SEARCHING FOR THE FACTOR OF SAFETY: A STUDY OF WOOD ROOF SYSTEM CAPACITY AS IT APPLIES TO SOLAR PHOTOVOLTAIC INSTALLATION

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SEARCHING FOR THE FACTOR OF SAFETY:
A STUDY OF WOOD ROOF SYSTEM CAPACITY AS IT APPLIES TO SOLAR
PHOTOVOLTAIC INSTALLATION

BY

ALFRED DAVID SANCHEZ III

Bachelor of Business Administration
University of New Mexico 2004

THESIS

Submitted in Partial Fulfillment of the
Requirements for the Degree of

MASTER OF SCIENCE

Civil Engineering

The University of New Mexico
Albuquerque, New Mexico

May, 2014
DEDICATION

To my newborn son:

Follow your heart.
Trust your instincts and never stop exploring.
ACKNOWLEDGEMENTS

My sincerest gratitude to Dr. Walter Gerstle, your help and mentorship has been beyond measure and I aspire to your example. I would also like to thank Dr. Stephen Dwyer of, Sandia National Labs, for your trust and insight into our shared research objectives. Furthermore Brian Dwyer, David Modisette, and Raybeau Richardson, you have all unselfishly donated to this research and for that I am forever grateful.

Throughout this research and my graduate course work I have been continuously inspired by many of my professors. Dr. Arup Maji has been one such professor that has been of great inspiration, I thank you for your commitment to your craft and your continuous help throughout my coursework. I would also like to thank Dr. Tim Ross for his continued support throughout my course work and the expertise that he has brought to my thesis committee.

Finally, my wife, Kira J. Carbonneau, I never imagined that we could achieve what we have and I can’t imagine the journey we are about to begin. I do know that I couldn’t do any of it without you. Thank you.
SEARCHING FOR THE FACTOR OF SAFETY:
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INSTALLATION

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ABSTRACT

The recent drive to install residential installation of photovoltaic roof systems has generated the need to re-examine the structural capacity of residential structures. Currently, a perspective exists that residential wood roofs may not be able to carry the additional load of a photovoltaic (PV) array. This research seeks to address how the addition of PV installations can affect the structural capacity of residential wood roof systems and to investigate the current methods in which roof systems are assigned their working strengths.

With the goal of finding ultimate capacities of wood roof systems, empirical tests were conducted in a laboratory setting and results were compared with current building and design codes. These comparisons help to identify factors of safety and other assumptions that exist in current building codes and therefore elicit a better understanding of the structural capacity of wood roof systems. Finally, a computer analysis was conducted to better understand the way in which a PV installation affects the structural performance of wood roof systems.
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CHAPTER 1

Introduction

The recent drive to install solar photovoltaic roof systems has generated the need to evaluate the structural capacity of residential roof structures. Currently, a perspective exists that residential wood roofs may not be able to carry the additional dead load that is incurred with the installation of a photovoltaic (PV) array. This study examines the structural capacity of existing and new residential roof systems and will help to quantify the load that a roof system can support. It is hypothesized that current roof structures can support significantly more load than the International Residential Building Code (IRC) allows. The unclear nature of the IRC’s assumed factor of safety has caused uncertainty in determining the ability of residential roof system to carry the additional loading associated with PV installations.

In exploring this hypothesis, laboratory tests were conducted using full scale physical models of roof panels subjected to distributed loading. It was also initially planned that testing be conducted with and without the installation of PV panels; however the installation methods for PV vary considerably. Therefore it was concluded that full scale testing be limited to typical roof assemblies without the installation of PV panels and computer simulation be used to understand how roof assemblies perform with various means of PV installation.

As a method of applying load to a roof panel, a uniformly distributed loading condition was chosen due to its ability to best represent in situ snow loading conditions. It was decided that a uniformly distributed load applied gradually best represents the typical loading condition that a roof assembly will encounter within its service life. The tested
roof panels thus closely represent typical residential roofs. Previous research indicates that wood roof systems exhibit partial composite action amongst their structural components (Campos 2013). Due to this partial composite action, a nonlinear interaction is created between structural components, therefore not allowing the use of superposition when analyzing structural components. In consideration of this complex behavior, roof assemblies were tested as systems rather than testing the structural components individually and then analyzing systems using conventional methods.

To better understand the building code used during construction of homes the 2009 International Building Code as it pertains to joist spans and roof assemblies was reviewed. In examining the International Building Code it became evident that the code is prescriptive in style and does not account for designs based on material properties and site-specific conditions. Given the prescriptive nature of the code, the factor of safety is not specified and assumptions regarding behavior of system components are not offered. Ultimately, this lack of information prevents the designer from truly knowing and understanding the capacity of a roof system. This research elucidated the assumptions that exist in the current building codes and helped us to relate those assumptions to empirical testing to gain a better understanding of the structural behavior of wood roof systems.

By developing a clearer understanding of wood roof capacity, the PV industry can approach roof top installations with the knowledge of how a PV rooftop installation affects the structural capacity of a roof system. Such knowledge can then be used to develop guidelines for the public and ultimately streamline the process for rooftop PV installations. Such streamlined approaches to PV installation would decrease overall cost.
and increase national energy independence. As PV becomes more popular within the United States, local building officials are increasingly burdened with the task of issuing permits for installations on pre-existing roof structures. Often, building officials have little choice but to require a professional engineer to certify the structural capacity of roof systems prior to PV installation. Such certifications by professional engineers are both costly and time consuming and often result in civil engineers using their ‘engineering judgment’ rather than rigorous analytical modeling and concise research. By eliminating the need for structural engineering on many residential PV installations, the liability exposure to the engineer profession can be greatly reduced while eliminating a costly burden to the consumer.

In 2013, US PV installations accounted for more than 5,000 MW of new power generation (Solar Energy Industries Association, 2013). The associated current cost per installed watt of PV is approximately $3.00 to $4.43 (National Renewable Energy Laboratory, 2013; Solar Energy Industries Association, 2013), bringing the total US PV market share between 15 billion and 22.5 billion dollars. Out of all United States PV installations, the residential PV market accounts for approximately 16 percent of total power generation capacity (800 MW); however, residential installations account for 90% of all PV installations or 129,000 installations. The average size of residential installation is approximately 6.2 kW with an average cost of approximately $23,000 per residential installation (Interstate Renewable Energy Council, 2013). Therefore, the total cost of all United States residential installations falls between 2.4 billion and 3.5 billion dollars. Since 90% of all installations occur within the residential market segment, the influence of cost saving measures can affect a broad range of stakeholders.
Additionally, the current hourly rate of a structural engineer can be assumed to be $150, with an assumed four hour minimum charge of $600. It can be quickly deduced that if all 129,000 residential PV installation used a structural engineer, the total engineering cost for all installations would be equal to approximately 77 million dollars. Thus, the total cost saving due to the elimination of a structural engineer is a small percent saving. But the reduction of engineering cost to the consumer could allow an increase in total capacity of residential installations thus furthering national energy independence.
CHAPTER 2

Literature Review

2.1 Introduction to Literature Review

Under current design practice, residential roofs are usually designed using design tables included in the International Residential Code (IRC), or using allowable stresses provided by the National Design Standards (NDS). In either case, the factor of safety is not made explicit. The present study seeks to identify the factor of safety built into current code design specifications and compares the factor of safety to results from empirical testing. This literature review first provides a brief history of stress grading. Secondly, it reviews the adopted codes and testing standards associated with the International Residential Building Code (IRC), the National Design Standard (NDS), and the testing standards of ASTM International (ASTM), and lastly reviews research conducted at USDA Forest Service, Forest Products Laboratory.

2.2 History of Stress Grading

The visual stress grading of lumber has officially existed since the early part of the twentieth century when the United States Department of Agriculture (USDA) Forest Service Forest Products Laboratory published a set of basic rules with assigned stress values in 1923 (Galligan & McDonald, 2000). During World War II these assigned stress values were increased by 85% as a result of the United States Army dictating an increase to initial design values as a consequence of the war effort. After the war ended some of the changes made by the military became permanent design values. The changes made by the military and demand for lumber created constant changes to the lumber grading system and therefore uncertainty in the design values. To aid the process of creating
standard design values and increase confidence in these values, changes to visual grading procedures came with the adoption of American Lumber Standards (ALS) PS 20-70. The standards set by the ALS, brought recognition to several factors such as moisture content and shrinkage that influence grading. Under the ALS, a National Grading Rule was developed (Galligan & McDonald, 2010).

The newly developed grading rule established uniform grading methods that could be applied to all species of lumber. While standardized grading rules now existed, the need to verify baseline design values was becoming increasingly important. In 1977, the North American In-Grade Testing Program went into effect, in hopes of standardizing design values with the use of proof testing of full size samples. As a consequence of such testing, the current visual grading system can claim to be based on empirical full-scale testing. Due to these empirical tests, changes to historical design values were made.

Concurrent with the standardization of visual grading of lumber, a new method of machine rating was gaining acceptance within the lumber industry. This new method of machine rating made use of an observed statistical correlation between stiffness and strength that was found to exist in all species of wood. By employing this non-destructive machine testing to find a modulus of elasticity (i.e. stiffness), the method of machine rating was also able to assign an associated strength or stress grade. As of 1996, the amount of machine stress rated lumber produced in the United States had increased from insignificant levels of production to 1.1 billion board feet annually (Galligan & McDonald, 2010). Machine stress rated lumber reached an all time high in 2005, with an estimated production of almost 3 billion board feet (Logan, Allen, Uskoski, & Nelson, 2010). Currently, it is becoming increasingly difficult to acquire purely visually rated
dimensional lumber as machine rated lumber allows for higher efficiency in lumber production and is used almost exclusively.

