test setup consisted of a reaction frame that carried an automated screw jack (Fig. 7.8).
Figure 7.8. Experimental setup for laboratory uplift tests.

Load was applied on the metal clip connection by means of a steel cable attached to the load cell. This loading mechanism was implemented to ensure that the resultant of the uplift load was applied directly over the connector. The size of the steel cable was selected such as to minimize its elastic deformation under the expected loads. An LVDT (Linear Variable Displacement Transducer) was attached to the reaction frame and mounted on the top of rafter/ bottom chord (depending on the location of metal clip) to measure the relative displacement between the top plate and rafter/bottom chord. National Instruments Data Acquisition devices were used to collect data from the load cell and LVDT and to control the displacement rate of the jack. The load cell is compression/tension capable and has a capacity of 5 kips. The LVDT has a stroke length
of 2 inch and a spring return armature for easy installation. The screw jack, driven by micro stepping motor, has a capacity of 5 kips.

7.5.1 Experimental procedure

The experimental procedure adopted for the laboratory study is the same as the insitu test except that the tests were carried out on individual connections. The uplift capacity is the maximum load sustained by the connection. No deduction for dead load is made as the connection is not subjected to any load other than the uplift load. As the LVDT’s and load cells were located concentric to the metal clip connection no correction was applied to account for their placements.

7.6 RESULTS AND DISCUSSION

7.6.1 Uplift capacity

Retrofit roof – to – wall connection

Retrofits are fixed in existing homes in three ways 1) from the roof side while reroofing 2) from the inside by removing a portion of the drywall and ceiling and 3) from the outside by removing the soffit and exterior cladding. If the exterior cladding is brick wall, it is difficult to fix the ties as a portion of the wall has to be removed. Nevertheless manufacturer’s guidelines specify that the hurricane ties have to be installed outside the wall in order to avoid eccentricity in the vertical load path. Failure to do so will result in reduced uplift capacity due to the rotation of ceiling joist/rafters.
In the present case, as the exterior had brick veneer cladding, the retrofit was fixed from inside. A portion of the drywall and ceiling was removed around the perimeter of the roof, exposing the roof to wall connection as shown in Fig. 7.9. As the wall studs were precision framed there was no difficulty in installing the retrofits and also in maintaining the vertical load path. Three types of Simpson’s strong tie - single H2.5, double H2.5 and H1.0 were employed to retrofit the existing roof to wall toenail connections.

H2.5 (see Fig. 7.10 (b)) is a twisted metal strap that is used to attach ceiling joist with the top plate/rafter. The lower leg of H2.5 is long enough to be attached to double top plate and hence it is used in houses constructed with double top plates. Special type of Simpson’s 8d galvanized nails was used to attach the strap to the ceiling joist and top plate. These nails are shorter than ordinary 8d nails in order to prevent the longitudinal splitting of rafter/ceiling joist when subjected to extreme wind load. Five 8d nails were
used to attach the lower leg to the side of the top plate and five 8d nails were used to attach to the upper leg to the ceiling joist. 32 connections (8 systems of connections) were retrofitted using H2.5 and their uplift capacity was evaluated. The mean ultimate capacity was estimated to be 5.34 kNs (1200 lbs) with a coefficient of variation (COV) of 0.20.

![Diagram of H1.0 hurricane tie and H2.5 hurricane metal strap]

**Figure 7.10.** a) H1.0 hurricane tie b) H2.5 hurricane metal strap

32 connections (8 systems of connections) were retrofitted using double H2.5s. Double H2.5s was constructed using a single H2.5 on either side of the ceiling joist. The lower legs of the H2.5s should be facing away from each other as shown in Fig. 7. 11. This is to avoid intersecting nails in the ceiling joist and thereby to prevent longitudinal splitting of joist. The mean estimate of the uplift capacity of double H2.5s is 7.20 kN (1618 lbs) with a COV of 0.24.
Figure 7.11. Double H2.5 hurricane tie

Figure 7.12. Failure of connection due to a) strap tear b) top plate split.
H1.0 is a square plate with a slot at the diagonal edge that holds the rafter/joist (Fig. 7.10 (a)). 6 -8d galvanized nails, 3 on each side of the slot and 4-8d nails on the square plate were used to fasten the H1.0 tie to the rafter and ceiling joist respectively. H1.0 was used, in the present study, to retrofit 16 connections (4 systems of connections) and its average ultimate capacity was evaluated to be 4.72 kN (1062 lbs) with a COV of 0.24.

