



# Design of Residential Structures Against Strong Wind Forces

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A Major Qualifying Project

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4/19/2012

## **Abstract**

Through the completion of this Major Qualifying Project, we sought to design a two story, single family home capable of withstanding extreme wind loads. Over the last several decades the intensity and number of extreme wind storms, like tornadoes and hurricanes, has escalated at an alarming rate. Consequently, there has been an increase in damage to homes exposed to these storms. Therefore, the primary intent of this MQP was to identify methods and approaches to affordably and effectively improve residential home design and construction. From the information our team gathered through research, interviews, and experimentation, we designed an affordable structure capable of withstanding 110 mph 3 second wind gusts.

## **Executive Summary**

Over the last couple decades, the global community has seen a surge in high speed wind storms, which include, but are not limited to tornados and hurricanes. The United States is no exception to this alarming trend. The Midwest, South East, and East Coast are struck by these storms during the spring and fall months each year. Subsequently, as the number and intensity of these storms swells, the amount of resulting damage to our country's infrastructure and number of human casualties have also been mounting. Perhaps the sector of infrastructure that is affected most by these destructive storms is residential infrastructure. It is not uncommon to witness one of these extreme storms to decimate entire towns and cities. This is because the IBC only specifies the minimum regulations to withstand 85 to 90 mph wind gusts. However, investigations have shown poor and/or careless craftsmanship during construction by contractors decreases the maximum wind load a home can withstand. Therefore, homes being built to minimum code may, in fact, not be able to tolerate minimum designed wind load, and will certainly not be able to endure anything more forceful.

The United States was subjected to hundreds of devastating wind storms during the spring 2011. Between April 25 and April 28, a record 358 confirmed tornadoes touched down in an area that spanned from the Midwest, to the South, all the way up to the New England. Less than a month later, another tornado outbreak occurred in which another 242 confirmed twisters touched down. The most notable of these lethal tempests was in Joplin, Missouri on May 22. The Joplin tornado was a rated a catastrophic F5 and reached a maximum width that exceeded just over a mile. When the powerful wind storm had finally concluded, after reigning devastation to the area for approximately 45 minutes, it had claimed 160 lives and caused approximately \$2.8 billion in damage. Ultimately, this tornado, the aforementioned tornado outbreaks, and the tornadoes that touched down in Springfield and Worcester County, Massachusetts on June 1, 2011 inspired the Wind Effects on Structures Major Qualifying Project. The main focus of this MQP was to determine the most effective and affordable practices to ensure a residential structure can weather winds greater than the minimum code. As a result, over the course of the academic year, we designed a two story, single family home capable of withstanding 110 mph wind gusts.

We conducted extensive research that supplied us with a strong background regarding the necessity to transform residential home design and construction. We focused our initial investigations on climate change, specifically its effect on hurricanes and tornadoes in the United States, and how the increase in quantity and intensity of these storms is ravaging the Midwest and Eastern seaboard's infrastructure. From this research we became familiar with the International Residential Code (IRC). The IRC is the standardized building code for residential structures currently being used in the United States. Prior to its installation, there was not a standardized code and different codes were used throughout various regions of the country. From our analysis of the IRC we became familiar with typical wind deficiencies most homes experience. Some of these deficiencies include anchorage issues and wall failures. However, our research indicated that the majority of major structural damage could be traced back to an initial failure in the roof. Findings from this research then enabled us to identify current solutions available to combat these failures. A few of these solutions are installing hurricane straps or using fiber reinforced polymer between roof rafters and the top plate. Another solution includes ensuring there is a continuous load path from the roof all the way down to the foundation. Finally, after developing a strong understanding of the destructive nature of these storms and common caused wind failures, we analyzed the effect high speed wind storms have on the economy by examining insurance policies, insurance losses, and possible incentives insurance agencies can offer to their customers, should they make their homes less susceptible to wind caused failures.

We identified qualitative and quantitative tools, including further research, interviews, field testing and calculations to gather the necessary information that we used to fulfill our objectives and complete our project. Our project objectives were:

- To identify areas vulnerable to wind caused failure
- To determine and test best available features to prevent wind damage
- To design a two story, family home capable of withstanding 110 mph wind gusts
- To suggest simple, affordable changes to the IRC



In order to identify areas vulnerable to wind caused failure, we conducted extensive research on this topic. As mentioned earlier, all our research indicated that significant structural damage usually begins with an initial failure in the roof. To confirm our research, we contacted Hanover Insurance Group located in Worcester, Massachusetts and spoke with Jack Burlas, a Property Adjuster. Burlas verified our research by presenting us with data that revealed the majority of damage claims Hanover Insurance receives from wind caused failures are failures in the roof. Upon attaining this information and confirming our initial research, we were determined to focus our efforts on designing a roof that is less vulnerable to fail when subject to high speed winds.

We then began conducting more research to determine the best available design and physical features to implement in our structure to prevent wind damage. Through our research we were able to identify several features that are presently being employed in some newly constructed and renovated residential structures. The main design feature that we identified was the hip roof design which, unlike a gable roof, has all sides slope downwards to the wall at a fairly gentle slope. A hip roof performs better than any other roof structure when faced with enduring high winds. To determine the most effective roof connection at combating vertical uplift, we developed five tests, which included a combination of four different physical features. The five connections tested included a toenail, hurricane strap and toenail, liquid nail (epoxy), liquid nail and hurricane strap, and liquid nail and Kevlar strap. Each test was conducted five times to ensure accuracy and precision. From our tests we were determined the toenail and hurricane strap combination was the strongest connection. Additionally, it was also the second most affordable; making it the most viable option to employ in newly constructed and renovated homes.

Amidst our testing we also designed our two story, single family residential structure. Before we could begin calculations, we had to first determine where our structure would be located. This was necessary because it enabled us to obtain loading and other important factors, such as basic wind speed, ground snow load, and soil conditions. Therefore, we decided to presume we were designing a house to be constructed in Dennis, Massachusetts. Dennis is a town located in Cape Cod and was an ideal location to choose for this project because it is exposed to a variety of extreme weather conditions. During the fall

months, the Cape is often pounded by hurricanes and tropical storms that bring torrential rain and high speed winds. The next several months subject the area to nor'easters and heavy snow falls that do not usually relent until March. Additionally, choosing a specific location also allowed us to determine the species of wood that is typically used in the area for residential construction. After choosing Dennis as our location we had to determine the information above to calculate accurate loads. Therefore, we examined tables and figures from the IBC, and discovered the basic wind speed to be 110 mph and the ground snow load to be 30 psf. We then called the local Lowes Home Improvement to determine which species of wood is sold in the area and were informed SPF (Spruce-Pine-Fir) #2 is typical. Finally, we acquired a boring log from the area to analyze the soil conditions for our foundation design. After gathering all the required information we utilized the information in *Design of Wood Structures* and the *2012 International Residential Code for One- and Two- Family Dwellings* to assist us in the complete design of our two story structure. Each member and connection was designed to ensure both Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) specifications were met.

Since we were able to make the necessary calculations and design a structure that is theoretically capable of enduring 110 mph wind gusts, we believe our home could be successfully constructed and tolerate the loads it was designed to handle. We would also propose that some of the features we incorporated in our dwelling eventually become minimum code. Our primary suggestion is the connection detail that calls for the installation of hurricane straps in conjunction with toenails for all roof rafter to top plate connections. The cost of hurricane straps is inexpensive and the time required for installation is minimal, as well. Furthermore, our testing has shown this connection detail tremendously enhances a roof's ability to handle greater uplift forces caused by high speed winds. Ultimately, we believe our suggestions and design can be utilized in the future as a template for designing any home faced with the task of withstanding high speed winds to save countless residential wood structures, lives, and money.

## Capstone Design Experience

The completion of a Major Qualifying Project must address the Capstone Design Experience.

This experience states,

*“Students must be prepared for engineering practice through the curriculum culminating in a major design experience based on the knowledge and skills acquired in earlier course work and incorporating engineering standards and realistic constraints that include most of the following considerations: economic; environmental; sustainability; manufacturability; ethical; health and safety; social; and political.”*

During the completion of our project, Wind’s Effect on Structures, we utilized the knowledge we obtained from courses over the previous four years, including Structural Engineering, Materials of Construction, Foundation Engineering, Fundamental of Civil Engineering AutoCad, and others to address several of the aforementioned considerations. The considerations our project specifically addressed were economic, environmental, manufacturability, health and safety, and social.

The first and most important consideration our project addressed was health and safety. Our project’s main focus was designing a residential structure capable of withstanding high speed winds. The intensity and number of high speed wind storms have increased over the last couple decades. Consequently, because many homes are not designed to support these types of extreme wind loads, homeowners are at tremendous risk of losing their homes and possibly their lives. Therefore, by completing our MQP, and developing a design that is capable of withstanding such extreme wind speeds, we are partaking in an effort to ensure the health and safety of homes and homeowners.

The next consideration our project includes is economics. Our project covered this consideration two fold. Because our design is capable of supporting these loads, if used for construction, the relative damage should theoretically be limited. This will save homeowners, insurance companies, and the government money. In addition, to address the areas that experience the most damage during high speed wind storms, we contacted a local insurance agency to focus our design on strengthening that particular

area. We also designed our house to be affordable. While completing the design, we specified the minimum required member that was capable of handling the design loads. This would limit costs by preventing the use of unnecessarily strong and expensive members. Furthermore, we also completed our entire design without needing to specify blocking. Blocking is time consuming to install and therefore, would increase the labor costs. By eliminating blocking from our design, but still having enough strength in our structure, we were able to design a more economical structure.

Some of the lesser topics addressed in our project listed in the design experience include, environmental, manufacturability, and social considerations. The environmental considerations were the primary focus behind our project. As stated previously, the number and intensity of high speed wind storms is increasing in the United States. We analyzed current data regarding climate change, tornadoes, and hurricanes to garner a firm grasp on the environmental aspects of our project. The next topic our project addressed was manufacturability. Through our design, we specify how to construct or manufacture our home. To accomplish this we had to complete calculations to determine the number or size of every member. These calculations and specifications would be absolutely essential to the manufacturing of the house itself. Finally, our project addresses current social considerations. The fundamental inspiration behind our project were the tornado outbreaks in 2010 and 2011. These outbreaks even affected parts of Massachusetts in close proximity to WPI. They were also covered heavily in the news, with the story focusing on what can be done in the future. Our project has helped provide answers to some of those questions.

Ultimately, by completing this Major Qualifying Project, we have completed our Capstone Design requirement. We have used the knowledge obtained from the classes we have taken over the last four years and applied it to successfully complete our project and address the above considerations.

## **Acknowledgements**

The authors wish to thank several people. We would like to thank Jack Burlas, Francis Harvey & Sons, Inc., and F.L. Caulfield & Sons, Inc. for providing us with essential information to complete our analysis. We would also like to thank Russ Lang and Don Pellegrino for their assistance in the lab. Finally, we would like to thank our project advisors Professor Tahar El-Korchi and Professor Edward Swierz for their help and direction with this project.

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## 1.0 Introduction

The last two decades have seen a surge in the intensity and number of high speed wind storms like hurricanes and tornados in the United States. Some experts believe the causes of these destructive storms are the result of global warming and climate change. Many also argue climate change is directly related to man's interaction and/or interference with nature. The intense debate regarding the origins of climate change has been affecting the everyday life of Americans for almost a decade. From the light bulbs being used to the cars being driven, Americans use or are exposed to products every day designed to produce less emissions than their predecessors. Nevertheless, determining whether or not carbon emissions are the true cause of the amplified strength and quantity of these storms is not as important as mitigating the devastation these storms are reaping on the United States' residential infrastructure and subsequently the economy.

High speed wind storms are common occurrences for several regions of the United States. The Midwest, Southeast, and Atlantic coast are generally exposed to the strongest winds of any other region. Tornadoes terrorize the Midwest, while hurricanes frequently assault the south east and Atlantic coast. Tornadoes are perhaps nature's most violent storm; twisting at an average speed of 112 mph with the possibility of exceeding wind gusts of 300 mph. Hurricanes are powerful, swirling storms that form and strengthen over the warm southern Atlantic seas to generating wind gusts between 75-200 mph. Residential structures exposed to wind loads in excess of 140-150 mph are undoubtedly going to incur excessive damage. However, while some structures may be destined to be destroyed, many structural failures occur as a result of 80-90 wind loads and can be easily avoided.

Common structural failures associated with wind loading on residential structures include joint connections, roof, wall, and foundation failures. The International Residential Code (IRC) should be followed to diminish these failures triggered by wind loading. The IRC is a comprehensive residential code established in 2000 that standardized the minimum regulations for one and two family homes nationwide. Prior to the IRC, building regulations were developed regionally. However, despite the IRC being the stand alone residential code in the United States, it is not always followed during the

construction phase. When the conditions specified by the designer, which were obtained by using the IRC, are not followed structural failures prompted by wind loading are more common. Additionally, because the code only sets the *minimum* regulations, many residential structures will not contain simple, affordable solutions which would permit the structure to support superior wind loads, such as hurricane straps. Some solutions are a small expense in relation to the overall construction of the structure and can save home owners, insurance agencies, and the United States a tremendous amount of money after tornadoes or hurricanes.

The United States has developed the International Residential Code to standardize residential building regulations nationwide. This code was developed to provide a single coordinated set of national model building codes. To further advance these codes we conducted an extensive study to examine the building code, focusing specifically on building design under extreme wind loading conditions. We worked in collaboration with a local insurance agency to identify the most vulnerable structure of a house. This information obtained from our collaboration was utilized to determine solutions, which were then load tested, in an effort to strengthen this particular area. Finally, all the information was combined to design a two story single family home capable of withstand extreme wind loads. Ultimately, this study produced this detailed report that we hope will emphasize practical and reasonable changes to the IRC to allow residential structures to sustain greater wind loads.

## **2.0 Literature Review**

Our project goal was to design a single family home that was capable of withstanding extreme wind loads that homes typically experience during hurricanes and tornadoes. While some homes do experience devastating wind loads, in excess of 150 mph, and are doomed for destruction, many homes can be saved. Minimum protection mandated by the current residential code and poor workmanship during construction are common causes contributing to the unnecessary destruction of homes. We designed our structure to incorporate inexpensive but effective measures that can mitigate damage caused by wind loading. To complete this project successfully, we conducted research on climate change and its role in strengthening storms, the International Building Code, common failures in residential homes, and current solutions to combat these failures. In addition, we also investigated the financial losses and the role insurance plays following a disastrous wind storm. Finally we researched further ways insurance companies can help mitigate damage and losses to protect themselves and homeowners from substantial financial losses. We utilized these findings to develop an efficient plan of action to accomplish our goals.

### **2.1 The Necessity for Design Change**

In recent decades the topic of climate change has become a serious and urgent issue that has been the center of an enormous debate. The object of this debate is not centered on whether or not the earth's climate is changing, because almost all experts studying the change in Earth's temperature are in consensus that it is. The focus of the debate is whether the slight rise in temperature is the direct result of human interaction or one of the natural warming periods in the Earth's cyclical lifecycle. Although this intense debate may never end, there is now an overwhelming body of scientific evidence that human activity is contributing to global warming, with the main source of greenhouse gasses being emitted by, in order of global importance, electricity generation, land use changes (particularly deforestation), agriculture, and transport (Stern,2006). The pie graph below in Figure 1 portrays the percentage each source is contributing to the total amount of greenhouse gas emissions. This change in climate and rise in temperature is producing stronger and more violent storms that are reaping havoc on our country's



residential infrastructure and economy. Recent natural disasters such as Hurricane Katrina in August 2005 and the tornado that annihilated Joplin, Missouri in May 2011 were two of the strongest storms the United States has ever seen. The damage caused by these storms was astronomical both physically and financially. The economy, already suffering from a mounting debt, is also being hit harder by these types of storms that are causing Insurance companies and the federal government to lose millions of dollars, while thousands of citizens lost more than just money. Residents of the areas lost just about everything they had, including and most importantly, their homes. Consequently, natural disasters are threatening the American dream two fold, by potentially leveling homes and weakening the economy further and impeding a financial recovery. With the adverse effects of global warming expected to worsen and the economy struggling to stay afloat, changes must be made to not only to the way residential structures are built, but also to insurance policies to better protect the American citizens.

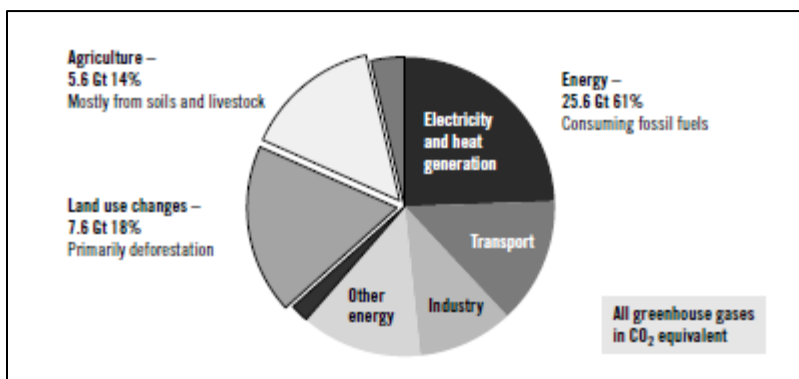


Figure 1: Global Emissions of Greenhouse Gases by Source (ODP. Web. Nov.-Dec. 2011).

### 2.1.1 Climate Change: Causes and Effects

The Earth's atmosphere functions like a blanket that surrounds the earth to keep it at temperature that is able to sustain life (Cleaner Climate, 2011). There are many gases that compose this blanket and some of these gases, carbon dioxide, methane, and nitrous oxide, are known as greenhouse gases. Currently, unhealthy levels of these greenhouse gases are present in the atmosphere and are threatening the stability of the earth's livable temperature. Scientists believe that greenhouses gases are altering the Earth's climate through a process known as the enhanced greenhouse effect.

The sun emits visible light and ultraviolet radiation toward the Earth. The Earth's atmosphere converts some of the visible light and ultraviolet radiation into heat energy, known as infrared radiation. The Earth then absorbs some of that heat energy and radiates the rest back toward the atmosphere. Some of the radiation escapes through the atmosphere but the rest is captured by the greenhouse gases and used to heat the earth at a livable temperature. This process is known as the greenhouse effect and is essential to maintain life on earth. However, recently there has been an increase in the concentration of greenhouse gases within the atmosphere due to human interaction with the environment (Jonathan, 2010). As a result, this increased concentration is also increasing the amount of heat energy that is being captured by the atmosphere. Therefore, temperatures around the world are slightly rising. At first glance, this marginal rise in temperature may appear negligible, but it is affecting the global climate (Jonathan, 2010).

The enhanced greenhouse effect is adversely affecting the global climate. The Earth is experiencing higher average sea and air temperatures. In the continental United States the annual average temperature has increased by 2°F since 1970, with winter temperatures rising twice as much as that. Warming has resulted in many other climate related changes including more frequent extremely hot days, a longer growing season, an increase in heavy downpours, less winter precipitation as snow, earlier breakup of winter ice resulting in earlier peak river flow, rising sea levels, rising sea surface temperatures, and finally more powerful storms, like hurricanes (U.S. Global Change Research Program, 2011).

A correlation between rising sea surface temperatures and increased hurricane intensity makes intuitive sense. The power of hurricanes comes from the energy held in water. Warm water is the "food" that feeds hurricanes. This also helps explain why hurricane season occurs at the end of the summer. As clouds move over warm water, they are energized by the addition of water droplets that evaporate from the ocean's surface. As clouds increase, so do the height and strength of the storm. Furthermore, the warmer temperatures present in the area increase the humidity levels just above the surface. This facilitates the evaporation of warm water droplets into the atmosphere. Finally, the warmer surface ocean temperatures decrease atmospheric stability, increasing the penetration depth of a vortex, which makes

developing tropical cyclones more resistant to vertical wind shear that inhibits the formation and intensification of tropical cyclones. Consequently, proponents of the climate change hypothesis assert there is a correlation between increasing air and sea temperatures and increased hurricane intensity. And both future simulation models and actual historical data on past storms support the conclusion that hurricanes may become more destructive as tropical sea surface temperatures increase (Glicksman, 2006). Since 1970s, the average global surface temperature increased between 0.5°C and 0.9°C per decade. An examination of hurricane intensity since 1970 shows a substantial change in the intensity distribution of hurricanes globally. According to the study, the number of category 1, 2, and 3 hurricanes remained approximately constant. In stark contrast however, the number of hurricanes reaching the level of a category 4 or 5 almost doubled and occurred in all ocean basins (Webster et al, 2005). Additionally, future models expect this trend to continue. A new model developed by the National Oceanic and Atmospheric Administration's Geophysical Fluid Dynamics Laboratory in Princeton, New Jersey is believed to be the most accurate model created, predicts the number of category 4 and 5 storms, with winds reaching 135 mph, to double by the end of the century, and the strongest storms, wind speeds exceeding 145 mph, to triple (Kerr, 2010). The typical hurricane activity in the current climate and the projected activity for a warmer climate developed by the Princeton lab are available in Figure 2. This increase in hurricane intensity has the potential to wipe out entire coast lines and cripple the economy. Therefore, it is an absolute necessity for homeowners to protect themselves from the threat of destruction.

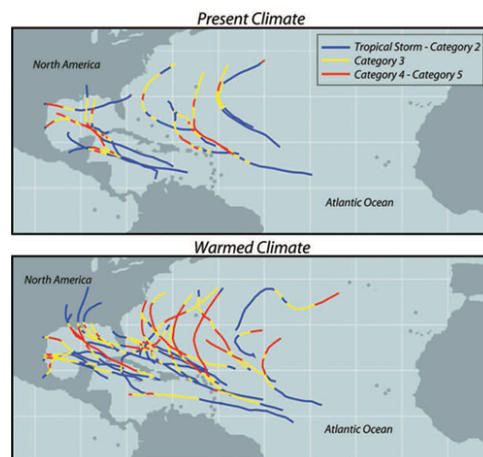


Figure 2: Hurricanes Comparisons for Present and Predicted Future Warmed Climate Geophysical Fluid Dynamics Laboratory. Web.

Nov. 2011.

## 2.1.2 Tornadoes and Hurricanes

Tornadoes and hurricanes are known for the chaos and millions of dollars of damage that is inflicted after one rips through a town or city. Although hurricanes typically form over water and tornadoes over land, both develop under similar conditions where warm, moist air winds blow into each other from opposing directions. Although both are extreme forces of nature, there is a big difference between the two, not only in how they act but how they are measured as well. Tornado winds reach much higher speeds than that of a hurricane but hurricanes generally cause more damage individually and over an entire season. Hurricanes are classified as tropical storms that reach wind speeds of 74 miles per hour or more and move in a large spiral around “the eye” which is a more calm center that can extend anywhere from 20 to 400 miles wide (Stathopoulos & Baniotopoulos, 2004). The duration of an individual hurricane is generally several hours and hurricane season itself lasts from the beginning of June to the end of November (<http://www.whathappensnow.com>, 2009). Hurricanes are measured on the Saffir Simpson scale from 1 to 5, ranging from very dangerous to catastrophic, depending on the intensity of the storm. Each increase in the scale means a potential increase of damage by a factor four (NHC-NOAA, 2009)

Tornadoes on the other hand are classified as windstorms and form from funnel clouds, thunderstorms, and even from hurricanes. The diameter of a tornado is much smaller than a hurricane averaging approximately a mile wide or smaller. A tornado can last from 10 minutes to as long as an hour and about 80% occur between noon and midnight. Tornado season is generally from March until the end of August (<http://www.whathappensnow.com>, 2009). Tornadoes are measured on the Fujita scale which rates the intensity of a tornado by examining structures that have been hit by tornadoes. The Fujita scale ranges from a gale tornado to an inconceivable tornado which could reach top speeds close to 380 miles per hour (Stathopoulos & Baniotopoulos, 2004). A depiction of both the Fujita and Saffir Simpson Scales are presented in Table 1. Although Tornadoes often occur in certain areas of the United States, such as the Midwest and Hurricanes often occur in the Southeast and along the East Coast. These violent winds can

strike any part of the country at any time and if cities and communities are not prepared, then the effects of these natural disasters can be devastating

**Table 1: Hurricane and Tornado Scales "Tornadoes." *National Hurricane Center. Web. 10 Dec. 2012***

Scale	Scale Number	Wind Speed (MPH)	Damage
	1	74-95	Minimal
Saffir Simpson Hurricane	2	96-110	Moderate
	3	111-130	Extensive
	4	131-155	Extreme
	5	>155	Catastrophic
	F-0	40-72	Gale Tornado
	F-1	73-112	Moderate Tornado
Fujita Tornadoes	F-2	113-157	Significant Tornado
	F-3	158-206	Severe Tornado
	F-4	207-260	Devastating Tornado
	F-5	261-318	Incredible Tornado
	F-6	319-379	Inconceivable Tornado

## 2.2 Importance of a Model Building Code

Model building codes are codes developed by an organization autonomous to the group responsible for executing the code. The division between the group establishing the code and people operating under the regulations is imperative to ensure a reliable system where the wellbeing and safety of the building owner and occupants are first priorities. A majority of the time, contractors constructing residential structures are working off a lump sum budget. The contractor takes one hundred percent of the risk, because the lump sum bid includes all costs involved in the construction. Therefore any additions to bid price are taken out of the contractor's profit. The one hundred percent risk on the contractor results in shortcuts being taken. Simple changes in the spacing and quantity of connections can result in major damage when faced with abnormal wind temperatures.

### **2.2.1 Pre International Building Code**

Before the International Building Code was created, there were several different codes in operation specific to where in the United States the construction was occurring. In the northeast, Building Officials and Code Administration (BOCA) created National Building Code (NBC). The Midwest and western United states used the International Conference of Building Officials (ICBO) which developed the Uniform Building Code (UBC). While Southern United States used Standard Building Code (SBC) establish by the Southern Building Code Congress International (SBCCI). Although each organization's goal was to provide standards of construction insuring the safety and welfare; there was confusion regarding the code each state should be regulated under. With the codes being very analogous to one another the three organizations (BOCA/ICBO/SBCCI) formed the International Code Council (ICC), as way of unifying the separate standards in use and strengthen the amending process. With all major bodies working under one council there would be no regional code division.

The International Code Councils first creation was the International Building Code (IBC) in 1997. This first version of the IBC was imperfect and had several faults. Therefore the 1997 International Building Code was not accepted by BOCA, ICBO, or SBCCI. Three years later in 2000, the second International Building Code was established. This code was a large improvement on the previous IBC and was therefore accepted by the associations who created it. BOCA, ICBO, and SBCCI all adopted the 2000 2<sup>nd</sup> edition of the IBC and ceased development of their fore mentioned codes.

### **2.2.2 Current State of the International Residential Code**

The International Residential Code is now the main set of standards in use for the entire United States. It is a universal code which makes construction easier for contractors since they only need to know one code. But also allows for individual states to amend the IRC to adapt to safety requirements of each state. Massachusetts has the Massachusetts State Building Code which uses the International Residential Code with provisions regarding the hazard index.

Even though the International Residential Code is applied to all new construction in the United States, it is impossible to renovate all pre-existing buildings to abide by the new code regulations. Therefore, the majority of pre-existing buildings do not fully abide by the International Code Councils set of standards, but are considered to be acceptable.

### **2.2.3 Buildings Grandfathered into the International Residential Code**

Older buildings are more susceptible to wind loads than newly constructed buildings under the IRC. One problem facing older structures is nail capacity. When analyzed, many of the nails used in the assembly of older buildings are required to support 250 pounds per linear foot. This load per fastener capacity is extremely high and exceeds current code regulations and the load the nails were designed for. If these buildings were constructed using the current code, more nails would have been installed to decrease the load per fastener. Nevertheless, because they were not built under current codes, when these structures are exposed to tornado and hurricane force winds, some form of failure is inevitable.

Another common failure point of old construction is the design of the shear walls. Current IRC specifies a specific ratio, material, and anchorage of the shear walls. The International Residential Code also provides acceptable products for different forms of construction. Without these provisions established through the IRC older buildings only have a shear capacity of 4000 lbs for the entire structure. The 4000 lbs of shear capacity is only sufficient under medium to high wind loads. Once the shear walls experience wind exceeding 100 mph damage occurs.

### **2.2.4 Most Frequent Wind Deficiencies**

The International Code Council's main objective is to provide safety to the occupants and owners of the building after construction. There are still common wind deficiencies in newer construction which need to be addressed. The most prevalent deficiencies from wind damage are the foundation anchorage with hold downs, anchor bolt spacing, rafter ties and wall ties. The main issue in the most frequent wind deficiencies involves connections. The spacing, type, and material used in each connection are what allow the shear walls to stay attached to the foundation or the roof to maintain its structural integrity.

Other wind deficiencies occur due to a lack of information from the International Building Code. These inadequacies include wind borne debris protection and wind and door pressure ratings. More research needs to be done by the International Code Council to develop a set of standards that can be used as solutions to these problems. Nonetheless pressure ratings and debris protection can be the difference of thousands of dollars in damage, and can even prevent casualties from extreme weather.

### **2.3 Types of Wind Forces**

One of the main issues that make these strong winds so dangerous is the effects it has on people as individuals and buildings and homes. A majority of the damage is not from flying debris released from these winds but because of the intense wind loads that are experienced by the millions of man-made structures. One of the major forces from wind load experienced by most homes is an uplift force which is the result of wind flowing over a roof and pulling up on the roof and all its components which puts stress on the fasteners of the shingles and the connections between the roof, the rafters, and the walls. Another load, known as racking, is a horizontal force which applies pressure on the corners of a structure and forces the house to tilt. If the corners of the structure are strong and well-built, but the floor to foundation connection is weak then a structure may experience another horizontal pressure that will cause sliding which may push a house off its foundation. If a house is not able to rack or slide, the lateral forces from the wind may cause the house to flip and rotate off of its foundation, which is known as overturning (<http://www.safestronghome.com/highwind/05>, 2011). These different types of wind loads acting alone or together can have a tremendous effect on a structure. Figure 3 displays the probability of failure as a result of these different types of wind forces. As wind speeds begin to reach 100 mph, failure is much more common.



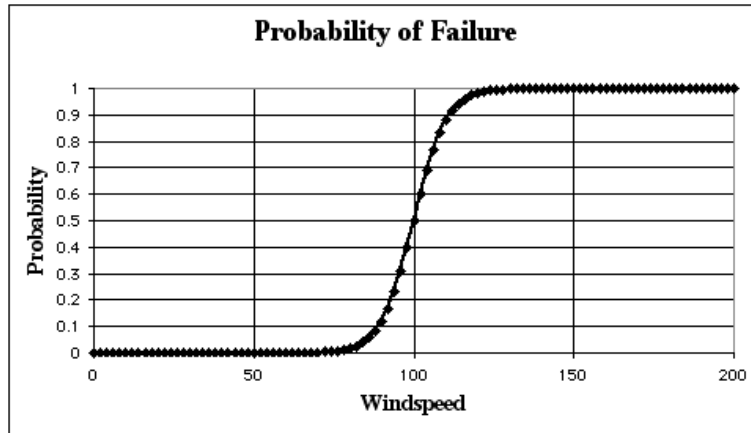


Figure 3: Probability of Complete Structural Failure vs. Wind Speed (mph) A Guide to F-Scale Damage Assessment." U.S. Dept. of Commerce 1 (1999).

## 2.4 Types of Wind Failures

A house is comprised of several structures that are constructed from smaller components. Each of these structures and smaller components can experience various types of failure. Poor construction due to materials, connections, or workmanship is one of the primary reasons wind loads will adversely effect housing structures. A building's geometry and miscalculated design wind speeds can also play a role in the destruction of a house due to high winds.

### 2.4.1 Concrete Slab Foundation Failures

There are many different potential failure points in one house that can contribute to deformation as a result of high winds. One of the main areas involves the foundation and how it's connected to the rest of the house. In the case of concrete slab foundation, there are several ways in which a foundation may experience failure during high winds. Homes using these types of foundations are often bolted or strapped to their foundations (Evans & Mandt, 2003). Bolts can be placed too deep in the concrete so that they will not be able to connect with the nuts that are supposed to be screwed to them. There have also been instances where washers were neglected to be placed on existing bolts. Bolts have also been found to be placed wrongly so that they are outside of the bottom plates, which mean they were not even anchored to the foundation. It is also more common for there to be failure where the wall stud connects to the bottom plate. The bottom plate is usually straight nailed into the wall stud or nailed in using a toenail connection.

Although this type of connection will generally meet building codes, it does not provide much resistance to uplift forces. Shot pins and cut nails used around perimeter foundations often break under wind loads or are bent and allow the foundation to move out of place. Shot pins and cut nails are generally a violation of building code around the perimeter foundation but have been found in some houses. Instead of shot pins and cut nails, the IRC specifies that anchor bolts should be used (IRC, 2012).

### **2.4.2 Poured Concrete Wall and Pier-Beam Failures**

Pier and beam foundations involve anchoring the floor beams to piers or footings, which also leave a crawl space underneath the house. The problem with this type of foundation is that the floor supported by the piers is rarely anchored to the piers. Additionally, the walls are not bolted to the foundation and make the structure more susceptible to failure. When facing the high winds that come with tornadoes and hurricanes, pier and beam foundations tend to shift off their foundations. A poured concrete wall foundation is another type of foundation that with poor workmanship becomes vulnerable to wind loads. When the bottom plates are connected to the floor, nails or bolts often miss or don't reach the connection to the floor joists which gives the connection little wind resistance. Nails between the floor and wall don't strengthen the connection. These nails will be driven into the open joint, not through to the floor connection. This virtually leaves the wall unattached.

### **2.4.3 Masonry and Pile Foundation Failures**

Masonry foundation walls are another type of construction that have weak resistance against lateral and uplifting wind forces. These type of foundations in strong winds often result in homes shifting off their foundation because the foundations are not embedded enough or do not have reinforcements down to their footings. The top cell of these foundations uses a mortar where anchor bolts or straps are generally placed. Brick or block foundations are extremely susceptible to high winds because the foundations are commonly not anchored which means the house can be easily moved from its original location with much less force than anchored homes. Pile or column foundations are more elevated with more emphasis on the anchoring of the floor. The piles or columns can fail when they are not braced

properly or dug deep enough into the earth's surface. This type of foundation is also weak where the floor attaches to the wall and the wall studs are nailed to the bottom plates. It is common in these types of structures for the wall to be damaged or lost and the floor to remain intact (Evans & Mandt, 2003).

#### **2.4.4 Wall Failures**

When facing extreme wind loads the wall connections can also experience different types of failure. Some of the failure types involve the type of materials that are used and the quality of work in the construction. Walls that are installed improperly such as freestanding walls that are not connected to other walls, the floor, or roof, make homes much weaker than ones that are properly installed. Loose siding or brick ties that are not connected properly to mortar joints can be dangerous because the structural integrity of the building will be in a weakened state. Furthermore, these materials can become dangerous debris in high winds in the event they are dislodged from the structure.

Large windows and doors also can play a large role in the strength of a home faced with enduring high speed winds because they have less bracing and are more susceptible to failure. When larger windows and doors fail, it allows wind to enter the structure and can create uneven pressures inside the structure. This places more stress on the structure. Consequently, the increase in stress also increases the probability of failure. Having enough doors or windows that will disrupt the framing continuity can also diminish the overall strength of the walls and therefore, also increase the probability of failure. This is often the reason why homes with attached garages are so vulnerable to wind damage. Garage doors are so susceptible because they are so large and lack enough support to withstand high speed winds. As a result, if the garage door fails, the collapse of the sidewalls is not uncommon.

#### **2.4.5 Roof Failures**

Lastly the roof is the most critical area that suffers significant amounts of damage during tornado and hurricane force winds. The rafter or trusses that support a roof structure up are typically connected to the top plate using a simple toenail connection. Although the IRC explains the construction of this connection and specifies it as minimum code, this connection is weak and highly susceptible to failure

due to high uplift force. Additionally, if the wood used to construct the rafters or trusses is connected with too many nails, in an attempt to strengthen the connection, splitting of the wood can occur. This makes the structure even more prone to failure. When it is a roof truss that fails due to lateral forces, it is usually because there is a lack of lateral bracing in the structure. Fasteners that do not reach the framing or staples that miss rafters can lead to the loss of roof decking, twisting, and uplifting.

There are also many different types of roofing materials (asphalt, shingles, tile, metal wood, shakes, etc.) that must be installed properly to protect a home from high winds. The way these materials are installed can also open up chances for failure. Overdriving and under-driving fasteners while attaching the roofing materials can cause it to crack or be loose. Misplaced fasteners can also allow for shingles to blow off in high winds (Evans & Mandt, 2003). The mortar or adhesive placed under the roofing material also is important in supporting the structure and protecting it from disengaging from a home during tornado or hurricane type winds.

## **2.5 Solutions to Combat Common Wind Induced Failures**

As hurricanes and tornadoes become a larger problem throughout the United States, more and more research is being conducted to develop possible solutions to combat the most common wind caused failures. With 90 percent of residential buildings in the United States being constructed of light-frame wood structures, solutions must be geared toward this type of building (Canbek et al, 2011). Extreme winds continue to cause catastrophic damage to residential buildings throughout the country and with people continuing to move into hurricane prone areas, it is imperative that solutions are developed to save lives and homes. In 2003, approximately 153 million people, representing 53 percent of the nation's population, lived in coastal areas of the United States (NOS, 2006). Figure 4 displays the increase of US population density along the coastal counties and states of the country. Throughout the last 30 years, the density along the coast continues to increase substantially. This number has continued to grow in recent years and is expected to grow in the future. With so many people living in hurricane prone areas, the amount of residential buildings on the coast has also grown substantially. Although coastal areas are at

the highest risk for hurricanes, other parts of the country are at risk as well, for both hurricanes and tornadoes. With this increased risk, the need to develop solutions is pressing now more than ever.

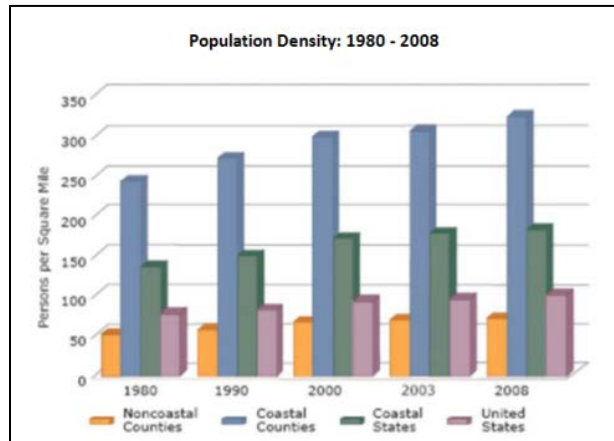


Figure 4: United States Population Density Coastal Counties and States *Coastal Trend Report Series*. Rep. National Oceanic and Atmospheric Administration, Sept. 2004. Web.

## 2.5.1 Construction Solutions

Some structural failures due to wind can be attributed to poor workmanship and inferior building materials. Generalities for resisting wind failure include correct installation. Based on the design specifications a structure should not fail, but a simple incorrectly installed connection could result in failure. Another common failure cause is the use of poor building materials. Contractors and builders may use inferior materials when building residential structures. The materials can include but are not limited to inferior lumber, inadequate connections, and low quality bonding agents. The lower grade materials used may not be capable to withstand the wind force while often a different, slightly higher quality material would have sufficed. A solution to some wind failures is as simple as ensuring contractors build the structure to meet the design specifications outlined by the designer, not just to meet just the minimum requirements of the building codes. The design specifications outlined by the designer are obtained from using the code but are specific to the structure and the area. Therefore, these specifications are usually above minimum code. Although building codes may serve as the general minimum requirements it is often not in accordance with designer's specifications to withstand extreme winds.

## **2.5.2 Continuous Load Path**

The common goal of all solutions to resist wind induced failure to residential buildings is to develop a continuous load path from the roof through the wall and to the foundation of a residential building. To develop this continuous load path, engineers must design connections from the roof to the walls, as well as from the walls to the foundation. Different connections throughout the building include: rafter to top plate, top plate to stud, floor to floor, stud to mudsill, and mudsill to foundation. Not only must a continuous load path be developed, but the materials used to create it must be capable of withstanding the force induced by the extreme wind.

## **2.5.3 Garage Doors, Doors, Windows**

A common failure in a residential building is the collapsing of “openings”. Openings refer to windows and doors of a building or anything that disrupts the continuous structure of the wall. These openings are often the first place of failure and can cause a domino effect on the structure by causing other failures. When a door or window is blown out, wind can enter the building, causing a buildup of pressure inside. This uplift force can cause the roof to fail in a way that would be uncommon if the openings had been able to withstand the wind. Garage doors are especially prone to failure because of the large surface area and few connections to hold it in place. Regular doors are similar to garage doors, in terms of being susceptible to failure. The most common solution to combat this problem is reinforcing the doors with stronger building materials. Another solution for regular doors in a building is as simple as strengthening the connections between the door to the wall. The stronger the nails in the connection, the more pressure the door can endure. For double doors, a center bolt can be used to reinforce the door. Windows and glass doors have similar solutions as well. The collapsing of these openings not only can allow wind into the building but also can be extremely dangerous because of the shattered glass debris and the danger that goes along with it. If a window is being built to withstand extreme winds, they should be made with shatterproof glass such as plexiglass. For all openings, the frame that supports these

windows should be reinforced and exceed the current building code specifications. Simple solutions have proven to be the best when dealing with openings in a residential structure.

#### **2.5.4 Roof to Wall**

The most research and testing has been conducted to find solutions to combat roof failures. Roof failures occur most frequently in extreme winds. Moreover, when they fail, catastrophic damage typically occurs from a variety of sources. The most vulnerable areas of a roof include the edges, corners, overhangs, and connections. Large overhangs allow the buildup of up-lift forces to generate under the overhang itself, which significantly increase the probability of the roof being torn off the rest of the structure. Solutions to overhang failure are as simple as limiting the length of the overhang or designing the overhang to detach from the roof under extreme winds. This will allow for the overhang to be blown away without damaging the rest of the roof. The only minor consequence involved with implementing this solution is that the blown away overhang will then become wind born debris. Although this is a concern, it will help diminish the number of roofs that are torn off completely, and therefore, theoretically, also decrease the amount of wind borne debris. Nevertheless, to limit the damage caused by the blow away overhangs, it should be constructed of light weight material.

Roof to wall connections are the first step in the continuous load path which is so important to the resistance of a building to extreme winds. Rafter to top plate and top plate to stud connections are needed to develop a continuous load path from the roof to the wall. Various connectors, including hurricane straps, hurricane tie downs, toe nail, roof clips, clinchers, typhoon clip, fiber reinforced polymer, and pocket clips among others, have been developed in an attempt to form this continuous load path. The most common connection used to mitigate damage due to extreme winds are hurricane straps. While these connectors can perform well in extreme winds, if they are installed incorrectly they will fail well before they are designed to. Another reason for failure is the presumption that the strength of the connection is directly proportional to the number of straps used. Many believe the more straps used, the

stronger the connection. This presumption has been proven false in multiple studies because of the intrusive nature of the nails, but is still widely misunderstood.

All but one of the connections mentioned are metallic and therefore intrusive connectors. When installing a metallic connection, nails or screws are used to secure the connector to the wood member. This intrusive action can reduce the strength of the wood members and cause the wood to fail before the actual connection. Therefore new developments are being made for “nail-less” connections that are adhesive based. (Ahmed, Canino, Chowdhury, Mirmiran, Suksawang, 2011).

### **2.5.5 Fiber Reinforced Polymer**

Fiber Reinforced Polymer (FRP) is garnering recognition as a potentially useful solution to strengthen connection. FRP is an epoxy type connection using fibrous connections to reinforce the strength of the overall connection. The FRP connection is being researched lately and it has been found to have many advantages. The advantages of this connection include its high tensile strength, ability to withstand harsh environments where metal connectors would deteriorate, flexibility, and its non – intrusive installation. Our research showed FRP has been tested against some metal connectors and has shown to outperform the metal in several aspects. It performed roughly 2 times better than hurricane straps in uplift capacity in two laboratory tests. The FRP connection also performed better than hurricane straps in both in and out of plane lateral load capacity. The FRP research is still relatively premature but has promising qualities. Nevertheless, further research will help decide whether this connection can be used to better resist roof to wall failure in extreme winds.



**Figure 5: Fiber Reinforced Polymer**



## **2.5.6 Anchoring the Structure to the Foundation**

If a continuous load path is developed from the roof to wall, the load must then be transferred from the wall to the foundation. Connections involved in this continuous load path are stud to sill plate and sill plate to foundation. Although failure in the foundation is much less common than roof or wall failures, they can happen. Sealing cracks in the foundation with epoxy are necessary to reduce the chance of failure in the foundation. Additionally, ensuring tie and bolt anchors are installed correctly from the wall frame deep into the foundation is necessary to diminish the probability failures like overturning. When installed correctly these anchorage systems have proven to resist the loads produced by extreme winds and continue the load to the ground.

## **2.6 Insurance, Losses, and Additional Mitigation Methods**

Insurance companies are greatly affected by wind damaged homes and high speed wind natural disasters. Most homes are insured with homeowner's insurance and under all homeowner's insurance policies wind damage is covered. Some forces of nature, such as flood and earthquakes however, are not covered under basic homeowner's insurance policies and require additional policies. With the jaw dropping losses insurance companies are suffering from recent natural disasters discussions among law and policymakers have surfaced regarding alternative insurance methods. These methods would include additional cover by developing a disaster insurance policy or the implementation of long term insurance coverage. These methods could protect both the insurance companies and homeowner better than the current system and be less harsh on the economy.

### **2.6.1 Homeowners Insurance**

Homeowners insurance is absolutely essential for all homeowners. Although it is not required by law, homeowners insurance can save a policy holder a tremendous amount of money in the event that anything happens to their home or belongings. As of May 2011, the national average premium for home insurance in the United States was \$730.28 (HomeInsurance.com, 2011). In calculating this cost,

insurance companies considered many different factors that may include but are not limited to, propensity for disaster to occur in the area, building material used in the home, building costs, neighborhood crime levels, size of the house, condition of the home, and distance to nearest fire hydrant and fire station. The answer to these questions will determine the cost of the policy an insurance company is willing to offer. There are different policies that exist, but the most common coverage homeowners will receive is called an HO-3 policy. Coverage under this policy can include something as small as vandalism or as large as damaged caused by most major disaster. However, the amount of coverage is determined by a liability limit. The liability limit determines how much a policy holder may receive if something happens to their home. The amount of coverage a holder has refers to the amount of money it would cost to rebuild their home given the price of materials and labor in the area. The amount is not the same as the purchased amount of the home, which accounts also for the value of the land. These limits usually start as low as \$100,000, but policies can be purchased with a much higher limit. Many experts recommend having a policy that provides a minimum of \$300,000 to \$500,000 of coverage, depending on the value of the home (Silverman, 2006). However, because wind damage is covered under homeowner's insurance, unlike damage caused by flooding and earthquakes, liability limits leave insurance companies particularly vulnerable to substantial losses in the wake of disaster such as a tornado or hurricane. Thus, with the recent history of these types of storms intensifying, it is imperative that insurance companies reform their methods for providing homeowners insurance in a way that is beneficial to both the consumer and the provider.