2.3 International Residential Building Code (IRC)

One of the most commonly adopted building codes in the United States is the International Residential Building Code (IRC). The IRC is authored by the International Code Council (ICC), which was founded in 1994 by the merger of several regional councils to form a “comprehensive and coordinated national model of construction codes” (ICC, 2013 p.7). The founding members of the IRC include three regional councils: 1) the Building Officials and Code Administrators International, Inc. (BOCA), used throughout the east coast and the Midwest portions of the U.S., 2) the International Conference of Building Officials (ICBO), whose model building code was used in the western U.S. and; lastly, 3) the Southern Building Code Congress International, Inc. (SBCCI) whose code was implemented in the southern region of the country. Predating the ICC, the establishment of building codes was the responsibility of these three regional councils and local governments were encouraged to adopt the building codes of the council nearest in proximity. While the ICC publishes building codes based upon these three regional councils, a United States governmentally-mandated building code does not officially exist. All fifty states and incorporated municipalities are allowed to adopt codes of their own choosing; however, most municipalities have partially or fully adopted the IRC codes put forth by the ICC.

One resource within the IRC are the span tables as shown in Table I, which presents an example of a span table produced by the IRC. Span tables allow users to
<table>
<thead>
<tr>
<th>Ceiling Joist Spacing (inches)</th>
<th>Species and Grade</th>
<th>Maximum Ceiling Joist Spans (Feet-Inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Douglas fir-Larch SS</td>
<td>13-2 20-8 - -</td>
</tr>
<tr>
<td></td>
<td>Douglas fir-Larch #1</td>
<td>12-8 19-11 - -</td>
</tr>
<tr>
<td></td>
<td>Douglas fir-Larch #2</td>
<td>12-5 19-6 25-8 -</td>
</tr>
<tr>
<td></td>
<td>Douglas fir-Larch #3</td>
<td>10-10 15-10 20-1 24-6</td>
</tr>
<tr>
<td></td>
<td>Hem-fir SS</td>
<td>12-5 19-6 25-8 -</td>
</tr>
<tr>
<td></td>
<td>Hem-fir #1</td>
<td>12-2 19-1 25-2 -</td>
</tr>
<tr>
<td></td>
<td>Hem-fir #2</td>
<td>11-7 18-2 24-0 -</td>
</tr>
<tr>
<td></td>
<td>Hem-fir #3</td>
<td>10-10 15-10 20-1 24-6</td>
</tr>
<tr>
<td></td>
<td>Southern pine SS</td>
<td>12-11 20-3 - -</td>
</tr>
<tr>
<td></td>
<td>Southern pine #1</td>
<td>12-8 19-11 - -</td>
</tr>
<tr>
<td></td>
<td>Southern pine #2</td>
<td>12-5 19-6 25-8 -</td>
</tr>
<tr>
<td></td>
<td>Southern pine #3</td>
<td>11-6 17-0 21-8 25-7</td>
</tr>
<tr>
<td></td>
<td>Spruce-pine-fir SS</td>
<td>12-2 19-1 25-2 -</td>
</tr>
<tr>
<td></td>
<td>Spruce-pine-fir #1</td>
<td>11-10 18-8 24-7 -</td>
</tr>
<tr>
<td></td>
<td>Spruce-pine-fir #2</td>
<td>11-10 18-8 24-7 -</td>
</tr>
<tr>
<td></td>
<td>Spruce-pine-fir #3</td>
<td>10-10 15-10 20-1 24-6</td>
</tr>
</tbody>
</table>

|                               | Douglas fir-Larch SS | 10-5 16-4 21-7 - - | 24 |
|                               | Douglas fir-Larch #1 | 10-0 15-9 20-1 24-6 | |
|                               | Douglas fir-Larch #2 | 9-10 14-10 18-9 22-11 | |
|                               | Douglas fir-Larch #3 | 7-8 11-2 14-2 17-4 | |
|                               | Hem-fir SS          | 9-10 15-6 20-5 - | |
|                               | Hem-fir #1          | 9-8 15-2 19-7 23-11 | |
|                               | Hem-fir #3          | 9-2 14-5 18-6 22-7 | |
|                               | Southern pine SS    | 10-3 16-1 21-2 - | |
|                               | Southern pine #1    | 10-0 15-9 20-10 - | |
|                               | Southern pine #2    | 9-10 15-6 20-1 23-11 | |
|                               | Southern pine #3    | 8-2 12-0 15-4 18-1 | |
|                               | Spruce-pine-fir #1  | 9-5 14-9 18-9 22-11 | |
|                               | Spruce-pine-fir #2  | 9-5 14-9 18-9 22-11 | |
|                               | Spruce-pine-fir #3  | 7-8 11-2 14-2 17-4 | |

Table I. Span table adapted from the IRC
choose from several species of dimensional lumber and from several dead and live load combinations to determine the required lumber dimension for a given span. The IRC also takes into account the spacing between joists, ‘rafter spacing’, when determining a required span length. With regard to the rafter spacing, the IRC allows users to choose between four values of rafter spacing: 12”, 16”, 19.2”, and 24” on center. The four species of lumber listed by the IRC include douglas fir-larch, hem-fir, southern pine, and spruce-pine fir. Variations in allowable spans also take into account various grades of lumber ranging from SS (select structural), #1, #2, and #3. In the IRC, the spans of dimensional lumber are limited to two dead loading situations; 10 psf and 20 psf. The IRC also provides four live load conditions 20 psf, 30 psf, 50 psf, and 70 psf. As shown in Table I, if a user wished to specify a joist that could accommodate a 10 psf dead load with a 20 psf live load, spanning 23 feet, with a 24 inch on center spacing, the code would specify a 2 x 10 Hem-fir #1, or a 2 x 10 Southern pine #2.

Due to the prescriptive nature of the IRC span tables, the question arises as to how the authors arrived at their prescribed spans and what was the presumed factor of safety while developing the tables. An investigation of the factor of safety built into the IRC span tables highlights the lack of any documented factor of safety explicitly or implicitly stated within the code. Although there is no stated factor of safety in the IRC, there is a reference that credits the span tables to another organization, the American Forest & Paper Association (AFPA). In 1944 the AFPA, also known as the American Wood Council, put forth an additional set of standards for building known as the National Design Specification (NDS 2012).
2.4 National Design Specification (NDS)

While the IRC is a code of prescribed requirements in tabular form, geared mainly to an audience outside the engineering profession, the NDS due, to its numerically specific and adjustable design values allows users more specificity in designing members. The NDS is the code most commonly utilized by engineers. As shown in Table II, adjustment factors are utilized to adjust baseline design values to better suit in situ conditions. In order to adjust the baseline allowable properties of lumber, the NDS utilizes a table that helps users gather applicable factors and apply them to a base design value. The development of a design value based on the adjustment factor approach is shown in Table II.

Table II. Applicability of adjustment factors for sawn lumber adapted from NDS.

<table>
<thead>
<tr>
<th></th>
<th>ASD</th>
<th>ASD and LRFD</th>
<th>LRFD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Duration Factor</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wet Service Factor</td>
<td>C_D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Temperature Factor</td>
<td>C_M</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam Stability Factor</td>
<td>C_L</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Size Factor</td>
<td>C_F</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat Use Factor</td>
<td>C_fu</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Incising Factor</td>
<td>C_i</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Repetitive Member Factor</td>
<td>C_r</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column Stability Factor</td>
<td>C_F</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Buckling Stiffness Factor</td>
<td>C_F</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bearing Area Factor</td>
<td>C_F</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Format Conversion Factor</td>
<td>C_F</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance Factor</td>
<td>C_F</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Time Effect Factor</td>
<td>C_F</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[
F_b' = F_b \times C_D \times C_M \times C_L \times C_F \times C_i \times C_r \times K_F \times \phi_b \times \lambda \\
F_t' = F_t \times C_D \times C_M \times C_l \times C_F \times C_i \times K_F \times \phi_t \times \lambda \\
F_v' = F_v \times C_D \times C_M \times C_l \times C_F \times C_i \times K_F \times \phi_v \times \lambda \\
F_c' = F_c \times C_D \times C_M \times C_l \times C_F \times C_i \times C_p \times K_F \times \phi_c \times \lambda \\
E' = E \times C_M \times C_l \times C_F \times C_i \times K_F \times \phi_e \times \lambda \\
E_{min}' = E_{min} \times C_M \times C_l \times C_i \times C_T \times K_F \times \phi_e \times \lambda 
\]
The current NDS contains design values for both visually rated lumber and mechanically graded dimensional lumber. For visually graded lumber the NDS contains 29 different species of wood and six corresponding design values for each species. Such design stress values include fiber bending ($F_b$), tension parallel to grain ($F_t$), shear parallel to grain ($F_s$), compression perpendicular to grain ($F_c$), compression parallel to grain ($F_{cp}$), and modulus of elasticity ($E$). Mechanically graded lumber has tables like those for visually graded lumber; however rather than listing values for every species, the tables for mechanically graded lumber ignore species type and simply list grades that correspond to mechanically determined values of $E$ and $F_b$.

While mechanically-graded lumber tables still utilize adjustment factors to arrive at design values, there is an implicitly generated grade name that is representative of a presumably mechanically derived $E$ and $F_b$ value. For example, the machine stress rated grade name of 900f-1.0E corresponds to $F_b = 900$ psi and $E = 1,000,000$ psi, $F_t = 350$ psi, and $F_c = 1050$. Design values for machine stress graded lumber rely upon grade types that are presumably found from a machine test. Variations due to species of lumber are not directly addressed within the NDS; however the notes associated with the design tables state:

for any given bending design value, $F_b$, the modulus of elasticity $E$, and tension parallel to grain, $F_t$, design value may vary depending upon species, timber source or other variables. The “$E$” and “$F_t$” values included in the “$F_b$-E” grade designations in Table 4c are those usually associated with each $F_b$ level. Grade stamps may show higher or lower values if machine rating indicates the assignment is appropriate (NDS, 2012 p. 43).

This note in the design tables casts doubt on the accuracy of the design tables and allows properties to be changed, presumably based upon the judgment of machine rating operators and managers. Further doubt is cast upon the accuracy of the design values due
to an additional note that indicates “the gain in load carrying capacity due to increased strength and stiffness resulting from drying more than offsets the design effect of size reductions due to shrinkage” (NDS Supplement, 2012 p.43). This statement highlights that the effect of shrinkage is neglected and that any change in cross sectional area is more than counterbalanced by increases in capacity due to drying. The phrase “more than offsets”, does not quantify the gains in strength due to drying. Beyond this statement, the NDS provides no further explanation as to the increase in capacity due to drying effects.