![Image of H1.0 and double H2.5](image1.jpg)

**Figure 7.13.** Top plate failure of a) H 1.0 strap b) double H2.5

Two types of failure modes were observed in the present study - 1) strap tear and 2) top plate split. When retrofitted with a single H2.5 21 connections failed due to strap tear (Fig. 7.12 (a)) and 11 connections failed due to top plate split (Fig. 7.12 (b)). The connections retrofitted with H1.0 and double H2.5 failed mostly due to top plate split (see Fig. 7.13) except for two cases where the connections did not fail at all. In those two cases, connections retrofitted with double H2.5 started pulling the wall below, resulting in cracks along the length of the drywall. In the above two cases, application of uplift load was stopped once the ultimate capacity was reached.
Figure 7.14. Crack in the drywall.

The reason for no strap tear when retrofitted with double H2.5 and H1.0 is equal sharing of load by the connectors/nails on either side of the joist. But in the case of single H2.5, there is a small eccentricity in the resistance offered by the strap, which resulted in the strap tear. The load–displacement response of single H2.5 that failed due to strap tear and top plate is depicted in Fig. 15 a. and 10 b. respectively.

Trip – L – Grip metal clip is a pre bent fastener used to attach the rafter/ceiling joist with the top plate. The metal clip uses 2 nails to attach to the rafter/ceiling joist, 2 nails to connect to the top side of the top plate and 4 nails to fasten to the inner side of the top plate. However, in the present case, four nails connecting the metal clip to the side of the top plate were found missing. The top plate to rafter/ceiling joist connecting unit, therefore, comprised of two toenails and four metal clip fastener nails (two attaching to the rafter/ceiling joist and two attaching to the top side of the top plate).
Figure 7.15. Typical response of connection which failed due to (a) Strap tear (b) Top plate split

7.6.2 Trip – L – Grip metal clip connection

Figure 7.16. Failure by a) nail withdrawal b) yielding of nail and nail pull out
Sixty five metal clip connections were harvested from two thornhill residential structure and were tested in the laboratory. The mean uplift capacity was estimated to be 2.47 kNs (555 lbs) with a COV of 0.35. Four types of failure modes were observed including 1) withdrawal of nails (Fig. 7.16(a)) 2) failure due to nail pull out and wood split (Fig. 7.17(a)) 3) yielding of nail along with nail pull out (Fig. 7.16(b)) and 4) Metal clip failure (Fig. 7.17(b)). Majority of the connections failed due to nail withdrawal marking it as the dominant mode of failure. Fig. 7.18(a) shows the response of metal clip connection that failed due to nail withdrawal. The response of metal clip connection that failed due to wood split and nail withdrawal was captured in Fig. 7.18(b). The sudden drop in the capacity of the connection as shown in Fig. 7.18(b) is due to the brittle failure of the wood.

Table 7.1 compares uplift capacities of Trip – L – Grip metal clip, toenails and retrofits such as single H2.5, double H2.5, and H1.0. Among the retrofits, single H2.5