### **2.6.2 Insurance Losses after Katrina and Cause for Reform**

When Katrina made its second landfall on August 29, 2005, the category three storm, just downgraded from a category five one day prior, smashed the Gulf Coast with high velocity winds, 30 foot storm surge, heavy rain, flooding, coastal erosion, hail, and tornadoes. The storm caused an inconceivable amount of death, destruction, economic loss, and human suffering to the coastal regions of Louisiana, Mississippi, and Alabama. It is estimated that Hurricane Katrina generated approximately \$75 billion

worth of damage just to residential homes and structures. This gargantuan cost resulted in the approximate loss of \$40 to \$60 billion for private insurers of homes damaged or demolished as a result of Katrina, making it the costliest insured loss from a single event in United States history, exceeding both Hurricane Andrew in 1992 and the terrorist attacks on September 11, 2001. However, estimates of total economic losses, including insured and uninsured property and flood damage exceed \$200 billion dollars. Despite the severity of damages, insurers were well-equipped to manage the financial impact of the catastrophe. However, the losses sustained by insurers highlight the potential vulnerability if faced with a mega catastrophe such as two hurricanes similar to Katrina's strength in the same season. As a result, there appears to be growing support among policymakers and disaster policy experts to reexamine how the United States manages and finances disaster risk and seek new and innovative ways to do both. With this in mind economists suggests individual households have two options to reduce losses from disasters, pre-disaster mitigation, and risk financing (King, 2005) Together these two options can mitigate financial losses for insurance agencies, policyholders, and the economy.

### **2.6.3 Disaster Insurance**

Natural disasters and other catastrophes have had a more devastating impact on insurers over the last 20 years than any other time in history. Between 1970 and the mid-1980s, annual insured losses from natural disasters averaged between \$3-\$4 billion. The insured losses from Hurricane Hugo in 1989 exceeded \$4 billion and was the first natural disaster to inflict more than a billion dollars of insured losses. Since Hugo there has been a radical increase in the amount of insured losses caused by natural disasters. Hurricane Andrew in 1990 cost \$20 billion, the four hurricanes in 2004 (Charley, Frances, Ivan, and Jeanne) total \$29 billion, and Hurricane Katrina was responsible for approximately \$50 billion (Kunreuther, 2008). With the substantial increases in insured damaged disaster insurance is a possibility many policymakers and insurers are starting to consider. Disaster insurance would be similar to homeowner's insurance in that it would cover a policyholder's home and belongings if they were to get destroyed in a natural disaster but be a separate and additional cost to home insurance. It is beneficial to

both parties because it will offer extra coverage for homeowners while providing extra funds for insurers which will mitigate losses in the event of a natural disaster. The one pitfall regarding disaster insurance currently is price. Because this type of insurance would be entirely new, it is difficult for insurers to set accurate and competitive prices that would entice homeowners to purchase it. Nevertheless, disaster insurance may present effective measures to mitigate financial losses caused by a natural disaster.

#### **2.6.4 Long Term Insurance**

Another possibility insurance agencies should consider is marketing long term insurance contracts on residential property to provide stability to homeowners and encourage cost effective mitigation measures. Short term insurance policies often create significant social costs. Evidence from recent disasters reveals that many consumers fail to adequately protect or even insure their homes. This creates a significant welfare cost to themselves and a possible cost to all tax payers in the form of government disaster assistance. According to the Department of Housing and Urban Development, 41 percent of the homes damaged from the 2005 hurricane season were either underinsured or completely uninsured. Additionally, of the 60,196 homes that suffered severe wind damage, 23,000 (38 percent) did not have insurance against wind loss (Kunreuther, 2008). Because wind damage is usually covered under a basic homeowner's insurance policy, meaning approximately 23,000 homes were uninsured in a hazard prone area.

For a long term policy to be feasible, insurers would have to be able to charge a rate that reflects their best estimate of the risk over the particular time period. The uncertainty surrounding these particular estimates could be reflected in the premium as a function of time similar to the interest rates of a fixed mortgage rate that varies between 15, 25, and 30 year loans. This type of policy is advantageous from both the point of view of the policyholder and insurer. For the policyholder long term insurance provides the m with the stability and an assurance that their property will be protected for as long as they own it. This has been a major concern in hazard prone areas where insurers have cancelled policies after major disaster (Kunreather, 2008). For the insurer it increases revenue and make it difficult for homeowners to

cancel policies. Thus, both sides are protected from losing large sums of money in the wake of an insured disaster.

### **2.6.5 Encouraging Adoption of Mitigation Measures**

Long term insurance also provides economic incentives for homeowners to invest in cost effective mitigation measures where current annual insurance policies are unlikely, even if they are risked based, to do so. To illustrate this point, Howard Kunreuther provides a simple example in his paper *Reducing Losses from Catastrophic Risks Through Long-term Insurance and Mitigation*. In the article, Kunreuther writes:

“Suppose a family could invest \$1,500 to strengthen the roof of their house to reduce the damage by \$30,000 from a future hurricane with an annual probability of 1/100. An insurer charging a risk based premium would be willing to reduce the annual charge by \$300 (i.e.  $1/100 \times 30,000$ ) to reflect the lower expected losses that would occur if a hurricane hit in the area in which the policyholder was residing.... Under the current annual insurance contracts many property owners would be reluctant to incur the \$1,500 cost because they would only receive \$300 back the next year” (Kunreuther, 2008).

This means that because in the current system insurance policies are renewed annually, an insurer would only credit the policyholder for their efforts to reduce damage to their home one time. If a long term insurance policy system was used, policyholders may be more willing to protect their house because as in the example presented above, the roof renovations would pay for itself in 5 years. In the current system incurring an upfront cost of \$1,500 and only receiving \$300 may not seem worth the \$1,200 expense overall. Additionally, by linking mitigation expenditures to the structure, rather than the property owner, annual payments would be lowered and this could be a selling point for mortgages. As a result, banks would be more protected against catastrophic loss of property, insurer’s potential losses from major disasters would be reduced, and the, most importantly, the general public would be less likely to need large amounts of tax dollars going to disaster relief (Kunreuther, 2008). Finally, because older houses are

not up to code, insurers can have inspectors examine a potential policyholder's house to determine where affordable and effective mitigation measures could be installed. Ultimately, long term insurance could potentially reduce the amount of physical and financial damage natural disasters threaten to inflict upon the United States.

## **2.7 Summary**

Due to the increasing concerns regarding climate change and increased storm strengths, it is necessary to design residential buildings to support greater wind loads. Through this project, we sought to design a single family home with affordable and effective preventative measures to limit wind damage. We believe adjustment can be made to existing code to better protect homeowner's from having their homes destroyed. These adjustments will also help the economy by limiting the damaged caused and the resulting insurance pay outs and government emergency funds. Our research to familiarize ourselves with the causes and effects of climate change, current residential codes, common wind loading failures, current solutions to limit such failures, and its effect of insurance companies and the economy assisted our efforts to identify the most effective methods and measures to available to utilize in the design of our structure. Ultimately, we hope this design will become a model example for contractors and code developers to best protect homes from extreme wind loads.

### 3.0 Methodology

The challenge of our project was to design a two story residential structure capable of better resisting extreme wind loads. To accomplish this goal we employed the help of a local insurance agency, construction contractor, our project advisors, and WPI's Civil Engineering Lab Staff to address the following objectives:

- To identify the most vulnerable areas of a home exposed to extreme wind loading
- To determine features available to limit the damage caused by wind loading on those vulnerable areas
- To test and compare available features' strengths against the individual and labor costs necessary to install in a home
- To design a single family home with the most effective and affordable features to limit damage from extreme wind loading

Fully completing each objective was absolutely essential to the success of the project because all the objectives build off one another. Because wind caused failures can occur at various locations around a house, it was important to identify the areas that are most vulnerable. Identifying these areas enabled us to focus on designing the most critical structural elements with the potential to combat extreme wind forces more efficiently. Strengthening these structural elements required us to determine various connection details, anchoring mechanisms, or technologies available to prevent or limit wind related damage. After compiling a list of these features, we tested them in the lab and compared their relative strength's against uplift force verse total costs to install within the structure. This was necessary to determine the most effective and most affordable preventative measures available to limit or prevent wind damage. Finally, we utilized the features with the best test results and both the Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) to design our structure with the capability to withstand 110 mile per hour wind gusts. Ultimately, the final design of our house was the result of both qualitative and quantitative methodological approaches that are presented in greater detail throughout this chapter.

### **3.1 Identifying the Most Critical Areas Vulnerable to Wind Caused Failure**

Our first objective required us to determine the features of a residential home that were most susceptible to wind caused failures. We needed to understand the different types of failures to ensure we focused our attention on designing our home to combat the most common failures for the most critical elements. This information also facilitated our determination of features to test that we considered installing within our structure. Essentially, the information gathered to complete this objective laid the ground work for our project and several other project objectives. To fulfill this objective we conducted extensive research regarding wind caused failures. Ultimately, our research revealed several areas vulnerable to wind caused failures. Some common areas susceptible to wind related failures included the foundation and shearwalls. However, we determined from our research that the most critical element, at the greatest risk of wind caused failure was the roof. To confirm our research was correct, we contacted Hanover Insurance Group in Worcester, Massachusetts and interviewed a Property Adjuster to confirm our data. We tried contacting other authorities on this issue, but they would not return our phone calls or e-mails.

#### **3.1.2 Interview with Hanover Insurance Group**

Upon contacting Hanover Insurance Group for information regarding coverage for single family homes, we were put in contact with Jack Burlas, a property adjuster. Burlas initially provided us with information regarding the different policies they offered for homeowners insurance. He explained examples of different types of damages and how they were covered under each policy. He also informed us that if a homeowner strengthens their home against high speed winds by increasing the strength of the connections or retrofitting the home, they can decrease the price of their policy by a percentage based on the number or types of retrofits. Finally, to conclude the interview we asked Burlas what type of damage claims were most common after high speed wind storms. Without hesitation he replied roof damage was absolutely the most common. To confirm this assertion for us, he provided us with data on the different



types of roof damage that were been evaluated by Hanover Insurance. This information was crucial for the completion of our first objective and the project.

## **3.2 Identifying Measures Available to Prevent Wind Damage to the Roof**

Our project's second objective focused on identifying affordable and effective measures capable of preventing wind related damage to the roof structure. In our first objective, we identified the roof as being the most critical structural element of a house, at the greatest risk of wind failure. Therefore, to accomplish this objective, we utilized the information obtained from our research and interview with Jack Burlas to identify appropriate measures available to improve roof design. In addition, we analyzed different roof designs and measures available to strengthen roof connections. From our background research, analysis of architectural drawings for homes recently renovated to withstand greater wind loads, and suggestions from our advisors, we determined the roof rafter to top plate connection was the most critical aspect of the roof to focus our experimental testing program on. The test program is designed to help us identify critical features and details of the roof that could benefit from an additional strengthening and reinforcement.

### **3.2.1 Focusing Testing on the Rafter to Top Plate Connection**

The next phase involved testing the measures we identified to complete this objective. Our project advisors recommended we concentrate our testing on a one specific area of the roof because the amount of time required to produce reliable results is immense. As a result, we decided to specifically focus on the rafter to top plate connection because we determined this was the most critical element of a roof structure. These connections transfer loads from the structural ridges of the roof the top plate of the home. If they fail, the entire roof is at a greater risk of being blown off, exposing the rest of the structure to the outside elements. Once the structure is exposed to these elements, wind damage is no longer the only cause for concern. Thus, identifying ways to strengthen these connections through testing and relying on our research to improve other elements of the roof design was essential for the successful completion of the project.

### **3.2.2 Preventative Measures to Strengthen the Rafter to Top Plate Connection**

To determine the most effective measures available to strengthen a roof and the respective costs of these measures we contacted two contractors, Francis Harvey & Sons, Inc. and F.L. Caulfield & Sons, Inc. F.L. Caulfield & Sons recently renovated a house in Cape Cod to withstand greater wind loads. To assist in our analysis they provided us with the architectural drawings outlining the renovations made to the structure. The primary preventative measure utilized to strengthen the rafter to top plate connection was the installation of hurricane straps. A hurricane strap was installed at each rafter to top plate connection to supplement the toenail connection already in place. Thomas Caulfield, the owner of F.L. Caulfield & Sons, assured us that in his experience hurricane straps were the premier preventative measure to diminish the probability of a roof blowing off from high speed winds. He also added that he believed they work so well that he even installed them in his house in Cape Cod. Nevertheless, some of our research claimed that adhesive connections may be stronger.

We then focused our efforts on identifying a nail-less wood connection by examining different wood adhesives. Some of our research claimed that the intrusive nature of the nails caused wood to fail before the actual connection. As presented in our background chapter, we identified the recently developed fiber reinforced polymer, FRP, as the most advance adhesive available today. However, because the advanced technology behind FRP was developed within the last half decade, the cost to obtain the stalwart adhesive was too expensive. An alternative to FRP, our project advisors recommended we test Kevlar straps. They suggested this because Kevlar straps required a special adhesive to bind to the wood, which combined with the exceptional strength of the Kevlar, could simulate the strength of FRP. Finally, we also chose to test a wood adhesive to obtain information on nail-less connection. The wood adhesive we selected was Liquid Nails Adhesive. Ultimately, we utilized and combined these three features to develop four different connections to test.

Once we identified these three measures as having the capability to strengthen the rafter to top plate connection we contacted Francis Harvey & Sons to inquire about labor costs associated with installing each of our four connections versus a typical toenail connection. Francis Harvey & Sons

provided us with these labor costs along with additional information regarding the amount of time required for installation. From this information, we were able to begin the process of determining the most effective and affordable connection available to mitigate wind related damage.

### **3.3 Testing Connection Types to Strengthen the Rafter/Top Plate Connection**

After identifying different connection types to strengthen the rafter to top plate connection, our next objective required us to load test these different connections. For all tests, two 14.5” long Spruce-Pine-Fir (SPF) No.2 2 x 4’s were used to simulate the top plate. The two pieces of lumber were nailed together using 10d common nails with one nail located 3.5” from the ends of the sample. After the top plate sections were assembled, we drilled 1” diameter holes using a drill press with a 1” drill bit, centered 1” from the ends of each section (See Figure 6). These holes enabled us to install the specimens into the Tinius Olsen Universal Testing Machine. Before placing the sample in the machine, we had to connect our simulated rafter to the top plate. We cut 15” long sections from Spruce-Pine-Fir (SPF) No.2 2 x 12’s. These samples were then connected to the top plate with the connection types we identified in our second objective. Once the specimens were fully constructed, they were placed in a design apparatus that allowed us to simulate uplift force capable of pulling the rafter from the top plate. Each connection type was constructed and tested five times. Photographs of the design, construction, and testing of the specimens are available in below and in Appendix F. The different specimens and apparatus were positioned in the Universal Testing Machine and tested. The computer controlled Tinius Olsen testing machine applied a tensile force to the connection at a rate of a half inch per minute (1/2 inch/min.) and was set to monitor load versus cross head displacement. Once each specimen failed, the test machine provided us with computerized data that allowed us to analyze our results more efficiently.



Figure 6: Drilling 1” holes in top plate section segment (left) and cutting 2 x 4 and 2 x 12 sections (right)

### 3.3.1 Nail Based Connections

Our first test focused on the most basic of all roof rafter to top plate connections, a toenail connection. This connection type bonded the rafter to the top plate with 3 16d common nails. The fasteners were nailed at as close to a 30° angle as possible to ensure one third of the nail height was driven through the rafter into the top plate. Two nails were used on one side while the third was installed on the opposite face of the rafter between the original two nails. Our second tests followed the same procedure as the first but included the installment of a hurricane straps to supplement the toenail connection. The hurricane straps were aligned and installed using the manufacturing standards and 5 8d hot-dip galvanized nails to connect the strap to both the rafter and top plate. (See Appendix E for Toenail Design Analysis)



Figure 7: Toenail and hurricane strap connection (left) toenail connection (right)

### 3.3.2 Adhesive Based Connections

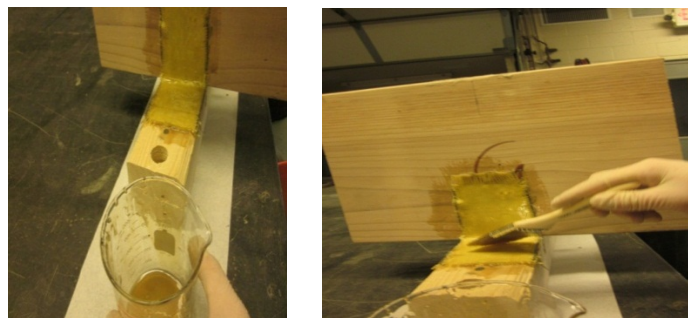
The next batch of tests involved our adhesive based connections. The first of these connections we tested were Liquid Nails. We applied the Liquid Nail Adhesive to the area where the rafter and top plate connected to one another. After the samples were attached the adhesive was also applied to the outer

edges of the connection to ensure a sturdy connection. The specimens with the Liquid Nail connection were then set aside to cure for 36 hours. The fourth tests used the same Liquid Nail connection in test three but also included the installment of a hurricane strap. This test was similar to test 2, but provided us with important information regarding the claims we encountered in our research that argued the intrusive nature of the toenail can weaken the wood and cause the wood to fail before the connection.



**Figure 8: Liquid Nails being applied on bottom (left) and along edges (center) and Liquid Nails with hurricane strap (right)**

Our final tests examined the strength of Kevlar straps bonded to the wood samples with a strong adhesive to simulate the cutting edge fiber reinforced polymer (FRP). The Kevlar straps and adhesives were used as an alternative to FRP because FRP was hard to obtain and very expensive. The adhesive, which was a mix of a fiberglass resin and liquid hardener, was applied to the faces and intersection point of the rafter and top plate. The 8" x 3.5" strap was then placed the area covered by the adhesive. Once the straps were in place the adhesive was then reapplied atop the Kevlar straps and set aside to cure. The amount of time required to install this connection was approximately 2 hours. However, the connection did not reach full strength until a full day of curing.



**Figure 9: Kevlar strap connection with adhesive**

## **3.4 Designing the Single Family Home**

After thoroughly researching and identifying the areas of residential structures that are vulnerable to wind caused failures, our next objective was to design our single family, two story home to withstand extreme wind loading scenarios. The structure was designed to meet both the Allowable Stress Design (ASD) and the Load and Resistance Factor Design (LRFD) specifications. To accomplish this we had to select a hypothetical location for construction, calculate the different loads, and apply the appropriate load combinations that would be exerted on the structure. Once the structure was designed entirely, we drafted our structure with AutoCad and Revit Architecture.

Before we could commence our calculations, we had to select a hypothetical location for our house to be constructed. This was essential because it provided us with accurate loading magnitudes and information regarding the types of lumber species available for wood construction. After selecting a location, we investigated the different types of lumber used for wood construction. To gather this information we contacted a local home improvement store.

### **3.4.1 Load Calculations**

Prior to starting the actual design, we had to identify and calculate all of our loads. Loads can act in a variety of different ways. Some loads act only vertically, some only act horizontally, and some can have both a vertical and horizontal components. Vertical loads are primarily carried by a system of joists, beams, walls, or posts, while lateral loads are handled by shearwalls. It was essential that all these load carrying members were designed properly to ensure they will support the force or load that will be exacted upon it. Ultimately we identified several loads that we were required to calculate. These loads included a dead load for our roof, floor, and walls, a live load for our roof and floors, a snow load, and a vertical and lateral wind load. To calculate our loads the design examples in *Design of Wood Structure* to ensure all our load calculations were calculated correctly.

### **3.4.2 Load Combinations**

After computing the design values of each load that will be exerted on our structure, we combined the loads by using the proper load combinations. Load combinations are used to check the strength of a structure. ASD and LRFD specifications provide several load combinations and are the only two methods of design permitted by ASCE 7, IBC, NDS, and SDPWS. Both of these methods compare the demand on a structure to the provided strength capacity of a particular member. Prior to 2005, LRFD was not recognized as an acceptable design method. However, unlike ASD, this method addresses factors of safety by specifically accounting for possible variations in demand with a load factor, and possible variations in capacity with a resistance factor. On the contrary, ASD demands are calculated using loads that would be commonly anticipated to occur and, although its factors of safety do provide ample protection of life and serviceability, they have not been rationalized to the same extent as the newer LRFD method. Nevertheless, because both methods are recognized as acceptable design methods, we decided it was essential to design each member using both ASD and LRFD load combinations and specifications.

### **3.4.3 Drafting the Home**

Once the home design was calculated, the ensuing progression was the visualization and creation of the house using computer aided drafting. The drafting took place in three major phases. The first phase used AutoCAD 2D drafting to create a framing plan. The framing plan consisted of North, South, East, and West elevations and a plan view of the home. This drafting stage allowed for the determination of stud, joist, and opening layout.

After the framing plan was complete AutoCAD 3D drafting was used to conceptualize the different components of the hip roof in three dimensional space. This design for the hip roof consisted of several different truss combinations before the final design was decided upon. The three dimensional drafting allowed us to develop the roofs truss system, common rafters, joists, top plates, and structural ridges to conjoin and function as a structure.

The final phase of drafting utilized Revit. Revit transformed the previous drawings into a realistic architectural representation of the home. This allowed for the completed home to be viewed from any perspective. The drafting of the home converted the calculations and dimensions determined, into the actual home, completing the design.

### **3.5 Summary**

The goal for our project was to design a single family, two story home that was capable of withstanding extreme wind loads. To achieve our goals, we completed four objectives. These objectives included:

- Identifying the most vulnerable areas of a home subjected to extreme wind loading
- Determining features available to limit the damage caused by wind loading of such vulnerable areas
- Testing and comparing available features' strengths against the individual and labor costs necessary to install in a home
- Designing a single family home with the most effective and affordable features to limit damage from extreme wind loading

To fulfill these objectives we conducted significant research to better understand the scope of our project. We focused our research on single family home construction, failures caused by wind loading, and current solutions available to mitigate those failures from occurring. To fulfill our first objective we conducted an interview with property adjusters from Hanover Insurance Group. The information we obtained from that interview was essential for the completion of our second objective because it enabled us to concentrate our research specifically on the roof. By analyzing drawings and speaking to people with experience in the construction and design field, we were able to determine the best available features to limit damage to the roof. These features were then load tested and compared financially to and determine the most effective and most affordable. Finally, the rest of the structure was designed to meet the specification both the Allowable Stress Design and Load and Resistance Factored Design. When the structure was



completely finished, AutoCad and Revit Architecture were employed to develop two and three dimensional renderings of our house.

## **4.0 Results**

The fundamental principle behind this project was to prevent the unwarranted destruction of homes due to extreme wind loads. Utilizing an effective wind design can help protect entire communities, which is important for the financial well-being of the homeowner and the rest of the economy. When an entire community is destroyed by a tornado or hurricane the insurance and government funds required to manage the situation can be astronomical. Therefore, it was our goal to design a home capable of withstanding these loads to safeguard homes in the future. To achieve this goal we established several objectives that we wished to accomplish. Consequently, this chapter is devoted to demonstrating the findings we obtained from our methodological approaches to complete these objectives.

### **4.1 Testing Results**

For the joint pullout test, load and deformation data was collected through the Tinius Olsen Universal Testing Machine. With this information, we were able to transfer the data to Microsoft Excel where we were able to work with the data to produce graphs and tables.

The first step in analyzing our data was to produce a load versus deformation curve for each individual test. These graphs varied from test to test, as well as within some tests of the same connection type. Through the load versus deformation graphs we were able to obtain valuable information per each connection. The most vital information obtained from the graphs was the load at failure. We decided to define failure in two unrelated sets. The first failure set, Limit State 1, the load at failure was identified as the point where the load versus deformation graph deviated from a linear relationship and began to fail locally. This limit state simulated complete failure. For the second failure set, Limit State 2, the load at failure was the maximum load before deformation reached 1/8 of an inch, which we defined as an acceptable deformation. The graph below displays the point where each failure set was determined.

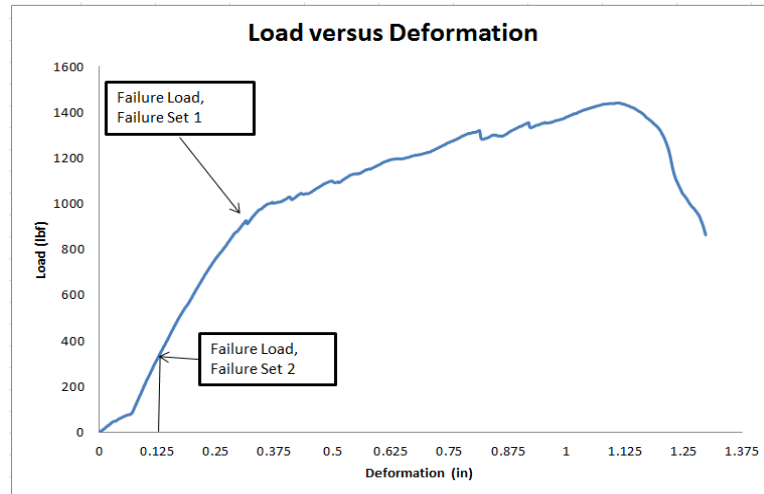


Figure 10: Failure Loads, Limit State 1 & 2 Showing Pullout Load vs. Axial Deformation

Limit State 1 shows the load the connection was able to withstand before complete failure. This was important to determine because the connection was still intact at this point and kept the rafter from separating completely from the top plate. The disadvantage of Limit State 1 is the deformation at this load. The deformation values along the x axis of the load versus deformation graphs represent the distance the rafter and top plate separate as the load is applied. For many of the failure loads in Limit State 1, the corresponding deformation is undesirable. A larger deformation will allow destructive elements such as rain and debris to enter the house and cause severe damage.

Limit State 2 was important because it demonstrated the maximum load the connection can withstand without deforming more than a 1/8". This smaller deformation will diminish the separation between the rafter and top plate, preventing water and debris from entering the house. The failure load for Limit State 2 was determined before the connection failed completely, but utilizing an acceptable deformation permitted us to analyze our results more efficiently.

After the failure loads were determined for each test, a load versus deformation graph (one for each Limit State) was created with the maximum load at Limit State 1. The new graphs allowed us to focus our data and eliminate data we considered unnecessary.

Two tables were created for each connection type, one for Limit State 1 and one for Limit State 2. For Limit State 1, the table includes the failure load and ultimate deformation. The table for Limit State 2

includes only the failure load, because the deformation was consistently regulated to be 1/8". In each table, an average and standard deviation was calculated for each property. Ultimately, these tables were crucial to our comparisons of each connection at the conclusion of the testing.

### 4.1.1 Toenail Connection

The toenail connection was the first connection tested. This served as our baseline for comparison because it represented the minimum connection required by code. We followed code, using three 16 d common nails. The results for Limit State 1 and Limit State 2 were as follows:

**Table 2: Accumulation; Toenail, Limit State 1 (left) & 2 (right)**

Toenail Test Accumulation		
Limit State 1		
Test	Failure Load (lbf)	Failure Deformation (in)
1	134	0.06
2	207	0.12
3	170	0.12
4	251	0.08
5	151	0.07
Avg	182.6	0.09
Std Dev	46.89	0.03

Toenail Test Accumulation	
Limit State 2	
Peak Load before 1/8" Deformation	
Test	Failure Load (lbf)
1	174
2	216
3	172
4	316
5	197
Avg	215
Std Dev	59.28

The toenail connection performed slightly worse than expected. The average withdrawal value per nail is approximately 80 pounds per nail. Because we used three nails, the average failure load should be 240 pounds per foot. However, according to the Table 2 above our average was 182.6 pounds per foot. In all five tests, the toenails pulled from the top plate, often in combination with wood cracking. When comparing the two Limit States, Limit State 2 outperformed Limit State 1 by roughly 30 lbf but had a larger standard deviation. This information tells us that the toenail connection began to fail, on average, before it reached an 1/8" deformation, which is defined in the IRC as an acceptable deformation. As shown in table 2, the average deformation for Limit State 1 was only .09 inches. The load versus deformation graphs for Limit State 1 and Limit State 2 represent the same data for each test, with the graphs for Limit State 2 continuing further than those of Limit State 1. A typical load versus deformation graph, up to the failure load, for each limit state is below.

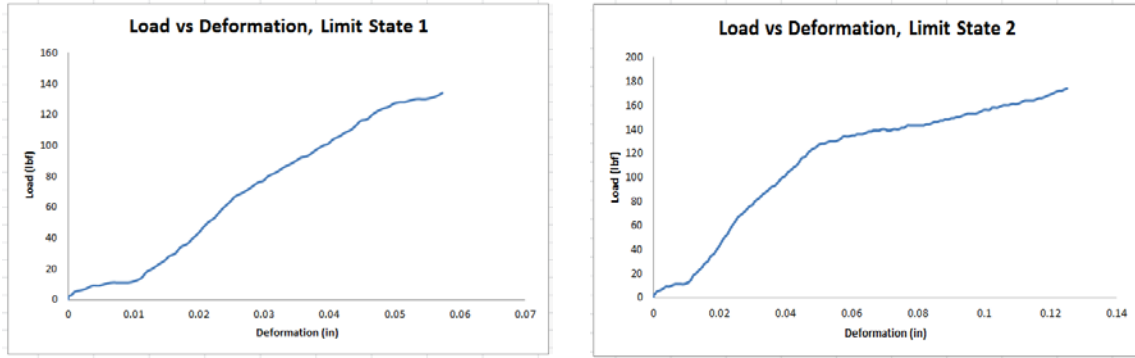


Figure 11: Load vs. Deformation Curves for Toenails for Limit States 1 (left) & 2 (right)

The graphs represent the relationship between the load applied and the deformation that occurred in the connection. The first graph, Limit State 1, represents a portion of the second graph, Limit State 2. This is because the failure load was determined to occur before deformation reached 1/8 of an inch.

#### 4.1.2 Toenails Combined with Hurricane Straps

The next connection tested was a toenail connection in conjunction with a hurricane strap. Hurricane straps are commonly used in areas prone to high winds because they are effective, inexpensive, and easy to install. The strap is attached to the rafter and the top plate using five 8d common nails at each connection. Testing hurricane straps allowed us to compare the results against that of only toenailing and observe how much strength the hurricane strap actually adds. The results of the test for Limit States 1 and 2 were as follows:

Table 3: Accumulation; Toenail & Hurricane Strap, Limit State 1 (left) & 2 (right)

Toenail with Straps Accumulation			Toenail with Straps Accumulation	
Limit State 1			Limit State 2	
Test	Failure Load (lbf)	Failure Deformation (in)	Test	Failure Load (lbf)
1	840	0.25	1	440
2	967	0.24	2	642
3	925	0.31	3	321
4	1032	0.25	4	651
5	960	0.28	5	430
Avg	944.80	0.27	Avg	496.8
Std Dev	70.18	0.03	Std Dev	144.44

The addition of the hurricane strap added significant strength to the rafter to top plate connection. For Limit State 1, the hurricane strap made the connection more than five times stronger than toenails

alone. For Limit State 2, the hurricane strap more than doubled the failure load, which is still a significant improvement. In this connection, Limit State 1 had a higher failure load, but also deformed over a quarter of an inch. A sample of the load versus deformation graphs (to failure) for test 1, are available below.

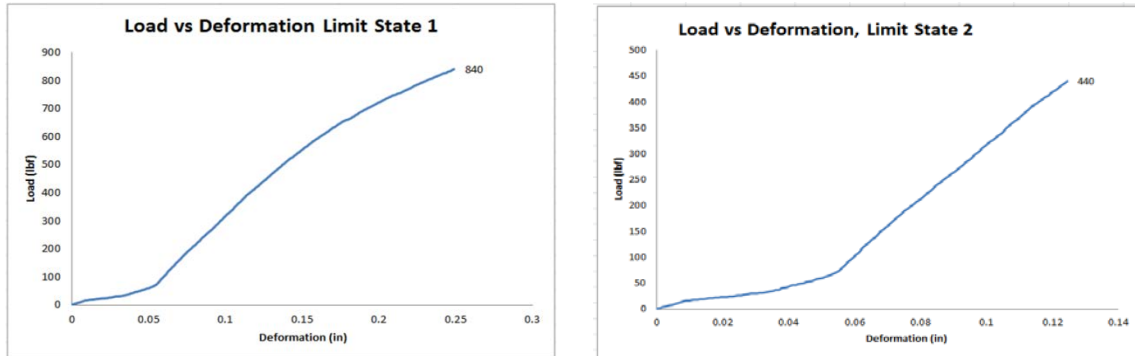


Figure 12: Load vs. Deformation Curves for Toenail and Hurricane Strap for Limit States 1 (left) & 2 (right)

The data from the graph depicts Limit State 1 allowed for a higher failure load but also a higher deformation, while Limit State 2 ends when deformation reaches .125 inches. Both Limit States are helpful when comparing tests to one another.

### 4.1.3 Liquid Nails

As an alternative to the toenail connection, we wanted to design a connection using an adhesive. We chose to use an adhesive because of the non-intrusive behavior of the connection. As presented earlier in this report, some of our research had indicated that the intrusive nature of a nail weakened the wood and caused the wood to fail, not the connection. The adhesive used was heavy duty Liquid Nails, a construction adhesive used for a variety of application, including treated lumber. The data from the tests is presented below.

Table 4: Accumulation; Liquid Nails, Limit States 1 (left) & 2 (right)

Liquid Nails Accumulation		
Limit State 1		
Test	Peak Load (lbf)	Failure Deformation (in)
1	259	0.04
2	470	0.12
3	336	0.12
4	103	0.01
Avg	292	0.07
Std Dev	153.22	0.06

Liquid Nails Accumulation	
Limit State 2	
Peak Load before 1/8" Deformation	
Test	Peak Load (lbf)
1	259
2	470
3	336
4	103
Avg	292
Std Dev	153.22

In the Liquid Nail connection, the connection begins to fail locally extremely early and the linear relationship in the load versus deformation graph is lost, on average at .07 inches, well below the .125 inches allowed for Limit State 2. This tells us the Liquid Nail connection, on average fails locally before deformation reaches .125 inches. The Liquid Nail connection is best compared to the toenail connection and performed better for both Limit States. The load versus deformation curves for Liquid Nails were different than the previous connection curves because the Liquid Nails would completely fail suddenly. This typical relationship for Limit State 1 and 2 for the Liquid Nail test can be viewed below.

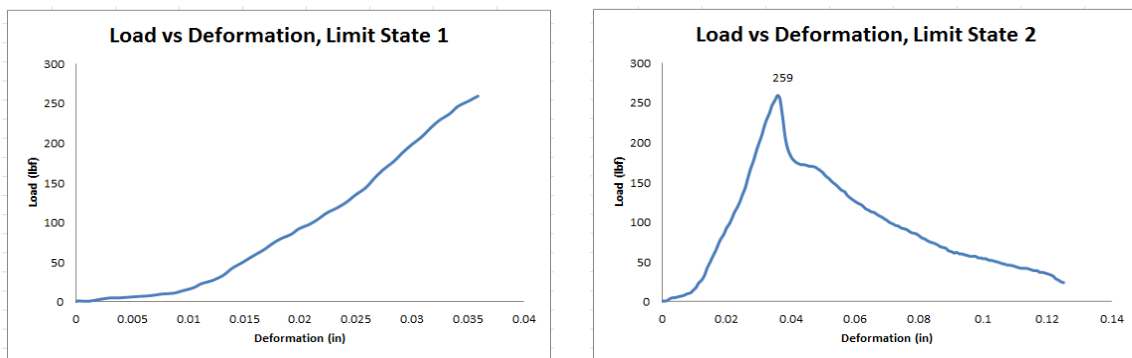


Figure 13: Load vs. Deformation Curve for Liquid Nails for Limit States 1 (left) & 2 (right)

#### 4.1.4 Liquid Nails in Combination with Hurricane Straps

The next connection tested was the Liquid Nail adhesive in conjunction with a hurricane strap. This test was similar to the toenail and hurricane strap connection, but replaced the toenail with the less intrusive adhesive. The information obtained from this test supplied us with more data regarding the strengths of the nailed and nail-less connections. The information gathered from the tests:

Table 5: Accumulation; Liquid Nails and Hurricane Strap, Limit States 1 (left) & 2 (right)

Liquid Nails and Straps Test Accumulation		
Limit State 1		
Test	Peak Load (lbf)	Failure Deformation (in)
1	1397	1.10
2	5833	1.55
3	5370	0.93
4	1738	1.26
5	1675	1.19
Avg	3202.6	1.20
Std Dev	2199.74	0.23

Liquid Nails and Straps Test Accumulation	
Limit State 2	
Peak Load before 1/8" Deformation	
Test	Peak Load (lbf)
1	82
2	4340
3	4499
4	523
5	628
Avg	2014.4
Std Dev	2205.80

With respect to Limit State 1, the Liquid Nails in combination with hurricane straps far outperformed toenails in combination with hurricane straps. The average peak load more than tripled in our tests. The failure deformation, however, also increased by more than four times when using Liquid Nails in place of toenails.

When observing Limit State 2, it is clear the Liquid Nails and hurricane strap again outperformed the toenails and hurricane strap connection. With the failure deformation kept constant at .125 inches, the Liquid Nails combination had a peak load more than four times the toenails combination.

The performance consistency of the Liquid Nails in conjunction with the hurricane straps was a negative though. The standard deviation of more than 2000 for both Limit States far exceeds the standard deviation for any other test. This can be attributed to tests two and three, which far exceeded the peak load of any test. However, tests one, four, and five performed similarly to the other tests. We believe tests two and three exceptionally high peak loads were the result of our wood specimens performing better than usual. Nevertheless, a typical load versus deformation graph for both Limit State 1 and Limit State 2 are below:

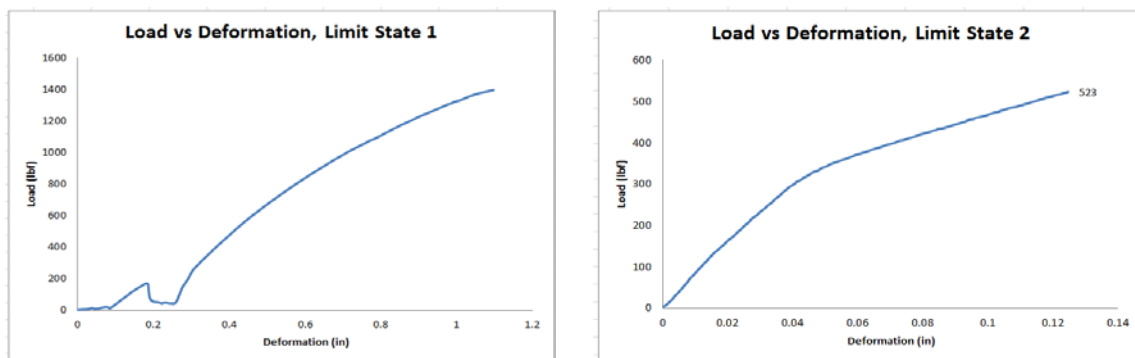


Figure 14: Load vs. Deformation Curve for Liquid Nails and Hurricane Strap for Limit States 1 (left) & 2 (left)

#### 4.1.5 Kevlar

The final connection tested was the use of Kevlar straps attached to the wood using a special adhesive. This served as an alternative connection to the cutting edge FRP. The results from our tests were as follows:



Table 6: Accumulation; Kevlar Straps, Limit States 1 (left) and 2 (right)

Kevlar Test Accumulation		
Limit State 1		
Test	Peak Load (lbf)	Failure Deformation (in)
1	115	0.07
2	515	0.09
3	362	0.09
4	160	0.16
5	620	0.12
Avg	354.4	0.10
Std Dev	218.80	0.03

Kevlar Test Accumulation	
Limit State 2	
Peak Load before 1/8" Deformation	
Test	Peak Load (lbf)
1	132
2	515
3	362
4	106
5	620
Avg	347
Std Dev	227.64

As the tables display, the Kevlar connection did not perform as well as many of the other connections tested. The tests were rather inconsistent, producing a standard deviation of over 200. A typical load versus deformation graph for each Limit State is below:

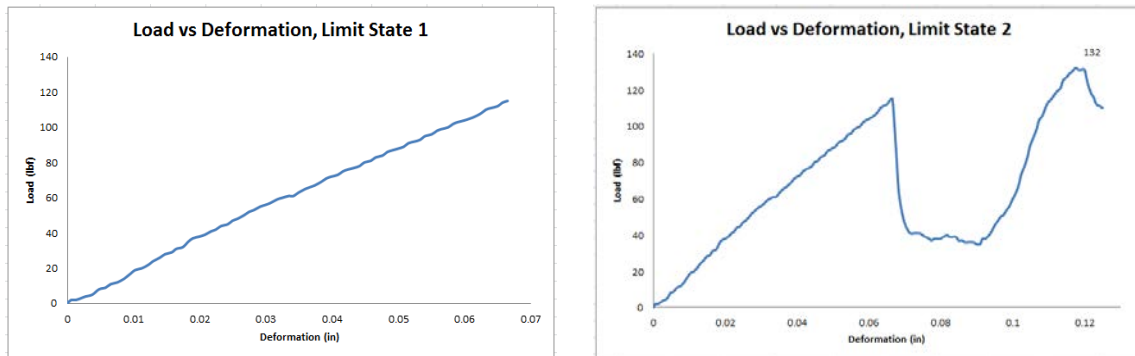


Figure 15: Load vs. Deformation Curve for Kevlar for Limit States 1 (left) & 2 (right)

The graph for Limit State 2 continues past the end of the graph for Limit State 1 because the deformation has yet to reach .125 inches. This is consistent for the Kevlar tests, as the average failure deformation for Limit State 1 is .1 inches.

The failure of the Kevlar tests was consistently due to the adhesive failing to stay attached to the wood, not failure of the Kevlar itself.

## 4.2 Connection Comparison

The goal of this study was to test and compare the multiple variations of rafter to top plate connections. With the information obtained we were able to compare our results in several ways. A table

displaying the average failure load and standard deviation for each feature was created for both Limit State 1 and Limit State 2.

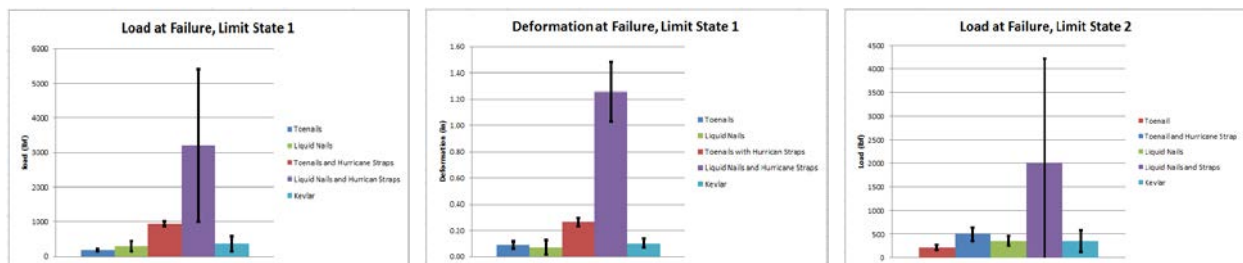
**Table 7: Comparison of All Tests for Limit State 1**

Total Accumulation, Limit State 1					
Test	Toenail	Toenail w/ Strap	Liquid Nail	Liquid Nail w/ Strap	Kevlar
Failure Load (lbf)	182.60	944.80	292.00	3202.60	354.40
Std Dev	46.89	70.18	153.22	2199.74	218.80
Failure Deformation (in)	0.09	0.27	0.17	1.20	0.10
Std Dev	0.03	0.03	0.04	0.23	0.04

**Table 8: Comparison of All Tests for Limit State 2**

Total Accumulation, Limit State 2					
Test	Toenail	Toenail w/ Strap	Liquid Nail	Liquid Nail w/ Strap	Kevlar
Failure Load (lbf)	215.00	496.80	355.00	2014.40	347.00
Std Dev	59.28	144.44	106.78	2205.80	227.64

These results can also be compared graphically. We developed three graphs with the information contained in the tables above to demonstrate the load and deformation at complete failure and the load at the acceptable deformation. Ultimately, these graphs, which are presented below, were crucial in our determination of the most effective connection type.



**Figure 16: Comparison of All Tests for Failure Load at Limit State 1 (left), Deformation at Limit State 1 (center) Failure Load at Limit State 2 (right)**

For Limit State 1, the Liquid Nails outperformed the toenails in terms of failure load but not in failure deformation. Also, the standard deviation for the Liquid Nails was far higher than the standard deviation for the toenails. This higher standard deviation for the Liquid Nails can be attributed to the

inconsistency in test results which was not apparent in the toenail test results because the toenails were more consistent.

The combination of Liquid Nails and a hurricane strap outperformed the toenail and hurricane strap combination in terms of failure load but did not in terms of failure deformation. The standard deviation when using Liquid Nails and a hurricane strap was extremely high for the failure load, nearly 20 times higher than that of the toenail and hurricane strap. This high standard deviation was also a result of the Liquid Nail inconsistent performance.

Kevlar performed rather poorly in both Limit State 1 and Limit State 2. The load at failure fell well short of the toenail in combination with a hurricane strap as well as Liquid Nails in combination with a hurricane strap. The only positive for the Kevlar was the deformation at failure for Limit State 1; however this was far outweighed by the poor performance in terms of load at the acceptable deformation.

When comparing Liquid Nails to toenails, whether in combination with a hurricane strap or not, the Liquid Nails had a higher failure load but also deformed more at this point of failure. The Liquid Nails also had less consistent results than that of the toenails and resulted in a higher standard deviation.

Another important comparison is the effect the hurricane strap had on the connection. When a hurricane strap was added to the toenail connection, the failure load was increased roughly five times. The addition of the hurricane strap to the Liquid Nails increased the failure load nearly nine times.

Another aspect to consider when comparing the different types of connections is the cost of each one. As a group, we priced out each connection for materials as well as labor for installation. The specific costs for each connection can be found in the appendix but the following table represents a comparison of the cost for each connection.

**Table 9: Cost Analysis**

Costs					
Test	Toenail	Toenail w/ Strap	Liquid Nail	Liquid Nail w/ Strap	Kevlar
Material Cost	\$24.78	\$99.12	\$16.03	\$90.37	\$396.82
Labor Cost	\$294.00	\$596.90	\$739.45	\$1,069.08	\$739.45
Total Cost	\$318.78	\$696.02	\$755.48	\$1,159.45	\$1,136.27

The labor was the major factor in the cost for each connection. This had a major impact on the connection we selected because it must be easy to install as well as strong to be chosen. In order to do this, we developed strength versus cost ratio table:

**Table 10: Strength to Cost Ratio**

Strength to Cost Ratio					
Test	Toenail	Toenail w/ Strap	Liquid Nail	Liquid Nail w/ Strap	Kevlar
Limit State 1	0.57	1.36	0.39	2.76	0.31
Limit State 2	0.67	0.71	0.39	1.74	0.31

The table displays the strength to cost ratio for both Limit State 1 and Limit State 2. Liquid Nails in combination with hurricane straps as the best ratio but again this is skewed due to the inconsistency in the tests for this connection. The second highest ratio belonged to toenails and hurricane strap connection. The other three connections were well below these two and therefore were eliminated from the discussion as possible rafter to top plate connection solutions.

### **4.3 Testing Conclusions and Connection Selection**

The objective for these tests was to gather data about each connection and decide which would be best for our house design. We wished to observe just how much hurricane straps strengthened rafter to top plate connections and whether an adhesive was a realistic replacement for toenails.

The addition of a hurricane strap significantly increases the strength of the connection. Hurricane straps are easy to install and inexpensive (about 35 cents each) making them an ideal addition to the rafter to top plate connection. The extra labor required for installation is far outweighed by the benefits the hurricane straps provide.