Although the NDS addresses many different properties of wood, the present study mostly pertains to wood properties associated with bending. Due to this focus, the NDS was examined with the specific interest in fiber bending strength ($F_b$). One of the overarching factors affecting the strength of a joist and therefore the strength of a roof system is the system itself. The ability of a system to resist more load than the sum of its individual components is referred to within the industry as system effects. Due to these system effects the NDS allows users to increase the load carrying capacity of a joist if it is a member of a composite assembly. The increase in capacity due to system effects is represented by a ‘repetitive member factor’ ($C_r$), and provides an increase to allowable design values of 15% if the joists meet specific requirements. These requirements are stated as follows:

bending design values $F_b$, for dimensional lumber 2” to 4” thick shall be multiplied by the repetitive member factor $C_r = 1.15$, when such members are used as joist, truss cords, rafters, studs, planks, decking or similar members which are in contact or spaced not more than 24 on center, are not less than 3 in number and are joined by floor, roof or other load distributing elements adequate to support the design load (NDS 2012).

It is interesting to note that the repetitive member factor $C_r$ is a factor that is not influenced by any observed or measurable characteristic of sawn lumber; but rather the
increase in allowable capacity is based solely on the geometric properties of the assembly, which can supposedly resist more loading than the sum of individual components.

Due to the fact that the NDS allows for variation of more factors, it far surpasses the IRC in terms of completeness in allowing its users the ability to determine the design values that best reflect in situ conditions. Although the NDS allows for greater flexibility than that of the IRC, it continues to lack a stated value for the nominal factor of safety. Furthermore, the NDS casts doubt upon both the accuracy and finality of design values by allowing offset of un-quantified strength losses, due to shrinkage, with supposedly greater un-quantified strength gains, due to drying.

Ultimately the NDS provides valuable design information for a wide array of various usages and types of lumber; however in our quest to quantify a factor of safety the NDS lacks information. Although the NDS does not provide explicit factors of safety, it does refer users to the ASTM standards and the North American In-Grade Testing Program. The commentary of the NDS section 4.2.3.2 states:

Changes in the 1991 NDS to dimension lumber design values are based on a comprehensive testing program conducted by the North American forest products industry called In-Grade Testing…. A new test method standard, ASTM D4761, was developed to cover the mechanical test methods used in the program. A new standard practice, ASTM D1990, was developed to codify procedures for establishing design values for visually graded dimension lumber from test results obtained from in-grade test programs (NDS, 2013).

This new insight into the genesis of design values leads us to investigate the testing procedures and standards that have been published by the ASTM wood subcommittee D07 and to investigate the North American In-Grade Testing Program.
2.5 ASTM International

ASTM international publications have greatly influenced the field of structural lumber testing and current wood design standards. While ASTM once stood for American Society for Testing and Materials, the current organization does not recognize the acronym and is simply named ASTM International. The ASTM wood sub-committee (D07) is tasked with the responsibility of quantifying and documenting testing procedures. To fulfill this responsibility ASTM determines the procedures for establishing mechanical properties of all wood-based products. As earlier indicated, the NDS specifies some adjustment factors based on various characteristics of both the material and the systems; however the NDS does not specify how those factors were found but rather refers the users to ASTM standards. For example, the addition of a 15% upward adjustment due to repetitive-member performance, stated as appropriate by the NDS stems from ASTM Standard *Evaluating System Effects in Repetitive-Member Wood Assemblies*.

The ASTM D6555-03 standard recognizes an increase in load-carrying capacity due to three factors which include: load sharing, composite action, and residual capacity. Within this standard, a method for quantifying system effects using empirical test results is presented. The ASTM standard indicates that at least 28 assemblies need to be tested in order to quantify system effects (ASTM D6555 Section 8.3). The sample size of 28 specimens stems from ASTM Standard D2915 titled “*Standard Practice for Evaluating Allowable Properties for Grades of Structural Lumber.*” ASTM D 2915 seeks to identify grade assignments based on empirically derived mechanical properties found during the
testing of representative samples. By this standard, a lumber grade can be established which is probabilistically representative of a sample population through the use of statistics. Due to this representation ASTM D2915 allows small sample sizes for empirical testing, thus increasing efficiency for both visually and mechanically graded lumber.

To establish a grade, empirical testing is conducted on a sample size that is representative of the total population, which ASTM has established at a lower bound of 28. An example of this process is shown in Figures 1 and 2. After testing is completed, a regression line to the data is determined. The regression line is then shifted downward to ensure that 95% of the data points fall above the regression line. This new offset regression line is then said to be indicative of the population and cutoffs can be established to represent different grades within the entire population.

![Figure 1. Example of prediction of strength by regression analysis](image-url)
Figure 2. Example of the typical relationship between strength predictor (MOE) and strength (MOR). Regression line is shifted downwards to below 95% of the data.

In addition to ASTM Standard D2915, machine stress rated lumber is assigned design values using ASTM Standard D6570 *Standard Practice for Assigning Allowable Properties for Mechanically Graded Lumber*, which includes factors aimed at addressing multiple scenarios and factors including: multiple-member systems, normal duration of load, growth ring position, moisture content, size factors, different than normal duration of load, decay, treated wood, temperature, and bearing areas. In addition to discussing these factors and scenarios this ASTM standard helps to allow non-destructive rating of lumber by relating a physically found modulus of elasticity to a hypothetically correlated modulus of rupture. This hypothetical correlation between stiffness and bending strength is the basic assumption in nondestructive testing.

In the 1960s the correlation between Modulus of Elasticity (MOE) and Modulus of Rupture (MOR) had been recognized, and the lumber rating industry began to develop machines that could quickly test individual pieces of lumber. More recent development of
these machines incorporates components that not only determine MOE values but also automatically inspect for visual characteristics such as knots and grain pattern using optical scanners. These characteristics also influence final grade assignments. Due to the widespread acceptance of mechanically graded lumber beginning in the 1970s, the vast majority of all dimensional lumber available today is machine stress rated.

The correlation presented in ASTM Standard D6570 between MOE and MOR provides an efficient and accurate assignment of grades; however it does not provide explicit information concerning the factor of safety that is built into the grading system. In the continued search for an established underlying factor of safety, additional information can be located in ASTM Standard D245 *Standard Practice of Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber*. Within this standard the method of establishing allowable properties is addressed in Section 6.2, which indicates, “properties when divided by the factors given in Table 8 give the respective allowable design properties for clear straight-grained wood. The factors include an adjustment for normal duration of load and a factor of safety”. Table III is an example of adjustment factors provided by ASTM D245 Table 8.
Table III. Adjustment factors to be applied to the clear wood properties provided by ASTM. (Adapted from ASTM D245 Table 8)

<table>
<thead>
<tr>
<th></th>
<th>Bending Strength</th>
<th>Modulus of Elasticity in Bending</th>
<th>Tensile Strength Parallel to Grain</th>
<th>Compressive Strength Parallel to Grain</th>
<th>Horizontal Shear Strength</th>
<th>Proportional Limit and Stress at Deformation in Compression Perpendicular to Grain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Softwoods</td>
<td>2.1</td>
<td>0.94</td>
<td>2.1</td>
<td>1.9</td>
<td>2.1</td>
<td>1.67</td>
</tr>
<tr>
<td>Hardwoods</td>
<td>2.3</td>
<td>0.94</td>
<td>2.3</td>
<td>2.1</td>
<td>2.3</td>
<td>1.67</td>
</tr>
</tbody>
</table>

Additionally, ASTM D245 provides examples of Stress-Grade Development that clearly show how adjustment factors affect the overall design values of mechanically and visually rated lumber. Tables IV and V provide examples of how ASTM implements adjustment factors. ASTM standard D245 contains the first explicit mention of a factor of safety, which is an established factor of 2.1. However, this factor does not apply to all wood properties. As can be seen in Table III, safety factors vary in both property type and wood classification. It is important to note that any prescribed factor of safety is applied in addition to the statistical 5% exclusion limit. ASTM standard D245 also addresses the age of lumber and its working stress values, indicating that old lumber can be assigned the same working stress values as new lumber.
Table IV. Example of how ASTM implements adjustment factors for limiting characteristics.

<table>
<thead>
<tr>
<th>Property</th>
<th>Limiting Characteristic</th>
<th>Strength Ratio %</th>
<th>From Table</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>Narrow face knot = ¼ in</td>
<td>62</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Knot centerline of wide face = 2 3/8 in</td>
<td>60</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Knot at edge of wide face = 1 3/8 in</td>
<td>60</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Slope of grain 1 in 10</td>
<td>61</td>
<td>1</td>
</tr>
<tr>
<td>Compression strength parallel to grain</td>
<td>Knot on any face = 2 ½ in</td>
<td>65</td>
<td>3</td>
</tr>
<tr>
<td>Shear</td>
<td>Slope of grain 1 in 8</td>
<td>66</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Size of shake or check = ½ in</td>
<td>50</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Length of end split = 4 ¼ in.</td>
<td>50</td>
<td>1</td>
</tr>
</tbody>
</table>

Table V. Example of ASTM’s allowable properties for the sample stress-grade

<table>
<thead>
<tr>
<th>Property</th>
<th>Strength Value psi</th>
<th>Adjustment Factor</th>
<th>Strength Ratio</th>
<th>Seasoning Adjustment</th>
<th>Special Features</th>
<th>Allowable Property psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>4432</td>
<td>1/2.1</td>
<td>0.6</td>
<td>1.25</td>
<td>0.89</td>
<td>1400</td>
</tr>
<tr>
<td>Compression parallel to grain</td>
<td>2174</td>
<td>1/1.9</td>
<td>0.65</td>
<td>1.5</td>
<td></td>
<td>1100</td>
</tr>
<tr>
<td>Horizontal shear</td>
<td>576</td>
<td>1/2.1</td>
<td>0.5</td>
<td>1.08</td>
<td></td>
<td>150</td>
</tr>
<tr>
<td>Tension parallel to grain</td>
<td>4432</td>
<td>1/2.1</td>
<td>0.60x0.55</td>
<td>1.25</td>
<td></td>
<td>850</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>1304000</td>
<td>1/0.94</td>
<td>1</td>
<td>1.14</td>
<td></td>
<td>1580000</td>
</tr>
<tr>
<td>Compression Perpendicular A</td>
<td>282</td>
<td>1/1.67</td>
<td>1</td>
<td>1.5</td>
<td></td>
<td>255</td>
</tr>
<tr>
<td>Compression Perpendicular B</td>
<td>491</td>
<td>1/1.67</td>
<td>1</td>
<td>1.5</td>
<td></td>
<td>440</td>
</tr>
</tbody>
</table>

*A Compression perpendicular to grain for proportional limit stress.
*B Compression perpendicular to grain at 0.04 in (1 mm) deformation.
Out of the three entities providing recommendations to the construction industry, ASTM standards are the only set of guidelines that provide an explicit factor of safety. In addition to providing a factor of safety, the ASTM standards provide insight into how grades are assigned using both the correlation between MOE and MOR and visual inspection. ASTM also has increased the efficiency of grading lumber by setting standards associated with empirically testing small samples of wood species to gain knowledge about the larger population.