Figure 7.17. a) Failure by nail withdrawal and wood split b) Metal clip failure
performs better than H1.0. Double H2.5 has only 35% higher uplift capacity than single H2.5 as against the expected 100% increase. The possible reason for this reduction in expected capacity is the mode of failure of the double H2.5 – all the connections retrofitted using two H2.5s failed due to top plate split. When retrofitted with two H2.5 the connections become stiffer and stronger than single H2.5 connections and therefore the failure of the metal strap (strap tear) becomes highly unlikely. As a result failure occurs at the weakest member i.e., at the top plate. The tearing strength of the wood member is considerably less than that of the metal connector resulting in strength reduction of the connection. Hence it can be inferred that no significant advantage is gained by using H2.5s on either side of the joist in place of a single H2.5. Double H2.5 and H1.0 have higher COV than single H2.5. Both double H2.5 and H1.0 fail by the splitting of wood top plate. The highly uncertain nature of wood and the undefined effect

Figure 7.18. Typical response of connection which failed due to (a) Nail withdrawal (b) Combination
of aging on its capacity is the reason for the higher COV compared to H2.5 whose dominant mode of failure is by strap tear.

Table 7. 1 compares the field and lab tested uplift strength of retrofit. The lab tested H2.5 has 37% higher uplift strength and lesser COV. While the lesser COV is predictable due to the precise fabrication of connections, selective wood pieces and qualified workmanship, the probable reason for increased capacity is rather unclear. The influence of age – fatigue and deterioration - on the capacity of in-situ connections is the likely suspect for the strength reduction. Table 7. 1 clearly highlights the advantage of using retrofitted connection over the traditional toenail connections. Single H2.5 has 3.5 times the capacity of two nail toenail connections and 2.5 times the uplift strength 3 nail toenail connection. Similarly double H2.5s and H1.0 have uplift capacities appreciably greater than 2-16d and 3-16d nail capacities.

Uplift wind pressure acting on each of the retrofit connections was calculated based on the dimensions of the baseline structure and the obtained uplift capacity. The evaluated uplift pressure was 3.68 kPa (76.8 psf), 3.25 kPa (67.97 psf) and 4.96 kPa (103.552 psf) for single H2.5, H1.0 and double H2.5 respectively. Applying a factor of safety of 2 to this calculated wind pressures, the design uplift pressures were 1.84 kPa (38.4 psf), 1.63 kPa (33.98 psf) and 2.48 kPa (51.78 psf). It is evident from the design uplift pressures that the retrofitted connections are safe against wind pressures generated during high wind storms. Further analysis clearly demonstrated the safety of these retrofit connections even during Category 3 hurricanes (wind speed -130mph).
Table 7.1. Comparative table of uplift capacities.

<table>
<thead>
<tr>
<th>Type of connection</th>
<th>No. of specimen</th>
<th>Average Ultimate capacity kN ( lbs)</th>
<th>COV</th>
<th>Study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toenail</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 – 16d</td>
<td>81</td>
<td>1.52 (342)</td>
<td>0.35</td>
<td>[4]</td>
</tr>
<tr>
<td>3 – 16d</td>
<td>19</td>
<td>1.98 (445)</td>
<td>0.37</td>
<td></td>
</tr>
<tr>
<td>Retrofit connections</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H 2.5-one</td>
<td>32</td>
<td>5.34 (1200)</td>
<td>0.20</td>
<td>Present study</td>
</tr>
<tr>
<td>H2.5 - two</td>
<td>32</td>
<td>7.20 (1618)</td>
<td>0.24</td>
<td>(Insitu)</td>
</tr>
<tr>
<td>H1.0</td>
<td>16</td>
<td>4.72 (1062)</td>
<td>0.24</td>
<td></td>
</tr>
<tr>
<td>Trip- L-Grip metal clip</td>
<td>65</td>
<td>2.47 (555)</td>
<td>0.35</td>
<td>Present study</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(Laboratory)</td>
</tr>
<tr>
<td>Retrofit connection</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Small metal strap</td>
<td>19</td>
<td>7.20 (1640)</td>
<td>0.10</td>
<td>[2]</td>
</tr>
<tr>
<td>(similar to H2.5)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 7.19. Proper installation of Trip – L- Grip metal clip connector.
Factor of safety (FOS) is obtained by dividing the field tested uplift capacity of retrofits from the present study by the capacity from the manufacturer’s specification – single H2.5 has FOS of 2.9 and H1.0 has 2.3. This factor of safety can be further improved by installing the retrofits on the outside of the wall.