The replacement of toenails with Liquid Nails seems to be a realistic option and should be taken into consideration. Liquid Nails were able to withstand a greater load before failure but the gap allowed between the rafter and top plate (deformation) was greater with Liquid Nails. This higher deformation may make some builders stray from the use of Liquid Nails because the gap may allow other elements such as rain to enter and affect the building. With further testing, a wood adhesive may be a viable option

for a rafter to top plate connection but our tests, especially when in combination with a hurricane strap, were far too inconsistent to be chosen for our house.

Kevlar or another polymer connection seems feasible but work needs to be done with the material. Our Kevlar connections performed poorly and cannot be considered a replacement for current connections. With research and development, a polymer may be the connection of the future but the technology is just not available today.

After reviewing the test results as well as the costs, and comparing the data we decided to use toenails in combination with a hurricane strap in our house design. The decision was made primarily because the hurricane straps and toenail connections tested the most effectively with the smallest deviation. With hurricane straps in common practice today, the labor involved in this construction is quick and easy, keeping costs down. The overall cost for the installation of this connection was the second cheapest of all the connections. Another factor in choosing the toenails and straps over the other connections was that the deformation of the connection is the smallest compared to the load that it can withstand. If the rafter separates too far from the top plate, rain and wind could enter the house and inflict heavy damage as well as make the house structurally inadequate. Although this connection is currently used, it is not a common practice and should be implemented into the newest edition of the building code (IBC).

#### **4.4 Designing the Two Story Single Family House**

After thoroughly researching and identifying the areas of residential structures that are vulnerable to wind cause failures, we designed our single family, two story home with effective design features to combat these failures. One of these features includes a hip roof, opposed to a traditional gable roof. In addition, each member was designed specifically to handle extreme wind loads rather than for minimum code regulations. Every member was designed to meet both the Allowable Stress Design (ASD) and the Load and Resistance Factor Design (LRFD) specifications. To successfully achieve this, we utilized information contained within the *2012 International Residential Code for One and Two Family*

*Dwellings, American Society of Civil Engineers (ASCE) 7: Minimum Design Loads for Buildings and Other Structures, 2008 Special Design Provisions for Seismic and Wind Design, and National Design Specification for Wood Construction.* All calculations are available in Appendix A.

#### **4.4.1 Location and Wood Species**

We initially considered designing our home in the Midwest where it is not uncommon for a tornado to demolish an entire town or community. However, because the 2011 tornado outbreak extended into Massachusetts on June 1, we experienced firsthand, the devastation and destruction these twisters beget, we decided to select a location within the state. We felt that if we designed for a location within close proximity to WPI, our design and results could be implemented more quickly into society. Ultimately, we decided upon Dennis, Massachusetts, a small town located in Cape Cod. We felt this was an optimal location because, although it is not prone to tornadoes, it is prone to hurricanes and hurricane force winds. To determine the structural lumber used in the area we contacted the local Lowes Home Improvement store. We were informed that the structural lumber they sell for wood construction is Spruce-Pine-Fir (SPF) No. 2. After obtaining this information, we were able to begin our calculations.

#### **4.4.2 Dead Loads**

The first loads we calculated were the gravity loads. Gravity load design is the most elementary aspect of designing because it is perpetual loading condition, and has always been the traditional design concern. The first gravity loads we calculated were our dead loads. A dead load is a structures weight and all the materials that are permanently attached to it. Ultimately, we had to calculate three different dead loads, one for the roof, walls, and floors. To ensure the dead load calculations were precise and accurate, we conducted further research on roof, wall, and floor construction to determine the typical materials contained within these three structures. After determining which components would be included in our structures, we analyzed ASCE 7 Table C3-1, Minimum Design Dead Loads, and Appendix B in *Design of Wood Structures, Weights of Building Materials*, to determine the individual weights of the material that

would be used for construction. From these resources, we determined the roof, wall, and floor dead loads to be, 10 pounds per square foot (psf), 12.1 psf, and 10 psf respectively.

### **4.4.3 Live Load**

The second gravity loads we calculated were the live loads. Our design required us to determine two live loads, one for the roof and another for the floors. Live loads are associated with the use or occupancy of a particular structure. Where dead loads are permanently applied, live loads tend to fluctuate with time and typically account for loads produced by people or furniture. ASCE 7 and the IBC specify minimum roof and floor live loads that must be used in the design of a structure. Therefore, we examined Table 4-1, Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads, in ASCE 7 to determine the live loads. First, we determined the roof live load. According to the table, the design value for the live load of an ordinary flat, pitched or curved roof is 20 psf. However, live load reductions are permitted if the tributary area supported by structural members is greater than 200 ft<sup>2</sup> and/or the roof slope is steeper than 4 inches per foot. The tributary area supported by the structural members was not greater than 200 ft<sup>2</sup>, but the roof slope was 5 inches per foot. Thus, we were able to reduce the roof live load by using the equation  $L_0R_1R_2$ , where  $L_0$  is the original live load,  $R_1$  is a function of the supported tributary area, and  $R_2$  is a function of the roof slope. Because the tributary area was less than 200 ft<sup>2</sup>,  $R_1$  was equivalent to 1. To solve for  $R_2$  we multiplied the roof slope by 0.05 and subtracted that value from 1.2. We then multiplied all the values together and obtained a new design value of 19 psf. We then determined the floor live loads to be 40 psf. This value was taken directly from Table 4-1 and not reduced.

### **4.4.4 Snow Load**

The next gravitational load we designed for was the snow load. Snow loads primarily affect roof structures. The magnitude of a snow load can vary tremendously over a relatively small geographic region. According to *Design of Wood Structures*, in a certain mountainous region of Southern California the snow load is 100 psf, but less than 10 miles away, at the same elevation, the snow load is only 50 psf

(Breyer et. al, 2006). This further emphasizes the importance of recognizing the local condition and affirmed the rationale for selecting a hypothetical location for construction. Nevertheless, to calculate the snow load we had to initially calculate the flat roof snow load. This was done by employing the equation,  $S = 0.7C_eC_tIp_g$ , where  $C_e$ ,  $C_t$ ,  $I$  are the respective exposure, thermal, and importance factors for a particular structure, and  $p_g$  is the ground snow load for the area. By examining ASCE 7 Tables 7-2, 7-3, and 1-1, we determined the three flat roof factors were all equivalent to one. Next, we analyzed ASCE Figure 7-1 to obtain a ground snow load. According to the figure, the ground snow load for Cape Cod, Massachusetts is 30 psf. We multiplied the values together to attain a reduced snow load of 18.9 psf.

To acquire the true design snow load, we then multiplied the reduced snow load by  $C_s$ , the roof slope factor. This factor provides reduced snow loads for roof slopes, type of roof surfaces, and thermal conditions. After taking into account all necessary conditions, we deduced from ASCE Section 7.4 and Table 7.2A the slope factor was also equal to one. Therefore, the final design snow load was 18.9 psf, which was adjusted and rounded up to 19 psf.

#### **4.4.5 Wind Load**

The final load we had to calculate prior to commencing the design of structural members was the wind load. Wind loads are not gravitational loads and the ASCE 7 provisions for determining the magnitude of wind gusts are based on the results of extensive research regarding loading on structures and components of various sizes and configurations, in a variety of simulated exposure conditions. Since the fundamental focus of the project was wind loading and enhancing the design of residential structures to combat wind caused failure, it was absolutely vital to the success of the project that the wind load be calculated correctly and accurately. Wind loads have both a vertical and horizontal component. To ensure we were using accurate loading values, we computed the vertical component which acts on the roof and the horizontal component acts on the shearwalls. Both values were attained from the formula,  $\lambda K_{zt}Ip_{s30}$ , which is the basic formula for calculating wind pressure. In this formula  $\lambda$  and  $K_{zt}$  are the respective



height and topographic factors, while  $I$ , still denotes the previously determined importance factor, and  $p_{s30}$  is the simplified design wind pressure.

To compute an accurate  $p_{s30}$  we examined ASCE 7 Figure 6-1 to acquire the nominal design 3 second wind gust speed for Cape Cod. From Figure 6-1 we learned the nominal 3 second wind gust speed was 110 mph. Then, we located 110 mph in Figure 6-2 to acquire the maximum horizontal and vertical pressures. The magnitudes of the corresponding pressures are based on roof angles. The figure only provided values for five degree increments from  $0^\circ$  to  $25^\circ$ . Since the roof angle is  $22.62^\circ$ , we used linear interpolation to determine the maximum horizontal and vertical pressures, which were 25.4 psf and 16.6 psf respectively.

Also included in Figure 6-2 are  $\lambda$  values.  $\lambda$  values are a function of the structure's mean roof height and exposure to wind. There are three exposure categories, B, C, and D. A description of each category is available in ASCE 7 Section 6.5.6.3. These descriptions are available to assist the designer in selecting a proper  $\lambda$  value. From these explanations, we identified our location was most similar to the description afforded in category C. The figure provides values for each exposure category according to the mean roof height. However, values are only available for five foot increments beginning at 15 feet. Because the mean roof height was 28 feet, we utilized linear interpolation again to ensure we were using the appropriate  $\lambda$  value. From our calculations we obtained a  $\lambda$  value of 1.38.

The next step was to determine the value of the topographic factor,  $K_{zt}$ . Topographic effects are discussed in detail in ASCE 7 Section 6.5.7. The section 6.5.7.2 is devoted entirely to computing the topographic factor. It explains that if the site conditions and location of structures do not meet all the conditions specified in Section 6.5.7.1 then  $K_{zt}$  is equal to one. Therefore, we perused the preceding section and discovered that our site conditions did not meet all the specified condition, making out  $K_{zt}$  value one. Finally, we multiplied all the values together to obtain the respective horizontal and vertical wind design values of 35 psf and 23 psf.

#### **4.4.6 Load Combinations**

The basic load combinations for ASD and LRFD are presented in ASCE Section 2.4.1 and 2.3.2 and the combinations we utilized for the calculations are available in Appendix A. In these load combinations, each load is preceded by a coefficient that it was multiplied by and then added to the other loads. The maximum calculated load was then selected as the design value. Overall, the respective maximum calculated roof, floor, and wall loads were 55.8 psf, 52.1 psf, and 771 pounds per foot (lb/ft) for ASD and 67.8 psf, 78.5 psf, and 1,088 lb/ft for LRFD. A lateral wind load acting upon the walls was also calculated. The maximum values for these loads were 350 lb/ft for ASD and 560 lb/ft for LRFD.

#### **4.4.7 Roof Design**

The first structure we designed was the roof. Through our methodological approaches, we identified the roof as the most critical structure of the house, so it was imperative that we design a stable and strong roof. basic design considerations for a roof include roof layout, truss design, and sheathing thickness and nailing.

The roof layout was the first aspect of the design that we had to determine. There are several different types of roof layouts that can be used for a variety of reasons. Some of these roof layouts include a gable roof, gambrel roof, flat roof, and hip roof. A gable roof is the traditional style of roofs that is constructed with two sloping sides. A gambrel roof is similar to a gable roof but slopes down twice on each side, with the lower slope being the steeper of the two. A flat roof is a roof constructed exactly as it is named. Finally, the hip roof slopes down to the walls on all four sides. After conducting thorough research for each roof layout, we decided to select a hip roof layout. Our research indicated that hip roofs stand up wind storms better than any other roof layout. The shape of gable and gambrel roofs resemble that of an airfoil and wind moving across it will try to lift it similar to the wing of an airplane.

After deciding upon the layout we then had to design the structural members. Due to the unique structure of the hip roof we had to construct the roof with a series of trusses. In our preliminary sketch we decided to space the trusses at 16" on center (o.c.). To determine if this was acceptable we began the

initial roof design by completing a truss analysis. This was done to ensure the truss design was capable of supporting the design loads. Ultimately, we developed trusses to support the common rafters and hip rafters. Upon completing the truss design, we then had to determine the size of the structural members used to construct the trusses. Using both ASD and LRFD design methods, we determined that 2 x 12 SPF No. 2 members were acceptable members to handle the design loads.

Finally, the last elements we had to design were the sheathing thickness and nailing. These elements are members of the roof diaphragm and therefore, the process required to determine these aspects are covered in further detail in the next section, diaphragm design. Nevertheless, both design methods concurred that 19/32" APA rated SPF 3-ply plywood panels or better, nailed perpendicular to the rafters with 10d common nails spaced at 6" o.c. at supported edges and 12" o.c. field without blocking was adequate to support the design loads.

#### **4.4.8 Diaphragm Design**

The next element we designed was the floor diaphragm. A diaphragm is made up of the structural members that combine to form the horizontal plane(s) of the building. The basic design considerations for a diaphragm include joist, chord, and strut design, sheathing thickness, and nailing.

The first of these elements that we designed were the structural beams. Joists, chords, and struts are the structural members that make up part a diaphragm's frame. We began the initial diaphragm design by identifying the required size of the floor joists. When we sketched the preliminary plan view of the structure, we decided to have the floor joists lay perpendicular to the front and back walls at 16" o.c. These members are supported by the interior load bearing walls from the floor below and therefore, do not require any girders. Furthermore, we selected to space these members at 16" o.c. because this is a typical spacing used for these members in residential home construction, and we did not feel it was necessary to decrease the distance to 12" o.c unless our calculations revealed otherwise. After determining these conditions we were able to calculate the necessary size of the floor joists using both the ASD and LRFD methods. The two methods required us to perform different calculations, but were similar

in regards to checking strength and serviceability of each member. In the end, both methods agreed that 2 x 12 SPF No. 2 members were more than adequate to handle the design loads.

The next elements we designed were the diaphragm chords and struts. Diaphragm chords and struts, also known as the top plate, are the perimeter members of a diaphragm. These members are actually the same but are given different names based upon where the load is being applied. The member is called a chord if the lateral load is being applied parallel to the member and a strut if the lateral load is applied perpendicular to the member. The maximum chord and strut forces are compared to one another, and the design is based on the critical force. We calculated the chord and strut forces in both the transverse and longitudinal directions and upon comparison, discovered that the chord forces were critical. We then used these forces to calculate the required number and size of the beam(s). Ultimately, both ASD and LRFD agreed that two 2 x 6 Spruce-Pine-Fir No. 2 members were acceptable to handle the load we had designed for.

After designing all the structural members, we focused our attention on the sheathing thickness and required nailing. To determine the thickness and nailing, we needed to determine the minimum required span rating. The span rating of sheathing is a set of two numbers. The two numbers indicate the maximum recommended span in inches when used as roof or floor sheathing. The left hand number denotes roof sheathing and the right hand number denotes floor sheathing. We chose to calculate the strength of 3 ply span rated plywood because it is constructed with the fewest plies, and therefore, represented the minimum acceptable sheathing that can be utilized for construction. We then calculated the strength of 3 ply span rated plywood, nailed with the strength axis perpendicular and parallel to the joists for both the applied and total uniform loads. To determine the require rating we compared the maximum adjusted load to the actual load. We obtained the allowable uniform applied and total loads from Table 1 in APA's *Load Span Tables for APA Structural-Use Panels*, and the necessary adjustment factors to multiply them by from Table 3 and 4. Both methods revealed 3-ply plywood was only acceptable to handle the design loads when nailed with its strength axis perpendicular to the joists.

Additionally, when it is nailed perpendicular to the joist, the minimum span rating, 32/16, was acceptable for the design loads.

The next step was to determine the thickness and nailing of the sheathing. We located the span rating on Table 5 from APA's *Panel Design Specifications* to identify the nominal thicknesses available for 32/16 span rating. These thicknesses were then located for sheathing and single floor sheathing grade in the *Special Design Provisions for Wind and Seismic* Table 4.2C, Nominal Unit Shear Capacities for Wood Frame Diaphragms, to determine the maximum unit shear and nailing requirements for wind loading case 1. The loading cases are described in detail in Tables 4.2A and 4.2B. Nevertheless, the maximum unit shears presented in the table are for Douglas Fir and Southern Pine wood species only, and subsequently, needed to be multiplied by the specific gravity of SPF to render the proper design value. The adjusted maximum unit shears for SPF, were further modified by both design methods, which are explained in SDPWS Section 4.2.3, and then compared to each method's calculated shear. In the end, both methods concurred that 19/32" APA rated 32/16 3-ply plywood panels, nailed with 10d common nails, perpendicular to joists, at 6" o.c. at supported edges and 12" o.c field were adequate to support the design loads. Additionally, our calculations also revealed that blocking was not required. This is important because it will save time and money during construction.

#### **4.4.9 Shearwall Design**

The third structure we designed were the shearwalls. A shearwall is made up of the structural members that combine to form the vertical plane(s) of the building. The basic design considerations for a diaphragm include stud and chord design, sheathing thickness, and nailing.

The first structural members we designed for were the studs. Similar to the joists, we decided to space the studs at 16" o.c. because it was a typical spacing for residential construction. After deciding upon this, we began to calculate the required stud size. To achieve this, we had to establish the studs could withstand the greatest axial and lateral loads. First, we computed the ultimate concentrated load on one stud from the gravity loads. When we discovered that a 2 x 6 SPF No. 2 member was acceptable for

the axial loading scenario for both methods, we moved onto the lateral loading scenario. Our calculations also revealed that the 2 x 6 member was adequate for the maximum lateral loads. To ensure that the 2 x 6 studs was adequate under simultaneous loading conditions, we utilized the interaction formula, which incorporated the actual and adjusted allowable compression stress from the axial loading and the actual and adjusted bending stresses from the lateral loading. The formula is set up to combine the values so that an acceptable member will produce a value less than or equal to one. After executing the formula for a 2 x 6 member, both design methods produced acceptable values. Therefore, we determined that 2 x 6 SPF No. 2 members were acceptable for all studs.

We then designed the shearwall chords. Shearwall chords are the vertical members at the end of each segmented shearwall. To calculate the size of the shearwall chords we calculated the compression and tension forces acting on it. The compression was calculated to be substantially larger due to the axial loading and subsequently, was the governing force. The adjusted compression values perpendicular and parallel to the grain were compared next. In the ASD calculations, we discovered that compression perpendicular to the grain was the ultimate governing force, but the LRFD demonstrated the opposite. However, upon further calculation, both methods demonstrated that a 2 x 6 SPF No. 2 member was not acceptable, rather a 4 x 6 SPF No. 2 post would be required to support the design load for the shearwall chords.

Finally, we had to determine the sheathing thickness, nailing, and blocking requirements. To determine the sheathing thickness the actual unit shear of each shearwall was compared to an allowable unit shear for plywood. We obtained the allowable unit shear for a particular thickness and nailing of plywood from Table 4.3A in the *Special Design Provisions for Wind and Seismic*. Analogous to the diaphragm sheathing, the values in Table 4.3A are for Douglas Fir and Southern Pine species only. Thus, we had to multiply the values by the specific gravity of SPF and other mathematical adjustment specific to each design method addressed in SDPWS Section 4.3.3, to acquire the true allowable unit shear for an SPF sheathing panel. Our initial calculations demonstrated that 7/16" structural panels-sheathing with 8d

common nails at 6" o.c. at supported edges and 12" o.c. field would be acceptable for all the shearwall sheathing. However, this did not account for the shearwalls being unblocked.

Because the shearwalls were to be designed without blocking, we learned that we needed to utilize thicker and stronger APA span rated sheathing, and closer nailing to satisfy the deflection requirement. According to the SDPWS, the maximum shearwall deflection cannot exceed  $0.02 * height (ft) * 12$ . The maximum allowable shearwall deflection is equal to the sum of the bending and shear deflection and nail and anchorage slips multiplied by the deflection amplification factor. To calculate the deflection we calculated each of the aforementioned values. First we had to divide the maximum shear force by the unblocked shearwall adjustment factor, which we obtained from SDPWS Table 4.3.3.2. Ultimately, this required us to decrease the intermediate framing, field, nailing to 6" o.c. to achieve an acceptable deflection value. Next we calculated the four values that were added together and multiplied by four, the deflection amplification factor. These calculations can be viewed in Appendix A.  $G_t$  presented in the shear deflection equation is the panel rigidity which were obtained from Table C4.2.2A of SDPWS 2008. The  $e_n$  contained in the nail slip equation is the nail deformation in inches and is based on nail size and load per fastener. These values were obtained from SDPWS 2008 Table C4.2.2D, and were subjected to a 20% increase if any other grade of plywood other than Structural I was being utilized. Finally,  $d_a$  in the anchorage slip equation is the total slip between the chord and anchorage bracket and is assumed to be  $\frac{1}{4}$ ". Ultimately, both design methods determined that 15/32" APA rated 32/16 3-ply wood structural panels—structural with 10d common nail at 6" o.c at supported edges and field were required to withstand the lateral force without any blocking.

#### **4.4.10 Foundation Design**

The foundation was the subsequent design after the shearwall. The foundation transfers loads carried to it by the shearwall into the soil beneath the home. The design for the foundation consisted of two major components. The first component was the footing. The specifications for footing design was a continuous 12" wide footing, supporting light frame design and 8' deep basement retaining walls. The

weight of the footing was determined by multiplying the density of concrete by the cross sectional area of the footing, resulting in 1200 lb/ft. To determine the bearing pressure, the weight of the foundation was then used in equation  $q = \frac{P+W_f}{A} - u_D$ , where P was equal to the vertical load,  $W_f$  was the foundation weight, A was the base area, and  $u_D$  was equal to the pore water pressure at the bottom of the foundation. Once the footing was established, the normal force acting between the soil and the basement retaining wall was calculated. The coefficient of lateral earth pressure at rest,  $K_0$  was calculated using equation  $K_0 = (1 - \sin \phi')OCR^{\sin \phi'}$ . In this equation  $\phi'$  and OCR were equal to the respective friction angle and over consolidation ratio of the soil. Then  $K_0$ , with unit weight of soil and the height of the wall were combined in equation  $P_0/b = \frac{\gamma H^2 K_0}{2}$ , to calculate the normal force acting between soil and the retaining wall. In this equation  $\gamma$  and H were equal to the respective soil unit weight and wall height.

#### **4.4.11 Connection Design**

The last element of the structure we designed were the connections. The connections are the most essential element of the design because they keep the structure standing. To determine the necessary number of fasteners per connection, we had to compute the basic strength of a single dowel-type fastener subjected to a lateral load. This is known as the reference design value and was taken as the smallest load capacity obtained from evaluating all six single shear yield limit equations for each connection. Next we multiplied the reference design value by the appropriate adjustment factors to obtain the adjusted design value. The load per connection was then divided by the adjusted design value to determine the required number of fasteners. In the end, both design methods produced very similar results and a fastening schedule outlining the number of fasteners per connection is available in Appendix A.



## 5.0 Conclusion and Recommendations

To raise awareness regarding the typical deficiencies residential structures contain in the face of high speed wind storms, we completed this extensive study. The last couple decades have seen an escalation in the intensity and number of high speed wind storms. These storms are devastating the residential infrastructure of United States, specifically in areas like the Midwest, Southeast, and Atlantic coast. In 2005, Hurricane Katrina decimated New Orleans with torrential downpours and winds in excess of 140 mph. More recently, in 2011, Hurricane Irene whipped winds in excess of 120 mph while making its way up the Atlantic coast. Over the storm's week long life, it damaged and destroyed homes as far north as Vermont. Additionally, during the years of 2010 and 2011, the Midwest and Southeast confirmed a record number of tornadoes touching down. Several of these twisters, including the 2011 tornado in Joplin, Missouri, reached the maximum grade, F5, on the Fujita Scale. Although residential structures exposed to storms with this magnitude may be destined for destruction, many structural failures occur as a result of weaker wind loads and can be easily avoided. The most recent edition of the IRC sets minimum design code regulations that are not capable of handling the strength and ferocity of many of the storms we are seeing each year. Therefore, to develop a design the IRC can use as a template to limit wind caused failures in the future, we conducted extensive testing on roof connections and designed our own structure capable of withstanding 110 mph winds.

During our testing phase we examined five rafter to top plate connections. We identified this connection as the most critical element of any other roof structure. These five connections included:

- Toenailing with 3 16d common nails
- Toenailing with 3 16d common nails combined with Hurricane Strap
- Liquid Nails
- Liquid Nails and Hurricane Strap
- Kevlar Strap

Each connection was constructed and tested five times to produce ample data for analysis. Upon completing the tests for all connections we compared their relative strengths and consistency in regards to performance. Finally, the individual costs and labor costs to install each connection was included in our analysis to identify the most effective and affordable solution. Ultimately, we determined from our data the combination of the toenail and hurricane strap was the best solution for our structure.

While testing for our roof was ongoing, we were also concentrated on designing the rest of our two story, single family home. We were able to utilize information from various sources including *Design of Wood Structures*, *Special Design Provision for Wind and Seismic (SDPWS)*, and *ASCE 7-05: Minimum Design Loads for Buildings and Other Structures* to design our home to meet both ASD and LRF design specifications.

Overall, the initiative of our project has been successful for developing a design template for homes faced with enduring high speed wind storms. After completing our project, we believe that there should be adjustments made to the subsequent edition of the IRC. Our primary recommendation is to standardize code to include hurricane straps as minimum code. Our tests exhibited hurricane straps increased the strength of the most critical element of the most critical structure, the rafter to top plate connection in the roof, approximately 500%. Moreover, hurricane straps are inexpensive and easy to install. The overall cost for labor and materials is miniscule compared to the cost of the damage a house will incur if the roof is blown off.

Our second recommendation involves the design of homes. We believe that all newly constructed one or two story homes should be designed capable of withstanding at least 110 mph wind gusts. The strength of high speed wind storms is increasing. The current average wind speed for a tornado in the United States is 112 mph. Tornadoes are typically nature's most violent wind storms and thus, it is our belief that all newly constructed residential structure have the capability to handle the average wind speed. We believe it is not unreasonable to expect houses to only be destroyed by the occasional storm that is excessively strong and ferocious. Although some areas may not be prone to tornado or hurricane force winds, we are of the firm belief it is better to be prepared for the unexpected.

Ultimately, we believe the work we have done to complete this study is only the beginning of a movement to inspire change to residential code and construction. From our project, the IRC can develop better minimum code requirements to diminish the probability of failure caused by wind and secure a better future for homeowner around the nation.

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## **Appendix A: Allowable Stress Design (ASD) Calculations**

# ASD LOADS

# Loads

## Gravity Loads

### **Roof Dead Load: D**

Asphalt Shingles =	2.50 psf
19/32 -in. Wood Sheathing (3.7 psf x 15/32 in.) =	2.20 psf
Waterproofing Membrane (Single Ply Sheet) =	0.60 psf
4 in. Polystyrene Foam Insulation (0.2 psf x 4 in.) =	0.80 psf
Framing (2 x 12 @ 16 in o.c.) =	2.90 psf
Flashing (Copper/Tin) =	1.00 psf
	<hr/>
	10.00 psf

**Roof Dead Load: D = 10.00 psf**

### **Wall Dead Load: D**

2 x 6 @ 16-in., 5/8-in. gypsum, insulated, 15/32-in. siding =	12.00 psf
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**Wall Dead Load: D = 12.00 psf**

### **Floor Dead Load: D**

Framing (2 x 12 @ 16 in o.c.) =	2.90 psf
Subflooring 3/4 in. =	3.00 psf
Hardwood Flooring =	4.00 psf
Ceiling (Gypsum Board 1/2 in.) =	2.20 psf
	<hr/>
	12.10 psf

**Floor Dead Load: D = 12.10 psf**

### **Roof Live Load: L<sub>r</sub>**

$L_0 R_1 R_2 =$	19.00 psf
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**Roof Live Load: L<sub>r</sub> = 19.00 psf**

### **Floor Live Load: L**

ASCE- 7 Table 4-1	40.00 psf
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**Floor Live Load: L = 40.00 psf**

### **Roof Snow Load: S**

$C_s(0.7)(C_e)(C_t)(I)(p_g) =$	18.90 psf
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**Roof Snow Load: S ≈ 19.00 psf**

**Roof Wind Load: W**

$(\lambda)(K_{zt})(I)(p_{s30}) =$

**23.00 psf**

**Roof Wind Load: W =**

**23.00 psf**

**Lateral Loads**

**Wind Load: W**

$(\lambda)(K_{zt})(I)(p_{s30}) =$

**35.00 psf**

**Wind Load: W =**

**35.00 psf**

See Appendix C for Necessary Tables and Figures



# ASD Load Combinations

### Applicable Roof Load Combinations (ASD)

D	10.0 psf	Non Critical
D+L <sub>r</sub>	29.0 psf	Non Critical
D+S	29.0 psf	Non Critical
D + 0.75L <sub>r</sub> + 0.75S	38.5 psf	Non Critical
D+W	33.0 psf	Non Critical
D + 0.75W + 0.75L <sub>r</sub> + 0.75S	55.8 psf	CRITICAL

### Applicable Floor Load Combinations (ASD)

D	12.1 psf	Non Critical
D+L	52.1 psf	CRITICAL

### Applicable Wall Load Combinations (ASD)

#### Gravity Load Combination

D	417 lb/ft	Non Critical
D+L	737 lb/ft	Non Critical
D+S	569 lb/ft	Non Critical
D + 0.75L + 0.75S	771 lb/ft	CRITICAL

#### Lateral Load Combinations

W	350 lb/ft	CRITICAL
0.75W	263 lb/ft	Non Critical

# ASD Roof Truss Members

### **Wood Properties**

Spruce-Pine-Fir	No.2
Bending ( $F_b$ )	875 psi
Tension Parallel to Grain ( $F_t$ )	450 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	1,150 psi
Modulus of Elasticity (E.)	1,400,000 psi
Modulus of Elasticity ( $E_{min}$ )	510,000 psi

### **Factors**

$C_D$ (Load Duration)	1.60
$C_F$ (Size)	
Bending	1.10
Tension	1.10
Compression	1.00
$C_i$ (Incising)	1.00
$C_L$ (Stability)	1.00
$C_M$ (Wet Service)	1.00
$C_r$ (Repetitive Member)	1.15
$C_t$ (Thermal)	1.00

### **Wood Dimensions and Spacing**

Common Rafter Length: L 18.34 ft

Rafter Spacing: 16.00 in

Section Modulus: S

2 x 6 =	7.56 in <sup>3</sup>
2 x 8 =	14.06 in <sup>3</sup>
2 x 10 =	22.56 in <sup>3</sup>
2 x 12 =	33.06 in <sup>3</sup>

Area: A

2 x 6 =	8.25 in <sup>2</sup>
2 x 8 =	11.25 in <sup>2</sup>
2 x 10 =	14.25 in <sup>2</sup>
2 x 12 =	17.25 in <sup>2</sup>

Moment of Inertia: I

2 x 6 =	20.80 in <sup>3</sup>
2 x 8 =	52.73 in <sup>3</sup>
2 x 10 =	107.17 in <sup>3</sup>
2 x 12 =	190.11 in <sup>3</sup>

### **Loads**

Total Load: $w_{TL}$ (D + 0.75W + 0.75L + 0.75S) x Rafter Spacing	74.3 lb/ft		
Shear: V ( $w_{TL} \times L$ )/2 =	682 lb		
Moment: M ( $w_{TL} \times L^2$ )/8 =	3,125 ft-lb 37.5 in-k		
<b><u>Bending</u></b>			
Adjusted Bending Design Value: $F'_b$ $F_b (C_D)(C_M)(C_t)(C_L)(C_F)(C_r)(C_i) =$	1,771 psi		
Required Section Modulus: Req'd S $M/F'_b =$	21.2 in <sup>3</sup>	=>	Try 2x10
Actual Bending Stress Design Value: $f_b$ M/S =	1,662 psi		
$F'_b > f_b$	1,771	>	1,662 TRUE
<b><u>Shear</u></b>			
Adjusted Shear Design Value Parallel to Grain: $F'_v$ $F_v (C_D)(C_M)(C_t)(C_i) =$	216 psi		
Actual Shear Stress Parallel to Grain: $f_v$ 1.5V/A =	71.75 psi		
$F'_v > f_v$	216.0	>	71.75 TRUE
<b><u>Deflection</u></b>			
Adjusted Modulus of Elasticity: E' $E(C_M)(C_t)(C_i) =$	1,400,000 psi		
Actual Deflection Under Snow Load: $\Delta_s$ $5w_s L^4 / 384E'I =$	0.43 in		
Allowable Deflection Under Snow Load: Allow. $\Delta_s$ L/240 =	0.92 in		
Allow. $\Delta_s > \Delta_s$	0.92	>	0.43 TRUE

Actual Deflection Under Total Load:  $\Delta_{TL}$

$$\Delta_S(w_{TL}/w_S) = 1.26 \text{ in}$$

Allowable Deflection: Allow.  $\Delta_{TL}$

$$L/180 = 1.22 \text{ in}$$

$$\text{Allow. } \Delta_{TL} > \Delta_{TL} \quad 1.22 > 1.26 \quad \text{FALSE}$$

### **Bending**

Adjusted Bending Design Value:  $F'_b$

$$F_b (C_D)(C_M)(C_t)(C_L)(C_F)(C_r)(C_i) = 1,771 \text{ psi}$$

Required Section Modulus: Req'd S

$$M/F'_b = 21.2 \text{ in}^3 \quad \Rightarrow \quad \text{Try 2x12}$$

Actual Bending Stress Design Value:  $f_b$

$$M/S = 1,134 \text{ psi}$$

$$F'_b > f_b \quad 1,771 > 1,134 \quad \text{TRUE}$$

### **Shear**

Adjusted Shear Design Value Parallel to Grain:  $F'_v$

$$F_v (C_D)(C_M)(C_t)(C_i) = 216 \text{ psi}$$

Actual Shear Stress Parallel to Grain:  $f_v$

$$1.5V/A = 59.3 \text{ psi}$$

$$F'_v > f_v \quad 216 > 59.3 \quad \text{TRUE}$$

### **Deflection**

Adjusted Modulus of Elasticity:  $E'$

$$E(C_M)(C_t)(C_i) = 1,400,000 \text{ psi}$$

Actual Deflection Under Snow Load:  $\Delta_S$

$$5w_s L^4 / 384E'I = 0.24 \text{ in}$$

Allowable Deflection Under Snow Load: Allow.  $\Delta_S$

$$L/240 = 0.92 \text{ in}$$

$$\text{Allow. } \Delta_S > \Delta_S \quad 0.92 > 0.24 \quad \text{TRUE}$$

Actual Deflection Under Total Load:  $\Delta_{TL}$

$$\Delta_S(w_{TL}/w_S) = 0.71 \text{ in}$$

Allowable Deflection: Allow.  $\Delta_{TL}$

$L/180 =$

1.22 in

Allow.  $\Delta_{TL} > \Delta_{TL}$

1.22 > 0.71

TRUE

**Use 2 x 12 SPF No. 2 or Better for All Roof Truss Members**  
**MC  $\leq$  19%**

# ASD Roof Sheathing



**Loads**

Total Load:  $w_{TL}$

$$D + 0.75W + 0.75L + 0.75S = 55.8 \text{ psf}$$

Applied Load:  $w_{AL}$

$$0.75W + 0.75L + 0.75S = 45.8 \text{ psf}$$

**Factors**

$C_D$  (Load Duration) 1.60

$C_G$  (Grade and Construction)

Stiffness

Perpendicular to Joists 1.10

Parallel to Joists 1.00

Bending

Perpendicular to Joists 1.00

Parallel to Joists 1.00

Shear

Perpendicular to Joists 1.00

Parallel to Joists 2.80

$C_{SA}$  (Span Adjustment)

3-Span to 1-Span

Stiffness 0.53

Bending 0.80

Shear 1.20

$G_s$  (Specific Gravity) 0.92

**Strength Axis Perpendicular to Joists**

Span Rating: 24/0

Plywood Type: 3-Ply

Applied Load:  $w_{AL}$

$$0.75S + 0.75L + 0.75W = 45.8 \text{ psf}$$

Allowable Uniform Applied Load on APA Rated Plywood

Sheathing Across 16" o.c. Supports:  $W_{AL}$  147 psf

Adjusted Allowable Uniform Applied Load for 3-Ply

Plywood Sheathing:  $W'_{AL}$

$$W_{AL}(C_G)(C_{SA}) = 86 \text{ psf}$$

$$W'_{AL} > w_{AL} \quad 86 > 45.8 \quad \text{TRUE}$$

Total Load: $w_{TL}$				
$D + 0.75W + 0.75L + 0.75S =$		55.8	psf	
Allowable Uniform Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{TL}$		196	psf	
Adjusted Allowable Uniform Load for 3-Ply Plywood Sheathing: $W'_{TL}$				
$W_{TL}(C_G)(C_{SA}) =$		114	psf	
$W'_{TL} > w_{TL}$	114	>	55.8	TRUE
Actual Bending Stress: $f_b$				
$D + 0.75W + 0.75L + 0.75S =$		55.8	psf	
Allowable Bending Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_b$		117	psf	
Adjusted Allowable Bending Stress for 3-Ply Plywood Sheathing: $F'_b$				
$F_b(C_G)(C_D)(C_{SA}) =$		150	psf	
$F'_b > f_b$	150	>	55.8	TRUE
Actual Shear Stress: $f_v$				
$D + 0.75W + 0.75L + 0.75S =$		55.8	psf	
Allowable Shear Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_v$		228	psf	
Adjusted Allowable Shear Stress for 3-Ply Plywood Sheathing: $F'_v$				
$F_v(C_G)(C_D)(C_{SA}) =$		438	psf	
$F'_v > f_v$	438	>	55.8	TRUE

**APA Rated 24/0 3-Ply Plywood OK for Roof Sheathing Laid with Strength Axis Perpendicular to Joists**

**Strength Axis Parallel to Joists**

Span Rating:	24/0
Plywood Type:	3-Ply

Applied Load: $w_{AL}$ $0.75S + 0.75L + 0.75W =$	45.8 psf		
Allowable Uniform Applied Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{AL}$	9.00 psf		
Adjusted Allowable Uniform Applied Load for 3-Ply Plywood Sheathing: $W'_{AL}$ $W_{AL}(C_G)(C_{SA}) =$	4.77 psf		
$W'_{AL} > w_{AL}$	4.77	>	45.8      FAIL
Total Load: $w_{TL}$ $D + 0.75W + 0.75L + 0.75S =$	55.8 psf		
Allowable Uniform Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{TL}$	12.0 psf		
Adjusted Allowable Uniform Load for 3-Ply Plywood Sheathing: $W'_{TL}$ $W_{TL}(C_G)(C_{SA}) =$	6.36 psf		
$W'_{TL} > w_{TL}$	6.36	>	55.8      FAIL
Actual Bending Stress: $f_b$ $D + 0.75W + 0.75L + 0.75S =$	55.8 psf		
Allowable Bending Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_b$	25.0 psf		
Adjusted Allowable Bending Stress for 3-Ply Plywood Sheathing: $F'_b$ $F_b(C_G)(C_D)(C_{SA}) =$	32.0 psf		
$F'_b > f_b$	32.0	>	55.8      FAIL
Actual Shear Stress: $f_v$ $D + 0.75W + 0.75L + 0.75S =$	55.8 psf		
Allowable Shear Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_v$	145 psf		

Adjusted Allowable Shear Stress for 3-Ply Plywood

Sheathing:  $F'_v$

$$F_v (C_G)(C_D)(C_{SA}) = 780 \text{ psf}$$

$$F'_v > f_v \quad 780 > 55.8 \quad \text{TRUE}$$

Span Rating: 32 /16

Plywood Type: 3-Ply

Applied Load:  $w_{AL}$

$$0.75S + 0.75L + 0.75W = 45.8 \text{ psf}$$

Allowable Uniform Applied Load on APA Rated Plywood

Sheathing Across 16" o.c. Supports:  $W_{AL}$  20.0 psf

Adjusted Allowable Uniform Applied Load for 3-Ply

Plywood Sheathing:  $W'_{AL}$

$$W_{AL}(C_G)(C_{SA}) = 10.6 \text{ psf}$$

$$W'_{AL} > w_{AL} \quad 10.6 > 45.8 \quad \text{FAIL}$$

Total Load:  $w_{TL}$

$$D + 0.75W + 0.75L + 0.75S = 55.8 \text{ psf}$$

Allowable Uniform Load on APA Rated Plywood

Sheathing Across 16" o.c. Supports:  $W_{TL}$  27.0 psf

Adjusted Allowable Uniform Load for 3-Ply Plywood

Sheathing:  $W'_{TL}$

$$W_{TL}(C_G)(C_{SA}) = 14.3 \text{ psf}$$

$$W'_{TL} > w_{TL} \quad 14.3 > 55.8 \quad \text{FAIL}$$

Actual Bending Stress:  $f_b$

$$D + 0.75W + 0.75L + 0.75S = 55.8 \text{ psf}$$

Allowable Bending Stress for APA Rated Plywood Sheathing

Across 16" o.c. Supports:  $F_b$  43.0 psf

Adjusted Allowable Bending Stress for 3-Ply Plywood

Sheathing:  $F'_b$

$$F_b (C_G)(C_D)(C_{SA}) = 55.0 \text{ psf}$$

$F'_b > f_b$	55.0 > 55.8	FALSE
Actual Shear Stress: $f_v$ $D + 0.75W + 0.75L + 0.75S =$	55.8 psf	
Allowable Shear Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_v$	179 psf	
Adjusted Allowable Shear Stress for 3-Ply Plywood Sheathing: $F'_v$ $F_v(C_G)(C_D)(C_{SA}) =$	962 psf	
$F'_v > f_v$	962 > 55.8	TRUE
<hr/>		
Span Rating:	40 /20	
Plywood Type:	3-Ply	
Applied Load: $w_{AL}$ $0.75S + 0.75L + 0.75W =$	45.8 psf	
Allowable Uniform Applied Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{AL}$	44.0 psf	
Adjusted Allowable Uniform Applied Load for 3-Ply Plywood Sheathing: $W'_{AL}$ $W_{AL}(C_G)(C_{SA}) =$	23.3 psf	
$W'_{AL} > w_{AL}$	23.3 > 45.8	FAIL
Total Load: $w_{TL}$ $D + 0.75W + 0.75L + 0.75S =$	55.8 psf	
Allowable Uniform Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{TL}$	59.0 psf	
Adjusted Allowable Uniform Load for 3-Ply Plywood Sheathing: $W'_{TL}$ $W_{TL}(C_G)(C_{SA}) =$	31.3 psf	
$W'_{TL} > w_{TL}$	31.3 > 55.8	FAIL

Actual Bending Stress: $f_b$			
$D + 0.75W + 0.75L + 0.75S =$	55.8	psf	
Allowable Bending Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_b$	70.0	psf	
Adjusted Allowable Bending Stress for 3-Ply Plywood Sheathing: $F'_b$			
$F_b (C_G)(C_D)(C_{SA}) =$	89.6	psf	
$F'_b > f_b$	89.6	> 55.8	TRUE
Actual Shear Stress: $f_v$			
$D + 0.75W + 0.75L + 0.75S =$	55.8	psf	
Allowable Shear Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_v$	179	psf	
Adjusted Allowable Shear Stress for 3-Ply Plywood Sheathing: $F'_v$			
$F_v (C_G)(C_D)(C_{SA}) =$	962	psf	
$F'_v > f_v$	962	> 55.8	TRUE

**No APA Rated 3-Ply Plywood OK for Roof Sheathing Laid with Strength Axis Parallel to Joists**

**Thickness and Nailing**

3/8" Thick APA Rated 24/0 3-Ply Plywood Nailed Perpendicular to Joists with 6d Common Nails

Load Case:	Case 1		
Maximum Nominal Unit Shear for Wind Loading on Douglas Fir Plywood: $V_w$	460	lb/ft	
Adjusted Nominal Unit Shear for Wind Loading on SPF Plywood: $V'_w$			
$G_s V_w / 2 =$	212	lb/ft	
Actual Shear: $v$			
$0.75W$	263	lb/ft	
$V'_w > v$	212	> 263	FAIL

19/32" Thick APA Rated 32/16 3-Ply Plywood Nailed Perpendicular to Joists with 10d Common Nails

Load Case:	Case 1		
Maximum Nominal Unit Shear for Wind Loading on Douglas Fir Plywood: $V_w$	800 lb/ft		
Adjusted Nominal Unit Shear for Wind Loading on SPF Plywood: $V'_w$			
$G_s V_w / 2 =$	368 lb/ft		
Actual Shear: $v$			
$0.75W$	263 lb/ft		
$V'_w > v$	368 > 263		TRUE

**Use 19/32" APA Rated 32/16 SPF 3-Ply Plywood Panels or Better as Subroof Nailed Perpendicular to Supports with 10d Common Nails at 6" o.c. Supported Edges 12" o.c. Field No Blocking Required**

# ASD Floor Joists



### **Wood Properties**

Spruce-Pine-Fir	No.2
Bending ( $F_b$ )	875 psi
Tension Parallel to Grain ( $F_t$ )	450 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	1,150 psi
Modulus of Elasticity (E)	1,400,000 psi
Modulus of Elasticity ( $E_{min}$ )	510,000 psi

### **Factors**

$C_D$ (Load Duration)	1.00
$C_F$ (Size)	
Bending	1.00
Tension	1.00
Compression	1.00
$C_i$ (Incising)	1.00
$C_L$ (Stability)	1.00
$C_M$ (Wet Service)	1.00
$C_r$ (Repetitive Member)	1.15
$C_t$ (Thermal)	1.00

### **Wood Dimensions and Spacing**

Floor Joist Length: L	16.00 ft
Joist Spacing:	16.00 in
Section Modulus: S	
2 x 6 =	7.56 in <sup>3</sup>
2 x 8 =	14.06 in <sup>3</sup>
2 x 10 =	22.56 in <sup>3</sup>
2 x 12 =	33.06 in <sup>3</sup>
Area: A	
2 x 6 =	8.25 in <sup>2</sup>
2 x 8 =	11.25 in <sup>2</sup>
2 x 10 =	14.25 in <sup>2</sup>
2 x 12 =	17.25 in <sup>2</sup>
Moment of Inertia: I	
2 x 6 =	20.80 in <sup>3</sup>
2 x 8 =	52.73 in <sup>3</sup>
2 x 10 =	107.17 in <sup>3</sup>
2 x 12 =	190.11 in <sup>3</sup>

### **Loads**

Total Load:  $w_{TL}$   
 $(D + L) \times \text{Joist Spacing} = 69.5 \text{ lb/ft}$

Shear:  $V$   
 $(w_{TL} \times L)/2 = 556 \text{ lb}$

Moment:  $M$   
 $(w_{TL} \times L^2)/8 = 2,223 \text{ ft-lb}$   
 $26.7 \text{ in-k}$

**Bending**

Adjusted Bending Design Value:  $F'_b$   
 $F_b (C_D)(C_M)(C_t)(C_L)(C_F)(C_r)(C_i) = 1,006 \text{ psi}$   
 $1.01 \text{ ksi}$

Required Section Modulus: Req'd  $S$   
 $M/F'_b = 26.5 \text{ in}^3 \Rightarrow \text{Try } 2 \times 12$

Actual Bending Stress Design Value:  $f_b$   
 $M/S = 807 \text{ psi}$

$F'_b > f_b$   $1,006 > 807$  TRUE

**Shear**

Adjusted Shear Design Value Parallel to Grain:  $F'_v$   
 $F_v (C_D)(C_M)(C_t)(C_i) = 135 \text{ psi}$

Actual Shear Stress Parallel to Grain:  $f_v$   
 $1.5V/A = 48.3 \text{ psi}$

$F'_v > f_v$   $135 > 48.3$  TRUE

**Deflection**

Adjusted Modulus of Elasticity:  $E'$   
 $E(C_M)(C_t)(C_i) = 1,400,000 \text{ psi}$

Actual Deflection Under Live Load:  $\Delta_L$   
 $5w_L L^4/384E'I = 0.22 \text{ in}$

Allowable Deflection Under Snow Load: Allow.  $\Delta_L$   
 $L/360 = 0.53 \text{ in}$