2.6 USDA Forest Service, Forest Products Laboratory

During the middle of the 20th century, a need developed within the United States lumber industry to quantify and verify the mechanical properties of various species of 2-in thick dimensional lumber. During that time frame the bulk of lumber sold in the U.S. was visually graded, and although the development of machine stress grading standards had already been accomplished, its industry-wide integration had not yet been realized. In 1977, in order to verify mechanical properties and further the accuracy of machine stress grading the USDA Forest Service, Forest Products Laboratory, implemented the North American In-Grade Testing Program that included:

- testing of more than 70,000 specimens, totaling approximately 1,000,000 board feet of lumber, in bending, tension parallel to grain, and compression parallel to grain. This 10 year, $7 million dollar effort was one of the largest single research efforts ever undertaken in forest products research (Kretsmann, 2010).

The North American In-Grade Testing Program was a coordinated effort that utilized ASTM standards to test wood specimens to validate current design standards. The testing program also helped to establish new standards such as ASTM D 1990
Standard Practice for Establishing Allowable Properties for Visually Graded Dimensional Lumber from In-Grade Tests of Full Size Specimens. This standard addresses concerns associated with rapid rates of loading due to mechanical testing.

To accomplish the task of validating current design values, the North American In-Grade Testing Program incorporated many local agencies that independently evaluated lumber at a local level. The In-Grade testing program involved 33 species, or species groups, of lumber with considerations given to several different variable factors such as temperature, humidity conditions, moisture content, and differences in moisture meter reading. The goals of the testing program were not only to provide mechanical properties of various species of lumber but also to produce models that could be used to predict the strength of light-framed wood assemblies.

The culmination of this research helped to verify many historic design values of lumber that had existed for over seventy years. After the testing was completed in 1988, the results were quickly adopted by the NDS. The research also helped to adjust behavioral equations for column, beam, and beam-column design. To this day the NDS still reflects the results of the North American In-Grade Testing Program.

2.7 Chapter Summary

In summary, the mechanical properties of sawn lumber have been extensively studied and the methods of testing wood specimens have been well documented. However, questions still remain as to the exact testing standards that are used to develop building codes. This lack of clarity has caused great uncertainty in identifying factors of safety that exist within the governing codes. From the literature reviewed, it can be
concluded that a numerical factor of safety does not actually exist, but rather a range or a probability of failure would better describe how allowable values are determined. Moreover, the added weight applied to a roof system due to a PV installation is not a question of encroaching on the factors of safety but rather an issue that must be analyzed as to how it affects the probability of failure.

In order to further explore the performance of wood roof systems, full size laboratory testing was conducted as a means of observing structural behavior of roof systems. Although laboratory testing was conducted on full scale specimens the sheer number of tests that are needed to produce statistically significant results far exceeded the budgetary constraints of this study. The following chapter will describe the methods used to conduct laboratory testing.
CHAPTER 3

Testing Apparatus and Methods

3.1 Methods Overview

In order to accommodate full scale laboratory testing, a great amount of floor space was required to not only conduct testing but also to construct both test specimens and testing apparatus. All laboratory testing was conducted within University of New Mexico’s Centennial Engineering Structures Laboratory. The following sections focus on methods employed to conduct testing, construction of test specimens, procedures used to conduct tests, and data collection methods.

3.2 Laboratory Testing Methods

To apply a distributed load, an air bladder is used to pressurize the surface of the roof assembly, as shown in Figure 3. By inverting the roof assembly, the OSB surface faces the ground while the joists are on top. The air bladder is placed between the ground and the OSB surface of the assembly. As the roof assembly overlies the air bladder, a reaction header is placed on each end of the assembly to prevent the ends from displacing vertically, therefore acting in the same way as a bearing wall would support a roof. By allowing the air bladder to react against the ground, a uniformly distributed load equal to the air pressure in the bladder is applied to the inverted roof panel.
3.3 Materials

**Roof Panel Assembly.** When building a roof panel of any fixed length, the layout and cross-section geometry remains the same throughout the testing of joists or bearing members. Figure 4 provides an example of a roof panel.

The construction of each roof assembly utilized five joists spaced at 2 ft on center; the ends of each joist were then attached to like-sized end plates to complete a
rigid frame. Once a roof frame was constructed the frame was overlaid with 7/16” OSB decking as prescribed by the IRC 2009. The decking was then spaced 1/8” apart and nailed to the joist at 12” on center with 8d nails. All nails used in the assembly were collated 8d galvanized coated, clipped head nails, and were driven with a pneumatic framing nail gun. In preparation for a test the completed assembly is inverted and set on top of the deflated air bladder, then the reaction headers were placed above the assembly. All dimensional lumber and OSB decking was purchased from a local lumber retailer (Home Depot). Figure 5 provides a photograph of a roof assembly under load.

```
Figure 5. Photograph of roof assembly under load
```

**Air Bladders.** The first air bladders used were Coleman® Single High QuickBed® Twin Airbed and Coleman® Single High QuickBed® Queen Airbed purchased from a local retail store (Home Depot). Due to the fixed size of the airbeds the roof surface could not be completely covered by one airbed. Therefore multiple airbeds were used to load the majority of the roof surface, while not extending past the outer edge
of the OSB. Figure 6 shows an example of the assembly with Quickbed as the air bladders. After a few tests and analysis of the pressure results, the Quickbeds were unable to deform sufficiently to cause failure of the test assemblies. We therefore elected to manufacture custom air bladders capable of conforming to the larger deformations that the roof assemblies required at failure.

![Figure 6. Photograph of assembly with initial air bladder](image)

In designing the custom air bladders, 40 mill PVC sheet was selected due to its relative ease of acquisition, as it is commonly used as shower pan liner in residential and commercial construction. The PVC air bladders were glued together with PVC adhesive. The use of custom-made PVC air bladders started with test four and was used on all subsequent tests. Figures 7a and 7b show photographs of the PVC air bladders.
Figure 7a. Photograph of 10’x16’ PVC air bladder

Figure 7b. Photograph of 8’x10’ PVC air bladder
**Air bladder hoop stress calculations.** Using thin-walled pressure vessel theory, it was determined that the PVC bladders were adequate to exert the necessary levels of pressure to the roof assembly while maintaining acceptable levels of hoop stress. By equilibrium, the hoop stress is found to be \[ \sigma = \frac{pr}{t}. \] The hoop stress can be controlled by adjusting the radius of curvature of the sides of the bladder. By keeping the test assembly close to the ground the radius of air bladder inflation was minimized, thus allowing for pressures exerted on the assembly to be a sufficient to fail the assembly.

### 3.4 Data Acquisition

Two different data acquisition methods were employed. The first method used video recorders to visually record each testing session, allowing for data to be read from the recording and then manually entered into a spreadsheet at a later time. In addition, observers were present to help monitor air pressure and to record roof assembly cracking and failure. The majority of tests were recorded visually and relied upon analog measurement systems. Components of the analog measurement systems include two water manometers to measure pressure and five reference string lines with rulers to measure deflection at the center of each joist.

In subsequent tests, automated digital data collection was utilized to measure both pressure and displacement at panel center to help ensure uniformity in measurement and eliminate the work associated with collecting data from a video recording. Digital data acquisition was accomplished using a National Instruments eDaq in conjunction with LabView software. Pressure measurements were gathered with a Honeywell pressure transducer, while displacement was monitored through the use of a Firstmark...
Controls yo-yo potentiometer. Figure 8 provides a photograph of the digital data collection system.

![Digital data collection system](image)

Figure 8. Digital data collection system

**Pressure.** During the inflation the air bladders pressure were monitored through water column manometers as shown in Figure 9 and 10.

![Manometer diagram](image)

Figure 9. Manometer
A water column manometer measured the pressure in each individual air bladder, to ensure that the pressure was uniform across all bladders. If multiple air bladders were used, all air bladders were linked together through a neutral cross over tube. Before the implementation of digital data acquisition, manometer readings were recorded using a digital video camera. The use of multiple manometers helped to control uniformity in pressure between air bladders during testing. The air operator ensured that all manometer readings were equal across air bladders when adjusting the speed with which air was added to the system to achieve uniform loading. Once digital data acquisition was enabled, pressure data was gathered from the Honeywell pressure transducer and a National Instruments (NI) eDaq.

![Figure 10. Photograph of double manometer use in laboratory](image-url)
**Deflection.** To measure deflection, a ruler was initially attached at the mid-span in of each joist. The ruler was set perpendicular to the axis of the joist and attached to side of the joist. Against the top of each joist and consequently 3.5 inches high on the ruler (in the case of 2x4 joist) a string line was attached tautly to the reaction headers. As the roof assembly began to deflect, the rulers attached to the joist rise against the fixed string line, allowing for deflection to be measured visually with approximately 1/8” precision.