The uplift capacity of metal clip connection, toenail connection and retrofit connections (H2.5, H1) can be compared using Table 7.1. This comparison is crucial in highlighting the need for proper installation of hurricane ties. As mentioned earlier, the Trip – L – Grip metal clip should use 2 nails to connect to the top side of the top plate, 2 nails to attach to the ceiling joist and four nails to fasten to the side of the top plate. Fig. 7.19 shows the correct method of installation of Trip – L – Grip metal clips. However metal clip connections in the thorn hill houses, lacked the four nails that connected the metal clip to the side of the top plate.

The capacity of the Trip – L – Grip metal clips was way lower than retrofit connections and only slightly higher than the toenail connections. The capacity of single H2.5 and H1.0 was almost double that of the Trip – L – Grip metal clips and double H2.5s had thrice the capacity of the metal clips. While the field tested toenail connections had capacities 62%-78% lower than the metal clips, the lab tested ones had capacities almost equal to the metal clips. Though the role of aging in the reduction of capacity cannot be over ruled, it is the lack of nails on the metal clip that proved detrimental to the uplift strength of metal clip. The negligence on the part of construction workers and supervisors to use prescribed number of nails to connect the metal clip to the side of the top plate resulted in the significant decrease in the uplift capacity. The high
COV of trip – L – Grip metal clips can be attributed the different failure modes of the connections and to the effect of aging on the wood and metal clip connections. The capacity varied significantly with each failure mode resulting in a wide spread of data and a high COV.

7.6.3 Stiffness:

The initial stiffness of both H2.5 and Trip – L – Grip metal connection was calculated as the secant stiffness at 1.6 mm (0.0625 inch) in consistent with a previous work. The motivation for this selection is that the stiffness is approximately linear in this range and the obtained load displacement data is fairly stable over this range, as compared to other displacements considered (0.254 mm, 3.2 mm). The secant stiffness at 0.254 mm and 3.2 mm were unfit for effective stiffness calculation due to either instability of data or non-linearity in the range considered.

The mean effective initial stiffness of eighteen H2.5 retrofit connection was evaluated to be 1.61 kN/mm (9199 lbs/in.) with a COV of 0.31. Fifty nine Trip – L – Grip connections were considered for the estimation of initial effective stiffness. The average initial stiffness is 1.06 kN/mm (6061 lbs/in.) and the COV is 0.50. (Table 7. 2)

7.6.4 Probability Models

Probability distribution model of uplift capacity is essential for holistic development of loss calculation and damage prediction models of light frame wood structures subjected to hurricanes. Competent distribution fits can be identified only if valid and sufficient data are available. Statistically significant information on uplift capacity and stiffness of retrofit and metal clip connections collected in the present study enabled the
identification of distribution fits and parameters. Goodness of fit tests was used to ascertain the plausibility of underlying distribution of uplift capacity and stiffness. Kolmogrov-Smirnov GOF test with a 5% level of significance was employed to check the feasibility of various distribution fits. Method of maximum likelihood was used to estimate the distribution parameters. In addition Anderson–Darling (AD), a GOF test that is sensitive to data in the tails, was also employed to further confirm the findings from K-S test.

**Table 7.2.** Comparative table for stiffness

<table>
<thead>
<tr>
<th>Type of connection</th>
<th>No. of specimen</th>
<th>Average stiffness kN/mm (lbs/in)</th>
<th>COV</th>
<th>Study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toenail</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 – 16d</td>
<td>75</td>
<td>0.45 (2590)</td>
<td>0.26</td>
<td>[4]</td>
</tr>
<tr>
<td>3 – 16d</td>
<td>19</td>
<td>0.55 (3159)</td>
<td>0.33</td>
<td></td>
</tr>
<tr>
<td>Trip- L-Grip metal clip</td>
<td>59</td>
<td>1.06 (6061)</td>
<td>0.50</td>
<td>Present study (Laboratory)</td>
</tr>
<tr>
<td>Retrofit connections</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H 2.5-one</td>
<td>18</td>
<td>1.61 (9199)</td>
<td>0.31</td>
<td>in-situ study</td>
</tr>
</tbody>
</table>