$0.53 > 0.22$  TRUE

Actual Deflection Under Total Load:  $\Delta_{TL}$   
 $\Delta_L (w_{TL}/w_L) = 0.29 \text{ in}$

Allowable Deflection: Allow.  $\Delta_{TL}$

$L/240 =$

0.80 in

0.80 > 0.29

TRUE

**Use 2 x 12 SPF No. 2 or Better for All Floor Joists**  
**MC  $\leq$  19%**

# ASD Diaphragm Chords & Struts

### **Wood Properties**

Spruce-Pine-Fir	No.2
Bending ( $F_b$ )	875 psi
Tension Parallel to Grain ( $F_t$ )	450 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	1,150 psi
Modulus of Elasticity (E)	1,400,000 psi
Modulus of Elasticity ( $E_{min}$ )	510,000 psi

### **Factors**

$G_s$ (Specific Gravity)	0.92
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### **Wood Dimensions and Spacing**

Wall Height: h	10.00 ft
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#### Section Modulus: S

2 x 6 =	7.56 in <sup>3</sup>
2 x 8 =	14.06 in <sup>3</sup>
2 x 10 =	22.56 in <sup>3</sup>
2 x 12 =	33.06 in <sup>3</sup>

#### Area: A

2 x 6 =	8.25 in <sup>2</sup>
2 x 8 =	11.25 in <sup>2</sup>
2 x 10 =	14.25 in <sup>2</sup>
2 x 12 =	17.25 in <sup>2</sup>

#### Moment of Inertia: I

2 x 6 =	20.80 in <sup>3</sup>
2 x 8 =	52.73 in <sup>3</sup>
2 x 10 =	107.17 in <sup>3</sup>
2 x 12 =	190.11 in <sup>3</sup>

### **Loads**

Transverse Lateral Force: $w_T$ (Lateral Wind Load) x (Wall Height) =	350 lb/ft
--	-----------

Longitudinal Lateral Force: $w_L$ (Lateral Wind Load) x (Wall Height) =	350 lb/ft
--	-----------

Transverse Moment: $M_T$ $(w_T \times L^2)/8 =$	109 ft-k
--	----------

Logintudinal Moment:  $M_L$   
 $(w_T \times L^2)/8 =$  44.8 ft-k

**Transverse Chord Forces**

Compression:  $C_u$   
 $M_T/b =$  3.42 k

Tension:  $T_u$   
 $M_T/b =$  3.42 k

Actual Compression Stress Parallel to Grain in Two (2)  
 2 x 6 Wood Members:  $f_c$   
 $C_u/A =$  0.21 ksi  
 207 psi

Actual Tension Stress Parallel to Grain in Two (2)  
 2 x 6 Wood Members:  $f_t$   
 $T_u/A =$  0.21 ksi  
 207 psi

Allowable ASD Tension Stress Parallel to Grain for SPF No. 2:  
 $F_t =$  450 psi

$F_t > f_t$  450 > 207 TRUE

Allowable ASD Compression Stress Parallel to Grain for SPF No. 2:  
 $F_c =$  425 psi

$F_c > f_c$  425 > 207 TRUE

**Two (2) 2 x 6 SPF No. 2 OK for Transverse Chord Forces**

**Longitudinal Chord Forces:**

Compression:  $C_u$   
 $M_T/b =$  0.90 k

Tension:  $T_u$   
 $M_T/b =$  0.90 k

Actual Compression Stress Parallel to Grain in Two (2)  
 2 x 6 Wood Members:  $f_c$   
 $C_u/A =$  0.05 ksi  
 54.3 psi

Actual Tension Stress Parallel to Grain in Two (2)

2 x 6 Wood Members:  $f_t$

$T_u/A =$  0.05 ksi  
54.3 psi

Allowable Tension Stress Parallel to Grain for SPF No. 2:

$F_t =$  135 psi

$F_t > f_t$  135 > 54.3 TRUE

Allowable Compression Stress Parallel to Grain for SPF No. 2:

$F_c =$  425 psi

$F_c > f_c$  425 > 54.3 TRUE

**Two (2) 2 x 6 SPF No. 2 OK for Longitudinal Chord Forces**

**Longitudinal Lateral Forces**

Unit Shear:  $v_u$

$V_u/b =$  112.0 lb/ft

Actual Unit Shear:  $V$

$1.0v_u =$  112.0 lb/ft

Load Case: Case 3

Maximum Nominal Unit Shear for Wind with 15/32" APA  
Rated 32/16 3-Ply Plywood Panels with 8d Common Nails  
at 6" o.c. :  $v_w$

505 lb/ft

Allowable Unit Shear: Allow.  $v$

$(G*v_w)/2 =$  232 lb/ft

Allow.  $v > V$  232 > 112.0 TRUE

**No Blocking Required for Longitudinal Lateral Force**

**Transverse Strut Forces**

Diaphragm Unit Shear:  $v_R$

$V_u/b =$  273 lb/ft

Shear Wall Unit Shear:  $v_w$

$V_u/b =$  307 lb/ft

Tension:  $T_A$

$$v_R(L/2) - v_W(\Sigma \text{ Opening}) = 2.79 \text{ k}$$

Compression:  $C_A$

$$v_R(L/2) - v_W(\Sigma \text{ Opening}) = 2.79 \text{ k}$$

Chord vs. Strut Forces: 3.42 > 2.79 TRUE

**Two (2) 2 x 6 SPF No.2 OK for Transverse Strut Forces\***

\*Chords and struts are the same member designed for forces from different direction (perpendicular or parallel). Because, in this case, the chord forces in the transverse direction are larger than the strut forces the chord design governs and can be used for the struts

**Longitudinal Strut Forces**

Diaphragm Unit Shear:  $v_R$

$$V_U/b = 112 \text{ lb/ft}$$

Shear Wall Unit Shear:  $v_W$

$$V_U/b = 174 \text{ lb/ft}$$

Tension:  $T_A$

$$v_R(L/2) - v_W(\Sigma \text{ Opening}) = 0.51 \text{ k}$$

Compression:  $C_A$

$$v_R(L/2) - v_W(\Sigma \text{ Opening}) = 0.51 \text{ k}$$

Chord vs. Strut Forces: 0.90 > 0.51 TRUE

**Use Two (2) 2 x 6 SPF No. 2 or Better for All Diaphragm Chords and Struts (Top Plates)  
MC ≤ 19%**



# ASD Floor Sheathing

**Loads**

Total Load:  $w_{TL}$

$D + L =$  52.1 psf

Applied Load:  $w_{AL}$

$L =$  40.0 psf

**Factors**

$C_D$  (Load Duration) 0.90

$C_G$  (Grade and Construction)

Stiffness

Perpendicular to Joists 1.10

Parallel to Joists 1.00

Bending

Perpendicular to Joists 1.00

Parallel to Joists 1.00

Shear

Perpendicular to Joists 1.00

Parallel to Joists 2.80

$C_{SA}$  (Span Adjustment)

3-Span to 1-Span

Stiffness 0.53

Bending 0.80

Shear 1.20

$G_s$  (Specific Gravity) 0.92

**Strength Axis Perpendicular to Joists**

Span Rating: 32 /16

Plywood Type: 3-Ply

Applied Load:  $w_{AL}$

$L =$  40.0 psf

Allowable Uniform Applied Load on APA Rated Plywood

Sheathing Across 16" o.c. Supports:  $W_{AL}$  282 psf

Adjusted Allowable Uniform Applied Load for 3-Ply

Plywood Sheathing:  $W'_{AL}$

$W_{AL}(C_G)(C_{SA}) =$  164 psf

$W'_{AL} > w_{AL}$  164 > 40.0 TRUE

Total Load: $w_{TL}$				
D + L =		52.1 psf		
Allowable Uniform Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{TL}$		376 psf		
Adjusted Allowable Uniform Load for 3-Ply Plywood Sheathing: $W'_{TL}$				
$W_{TL}(C_G)(C_{SA}) =$		219 psf		
$W'_{TL} > w_{TL}$	219 > 52.1			TRUE
Actual Bending Stress: $f_b$				
D + L =		52.1 psf		
Allowable Bending Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_b$		173 psf		
Adjusted Allowable Bending Stress for 3-Ply Plywood Sheathing: $F'_b$				
$F_b(C_G)(C_D)(C_{SA}) =$		125 psf		
$F'_b > f_b$	125 > 52.1			TRUE
Actual Shear Stress: $f_v$				
D + L =		52.1 psf		
Allowable Shear Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_v$		290 psf		
Adjusted Allowable Shear Stress for 3-Ply Plywood Sheathing: $F'_v$				
$F_v(C_G)(C_D)(C_{SA}) =$		209 psf		
$F'_v > f_v$	209 > 52.1			TRUE

**APA Rated 32/16 3-Ply Plywood OK for Floor Sheathing Laid with Strength Axis Perpendicular to Joists**

**Strength Axis Parallel to Joists**

Span Rating:	32 /16
Plywood Type:	3-Ply

Applied Load: $w_{AL}$ L =	40.0 psf		
Allowable Uniform Applied Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{AL}$	20.0 psf		
Adjusted Allowable Uniform Applied Load for 3-Ply Plywood Sheathing: $W'_{AL}$ $W_{AL}(C_G)(C_{SA}) =$	10.6 psf		
$W'_{AL} > w_{AL}$	10.6	>	40.0 FAIL
Total Load: $w_{TL}$ D + L =	52.1 psf		
Allowable Uniform Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{TL}$	27.0 psf		
Adjusted Allowable Uniform Load for 3-Ply Plywood Sheathing: $W'_{TL}$ $W_{TL}(C_G)(C_{SA}) =$	14.3 psf		
$W'_{TL} > w_{TL}$	14.3	>	52.1 FAIL
Actual Bending Stress: $f_b$ D + L =	52.1 psf		
Allowable Bending Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_b$	43.0 psf		
Adjusted Allowable Bending Stress for 3-Ply Plywood Sheathing: $F'_b$ $F_b(C_G)(C_D)(C_{SA}) =$	31.0 psf		
$F'_b > f_b$	31.0	>	52.1 FAIL
Actual Shear Stress: $f_v$ D + L =	52.1 psf		
Allowable Shear Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_v$	179 psf		

Adjusted Allowable Shear Stress for 3-Ply Plywood

Sheathing:  $F'_v$

$F_v (C_G)(C_D)(C_{SA}) = 541 \text{ psf}$

$F'_v > f_v \quad 541 > 52.1 \quad \text{TRUE}$

Span Rating: 40 /20

Plywood Type: 3-Ply

Applied Load:  $w_{AL}$

$L = 40.0 \text{ psf}$

Allowable Uniform Applied Load on APA Rated Plywood

Sheathing Across 16" o.c. Supports:  $W_{AL}$  44.0 psf

Adjusted Allowable Uniform Applied Load for 3-Ply

Plywood Sheathing:  $W'_{AL}$

$W_{AL}(C_G)(C_{SA}) = 23.3 \text{ psf}$

$W'_{AL} > w_{AL} \quad 23.3 > 40.0 \quad \text{FAIL}$

Total Load:  $w_{TL}$

$D + L = 52.1 \text{ psf}$

Allowable Uniform Load on APA Rated Plywood

Sheathing Across 16" o.c. Supports:  $W_{TL}$  59.0 psf

Adjusted Allowable Uniform Load for 3-Ply Plywood

Sheathing:  $W'_{TL}$

$W_{TL}(C_G)(C_{SA}) = 31.3 \text{ psf}$

$W'_{TL} > w_{TL} \quad 31.3 > 52.1 \quad \text{FAIL}$

Actual Bending Stress:  $f_b$

$D + L = 52.1 \text{ psf}$

Allowable Bending Stress for APA Rated Plywood Sheathing

Across 16" o.c. Supports:  $F_b$  70.0 psf

Adjusted Allowable Bending Stress for 3-Ply Plywood

Sheathing:  $F'_b$

$F_b (C_G)(C_D)(C_{SA}) = 50.4 \text{ psf}$

$F'_b > f_b$	50.4 > 52.1	FAIL
Actual Shear Stress: $f_v$ D + L =	52.1 psf	
Allowable Shear Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_v$	228 psf	
Adjusted Allowable Shear Stress for 3-Ply Plywood Sheathing: $F'_v$ $F_v (C_G)(C_D)(C_{SA}) =$	689 psf	
$F'_v > f_v$	689 > 52.1	TRUE

**No APA Rated 3-Ply Plywood OK for Floor Sheathing Laid with Strength Axis Parallel to Joists**

**Thickness and Nailing**

3/8" Thick APA Rated 32/16 3-Ply Plywood Nailed Perpendicular to Joists with 6d Common Nails

Load Case:	Case 1	
Maximum Nominal Unit Shear for Wind Loading on Douglas Fir Plywood: $V_w$	460 lb/ft	
Adjusted Nominal Unit Shear for Wind Loading on SPF Plywood: $V'_w$ $G_s V_w / 2 =$	212 lb/ft	
Actual Shear: $v$ 1.0W	350 lb/ft	
$V'_w > v$	212 > 350	FAIL

---

19/32" Thick APA Rated 32/16 3-Ply Plywood Nailed Perpendicular to Joists with 10d Common Nails

Load Case:	Case 1	
Maximum Nominal Unit Shear for Wind Loading on Douglas Fir Plywood: $V_w$	800 lb/ft	
Adjusted Nominal Unit Shear for Wind Loading on SPF Plywood: $V'_w$ $G_s V_w / 2 =$	368 lb/ft	

Actual Shear:  $v$

1.0W

350 lb/ft

$V'_w > v$

368 > 350

TRUE

**Use 19/32" APA Rated 32/16 3-Ply Plywood Panels or Better as Subfloor  
With 1/4" Underlayment Grade Panel Installed Over Subfloor  
Nailed Perpendicular to Joists with 10d Common Nails  
at 6" o.c. Supported Edges  
12" o.c. Field  
No Blocking Required**

# ASD Studs



### **Wood Properties**

Spruce-Pine-Fir	Stud
Bending ( $F_b$ )	675 psi
Tension Parallel to Grain ( $F_t$ )	350 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	725 psi
Modulus of Elasticity (E)	1,200,000 psi
Modulus of Elasticity ( $E_{min}$ )	440,000 psi

### **Factors**

$C_D$ (Load Duration)	
Wind	1.60
Snow	1.15
Dead Load	0.90
$C_F$ (Size)	
Bending	1.10
Tension	1.10
Compression	1.05
$C_i$ (Incising)	1.00
$C_L$ (Stability)	1.00
$C_M$ (Wet Service)	1.00
$C_r$ (Repetitive Member)	1.15
$C_t$ (Thermal)	1.00

### **Wood Dimensions and Spacing**

Stud Length: L	10.00 ft
Stud Spacing:	16.00 in
Section Modulus: S	
2 x 4 =	3.06 in <sup>3</sup>
2 x 6 =	7.56 in <sup>3</sup>
Area: A	
2 x 4 =	5.25 in <sup>2</sup>
2 x 6 =	8.25 in <sup>2</sup>
Moment of Inertia: I	
2 x 4 =	5.36 in <sup>3</sup>
2 x 6 =	20.80 in <sup>3</sup>

### **Load Case 1: Gravity Loads Only**

Ultimate Concentrated Load: P	
(D + 0.75L + 0.75S) x Stud Spacing =	1,028 lb
	1.03 k

Shear: V  
 $(P \times L)/2 =$  5,139 lb  
 5.14 k

Moment: M  
 $(P \times L^2)/8 =$  12,847 ft-lb  
 154 k-in

**Column Capacity:**

Column Buckling About y-axis:

$(l_e/d)_y =$  0 => Sheathing

Column Buckling About x-axis:

$(l_e/d)_x =$  21.8

Adjusted Modulus of Elasticity for Stability:  $E'_{min}$

$E_{min}(C_M)(C_t)(C_i) =$  440,000 psi

Critical Buckling Value for Compression:  $F_{cE}$

$0.822E'_{min}/(l_e/d)^2 =$  760 psi  
 0.76 ksi

Reference Compression Design Value Parallel to Grain

Multiplied by all Adjustment Factors Except  $C_p$ :  $F^*_c$

$F_c(C_D)(C_M)(C_t)(C_F)(C_i) =$  875 psi  
 0.88 ksi

$F_{cE}/F^*_c =$  0.868

$(1+F_{cE}/F^*_c)/2c =$  1.167

Column Stability Factor:  $C_p$

$(1+F_{cE}/F^*_c)/2c - \text{Sqrt}(((1+F_{cE}/F^*_c)/2c)^2 - (F_{cE}/F^*_c)/c) =$  0.640

Adjusted Compression Design Value Parallel to Grain :  $F'_c$

$F_c(C_D)(C_M)(C_t)(C_F)(C_p)(C_i) =$  560 psi  
 0.56 ksi

Adjusted Lateral Design Value Parallel to Grain:  $P'$

$F'_c(A) =$  4,623 psi  
 4.62 ksi

$P' > P$  4.62 > 1.03 TRUE

**Bearing of Stud on Wall Plates**

Bearing Area Factor:  $C_b$

$(l_b + 0.375)/l_b =$  1.25

Adjusted Compression Design Value Perpendicular to

Grain:  $F'_{c\perp}$

$$F_{c\perp}(C_M)(C_t)(C_b) = \begin{array}{l} 531 \text{ psi} \\ 0.53 \text{ ksi} \end{array}$$

Adjusted Lateral Design Value Perpendicular to Grain:  $P'_{\perp}$

$$F'_{c\perp}(A) = 4.38 \text{ ksi}$$

$$P'_{\perp} > P_u \quad 4.38 > 1.03 \quad \text{TRUE}$$

**Load Case 2: Gravity Loads + Lateral Loads For .75W**

Lateral Load:  $w$

$$0.75W \times (\text{Stud Spacing}) = 35.00 \text{ lb/ft}$$

Shear:  $V$

$$(w \times L)/2 = \begin{array}{l} 175 \text{ lbs} \\ 0.18 \text{ k} \end{array}$$

Moment:  $M$

$$(w \times L^2)/8 = \begin{array}{l} 438 \text{ ft-lb} \\ 5.25 \text{ k-in} \end{array}$$

**Bending**

Actual Bending Stress:  $f_b$

$$M/S = 0.69 \text{ ksi}$$

Adjusted Bending Design Value:  $F'_b$

$$F_b(C_D)(C_M)(C_t)(C_L)(C_F)(C_r)(C_i) = \begin{array}{l} 1,366 \text{ psi} \\ 1.37 \text{ ksi} \end{array}$$

Adjusted Moment Design Value:  $M'$

$$F'_b(S) = 10.3 \text{ k-in}$$

$$M' > M \quad 10.3 > 5.25 \quad \text{TRUE}$$

**Axial**

Axial Load:  $P$

$$(D + 0.75L + 0.75S) \times \text{Stud Spacing} = \begin{array}{l} 1,028 \text{ lb} \\ 1.03 \text{ k} \end{array}$$

Actual Compression Stress Parallel to Grain:  $f_c$

$$P/A = 0.12 \text{ ksi}$$

**Combined Stress:**

Interaction Formula:

$$(f_c/F'_c)^2 + (f_b/(F'_b(1-f_c/F_{cE}))) \leq 1.0 \quad 0.657 \leq 1 \quad \text{TRUE}$$

**Gravity Loads + Lateral Loads For W**

Lateral Load:  $w$   
 $W \times (\text{Stud Spacing}) = 46.67 \text{ lb/ft}$

Shear:  $V$   
 $(w \times L)/2 = 233 \text{ lbs}$   
 $0.23 \text{ k}$

Moment:  $M$   
 $(w \times L^2)/8 = 583 \text{ ft-lb}$   
 $7.00 \text{ k-in}$

### **Bending**

Actual Bending Stress:  $f_b$

$M/S = 0.93 \text{ ksi}$

Adjusted Bending Design Value:  $F'_b$

$F_b (C_D)(C_M)(C_t)(C_L)(C_F)(C_r)(C_i) = 1,366 \text{ psi}$   
 $1.37 \text{ ksi}$

Adjusted Moment Design Value:  $M'$

$F'_b (S) = 10.3 \text{ k-in}$

$M' > M \quad 10.3 > 7.00 \quad \text{TRUE}$

### **Axial**

Axial Load:  $P$

$D \times \text{Stud Spacing} = 556 \text{ lb}$   
 $0.56 \text{ k}$

Actual Compression Stress Parallel to Grain:  $f_c$

$P/A = 0.07 \text{ ksi}$

### **Column Capacity:**

Column Buckling About y-axis:

$(l_e/d)_y = 0 \Rightarrow \text{Sheathing}$

Column Buckling About x-axis:

$(l_e/d)_x = 21.82$

Adjusted Modulus of Elasticity for Stability:  $E'_{\min}$

$E_{\min}(C_M)(C_t)(C_i) = 440,000 \text{ psi}$

Critical Buckling Value for Compression:  $F_{cE}$

$0.822E'_{\min}/(l_e/d)^2 = 760 \text{ psi}$   
 $0.76 \text{ ksi}$

Reference Compression Design Value Parallel to Grain  
Multiplied by all Adjustment Factors Except  $C_p$ :  $F^*_c$

$$F_c (C_D)(C_M)(C_t)(C_F)(C_i) = \begin{matrix} 685 \text{ psi} \\ 0.69 \text{ ksi} \end{matrix}$$

$$F_{cE}/F_c^* = 1.109$$

$$(1+F_{cE}/F_c^*)/2c = 1.318$$

Column Stability Factor:  $C_p$

$$(1+F_{cE}/F_c^*)/2c - \text{Sqrt}(((1+F_{cE}/F_c^*)/2c)^2 - (F_{cE}/F_c^*)/c) = 0.725$$

Adjusted Compression Design Value Parallel to Grain :  $F'_c$

$$F_c (C_D)(C_M)(C_t)(C_F)(C_p)(C_i) = \begin{matrix} 497 \text{ psi} \\ 0.50 \text{ ksi} \end{matrix}$$

Adjusted Lateral Design Value Parallel to Grain:  $P'$

$$F'_c (A) = \begin{matrix} 4,101 \text{ psi} \\ 4.10 \text{ ksi} \end{matrix}$$

$$P' > P \quad 4.10 > 0.56 \quad \text{TRUE}$$

**Combined Stress:**

Interaction Formula:

$$(f_c/F'_c)^2 + (f_b/(F'_b(1-f_c/F_{cE}))) \leq 1.0 \quad 0.762 \leq 1 \quad \text{TRUE}$$

**Use 2 x 6 SPF No. 2 or Better for All Studs  
MC ≤ 19%**

# ASD Shearwalls

### **Wood Properties**

Spruce-Pine-Fir	Stud
Bending ( $F_b$ )	675 psi
Tension Parallel to Grain ( $F_t$ )	350 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	725 psi
Modulus of Elasticity (E)	1,200,000 psi
Modulus of Elasticity ( $E_{min}$ )	440,000 psi

### **Factors**

$C_D$ (Load Duration)	1.60
$C_F$ (Size)	
Bending	1.10
Tension	1.10
Compression	1.05
$C_i$ (Incising)	1.00
$C_L$ (Stability)	1.00
$C_M$ (Wet Service)	1.00
$C_r$ (Repetitive Member)	1.15
$C_t$ (Thermal)	1.00

### **Wood Dimensions and Spacing:**

Wall Height: h	10.00 ft
Stud Spacing:	16.00 in
Section Modulus: S	
2 x 6 =	7.56 in <sup>3</sup>
4 x 6 =	17.65 in <sup>3</sup>
Area: A	
2 x 6 =	8.25 in <sup>2</sup>
4 x 6 =	19.25 in <sup>2</sup>
Moment of Inertia: I	
2 x 6 =	20.80 in <sup>3</sup>
4 x 6 =	48.53 in <sup>3</sup>

### **Loads**

Ultimate Uniform Wind Load: $w_u$ (Lateral Wind Load) x (Wall Height)	350 lb/ft
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**Wall 1 (First Floor Front Facing Wall)**

Ultimate Shear Force in Shearwall:  $V_u$

$$w_u(b/2) = \begin{matrix} 5,600 \text{ lb} \\ 5.6 \text{ k} \end{matrix}$$

$$\text{Ultimate Unit Shear in Shearwall: } v_u = 174 \text{ lb/ft}$$

$$\text{Unit Shear in Shearwall: } v \\ 1.0v_u = 174 \text{ lb/ft}$$

$$\begin{matrix} 5/16" \text{ Plywood Siding with 6d Common Nail @ 6" o.c.} \\ \text{Maximum Nominal Unit Shear for Wind: } v_w = 390 \text{ lb/ft} \end{matrix}$$

$$\begin{matrix} \text{Allowable Unit Shear for SPF: Allow. } v \\ (G_s \times v_w)/2 = 179 \text{ lb/ft} \end{matrix}$$

$$\text{Allow. } v > v \quad 179 > 174 \quad \text{TRUE}$$

**5/16" Plywood Siding with 6d Common Nails at 6" o.c. OK for Wall 1**

**Wall 2 (First Floor Back Facing Wall)**

Ultimate Shear Force in Shearwall:  $V_u$

$$w_u(b/2) = \begin{matrix} 5,600 \text{ lb} \\ 5.6 \text{ k} \end{matrix}$$

$$\text{Ultimate Unit Shear in Shearwall: } v_u = 160 \text{ lb/ft}$$

$$\text{Unit Shear in Shearwall: } v \\ 1.0v_u = 160 \text{ lb/ft}$$

$$\begin{matrix} 5/16" \text{ Plywood Siding with 6d Common Nail @ 6" o.c.} \\ \text{Maximum Nominal Unit Shear for Wind: } v_w = 390 \text{ lb/ft} \end{matrix}$$

$$\begin{matrix} \text{Allowable Unit Shear for SPF: Allow. } v \\ (G_s \times v_w)/2 = 179 \text{ lb/ft} \end{matrix}$$

$$\text{Allow. } v > v \quad 179 > 160 \quad \text{TRUE}$$

**5/16" Plywood Siding with 6d Common Nails at 6" o.c. OK for Wall 1**

**Wall 3 (First Floor Right Facing Wall)**

Ultimate Shear Force in Shearwall:  $V_u$

$$w_u(b/2) = \begin{matrix} 8,750 \text{ lb} \\ 8.75 \text{ k} \end{matrix}$$



Ultimate Unit Shear in Shearwall: $v_u$	273 lb/ft		
Unit Shear in Shearwall: $v$ $1.0v_u =$	273 lb/ft		
5/16" Plywood Siding with 6d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	390 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $(G_s \times v_w)/2 =$	179 lb/ft		
Allow. $v > v$	179	>	273 FAIL
3/8" Structural Panels -- Sheathing with 8d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	615 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $(G_s \times v_w)/2 =$	283 lb/ft		
Allow. $v > v$	283	>	273 TRUE

**3/8" Structural Panels -- Sheathing with 8d Common Nails at 6" o.c. OK for Wall 3**

**Wall 4 (First Floor Left Facing Wall)**

Ultimate Shear Force in Shearwall: $V_u$ $w_u(b/2) =$	8,750 lb 8.75 k		
Ultimate Unit Shear in Shearwall: $v_u$	307 lb/ft		
Unit Shear in Shearwall: $v$ $1.0v_u =$	307 lb/ft		
5/16" Plywood Siding with 6d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	390 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $(G_s \times v_w)/2 =$	179 lb/ft		
Allow. $v > v$	179	>	307 FAIL
7/16" Structural Panels -- Sheathing with 8d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	670 lb/ft		

Allowable Unit Shear for SPF: Allow. $v$ ( $G_s \times v_w$ )/2 =	308 lb/ft		
Allow. $v > v$	308	>	307      TRUE

**7/16" Structural Panels -- Sheathing with 8d Common Nails at 6" o.c. OK for Wall 4**

**Wall 5 (Second Floor Front Facing Wall)**

Ultimate Shear Force in Shearwall: $V_u$ $w_u(b/2) =$	5,600 lb 5.6 k		
Ultimate Unit Shear in Shearwall: $v_u$	174 lb/ft		
Unit Shear in Shearwall: $v$ $1.0v_u =$	174 lb/ft		
5/16" Plywood Siding with 6d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	390 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ ( $G_s \times v_w$ )/2 =	179 lb/ft		
Allow. $v > v$	179	>	174      TRUE

**5/16" Plywood with 6d Common Nails at 6" o.c. OK for Wall 5**

**Wall 6 (Second Floor Back Facing Wall)**

Ultimate Shear Force in Shearwall: $V_u$ $w_u(b/2) =$	5,600 lb 5.6 k		
Ultimate Unit Shear in Shearwall: $v_u$	145 lb/ft		
Unit Shear in Shearwall: $v$ $1.0v_u =$	145 lb/ft		
5/16" Plywood Siding with 6d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	390 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ ( $G_s \times v_w$ )/2 =	179 lb/ft		
Allow. $v > v$	179	>	145      TRUE

**5/16" Plywood with 6d Common Nails at 6" o.c. OK for Wall 6**

**Wall 7 (Second Floor Right Facing Wall)**

Ultimate Shear Force in Shearwall:  $V_u$

$$w_u(b/2) = \begin{array}{r} 8,750 \text{ lb} \\ 8.75 \text{ k} \end{array}$$

$$\text{Ultimate Unit Shear in Shearwall: } v_u = 297 \text{ lb/ft}$$

Unit Shear in Shearwall:  $v$

$$1.0v_u = 297 \text{ lb/ft}$$

5/16" Plywood Siding with 6d Common Nail @ 6" o.c.

$$\text{Maximum Nominal Unit Shear for Wind: } v_w = 390 \text{ lb/ft}$$

Allowable Unit Shear for SPF: Allow.  $v$

$$(G_s \times v_w)/2 = 179 \text{ lb/ft}$$

$$\text{Allow. } v > v \quad 179 > 297 \quad \text{FAIL}$$

7/16" Structural Panels -- Sheathing with 8d Common Nail

$$\text{@ 6" o.c. Maximum Nominal Unit Shear for Wind: } v_w = 670 \text{ lb/ft}$$

Allowable Unit Shear for SPF: Allow.  $v$

$$(G_s \times v_w)/2 = 308 \text{ lb/ft}$$

$$\text{Allow. } v > v \quad 308 > 297 \quad \text{TRUE}$$

**7/16" Structural Panels -- Sheathing with 8d Common Nails at 6" o.c. OK for Wall 7**

**Wall 8 (Second Floor Left Facing Wall)**

Ultimate Shear Force in Shearwall:  $V_u$

$$w_u(b/2) = \begin{array}{r} 8,750 \text{ lb} \\ 8.75 \text{ k} \end{array}$$

$$\text{Ultimate Unit Shear in Shearwall: } v_u = 273 \text{ lb/ft}$$

Unit Shear in Shearwall:  $v$

$$1.0v_u = 273 \text{ lb/ft}$$

5/16" Plywood Siding with 6d Common Nail @ 6" o.c.

$$\text{Maximum Nominal Unit Shear for Wind: } v_w = 390 \text{ lb/ft}$$

Allowable Unit Shear for SPF: Allow.  $v$   
 $(G_s \times v_w)/2 =$  179 lb/ft

Allow.  $v > v$  179 > 273 FAIL

3/8" Structural Panels -- Sheathing with 8d Common Nail  
 @ 6" o.c. Maximum Nominal Unit Shear for Wind:  $v_w$  615 lb/ft

Allowable Unit Shear for SPF: Allow.  $v$   
 $(G_s \times v_w)/2 =$  283 lb/ft

Allow.  $v > v$  283 > 273 TRUE

**3/8" Structural Panels -- Sheathing with 8d Common Nails at 6" o.c. OK for Wall 8**

**Tension Chord**

Load at Top of Shearwall:  $v$   
 $1.0v_u =$  307 lb/ft

Tension: T  
 $v h =$  3,070 lb  
 3.07 k

Net Area:  $A_n$   
 $bh =$  8.25 in<sup>2</sup>

Actual Tension Stress Parallel to Grain:  $f_t$   
 $T/A =$  372 psi

Adjusted ASD Tension Design Value:  $F'_t$   
 $F_t(C_D)(C_M)(C_t)(C_F) =$  616 psi

$F'_t > f_t$  616 > 372 TRUE

**One 2 x 6 OK for All Tension Chords of Shearwalls**

**Compression Chord**

Column Buckling About y-axis:  
 $(l_e/d)_y$  0.00 => Sheathing

Column Buckling About x-axis:  
 $(l_e/d)_x =$  21.8

Adjusted Modulus of Elasticity for Stability:  $E'_{\min}$   
 $E'_{\min}(C_M)(C_t)(C_i) =$  440,000 psi  
 Nominal Buckling Value for Compression:  $F_{cE}$   
 $0.822E'_{\min}/(l_e/d)^2 =$  760 psi

Nominal Compression Design Value Parallel to Grain  
 Multiplied by all Adjustment Factors Except  $C_p$ :  $F^*_c$   
 $F_c(C_D)(C_M)(C_t)(C_F)(C_i) =$  1,218 psi  
 1.22 ksi

$F_{cE}/F^*_c =$  0.624

$(1+F_{cE}/F^*_c)/2c =$  1.015

Column Stability Factor:  $C_p$   
 $(1+F_{cE}/F^*_c)/2c - \text{sqrt}(((1+F_{cE}/F^*_c)/2c)^2 - (F_{cE}/F^*_c)/c) =$  0.515

Adjusted ASD Compression Design Value Parallel  
 to Grain:  $F'_c$   
 $F_c(C_D)(C_M)(C_t)(C_F)(C_p) =$  627 psi

Adjusted ASD Compression Design Value  
 Perpendicular to Grain:  $F'_{c\perp}$   
 $F_{c\perp}(C_M)(C_t)(C_b) =$  425 psi

**$F'_{c\perp}$  Governs**

Total Dead Load Acting on Shearwall:  $w_{DL}$  417 lb/ft

Total Load Acting on Chord: P  
 (Tributary Area) x  $w_{DL} =$  6,669 lb  
 6.67 k

Allowable Compression Load on 2 x 6 Chord: Allow. P  
 $F_{c\perp}A =$  3,506 lb  
 3.51 k

Allow. P > P 3.51 > 6.67 FAIL

Allowable Compression Load on 4 x 6 Chord: Allow. P  
 $F_{c\perp}A =$  8,181 lb  
 8.18 k

Allow.  $P > P$

8.18 > 6.67

TRUE

**One 4 x 6 SPF Stud is OK for All Compression Shearwall Chords**

**Unblocked Shearwall Deflection**

Check 5/16" APA Rated 24/0 Wood Structural Panels -- Sheathing with 6d Common Nails at 6" o.c. at Supported Edges and 6" o.c.

Max Shear Force at Top of Wall:  $v_u$  307 lb/ft

Adjusted Shear Force for Unblocked Wall:  $v'_u$

$v_u / C_{ub} =$  384 lb/ft

Load Per Fastener: 192 lb/ft

Bending Deflection:  $\Delta_b$

$8v h^3 / EAb =$  0.004 in

Shear Deflection:  $\Delta_v$

$vh / Gt =$  0.154 in

Nail Slip:  $\Delta_n$

$0.75he_n =$  N/A\*

\* 192 lb/nail exceeds largest allowable load per fastener of 160 lb/ft for 6d common nails

Anchorage Slip:  $\Delta_a$

$(h/b)d_a =$  0.039 in

Story Drift:  $\Delta_s$

$\Delta_b + \Delta_v + \Delta_n + \Delta_a =$  N/A

Total Deflection:  $\Delta$

$C_d \Delta_s =$  N/A

Deflection Limit:  $\Delta_{limit}$

$0.02(h \times 12) =$  2.40 in

$\Delta_{limit} > \Delta$  2.40 > N/A FAIL

---

Check 3/8" APA Rated 24/0 Wood Structural Panels -- Sheathing with 8d Common Nails at 6" o.c. at Supported Edges and 6" o.c. Field

Max Shear Force at Top of Wall:  $v_u$  307 lb/ft

Adjusted Shear Force for Unblocked Wall: $v'_u$ $v_u/C_{ub} =$	384 lb/ft		
Load Per Fastener:	192 lb/ft		
Bending Deflection: $\Delta_b$ $8v h^3/EAb =$	0.004 in		
Shear Deflection: $\Delta_v$ $vh/Gt =$	0.154 in		
Nail Slip: $\Delta_n$ $0.75he_n =$	0.630 in		
Anchorage Slip: $\Delta_a$ $(h/b)d_a =$	0.039 in		
Story Drift: $\Delta_s$ $\Delta_b + \Delta_v + \Delta_n + \Delta_a =$	0.827 in		
Total Deflection: $\Delta$ $C_d \Delta_s =$	3.31 in		
Deflection Limit: $\Delta_{limit}$ $0.02(h \times 12) =$	2.40 in		
$\Delta_{limit} > \Delta$	2.40	>	3.31

FAIL

Check 15/32" APA Rated 32/16 Wood Structural Panels -- Sheathing with 10d Common Nails at 6" o.c. at Supported Edges and 6" o.c. Field

Max Shear Force at Top of Wall: $v_u$	307 lb/ft
Adjusted Shear Force for Unblocked Wall: $v'_u$ $v_u/C_{ub} =$	384 lb/ft
Load Per Fastener:	192 lb/ft
Bending Deflection: $\Delta_b$ $8v h^3/EAb =$	0.004 in
Shear Deflection: $\Delta_v$ $vh/Gt =$	0.142 in

Nail Slip: $\Delta_n$ $0.75he_n =$	0.423 in	
Anchorage Slip: $\Delta_a$ $(h/b)d_a =$	0.039 in	
Story Drift: $\Delta_s$ $\Delta_b + \Delta_v + \Delta_n + \Delta_a =$	0.608 in	
Total Deflection: $\Delta$ $C_d \Delta_s =$	2.43 in	
Deflection Limit: $\Delta_{limit}$ $0.02(h \times 12) =$	2.40 in	
$\Delta_{limit} > \Delta$	2.40 > 2.43	FAIL

Check 19/32" APA Rated 40/20 Structural Panels -- Sheathing with 10d Common Nails at 6" o.c. at Supported Edges and 6" o.c. Field

Max Shear Force at Top of Wall: $v_u$	307 lb/ft
Adjusted Shear Force for Unblocked Wall: $v'_u$ $v_u/C_{ub} =$	384 lb/ft
Load Per Fastener:	192 lb/ft
Bending Deflection: $\Delta_b$ $8v h^3/EAb =$	0.004 in
Shear Deflection: $\Delta_v$ $vh/Gt =$	0.135 in
Nail Slip: $\Delta_n$ $0.75he_n =$	0.423 in
Anchorage Slip: $\Delta_a$ $(h/b)d_a =$	0.039 in
Story Drift: $\Delta_s$ $\Delta_b + \Delta_v + \Delta_n + \Delta_a =$	0.601 in



Total Deflection: $\Delta$			
$C_d \Delta_s =$	2.40 in		
Deflection Limit: $\Delta_{limit}$			
$0.02(h \times 12) =$	2.40 in		
$\Delta_{limit} > \Delta$	2.40	>	2.40      FAIL

Check 15/32" APA Rated 32/16 Structural Panels -- Structural with 10d Common Nails at 6" o.c. at Supported Edges and 6" o.c. Field

Max Shear Force at Top of Wall: $v_u$	307 lb/ft		
Adjusted Shear Force for Unblocked Wall: $v'_u$			
$v_u / C_{ub} =$	384 lb/ft		
Load Per Fastener:	192 lb/ft		
Bending Deflection: $\Delta_b$			
$8v h^3 / EAb =$	0.004 in		
Shear Deflection: $\Delta_v$			
$vh / Gt =$	0.118 in		
Nail Slip: $\Delta_n$			
$0.75he_n =$	0.353 in		
Anchorage Slip: $\Delta_a$			
$(h/b)d_a =$	0.039 in		
Story Drift: $\Delta_s$			
$\Delta_b + \Delta_v + \Delta_n + \Delta_a =$	0.514 in		
Total Deflection: $\Delta$			
$C_d \Delta_s =$	2.06 in		
Deflection Limit: $\Delta_{limit}$			
$0.02(h \times 12) =$	2.40 in		
$\Delta_{limit} > \Delta$	2.40	>	2.06      TRUE

**Use 15/32" APA Rated 32/16 3-Ply Wood Structural Panels -- Structural or Better  
With 10d Common Nails  
at 6" o.c. Supported Edges  
6" o.c. Field  
AND  
One (1) 4 x 6 SPF Stud Post for All Shearwall Chords  
No Blocking Required**

# Foundation

### **Load**

Total Load:  $w_u$

$$D + L + S = 772 \text{ psf}$$

Weight of the foundation:  $W$

$$w_f/b = 1200 \text{ lb/ft}$$

Bearing Pressure:  $q$

$$(P/b + w_f/b)/B = 1972 \text{ lb/ft}^2$$

### **Basement Retaining Wall**

Overconsolidation ratio of soil: OCR = 2

Effective friction angle of soil:  $\Phi'$  = 30 Degrees

Coefficient of Lateral Earth Pressure at Rest:  $K_0$

$$(1 - \sin(\Phi'))(OCR^{\sin \Phi'}) = 0.707$$

Unit Weight of Soil:  $\gamma$  = 127 lb/ft<sup>3</sup>

Height of Wall:  $H$  = 8 ft

Normal Force Acting Between Soil and Wall per Unit

Length of Wall:  $P_0/b$

$$(\gamma)(H^2)(K_0)/2 = 2873.25 \text{ lb/ft}$$

<p style="text-align: center;"><b>Use 12" wide Continuous Footing Supported Light Frame design and 8' Deep Basement Retaining Walls</b></p>
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# ASD Headers

### **Wood Properties**

Spruce-Pine-Fir	No.2
Bending ( $F_b$ )	875 psi
Tension Parallel to Grain ( $F_t$ )	450 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	1,150 psi
Modulus of Elasticity (E)	1,400,000 psi
Modulus of Elasticity ( $E_{min}$ )	510,000 psi

### **Factors**

$C_D$ (Load Duration)	1.60
$C_F$ (Size)	
2 x 10	
Bending	1.10
Tension	1.10
Compression	1.00
2 x 6	
Bending	1.30
Tension	1.30
Compression	1.10
$C_i$ (Incising)	1.00
$C_L$ (Stability)	1.00
$C_M$ (Wet Service)	1.00
$C_r$ (Repetitive Member)	1.15
$C_t$ (Thermal)	1.00

### **Wood Dimensions and Spacing**

Header Length: L	6.67 ft
Section Modulus: S	
2 x 6 =	7.56 in <sup>3</sup>
2 x 8 =	14.06 in <sup>3</sup>
2 x 10 =	22.56 in <sup>3</sup>
2 x 12 =	33.06 in <sup>3</sup>
Area: A	
2 x 6 =	8.25 in <sup>2</sup>
2 x 8 =	11.25 in <sup>2</sup>
2 x 10 =	14.25 in <sup>2</sup>
2 x 12 =	17.25 in <sup>2</sup>
Moment of Inertia: I	
2 x 6 =	20.80 in <sup>3</sup>

$2 \times 8 =$	52.73 in <sup>3</sup>
$2 \times 10 =$	107.17 in <sup>3</sup>
$2 \times 12 =$	190.11 in <sup>3</sup>

**Loads**

Total Load:  $w_{TL}$

$D + 0.75L + 0.75S =$  771 lb/ft

Number of Beams Per Header: N 2.00

Shear: V

Total:  $(w_{TL} \times L)/2 =$  2,569 lb

Per Beam:  $V/N =$  1,285 lb

Moment: M

Total:  $(w_{TL} \times L^2)/8 =$  4,283 ft-lb

Per Beam:  $M/N =$  2,141 ft-lb  
25.7 in-k

**Bending**

Adjusted Bending Design Value:  $F'_b$

$F_b (C_D)(C_M)(C_t)(C_L)(C_F)(C_i) =$  1,540 psi

Required Section Modulus: Req'd S

$M/F'_b =$  16.7 in<sup>3</sup> => Try 2x10

Actual Bending Stress Design Value:  $f_b$

$M/S =$  1,139 psi

$F'_b > f_b$  1,540 > 1,139 TRUE

**Axial**

Adjusted Shear Design Value Parallel to Grain:  $F'_v$

$F_v (C_D)(C_M)(C_t)(C_i) =$  216 psi

Actual Shear Stress Parallel to Grain:  $f_v$

$1.5V/A =$  135 psi

$F'_v > f_v$  216 > 135 TRUE

Adjusted Compression Design Value Perpendicular to

Grain:  $F'_{c\perp}$

$$F_{c\perp}(C_b)(C_M)(C_t)(C_i) = 425 \text{ psi}$$

Required Bearing Area:  $A_B$

$$V/F'_{c\perp} = 3.02 \text{ in}^2$$

Minimum Seat Length:  $L_s$

$$A_B/\text{Support Thickness} = 2.02 \text{ in}$$

Number of Supports Required: N

Support Thickness >  $L_s$

1 Support	1.50	>	2.02	FAIL
2 Supports	3.00	>	2.02	TRUE

Adjusted Seat Length  $L'_s$

$$\text{Combined Thickness of Supports} = 3.00 \text{ in}$$

Total Bearing Area:  $A_T$

$$(\text{Support Thickness}) \times L'_s = 4.50 \text{ in}^2$$

Actual Compression Stress Perpendicular to Grain:  $f'_{c\perp}$

$$V/A_T = 285 \text{ psi}$$

$$F'_{c\perp} > f'_{c\perp} \quad 425 > 285 \quad \text{TRUE}$$

### **Deflection**

Adjusted Modulus of Elasticity:  $E'$

$$E(C_M)(C_t)(C_i) = 1,400,000 \text{ psi}$$

Actual Deflection Under Snow Load:  $\Delta_s$

$$5w_s L^4 / 384E'I = 0.01 \text{ in}$$

Allowable Deflection Under Snow Load: Allow.  $\Delta_s$

$$L/360 = 0.22 \text{ in}$$

$$\text{Allow. } \Delta_s > \Delta_s \quad 0.22 > 0.01 \quad \text{TRUE}$$

Actual Deflection Under Total Load:  $\Delta_{TL}$

$$\Delta_s(w_{TL}/w_s) = 0.12 \text{ in}$$



Allowable Deflection: Allow.  $\Delta_{TL}$

$$L/240 = 0.33 \text{ in}$$

$$\text{Allow. } \Delta_{TL} > \Delta_{TL} \quad 0.33 > 0.12 \quad \text{TRUE}$$

**Supports**

Adjusted Compression Design Value Parallel to Grain :  $F'_c$

$$F_c (C_D)(C_M)(C_t)(C_F)(C_P)(C_i) = 2,024 \text{ psi}$$

Actual Compressive Stress Parallel to Grain:  $f'_c$

$$V/A_T = 285 \text{ psi}$$

$$F'_c > f'_c \quad 2,024 > 285 \quad \text{TRUE}$$

**Use Two (2) 2 x 10 SPF No. 2 or Better for All Headers  
And  
Two (2) 2 x 6 SPF No. 2 or Better for All Header Support Jacks  
MC ≤ 19%**

# ASD Connections

# Connection Values

## Dowels

### Nails

16d Common Nails:

D (Diameter)	0.168 in
L (Length)	3.50 in
$F_{yb}$ (Bending Yield Strength of Fastener)	90.0 ksi
$K_D$ (Reduction Coefficient for Fasteners with $D < 1/4"$ )	2.20

10d Common Nails:

D (Diameter)	0.148 in
L (Length)	3.00 in
$F_{yb}$ (Bending Yield Strength of Fastener)	90.0 ksi
$K_D$ (Reduction Coefficient for Fasteners with $D < 1/4"$ )	2.20

8d Common Nails:

D (Diameter)	0.131 in
L (Length)	2.50 in
$F_{yb}$ (Bending Yield Strength of Fastener)	100 ksi
$K_D$ (Reduction Coefficient for Fasteners with $D < 1/4"$ )	2.20

### Bolts

A307 Bolt:

D (Diameter)	0.75 in
L (Length)	5.00 in
$F_{yb}$ (Bending Yield Strength of Fastener)	45.0 ksi

## Members

### Spruce-Pine-Fir Wood

$F_e$ (Dowel Bearing Strength)	3,500 psi
G (Specific Gravity)	0.42

### Concrete

$F_e$ (Dowel Bearing Strength)	7,500 psi
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## Adjustment Factors

$C_D$ (Load Duration)	1.60
$C_{di}$ (Diaphragm)	1.00
$C_{eg}$ (End Grain)	
End Nail Connection	0.67
Non-End Nail Connection	1.00
$C_g$ (Group)	1.00
$C_i$ (Incising)	1.00
$C_M$ (Wet Service)	1.00
$C_t$ (Thermal)	1.00
$C_{tn}$ (Toenail)	
Toenail Connection	0.83
Non-Toenail Connection	1.00
$C_\Delta$ (Geometry)	1.00

## Connection Glossary

$D$	Diameter (in)
$l$	Length of Nail (in)
$F_{yb}$	Bending Yield Strength of Fastener (psi)
$t_m$	Thickness of Main Member (in)
$t_s$	Thickness of Side Member (in)
$l_s$	Dowel Bearing Length of Fastener in Side Member (in)
$l_m$	Dowel Bearing Length of Fastener in Main Member (in)
$P_L$	Toenail Penetration of Nail in Main Member (in)
$P$	Penetration Of Nail in Main Member (in)
$G_m$	Specific Gravity of Wood Member
$F_{em}$	Dowel Bearing Strength of Main Member (psi)
$F_{es}$	Dowel Bearing Strength of Side Member (psi)
$K_D$	Reduction Factor for Fasteners with $D < 1/4"$
$\theta$	Maximum Angle of Load to Grain for Any Member in Connection ( $0 < \theta < 90$ )
$R_e$	$\frac{F_{em}}{F_{es}}$
$R_t$	$\frac{l_m}{l_s}$
$k_1$	$\frac{\sqrt{R_e + 2R_e^2(1 + R_t + R_t^2) + R_t^2R_e^3} - R_e(1 + R_t)}{1 + R_e}$
$k_2$	$-1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}l_m^2}}$
$k_3$	$-1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(2 + R_e)D^2}{3F_{em}l_s^2}}$
$K_\theta$	$1 + \frac{\theta}{360}$

**Rim Joist to Wall Plate**

D = 0.148	Mode I <sub>m</sub> :	
l = 3.00	$(D * l_m * F_{em}) / K_D$	360 lbs
F <sub>yb</sub> = 90,000		
t <sub>m</sub> = 3.00	Mode I <sub>s</sub> :	
t <sub>s</sub> = 1.50	$(D * l_s * F_{es}) / K_D$	225 lbs
l <sub>s</sub> = 1.00		
l <sub>m</sub> = 1.60	Mode II:	
P <sub>L</sub> = 1.60	$(k_1 * D * l_s * F_{es}) / K_D$	127 lbs
P = 1.50		
G <sub>m</sub> = 0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> = 3,350	$(k_2 * D * l_m * F_{em}) / [(1 + 2R_e) K_D]$	134 lbs
F <sub>es</sub> = 3,350		
K <sub>D</sub> = 2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> = 1.00	$(k_3 * D * l_s * F_{em}) / [(2 + R_e) K_D]$	96 lbs
1 + R <sub>e</sub> = 2.00		
1 + 2R <sub>e</sub> = 3.00	Mode IV:	
2 + R <sub>e</sub> = 3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1 + R_e)]$	100 lbs
R <sub>t</sub> = 1.60		
k <sub>1</sub> = 0.56	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> = 1.11	Smallest Value from Modes I-IV	96 lbs
k <sub>3</sub> = 1.28		
	Adjusted Design Value for Single Fastener: Z'	
	$Z(C_D)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	139.9 lbs
	Req'd Number of Nails: N	
	Load/Z'	57.7
	Req'd Spacing:	
	L/N	3.3 in

**Use 10d Common Nails @ 3" o.c. for all Rim Joist to Plate Connections, Toenailed**

**Ceiling Joist to Wall Plate**

D =	0.162	Mode I <sub>m</sub> :	
l =	3.50	$(D * l_m * F_{em}) / K_D$	493 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	3.00	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	$(D * l_s * F_{es}) / K_D$	288 lbs
l <sub>s</sub> =	1.17		
l <sub>m</sub> =	2.00	Mode II:	
P <sub>L</sub> =	1.86	$(k_1 * D * l_s * F_{es}) / K_D$	171 lbs
P =	2.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	$(k_2 * D * l_m * F_{em}) / [(1+2R_e)K_D]$	179 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	$(k_3 * D * l_s * F_{em}) / [(2+R_e)K_D]$	119 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1+R_e)]$	120 lbs
R <sub>t</sub> =	1.71		
k <sub>1</sub> =	0.60	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.09	Smallest Value from Modes I-IV	119 lbs
k <sub>3</sub> =	1.24		
		Adjusted Design Value for Single Fastener: Z'	
		$Z(C_D)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	174.3 lbs
		Req'd Number of Nails: N	
		Load/Z'	3.4

**Use 4 16d Common Nails for all Floor Joist to Plate Connections, Toenailed**

**Top Plate Splice**

D = 0.162	Mode I <sub>m</sub> :	
l = 3.50	$(D * l_m * F_{em}) / K_D$	493 lbs
F <sub>yb</sub> = 90,000		
t <sub>m</sub> = 5.50	Mode I <sub>s</sub> :	
t <sub>s</sub> = 1.50	$(D * l_s * F_{es}) / K_D$	370 lbs
l <sub>s</sub> = 1.50		
l <sub>m</sub> = 2.00	Mode II:	
P <sub>L</sub> = NONE	$(k_1 * D * l_s * F_{es}) / K_D$	182 lbs
P = 2.00		
G <sub>m</sub> = 0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> = 3,350	$(k_2 * D * l_m * F_{em}) / [(1 + 2R_e) K_D]$	179 lbs
F <sub>es</sub> = 3,350		
K <sub>D</sub> = 2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> = 1.00	$(k_3 * D * l_s * F_{em}) / [(2 + R_e) K_D]$	142 lbs
1 + R <sub>e</sub> = 2.00		
1 + 2R <sub>e</sub> = 3.00	Mode IV:	
2 + R <sub>e</sub> = 3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1 + R_e)]$	120 lbs
R <sub>t</sub> = 1.33		
k <sub>1</sub> = 0.49	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> = 1.09	Smallest Value from Modes I-IV	120 lbs
k <sub>3</sub> = 1.15		
	Adjusted Design Value for Single Fastener: Z'	
	$Z(C_D)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	191 lbs
	Req'd Number of Nails: N	
	Load/Z'	17.9

**Use 18 16d Common Nail Between all Splice Points, Lap Splice**



**Band Joist to Sole Plate**

D =	0.162	Mode I <sub>m</sub> :	
l =	3.50	$(D * l_m * F_{em}) / K_D$	493 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	5.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	$(D * l_s * F_{es}) / K_D$	370 lbs
l <sub>s</sub> =	1.50		
l <sub>m</sub> =	2.00	Mode II:	
P <sub>L</sub> =	NONE	$(k_1 * D * l_s * F_{es}) / K_D$	182 lbs
P =	2.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	$(k_2 * D * l_m * F_{em}) / [(1 + 2R_e) K_D]$	179 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	$(k_3 * D * l_s * F_{em}) / [(2 + R_e) K_D]$	142 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1 + R_e)]$	120 lbs
R <sub>t</sub> =	1.33		
k <sub>1</sub> =	0.49	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.09	Smallest Value from Modes I-IV	120 lbs
k <sub>3</sub> =	1.15		
		Adjusted Design Value for Single Fastener: Z'	
		$Z(C_D)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	191 lbs
		Req'd Number of Nails: N	
		Load/Z'	42.23
			40.0
		Req'd Spacing:	
		L/N	4.8 in

**Use 16d Common Nail at 4" o.c. for all Sole Plate to Band Joist Connections, Face Nailed**

**Continuous Header to Stud**

D =	0.131	Mode I <sub>m</sub> :	
l =	2.50	$(D * l_m * F_{em}) / K_D$	266 lbs
F <sub>yb</sub> =	100,000		
t <sub>m</sub> =	1.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	$(D * l_s * F_{es}) / K_D$	166 lbs
l <sub>s</sub> =	0.83		
l <sub>m</sub> =	1.33	Mode II:	
P <sub>L</sub> =	1.33	$(k_1 * D * l_s * F_{es}) / K_D$	93 lbs
P =	1.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	$(k_2 * D * l_m * F_{em}) / [(1 + 2R_e) K_D]$	101 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	$(k_3 * D * l_s * F_{em}) / [(2 + R_e) K_D]$	74 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1 + R_e)]$	82 lbs
R <sub>t</sub> =	1.60		
k <sub>1</sub> =	0.56	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.14	Smallest Value from Modes I-IV	74 lbs
k <sub>3</sub> =	1.34		
		Adjusted Design Value for Single Fastener: Z'	
		$Z(C_D)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	99 lbs
		Req'd Number of Nails: N	
		Load/Z'	3.4

**Use 4 8d Common Nail for All Header to Stud Connection, Toenailed**

**Band Joist to Joists**

D = 0.162	Mode I <sub>m</sub> :	
l = 3.50	$(D * l_m * F_{em}) / K_D$	493 lbs
F <sub>yb</sub> = 90,000		
t <sub>m</sub> = 3.00	Mode I <sub>s</sub> :	
t <sub>s</sub> = 1.50	$(D * l_s * F_{es}) / K_D$	288 lbs
l <sub>s</sub> = 1.17		
l <sub>m</sub> = 2.00	Mode II:	
P <sub>L</sub> = 1.86	$(k_1 * D * l_s * F_{es}) / K_D$	171 lbs
P = 2.00		
G <sub>m</sub> = 0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> = 3,350	$(k_2 * D * l_m * F_{em}) / [(1 + 2R_e) K_D]$	179 lbs
F <sub>es</sub> = 3,350		
K <sub>D</sub> = 2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> = 1.00	$(k_3 * D * l_s * F_{em}) / [(2 + R_e) K_D]$	119 lbs
1 + R <sub>e</sub> = 2.00		
1 + 2R <sub>e</sub> = 3.00	Mode IV:	
2 + R <sub>e</sub> = 3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1 + R_e)]$	120 lbs
R <sub>t</sub> = 1.71		
k <sub>1</sub> = 0.60	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> = 1.09	Smallest Value from Modes I-IV	119 lbs
k <sub>3</sub> = 1.24		
	Adjusted Design Value for Single Fastener: Z'	
	$Z(C_D)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	210 lbs
	Req'd Number of Nails: N	
	Load/Z'	2.8

**Use 3 16d Common Nails for all Floor Joist to Plate Connections, Face Nailed**

**Stud to Wall Plate**

D =	0.162	Mode I <sub>m</sub> :	
l =	3.50	(D * I <sub>m</sub> * F <sub>em</sub> ) / K <sub>D</sub>	493 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	3.00	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	(D * I <sub>s</sub> * F <sub>es</sub> ) / K <sub>D</sub>	288 lbs
l <sub>s</sub> =	1.17		
l <sub>m</sub> =	2.00	Mode II:	
P <sub>L</sub> =	1.86	(k <sub>1</sub> * D * I <sub>s</sub> * F <sub>es</sub> ) / K <sub>D</sub>	171 lbs
P =	2.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	(k <sub>2</sub> * D * I <sub>m</sub> * F <sub>em</sub> ) / [(1+2R <sub>e</sub> )K <sub>D</sub> ]	179 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	(k <sub>3</sub> * D * I <sub>s</sub> * F <sub>em</sub> ) / [(2+R <sub>e</sub> )K <sub>D</sub> ]	119 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	D <sup>2</sup> / K <sub>D</sub> * Sqrt[(2 * F <sub>em</sub> * F <sub>yb</sub> ) / 3 * (1+R <sub>e</sub> )]	120 lbs
R <sub>t</sub> =	1.71		
k <sub>1</sub> =	0.60	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.09	Smallest Value from Modes I-IV	126 lbs
k <sub>3</sub> =	1.24		
		Adjusted Design Value for Single Fastener: Z'	
		Z(C <sub>D</sub> )(C <sub>M</sub> )(C <sub>t</sub> )(C <sub>eg</sub> )(C <sub>di</sub> )(C <sub>tn</sub> )	200.8 lbs
		Load on One Stud Connection:	
		Load/Stud Spacing	263.2 lbs
		Req'd Number of Nails: N	
		Load/Z'	1.3

<b>Use 2 16d Common Nails for all Stud to Plate Connections, End Nailed</b>
---

**Floor Joist to Wall Plate**

D =	0.162	Mode I <sub>m</sub> :	
l =	3.50	(D * I <sub>m</sub> * F <sub>em</sub> ) / K <sub>D</sub>	493 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	3.00	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	(D * I <sub>s</sub> * F <sub>es</sub> ) / K <sub>D</sub>	288 lbs
l <sub>s</sub> =	1.17		
l <sub>m</sub> =	2.00	Mode II:	
P <sub>L</sub> =	1.86	(k <sub>1</sub> * D * I <sub>s</sub> * F <sub>es</sub> ) / K <sub>D</sub>	171 lbs
P =	2.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	(k <sub>2</sub> * D * I <sub>m</sub> * F <sub>em</sub> ) / [(1+2R <sub>e</sub> )K <sub>D</sub> ]	179 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	(k <sub>3</sub> * D * I <sub>s</sub> * F <sub>em</sub> ) / [(2+R <sub>e</sub> )K <sub>D</sub> ]	119 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	D <sup>2</sup> / K <sub>D</sub> * Sqrt[(2 * F <sub>em</sub> * F <sub>yb</sub> ) / 3 * (1+R <sub>e</sub> )]	120 lbs
R <sub>t</sub> =	1.71		
k <sub>1</sub> =	0.60	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.09	Smallest Value from Modes I-IV	119 lbs
k <sub>3</sub> =	1.24		
		Adjusted Design Value for Single Fastener: Z'	
		Z(C <sub>D</sub> )(C <sub>M</sub> )(C <sub>t</sub> )(C <sub>eg</sub> )(C <sub>di</sub> )(C <sub>tn</sub> )	174.3 lbs
		Req'd Number of Nails: N	
		Load/Z'	3.2

**Use 4 16d Common Nails for all Floor Joist to Plate Connections, Toenailed**

**Double Studs**

D =	0.148	Mode I <sub>m</sub> :	
l =	3.00	(D * l <sub>m</sub> * F <sub>em</sub> )/K <sub>D</sub>	338 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	1.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	(D * l <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	338 lbs
l <sub>s</sub> =	1.50		
l <sub>m</sub> =	1.50	Mode II:	
P <sub>L</sub> =	NONE	(k <sub>1</sub> * D * l <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	140 lbs
P =	1.50		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	(k <sub>2</sub> * D * l <sub>m</sub> * F <sub>em</sub> )/[(1+2R <sub>e</sub> )K <sub>D</sub> ]	127 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	(k <sub>3</sub> * D * l <sub>s</sub> * F <sub>em</sub> )/[(2+R <sub>e</sub> )K <sub>D</sub> ]	127 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	D <sup>2</sup> /K <sub>D</sub> * Sqrt[(2 * F <sub>em</sub> * F <sub>yb</sub> )/3 * (1+R <sub>e</sub> )]	100 lbs
R <sub>t</sub> =	1.00		
k <sub>1</sub> =	0.41	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.13	Smallest Value from Modes I-IV	100 lbs
k <sub>3</sub> =	1.13		
		Adjusted Design Value for Single Fastener: Z'	
		Z(C <sub>D</sub> )(C <sub>M</sub> )(C <sub>t</sub> )(C <sub>eg</sub> )(C <sub>di</sub> )(C <sub>tn</sub> )	160 lbs
		Load on One Stud Connection, P:	337.8 lbs
		Req'd Number of Nails: N	
		Load/Z'	2.1
			3
		Req'd Spacing:	
		L/N	16.0 in

<b>Use 10d Common Nail at 16" o.c. for all Double Stud Connections, Face Nailed</b>
---

**Continued Header**

D =	0.148	Mode I <sub>m</sub> :	
l =	3.00	(D * l <sub>m</sub> * F <sub>em</sub> ) / K <sub>D</sub>	338 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	1.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	(D * l <sub>s</sub> * F <sub>es</sub> ) / K <sub>D</sub>	338 lbs
l <sub>s</sub> =	1.50		
l <sub>m</sub> =	1.50	Mode II:	
P <sub>L</sub> =	NONE	(k <sub>1</sub> * D * l <sub>s</sub> * F <sub>es</sub> ) / K <sub>D</sub>	140 lbs
P =	1.50		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	(k <sub>2</sub> * D * l <sub>m</sub> * F <sub>em</sub> ) / [(1 + 2R <sub>e</sub> ) K <sub>D</sub> ]	127 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	(k <sub>3</sub> * D * l <sub>s</sub> * F <sub>em</sub> ) / [(2 + R <sub>e</sub> ) K <sub>D</sub> ]	127 lbs
1 + R <sub>e</sub> =	2.00		
1 + 2R <sub>e</sub> =	3.00	Mode IV:	
2 + R <sub>e</sub> =	3.00	D <sup>2</sup> / K <sub>D</sub> * Sqrt[(2 * F <sub>em</sub> * F <sub>yb</sub> ) / 3 * (1 + R <sub>e</sub> )]	100 lbs
R <sub>t</sub> =	1.00		
k <sub>1</sub> =	0.41	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.13	Smallest Value from Modes I-IV	100 lbs
k <sub>3</sub> =	1.13		
		Adjusted Design Value for Single Fastener: Z'	
		Z(C <sub>D</sub> )(C <sub>M</sub> )(C <sub>t</sub> )(C <sub>eg</sub> )(C <sub>di</sub> )(C <sub>tn</sub> )	160 lbs
		Req'd Number of Nails: N	
		Load / Z'	4.41
			5
		Req'd Spacing:	
		L / N	7.2 in

**Use 10d Common Nail at 7" o.c. for all Sole Plate to Band Joist Connections, Face Nailed**

**Rafter to Ridge Beam/Jack Rafters to Hip Rafters**

D =	0.162	Mode I <sub>m</sub> :	
l =	3.50	(D * l <sub>m</sub> * F <sub>em</sub> )/K <sub>D</sub>	460 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	1.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	(D * l <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	288 lbs
l <sub>s</sub> =	1.17		
l <sub>m</sub> =	1.86	Mode II:	
P <sub>L</sub> =	1.86	(k <sub>1</sub> * D * l <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	162 lbs
P =	2.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	(k <sub>2</sub> * D * l <sub>m</sub> * F <sub>em</sub> )/[(1+2R <sub>e</sub> )K <sub>D</sub> ]	168 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	(k <sub>3</sub> * D * l <sub>s</sub> * F <sub>em</sub> )/[(2+R <sub>e</sub> )K <sub>D</sub> ]	119 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	D <sup>2</sup> /K <sub>D</sub> * Sqrt[(2 * F <sub>em</sub> * F <sub>yb</sub> )/3 * (1+R <sub>e</sub> )]	120 lbs
R <sub>t</sub> =	1.60		
k <sub>1</sub> =	0.56	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.10	Smallest Value from Modes I-IV	119 lbs
k <sub>3</sub> =	1.24		
		Adjusted Design Value for Single Fastener: Z'	
		Z(C <sub>D</sub> )(C <sub>M</sub> )(C <sub>t</sub> )(C <sub>eg</sub> )(C <sub>di</sub> )(C <sub>tn</sub> )	158 lbs
		Req'd Number of Nails: N	
		Load/Z'	4.1

**Use 5 16d Common Nail for Rafer to Ridge Beam/ Jack Rafter to Hip Rafter Connections, Toenailed**



**Sole Plate to Foundation**

D =	0.75	Mode I <sub>m</sub> :	
l =	5.00	(D * l <sub>m</sub> * F <sub>em</sub> ) / 4K <sub>θ</sub>	4,922 lbs
F <sub>yb</sub> =	45,000		
t <sub>m</sub> =	3.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	(D * l <sub>s</sub> * F <sub>es</sub> ) / 4K <sub>θ</sub>	2,109 lbs
l <sub>s</sub> =	1.50		
l <sub>m</sub> =	3.50	Mode II:	
θ =	0.00	(k <sub>1</sub> * D * l <sub>s</sub> * F <sub>es</sub> ) / 3.6K <sub>θ</sub>	1,671 lbs
P =	3.50		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	7,500	(k <sub>2</sub> * D * l <sub>m</sub> * F <sub>em</sub> ) / [3.2(1+2R <sub>e</sub> )K <sub>θ</sub> ]	2,466 lbs
F <sub>es</sub> =	4,704		
K <sub>θ</sub> =	1.00	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.59	(k <sub>3</sub> * D * l <sub>s</sub> * F <sub>em</sub> ) / [3.2(2+R <sub>e</sub> )K <sub>θ</sub> ]	1,186 lbs
1+R <sub>e</sub> =	2.59		
1+2R <sub>e</sub> =	4.19	Mode IV:	
2+R <sub>e</sub> =	3.59	D <sup>2</sup> / 3.2K <sub>θ</sub> * Sqrt[(2 * F <sub>em</sub> * F <sub>yb</sub> ) / 3 * (1+R <sub>e</sub> )]	1,637 lbs
R <sub>t</sub> =	2.33		
k <sub>1</sub> =	1.14	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.44	Smallest Value from Modes I-IV	1,186 lbs
k <sub>3</sub> =	1.62		
		Adjusted Design Value for Single Fastener: Z'	
		Z(C <sub>D</sub> )(C <sub>M</sub> )(C <sub>t</sub> )(C <sub>g</sub> )(C <sub>Δ</sub> )	1,898
		Req'd Number of Anchor Bolts: N	
		Load/Z'	9.22
		Req'd Number of Anchor Bolts: N	
		Load/Z'	5.90

<p><b>Use Ten (10) 3/4" Diameter Anchor Bolts for Sole Plate to Foundation Connections for 50 ft Walls</b></p> <p><b>AND</b></p> <p><b>Six (6) 3/4" Diameter Anchors Bolts for 32 ft Walls</b></p>
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# ASD Fastener Schedule

<b><u>Connection</u></b>	<b><u>Fastening</u></b>	<b><u>Location</u></b>
Rim Joist to Wall Plate	10d Common Nails at 3" o.c.	Toenail
Ceiling Joist to Wall Plate	4 - 16d Common Nails	Toenail
Top Plate Splice	18 - 16d Common Nails	Lap Splice
Band Joist to Sole Plate	16d Common Nails at 4" o.c.	Face Nail
Continuous Header to Stud	4 - 8d Common Nails	Toenail
Band Joist to Joists	3 - 16d Common Nails	Face Nail
Stud to Wall Plate	2 - 16d Common Nails	End Nail
Floor Joist to Wall Plate	4 - 16d Common Nails	Toenail
Double Studs	10d Common Nails at 16" o.c.	Face Nail
Continued Header	10d Common Nails at 7" o.c.	Face Nail
Rafter to Ridge Beam/Jack Rafters to Hip Rafters	5 - 16d Common Nails	Toenail
Sole Plate to Foundation	10 - 3/4" Diameter Anchor Bolts for Long Walls 6 - 3/" Diameter Anchor Bolts for Short Walls	Face Nail

# LRFD LOADS

# Loads

## Gravity Loads

### **Roof Dead Load: D**

Asphalt Shingles =	2.50 psf
19/32 -in. Wood Sheathing (3.7 psf x 15/32 in.) =	2.20 psf
Waterproofing Membrane (Single Ply Sheet) =	0.60 psf
4 in. Polystyrene Foam Insulation (0.2 psf x 4 in.) =	0.80 psf
Framing (2 x 12 @ 16 in o.c.) =	2.90 psf
Flashing (Copper/Tin) =	1.00 psf
	<hr/>
	10.00 psf

**Roof Dead Load: D = 10.00 psf**

### **Wall Dead Load: D**

2 x 6 @ 16-in., 5/8-in. gypsum, insulated, 15/32-in. siding =	12.00 psf
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**Wall Dead Load: D = 12.00 psf**

### **Floor Dead Load: D**

Framing (2 x 12 @ 16 in o.c.) =	2.90 psf
Subflooring 3/4 in. =	3.00 psf
Hardwood Flooring =	4.00 psf
Ceiling (Gypsum Board 1/2 in.) =	2.20 psf
	<hr/>
	12.10 psf

**Floor Dead Load: D = 12.10 psf**

### **Roof Live Load: L<sub>r</sub>**

$L_0 R_1 R_2 =$	19.00 psf
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**Roof Live Load: L<sub>r</sub> = 19.00 psf**

### **Floor Live Load: L**

ASCE- 7 Table 4-1	40.00 psf
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**Floor Live Load: L = 40.00 psf**

### **Roof Snow Load: S**

$C_s(0.7)(C_e)(C_t)(I)(p_g) =$	18.90 psf
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**Roof Snow Load: S ≈ 19.00 psf**

**Roof Wind Load: W**

$$(\lambda)(K_{zt})(I)(p_{s30}) =$$

**23.00 psf**

**Roof Wind Load: W =**

**23.00 psf**

**Lateral Loads**

**Wind Load: W**

$$(\lambda)(K_{zt})(I)(p_{s30}) =$$

**35.00 psf**

**Wind Load: W =**

**35.00 psf**

See Appendix C for Necessary Tables and Figures

# LRFD Load Combinations

### Applicable Roof Load Combinations (LRFD)

1.4D	14.0 psf	Non Critical
1.2D + 1.6L <sub>r</sub> + 0.5S	51.9 psf	Non Critical
1.2D + 1.6S + 0.8W	60.8 psf	CRITICAL
1.2D + 1.6W + 0.5L <sub>r</sub> + 0.5S	67.8 psf	Non Critical

### Applicable Floor Load Combinations (LRFD)

1.4D	16.9 psf	Non Critical
1.2D + 1.6L	78.5 psf	CRITICAL

### Applicable Wall Load Combinations (LRFD)

#### Gravity Load Combination

1.4D	584 lb/ft	Non Critical
1.2D + 1.6L + 0.5S	1,088 lb/ft	CRITICAL
1.2D + 1.6S + 0.5L	903 lb/ft	Non Critical
1.2D + 0.5L + 0.5S	936 lb/ft	Non Critical

#### Lateral Load Combinations

0.8W	280 lb/ft	Non Critical
1.6W	560 lb/ft	Critical



# LRFD Roof Truss Members

### **Wood Properties**

Spruce-Pine-Fir	No.2
Bending ( $F_b$ )	875 psi
Tension Parallel to Grain ( $F_t$ )	450 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	1,150 psi
Modulus of Elasticity (E)	1,400,000 psi
Modulus of Elasticity ( $E_{min}$ )	510,000 psi

### **Factors**

$\lambda$ (Time Effect)	1.00
$\varphi_b$ (Bending Resistance)	0.85
$\varphi_t$ (Tension Resistance)	0.80
$\varphi_v$ (Shear Resistance)	0.75
$\varphi_c$ (Compression Resistance)	0.90
$\varphi_s$ (Stability Resistance)	0.85
$K_F$ (Format Conversion)	
Bending	2.54
Tension	2.70
Shear	2.88
Compression	
Perpendicular to Grain	2.08
Parallel to Grain	2.40
Stability	1.76
$C_F$ (Size)	
2 x 10	
Bending	1.10
Tension	1.10
Compression	1.00
2 x 12	
Bending	1.00
Tension	1.00
Compression	1.00
$C_i$ (Incising)	1.00
$C_L$ (Stability)	1.00
$C_M$ (Wet Service)	1.00
$C_r$ (Repetitive Member)	1.15
$C_t$ (Thermal)	1.00

### **LRFD Design Values**

Nominal Bending Design Value:  $F_{bn}$

$$F_b (K_F) = \begin{array}{r} 2,224 \text{ psi} \\ 2.22 \text{ ksi} \end{array}$$

Nominal Tension Parallel to Grain Design Value:  $F_{tn}$

$$F_t (K_F) = \begin{array}{r} 1,215 \text{ psi} \\ 1.22 \text{ ksi} \end{array}$$

Nominal Shear Design Value Parallel to Grain:  $F_{vn}$

$$F_v (K_F) = \begin{array}{r} 2,835 \text{ psi} \\ 2.84 \text{ ksi} \end{array}$$

Nominal Compression Perpendicular to Grain Design Value:  $F_{c \perp n}$

$$F_{c \perp} (K_F) = \begin{array}{r} 885 \text{ psi} \\ 0.89 \text{ ksi} \end{array}$$

Nominal Compression Parallel to Grain Design Value:  $F_{cn}$

$$F_c (K_F) = \begin{array}{r} 2,760 \text{ psi} \\ 2.76 \text{ ksi} \end{array}$$

Nominal Modulus of Elasticity Design Value:  $E_{min n}$

$$E_{min} (K_F) = \begin{array}{r} 900,000 \text{ psi} \\ 900 \text{ ksi} \end{array}$$

### **Wood Dimensions and Spacing**

Common Rafter Length: L 18.34 ft

Rafter Spacing: 16.00 in

Section Modulus: S

$$\begin{array}{r} 2 \times 6 = 7.56 \text{ in}^3 \\ 2 \times 8 = 14.06 \text{ in}^3 \\ 2 \times 10 = 22.56 \text{ in}^3 \\ 2 \times 12 = 33.06 \text{ in}^3 \end{array}$$

Area: A

$$\begin{array}{r} 2 \times 6 = 8.25 \text{ in}^2 \\ 2 \times 8 = 11.25 \text{ in}^2 \\ 2 \times 10 = 14.25 \text{ in}^2 \\ 2 \times 12 = 17.25 \text{ in}^2 \end{array}$$

Moment of Inertia: I

$$\begin{array}{r} 2 \times 6 = 20.80 \text{ in}^3 \\ 2 \times 8 = 52.73 \text{ in}^3 \\ 2 \times 10 = 107.17 \text{ in}^3 \\ 2 \times 12 = 190.11 \text{ in}^3 \end{array}$$

### **Loads**

Total Load:  $w_u$

$$(1.2D + 1.6W + 0.5L + 0.5S) \times \text{Rafter Spacing} = 90.4 \text{ lb/ft}$$

Shear:  $V_u$

$$(w_u \times L)/2 = 829 \text{ lb}$$

Moment:  $M_u$

$$(w_u \times L^2)/8 = 3,801 \text{ ft-lb}$$
$$45.6 \text{ in-k}$$

### **Bending**

Adjusted Bending Design Value:  $F'_{bn}$

$$F_{bn} (\phi_b)(\lambda)(C_M)(C_t)(C_L)(C_F)(C_r)(C_i) = 2,630 \text{ psi}$$
$$2.63 \text{ ksi}$$

Required Section Modulus: Req'd S

$$M_u/F'_{bn} = 17.34 \text{ in}^3 \quad \Rightarrow \quad \text{Try 2x10}$$

Adjusted Moment Resistance:  $M'_n$

$$F'_{bn} * S = 50.2 \text{ in-k}$$

$$M'_n > M_u \quad 50.2 > 45.6 \quad \text{TRUE}$$

### **Shear**

Adjusted Shear Design Value Parallel to Grain:  $F'_{vn}$

$$F_{vn} (\phi_v)(\lambda)(C_M)(C_t)(C_i) = 321 \text{ psi}$$
$$0.32 \text{ ksi}$$

Adjusted Shear Resistance Parallel to Grain:  $V'_n$

$$2/3(F'_{vn})(A) = 3.05 \text{ k}$$

$$V'_n > V_u \quad 3,047 > 829 \quad \text{TRUE}$$

### **Deflection**

Adjusted Modulus of Elasticity:  $E'$

$$E(C_M)(C_t)(C_i) = 1,400,000 \text{ psi}$$

Actual Deflection Under Snow Load:  $\Delta_s$

$$5w_s L^4 / 384E'I = 0.43 \text{ in}$$

Allowable Deflection Under Snow Load: Allow.  $\Delta_s$

$$L/240 = 0.92 \text{ in}$$

$$\text{Allow. } \Delta_s > \Delta_s \quad 0.92 > 0.43 \quad \text{TRUE}$$

Actual Deflection Under Total Load:  $\Delta_u$

$$\Delta_s(w_u/w_s) = 1.53 \text{ in}$$

Allowable Deflection: Allow.  $\Delta_u$

$$L/180 = 1.22 \text{ in}$$

$$\text{Allow. } \Delta_u > \Delta_u \quad 1.22 > 1.53 \quad \text{FAIL}$$

### **Bending**

Required Section Modulus: Req'd S

$$M_u/F'_{bn} = 17.3 \text{ in}^3 \quad \Rightarrow \quad \text{Try 2x12}$$

Adjusted Moment Resistance:  $M'_n$

$$F'_{bn} * S = 50.2 \text{ in-k}$$

$$M'_n > M_u \quad 50.2 > 45.6 \quad \text{TRUE}$$

### **Shear**

Adjusted Shear Resistance Parallel to Grain:  $V'_n$

$$2/3(F'_{vn})(A) = 3.69 \text{ k}$$

$$V'_n > V_u \quad 3,689 > 829 \quad \text{TRUE}$$

### **Deflection**

Adjusted Modulus of Elasticity:  $E'$

$$E(C_M)(C_t)(C_f) = 1,400,000 \text{ psi}$$

Actual Deflection Under Snow Load:  $\Delta_s$

$$5w_s L^4 / 384E'I = 0.24 \text{ in}$$

Allowable Deflection Under Snow Load: Allow.  $\Delta_s$

$$L/240 = 0.92 \text{ in}$$

$$\text{Allow. } \Delta_s > \Delta_s \quad 0.92 > 0.24 \quad \text{TRUE}$$

Actual Deflection Under Total Load:  $\Delta_u$

$$\Delta_s(w_u/w_s) = 0.86 \text{ in}$$

Allowable Deflection: Allow.  $\Delta_u$

$$L/180 = 1.22 \text{ in}$$

Allow.  $\Delta_u > \Delta_u$

1.22 > 0.86

TRUE

**Use 2 x 12 SPF No. 2 or Better for All Roof Truss Members**  
**MC ≤ 19%**

# LRFD Roof Sheathing

### **Loads**

Total Load:  $w_u$

$$1.2D + 1.6W + 0.5L + 0.5S = 67.8 \text{ psf}$$

Applied Load:  $w_{AL}$

$$1.6W + 0.5L + 0.5S = 55.8 \text{ psf}$$

### **Factors**

$\lambda$  (Time Effect) 0.80

$\varphi_b$  (Bending Resistance) 0.85

$\varphi_v$  (Shear Resistance) 0.75

$\varphi_D$  (Resistance) 0.80

$K_F$  (Format Conversion)

Bending 2.54

Shear 2.88

$C_G$  (Grade and Construction)

Stiffness

Perpendicular to Joists 1.10

Parallel to Joists 1.00

Bending

Perpendicular to Joists 1.00

Parallel to Joists 1.00

Shear

Perpendicular to Joists 1.00

Parallel to Joists 2.80

$C_{SA}$  (Span Adjustment)

3-Span to 1-Span

Stiffness 0.53

Bending 0.80

Shear 1.20

$G_s$  (Specific Gravity) 0.92

### **Strength Axis Perpendicular to Joists**

Span Rating: 24/0

Plywood Type: 3-Ply

Applied Load:  $w_{AL}$

$$1.6W + 0.5L + 0.5S = 55.8 \text{ psf}$$

Allowable Uniform Applied Load on APA Rated Plywood

Sheathing Across 16" o.c. Supports:  $w_{AL}$  147 psf



Adjusted Allowable Uniform Applied Load for 3-Ply Plywood Sheathing:  $W'_{AL}$

$$W_{AL}(C_G)(C_{SA}) = 85.7 \text{ psf}$$

$$W'_{AL} > w_{AL} \quad 85.7 > 55.8 \quad \text{TRUE}$$

Total Load:  $w_u$

$$1.2D + 1.6W + 0.5L + 0.5S = 67.8 \text{ psf}$$

Allowable Uniform Load on APA Rated Plywood Sheathing Across 16" o.c. Supports:  $W_u$

$$196 \text{ psf}$$

Adjusted Allowable Uniform Load for 3-Ply Plywood Sheathing:  $W'_u$

$$W_u(C_G)(C_{SA}) = 114 \text{ psf}$$

$$W'_u > w_u \quad 114 > 67.8 \quad \text{TRUE}$$

Actual Bending Stress:  $f_b$

$$1.2D + 1.6W + 0.5L + 0.5S = 67.8 \text{ psf}$$

Allowable Bending Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports:  $F_b$

$$117 \text{ psf}$$

Adjusted Allowable Bending Stress for 3-Ply Plywood Sheathing:  $F'_b$

$$F_b(K_F)(\lambda)(\phi_b)(C_G)(C_{SA}) = 162 \text{ psf}$$

$$F'_b > f_b \quad 162 > 67.8 \quad \text{TRUE}$$

Actual Shear Stress:  $f_v$

$$1.2D + 1.6W + 0.5L + 0.5S = 67.8 \text{ psf}$$

Allowable Shear Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports:  $F_v$

$$228 \text{ psf}$$

Adjusted Allowable Shear Stress for 3-Ply Plywood Sheathing:  $F'_v$

$$F_v(K_F)(\lambda)(\phi_v)(C_G)(C_{SA}) = 473 \text{ psf}$$

$$F'_v > f_v \quad 473 > 67.8 \quad \text{TRUE}$$

**APA Rated 24/0 3-Ply Plywood OK for Roof Sheathing Laid with Strength Axis Perpendicular to Joists**

**Strength Axis Parallel to Joists**

Span Rating:	24/0		
Plywood Type:	3-Ply		
Applied Load: $w_{AL}$ $1.6W + 0.5L + 0.5S =$	55.8 psf		
Allowable Uniform Applied Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{AL}$	9.00 psf		
Adjusted Allowable Uniform Applied Load for 3-Ply Plywood Sheathing: $W'_{AL}$ $W_{AL}(C_G)(C_{SA}) =$	4.77 psf		
$W'_{AL} > w_{AL}$	4.77	< 55.8	FAIL
Total Load: $w_u$ $1.2D + 1.6W + 0.5L + 0.5S =$	67.8 psf		
Allowable Uniform Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_u$	12.0 psf		
Adjusted Allowable Uniform Load for 3-Ply Plywood Sheathing: $W'_u$ $W_u(C_G)(C_{SA}) =$	6.36 psf		
$W'_u > w_u$	6.36	> 67.8	FAIL
Actual Bending Stress: $f_b$ $1.2D + 1.6W + 0.5L + 0.5S =$	67.8 psf		
Allowable Bending Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_b$	25.0 psf		
Adjusted Allowable Bending Stress for 3-Ply Plywood Sheathing: $F'_b$ $F_b (K_F)(\lambda)(\phi_b)(C_G)(C_{SA}) =$	34.5 psf		
$F'_b > f_b$	34.5	> 67.8	FAIL
Actual Shear Stress: $f_v$ $1.2D + 1.6W + 0.5L + 0.5S =$	67.8 psf		

Allowable Shear Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_v$	145 psf		
Adjusted Allowable Shear Stress for 3-Ply Plywood Sheathing: $F'_v$			
$F_v (K_F)(\lambda)(\phi_v)(C_G)(C_{SA}) =$	842 psf		
$F'_v > f_v$	842	>	67.8 TRUE
Span Rating:	32 /16		
Plywood Type:	3-Ply		
Applied Load: $w_{AL}$			
$1.6W + 0.5L + 0.5S =$	55.8 psf		
Allowable Uniform Applied Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{AL}$	20.0 psf		
Adjusted Allowable Uniform Applied Load for 3-Ply Plywood Sheathing: $W'_{AL}$			
$W_{AL}(C_G)(C_{SA}) =$	10.6 psf		
$W'_{AL} > w_{AL}$	10.6	>	55.8 FAIL
Total Load: $w_u$			
$1.2D + 1.6W + 0.5L + 0.5S =$	67.8 psf		
Allowable Uniform Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_u$	27.0 psf		
Adjusted Allowable Uniform Load for 3-Ply Plywood Sheathing: $W'_u$			
$W_u(C_G)(C_{SA}) =$	14.3 psf		
$W'_u > w_u$	14.3	>	67.8 FAIL
Actual Bending Stress: $f_b$			
$1.2D + 1.6W + 0.5L + 0.5S =$	67.8 psf		
Allowable Bending Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_b$	43.0 psf		

Adjusted Allowable Bending Stress for 3-Ply Plywood

Sheathing:  $F'_b$

$$F_b (K_F)(\lambda)(\phi_b)(C_G)(C_{SA}) = 59.4 \text{ psf}$$

$$F'_b > f_b \quad 59.4 > 67.8 \quad \text{FAIL}$$

Actual Shear Stress:  $f_v$

$$1.2D + 1.6W + 0.5L + 0.5S = 67.8 \text{ psf}$$

Allowable Shear Stress for APA Rated Plywood Sheathing  
Across 16" o.c. Supports:  $F_v$

$$179 \text{ psf}$$

Adjusted Allowable Shear Stress for 3-Ply Plywood

Sheathing:  $F'_v$

$$F_v (K_F)(\lambda)(\phi_v)(C_G)(C_{SA}) = 1039 \text{ psf}$$

$$F'_v > f_v \quad 1039 > 67.8 \quad \text{TRUE}$$

Span Rating: 40 /20

Plywood Type: 3-Ply

Applied Load:  $w_{AL}$

$$1.6W + 0.5L + 0.5S = 55.8 \text{ psf}$$

Allowable Uniform Applied Load on APA Rated Plywood  
Sheathing Across 16" o.c. Supports:  $W_{AL}$

$$44.0 \text{ psf}$$

Adjusted Allowable Uniform Applied Load for 3-Ply

Plywood Sheathing:  $W'_{AL}$

$$W_{AL}(C_G)(C_{SA}) = 23.3 \text{ psf}$$

$$W'_{AL} > w_{AL} \quad 23.3 > 55.8 \quad \text{FAIL}$$

Total Load:  $w_u$

$$1.2D + 1.6W + 0.5L + 0.5S = 67.8 \text{ psf}$$

Allowable Uniform Load on APA Rated Plywood  
Sheathing Across 16" o.c. Supports:  $W_u$

$$59.0 \text{ psf}$$

Adjusted Allowable Uniform Load for 3-Ply Plywood

Sheathing:  $W'_u$

$$W_u(C_G)(C_{SA}) = 31.3 \text{ psf}$$

$W'_u > w_u$	31.3 > 67.8	FAIL
Actual Bending Stress: $f_b$ $1.2D + 1.6W + 0.5L + 0.5S =$	67.8 psf	
Allowable Bending Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_b$	70.0 psf	
Adjusted Allowable Bending Stress for 3-Ply Plywood Sheathing: $F'_b$ $F_b (K_F)(\lambda)(\phi_b)(C_G)(C_{SA}) =$	96.7 psf	
$F'_b > f_b$	96.7 > 67.8	TRUE
Actual Shear Stress: $f_v$ $1.2D + 1.6W + 0.5L + 0.5S =$	67.8 psf	
Allowable Shear Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_v$	228 psf	
Adjusted Allowable Shear Stress for 3-Ply Plywood Sheathing: $F'_v$ $F_v (K_F)(\lambda)(\phi_v)(C_G)(C_{SA}) =$	1324 psf	
$F'_v > f_v$	1324 > 67.8	TRUE

**No APA Rated 3-Ply Plywood OK for Roof Sheathing Laid with Strength Axis Parallel to Joists**

**Thickness and Nailing**

3/8" Thick APA Rated 24/0 3-Ply Plywood Nailed Perpendicular to Joists with 6d Common Nails

Load Case:	Case 1
Maximum Nominal Unit Shear for Wind Loading on Douglas Fir Plywood: $V_w$	460 lb/ft
Adjusted Nominal Unit Shear for Wind Loading on SPF Plywood: $V'_w$ $G_s(V_w)(\phi_D) =$	339 lb/ft
Actual Shear: $v$ $1.6W =$	560 lb/ft

$V'_w > v$  339 > 560 FAIL

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19/32" Thick APA Rated 32/16 3-Ply Plywood Nailed Perpendicular to Joists with 10d Common Nails

Load Case: Case 1

Maximum Nominal Unit Shear for Wind Loading on Douglas  
Fir Plywood:  $V_w$  800 lb/ft

Adjusted Nominal Unit Shear for Wind Loading on SPF  
Plywood:  $V'_w$   
 $G_s(V_w)(\phi_D) =$  589 lb/ft

Actual Shear:  $v$   
 $1.6W =$  560 lb/ft

$V'_w > v$  589 > 560 TRUE

**Use 19/32" APA Rated 32/16 3-Ply Plywood Panels or Better as Subroof  
Nailed Perpendicular to Supports with 10d Common Nails  
at 6" o.c. Supported Edges  
12" o.c. Field  
No Blocking Required**

# LRFD Floor Joists

### **Wood Properties**

Spruce-Pine-Fir	No.2
Bending ( $F_b$ )	875 psi
Tension Parallel to Grain ( $F_t$ )	450 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	1,150 psi
Modulus of Elasticity (E)	1,400,000 psi
Modulus of Elasticity ( $E_{min}$ )	510,000 psi

### **Factors**

$\lambda$ (Time Effect)	0.80
$\varphi_b$ (Bending Resistance)	0.85
$\varphi_t$ (Tension Resistance)	0.80
$\varphi_v$ (Shear Resistance)	0.75
$\varphi_c$ (Compression Resistance)	0.90
$\varphi_s$ (Stability Resistance)	0.85
$K_F$ (Format Conversion)	
Bending	2.54
Tension	2.70
Shear	2.88
Compression	
Perpendicular to Grain	2.08
Parallel to Grain	2.40
Stability	1.76
$C_F$ (Size)	
Bending	1.00
Tension	1.00
Compression	1.00
$C_i$ (Incising)	1.00
$C_L$ (Stability)	1.00
$C_M$ (Wet Service)	1.00
$C_r$ (Repetitive Member)	1.15
$C_t$ (Thermal)	1.00

### **LRFD Design Values**

Nominal Bending Design Value:  $F_{bn}$

$$F_b (K_F) = \begin{matrix} 2,224 \text{ psi} \\ 2.22 \text{ ksi} \end{matrix}$$

Nominal Tension Parallel to Grain Design Value:  $F_{tn}$

$$F_t (K_F) = \begin{matrix} 1,215 \text{ psi} \\ 1.22 \text{ ksi} \end{matrix}$$

Nominal Shear Design Value Parallel to Grain:  $F_{vn}$

$$F_v (K_F) = \begin{matrix} 389 \text{ psi} \\ 0.39 \text{ ksi} \end{matrix}$$

Nominal Compression Perpendicular to Grain Design



Value:  $F_{c \perp n}$

$$F_{c \perp}(K_F) = \begin{array}{l} 885 \text{ psi} \\ 0.89 \text{ ksi} \end{array}$$

Nominal Compression Parallel to Grain Design Value:  $F_{cn}$

$$F_c(K_F) = \begin{array}{l} 2,760 \text{ psi} \\ 2.76 \text{ ksi} \end{array}$$

Nominal Modulus of Elasticity Design Value:  $E_{min n}$

$$E_{min n}(K_F) = \begin{array}{l} 900,000 \text{ psi} \\ 900 \text{ ksi} \end{array}$$