During the testing of the assemblies, deflection and pressure were monitored as a function of time. Due to the loading configuration, the center of each span experienced the largest deflection. A graphical user interface was built in LabView to monitor the data in real time allowing for both data logging and real time verification of results. A Fluke 189 data-logging multimeter was also utilized to add a layer of redundancy to the data acquisition from the pressure transducer. It is of interest to note that a multimeter can be used to monitor and subsequently record the change in voltage of a transducer; this change in voltage can then be converted to a change in pressure or position through appropriate calibration procedures. The multimeter was set to acquire a measurement every five seconds. The results were then converted to a pressure measurement and compared to the measurements made by the NI eDaq system. It was observed that measurements of the two devices were nearly identical and the error in measurement was acceptable. Figures 11 and 12 show the graphical user-interface and underlying data acquisition that were programmed in LabView. Considering the agreement of both devices, the data from the eDaq system was found to be sufficiently accurate.
Figure 11. Real time testing display data

Figure 12. LabView program used in data acquisition
3.5 Testing Procedure

For each test, the reaction headers are set at the appropriate distance to accommodate the chosen span. After the reaction headers are in place, the assembled inverted roof panel is rolled onto the bladder and below the reaction headers and subsequently clamped to the headers. Figure 13 shows a photograph of an assembly being rolled under the reaction headers.

![Figure 13. Photograph of assembly being rolled onto air bladder](image)

As the air bladders are inflated, the ends of the roof assembly are pushed against the reaction headers while the center of the roof assembly is free to deflect upwards. Once the air bladder/bladders are sufficiently inflated to support the full weight of the reaction headers, the clamps that are used to raise the assembly are released. Control of the air bladder inflation is accomplished with the use of a ball valve, directly connected to a laboratory air compressor. The air flow is adjusted throughout the test to keep the air pressure constant among all air bladders. The loading rate is approximately 10 psf per minute. As pressure increased the roof panel begins to deflect upwards with the
maximum deflection typically occurring in the center of the assembly at the center joist. As the first joist fractures, the pressure drops in response to the sudden increase in bladder volume. After the first major fracture of the assembly occurs, the air flow is increased to further pressurize the air bladder until the next major crack occurred. When the deflection becomes very large (4” or more), the test is concluded and the pressure in the airbladders is released. All data recoding then ends.

3.6 Chapter Summary

Testing of full scale roof panels presented many unique challenges. One challenge was the development of the testing apparatus. Construction of the apparatus was an iterative process that required time and knowledge of construction methods to ensure that the apparatus was able to fail roof panels of multiple sizes. A second challenge was the collection of data during the actual testing of roof panels. Again, this process was developed incrementally and evolved throughout the study. Once the testing procedures were developed it proved to be effective in producing results. Chapter four presents the results of the testing conducted with above described procedures.
CHAPTER 4

Test Results

4.1 Results Overview

Full scale laboratory testing of wood roof assemblies has been conducted to better understand how wood roof assemblies fail under load. Initial testing focused primarily on creating the testing methods sufficient to cause roof panel failure. Once the design and calibration of the equipment needed for testing was completed, the focus of testing switched to recovering accurate data from every test. Pressure and deflection were the main responses monitored. The early testing results were based on visually reading values from video recordings of manometer responses and string line deflections. These early tests were conducted with extreme caution due to unfamiliarity with the testing equipment and procedures. Consequently, the rate of loading applied to roof sections was inconsistent. The inconsistencies in rates of loading coupled with non-digital data acquisition contributed to the large error bias of some reported data. The accuracy of test results improved drastically as data acquisition became more sophisticated and researchers became more comfortable with the testing procedures. In the following sections each of the test results are graphically compared to the design values prescribed by the NDS.

4.2 Anticipated Results

There exists an analytical method for determining the bending capacity of a wood roof system in lieu of destructive test methods. The method used to calculate the capacity of a roof system is Euler-Bernoulli beam theory. This theory states that the
maximum fiber bending stress is \( F_b = \frac{Mc}{l} \), where \( M \) equals the maximum moment, \( c \) equals the distance from centroid to the extreme fibers and \( I \) equals the moment of inertia of the beam. The use of this method allows for the strengths of different sized joist to be calculated. The shear capacity did not control in any of the tested panels by design.

In order to compare NDS design values to test results the design values \( E \) (Young’s modulus) and \( F_b \) (fiber bending strength) must be converted to pressure and deflections. In order to convert \( F_b \) from a fiber bending stress to a uniform distributed pressure the following sequence was used

\[
M = F_b I \frac{c}{l}
\]

\[
\omega = \frac{M \times 8}{l^2}
\]

\[
p = \frac{\omega}{s}, \text{ where}
\]

\( \omega \) = distributed load

\( M \) = moment

\( l \) = length of joist (span length)

\( I \) = moment of inertia of joist about strong axis

\( c \) = distance from centroid of joist to extreme fiber of joist

\( s \) = joist spacing

\( p \) = failure pressure.

Thus, \( p = \left( \frac{F_b I}{c} \right) \frac{8}{l^2 s} \).
Furthermore, displacement $\Delta$ is calculated as

$$\Delta = \frac{5\omega l^4}{384EI}, \text{ where}$$

$$E = \text{Young’s modulus prescribed by the NDS, so}$$

$$\Delta_{\text{failure}} = \frac{5(p \times s)l^4}{384EI}$$

With the use of the aforementioned formulas the results of testing can be directly compared to NDS design values.

Laboratory testing was limited due to the nature of the laboratory floors anchoring system. Laboratory testing was conducted with joist spans that were in close proximity (rounded to the nearest 2 foot increment) to the prescribed spans listed by the IRC. By conducting tests of joist spans that were close to IRC specification, bending controlled the mode of failure, and consequently shear was not a contributing factor to ultimate failure. In accordance with common practice, the contribution of composite behavior to strength is ignored.

4.3 Preliminary Results

The first six tests were exploratory in nature, used to establish acceptable methods and equipment to accomplish the required testing. Tests (7-25) utilize the successful procedures that were developed. Results of these tests are presented in the following section. Within the following sections all tests that recorded pressure and deflection are compared to allowable NDS values.

2x4 at 8ft Span: Tests One through Six
The first testing process began with spans that were taken from rafter span charts listed in the 2003 IRC. The prescribed span for a 2x4 Douglas fir-larch #2 is 7’-10”. As previously mentioned, laboratory constraints of our apparatus limited our spans to 2ft intervals and as a result spans of 8ft were selected. An added 2” longer span was advantageous in reducing the pressure necessary to break an assembly and therefore reducing stress on air bladders. However, deflection became more pronounced and the air mattresses were unable to accommodate this added deflection. The need for a balance between span and deflection was evident. Deflections posed a larger problem than a slight increase in pressure. Upon several trials it was decided to round down the specified spans to the nearest even span length of 6ft.

**Test One.** Test one was conducted as a ‘proof of concept’ test with reaction headers spaced 8ft apart and attached to the floor with threaded steel rod allowing the roof panel to simulate an 8ft roof span. No data was taken with this test. However the completion of this test allowed us to better design how the reaction headers were held in place and determine how to best measure pressure with the use of manometers.

**Test Two.** Test two was conducted with an 8ft panel span with reaction headers spaced 8ft apart. Data collection was only focused on pressure that was verbally read out and recorded. Two air mattresses were used for this test to best cover the area of the roof panel. The measurement for pressure versus time from the two air bladders is presented in Figure 14. Two tests were conducted with this preliminary setup and it was determined that a 6ft roof assembly would best meet the objectives of this study for two reasons. First, the lumber used for this study was a mix of “whitewood #3” and according to the building code span tables using this material, 6ft spans were closer to the
recommendations. Secondly, deflection would be less using 6ft spans thus allowing the air mattresses to expand freely without reaching their inflation threshold.

The roof assembly from test two appears to carry a larger load as shown in Figure 14. This is perhaps due to the air mattresses inability to confirm to the deflected shape of the roof assembly. Therefore the excess loading can be attributed to the internal webs of the air mattresses.

![Pressure measured 2-19-13](image)

Figure 14. Pressure for test two, no deflection data was collected.

**Test Three.** Test three was conducted with a 6ft span roof assembly. Figure 15 illustrates the data collected during this test.
The failure of joists can be distinctly seen in the graph by sharp decreases in pressure. The failure of the center joist exhibits the most dramatic effect at the center deflection value of 1.37 inches. Additional failure occurs at the flanking joist with a deflection of 2.95 inches. It should be noted that after initial failure the system is still able to carry load in excess of the load that causes the first joist failure. After this test, it was observed that the use of pre-made air mattresses was restricting pressure being applied to the assembly due to webbing that is used within the mattress. It was decided that an air bladder with no internal webbing should be constructed and used for future tests.

**Test Four.** Test four was conducted on a 6ft span roof assembly and utilized the newly constructed 8x10 air bladder shown in Figure 7b. Figure 16 presents the data collected during this test.
The initial failure occurred with the left exterior joist at around 1.37 inches of deflection with pressure at approximately 93 psf. A second failure occurred with the adjacent joist at 1.87 inches with a pressure of 108 psf. During the test, the airbladder was bulging outside the roof assembly resulting in a line load in excess of the distributed load. In response to the airbladder bulging around the sides of the roof assembly it was determined that the roof assembly should be raised to prevent the extra line load that was caused by the bulge at the panel edges.

**Test Five.** Test five was conducted with a 6ft span roof assembly with the 8’x10’ custom air bladder. Additionally, the roof assembly was raised to 14 inches to accommodate the 8’x10’ airbladder and prevent any bulging around the side joist and therefore diminishing the effects of any line load. Figure 17 presents the data collected during this test. Figure 18 is a photograph showing the initial failure of the center joist and the joist right of the center occurring around 1.75 inches of deflection with approximately 135 psf. The testing continued until it was noted that the testing apparatus
hold-downs were beginning to rotate. Testing was suspended until a fix was made to the hold-down problem. A rocker plate was added to the bottom of the hold-downs thus reducing the effect of rotation of the hold-down headers. It is important to note that NDS values are constant for each test; however test results differ due to the variable nature of both testing and the mechanical properties of lumber.