P-value from the GOF tests control the choice of distribution fit. If the p-value of a distribution is greater than the adopted level of significance, then that distribution is a plausible fit. Higher the value of p, stronger this statement becomes. Distributions like Normal, lognormal, gamma and extreme value were considered for the best fit study.
Finally lognormal distribution was identified as best fit for both uplift capacity and stiffness, from K-S and AD GOF tests. The uplift capacity and stiffness of retrofit H2.5 connection and Trip – L – Grip connection can therefore be expressed as jointly lognormal. Table 7. 3 lists the p-value from K-S and AD GOF tests for lognormal distribution.

The behavior of roof to wall connection is a function of uplift capacity and stiffness, both of which are interrelated. Correlation coefficient is an important parameter to measure the dependence between uplift capacity and stiffness. Knowledge on correlation coefficient becomes imperative to completely describe a joint probability distribution if the means and variances of the dependent variables are already known. Hence the correlation coefficient between the uplift capacity and stiffness of roof to wall connections is provided herein. Uplift capacity and stiffness of H2.5 retrofit connections were found to be positively correlated with a value of 0.70. Similarly for Trip – L – Grip
connection the two connection parameters were correlated positively with a value of 0.757. Table 7.4 provides the correlation between the natural logarithms of uplift capacity and stiffness for the sake of convenience.

Table 7.4. Maximum likelihood estimates of probability distribution parameters.

<table>
<thead>
<tr>
<th>Connection</th>
<th>Uplift capacity kN (lbs)</th>
<th>Correlation</th>
<th>Stiffness kN/m (lb/inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dist.</td>
<td>λ</td>
<td>ζ</td>
</tr>
<tr>
<td>H 2.5 – Single</td>
<td>LN</td>
<td>1.656</td>
<td>0.198</td>
</tr>
<tr>
<td>H 1.0</td>
<td>LN</td>
<td>1.524</td>
<td>0.236</td>
</tr>
<tr>
<td>H 2.5 – double</td>
<td>LN</td>
<td>1.946</td>
<td>0.236</td>
</tr>
<tr>
<td>Trip – L – Grip</td>
<td>LN</td>
<td>0.846</td>
<td>0.340</td>
</tr>
<tr>
<td>Metal clip</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Correlation is measured between the natural logarithm of both the uplift capacity and stiffness.

7.7 CONCLUSION

The in-situ capacity of retrofitted hurricane ties were evaluated and compared with laboratory values. The lab uplift capacity values were found to be higher than the in-situ capacity. In addition the variability of the laboratory data was lesser than the field tested values of hurricane ties. Aging of wood and variability associated with field construction practices were found to be the reason for the difference in both the uplift capacity and COV. Lognormal probability distribution was found to best describe both the uplift capacity and initial stiffness for H2.5 metal ties. Laboratory testing of Trip – L – Grip
metal connectors revealed that the uplift capacity and initial stiffness followed lognormal distribution. It was also found that the improper installation of the metal clip resulted in reduction in the mean uplift capacity. The results from the current research can be used in roof to wall connection fragility estimation of retrofitted structures and old buildings with metal clip connections.

7.8 ACKNOWLEDGEMENT

The testing on Douthill houses was supported by National Science Foundation (NSF) under award number CMS-0642455 and the Thornhill project was given as a sub award through University of Florida by Florida’s Department of Community Affairs. The authors gratefully acknowledge the generous funding and support of NSF and Florida’s Department of Community Affairs. The authors would like to acknowledge the guidance and encouragement provided by Dr. Scott Schiff for this project. The views expressed in this paper are solely that of the authors and do not reflect the opinion of NSF or Florida’s Department of Community affairs.

7.9 References