### **Wood Dimensions and Spacing**

Floor Joist Length: L 16.00 ft

Joist Spacing: 16.00 in

Section Modulus: S

$$\begin{array}{l} 2 \times 6 = 7.56 \text{ in}^3 \\ 2 \times 8 = 14.06 \text{ in}^3 \\ 2 \times 10 = 22.56 \text{ in}^3 \\ 2 \times 12 = 33.06 \text{ in}^3 \end{array}$$

Area: A

$$\begin{array}{l} 2 \times 6 = 8.25 \text{ in}^2 \\ 2 \times 8 = 11.25 \text{ in}^2 \\ 2 \times 10 = 14.25 \text{ in}^2 \\ 2 \times 12 = 17.25 \text{ in}^2 \end{array}$$

Moment of Inertia: I

$$\begin{array}{l} 2 \times 6 = 20.80 \text{ in}^3 \\ 2 \times 8 = 52.73 \text{ in}^3 \\ 2 \times 10 = 107.17 \text{ in}^3 \\ 2 \times 12 = 190.11 \text{ in}^3 \end{array}$$

### **Loads**

Total Load:  $w_u$

$$(1.2D + 1.6L) \times \text{Joist Spacing} = 105 \text{ lb/ft}$$

Shear:  $V_u$

$$(w_u \times L)/2 = 838 \text{ lb}$$

Moment:  $M_u$

$$(w_u \times L^2)/8 = \begin{array}{l} 3350 \text{ ft-lb} \\ 40.2 \text{ in-k} \end{array}$$

**Bending**

Adjusted Bending Design Value:  $F'_{bn}$

$$F_{bn}(\phi_b)(\lambda)(C_M)(C_t)(C_L)(C_F)(C_r)(C_i) = \begin{matrix} 1739 \text{ psi} \\ 1.74 \text{ ksi} \end{matrix}$$

Required Section Modulus: Req'd S

$$M_u / F'_{bn} = 23.12 \text{ in}^3 \quad \Rightarrow \quad \text{Try 2x12}$$

Adjusted Moment Resistance:  $M'_n$

$$F'_{bn} * S = 57.49 \text{ in-k}$$

$$M'_n > M_u \quad 57.49 > 40.20 \quad \text{TRUE}$$

**Shear**

Adjusted Shear Design Value Parallel to Grain:  $F'_{vn}$

$$F_{vn}(\phi_v)(\lambda)(C_M)(C_t)(C_i) = \begin{matrix} 233 \text{ psi} \\ 0.233 \text{ ksi} \end{matrix}$$

Adjusted Shear Resistance Parallel to Grain:  $V'_n$

$$2/3 F'_{vn} A = \begin{matrix} 2,216 \text{ lb} \\ 2.22 \text{ k} \end{matrix}$$

$$V'_n > V_u \quad 2,216 > 838 \quad \text{TRUE}$$

**Deflection**

Adjusted Modulus of Elasticity:  $E'$

$$E(C_M)(C_t)(C_i) = 1,400,000 \text{ psi}$$

Actual Deflection Under Live Load:  $\Delta_L$

$$5w_L L^4 / 384E'I = 0.39 \text{ in}$$

Allowable Deflection Under Snow Load: Allow.  $\Delta_L$

$$L/360 = 0.53 \text{ in}$$

$$0.53 > 0.39 \quad \text{TRUE}$$

Actual Deflection Under Total Load:  $\Delta_u$

$$\Delta_L (w_u / w_L) = 0.77 \text{ in}$$

Allowable Deflection: Allow.  $\Delta_u$

$$L/240 = 0.80 \text{ in}$$

$$0.80 > 0.77 \quad \text{TRUE}$$

**Use 2 x 12 SPF No. 2 or Better for All Floor Joists  
MC ≤ 19%**

# LRFD Diaphragm Chords & Struts

### **Wood Properties**

Spruce-Pine-Fir	No.2
Bending ( $F_b$ )	875 psi
Tension Parallel to Grain ( $F_t$ )	450 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	1,150 psi
Modulus of Elasticity (E)	1,400,000 psi
Modulus of Elasticity ( $E_{min}$ )	510,000 psi

### **Factors**

$\phi_b$ (Bending Resistance)	0.85
$\phi_t$ (Tension Resistance)	0.80
$\phi_v$ (Shear Resistance)	0.75
$\phi_c$ (Compression Resistance)	0.90
$\phi_s$ (Stability Resistance)	0.85
$\phi_D$ (Resistance)	0.80
$K_F$ (Format Conversion)	
Bending	2.54
Tension	2.70
Shear	2.88
Compression	
Perpendicular to Grain	2.08
Parallel to Grain	2.40
Stability	1.76
$G_s$ (Specific Gravity)	0.92

### **LRFD Design Values**

Nominal Bending Design Value:  $F_{bn}$

$$F_b (K_F) = \begin{matrix} 2,224 \text{ psi} \\ 2.22 \text{ ksi} \end{matrix}$$

Nominal Tension Parallel to Grain Design Value:  $F_{tn}$

$$F_t (K_F) = \begin{matrix} 1,215 \text{ psi} \\ 1.22 \text{ ksi} \end{matrix}$$

Nominal Shear Design Value Parallel to Grain:  $F_{vn}$

$$F_v (K_F) = \begin{matrix} 389 \text{ psi} \\ 0.39 \text{ ksi} \end{matrix}$$

Nominal Compression Perpendicular to Grain Design Value:  $F_{c\perp n}$

$$F_{c\perp} (K_F) = \begin{matrix} 885 \text{ psi} \\ 0.89 \text{ ksi} \end{matrix}$$

Nominal Compression Parallel to Grain Design Value:  $F_{cn}$

$$F_c (K_F) = \begin{array}{l} 2,760 \text{ psi} \\ 2.76 \text{ ksi} \end{array}$$

Nominal Modulus of Elasticity Design Value:  $E_{min}$

$$E_{min} (K_F) = \begin{array}{l} 900,000 \text{ psi} \\ 900 \text{ ksi} \end{array}$$

### **Wood Dimensions and Spacing**

$$\text{Wall Height: } h = 10.00 \text{ ft}$$

Section Modulus: S

$$2 \times 6 = 7.56 \text{ in}^3$$

$$2 \times 8 = 14.06 \text{ in}^3$$

$$2 \times 10 = 22.56 \text{ in}^3$$

$$2 \times 12 = 33.06 \text{ in}^3$$

Area: A

$$2 \times 6 = 8.25 \text{ in}^2$$

$$2 \times 8 = 11.25 \text{ in}^2$$

$$2 \times 10 = 14.25 \text{ in}^2$$

$$2 \times 12 = 17.25 \text{ in}^2$$

Moment of Inertia: I

$$2 \times 6 = 20.80 \text{ in}^3$$

$$2 \times 8 = 52.73 \text{ in}^3$$

$$2 \times 10 = 107.17 \text{ in}^3$$

$$2 \times 12 = 190.11 \text{ in}^3$$

### **Loads**

Transverse Lateral Force:  $w_T$

$$(\text{Lateral Wind Load}) \times (\text{Wall Height}) = 350 \text{ lb/ft}$$

Longitudinal Lateral Force:  $w_L$

$$(\text{Lateral Wind Load}) \times (\text{Wall Height}) = 350 \text{ lb/ft}$$

Transverse Moment:  $M_T$

$$(w_T \times L^2)/8 = 109 \text{ ft-k}$$

Logintudinal Moment:  $M_L$

$$(w_T \times L^2)/8 = 44.8 \text{ ft-k}$$

**Transverse Chord Forces**

Compression:  $C_u$

$$M_T/b = 3.42 \text{ k}$$

Tension:  $T_u$

$$M_T/b = 3.42 \text{ k}$$

Actual Compression Stress Parallel to Grain in Two (2)

2 x 6 Wood Members:  $f_c$

$$C_u/A = \begin{matrix} 0.21 \text{ ksi} \\ 207 \text{ psi} \end{matrix}$$

Actual Tension Stress Parallel to Grain in Two (2)

2 x 6 Wood Members:  $f_t$

$$T_u/A = \begin{matrix} 0.21 \text{ ksi} \\ 207 \text{ psi} \end{matrix}$$

Allowable LRFD Tension Stress Parallel to Grain for SPF No. 2:

$$F_{tn} = 1215 \text{ psi}$$

$$F_{tn} > f_t \quad 1215 > 207 \quad \text{TRUE}$$

Allowable LRFD Compression Stress Parallel to Grain for SPF No. 2:

$$F_c = 2,760 \text{ psi}$$

$$F_{cn} > f_c \quad 2,760 > 207 \quad \text{TRUE}$$

**Two (2) 2 x 6 SPF No. 2 OK for Transverse Chord Forces**

**Longitudinal Chord Forces:**

Compression:  $C_u$

$$M_L/b = 0.90 \text{ k}$$

Tension:  $T_u$

$$M_L/b = 0.90 \text{ k}$$

Actual Compression Stress Parallel to Grain in Two (2)

2 x 6 Wood Members:  $f_c$

$$C_u/A = \begin{matrix} 0.05 \text{ ksi} \\ 54.3 \text{ psi} \end{matrix}$$

Actual Tension Stress Parallel to Grain in Two (2)

2 x 6 Wood Members:  $f_t$

$$T_u/A = \begin{matrix} 0.05 \text{ ksi} \\ 54.3 \text{ psi} \end{matrix}$$

Allowable LRFD Tension Stress Parallel to Grain for SPF No. 2:

$$F_{tn} = 1215 \text{ psi}$$

$$F_{tn} > f_t \quad 1215 > 54.3 \quad \text{TRUE}$$

Allowable LRFD Compression Stress Parallel to Grain for SPF No. 2:

$$F_c = 2,760 \text{ psi}$$

$$F_{cn} > f_c \quad 2,760 > 54.3 \quad \text{TRUE}$$

**Two (2) 2 x 6 SPF No. 2 OK for Longitudinal Chord Forces**

**Longitudinal Lateral Forces**

Unit Shear:  $v_u$

$$V_u/b = 112 \text{ lb/ft}$$

Adjusted Unit Shear:  $V$

$$1.6v_u = 179 \text{ lb/ft}$$

Load Case: Case 3

Maximum Nominal Unit Shear for Wind with 15/32" APA  
Rated 32/16 3-Ply Plywood Panels with 8d Common Nails  
at 6" o.c. :  $v_w$

$$505 \text{ lb/ft}$$

Allowable Unit Shear: Allow.  $v$

$$(G*v_w)/2 = 372 \text{ lb/ft}$$

$$\text{Allow. } v > V \quad 372 > 179 \quad \text{TRUE}$$

**No Blocking Required for Longitudinal Lateral Force**

**Transverse Strut Forces**

Diaphragm Unit Shear:  $v_R$

$$V_u/b = 273 \text{ lb/ft}$$

Shear Wall Unit Shear:  $v_w$

$$V_u/b = 307 \text{ lb/ft}$$

Tension:  $T_A$

$$v_R(L/2) - v_w(\Sigma \text{ Opening}) = 2.79 \text{ k}$$

Compression:  $C_A$

$$v_R(L/2) - v_W(\Sigma \text{ Opening}) = 2.79 \text{ k}$$

Chord vs. Strut Forces: 3.42 > 2.79 TRUE

**Two (2) 2 x 6 SPF No.2 OK for Transverse Strut Forces\***

\*Chords and struts are the same member designed for forces from different direction (perpendicular or parallel). Because, in this case, the chord forces in the transverse direction are larger than the strut forces the chord design governs and can be used for the struts

**Longitudinal Strut Forces**

Diaphragm Unit Shear:  $v_R$

$$V_U/b = 112 \text{ lb/ft}$$

Shear Wall Unit Shear:  $v_W$

$$V_U/b = 174 \text{ lb/ft}$$

Tension:  $T_A$

$$v_R(L/2) - v_W(\Sigma \text{ Opening}) = 0.51 \text{ k}$$

Compression:  $C_A$

$$v_R(L/2) - v_W(\Sigma \text{ Opening}) = 0.51 \text{ k}$$

Chord vs. Strut Forces: 0.90 > 0.51 TRUE

**Use Two (2) 2 x 6 SPF No. 2 or Better for All Diaphragm Chords and Struts (Top Plates)  
MC 19%**



# LRFD Floor Sheathing

**Loads**

Total Load:  $w_u$   
 $1.2D + 1.6L =$  78.5 psf

Applied Load:  $w_{AL}$   
 $1.6L =$  64.0 psf

**Factors**

$\lambda$  (Time Effect) 0.80

$\varphi_b$  (Bending Resistance) 0.85

$\varphi_v$  (Shear Resistance) 0.75

$\varphi_D$  (Resistance) 0.80

$K_F$  (Format Conversion)

Bending 2.54

Shear 2.88

$C_G$  (Grade and Construction)

Stiffness

Perpendicular to Joists 1.10

Parallel to Joists 1.00

Bending

Perpendicular to Joists 1.00

Parallel to Joists 1.00

Shear

Perpendicular to Joists 1.00

Parallel to Joists 2.80

$C_{SA}$  (Span Adjustment)

3-Span to 1-Span

Stiffness 0.53

Bending 0.80

Shear 1.20

$G_s$  (Specific Gravity) 0.92

**Strength Axis Perpendicular to Joists**

Span Rating: 32 /16

Plywood Type: 3-Ply

Applied Load:  $w_{AL}$   
 $1.6L =$  64.0 psf

Allowable Uniform Applied Load on APA Rated Plywood  
 Sheathing Across 16" o.c. Supports:  $W_{AL}$  282 psf

Adjusted Allowable Uniform Applied Load for 3-Ply  
 Plywood Sheathing:  $W'_{AL}$   
 $W_{AL}(C_G)(C_{SA}) =$  164 psf

$W'_{AL} > w_{AL}$	164	>	64.0	TRUE
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Total Load:  $w_u$

$1.2D + 1.6L =$	78.5	psf
-----------------	------	-----

Allowable Uniform Load on APA Rated Plywood

Sheathing Across 16" o.c. Supports: $W_u$	376	psf
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Adjusted Allowable Uniform Load for 3-Ply Plywood

Sheathing:  $W'_u$

$W_u(C_G)(C_{SA}) =$	219	psf
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$W'_u > w_u$	219	>	78.5	TRUE
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Actual Bending Stress:  $f_b$

$1.2D + 1.6L =$	78.5	psf
-----------------	------	-----

Allowable Bending Stress for APA Rated Plywood Sheathing

Across 16" o.c. Supports: $F_b$	173	psf
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Adjusted Allowable Bending Stress for 3-Ply Plywood

Sheathing:  $F'_b$

$F_b(K_F)(\lambda)(\phi_b)(C_G)(C_{SA}) =$	239	psf
--	-----	-----

$F'_b > f_b$	239	>	78.5	TRUE
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Actual Shear Stress:  $f_v$

$1.2D + 1.6L =$	78.5	psf
-----------------	------	-----

Allowable Shear Stress for APA Rated Plywood Sheathing

Across 16" o.c. Supports: $F_v$	290	psf
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Adjusted Allowable Shear Stress for 3-Ply Plywood

Sheathing:  $F'_v$

$F_v(K_F)(\lambda)(\phi_v)(C_G)(C_{SA}) =$	601	psf
--	-----	-----

$F'_v > f_v$	601	>	78.5	TRUE
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**APA Rated 32/16 3-Ply Plywood OK for Floor Sheathing Laid with Strength Axis Perpendicular to Joists**

**Strength Axis Parallel to Joists:**

Span Rating:	32 / 16
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Plywood Type:	3-Ply
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Applied Load: $w_{AL}$				
1.6L =			64.0 psf	
Allowable Uniform Applied Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{AL}$			20.0 psf	
Adjusted Allowable Uniform Applied Load for 3-Ply Plywood Sheathing: $W'_{AL}$				
$W_{AL}(C_G)(C_{SA}) =$			10.6 psf	
$W'_{AL} > w_{AL}$		10.6 < 64.0		FAIL
Total Load: $w_u$				
1.2D + 1.6L =			78.5 psf	
Allowable Uniform Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_u$			27.0 psf	
Adjusted Allowable Uniform Load for 3-Ply Plywood Sheathing: $W'_u$				
$W_u(C_G)(C_{SA}) =$			14.3 psf	
$W'_u > w_u$		14.3 < 78.5		FAIL
Actual Bending Stress: $f_b$				
1.2D + 1.6L =			78.5 psf	
Allowable Bending Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_b$			43.0 psf	
Adjusted Allowable Bending Stress for 3-Ply Plywood Sheathing: $F'_b$				
$F_b(K_F)(\lambda)(\phi_b)(C_G)(C_{SA}) =$			59.4 psf	
$F'_b > f_b$		59.4 < 78.5		FAIL
Actual Shear Stress: $f_v$				
1.2D + 1.6L =			78.5 psf	
Allowable Shear Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_v$			179 psf	
Adjusted Allowable Shear Stress for 3-Ply Plywood Sheathing: $F'_v$				
$F_v(K_F)(\lambda)(\phi_v)(C_G)(C_{SA}) =$			1039 psf	

$F'_v > f_v$	1039 > 78.5	TRUE
Span Rating:	40 /20	
Plywood Type:	3-Ply	
Applied Load: $w_{AL}$ 1.6L =	64.0 psf	
Allowable Uniform Applied Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_{AL}$	44.0 psf	
Adjusted Allowable Uniform Applied Load for 3-Ply Plywood Sheathing: $W'_{AL}$ $W_{AL}(C_G)(C_{SA}) =$	23.3 psf	
$W'_{AL} > w_{AL}$	23.3 < 64.0	FAIL
Total Load: $w_u$ 1.2D + 1.6L =	78.5 psf	
Allowable Uniform Load on APA Rated Plywood Sheathing Across 16" o.c. Supports: $W_u$	59.0 psf	
Adjusted Allowable Uniform Load for 3-Ply Plywood Sheathing: $W'_u$ $W_u(C_G)(C_{SA}) =$	31.3 psf	
$W'_u > w_u$	31.3 < 78.5	FAIL
Actual Bending Stress: $f_b$ 1.2D + 1.6L =	78.5 psf	
Allowable Bending Stress for APA Rated Plywood Sheathing Across 16" o.c. Supports: $F_b$	70.0 psf	
Adjusted Allowable Bending Stress for 3-Ply Plywood Sheathing: $F'_b$ $F_b (K_F)(\lambda)(\phi_b)(C_G)(C_{SA}) =$	96.7 psf	
$F'_b > f_b$	96.7 > 78.5	TRUE
Actual Shear Stress: $f_v$ 1.2D + 1.6L =	78.5 psf	

Allowable Shear Stress for APA Rated Plywood Sheathing  
Across 16" o.c. Supports:  $F_v$

228 psf

Adjusted Allowable Shear Stress for 3-Ply Plywood  
Sheathing:  $F'_v$

$F_v (K_F)(\lambda)(\phi_v)(C_G)(C_{SA}) =$

1324 psf

$F'_v > f_v$

1324 > 78.5 TRUE

**No APA Rated 3-Ply Plywood OK for Floor Sheathing Laid with Strength Axis Parallel to Joists**

**Thickness and Nailing**

3/8" Thick APA Rated 24/0 3-Ply Plywood Nailed Perpendicular to Joists with 6d Common Nails

Load Case:

Case 1

Maximum Nominal Unit Shear for Wind Loading on Douglas  
Fir Plywood:  $V_w$

460 lb/ft

Adjusted Nominal Unit Shear for Wind Loading on SPF  
Plywood:  $V'_w$

$G_s(V_w)(\phi_D) =$

339 lb/ft

Actual Shear:  $v$

$1.6W =$

560 lb/ft

$V'_w > v$

339 > 560 FAIL

---

19/32" Thick APA Rated 32/16 3-Ply Plywood Nailed Perpendicular to Joists with 10d Common Nails

Load Case:

Case 1

Maximum Nominal Unit Shear for Wind Loading on Douglas  
Fir Plywood:  $V_w$

800 lb/ft

Adjusted Nominal Unit Shear for Wind Loading on SPF  
Plywood:  $V'_w$

$G_s(V_w)(\phi_D) =$

589 lb/ft

Actual Shear:  $v$

$1.6W =$

560 lb/ft

$V'_w > v$

589 > 560 TRUE

**Use 19/32" APA Rated 32/16 3-Ply Plywood Panels or Better as Subfloor  
With 1/4" Underlayment Grade Panel Installed Over Subfloor  
Nailed Perpendicular to Joists with 10d Common Nails  
at 6" o.c. Supported Edges  
12" o.c. Field  
No Blocking Required**

# LRFD Studs



### **Wood Properties**

Spruce-Pine-Fir	Stud
Bending ( $F_b$ )	675 psi
Tension Parallel to Grain ( $F_t$ )	350 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	725 psi
Modulus of Elasticity (E)	1,200,000 psi
Modulus of Elasticity ( $E_{min}$ )	440,000 psi

### **Factors**

$\lambda$ (Time Effect)	
1.2D + 1.6W + 0.5L + 0.5S	1.00
1.2D + 1.6L + 0.5S	0.80
$\varphi_b$ (Bending Resistance)	0.85
$\varphi_t$ (Tension Resistance)	0.80
$\varphi_v$ (Shear Resistance)	0.75
$\varphi_c$ (Compression Resistance)	0.90
$\varphi_s$ (Stability Resistance)	0.85
$K_F$ (Format Conversion)	
Bending	2.54
Tension	2.70
Shear	2.88
Compression	
Perpendicular to Grain	2.08
Parallel to Grain	2.40
Stability	1.76
$C_F$ (Size)	
Bending	1.10
Tension	1.10
Compression	1.05
$C_i$ (Incising)	1.00
$C_L$ (Stability)	1.00
$C_M$ (Wet Service)	1.00
$C_r$ (Repetitive Member)	1.15
$C_t$ (Thermal)	1.00

### **LRFD Design Values**

Nominal Bending Design Value:  $F_{bn}$

$$F_b (K_F) = \begin{matrix} 1,715 \text{ psi} \\ 1.72 \text{ ksi} \end{matrix}$$

Nominal Tension Parallel to Grain Design Value:  $F_{tn}$

$$F_t (K_F) = \begin{matrix} 945 \text{ psi} \\ 0.95 \text{ ksi} \end{matrix}$$

Nominal Shear Design Value Parallel to Grain:  $F_{vn}$

$$F_v (K_F) = \begin{array}{l} 389 \text{ psi} \\ 0.39 \text{ ksi} \end{array}$$

Nominal Compression Perpendicular to Grain Design

Value:  $F_{c \perp n}$

$$F_{c \perp} (K_F) = \begin{array}{l} 885 \text{ psi} \\ 0.89 \text{ ksi} \end{array}$$

Nominal Compression Parallel to Grain Design Value:  $F_{cn}$

$$F_c (K_F) = \begin{array}{l} 1,740 \text{ psi} \\ 1.74 \text{ ksi} \end{array}$$

Nominal Modulus of Elasticity Design Value:  $E_{min n}$

$$E_{min n} (K_F) = \begin{array}{l} 776,471 \text{ psi} \\ 776 \text{ ksi} \end{array}$$

### **Wood Dimensions and Spacing**

$$\text{Stud Length: } L = 10.00 \text{ ft}$$

$$\text{Stud Spacing: } = 16.00 \text{ in}$$

Section Modulus: S

$$2 \times 4 = 3.06 \text{ in}^3$$

$$2 \times 6 = 7.56 \text{ in}^3$$

Area: A

$$2 \times 4 = 5.25 \text{ in}^2$$

$$2 \times 6 = 8.25 \text{ in}^2$$

Moment of Inertia: I

$$2 \times 4 = 5.36 \text{ in}^3$$

$$2 \times 6 = 20.80 \text{ in}^3$$

### **Load Case 1: Gravity Loads Only**

Total Load:  $w_u$

$$(1.2D + 1.6L + 0.5S) \times \text{Stud Spacing} = \begin{array}{l} 1,451 \text{ lbs} \\ 1.45 \text{ k} \end{array}$$

Shear:  $V_u$

$$(w_u \times L)/2 = \begin{array}{l} 7,254 \text{ lbs} \\ 7.25 \text{ k} \end{array}$$

Moment:  $M_u$

$$(w_u \times L^2)/8 = \begin{array}{l} 18,136 \text{ ft-lb} \\ 218 \text{ k-in} \end{array}$$

### **Column Capacity:**

Column Buckling About y-axis:

$$(l_e/d)_y = 0 \Rightarrow \text{Sheathing}$$

Column Buckling About x-axis:

$$(l_e/d)_x = 21.8$$

Adjusted Modulus of Elasticity for Stability:  $E'_{min n}$

$$E_{min n} (\varphi_s)(\lambda)(C_M)(C_t)(C_i) = 528 \text{ ksi}$$

Nominal Buckling Value for Compression:  $F_{cEn}$

$$0.822E'_{min n} / (l_e/d)^2 = 0.91 \text{ ksi}$$

$$912 \text{ psi}$$

Nominal Compression Design Value Parallel to Grain

Multiplied by all Adjustment Factors Except  $C_p$ :  $F^*_{cn}$

$$F_{cn} (\varphi_c)(\lambda)(C_M)(C_t)(C_F)(C_i) = 1378 \text{ psi}$$

$$1.38 \text{ ksi}$$

$$F_{cEn} / F^*_{cn} = 0.662$$

$$(1 + F_{cEn} / F^*_{cn}) / 2c = 1.038$$

Column Stability Factor:  $C_p$

$$(1 + F_{cEn} / F^*_{cn}) / 2c - \sqrt{((1 + F_{cEn} / F^*_{cn}) / 2c)^2 - (F_{cEn} / F^*_{cn}) / c} = 0.537$$

Adjusted Compression Design Value Parallel to Grain:  $F'_{cn}$

$$F_{cn} (\varphi_c)(\lambda)(C_M)(C_t)(C_F)(C_p)(C_i) = 740 \text{ psi}$$

$$0.74 \text{ ksi}$$

Nominal Lateral Design Value Parallel to Grain:  $P_n$

$$F'_{cn} (A) = 6,105 \text{ psi}$$

$$6.11 \text{ ksi}$$

$$P_n > w_u \quad 6.11 > 1.45 \quad \text{TRUE}$$

**Bearing of Stud on Wall Plates**

Bearing Area Factor:  $C_b$

$$(l_b + 0.375) / l_b = 1.25$$

Adjusted Compression Design Value Perpendicular to

Grain:  $F'_{c \perp n}$

$$F_{c \perp n} (\varphi_c)(\lambda)(C_M)(C_t)(C_b) = 797 \text{ psi}$$

$$0.80 \text{ ksi}$$

Nominal Lateral Design Value Perpendicular to Grain:  $P_{\perp n}$

$$F'_{c \perp n} (A) = 6.57 \text{ ksi}$$

$$P_{\perp n} > w_u \quad 6.57 > 1.45 \quad \text{TRUE}$$

### **Load Case 2: Gravity Loads + Lateral Loads**

Lateral Load:  $w_u$

$$1.6W \times \text{Stud Spacing} = 74.7 \text{ lb/ft}$$

Shear:  $V_u$

$$(w_u \times L)/2 = 373 \text{ lbs}$$
$$0.37 \text{ k}$$

Moment:  $M_u$

$$(w_u \times L^2)/8 = 933 \text{ ft-lb}$$
$$11.2 \text{ k-in}$$

### **Bending**

Nominal Bending Stress:  $f_{bu}$

$$M_u/S = 1.48 \text{ ksi}$$

Adjusted Bending Design Value:  $F'_{bn}$

$$F_{bn}(\phi_b)(\lambda)(C_M)(C_t)(C_L)(C_F)(C_r)(C_i) = 1.84 \text{ ksi}$$

Adjusted Moment Design Value:  $M_n$

$$F'_{bn}(S) = 13.9 \text{ k-in}$$

$$M_n > M_u \quad 13.9 > 11.2 \quad \text{TRUE}$$

### **Axial**

Axial Load:  $P_u$

$$(1.2D + 0.5L + 0.5S) \times \text{Stud Spacing} = 1248 \text{ lb}$$
$$1.25 \text{ k}$$

Nominal Compression Stress Parallel to Grain:  $f_{cu}$

$$P_u/A = 0.15 \text{ ksi}$$

Column Buckling About x-axis:

$$(l_e/d)_x = 21.8$$

Adjusted Modulus of Elasticity for Stability:  $E'_{min n}$

$$E_{min n}(\phi_s)(\lambda)(C_M)(C_t)(C_i) = 660 \text{ ksi}$$

Nominal Buckling Value for Compression:  $F_{cEn}$

$$0.822E'_{min n}/(l_e/d)^2 = 1.14 \text{ ksi}$$
$$1,140 \text{ psi}$$

Nominal Compression Design Value Parallel to Grain

Multiplied by all Adjustment Factors Except  $C_p$ :  $F^*_{cn}$

$$F_{cn}(\phi_c)(\lambda)(C_M)(C_t)(C_F)(C_i) = 1,723 \text{ psi}$$
$$1.72 \text{ ksi}$$

$$F_{cEn}/F_{cn}^* = 0.662$$

$$(1+F_{cEn}/F_{cn}^*)/2c = 1.04$$

Column Stability Factor:  $C_p$

$$(1+F_{cEn}/F_{cn}^*)/2c - \sqrt{((1+F_{cEn}/F_{cn}^*)/2c)^2 - (F_{cEn}/F_{cn}^*)/c} = 0.537$$

Adjusted Compression Design Value Parallel to Grain:  $F'_{cn}$

$$F_{cn}(\phi_c)(\lambda)(C_M)(C_t)(C_F)(C_P)(C_i) = \begin{matrix} 925 \text{ psi} \\ 0.93 \text{ ksi} \end{matrix}$$

Nominal Lateral Design Value Parallel to Grain:  $P'_n$

$$F'_{cn}(A) = \begin{matrix} 7,632 \text{ psi} \\ 7.63 \text{ ksi} \end{matrix}$$

$$P'_n > P_u \quad 7.63 > 1.25 \quad \text{TRUE}$$

**Combined Stress:**

Interaction Formula:

$$(f_{cu}/F'_{cn})^2 + (f_{bu}/(F'_{bn}(1-f_{cu}/F_{cEn}))) \leq 1.0 \quad 1.00 \leq 1 \quad \text{TRUE}$$

**Use 2 x 6 SPF No. 2 or Better for All Studs  
MC ≤ 19%**

# LRFD Shearwalls

**Wood Properties**

Spruce-Pine-Fir	Stud
Bending ( $F_b$ )	675 psi
Tension Parallel to Grain ( $F_t$ )	350 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	725 psi
Modulus of Elasticity (E)	1,200,000 psi
Modulus of Elasticity ( $E_{min}$ )	440,000 psi

**Factors**

$\lambda$ (Time Effect)	1.00
$\varphi_b$ (Bending Resistance)	0.85
$\varphi_t$ (Tension Resistance)	0.80
$\varphi_v$ (Shear Resistance)	0.75
$\varphi_c$ (Compression Resistance)	0.90
$\varphi_s$ (Stability Resistance)	0.85
$\varphi_D$ (Resistance)	0.80
$K_F$ (Format Conversion)	
Bending	2.54
Tension	2.70
Shear	2.88
Compression	
Perpendicular to Grain	2.08
Parallel to Grain	2.40
Stability	1.76
$C_d$ (Deflection Amplification)	4.00
$C_F$ (Size)	
Bending	1.10
Tension	1.10
Compression	1.05
$C_i$ (Incising)	1.00
$C_L$ (Stability)	1.00
$C_M$ (Wet Service)	1.00
$C_r$ (Repetitive Member)	1.15
$C_t$ (Thermal)	1.00
$G_s$ (Specific Gravity)	0.92

**LRFD Design Values**Nominal Bending Design Value:  $F_{bn}$ 

$F_b$ ( $K_F$ ) =	1,715 psi
	1.72 ksi

Nominal Tension Parallel to Grain Design Value: $F_{tn}$	
$F_t (K_F) =$	945 psi 0.95 ksi
Nominal Shear Design Value Parallel to Grain: $F_{vn}$	
$F_v (K_F) =$	389 psi 0.39 ksi
Nominal Compression Perpendicular to Grain Design Value: $F_{c \perp n}$	
$F_{c \perp} (K_F) =$	885 psi 0.89 ksi
Nominal Compression Parallel to Grain Design Value: $F_{cn}$	
$F_c (K_F) =$	1,740 psi 1.74 ksi
Nominal Modulus of Elasticity Design Value: $E_{min n}$	
$E_{min n} (K_F) =$	776,471 psi 776 ksi

**Wood Dimensions and Spacing:**

Wall Height: h	10.00 ft
Stud Spacing:	16.00 in
Section Modulus: S	
2 x 6 =	7.56 in <sup>3</sup>
4 x 6 =	17.65 in <sup>3</sup>
Area: A	
2 x 6 =	8.25 in <sup>2</sup>
4 x 6 =	19.25 in <sup>2</sup>
Moment of Inertia: I	
2 x 6 =	20.80 in <sup>3</sup>
4 x 6 =	48.53 in <sup>3</sup>

**Loads**

Ultimate Uniform Wind Load: $w_u$	
(Lateral Wind Load) x (Wall Height) =	350 lb/ft

**Wall 1 (First Floor Front Facing Wall)**

Ultimate Shear Force in Shearwall: $V_u$	
$w_u(b/2) =$	5,600 lb 5.60 k



Ultimate Unit Shear in Shearwall: $v_u$	174 lb/ft		
Unit Shear in Shearwall: $v$ $1.6v_u =$	279 lb/ft		
5/16" Plywood Siding with 6d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	390 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $G_s(v_w)(\phi_D) =$	287 lb/ft		
Allow. $v > v$	287 > 279		TRUE

**5/16" Plywood Siding with 6d Common Nails at 6" o.c. OK for Wall 1**

**Wall 2 (First Floor Back Facing Wall)**

Ultimate Shear Force in Shearwall: $V_u$ $w_u(b/2) =$	5,600 lb 5.60 k		
Ultimate Unit Shear in Shearwall: $v_u$	160 lb/ft		
Unit Shear in Shearwall: $v$ $1.6v_u =$	256 lb/ft		
5/16" Plywood Siding with 6d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	390 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $G_s(v_w)(\phi_D) =$	287 lb/ft		
Allow. $v > v$	287 > 256		TRUE

**5/16" Plywood Siding with 6d Common Nails at 6" o.c. OK for Wall 2**

**Wall 3 (First Floor Right Facing Wall)**

Ultimate Shear Force in Shearwall: $V_u$ $w_u(b/2) =$	8,750 lb 8.75 k		
Ultimate Unit Shear in Shearwall: $v_u$	273 lb/ft		
Unit Shear in Shearwall: $v$ $1.6v_u =$	438 lb/ft		

5/16" Plywood Siding with 6d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	390 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $G_s(v_w)(\phi_D) =$	287 lb/ft		
Allow. $v > v$	287	>	438      FAIL
3/8" Structural Panels -- Sheathing with 8d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	615 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $G_s(v_w)(\phi_D) =$	453 lb/ft		
Allow. $v > v$	453	>	438      TRUE

**3/8" Structural Panels -- Sheathing with 8d Common Nails at 6" o.c. OK for Wall 3**

**Wall 4 (First Floor Left Facing Wall)**

Ultimate Shear Force in Shearwall: $V_u$ $w_u(b/2) =$	8,750 lb 8.75 k		
Ultimate Unit Shear in Shearwall: $v_u$	307 lb/ft		
Unit Shear in Shearwall: $v$ $1.6v_u =$	491 lb/ft		
7/16" Structural Panels -- Sheathing with 8d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	670 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $G_s(v_w)(\phi_D) =$	493 lb/ft		
Allow. $v > v$	493	>	491      TRUE

**7/16" Structural Panels -- Sheathing with 8d Common Nails at 6" o.c. OK for Wall 4**

**Wall 5 (Second Floor Front Facing Wall)**

Ultimate Shear Force in Shearwall: $V_u$ $w_u(b/2) =$	5,600 lb 5.60 k		
Ultimate Unit Shear in Shearwall: $v_u$	174 lb/ft		

Unit Shear in Shearwall: $v$			
$1.6v_u =$		279 lb/ft	
5/16" Plywood Siding with 6d Common Nail @ 6" o.c.			
Maximum Nominal Unit Shear for Wind: $v_w$		390 lb/ft	
Allowable Unit Shear for SPF: Allow. $v$			
$G_s(v_w)(\phi_D) =$		287 lb/ft	
Allow. $v > v$		287 > 279	TRUE

**5/16" Plywood Siding with 6d Common Nails at 6" o.c. OK for Wall 5**

**Wall 6 (Second Floor Back Facing Wall)**

Ultimate Shear Force in Shearwall: $V_u$			
$w_u(b/2) =$		5,600 lb	
		5.60 k	
Ultimate Unit Shear in Shearwall: $v_u$		145 lb/ft	
Unit Shear in Shearwall: $v$			
$1.6v_u =$		233 lb/ft	
5/16" Plywood Siding with 6d Common Nail @ 6" o.c.			
Maximum Nominal Unit Shear for Wind: $v_w$		390 lb/ft	
Allowable Unit Shear for SPF: Allow. $v$			
$G_s(v_w)(\phi_D) =$		287 lb/ft	
Allow. $v > v$		287 > 233	TRUE

**5/16" Plywood Siding with 6d Common Nails at 6" o.c. OK for Wall 6**

**Wall 7 (Second Floor Right Facing Wall)**

Ultimate Shear Force in Shearwall: $V_u$			
$w_u(b/2) =$		8,750 lb	
		8.75 k	
Ultimate Unit Shear in Shearwall: $v_u$		297 lb/ft	
Unit Shear in Shearwall: $v$			
$1.6v_u =$		475 lb/ft	

5/16" Plywood Siding with 6d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	390 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $G_s(v_w)(\phi_D) =$	287 lb/ft		
Allow. $v > v$	287	>	475      FAIL
7/16" Structural Panels -- Sheathing with 8d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	670 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $G_s(v_w)(\phi_D) =$	493 lb/ft		
Allow. $v > v$	493	>	475      TRUE

**7/16" Structural Panels -- Sheathing with 8d Common Nails at 6" o.c. OK for Wall 7**

**Wall 8 (Second Floor Left Facing Wall)**

Ultimate Shear Force in Shearwall: $V_u$ $w_u(b/2) =$	8,750 lb 8.75 k		
Ultimate Unit Shear in Shearwall: $v_u$	273 lb/ft		
Unit Shear in Shearwall: $v$ $1.6v_u =$	438 lb/ft		
5/16" Plywood Siding with 6d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	390 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $G_s(v_w)(\phi_D) =$	287 lb/ft		
Allow. $v > v$	287	>	438      FAIL
3/8" Structural Panels -- Sheathing with 8d Common Nail @ 6" o.c. Maximum Nominal Unit Shear for Wind: $v_w$	615 lb/ft		
Allowable Unit Shear for SPF: Allow. $v$ $G_s(v_w)(\phi_D) =$	453 lb/ft		
Allow. $v > v$	453	>	438      TRUE

### 3/8" Structural Panels -- Sheathing with 8d Common Nails at 6" o.c. OK for Wall 3

#### Tension Chord

Load at Top of Shearwall:  $v$

$$1.6v_u = 491 \text{ lb/ft}$$

Tension:  $T$

$$v h = 4,912 \text{ lb}$$

$$4.91 \text{ k}$$

Net Area:  $A_n$

$$bh = 8.25 \text{ in}^2$$

Actual Tension Stress Parallel to Grain:  $f_t$

$$T/A = 595 \text{ psi}$$

Adjusted LRFD Tension Design Value:  $F'_{tn}$

$$F_{tn}(\phi_t)(\lambda)(C_M)(C_t)(C_F) = 832 \text{ psi}$$

$$F'_t > f_t \quad 832 > 595 \quad \text{TRUE}$$

#### One 2 x 6 OK for All Tension Chords of Shearwalls

#### Compression Chord

Column Buckling About y-axis:

$$(l_e/d)_y = 0.00 \Rightarrow \text{Sheathing}$$

Column Buckling About x-axis:

$$(l_e/d)_x = 21.8$$

Adjusted Modulus of Elasticity for Stability:  $E'_{min n}$

$$E_{min n}(\phi_s)(\lambda)(C_M)(C_t)(C_i) = 374,000 \text{ psi}$$

Nominal Buckling Value for Compression:  $F_{cEn}$

$$0.822E'_{min n} / (l_e/d)^2 = 646 \text{ psi}$$

Nominal Compression Design Value Parallel to Grain

Multiplied by all Adjustment Factors Except  $C_p$ :  $F^*_{cn}$

$$F_{cn}(\phi_c)(\lambda)(C_M)(C_t)(C_F)(C_i) = 1,644 \text{ psi}$$

$$1.64 \text{ ksi}$$

$$F_{cEn} / F^*_{cn} = 0.393$$

$$(1+F_{cEn}/F'_{cn})/2c = 0.870$$

Column Stability Factor:  $C_p$

$$(1+F_{cEn}/F'_{cn})/2c - \text{sqrt}(((1+F_{cEn}/F'_{cn})/2c)^2 - (F_{cEn}/F'_{cn})/c) = 0.354$$

Adjusted LRFD Compression Design Value Parallel to

Grain:  $F'_{cn}$

$$F_{cn}(\varphi_c)(\lambda)(C_M)(C_t)(C_F)(C_P)(C_i) = 582 \text{ psi}$$

Adjusted LRFD Compression Design Value

Perpendicular to Grain:  $F'_{c\perp n}$

$$F_{c\perp n}(C_M)(C_t)(C_b) = 885 \text{ psi}$$

**$F'_{cn}$  Governs**

Total Dead Load Acting on Shearwall:  $w_{DL}$

$$1.4D = 584 \text{ lb/ft}$$

Total Load Acting on Chord: P

$$\text{(Tributary Area)} \times w_{DL} = 9,336 \text{ lb}$$

$$9.34 \text{ k}$$

Allowable Compression Load on 2 x 6 Chord: Allow. P

$$F'_{cn}A = 4,802 \text{ lb}$$

$$4.80 \text{ k}$$

$$\text{Allow. P} > P \quad 4.80 > 9.34 \quad \text{FAIL}$$

Allowable Compression Load on 4 x 6 Chord: Allow. P

$$F'_{cn}A = 11,204 \text{ lb}$$

$$11.20 \text{ k}$$

$$\text{Allow. P} > P \quad 11.20 > 9.34 \quad \text{TRUE}$$

**One (1) 4 x 6 SPF Stud Post is OK for All Compression Shearwall Chords**

**Unblocked Shearwall Deflection**

Check 5/16" APA Rated 24/0 Wood Structural Panels -- Sheathing with 6d Common Nails at 6" o.c. at Supported Edges and 6" o.c.

$$\text{Max Shear Force at Top of Wall: } v_u = 307 \text{ lb/ft}$$

Adjusted Shear Force for Unblocked Wall:  $v'_u$

$$v_u/C_{ub} = 384 \text{ lb/ft}$$

Load Per Fastener:	192 lb/ft		
Bending Deflection: $\Delta_b$ $8v h^3/EAb =$	0.004 in		
Shear Deflection: $\Delta_v$ $vh /Gt =$	0.154 in		
Nail Slip: $\Delta_n$ $0.75he_n =$	N/A*		
* 192 lb/nail exceeds largest allowable load per fastener of 160 lb/ft for 6d common nails			
Anchorage Slip: $\Delta_a$ $(h/b)d_a =$	0.039 in		
Story Drift: $\Delta_s$ $\Delta_b + \Delta_v + \Delta_n + \Delta_a =$	N/A		
Total Deflection: $\Delta$ $C_d \Delta_s =$	N/A		
Deflection Limit: $\Delta_{limit}$ $0.02(h \times 12) =$	0.24 in		
$\Delta_{limit} > \Delta$	0.24	> N/A	FAIL

Check 3/8" APA Rated 24/0 Wood Structural Panels -- Sheathing with 8d Common Nails at 6" o.c. at Supported Edges and 6" o.c. Field

Max Shear Force at Top of Wall: $v_u$	307 lb/ft
Adjusted Shear Force for Unblocked Wall: $v'_u$ $v_u/C_{ub} =$	384 lb/ft
Load Per Fastener:	192 lb/ft
Bending Deflection: $\Delta_b$ $8v h^3/EAb =$	0.004 in
Shear Deflection: $\Delta_v$ $vh /Gt =$	0.154 in
Nail Slip: $\Delta_n$ $0.75he_n =$	0.630 in

Anchorage Slip: $\Delta_a$ $(h/b)d_a =$	0.039 in	
Story Drift: $\Delta_s$ $\Delta_b + \Delta_v + \Delta_n + \Delta_a =$	0.827 in	
Total Deflection: $\Delta$ $C_d \Delta_s =$	3.31 in	
Deflection Limit: $\Delta_{limit}$ $0.02(h \times 12) =$	2.40 in	
$\Delta_{limit} > \Delta$	2.40 > 3.31	FAIL

Check 15/32" APA Rated 32/16 Wood Structural Panels -- Sheathing with 10d Common Nails at 6" o.c. at Supported Edges and 6" o.c. Field

Max Shear Force at Top of Wall: $v_u$	307 lb/ft
Adjusted Shear Force for Unblocked Wall: $v'_u$ $v_u/C_{ub} =$	384 lb/ft
Load Per Fastener:	192 lb/ft
Bending Deflection: $\Delta_b$ $8v h^3/EAb =$	0.004 in
Shear Deflection: $\Delta_v$ $vh/Gt =$	0.142 in
Nail Slip: $\Delta_n$ $0.75he_n =$	0.423 in
Anchorage Slip: $\Delta_a$ $(h/b)d_a =$	0.039 in
Story Drift: $\Delta_s$ $\Delta_b + \Delta_v + \Delta_n + \Delta_a =$	0.608 in
Total Deflection: $\Delta$ $C_d \Delta_s =$	2.43 in



Deflection Limit: $\Delta_{limit}$			
$0.02(h \times 12) =$		2.40 in	
$\Delta_{limit} > \Delta$		2.40 > 2.43	FAIL

Check 19/32" APA Rated 40/20 Structural Panels -- Sheathing with 10d Common Nails at 6" o.c. at Supported Edges and 6" o.c. Field

Max Shear Force at Top of Wall: $v_u$		307 lb/ft	
Adjusted Shear Force for Unblocked Wall: $v'_u$			
$v_u/C_{ub} =$		384 lb/ft	
Load Per Fastener:		192 lb/ft	
Bending Deflection: $\Delta_b$			
$8v h^3/EAb =$		0.004 in	
Shear Deflection: $\Delta_v$			
$vh/Gt =$		0.135 in	
Nail Slip: $\Delta_n$			
$0.75he_n =$		0.423 in	
Anchorage Slip: $\Delta_a$			
$(h/b)d_a =$		0.039 in	
Story Drift: $\Delta_s$			
$\Delta_b + \Delta_v + \Delta_n + \Delta_a =$		0.601 in	
Total Deflection: $\Delta$			
$C_d \Delta_s =$		2.40 in	
Deflection Limit: $\Delta_{limit}$			
$0.02(h \times 12) =$		2.40 in	
$\Delta_{limit} > \Delta$		2.40 > 2.40	FAIL

Check 15/32" APA Rated 32/16 Structural Panels -- Structural with 10d Common Nails at 6" o.c. at Supported Edges and 6" o.c. Field