![Deflection & Pressure 4-12-13](image)

Figure 17. Pressure and deflection for test five

![Failure of center joist in test five](image)

Figure 18. Failure of center joist in test five
Test six. The introduction of rocker strips as shown in Figure 19 decreased the contact area between the roof assembly and the reaction headers. Additionally, the rocker strips assisted in ensuring centerline loading therefore minimizing the hold-down headers rotation by eliminating any eccentricity in the reaction headers.

The first joist to break during test six was an exterior joist at a maximum pressure at 83 psf with a deflection at the center of the assembly of 1.12 inches. It was noted that during the test, that the reaction headers did not rotate, therefore validating the ability of the rocker strip to eliminate the majority of the eccentricity. Data collected during this test is presented in Figure 20.
4.4 Test Results

Although some data was reported from the previous six tests, that data was only used to verify testing methods and equipment. The following test results represent the experimental focus of this research. The results of the following test comprise all the data that was used to analyze test results.

Tests 7 through 25. The following 19 tests were conducted with the same basic testing assembly and methods previously used in test six with the exception of two minor differences. The differences include digital data acquisition and changing the distance between reaction headers to accommodate different tested span lengths. The IRC span tables were used to dictate the span length for each 2in dimensional lumber test. Additionally trusses and composite wood I-joists were also tested.

2x4 at 6ft span. Once the test assembly and data collection methods were finalized during test one through six, further testing was conducted on 2x4 assemblies to develop a consistent set of results. Figure 21 displays the data collected form 2x4 testing.
The results indicate a nonlinear relationship between pressure and deflection. This nonlinearity was mainly due to the use of non-digital data acquisition and variation in the speed at which pressure was applied. Subsequent testing results do not present this nonlinear relationship as data was collected through the use of digital devices and air supply was consistent after first member failure.

Figure 21. Deflection and pressure of 2x4 at 6-foot span (Non-digital data acquisition)

The maximum pressure associated with the failures of the 2x4 test assemblies was 152.9 psf; the minimum measured pressure causing failure was 88.4 psf. With deflections reaching 1.9 inches and 1.5 inches respectively. Table 6 presents the results associated with each test conducted on 2x4 roof assemblies with an overview of maximum pressure and associated deflection. Figure 22 shows a catastrophic failure of a 2x4 roof assembly.

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum Pressure</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>135.2</td>
<td>1.7</td>
</tr>
<tr>
<td>Test 2</td>
<td>88.4</td>
<td>1.5</td>
</tr>
<tr>
<td>Test 3</td>
<td>152.9</td>
<td>1.9</td>
</tr>
<tr>
<td>Test 4</td>
<td>104</td>
<td>1.8</td>
</tr>
</tbody>
</table>
Figure 22. Photograph of the failure of a 2x4 roof assembly.

2x6 at 14ft Span. The testing of 2x6 joists was performed on two roof assemblies. Both tests utilized digital data collection to obtain results. Results from these tests reflect the increasingly common theme of displaying ultimate values that far surpasses the allowable design values. Figure 23 shows a well-defined linear slope up to first member failure. It can also be seen that one of the tests was able to achieve higher ultimate loads than the loading that caused first member failure. The added capacity that exists after first member failure is a prime example of ASTM’s factor of residual capacity.
The maximum measured pressure associated with the failures of the 2x6 test assemblies was 77.6 psf and the minimum measured pressure causing failure was 56.2 psf with deflections reaching 2.8 inches and 2.2 inches respectively. Table VII presents the results associated with each test conducted on 2x6 roof assemblies.

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum Pressure</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>56.2</td>
<td>2.2</td>
</tr>
<tr>
<td>Test 2</td>
<td>77.6</td>
<td>2.8</td>
</tr>
</tbody>
</table>

A unique mode of failure took place during the testing of a 2x6 roof assembly. This failure occurred, not at the expected center joist, but rather on the far side of the assembly. Figure 24 provides an image of this failure.
Figure 24. Photograph of unique exterior joist failure during the testing of 2x6 roof assemblies

**2x8 at 14ft span.** A total of five tests were conducted on 2x8 roof assemblies. All five tests relied upon visually collected data through the use of video recorded manometer readings and deflection measurements. Graphical representations of pressure and deflection are presented in Figure 25. As seen in this figure vertical lines in the graph represent deflections immediately before joist failure. Deflections after joist failure exceeded the measurement capabilities. Therefore the reporting of deflection values was given at the point of greatest deflection measured prior to failure.
The maximum measured pressure associated with the failures of the 2x8 test assemblies was 111.3 psf and the minimum measured pressure causing failure was 63.4 psf with deflections reaching 3.2 inches and 2.3 inches respectively. Table VIII presents the results associated with each test conducted on 2x8 roof assemblies.

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum Pressure</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>66.5</td>
<td>2.2</td>
</tr>
<tr>
<td>Test 2</td>
<td>78.0</td>
<td>2.2</td>
</tr>
<tr>
<td>Test 3</td>
<td>83.2</td>
<td>3.0</td>
</tr>
<tr>
<td>Test 4</td>
<td>63.4</td>
<td>2.3</td>
</tr>
<tr>
<td>Test 5</td>
<td>111.3</td>
<td>3.2</td>
</tr>
</tbody>
</table>

Figure 26 provides an image of Test 4 failure. The failure of the extreme fiber on the tension side of the center joist can be seen in this figure; simultaneously this point coincides with the intersection point of a knot in the lumber. This irregularity contributed to the early failure of the assembly.
2x10 at 18ft span. A total of two tests were conducted on 2x10 roof assemblies. All tests utilized digital data collection through the use of Labview software controls and the use of a pressure transducer and yoyo potentiometer. Figure 27 provides a graphical representation of pressure and deflection. An examination of this figure indicated that both assemblies exhibit similar modulus of elasticity curves leading up to first failure. It can also be seen that the ultimate load occurs after the first member failure, illustrating residual capacity.
The maximum measured pressure associated with the failures of the 2x10 test assemblies was 66.6 psf and the minimum measured pressure causing failure was 56 psf with deflections reaching 4.1 inches and 2.1 inches respectively. Table IX presents the results associated with each test conducted on 2x10 roof assemblies.

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum Pressure</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>66.6</td>
<td>4.1</td>
</tr>
<tr>
<td>Test 2</td>
<td>56.0</td>
<td>2.1</td>
</tr>
</tbody>
</table>

Figure 28 provides an image representative of the location of failure coinciding with the most visually obvious grain incursions.
2x12 at 20ft span. A total of two tests were conducted on 2x12 roof assemblies. All tests utilized digital data collection through the use of Labview software controls and the use of a pressure transducer and yoyo potentiometer. Figure 29 provides a graphical representation of both pressure and deflection for each test conducted on 2x12 roof assemblies.
The maximum measured distributed pressure associated with the failures of the 2x12 test assemblies was 104 psf, the minimum measured distributed pressure causing failure was 75.9 psf. With deflections reaching 3.4 inches and 6.0 inches respectively.

Table X presents the results associated with each test conducted on 2x12 roof assemblies.

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum Pressure</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>104.0</td>
<td>3.4</td>
</tr>
<tr>
<td>Test 2</td>
<td>100.1</td>
<td>4.6</td>
</tr>
<tr>
<td>Test 3</td>
<td>75.9</td>
<td>6.0</td>
</tr>
</tbody>
</table>

Figure 30 shows a dramatic failure of both the joist and the nail attachments to the decking as seen in Test 2.

Figure 30. Failure of both the joist and the attachments to the decking

**TJI at 20ft span.** A total of one test was conducted on a composite I joist (TJI) roof assembly. The test utilized digital data collection through the use of Labview
software controls and the use of a pressure transducer and yoyo potentiometer. As seen in Figure 31 the slope of the elasticity curve is nearly linear up to the point of ultimate failure. The uniformity between joist members can also be seen in this figure with all members failing simultaneously and therefore lacking any residual capacity. This test proved to be the best illustration of a simultaneous failure of an entire system.

![Test Results (TJI) @ 18'](image)

Figure 31. Deflection and pressure of TJI at 18-foot span

The maximum measured pressure and associated displacement with the failure of the TJI test assembly was 87.5 psf, with a deflection of 2.1 inches. Table XI presents the results associated with the test conducted on the TJI roof assembly. Figure 32 provides an image of the testing and failure of the roof assembly.

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum Pressure</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>87.5</td>
<td>2.1</td>
</tr>
</tbody>
</table>

Table XI. TJI at 18 foot span
Trusses at 18ft span. A total of one test was conducted on a truss roof assembly. The test utilized digital data collection as was utilized in previous testing. As seen in Figure 33 the slope of the elasticity curve is nearly linear, up to the point of ultimate failure. However, unlike the TJI, after the first member failure a noticeable residual capacity was seen and the trusses continued to carry load while plastically deforming until complete system failure. Although continued loading after first member failure was achieved, the magnitude of loading was less than needed for first member failure, thus residual capacity was not completely verifiable. Figure 33 provides a graphical representation of both pressure and deflection.
Figure 33. Deflection and Pressure of truss at 18-foot span

The maximum measured distributed pressure and associated displacement with the failures of the truss test assembly was 157.8 psf, and a deflection reaching 2.6 inches. Table XII presents the results associated with the test conducted on the truss roof assembly.

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum Pressure</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>157.8</td>
<td>2.6</td>
</tr>
</tbody>
</table>

Figure 34 provides an image of the testing associated with the truss assembly. This figure provides an example of the large air bladder folded to better accommodate the area of the test specimen. The technique of folding air bladders to best fit the loading area was conducted on several test assemblies. This figure also shows that failure occurred predominately in the top and bottoms cords of the truss. Figure 35 provides a second image of the truss testing. This figure shows the localized failures of gusset plates that connected both top and bottom cords. Although the gusset plates ultimately failed, the initial failure of the truss occurred from tension failures of the top and bottom cords.
Lastly, Figure 36 provides additional information concerning the testing of the truss assembly. In this figure it is of interest to note that the truss failure initiated at locations where knots existed in close proximity to the edge of the member.

Figure 34. Photograph of testing and failure of the truss roof assembly.