Max Shear Force at Top of Wall: $v_u$		307 lb/ft
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Adjusted Shear Force for Unblocked Wall: $v'_u$			
$v_u/C_{ub} =$		384 lb/ft	
Load Per Fastener:		192 lb/ft	
Bending Deflection: $\Delta_b$			
$8v h^3/EAb =$		0.004 in	
Shear Deflection: $\Delta_v$			
$vh/Gt =$		0.118 in	
Nail Slip: $\Delta_n$			
$0.75he_n =$		0.353 in	
Anchorage Slip: $\Delta_a$			
$(h/b)d_a =$		0.039 in	
Story Drift: $\Delta_s$			
$\Delta_b + \Delta_v + \Delta_n + \Delta_a =$		0.514 in	
Total Deflection: $\Delta$			
$C_d \Delta_s =$		2.06 in	
Deflection Limit: $\Delta_{limit}$			
$0.02(h \times 12) =$		2.40 in	
$\Delta_{limit} > \Delta$		2.40 > 2.06	TRUE

**Use 15/32" APA Rated 32/16 3-Ply Wood Structural Panels -- Structural or Better**  
**With 10d Common Nails**  
**at 6" o.c. Supported Edges**  
**6" o.c. Field**  
**AND**  
**One (1) 4 x 6 SPF Stud Post for All Shearwall Chords**  
**No Blocking Required**

# Foundation

### **Load**

Total Load:  $w_u$

$$D + L + S = 772 \text{ psf}$$

Weight of the foundation:  $W$

$$w_f/b = 1200 \text{ lb/ft}$$

Bearing Pressure:  $q$

$$(P/b + w_f/b)/B = 1972 \text{ lb/ft}^2$$

### **Basement Retaining Wall**

Overconsolidation ratio of soil: OCR = 2

Effective friction angle of soil:  $\Phi'$  = 30 Degrees

Coefficient of Lateral Earth Pressure at Rest:  $K_0$

$$(1 - \sin(\Phi'))(OCR^{\sin \Phi'}) = 0.707$$

Unit Weight of Soil:  $\gamma$  = 127 lb/ft<sup>3</sup>

Height of Wall:  $H$  = 8 ft

Normal Force Acting Between Soil and Wall per Unit

Length of Wall:  $P_0/b$

$$(\gamma)(H^2)(K_0)/2 = 2873 \text{ lb/ft}$$

**Use 12" wide Continuous Footing  
Supported Light Frame design and 8' Deep Basement Retaining Walls**

# LRFD Headers

**Wood Properties**

Spruce-Pine-Fir	No.2
Bending ( $F_b$ )	875 psi
Tension Parallel to Grain ( $F_t$ )	450 psi
Shear Parallel to Grain ( $F_v$ )	135 psi
Compression Perpendicular to Grain ( $F_{c\perp}$ )	425 psi
Compression Parallel to Grain ( $F_c$ )	1,150 psi
Modulus of Elasticity (E)	1,400,000 psi
Modulus of Elasticity ( $E_{min}$ )	510,000 psi

**Factors**

$\lambda$ (Time Effect)	1.00
$\varphi_b$ (Bending Resistance)	0.85
$\varphi_t$ (Tension Resistance)	0.80
$\varphi_v$ (Shear Resistance)	0.75
$\varphi_c$ (Compression Resistance)	0.90
$\varphi_s$ (Stability Resistance)	0.85
$K_F$ (Format Conversion)	
Bending	2.54
Tension	2.70
Shear	2.88
Compression	
Perpendicular to Grain	2.08
Parallel to Grain	2.40
Stability	1.76
$C_F$ (Size)	
2 x 10	
Bending	1.10
Tension	1.10
Compression	1.00
2 x 6	
Bending	1.30
Tension	1.30
Compression	1.10
$C_i$ (Incising)	1.00
$C_L$ (Stability)	1.00
$C_M$ (Wet Service)	1.00
$C_r$ (Repetitive Member)	1.15
$C_t$ (Thermal)	1.00

**LRFD Design Values**

Nominal Bending Design Value: $F_{bn}$	
$F_b (K_F) =$	2,224 psi
	2.22 ksi

Nominal Tension Parallel to Grain Design Value:  $F_{tn}$

$$F_t(K_F) = \begin{array}{r} 1,215 \text{ psi} \\ 1.22 \text{ ksi} \end{array}$$

Nominal Shear Design Value Parallel to Grain:  $F_{vn}$

$$F_v(K_F) = \begin{array}{r} 389 \text{ psi} \\ 0.39 \text{ ksi} \end{array}$$

Nominal Compression Perpendicular to Grain Design Value:  $F_{c \perp n}$

$$F_{c \perp}(K_F) = \begin{array}{r} 884 \text{ psi} \\ 0.88 \text{ ksi} \end{array}$$

Nominal Compression Parallel to Grain Design Value:  $F_{cn}$

$$F_c(K_F) = \begin{array}{r} 2,760 \text{ psi} \\ 2.76 \text{ ksi} \end{array}$$

Nominal Modulus of Elasticity Design Value:  $E_{min n}$

$$E_{min n}(K_F) = \begin{array}{r} 900,000 \text{ psi} \\ 900 \text{ ksi} \end{array}$$

### **Wood Dimensions and Spacing**

$$\text{Header Length: } L = 6.67 \text{ ft}$$

Section Modulus: S

$$\begin{array}{r} 2 \times 6 = 7.56 \text{ in}^3 \\ 2 \times 8 = 14.06 \text{ in}^3 \\ 2 \times 10 = 22.56 \text{ in}^3 \\ 2 \times 12 = 33.06 \text{ in}^3 \end{array}$$

Area: A

$$\begin{array}{r} 2 \times 6 = 8.25 \text{ in}^2 \\ 2 \times 8 = 11.25 \text{ in}^2 \\ 2 \times 10 = 14.25 \text{ in}^2 \\ 2 \times 12 = 17.25 \text{ in}^2 \end{array}$$

Moment of Inertia: I

$$\begin{array}{r} 2 \times 6 = 20.80 \text{ in}^3 \\ 2 \times 8 = 52.73 \text{ in}^3 \\ 2 \times 10 = 107.17 \text{ in}^3 \\ 2 \times 12 = 190.11 \text{ in}^3 \end{array}$$

### **Loads**

Total Load:  $w_u$

$$1.2D + 1.6L + 0.5S = 1088 \text{ lb/ft}$$

Number of Beams Per Header: N 2.00

Shear:  $V_u$

Total:  $(w_{TL} \times L)/2 = 3,627 \text{ lb}$

Per Beam:  $V_u/N = 1,814 \text{ lb}$

Moment:  $M_u$

Total:  $(w_{TL} \times L^2)/8 = 6,046 \text{ ft-lb}$

Per Beam:  $M_u/N = 3,023 \text{ ft-lb}$   
36 in-k

### **Bending**

Adjusted Bending Design Value:  $F'_{bn}$

$F_{bn}(\phi_b)(\lambda)(C_M)(C_t)(C_L)(C_F)(C_i) = 2,079 \text{ psi}$   
2.08 ksi

Required Section Modulus: Req'd S

$M_u/F'_{bn} = 17.45 \text{ in}^3 \Rightarrow \text{Try } 2 \times 10$

Adjusted Moment Resistance:  $M'_n$

$F'_{bn} * S = 50.17 \text{ in-k}$

$M'_n > M_u$  50.17 > 36.28 TRUE

### **Axial**

Adjusted Shear Design Value Parallel to Grain:  $F'_{vn}$

$F_{vn}(\phi_v)(\lambda)(C_M)(C_t)(C_i) = 292 \text{ psi}$   
0.292 ksi

Adjusted Shear Resistance Parallel to Grain:  $V'_n$

$2/3 F'_{vn} A = 2.77 \text{ k}$

$V'_n > V_u$  2,770 > 1,814 TRUE

Adjusted Compression Design Value Perpendicular to Grain:  $F'_{c \perp n}$

$F_{c \perp n}(\phi_c)(\lambda)(C_M)(C_t)(C_i) = 796 \text{ psi}$

Required Bearing Area:  $A_B$

$V_u/F'_{c \perp n} = 2.05 \text{ in}^2$

Minimum Seat Length:  $L_s$

$A_B/\text{Support Thickness} = 1.37 \text{ in}$



Number of Supports Required: N

Support Thickness > L<sub>s</sub>

1 Support 1.50 > 1.37 TRUE

Total Bearing Area: A<sub>T</sub>

(Support Thickness) x L<sub>s</sub> = 2.25 in<sup>2</sup>

Actual Compression Perpendicular to Grain: f'<sub>c⊥n</sub>

V<sub>u</sub>/A<sub>T</sub> = 806 psi

F'<sub>c⊥n</sub> > f'<sub>c⊥n</sub> 884 > 806 TRUE

### **Deflection**

Adjusted Modulus of Elasticity: E'

E(C<sub>M</sub>)(C<sub>t</sub>)(C<sub>i</sub>) = 1,400,000 psi

Actual Deflection Under Snow Load: Δ<sub>S</sub>

5w<sub>s</sub>L<sup>4</sup>/384E'I = 0.01 in

Allowable Deflection Under Snow Load: Allow. Δ<sub>S</sub>

L/360 = 0.22 in

Allow. Δ<sub>S</sub> > Δ<sub>S</sub> 0.22 > 0.01 TRUE

Actual Deflection Under Total Load: Δ<sub>u</sub>

Δ<sub>S</sub>(w<sub>u</sub>/w<sub>s</sub>) = 0.18 in

Allowable Deflection: Allow. Δ<sub>u</sub>

L/240 = 0.33 in

Allow. Δ<sub>u</sub> > Δ<sub>u</sub> 0.33 > 0.18 TRUE

### **Supports**

Adjusted Compression Parallel to Grain Design Value: F'<sub>cn</sub>

F<sub>cn</sub>(φ<sub>c</sub>)(λ)(C<sub>M</sub>)(C<sub>t</sub>)(C<sub>i</sub>)(C<sub>p</sub>)(C<sub>F</sub>) = 2,732 psi

Actual Compressive Stress Parallel to Grain: f'<sub>cn</sub>

V<sub>u</sub>/A<sub>T</sub> = 806 psi

F'<sub>cn</sub> > f'<sub>cn</sub> 2,732 > 806 TRUE

**Use Two (2) 2 x 10 SPF No. 2 or Better for All Headers  
And  
One (1) 2 x 6 SPF No. 2 or Better for All Header Support Jacks  
MC ≤ 19%**

# LRFD Connections

# Connection Values

## Dowels

### Nails

16d Common Nails:

D (Diameter)	0.168 in
L (Length)	3.50 in
$F_{yb}$ (Bending Yield Strength of Fastener)	90.0 ksi
$K_D$ (Reduction Coefficient for Fasteners with $D < 1/4"$ )	2.20

10d Common Nails:

D (Diameter)	0.148 in
L (Length)	3.00 in
$F_{yb}$ (Bending Yield Strength of Fastener)	90.0 ksi
$K_D$ (Reduction Coefficient for Fasteners with $D < 1/4"$ )	2.20

8d Common Nails:

D (Diameter)	0.131 in
L (Length)	2.50 in
$F_{yb}$ (Bending Yield Strength of Fastener)	100 ksi
$K_D$ (Reduction Coefficient for Fasteners with $D < 1/4"$ )	2.20

### Bolts

A307 Bolt:

D (Diameter)	0.75 in
L (Length)	5.00 in
$F_{yb}$ (Bending Yield Strength of Fastener)	45.0 ksi

## Members

### Spruce-Pine-Fir Wood

$F_e$ (Dowel Bearing Strength)	3,500 psi
G (Specific Gravity)	0.42

### Concrete

$F_e$ (Dowel Bearing Strength)	7,500 psi
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## Adjustment Factors

$\lambda$ (Time Effect)	1.00
$\varphi_z$ (Resistance)	0.65

$K_F$ (Format Conversion)	3.32
$C_{di}$ (Diaphragm)	1.00
$C_{eg}$ (End Grain)	
End Nail Connection	0.67
Non-End Nail Connection	1.00
$C_g$ (Group)	1.00
$C_i$ (Incising)	1.00
$C_M$ (Wet Service)	1.00
$C_t$ (Thermal)	1.00
$C_{tn}$ (Toenail)	
Toenail Connection	0.83
Non-Toenail Connection	1.00
$C_\Delta$ (Geometry)	1.00

## Connection Glossary

$D$	Diameter (in)
$l$	Length of Nail (in)
$F_{yb}$	Bending Yield Strength of Fastener (psi)
$t_m$	Thickness of Main Member (in)
$t_s$	Thickness of Side Member (in)
$l_s$	Dowel Bearing Length of Fastener in Side Member (in)
$l_m$	Dowel Bearing Length of Fastener in Main Member (in)
$P_L$	Toenail Penetration of Nail in Main Member (in)
$P$	Penetration Of Nail in Main Member (in)
$G_m$	Specific Gravity of Wood Member
$F_{em}$	Dowel Bearing Strength of Main Member (psi)
$F_{es}$	Dowel Bearing Strength of Side Member (psi)
$K_D$	Reduction Factor for Fasteners with $D < 1/4"$
$\theta$	Maximum Angle of Load to Grain for Any Member in Connection ( $0 < \theta < 90$ )
$R_e$	$\frac{F_{em}}{F_{es}}$
$R_t$	$\frac{l_m}{l_s}$
$k_1$	$\frac{\sqrt{R_e + 2R_e^2(1 + R_t + R_t^2) + R_t^2R_e^3} - R_e(1 + R_t)}{1 + R_e}$
$k_2$	$-1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}l_m^2}}$
$k_3$	$-1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(2 + R_e)D^2}{3F_{em}l_s^2}}$
$K_\theta$	$1 + \frac{\theta}{360}$

**Rim Joist to Wall Plate**

D =	0.148	Mode I <sub>m</sub> :	
l =	3.00	$(D * l_m * F_{em}) / K_D$	360 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	3.00	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	$(D * l_s * F_{es}) / K_D$	225 lbs
l <sub>s</sub> =	1.00		
l <sub>m</sub> =	1.60	Mode II:	
P <sub>L</sub> =	1.60	$(k_1 * D * l_s * F_{es}) / K_D$	127 lbs
P =	1.50		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	$(k_2 * D * l_m * F_{em}) / [(1+2R_e)K_D]$	134 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	$(k_3 * D * l_s * F_{em}) / [(2+R_e)K_D]$	96 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1+R_e)]$	100 lbs
R <sub>t</sub> =	1.60		
k <sub>1</sub> =	0.56	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.11	Smallest Value from Modes I-IV	96 lbs
k <sub>3</sub> =	1.28		
		Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
		$Z(K_F)(\phi_z)(\lambda)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	172 lbs
		Req'd Number of Nails: N	
		Load/Z'	47.0
		Req'd Spacing:	
		L/N	4.1 in

**Use 10d Common Nails @ 4" o.c. for all Rim Joist to Plate Connections, Toenailed**

**Ceiling Joist to Wall Plate**

D =	0.162	Mode I <sub>m</sub> :	
l =	3.50	(D * I <sub>m</sub> * F <sub>em</sub> )/K <sub>D</sub>	493 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	3.00	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	(D * I <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	288 lbs
l <sub>s</sub> =	1.17		
l <sub>m</sub> =	2.00	Mode II:	
P <sub>L</sub> =	1.86	(k <sub>1</sub> * D * I <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	171 lbs
P =	2.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	(k <sub>2</sub> * D * I <sub>m</sub> * F <sub>em</sub> )/[(1+2R <sub>e</sub> )K <sub>D</sub> ]	179 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	(k <sub>3</sub> * D * I <sub>s</sub> * F <sub>em</sub> )/[(2+R <sub>e</sub> )K <sub>D</sub> ]	119 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	D <sup>2</sup> /K <sub>D</sub> * Sqrt[(2 * F <sub>em</sub> * F <sub>yb</sub> )/3 * (1+R <sub>e</sub> )]	120 lbs
R <sub>t</sub> =	1.71		
k <sub>1</sub> =	0.60	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.09	Smallest Value from Modes I-IV	119 lbs
k <sub>3</sub> =	1.24		
		Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
		Z(K <sub>F</sub> )(φ <sub>z</sub> )(λ)(C <sub>M</sub> )(C <sub>t</sub> )(C <sub>eg</sub> )(C <sub>di</sub> )(C <sub>tn</sub> )	214 lbs
		Req'd Number of Nails: N	
		Load/Z'	3.4

**Use 4 16d Common Nails for all Floor Joist to Plate Connections, Toenailed**

**Top Plate Splice**

D = 0.162	Mode I <sub>m</sub> :	
l = 3.50	$(D * l_m * F_{em}) / K_D$	493 lbs
F <sub>yb</sub> = 90,000		
t <sub>m</sub> = 5.50	Mode I <sub>s</sub> :	
t <sub>s</sub> = 1.50	$(D * l_s * F_{es}) / K_D$	370 lbs
l <sub>s</sub> = 1.50		
l <sub>m</sub> = 2.00	Mode II:	
P <sub>L</sub> = NONE	$(k_1 * D * l_s * F_{es}) / K_D$	182 lbs
P = 2.00		
G <sub>m</sub> = 0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> = 3,350	$(k_2 * D * l_m * F_{em}) / [(1+2R_e)K_D]$	179 lbs
F <sub>es</sub> = 3,350		
K <sub>D</sub> = 2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> = 1.00	$(k_3 * D * l_s * F_{em}) / [(2+R_e)K_D]$	142 lbs
1+R <sub>e</sub> = 2.00		
1+2R <sub>e</sub> = 3.00	Mode IV:	
2+R <sub>e</sub> = 3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1+R_e)]$	120 lbs
R <sub>t</sub> = 1.33		
k <sub>1</sub> = 0.49	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> = 1.09	Smallest Value from Modes I-IV	120 lbs
k <sub>3</sub> = 1.15		
	Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
	$Z(K_F)(\phi_z)(\lambda)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	258 lbs
	Req'd Number of Nails: N	
	Load/Z'	13.2

**Use 14 16d Common Nail Between all Splice Points, Face Nailed**



**Band Joist to Sole Plate**

D =	0.162	Mode I <sub>m</sub> :	
l =	3.50	(D * l <sub>m</sub> * F <sub>em</sub> )/K <sub>D</sub>	493 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	5.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	(D * l <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	370 lbs
l <sub>s</sub> =	1.50		
l <sub>m</sub> =	2.00	Mode II:	
P <sub>L</sub> =	NONE	(k <sub>1</sub> * D * l <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	182 lbs
P =	2.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	(k <sub>2</sub> * D * l <sub>m</sub> * F <sub>em</sub> )/[(1+2R <sub>e</sub> )K <sub>D</sub> ]	179 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	(k <sub>3</sub> * D * l <sub>s</sub> * F <sub>em</sub> )/[(2+R <sub>e</sub> )K <sub>D</sub> ]	142 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	D <sup>2</sup> /K <sub>D</sub> * Sqrt[(2 * F <sub>em</sub> * F <sub>yb</sub> )/3 * (1+R <sub>e</sub> )]	120 lbs
R <sub>t</sub> =	1.33		
k <sub>1</sub> =	0.49	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.09	Smallest Value from Modes I-IV	120 lbs
k <sub>3</sub> =	1.15		
		Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
		Z(K <sub>F</sub> )(φ <sub>z</sub> )(λ)(C <sub>M</sub> )(C <sub>t</sub> )(C <sub>eg</sub> )(C <sub>di</sub> )(C <sub>tn</sub> )	258 lbs
		Req'd Number of Nails: N	
		Load/Z'	31.28
			30.0
		Req'd Spacing:	
		L/N	6.4 in

**Use 16d Common Nail at 6" o.c. for all Sole Plate to Band Joist Connections, Face Nailed**

**Continuous Header to Stud**

D =	0.131	Mode I <sub>m</sub> :	
l =	2.50	$(D * l_m * F_{em}) / K_D$	266 lbs
F <sub>yb</sub> =	100,000		
t <sub>m</sub> =	1.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	$(D * l_s * F_{es}) / K_D$	166 lbs
l <sub>s</sub> =	0.83		
l <sub>m</sub> =	1.33	Mode II:	
P <sub>L</sub> =	1.33	$(k_1 * D * l_s * F_{es}) / K_D$	93 lbs
P =	1.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	$(k_2 * D * l_m * F_{em}) / [(1+2R_e)K_D]$	101 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	$(k_3 * D * l_s * F_{em}) / [(2+R_e)K_D]$	74 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1+R_e)]$	82 lbs
R <sub>t</sub> =	1.60		
k <sub>1</sub> =	0.56	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.14	Smallest Value from Modes I-IV	74 lbs
k <sub>3</sub> =	1.34		
		Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
		$Z(K_F)(\phi_z)(\lambda)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	133 lbs
		Req'd Number of Nails: N	
		Load/Z'	2.5

**Use 4 8d Common Nail for All Header to Stud Connection, Toenailed**

**Band Joist to Joists**

D = 0.162	Mode I <sub>m</sub> :	
l = 3.50	$(D * I_m * F_{em}) / K_D$	493 lbs
F <sub>yb</sub> = 90,000		
t <sub>m</sub> = 3.00	Mode I <sub>s</sub> :	
t <sub>s</sub> = 1.50	$(D * I_s * F_{es}) / K_D$	288 lbs
l <sub>s</sub> = 1.17		
l <sub>m</sub> = 2.00	Mode II:	
P <sub>L</sub> = 1.86	$(k_1 * D * I_s * F_{es}) / K_D$	171 lbs
P = 2.00		
G <sub>m</sub> = 0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> = 3,350	$(k_2 * D * I_m * F_{em}) / [(1+2R_e)K_D]$	179 lbs
F <sub>es</sub> = 3,350		
K <sub>D</sub> = 2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> = 1.00	$(k_3 * D * I_s * F_{em}) / [(2+R_e)K_D]$	119 lbs
1+R <sub>e</sub> = 2.00		
1+2R <sub>e</sub> = 3.00	Mode IV:	
2+R <sub>e</sub> = 3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1+R_e)]$	120 lbs
R <sub>t</sub> = 1.71		
k <sub>1</sub> = 0.60	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> = 1.09	Smallest Value from Modes I-IV	119 lbs
k <sub>3</sub> = 1.24		
	Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
	$Z(K_F)(\phi_z)(\lambda)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	258 lbs
	Req'd Number of Nails: N	
	Load/Z'	3.3

**Use 4 16d Common Nail for Hip Rafter to Ridge Beam Connections, Face Nailed**

**Stud to Wall Plate**

D =	0.162	Mode I <sub>m</sub> :	
l =	3.50	(D * I <sub>m</sub> * F <sub>em</sub> ) / K <sub>D</sub>	493 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	3.00	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	(D * I <sub>s</sub> * F <sub>es</sub> ) / K <sub>D</sub>	288 lbs
l <sub>s</sub> =	1.17		
l <sub>m</sub> =	2.00	Mode II:	
P <sub>L</sub> =	1.86	(k <sub>1</sub> * D * I <sub>s</sub> * F <sub>es</sub> ) / K <sub>D</sub>	171 lbs
P =	2.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	(k <sub>2</sub> * D * I <sub>m</sub> * F <sub>em</sub> ) / [(1+2R <sub>e</sub> )K <sub>D</sub> ]	179 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	(k <sub>3</sub> * D * I <sub>s</sub> * F <sub>em</sub> ) / [(2+R <sub>e</sub> )K <sub>D</sub> ]	119 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	D <sup>2</sup> / K <sub>D</sub> * Sqrt[(2 * F <sub>em</sub> * F <sub>yb</sub> ) / 3 * (1+R <sub>e</sub> )]	120 lbs
R <sub>t</sub> =	1.71		
k <sub>1</sub> =	0.60	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.09	Smallest Value from Modes I-IV	126 lbs
k <sub>3</sub> =	1.24		
		Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
		Z(K <sub>F</sub> )(φ <sub>z</sub> )(λ)(C <sub>M</sub> )(C <sub>t</sub> )(C <sub>eg</sub> )(C <sub>di</sub> )(C <sub>tn</sub> )	182 lbs
		Load on One Stud Connection:	
		Load/Stud Spacing	232.8 lbs
		Req'd Number of Nails: N	
		Load/Z'	1.3

**Use 2 16d Common Nails for all Stud to Plate Connections, End Nailed**

**Floor Joist to Wall Plate**

D =	0.162	Mode I <sub>m</sub> :	
l =	3.50	$(D * l_m * F_{em}) / K_D$	493 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	3.00	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	$(D * l_s * F_{es}) / K_D$	288 lbs
l <sub>s</sub> =	1.17		
l <sub>m</sub> =	2.00	Mode II:	
P <sub>L</sub> =	1.86	$(k_1 * D * l_s * F_{es}) / K_D$	171 lbs
P =	2.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	$(k_2 * D * l_m * F_{em}) / [(1+2R_e)K_D]$	179 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	$(k_3 * D * l_s * F_{em}) / [(2+R_e)K_D]$	119 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1+R_e)]$	120 lbs
R <sub>t</sub> =	1.71		
k <sub>1</sub> =	0.60	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.09	Smallest Value from Modes I-IV	119 lbs
k <sub>3</sub> =	1.24		
		Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
		$Z(K_F)(\phi_z)(\lambda)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	214 lbs
		Req'd Number of Nails: N	
		Load/Z'	3.9

**Use 4 16d Common Nails for all Floor Joist to Plate Connections, Toenailed**

**Double Studs**

D =	0.148	Mode I <sub>m</sub> :	
l =	3.00	(D * l <sub>m</sub> * F <sub>em</sub> )/K <sub>D</sub>	338 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	1.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	(D * l <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	338 lbs
l <sub>s</sub> =	1.50		
l <sub>m</sub> =	1.50	Mode II:	
P <sub>L</sub> =	NONE	(k <sub>1</sub> * D * l <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	140 lbs
P =	1.50		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	(k <sub>2</sub> * D * l <sub>m</sub> * F <sub>em</sub> )/[(1+2R <sub>e</sub> )K <sub>D</sub> ]	127 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	(k <sub>3</sub> * D * l <sub>s</sub> * F <sub>em</sub> )/[(2+R <sub>e</sub> )K <sub>D</sub> ]	127 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	D <sup>2</sup> /K <sub>D</sub> * Sqrt[(2 * F <sub>em</sub> * F <sub>yb</sub> )/3 * (1+R <sub>e</sub> )]	100 lbs
R <sub>t</sub> =	1.00		
k <sub>1</sub> =	0.41	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.13	Smallest Value from Modes I-IV	100 lbs
k <sub>3</sub> =	1.13		
		Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
		Z(K <sub>F</sub> )(φ <sub>z</sub> )(λ)(C <sub>M</sub> )(C <sub>t</sub> )(C <sub>eg</sub> )(C <sub>di</sub> )(C <sub>tn</sub> )	216 lbs
		Load on One Stud Connection, P:	337.8 lbs
		Req'd Number of Nails: N	
		Load/Z'	1.6
			3
		Req'd Spacing:	
		L/N	16.0 in

**Use 10d Common Nail at 16" o.c. for all Double Stud Connections, Face Nailed**

**Continued Header**

D =	0.148	Mode I <sub>m</sub> :	
l =	3.00	(D * I <sub>m</sub> * F <sub>em</sub> )/K <sub>D</sub>	338 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	1.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	(D * I <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	338 lbs
l <sub>s</sub> =	1.50		
l <sub>m</sub> =	1.50	Mode II:	
P <sub>L</sub> =	NONE	(k <sub>1</sub> * D * I <sub>s</sub> * F <sub>es</sub> )/K <sub>D</sub>	140 lbs
P =	1.50		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	(k <sub>2</sub> * D * I <sub>m</sub> * F <sub>em</sub> )/[(1+2R <sub>e</sub> )K <sub>D</sub> ]	127 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	(k <sub>3</sub> * D * I <sub>s</sub> * F <sub>em</sub> )/[(2+R <sub>e</sub> )K <sub>D</sub> ]	127 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	D <sup>2</sup> /K <sub>D</sub> * Sqrt[(2 * F <sub>em</sub> * F <sub>yb</sub> )/3 * (1+R <sub>e</sub> )]	100 lbs
R <sub>t</sub> =	1.00		
k <sub>1</sub> =	0.41	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.13	Smallest Value from Modes I-IV	100 lbs
k <sub>3</sub> =	1.13		
		Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
		Z(K <sub>F</sub> )(φ <sub>z</sub> )(λ)(C <sub>M</sub> )(C <sub>t</sub> )(C <sub>eg</sub> )(C <sub>di</sub> )(C <sub>tn</sub> )	216 lbs
		Req'd Number of Nails: N	
		Load/Z'	3.27
			5
		Req'd Spacing:	
		L/N	7.2 in

**Use 10d Common Nail at 8" o.c. for all Sole Plate to Band Joist Connections, Face Nailed**

**Rafter to Ridge Beam/Jack Rafters to Hip Rafters**

D =	0.162	Mode I <sub>m</sub> :	
l =	3.50	$(D * l_m * F_{em}) / K_D$	460 lbs
F <sub>yb</sub> =	90,000		
t <sub>m</sub> =	1.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	$(D * l_s * F_{es}) / K_D$	288 lbs
l <sub>s</sub> =	1.17		
l <sub>m</sub> =	1.86	Mode II:	
P <sub>L</sub> =	1.86	$(k_1 * D * l_s * F_{es}) / K_D$	162 lbs
P =	2.00		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	3,350	$(k_2 * D * l_m * F_{em}) / [(1+2R_e)K_D]$	168 lbs
F <sub>es</sub> =	3,350		
K <sub>D</sub> =	2.20	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.00	$(k_3 * D * l_s * F_{em}) / [(2+R_e)K_D]$	119 lbs
1+R <sub>e</sub> =	2.00		
1+2R <sub>e</sub> =	3.00	Mode IV:	
2+R <sub>e</sub> =	3.00	$D^2 / K_D * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1+R_e)]$	120 lbs
R <sub>t</sub> =	1.60		
k <sub>1</sub> =	0.56	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.10	Smallest Value from Modes I-IV	119 lbs
k <sub>3</sub> =	1.24		
		Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
		$Z(K_F)(\phi_z)(\lambda)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	214 lbs
		Req'd Number of Nails: N	
		Load/Z'	3.0

**Use 3 16d Common Nail for Rafter to Ridge Beam/ Jack Rafter to Hip Rafter Connections, Toenailed**



**Sole Plate to Foundation**

D =	0.75	Mode I <sub>m</sub> :	
l =	5.00	$(D * l_m * F_{em}) / 4K_{\theta}$	4,922 lbs
F <sub>yb</sub> =	45,000		
t <sub>m</sub> =	3.50	Mode I <sub>s</sub> :	
t <sub>s</sub> =	1.50	$(D * l_s * F_{es}) / 4K_{\theta}$	2,109 lbs
l <sub>s</sub> =	1.50		
l <sub>m</sub> =	3.50	Mode II:	
θ =	0.00	$(k_1 * D * l_s * F_{es}) / 3.6K_{\theta}$	1,671 lbs
P =	3.50		
G <sub>m</sub> =	0.42	Mode III <sub>m</sub> :	
F <sub>em</sub> =	7,500	$(k_2 * D * l_m * F_{em}) / [3.2(1+2R_e)K_{\theta}]$	2,466 lbs
F <sub>es</sub> =	4,704		
K <sub>θ</sub> =	1.00	Mode III <sub>s</sub> :	
R <sub>e</sub> =	1.59	$(k_3 * D * l_s * F_{em}) / [3.2(2+R_e)K_{\theta}]$	1,186 lbs
1+R <sub>e</sub> =	2.59		
1+2R <sub>e</sub> =	4.19	Mode IV:	
2+R <sub>e</sub> =	3.59	$D^2 / 3.2K_{\theta} * \text{Sqrt}[(2 * F_{em} * F_{yb}) / 3 * (1+R_e)]$	1,637 lbs
R <sub>t</sub> =	2.33		
k <sub>1</sub> =	1.14	Lateral Design Value for Single Fastener: Z	
k <sub>2</sub> =	1.44	Smallest Value from Modes I-IV	1,186 lbs
k <sub>3</sub> =	1.62		
		Adjusted Design Value for Single Fastener: Z' <sub>n</sub>	
		$Z(K_F)(\phi_z)(\lambda)(C_M)(C_t)(C_{eg})(C_{di})(C_{tn})$	1,898
		Req'd Number of Anchor Bolts: N	
		Load/Z'	9.22
		Req'd Number of Anchor Bolts: N	
		Load/Z'	5.90

**Use Ten (10) 3/4" Diameter Anchor Bolts for Sole Plate to Foundation Connections for 50 ft Walls  
AND  
Six (6) 3/4" Diameter Anchors Bolts for 32 ft Walls**

# LRFD Fastener Schedule

<b><u>Connection</u></b>	<b><u>Fastening</u></b>	<b><u>Location</u></b>
Rim Joist to Wall Plate	10d Common Nails at 4" o.c.	Toenail
Ceiling Joist to Wall Plate	4 - 16d Common Nails	Toenail
Top Plate Splice	14 - 16d Common Nails	Lap Splice
Band Joist to Sole Plate	16d Common Nails at 6" o.c.	Face Nail
Continuous Header to Stud	4 - 8d Common Nails	Toenail
Band Joist to Joists	4 - 16d Common Nails	Face Nail
Stud to Wall Plate	2 - 16d Common Nails	End Nail
Floor Joist to Wall Plate	4 - 16d Common Nails	Toenail
Double Studs	10d Common Nails at 16" o.c.	Face Nail
Continued Header	10d Common Nails at 8" o.c.	Face Nail
Rafter to Ridge Beam/Jack Rafters to Hip Rafters	3 - 16d Common Nails	Toenail
Sole Plate to Foundation	10 - 3/4" Diameter Anchor Bolts for Long Walls 6 - 3/" Diameter Anchor Bolts for Short Walls	Face Nail

## Appendix C: Tables, Figures, & Information Used to Complete Calculations

Table 11: ASCE Table 1-1: Occupancy Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Nature of Occupancy	Occupancy Category
<p>Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to:</p> <ul style="list-style-type: none"> <li>• Agricultural facilities</li> <li>• Certain temporary facilities</li> <li>• Minor storage facilities</li> </ul>	I
<p>All buildings and other structures except those listed in Occupancy Categories I, III, and IV</p>	II
<p>Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to:</p> <ul style="list-style-type: none"> <li>• Buildings and other structures where more than 300 people congregate in one area</li> <li>• Buildings and other structures with daycare facilities with a capacity greater than 150</li> <li>• Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250</li> <li>• Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities</li> <li>• Health care facilities with a capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities</li> <li>• Jails and detention facilities</li> </ul> <p>Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to:</p> <ul style="list-style-type: none"> <li>• Power generating stations<sup>a</sup></li> <li>• Water treatment facilities</li> <li>• Sewage treatment facilities</li> <li>• Telecommunication centers</li> </ul> <p>Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.</p> <p>Buildings and other structures containing toxic or explosive substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the toxic or explosive substances does not pose a threat to the public.</p>	III
<p>Buildings and other structures designated as essential facilities, including, but not limited to:</p> <ul style="list-style-type: none"> <li>• Hospitals and other health care facilities having surgery or emergency treatment facilities</li> <li>• Fire, rescue, ambulance, and police stations and emergency vehicle garages</li> <li>• Designated earthquake, hurricane, or other emergency shelters</li> <li>• Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response</li> <li>• Power generating stations and other public utility facilities required in an emergency</li> <li>• Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency</li> <li>• Aviation control towers, air traffic control centers, and emergency aircraft hangars</li> <li>• Water storage facilities and pump structures required to maintain water pressure for fire suppression</li> <li>• Buildings and other structures having critical national defense functions</li> </ul> <p>Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction.</p> <p>Buildings and other structures containing highly toxic substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the highly toxic substances does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.</p>	IV

<sup>a</sup>Cogeneration power plants that do not supply power on the national grid shall be designated Occupancy Category II.

**TABLE C3-1 MINIMUM DESIGN DEAD LOADS\***

Component	Load (psf)	Component	Load (psf)
<b>CEILINGS</b>		Decking, 2-in. wood (Douglas fir)	5
Acoustical Fiber Board	1	Decking, 3-in. wood (Douglas fir)	8
Gypsum board (per 1/8-in. thickness)	0.55	Fiberboard, 1/2-in.	0.75
Mechanical duct allowance	4	Gypsum sheathing, 1/2-in.	2
Plaster on tile or concrete	5	Insulation, roof boards (per inch thickness)	
Plaster on wood lath	8	Cellular glass	0.7
Suspended steel channel system	2	Fibrous glass	1.1
Suspended metal lath and cement plaster	15	Fiberboard	1.5
Suspended metal lath and gypsum plaster	10	Perlite	0.8
Wood furring suspension system	2.5	Polystyrene foam	0.2
<b>COVERINGS, ROOF, AND WALL</b>		Urethane foam with skin	0.5
Asbestos-cement shingles	4	Plywood (per 1/8-in. thickness)	0.4
Asphalt shingles	2	Rigid insulation, 1/2-in.	0.75
Cement tile	16	Skylight, metal frame, 3/8-in. wire glass	8
Clay tile (for mortar add 10 psf)		Slate, 3/16-in.	7
Book tile, 2-in.	12	Slate, 1/4-in.	10
Book tile, 3-in.	20	Waterproofing membranes:	
Ludowici	10	Bituminous, gravel-covered	5.5
Roman	12	Bituminous, smooth surface	1.5
Spanish	19	Liquid applied	1
Composition:		Single-ply, sheet	0.7
Three-ply ready roofing	1	Wood sheathing (per inch thickness)	3
Four-ply felt and gravel	5.5	Wood shingles	3
Five-ply felt and gravel	6	<b>FLOOR FILL</b>	
Copper or tin	1	Cinder concrete, per inch	9
Corrugated asbestos-cement roofing	4	Lightweight concrete, per inch	8
Deck, metal, 20 gage	2.5	Sand, per inch	8
Deck, metal, 18 gage	3	Stone concrete, per inch	12

\*Weights of masonry include mortar but not plaster. For plaster, add 5 lb/ft<sup>2</sup> for each face plastered. Values given represent averages. In some cases there is a considerable range of weight for the same construction.

(continued)

**Figure 17: Weights of Building Materials**

**TABLE C9-1 continued  
MINIMUM DESIGN DEAD LOADS\***

Component	Load (psf)	Component	Load (psf)
<b>FLOORS AND FLOOR FINISHES</b>		Windows, glass, frame, and sash	8
Asphalt block (2-in.), 1/2-in. mortar	30	Clay brick wythes:	
Cement finish (1-in.) on stone-concrete fill	32	4 in.	39
Ceramic or quarry tile (3/4-in.) on 1/2-in. mortar bed	16	8 in.	79
Ceramic or quarry tile (3/4-in.) on 1-in. mortar bed	23	12 in.	115
Concrete fill finish (per inch thickness)	12	16 in.	155
Hardwood flooring, 7/7-in.	4	Hollow concrete masonry unit wythes:	
Linoleum or asphalt tile, 1/4-in.	1	Wythe thickness (in inches)	4      6      8      10
Marble and mortar on stone-concrete fill	33	Density of unit (105 pcf)	
Slate (per mm thickness)	15	No grout	22      24      31      37
Solid flat tile on 1-in. mortar base	23	48 in. o.c.	29      38      47      55
Subflooring, 3/4-in.	3	40 in. o.c.      grout	30      40      49      57
Terrazzo (1-1/2-in.) directly on slab	19	32 in. o.c.      spacing	32      42      52      61
Terrazzo (1-in.) on stone-concrete fill	32	24 in. o.c.	34      46      57      67
Terrazzo (1-in.), 2-in. stone concrete	32	16 in. o.c.	40      53      66      79
Wood block (3-in.) on mastic, no fill	10	Full grout	55      75      95
Wood block (3-in.) on 1/2-in. mortar base	16	Density of unit (125 pcf)	
<b>FLOORS, WOOD-JOIST (NO PLASTER)</b>		No grout	26      28      36      44
<b>DOUBLE WOOD FLOOR</b>		48 in. o.c.	33      44      54      62
Joint sizes	12-in.      16-in.      24-in.	40 in. o.c.      grout	34      45      56      65
spacing	spacing	32 in. o.c.      spacing	36      47      58      68
(in.)	(1b/ft <sup>2</sup> )	24 in. o.c.	39      51      63      75
2 × 6	6	16 in. o.c.	44      59      73      87
2 × 8	6	Full grout	59      81      102
2 × 10	7	Density of unit (135 pcf)	
2 × 12	8	No grout	29      30      39      47
<b>FRAME PARTITIONS</b>		48 in. o.c.	36      47      57      66
Movable steel partitions	4	40 in. o.c.      grout	37      48      59      69
Wood or steel studs, 1/2-in. gypsum board each side	8	32 in. o.c.      spacing	38      50      62      72
Wood studs, 2 × 4, unplastered	4	24 in. o.c.	41      54      67      78
Wood studs, 2 × 4, plastered one side	12	16 in. o.c.	46      61      76      90
Wood studs, 2 × 4, plastered two sides	20	Full grout	62      83      105
<b>FRAME WALLS</b>		Solid concrete masonry unit wythes (incl. concrete brick):	
Exterior stud walls:		Wythe thickness (in mm)	4      6      8      10
2 × 4 @ 16-in., 5/8-in. gypsum, insulated, 3/8-in. siding	11	Density of unit (105 pcf)	32      51      69      87
2 × 6 @ 16-in., 5/8-in. gypsum, insulated, 3/8-in. siding	12	Density of unit (125 pcf)	38      60      81      102
Exterior stud walls with brick veneer	48	Density of unit (135 pcf)	41      64      87      110

\*Weights of masonry include mortar but not plaster. For plaster, add 5 lb/ft<sup>2</sup> for each face plastered. Values given represent averages. In some cases there is a considerable range of weight for the same construction.

(continued)

Figure 17: Weights of Building Materials Cont.

Table 12: ASCE Table 4-1: Minimum Uniformly Distributed Live Loads,  $L_o$ , and Minimum Concentrated Live Loads

Occupancy or Use	Uniform psf (kN/m <sup>2</sup> )	Conc. lb (kN)
Apartments (see <i>Residential</i> )		
Access floor systems		
Office use	50 (2.4)	2,000 (8.9)
Computer use	100 (4.79)	2,000 (8.9)
Armories and drill rooms	150 (7.18)	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)	
Lobbies	100 (4.79)	
Movable seats	100 (4.79)	
Platforms (assembly)	100 (4.79)	
Stage floors	150 (7.18)	
Balconies (exterior)	100 (4.79)	
On one- and two-family residences only, and not exceeding 100 ft <sup>2</sup> (9.3 m <sup>2</sup> )	60 (2.87)	
Bowling alleys, poolrooms, and similar recreational areas	75 (3.59)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dance halls and ballrooms	100 (4.79)	
Decks (patio and roof)		
Same as area served, or for the type of occupancy accommodated		
Dining rooms and restaurants	100 (4.79)	
Dwellings (see <i>Residential</i> )		
Elevator machine room grating (on area of 4 in. <sup>2</sup> [2,580 mm <sup>2</sup> ])		300 (1.33)
Finish light floor plate construction (on area of 1 in. <sup>2</sup> [645 mm <sup>2</sup> ])		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders		See Section 4.4
Garages (passenger vehicles only)	40 (1.92) <sup>a,b</sup>	
Trucks and buses		
Grandstands (see <i>Stadiums and arenas, Bleachers</i> )		
Gymnasiums—main floors and balconies	100 (4.79)	
Handrails, guardrails, and grab bars		See Section 4.4
Hospitals		
Operating rooms, laboratories	60 (2.87)	1,000 (4.45)
Patient rooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Hotels (see <i>Residential</i> )		
Libraries		
Reading rooms	60 (2.87)	1,000 (4.45)
Stack rooms	150 (7.18) <sup>c</sup>	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Manufacturing		
Light	125 (6.00)	2,000 (8.90)
Heavy	250 (11.97)	3,000 (13.40)
Marquees	75 (3.59)	
Office Buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first-floor corridors	100 (4.79)	2,000 (8.90)
Offices	50 (2.40)	2,000 (8.90)
Corridors above first floor	80 (3.83)	2,000 (8.90)
Penal Institutions		
Cell blocks	40 (1.92)	
Corridors	100 (4.79)	
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	10 (0.48)	
Uninhabitable attics with storage	20 (0.96)	
Habitable attics and sleeping areas	30 (1.44)	
All other areas except stairs and balconies	40 (1.92)	
Hotels and multifamily houses		
Private rooms and corridors serving them	40 (1.92)	
Public rooms and corridors serving them	100 (4.79)	
Reviewing stands, grandstands, and bleachers	100 (4.79) <sup>d</sup>	

Occupancy or Use	Uniform psf (kN/m <sup>2</sup> )	Conc. lb (kN)
Roofs		
Ordinary flat, pitched, and curved roofs	20 (0.96) <sup>h</sup>	
Roofs used for promenade purposes	60 (2.87)	
Roofs used for roof gardens or assembly purposes	100 (4.79)	
Roofs used for other special purposes	<sup>i</sup>	<sup>i</sup>
Awnings and canopies		
Fabric construction supported by a lightweight rigid skeleton structure	5 (0.24) nonreducible	
All other construction	20 (0.96)	
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000 (8.9)
All other occupancies		300 (1.33)
All roof surfaces subject to maintenance workers		300 (1.33)
Schools		
Classrooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
First-floor corridors	100 (4.79)	1,000 (4.45)
Scuttles, skylight ribs, and accessible ceilings		200 (0.89)
Sidewalks, vehicular driveways, and yards subject to trucking	250 (11.97) <sup>g</sup>	8,000 (35.60) <sup>g</sup>
Stadiums and arenas		
Bleachers	100 (4.79) <sup>d</sup>	
Fixed seats (fastened to floor)	60 (2.87) <sup>d</sup>	
Stairs and exit ways	100 (4.79)	<sup>e</sup>
One- and two-family residences only	40 (1.92)	
Storage areas above ceilings	20 (0.96)	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	125 (6.00)	
Heavy	250 (11.97)	
Stores		
Retail		
First floor	100 (4.79)	1,000 (4.45)
Upper floors	75 (3.59)	1,000 (4.45)
Wholesale, all floors	125 (6.00)	1,000 (4.45)
Vehicle barriers	See Section 4.4	
Walkways and elevated platforms (other than exit ways)	60 (2.87)	
Yards and terraces, pedestrian	100 (4.79)	

<sup>h</sup>Floors in garages or portions of a building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 4-1 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 lb (13.35 kN) acting on an area of 4.5 in. by 4.5 in. (114 mm by 114 mm) footprint of a jack; and (2) for mechanical parking structures without slab or deck that are used for storing passenger car only, 2,250 lb (10 kN) per wheel.

<sup>i</sup>Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provisions for truck and bus loadings.

<sup>d</sup>The loading applies to stack room floors that support nonmobile, double-faced library book stacks subject to the following limitations: (1) The nominal book stack unit height shall not exceed 90 in. (2290 mm); (2) the nominal shelf depth shall not exceed 12 in. (305 mm) for each face; and (3) parallel rows of double-faced book stacks shall be separated by aisles not less than 36 in. (914 mm) wide.

<sup>e</sup>In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 24 lb per linear ft of seat applied in a direction parallel to each row of seats and 10 lb per linear ft of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

<sup>g</sup>Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.

<sup>f</sup>The concentrated wheel load shall be applied on an area of 4.5 in. by 4.5 in. (114 mm by 114 mm) footprint of a jack.

<sup>h</sup>Minimum concentrated load on stair treads (on area of 4 in.<sup>2</sup> [2,580 mm<sup>2</sup>]) is 300 lb (1.33 kN).

<sup>i</sup>Where uniform roof live loads are reduced to less than 20 lb/ft<sup>2</sup> (0.96 kN/m<sup>2</sup>) in accordance with Section 4.9.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the greatest unfavorable effect.