Figure 35. Photograph failure of the gusset plates.
The preceding results of empirical testing have provided great insight to the structural values prescribed by the NDS. These empirical results have also raised questions to the viability of targeted factors of safety, prescribed by ASTM, and the real values that were demonstrated by destructive testing. The variance in observed factors of safety can be interpreted as both an influential justification for added capacity and a proof of the margin of error associated with the correlation between MOR and MOE. The implications of these findings are further explored in the following chapter.
CHAPTER 5
Discussion and Analysis

5.1 Discussion Overview

The results of testing revealed that within our samples there was always a greater ultimate capacity than the prescribed allowable capacity. This conclusion provides evidence that the factor of safety may be sufficiently high to offset any additional loading that occurs due to the installation of photovoltaic arrays. This chapter provides a discussion that compares the allowable bending strength from building codes with the observed bending strength found from testing. Additionally, this chapter discusses the findings in relation to the ASTM standards and provides an analysis of current practices for PV installations.

5.2 Test Data Comparison

When comparing different joist sizes and span lengths, it is helpful to calculate a fiber bending strength ($F_b$). The calculation of $F_b$ allows comparisons to be made across differing tests due to the incorporation of span length, cross-sectional area, and distributed loading. Calculation of $F_b$ is found by the use of the following equation:

$$F_b = \frac{M_{\text{max}} \times c}{I}.$$  

Although this formula represents the fiber bending stress, it is important to distinguish between the code provided allowable bending stress ($F_b$) and the ultimate experimentally observed bending strength ($F_{\text{exp}}$). A comparison between the IRC prescribed $F_b$ and the empirically found $F_{\text{exp}}$ can be made by converting IRC spans into $F_b$ values. Figure 37 provides a graphical representation of this comparison for all tested
dimensional lumber joist. Furthermore, it is noted that IRC spans translate into higher $F_b$ values than those published by the NDS.

Figure 37. NDS, IRC, and experimentally-observed bending strength values-compared for dimensional lumber.

Figure 38 compares the experimental data for all tests versus the allowable values of pressure and deflection; calculated from NDS values of $F_b$. The inconsistency between test results highlights the variability that exists among individual pieces of similarly graded lumber. This inconsistency raises concerns about the total discrepancy in strengths that exist within a joist type. As show within Figure 37 and 38, a lumber grade type could contain ultimate strengths that vary by as much as 73%. The variability of strengths that exists between tests cast further doubt upon a quantifiable factor of safety; but rather reveals further evidence that a factor of safety should not be expressed as a finite number but rather a probability.
Figure 38. Experimentally observed pressure and deflection values—compared to allowable pressure and deflection values.

These test results have shown strong evidence of greater factors of safety than prescribed by ASTM. However, according to ASTM standard 2915, a minimum of 28 tests on each joist size and type should be conducted in order to gain enough data to be representative of a sample population. Using this standard, findings would be expected to converge on the true factor of safety with these additional tests. Therefore, inferences concerning the factor of safety drawn from the current research should be used with caution.

5.3 ASTM Standard 6555

Early in this research it was observed that within the building codes the allowable strength of dimensional lumber was higher if the lumber was used in a roof assembly. The increase in allowable strength was attributed to system effects that occur when more than four joists are fastened to other elements to form a composite section and is
currently a 15% increase to the standard allowable values of dimensional lumber. During the course of this research the source of this additional capacity was found in ASTM standard 6555 Evaluating System Effects in Repetitive-Member Wood Assemblies. This standard has established guidelines for testing the effect of a system and has additional language that suggests the 15% increase in capacity is a conservative estimate.

Furthermore this guideline indicates that in order to allow any detectable increase in capacity the number of tests conducted must be large enough to be representative of the population as referenced in ASTM 2915. ASTM 6555 also attributes the 15% increase to capacity to three main effects 1) load sharing 2) composite action 3) residual capacity of the assembly. While ASTM recognizes all three of these effects, it admittedly does not fully quantify the contribution to capacity each effect has on a system. Due to this irresolute dialogue on the part of ASTM it becomes important to further explore ASTM 6555 as an avenue to justify non-engineered PV installations.

**Load Sharing.** During testing, it was observed that variations in grain pattern and knot distribution, known as coefficient of variation (COV), greatly differentiated the capacity of individual joists. Joists that had grain irregularities tended to fail before joists that were clear of incursions. When joists in the system exhibit a higher degree of COV the load sharing effect of the roof system increases. In other words, members that have more variation in grain and knot patterns, i.e. higher COV, cause load sharing to increase due to the fact that such members’ high COV will deflect more than members that are low in incursions, low COV, thus more load is shared.

This perspective of load sharing has historically contributed to allowing a 15% increase to roof capacities. However, recent studies have indicated that load sharing, due
to high COV, should not influence this increase in capacity (Verrill & Kretschmann, 2009). The results from the current study provide evidence of this relationship between COV and probability of system failure. This is seen in the tests conducted on both the TJIs and trusses, which naturally have low COVs due to the manufacturing process. Testing of TJIs exhibited a low COV as illustrated by the simultaneous failure of the majority of the members of the system. Refer to Figure 31 for a graphical representation of this failure. Additionally, it is also evident that trusses have a low COV due to the manufacturing processes ensuring low COV wood is used in essential components when creating the truss. This can be seen in Figure 33. As with the studies conducted by Verrill and Kretschmann (2009) our results have shown that variation in individual materials can contribute to the probability of system failure.

In relation to the goals of this research, our results may support the notion that if individual members of a roof system are highly homogenous (low COV) than the likelihood of roof failure due to PV installation is decreased. The opposite of this relationship is also true, in that a system comprised of heterogeneous members (high COV) may increase the likelihood of roof failure. Simple put, this relationship indicates that load sharing is a necessary but insufficient condition in calculating the strength of a system and therefore should not be the basis for increasing the capacity of system.

**Composite action.** A common practice within the commercial construction industry is to integrate structural steel joist and poured concrete floors to achieve composite action between the joist and the flooring. When a floor is loaded, the floor transfers load to individual joist, the transfer of load between floor and joist creates slip between the two elements. If the slip that occurs under design loads is completely
prevented then full composite action is said to exist. However, if the slip that occurs under design loads is only partially prevented then partial composite action is said to exist. In order to achieve composite action between steel joist and concrete floor systems the shear flow that exists between the two surfaces must be prevented by the installation of shear tabs. These shear tabs carry the shear flow between the concrete deck and the steel joist allowing for two individual members to act as a composite section. The composite sections that are created are both stronger and more cost effective, making composite action a relied upon design tool for structural engineers.

Although typical residential roof construction does not rely upon composite action to carry design loads it does share the potential for increased capacity due to composite action. Partial composite action can be shown to exist (Campos, 2013) in a typical wood roof assembly and its contribution to capacity is partial recognized, although not specifically quantified, in ASTM 6555. When full composite action exists between the joist and roof decking the structural capacity of a wood roof system can be shown to increase (150%). If such an increase in capacity could be achieved by a retrofitting technique the added capacity need due to an installation of a PV system could be more than offset by the capacity gained from such a retrofit.

*Residual capacity of the assembly.* Another interesting avenue of justifying the addition of PV arrays to a wood roof system is considering the availability of residual capacity. It was observed from laboratory testing that wood roof systems often experienced a first member failure (FMF) before ultimate capacity was reached. If a damage model is used to define roof failure the capacity of roof systems could be better accounted for and subsequently small amounts of damage could be tolerated when
considering ultimate capacities. By allowing damage to occur in significant loading
events the ultimate capacity of wood roof systems could justifiably be increased due in
part to this new accounting method. Although ASTM 6555 recognizes this capacity it has
chosen to exclude increases to system affects due to different probabilities in failure
between systems and individual members.

The committee chose to discourage the use of residual capacity in system factor
calculations based on the premise that traditional “safety factors” are calibrated to
a member-based design system. The committee believes that it is inappropriate to
extend these same safety factors to entire systems. In other words, engineers
should not design entire systems that have the same computed probability of
failure as individual members in today’s designs (ASTM D6555).

With the use of the MOR and MOE correlation the probability of failure has been
established for individual members, however if residual capacity is considered the
probability of failure within a system becomes further dependent on the total number of
joist in the system.

Conventional engineering design criteria do not include factors for residual
capacity after FMF (first member failure) in the design of single structural
members. The increased probability of FMF with increased number of members
can be derived using probability theory and is not unique to wood. The
contribution of residual capacity should not be included in the development of
system factors unless it can be combined with load sharing beyond FMF and
assembly performance criteria which take into account general structural integrity
requirements such as avoidance of progressive collapse (that is, increased safety
factor, load factor, or reliability index). Development of acceptable assembly
criteria should consider the desired reliability of the assembly (ASTM D6555).

However complicated the rationale for exclusion, there still exist a recognized capacity
that is not accounted for in the prescription of strength.
5.4 Computer Modeling

When a PV installation is added to a typical wood roof assembly the way in which live loads are transferred to structural elements is inevitably changed. This change in load path is most exemplified by the way in which common PV racking systems convert distributed live loads, to point loads. A typical roof top PV installation collects live load on top of PV panels and transfers the weight of the loaded panels to aluminum rails, those rails then transfer load to the roof deck by means of support feet. These support feet are the mechanism by which a distributed load is changed into a point load and therefore causing a disruption to the load path that the roof system was originally designed to accommodate. An example of such a load path disruption can be seen in Figure 39. A computer model was built to discover any effects this load path disturbance has on a typical wood roof system.

Figure 39. Photograph of L-feet creating point loads on a roof.
The Finite Element Method (FEM) of modeling was conducted using SAP2000. All models were designed as conventional wood roof assemblies that closely resembled the roof assemblies used in full scale laboratory testing. Only one roof assembly size was modeled. The roof assembly consisted of 2x8 rafters spaced 24” on centers with a span of 14ft. All roof assemblies shared the same end conditions of a roller and pin configuration. The only variation between models occurred in the way load was applied. This variation allowed for the structural comparison of point loads verses distributed loads.

Three models were constructed to model three different loading conditions (Figure 40).

1) Distributed loading as would be observed in any standard residential roof system subjected to a 30 PSF snow event. In this model the total load is equal to the size of the roof deck (14ftx18ft) multiplied by 30 lbs per square foot, with a total load of 7560 lbs.

Figure 40. Three different models examine the difference between a typical distributed load and the effects of two styles of PV installations.
2) Non-staggered point loads, that simulate the change in load path associated with the installation of a typical PV system. This loading scenario sums up all loading that was implemented in the distributed loaded model and divides the total load into 25 point loads. Those subsequent point loads are then applied in five rows (rows are transverse to joist) that are spaced three feet apart, simulating a typical PV racking system.

3) Staggered point loads (Figure 43) which also simulates the change in load path due to the installation of a PV system; however this model looks at the effect caused by staggering the support feet. The same amount of total load is applied to the roof system as both the distributed load and the in line point loads.

Figure 41. Non-staggered point loads.
During the design of the models it became necessary to adjust the attachments between plywood shell elements and sawn lumber joist elements. The standard attachment between plywood and joist is a typical 8d nailed connection. Previous research has shown that nailed connections are only capable of carrying approximately 200lbs of shear force therefore only allowing partial composite action between plywood sheeting and underlain joist (Campos, 2013). Due to this partial composite action existing in situ the computer models were designed with nail elements that are capable of carrying no more than 200lbs of shear force. This limited shear force capacity better reflects the partial composite action that exists within a standard wood roof assembly.

Another consideration taken into account when constructing the computer model is the center line geometry that SAP200 uses to define the location of elements. Due to this geometry a composite section, such as that created by the joist and the plywood, cannot be fully accounted for in calculating cross sectional area and hence a wrong...
interpretation of the moment of inertia. In response to this problem elements were created that could space members at the correct distances in order to achieve cross-sectional areas that are characteristic of real world roof assemblies. These compensating blocks were modeled as very stiff non-flexural elements that ultimately ensured that plane sections remain plane.

Figure 43. Computer modeling a composite section using compensating blocks spaced 2’ on center along the length of the joist.

Further considerations include membrane effects that occur in the deformation of the plywood elements. This membrane tension force is not accounted for in the computer model, however it is considered to be a conservative omission.

*Computer modeling results.* In order to validate the computer model a static analysis of the three models was conducted in order to identify the reactions at the supports. The support reactions that were found by hand calculations were then compared to the reactions that were produced by the finite element code. It was found that the support reactions that were calculated by the FEM analysis closely matched the hand
calculations. Due to this verification the results that emanated from models with different loading conditions were assumed to be accurate.

When discussing the results of FEM modeling it become necessary to focus the findings on the most significant results as they apply to the underline research, therefore it is assumed that the moment experienced within the joist is the most significant contribution to system failure. Therefore comparison between models will focus on the differences in maximum moments experienced by the joists.

The distributed loaded model can be considered the baseline case, with all subsequent models being compared to this baseline. By making comparison to the baseline, any moment experienced above baseline can be considered a detriment to the strength of the roof system. Furthermore, any excess moment can also be attributed as a direct consequence of the loading condition. The distributed loading model was found to have a maximum moment of 16.5 kips occurring at the mid-span of the utmost center joist. The maximum moment occurring from the point loaded model was 17.5 kips occurring at a point loaded joist near the mid-span. The third model with staggered point loads had a maximum moment of 15.8 kips occurring at the mid-span of a near center interior joist.

When comparing the baseline case (distributed loading model), it can be observed that a PV installation can both negatively and positively affect the structural capacity of a wood roof system based solely on the configuration of the rail supports. This result while not intuitive can be explained by a static analysis of a simply supported beam.
Figure 44. Shear and moment diagram of a simply supported beam subjected to distributed loading.

It can be observed from the shear and moment diagram of the simply supported beam; a maximum moment ratio of 1/8 exists at mid-span. This ratio represents the baseline moment of a joist and can be treated as the maximum moment that a joist can experience in order to stay within design specifications. Figure 45 shows that when a distributed load is broken into point loads it can be observed that the moment at mid-span can increase beyond baseline.
Although Figure 45 demonstrates that point loads can cause increases in moment beyond baseline it can also be seen in Figure 46 that as the point loads are moved further away from the mid-span, the maximum moment can equal the baseline case. It can be further deduced that as the point loads are spaced closer to the supports that the moment at mid-span can be reduced below the baseline case. As it pertains to the installation of PV, this reduction in moment can help to negate the need of further structural support for an existing roof system.
As demonstrated in the preceding examples, the maximum moment experienced by the joists can be significantly reduced by converting distributed loading into point loads. Within the context of these three examples the reduction in moment is accomplished by moving load away from the center of the joist. However it can also be seen from the FEM analyses that a staggered point load configuration can reduce the moment below baseline, therefore strengthening the overall roof system. Within the staggered point load model the spacing of point loads closer to end supports did not occur but rather the activation of previously under loaded joists was accomplished. This model exhibited a serendipitous balance between load transfer and load spacing that ultimately lead to a reduction in moments beyond baseline. When comparing the two point load models it can be deduced...
that the roof decking does an insufficient job of equally transferring load across all joists. Furthermore, it can be shown that the installation of PV can help to strengthen a roof if the installation can better activate all joist and shift load away from mid-spans.

Although these examples represent a generic method of PV installation much can be learned from the results that these models produce. It can be imagined that guidelines may be formulated for PV installations that can easily be followed by contractors and installers of these systems. Such guidelines could include specifications limiting the amount and location of support feet. Guidelines could also be made to optimize PV layouts for standard size roof configurations. In addition, spacing tables could be created to facilitate support feet locations that ensure moment equilibration compared to that of a distributed loading condition.

5.5 Chapter Summary

As stated, the results of testing revealed that within our samples, the roof panels were able to carry a higher load than the code prescribed allowable capacity. These results provide evidence that the current factor of safety may be large enough to allow for the additional load of PV arrays. However, these results should be cautioned due to the lack of a published factor of safety within the building codes and the small sample size of this study. In light of the probabilistic relationship between MOE and MOR; additional testing to achieve statistical significance should be conducted. In relation to the current building codes, neither the NDS nor IRC clearly lists a factor of safety, rather these codes focus only on design values. The inability of governing codes to list a factor of safety can
most certainly be attributed to the probabilistic nature of assigning mechanical properties to lumber.

As shown in the computer modeling the ability of a PV array to convert distributed loads into point loads can be a structural exploit. Due to this exploit a promising avenue exists in the development of structural guidelines. Such guidelines could assist installers in providing PV installation layouts that can structurally equal or outperform existing roof assemblies. The implications of the current research spawn directions for future research; such future directions are discussed in the subsequent chapter.
Chapter 6

Conclusions and Future Directions

To understand the results from this study it is important to first understand the premise of why this research was conducted. The current research stems from a contract between the U.S. Department of Energy, Sandia National Laboratories and the University of New Mexico. The contract requirements state that the University New Mexico will: 1) test roof structures to determine actual load bearing capacity; 2) calculate load bearing capacity based on applicable national structural regulations and lastly 3) compare the actual capacity to the calculated capacity. These three research requirements are ultimately aimed at addressing the issues associated with PV installations and at providing an avenue to increase the number of residential rooftop PV installation by: 1) addressing current market barriers due to structural issues; 2) developing standardized design and roof structure reinforcement techniques; 3) reducing the permit application time and associated soft costs and 4) eliminating the need for a structural engineer’s approval on a case-by-case basis for the vast majority of new residential rooftop installations. The contract requirements and current issues associated with PV installations set forth by Sandia National Laboratories fix the framework for both the testing requirements and the conclusions of the current study.

Ultimately this research has helped to reveal several arguments that can help advance the previously described issues associated with residential PV installations. One argument that can be made from the research is based upon the results of testing full-scale roof panel assemblies. The testing conducted within this study has shown the possibility of added capacity beyond that allowed by the current national standards. As
can be seen in Figure 47, the average factor of safety found during testing exceeded the current national standards for each joist type.

![Factor of Safety Comparison](image)

**Figure 47. Factor of Safety Comparison**

Although full-scale testing helped to reveal the potential for added capacity, the process of proving actual added capacity requires strict adherence to ASTM testing standards. ASTM requires that enough samples are tested to be representative of a population, with the number of samples tested being dependent on the population size. The required full-scale testing for each joist size is a large-scale research objective requiring costly high volume testing, and is outside the scope of the current research. Additionally, the end result of such a pursuit does not guaranteed a systemic change to allowable standards. The current results cannot be described as definitive; they do however, provide a path for DOE and Sandia National Labs to consider while pursuing their ultimate objectives.
With further regard to the framework set by Sandia National Labs, the literature review highlighted additional avenues to consider when discussing the issues associated with PV installation. Firstly, review of the literature pertaining to the structural capacity of room systems identified the possibility of reducing allowable design values due to the widespread misunderstanding of coefficient of variation (COV). Secondly, the literature provided insight into the development of allowable standards and the actual numerical value for the prescribed factor of safety of wood joists. Currently, a numerical value for the factor of safety is defined; however, the literature associated with the prescribed factor of safety also describes the highly variable process of developing such numbers. The development of a factor of safety is based upon mechanical grading of lumber, which is done in a probabilistic manner. In other words, the assignment of grades to lumber is based on a multitude of variables and the assignment of grade has unknown error associated with the process. Due to the probabilistic nature of grading lumber based upon the correlation between modulus of elasticity and modulus of rupture, we suggest that the actual factor of safety is, probabilistically, much greater than the prescribed factor of safety. This suggestion, coupled with other aspects described in the current study (i.e. computer modeling, drying effects, and system effects), suggests that roof strengths are probabilistically higher than current codes assume.

The current research points out several opportunities for the justification of allowing the majority of installations of rooftop PV panels to be accomplished without the use of a structural engineer. However, such blanket acceptance of non-engineered installations requires approval of a blue-ribbon ASCE or similar committee. Possible future follow-up research ideas include examination of the variability of structural
capacity due to: varying point loading configurations, different PV array configurations, the structural utilization of PV racking systems, and further understanding of wood properties.
References