<sup>j</sup>Roofs used for other special purposes shall be designed for appropriate loads as approved by the authority having jurisdiction.

$$L_r = L_0 R_1 R_2 \quad \text{and} \quad 12 \leq L_r \leq 20 \text{ psf}$$

$$\text{where } R_1 = \begin{cases} 1 & \text{for } A_T \leq 200 \text{ ft}^2 \\ 1.2 - 0.001A_T & \text{for } 200 \text{ ft}^2 < A_T < 600 \text{ ft}^2 \\ 0.6 & \text{for } A_T \geq 600 \text{ ft}^2 \end{cases}$$

$$R_2 = \begin{cases} 1 & \text{for } F \leq 4 \\ 1.2 - 0.05F & \text{for } 4 < F < 12 \\ 0.6 & \text{for } F \geq 12 \end{cases}$$

$A_T$  = tributary area supported by structural member, ft<sup>2</sup>

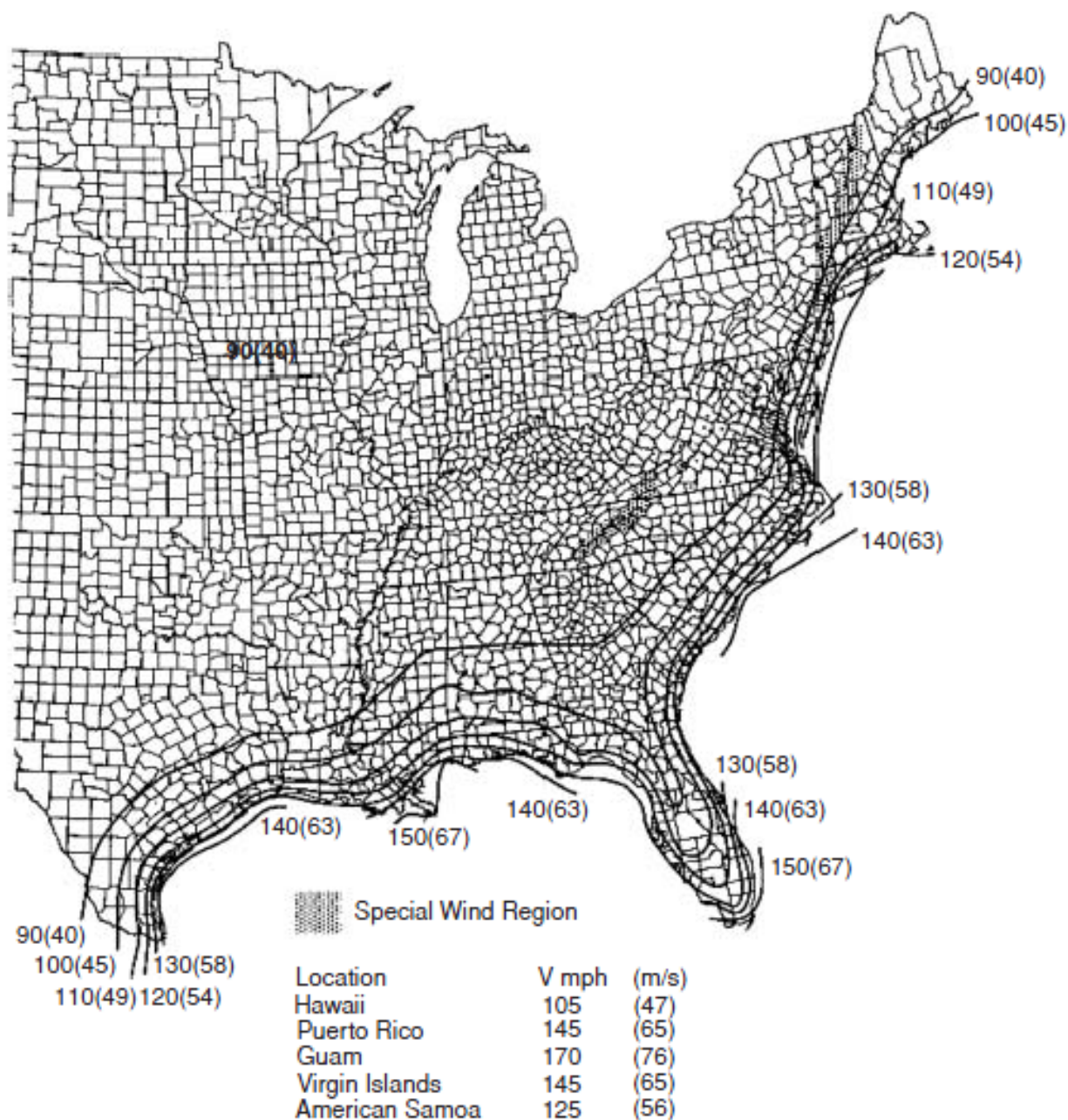
$F$  = the number of inches of rise per foot for a sloped roof

$L_0$  = minimum uniform live load per ASCE 7 Table 4-1 or IBC

Table 1607.1

Figure 18: Live Load Reduction





**Notes:**

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10 m) above ground for Exposure C category.
2. Linear interpolation between wind contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

Figure 19: ASCE Table 6-1: Basic Wind Speed

Main Wind Force Resisting System – Method 1				h ≤ 60 ft.									
Figure 6-2 (cont'd)		Design Wind Pressures		Walls & Roofs									
Enclosed Buildings													
Simplified Design Wind Pressure, p <sub>300</sub> (psf) (Exposure B at h = 30 ft, K <sub>e</sub> = 1.0, with I = 1.0)													
Basic Wind Speed (mph)	Roof Angle (degrees)	Lead Coeff	Zones										
			Horizontal Pressures				Vertical Pressures				Overhangs		
			A	B	C	D	E	F	G	H	Ex	Gx	
85	0 to 5°	1	11.5	-5.9	7.6	-9.5	-13.6	-7.8	-9.6	-6.1	-19.3	-15.1	
	10°	1	12.9	-5.4	8.6	-9.1	-13.6	-8.4	-9.6	-6.5	-19.3	-15.1	
	15°	1	14.4	-4.8	9.6	-2.7	-13.6	-9.0	-9.6	-6.9	-19.3	-15.1	
	20°	1	15.9	-4.2	10.6	-2.3	-13.6	-9.6	-9.6	-7.3	-19.3	-15.1	
	25°	1	14.4	2.3	10.4	2.4	-6.4	-8.7	-4.6	-7.0	-11.9	-10.1	
		2	-----	-----	-----	-----	-2.4	-4.7	-0.7	-3.0	-----	-----	
	30 to 45	1	12.9	8.8	10.2	7.0	1.0	-7.8	0.3	-6.7	-4.5	-5.2	
		2	12.9	8.8	10.2	7.0	5.0	-3.9	4.3	-2.8	-4.5	-5.2	
	90	0 to 5°	1	12.8	-6.7	8.5	-4.0	-15.4	-8.8	-10.7	-6.8	-21.6	-16.9
		10°	1	14.5	-6.0	9.6	-3.5	-15.4	-9.4	-10.7	-7.2	-21.6	-16.9
15°		1	16.1	-5.4	10.7	-3.0	-15.4	-10.1	-10.7	-7.7	-21.6	-16.9	
20°		1	17.8	-4.7	11.9	-2.6	-15.4	-10.7	-10.7	-8.1	-21.6	-16.9	
25°		1	16.1	2.6	11.7	2.7	-7.2	-9.8	-5.2	-7.8	-19.3	-11.4	
		2	-----	-----	-----	-----	-2.7	-5.3	-0.7	-3.4	-----	-----	
30 to 45		1	14.4	9.9	11.5	7.9	1.1	-8.8	0.4	-7.5	-5.1	-5.8	
		2	14.4	9.9	11.5	7.9	5.6	-4.3	4.3	-3.1	-5.1	-5.8	
100		0 to 5°	1	15.9	-8.2	10.5	-4.9	-19.1	-10.8	-13.3	-8.4	-26.7	-20.9
		10°	1	17.9	-7.4	11.9	-4.3	-19.1	-11.6	-13.3	-8.9	-26.7	-20.9
	15°	1	19.9	-6.6	13.3	-3.8	-19.1	-12.4	-13.3	-9.5	-26.7	-20.9	
	20°	1	22.0	-5.8	14.6	-3.2	-19.1	-13.3	-13.3	-10.1	-26.7	-20.9	
	25°	1	19.9	3.2	14.4	3.3	-8.8	-12.0	-6.4	-9.7	-16.5	-14.0	
		2	-----	-----	-----	-----	-3.4	-6.6	-0.9	-4.2	-----	-----	
	30 to 45	1	17.8	12.2	14.2	9.8	1.4	-10.8	0.5	-9.3	-6.3	-7.2	
		2	17.8	12.2	14.2	9.8	6.9	-5.3	5.9	-3.8	-6.3	-7.2	
	105	0 to 5°	1	17.5	-9.0	11.6	-5.4	-21.1	-11.9	-14.7	-9.3	-29.4	-23.0
		10°	1	19.7	-8.2	13.1	-4.7	-21.1	-12.8	-14.7	-9.8	-29.4	-23.0
15°		1	21.9	-7.3	14.7	-4.2	-21.1	-13.7	-14.7	-10.5	-29.4	-23.0	
20°		1	24.3	-6.4	16.1	-3.5	-21.1	-14.7	-14.7	-11.1	-29.4	-23.0	
25°		1	21.9	3.5	15.9	3.5	-9.7	-13.2	-7.1	-10.7	-18.2	-15.4	
		2	-----	-----	-----	-----	-3.7	-7.3	-1.0	-4.6	-----	-----	
30 to 45		1	19.6	13.5	15.7	10.8	1.5	-11.9	0.6	-10.3	-6.9	-7.9	
		2	19.6	13.5	15.7	10.8	7.6	-5.8	6.5	-4.2	-6.9	-7.9	
110		0 to 5°	1	19.2	-10.0	12.7	-5.9	-23.1	-13.1	-16.0	-10.1	-32.3	-25.3
		10°	1	21.6	-9.0	14.4	-5.2	-23.1	-14.1	-16.0	-10.8	-32.3	-25.3
	15°	1	24.1	-8.0	16.0	-4.6	-23.1	-15.1	-16.0	-11.5	-32.3	-25.3	
	20°	1	26.6	-7.0	17.7	-3.9	-23.1	-16.0	-16.0	-12.2	-32.3	-25.3	
	25°	1	24.1	3.9	17.4	4.0	-10.7	-14.6	-7.7	-11.7	-19.9	-17.0	
		2	-----	-----	-----	-----	-4.1	-7.9	-1.1	-5.1	-----	-----	
	30 to 45	1	21.6	14.8	17.2	11.8	1.7	-13.1	0.6	-11.3	-7.6	-8.7	
		2	21.6	14.8	17.2	11.8	8.3	-6.5	7.2	-4.6	-7.6	-8.7	
	120	0 to 5°	1	22.6	-11.9	15.1	-7.0	-27.4	-15.6	-19.1	-12.1	-38.4	-30.1
		10°	1	25.8	-10.7	17.1	-6.2	-27.4	-16.8	-19.1	-12.9	-38.4	-30.1
15°		1	28.7	-9.5	19.1	-5.4	-27.4	-17.9	-19.1	-13.7	-38.4	-30.1	
20°		1	31.6	-8.3	21.1	-4.6	-27.4	-19.1	-19.1	-14.5	-38.4	-30.1	
25°		1	28.6	4.6	20.7	4.7	-12.7	-17.3	-9.2	-13.9	-28.7	-20.2	
		2	-----	-----	-----	-----	-4.8	-9.4	-1.3	-6.0	-----	-----	
30 to 45		1	25.7	17.6	20.4	14.0	2.0	-15.6	0.7	-13.4	-9.0	-10.3	
		2	25.7	17.6	20.4	14.0	9.9	-7.7	8.6	-5.5	-9.0	-10.3	

Unit Conversions—1.0 ft = 0.3048 m; 1.0 psf = 0.0479 kN/m<sup>2</sup>

Figure 20: ASCE Figure 6-2: Simplified Design Wind Pressure

Table 13: ASCE Figure 6-2 Cont.: Adjustment Factor for Building Height and Exposure

Adjustment Factor for Building Height and Exposure, $\lambda$			
Mean roof height (ft)	Exposure		
	B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

Table 14: ASCE Table 6-1 Wind Importance Factor

Category	Non-Hurricane Prone Regions and Hurricane Prone Regions with $V = 85-100$ mph and Alaska	Hurricane Prone Regions with $V > 100$ mph
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

Note:

1. The building and structure classification categories are listed in Table 1-1.

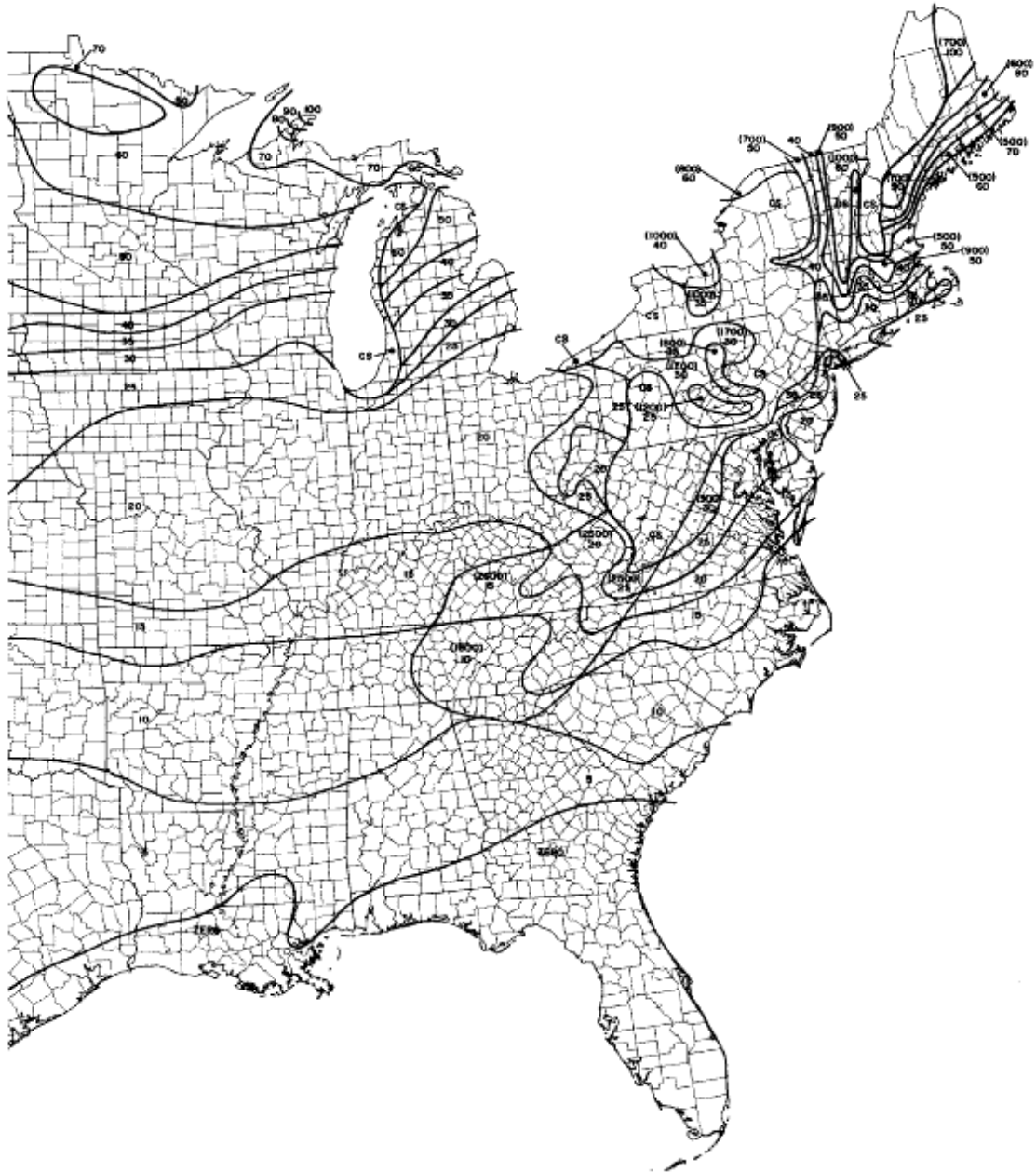


Figure 21: ASCE Figure 7-1: Ground Snow Loads,  $p_g$ , for the United States ( $\text{lb/ft}^2$ )

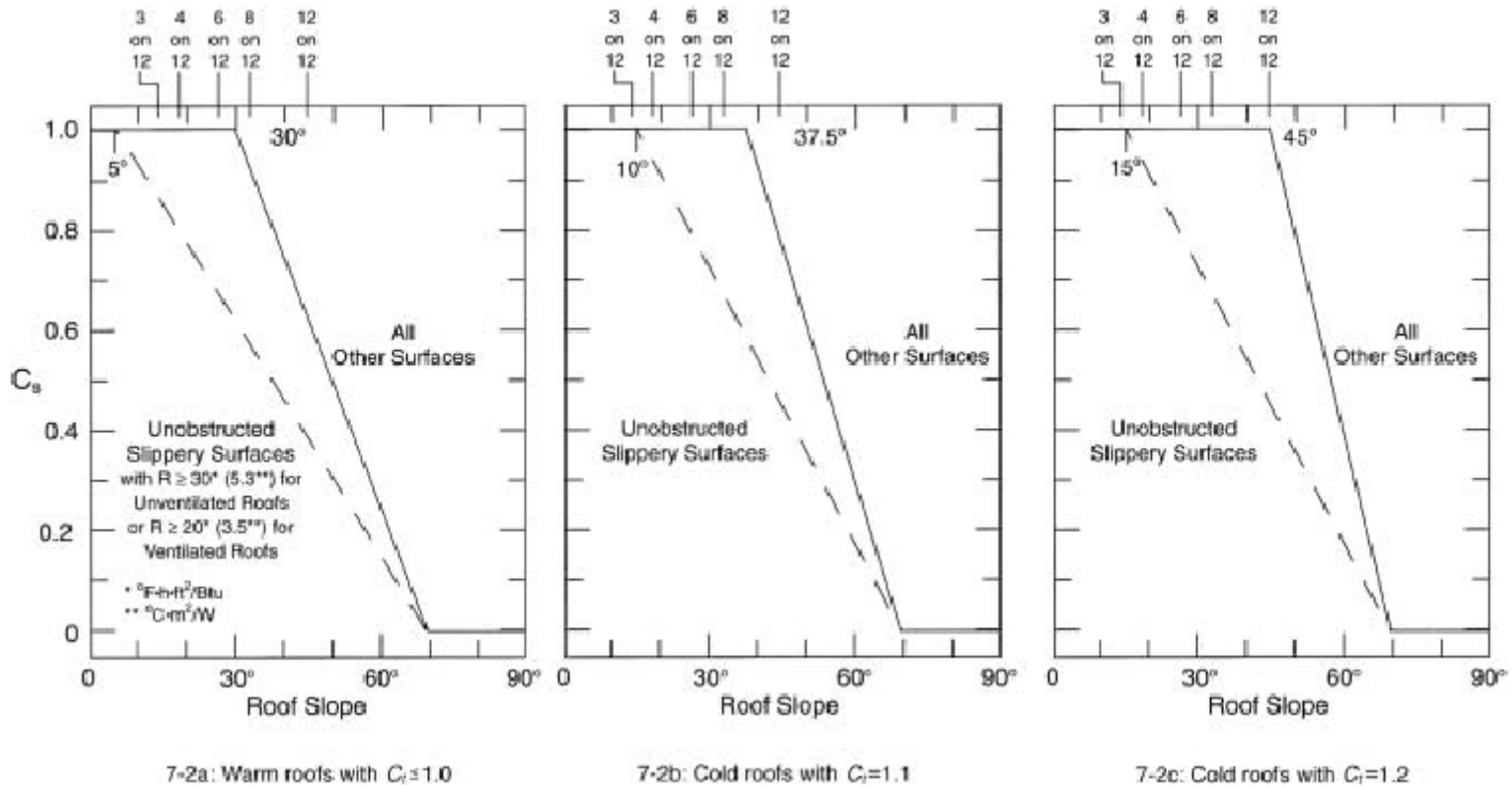


Figure 22: ASCE Figure 7-2: Graphs for Determining Roof Slope Factor  $C_s$ , for Warm and Cold Roof

**Table 15: ASCE Table 7-2: Exposure Factor,  $C_e$**

Terrain Category	Fully Exposed	Exposure of Roof <sup>a</sup> Partially Exposed	Sheltered
B (see Section 6.5.6)	0.9	1.0	1.2
C (see Section 6.5.6)	0.9	1.0	1.1
D (see Section 6.5.6)	0.8	0.9	1.0
Above the treeline in windswept mountainous areas.	0.7	0.8	N/A
In Alaska, in areas where trees do not exist within a 2-mile (3 km) radius of the site.	0.7	0.8	N/A

The terrain category and roof exposure condition chosen shall be representative of the anticipated conditions during the life of the structure. An exposure factor shall be determined for each roof of a structure.

<sup>a</sup>Definitions: Partially Exposed: All roofs except as indicated in the following text. Fully Exposed: Roofs exposed on all sides with no shelter<sup>b</sup> afforded by terrain, higher structures, or trees. Roofs that contain several large pieces of mechanical equipment, parapets that extend above the height of the balanced snow load ( $h_b$ ), or other obstructions are not in this category. Sheltered: Roofs located tight in among conifers that qualify as obstructions.

<sup>b</sup>Obstructions within a distance of  $10h_b$  provide "shelter," where  $h_b$  is the height of the obstruction above the roof level. If the only obstructions are a few deciduous trees that are leafless in winter, the "fully exposed" category shall be used. Note that these are heights above the roof. Heights used to establish the terrain category in Section 6.5.3 are heights above the ground.

**Table 16: ASCE Table 7-3: Thermal Factor,  $C_t$**

Thermal Condition <sup>a</sup>	$C_t$
All structures except as indicated below:	1.0
Structures kept just above freezing and others with cold, ventilated roofs in which the thermal resistance (R-value) between the ventilated space and the heated space exceeds $25 \text{ }^\circ\text{F} \times h \times \text{ft}^2/\text{Btu}$ ( $4.4 \text{ K} \times \text{m}^2/\text{W}$ ).	1.1
Unheated structures and structures intentionally kept below freezing.	1.2
Continuously heated greenhouses <sup>b</sup> with a roof having a thermal resistance (R-value) less than $2.0 \text{ }^\circ\text{F} \times h \times \text{ft}^2/\text{Btu}$ ( $0.4 \text{ K} \times \text{m}^2/\text{W}$ )	0.85

<sup>a</sup>These conditions shall be representative of the anticipated conditions during winters for the life of the structure.

<sup>b</sup>Greenhouses with a constantly maintained interior temperature of  $50 \text{ }^\circ\text{F}$  ( $10 \text{ }^\circ\text{C}$ ) or more at any point 3 ft above the floor level during winters and having either a maintenance attendant on duty at all times or a temperature alarm system to provide warning in the event of a heating failure.

**Table 17: ASCE Table 7-4: Snow Importance Factor**

Category <sup>a</sup>	$I$
I	0.8
II	1.0
III	1.1
IV	1.2

<sup>a</sup>See Section 1.5 and Table 1-1.

1. D (Equation 16-8)
2. D + L (Equation 16-9)
3. D + ( $L_r$  or S) (Equation 16-10)
4. D + 0.75L + 0.75( $L_r$  or S) (Equation 16-11)
5. D + (W or 0.7E) (Equation 16-12)
6. D + 0.75(W or 0.7E) + 0.75L + 0.75( $L_r$  or S) (Equation 16-13)
7. 0.6D + W (Equation 16-14)
8. 0.6D + 0.7E (Equation 16-15)

**Figure 213: ASD Load Combinations**

The IBC LRFD load combinations are:

- 1.4(D + F) (Equation 16-1)
- 1.2(D + F + T) + 1.6(L + H) + 0.5(L<sub>r</sub> or S or R) (Equation 16-2)
- 1.2D + 1.6(L<sub>r</sub> or S or R) + (f<sub>1</sub>L or 0.8W) (Equation 16-3)
- 1.2D + 1.6W + f<sub>1</sub>L + 0.5(L<sub>r</sub> or S or R) (Equation 16-4)
- 1.2D + 1.0E + f<sub>1</sub>L + f<sub>2</sub>S (Equation 16-5)
- 0.9D + 1.6W + 1.6H (Equation 16-6)
- 0.9D + 1.0E + 1.6H (Equation 16-7)

The variable load factor  $f_1$  is set to 1 for garages, places of public assembly, and tabulated live loads over 100 psf. Variable  $f_1$  is allowed to be 0.5 otherwise. This is consistent with ASCE 7. The variable  $f_2$  is 0.7 for roof configurations that do not shed snow, and 0.2 otherwise. This is specific to the IBC.

Figure 224: LRFD Load Combinations

**EXAMPLE 4.13 Resistance Factor (LRFD Only)**

Property	Symbol	Value
Bending $F_b$	$\phi_b$	0.85
Tension $F_t$	$\phi_t$	0.80
Shear $F_v$	$\phi_v$	0.75
Compression $F_c, F_{c\perp}$	$\phi_c$	0.90
Stability $E_{min}$	$\phi_s$	0.85

Shear is a highly variable strength value and may produce sudden, brittle failures. Conversely, compression is a very ductile mode of failure. Accordingly, the resistance factor for shear is significantly lower than that for compression.

Figure 235: LRFD Resistance Factors

**EXAMPLE 4.14 Format Conversion Factor (LRFD Only)**

Property	Value
Bending $F_b$	$2.16/\phi_b = 2.16/0.85 = 2.54$
Tension $F_t$	$2.16/\phi_t = 2.16/0.8 = 2.70$
Shear $F_v$	$2.16/\phi_v = 2.16/0.75 = 2.88$
Compression parallel to the grain $F_c$	$2.16/\phi_c = 2.16/0.9 = 2.40$
Compression perpendicular to the grain $F_{c\perp}$	$1.875/\phi_c = 1.875/0.9 = 2.083$
Stability $E_{min}$	$1.5/\phi_s = 1.5/0.85 = 1.765$

All conversions are based on the prescribed resistance factor  $\phi$ , and all except compression perpendicular to grain and modulus of elasticity used for stability have the same coefficient in the numerator. In ASD, compression perpendicular to grain is not adjusted for duration of load; that is,  $C_D$  is not applicable to  $F_{c\perp}$ . However, in LRFD, the time effect factor  $\lambda$  is applied to  $F_{c\perp}$ .  $E_{min}$  is not adjusted for load duration or time effect in either ASD or LRFD. Therefore, the format conversions for  $F_{c\perp}$  and  $E_{min}$  differ from those for other properties.

Figure 26: LRFD Conversion Factors



Table 18: NDS Table 4A: Reference Design Values for Spruce-Pine-Fir

**Table 4A Reference Design Values for Visually Graded Dimension Lumber (2" - 4" thick)<sup>1,2,3</sup>**

(All species except Southern Pine — see Table 4B) (Tabulated design values are for normal load duration and dry service conditions. See NDS 4.3 for a comprehensive description of design value adjustment factors.)

**USE WITH TABLE 4A ADJUSTMENT FACTORS**

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)							Grading Rules Agency
		Bending $F_b$	Tension parallel to grain $F_t$	Shear parallel to grain $F_v$	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain $F_c$	Modulus of Elasticity		
							E	$E_{min}$	
<b>REDWOOD</b>									
Clear Structural	2" & wider	1,750	1,000	160	650	1,850	1,400,000	510,000	RIS
Select Structural		1,350	800	160	650	1,500	1,400,000	510,000	
Select Structural, open grain		1,100	625	160	425	1,100	1,100,000	400,000	
No.1		975	575	160	650	1,200	1,300,000	470,000	
No.1, open grain		775	450	160	425	900	1,100,000	400,000	
No.2		925	525	160	650	950	1,200,000	440,000	
No.2, open grain		725	425	160	425	700	1,000,000	370,000	
No.3		525	300	160	650	550	1,100,000	400,000	
No.3, open grain		425	250	160	425	400	900,000	330,000	
Stud		2" & wider	575	325	160	425	450	900,000	
Construction	2" - 4" wide	825	475	160	425	925	900,000	330,000	
Standard		450	275	160	425	725	900,000	330,000	
Utility		225	125	160	425	475	800,000	290,000	
<b>SPRUCE-PINE-FIR</b>									
Select Structural	2" & wider	1,250	700	135	425	1,400	1,500,000	550,000	NLGA
No.1/No.2		875	450	135	425	1,150	1,400,000	510,000	
No.3		500	250	135	425	650	1,200,000	440,000	
Stud	2" & wider	675	350	135	425	725	1,200,000	440,000	
Construction	2" - 4" wide	1,000	500	135	425	1,400	1,300,000	470,000	
Standard		550	275	135	425	1,150	1,200,000	440,000	
Utility		275	125	135	425	750	1,100,000	400,000	

Table 19: NDS Size Factor,  $C_F$ , Table

Grades	Width (depth)	$F_b$		$F_t$	$F_c$
		Thickness (breadth)			
		2" & 3"	4"		
Select Structural, No.1 & Btr, No.1, No.2, No.3	2", 3", & 4"	1.5	1.5	1.5	1.15
	5"	1.4	1.4	1.4	1.1
	6"	1.3	1.3	1.3	1.1
	8"	1.2	1.3	1.2	1.05
	10"	1.1	1.2	1.1	1.0
	12"	1.0	1.1	1.0	1.0
Stud	14" & wider	0.9	1.0	0.9	0.9
	2", 3", & 4" 5" & 6"	1.1 1.0	1.1 1.0	1.1 1.0	1.05 1.0
Construction, Standard	2", 3", & 4"	Use No.3 Grade tabulated design values and size factors			
Utility	2", 3", & 4"	1.0	1.0	1.0	1.0
	4" 2" & 3"	1.0 0.4	1.0 —	1.0 0.4	1.0 0.6



EXAMPLE 4.10 Load Duration Factor (ASD Only)

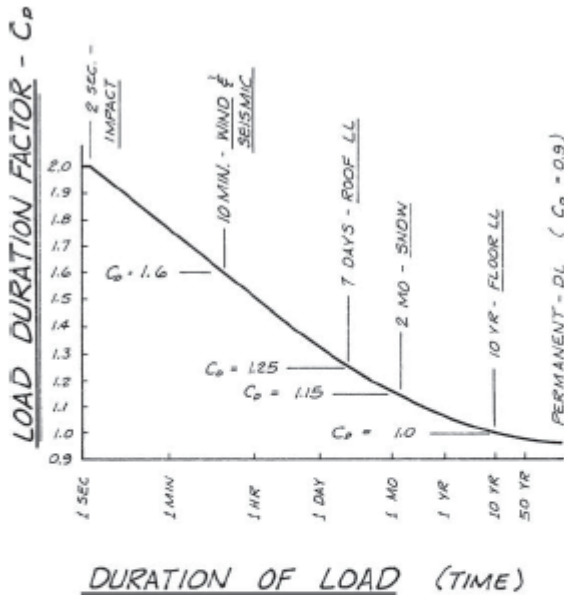


Figure 4.13 Madison curve.

Shortest duration load in combination	$C_D$
Dead load	0.9
Floor live load	1.0
Snow load	1.15
Roof live load	1.25
Wind or seismic force	1.6
Impact	2.0

NOTE: 1. Check all Code-required load and force combinations. 2. The  $C_D$  associated with the shortest duration load or force in a given combination is used to adjust the reference design values. 3. The critical combination of loads and forces is the one that requires the largest-size structural member.

Figure 247: Madison Curve for Load Duration Factor,  $C_D$

Recommended deflection limitations		
Use classification	Applied load only	Applied load + dead load
<b>Roof beams</b>		
Industrial	$L/180$	$L/120$
Commercial and institutional		
Without plaster ceiling	$L/240$	$L/180$
With plaster ceiling	$L/360$	$L/240$
<b>Floor beams</b>		
Ordinary usage*	$L/360$	$L/240$
Highway bridge stringers	$L/300$	
Railway bridge stringers	$L/300$ to $L/400$	

\*The ordinary usage classification is for floors intended for construction in which walking comfort and minimized plaster cracking are the main considerations. These recommended deflection limits may not eliminate all objections to vibrations such as in long spans approaching the maximum limits or for some office and institutional applications where increased floor stiffness is desired. For these usages the deflection limitations in the following table have been found to provide additional stiffness.

Figure 258: Beam Deflection Limits

Table 20: SDPWS 2008 Table 4.2C: Nominal Unit Shear Capacities for Wood-Frame Diaphragms

**Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms**

**Unblocked Wood Structural Panel Diaphragms<sup>1,2,3,4</sup>**

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)	A SEISMIC						B WIND			
					6 in. Nail Spacing at diaphragm boundaries and supporting members								6 in. Nail Spacing at diaphragm boundaries and supporting members	
					Case 1			Cases 2,3,4,5,6			Case 1	Cases 2,3,4,5,6		
$v_s$ (plf)	$G_s$ (kips/in.)		$v_s$ (plf)	$G_s$ (kips/in.)		$v_w$ (plf)	$v_w$ (plf)							
	OSB	PLY		OSB	PLY									
Structural I	6d	1-1/4	5/16	2	330	9.0	7.0	250	6.0	4.5	460	350		
					370	7.0	6.0	280	4.5	4.0	520	390		
	8d	1-3/8	3/8	2	480	8.5	7.0	360	6.0	4.5	670	505		
					530	7.5	6.0	400	5.0	4.0	740	560		
	10d	1-1/2	15/32	2	570	14	10	430	9.5	7.0	800	600		
					640	12	9.0	480	8.0	6.0	895	670		
Sheathing and Single-Floor	6d	1-1/4	5/16	2	300	9.0	6.5	220	6.0	4.0	420	310		
					340	7.0	5.5	250	5.0	3.5	475	350		
			3/8	2	330	7.5	5.5	250	5.0	4.0	460	350		
					370	6.0	4.5	280	4.0	3.0	520	390		
	8d	1-3/8	3/8	2	430	9.0	6.5	320	6.0	4.5	600	450		
					480	7.5	5.5	360	5.0	3.5	670	505		
			7/16	2	460	8.5	6.0	340	5.5	4.0	645	475		
					510	7.0	5.5	380	4.5	3.5	715	530		
	15/32	2	480	7.5	5.5	360	5.0	4.0	670	505				
			530	6.5	5.0	400	4.0	3.5	740	560				
	10d	1-1/2	15/32	2	510	15	9.0	380	10	6.0	715	530		
					580	12	8.0	430	8.0	5.5	810	600		
			19/32	2	570	13	8.5	430	8.5	5.5	800	600		
					640	10	7.5	480	7.0	5.0	895	670		

- Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
- For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor =  $[1 - (0.5 - G)]$ , where G = Specific Gravity of the framing lumber from the NDS (Table 11.3.2A). The Specific Gravity Adjustment Factor shall not be greater than 1.
- Apparent shear stiffness values  $G_s$  are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used,  $G_s$  values shall be permitted to be increased by 1.2.
- Where moisture content of the framing is greater than 19% at time of fabrication,  $G_s$  values shall be multiplied by 0.5

Table 21: SDPWS 2008 Table 4.3A: Nominal Unit Shear Capacities for Wood-Frame Shearwalls

**Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls<sup>1,3,6,7</sup>**

Wood-based Panels <sup>4</sup>																			
Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Fastener Type & Size	A SEISMIC								B WIND							
				Panel Edge Fastener Spacing (in.)								Panel Edge Fastener Spacing (in.)							
				6		4		3		2		6	4	3	2				
				$v_s$ (plf)	$G_s$ (kips/in.)	$v_s$ (plf)	$G_s$ (kips/in.)	$v_s$ (plf)	$G_s$ (kips/in.)	$v_s$ (plf)	$G_s$ (kips/in.)	$v_w$ (plf)	$v_w$ (plf)	$v_w$ (plf)	$v_w$ (plf)				
Wood Structural Panels - Structural <sup>1,4,5</sup>	5/16	1-1/4	Nail (common or galvanized box) 6d	OSB		PLY		OSB		PLY		OSB		PLY		560	840	1090	1430
	3/8 <sup>2</sup>	1-3/8	8d	460	19	14	720	24	17	920	30	20	1220	43	24	645	1010	1290	1710
	7/16 <sup>2</sup>			510	16	13	790	21	16	1010	27	19	1340	40	24	715	1105	1415	1875
	15/32	1-1/2	10d	560	14	11	860	18	14	1100	24	17	1460	37	23	785	1205	1540	2045
5/16	680			22	16	1020	29	20	1330	36	22	1740	51	28	950	1430	1860	2435	
Wood Structural Panels - Sheathing <sup>4,5</sup>	5/16	1-1/4	6d	360	13	9.5	540	18	12	700	24	14	900	37	18	505	755	980	1260
	3/8	1-3/8	8d	400	11	8.5	600	15	11	780	20	13	1020	32	17	560	840	1090	1430
	3/8 <sup>2</sup>			440	17	12	640	25	15	820	31	17	1060	45	20	615	895	1150	1485
	7/16 <sup>2</sup>	480	15	11	700	22	14	900	28	17	1170	42	21	670	980	1260	1640		
15/32	1-1/2	10d	520	13	10	760	19	13	980	25	15	1280	39	20	730	1065	1370	1790	
5/16			620	22	14	920	30	17	1200	37	19	1540	52	23	870	1290	1680	2155	
Plywood Siding	5/16	1-1/4	Nail (galvanized casing) 6d	280	13		420	16		550	17		720	21		390	590	770	1010
	3/8	1-3/8	8d	320	16		480	18		620	20		820	22		450	670	870	1150
Particleboard Sheathing - (M-S "Exterior Glue" and M-2 "Exterior Glue")	3/8		Nail (common or galvanized box) 6d	240	15		360	17		460	19		600	22		335	505	645	840
	3/8			260	18		380	20		480	21		630	23		365	530	670	880
	1/2			280	18		420	20		540	22		700	24		390	590	755	980
	5/8			370	21		550	23		720	24		920	25		520	770	1010	1290
Structural Fiberboard Sheathing	1/2		Nail (galvanized roofing) 11 ga. galv. roofing nail (0.120" x 1-1/2" long x 7/16" head)				340	4.0		460	5.0		520	5.5		475	645	730	
	25/32		11 ga. galv. roofing nail (0.120" x 1-3/4" long x 3/8" head)				340	4.0		460	5.0		520	5.5		475	645	730	

- Nominal unit shear values shall be adjusted in accordance with 4.3.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.3.6. For specific requirements, see 4.3.7.1 for wood structural panel shear walls, 4.3.7.2 for particleboard shear walls, and 4.3.7.3 for fiberboard shear walls. See Appendix A for common and box nail dimensions.
- Shears are permitted to be increased to values shown for 15/32 inch sheathing with same nailing provided (a) studs are spaced a maximum of 16 inches on center, or (b) panels are applied with long dimension across studs.
- For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor =  $[1 - (0.5 - G)]$ , where  $G$  = Specific Gravity of the framing lumber from the  $ND5$  (Table 11.3.2A). The Specific Gravity Adjustment Factor shall not be greater than 1.
- Apparent shear stiffness values  $G_s$  are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for shear walls constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used,  $G_s$  values shall be permitted to be increased by 1.2.
- Where moisture content of the framing is greater than 19% at time of fabrication,  $G_s$  values shall be multiplied by 0.5.
- Where panels are applied on both faces of a shear wall and nail spacing is less than 6" on center on either side, panel joints shall be offset to fall on different framing members. Alternatively, the width of the nailed face of framing members shall be 3" nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.
- Galvanized nails shall be hot-dipped or tumbled.

Table 22: SDPWS Table 4.3.3.2: Unblocked Shearwall Adjustment Factor,  $C_{ub}$

Table 4.3.3.2 Unblocked Shear Wall Adjustment Factor, $C_{ub}$					
Nail Spacing (in.)		Stud Spacing (in.)			
Supported Edges	Intermediate Framing	12	16	20	24
6	6	1.0	0.8	0.6	0.5
6	12	0.8	0.6	0.5	0.4

Table 23: Load Span Tables for APA Structural Use Panels Table 1: Uniform Loads on APA Rated Sheathing

TABLE 1  
UNIFORM LOADS (PSF) ON APA RATED SHEATHING. MULTI-SPAN, NORMAL DURATION OF LOAD, DRY CONDITIONS, PANELS 24 INCHES OR WIDER

Span Rating	Load Governed By	Strength Axis Across Supports Span Center-to-Center of Supports (inches)										Strength Axis Parallel to Supports Span, Center-to-Center of Supports (inches)		
		12	16	19.2	24	30	32	36	40	48	60	12	16	24
24/0	L/360	261	98	54	26	13	10	9				16	6	
	L/240	392	147	81	39	19	16	14				23	9	
	L/180	522	196	107	52	26	21	18				31	12	
	Bending	208	117	81	52	33	29	19				45	25	
	Shear	314	228	186	147	116	108	92				200	145	
24/16	L/360	339	128	70	34	17	14	12	9			23	9	
	L/240	509	191	105	51	25	20	18	13			34	13	
	L/180	679	255	140	68	33	27	24	17			45	17	
	Bending	267	150	104	67	43	38	24	19			53	30	
	Shear	362	262	215	169	133	125	106	95			200	145	
32/16	L/360	500	188	103	50	24	20	18	13			35	13	4
	L/240	750	282	154	75	37	30	26	19			53	20	7
	L/180	1001	376	206	100	49	40	35	25			70	27	9
	Bending	308	173	120	77	49	43	27	22			77	43	15
	Shear	400	290	237	187	147	138	117	105			248	179	111
40/20	L/360	979	368	201	98	48	39	34	25	16		78	29	10
	L/240	1468	552	302	146	72	58	51	37	24		117	44	15
	L/180	1958	736	403	195	96	78	69	49	32		157	59	20
	Bending	521	293	203	130	83	73	46	38	26		125	70	25
	Shear	505	366	299	236	186	174	147	132	114		314	228	141
48/24	L/360	1740	655	358	174	85	69	61	44	29	14	128	48	16
	L/240	2610	982	537	260	128	104	91	66	43	21	193	72	24
	L/180	3480	1309	716	347	170	139	122	88	57	28	257	97	33
	Bending	704	396	275	176	113	99	63	51	35	23	188	105	38
	Shear	648	469	384	302	239	223	189	170	147	116	362	262	162

Table 24: Load Span Tables for APA Structural Use Panels Table 3: Adjustments to Allowable Load Capacities Based on Panel Grade and Construction,  $C_G$

TABLE 3

**ADJUSTMENTS TO ALLOWABLE LOAD CAPACITIES  
BASED ON PANEL GRADE AND CONSTRUCTION,  $C_G$**

	Strength Axis <sup>(a)</sup>			
	Perpendicular to Supports		Parallel to Supports	
	Other	Structural I	Other	Structural I
<b>STIFFNESS (L/360, L/240, L/180)</b>				
3-Ply Plywood	1.1	1.1	1.0	1.5
4-Ply Plywood, COM-PLY	1.1	1.1	2.2	3.3
5-Ply Plywood <sup>(b)</sup>	1.1	1.1	3.1	5.2
OSB	1.0	1.0	3.1	5.2
<b>BENDING</b>				
3-Ply Plywood	1.0	1.0	1.0	1.3
4-Ply Plywood	1.1	1.1	1.2	1.7
COM-PLY	1.2	1.2	1.2	1.7
5-Ply Plywood <sup>(b)</sup> , OSB	1.2	1.2	1.8	2.8
<b>SHEAR</b>				
3-Ply Plywood	1.0	1.4	2.8	5.2
4-Ply Plywood	1.0	1.4	3.9	7.9
5-Ply Plywood <sup>(b)</sup>	1.1	1.6	1.0	1.4
OSB, COM-PLY	1.0	1.0	1.0	1.0

(a) The strength axis is the long panel dimension unless otherwise identified.  
 (b) Adjustments apply to plywood with 5 or more layers; for 5-ply/3-layer plywood, use adjustments for 4-ply.

Table 25: Load Span Tables for APA Structural Use Panels Table 4: Application Adjustment Factors

TABLE 4

**APPLICATION ADJUSTMENT FACTORS**

<b>Duration of Load, <math>C_D</math> (Applies to Bending and Shear Only):</b>	
Permanent load (over 10 years)	0.90
2 months, as for snow	1.15
7 days	1.25
Wind or earthquake	1.60
Impact	2.00
<b>Span Adjustments:</b>	
2-span to 1-span	
Deflection	0.42
Bending	1.00
Shear	1.25
3-span to 1-span	
Deflection	0.53
Bending	0.80
Shear	1.20
<b>Wet or Damp Locations, <math>C_M</math> (Moisture Content 16% or more):</b>	
Deflection	0.85
Bending	0.75
Shear	0.75

Table 26: Load Span Tables for APA Structural Use Panels Table 5: Typical APA Panel Construction

TABLE 5

**TYPICAL APA PANEL CONSTRUCTIONS<sup>(a)</sup>**

Span Rating	Plywood			COM-PLY	OSB
	3-Ply	4-Ply	5-Ply <sup>(b)</sup>		
<b>APA RATED SHEATHING</b>					
24/0	X				X
24/16					X
32/16	X	X	X		X
40/20	X	X	X		X
48/24		X	X		X
<b>APA RATED STURD-I-FLOOR</b>					
16 oc					
20 oc		X	X	X	X
24 oc		X	X	X	X
32 oc			X	X	X
48 oc			X	X	X

(a) Constructions may not be available in every area. Check with suppliers concerning availability.  
 (b) Applies to plywood with 5 or more layers.

Table 27: APA Panel Design Specifications Table 5: Nominal Thickness by Span Rating

TABLE 5

**NOMINAL THICKNESS BY SPAN RATING**  
 (The nominal thickness is given. The predominant thickness for each span rating is highlighted in bold type.)

Span Rating	Nominal Thickness (in.)										
	3/8	7/16	15/32	1/2	19/32	5/8	23/32	3/4	7/8	1	1-1/8
<b>APA Rated Sheathing</b>											
24/0	.375	.437	.469	.500							
24/16		.437	.469	.500							
32/16			.469	.500	.594	.625					
40/20					.594	.625	.719	.750			
48/24							.719	.750	.875		
<b>APA Rated Sturd-I-Floor</b>											
16 oc					.594	.625					
20 oc					.594	.625					
24 oc							.719	.750			
32 oc									.875	1.000	
48 oc											1.125

Note: 1 inch = 25.4 mm.



Table 28: SDPWS 2008 Table C4.2.2A: Gt values

**Table C4.2.2A Shear Stiffness,  $G_v t_v$  (lb/in. of depth), for Wood Structural Panels**

Span Rating <sup>4</sup>	Minimum Nominal Panel Thickness (in.)	Structural Sheathing				Structural I			
		Plywood			OSB	Plywood			OSB
		3-ply	4-ply	5-ply <sup>3</sup>		3-ply	4-ply	5-ply <sup>3</sup>	
<b>Sheathing Grades<sup>1</sup></b>									
24/0	3/8 <sup>2</sup>	25,000	32,500	37,500	77,500	32,500	42,500	41,500	77,500
24/16	7/16	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
32/16	15/32	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
40/20	19/32	28,500	37,000	43,000	88,500	37,000	48,000	47,500	88,500
48/24	23/32	31,000	40,500	46,500	96,000	40,500	52,500	51,000	96,000
<b>Single Floor Grades</b>									
16 oc	19/32	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
20 oc	19/32	28,000	36,500	42,000	87,000	36,500	47,500	46,000	87,000
24 oc	23/32	30,000	39,000	45,000	93,000	39,000	50,500	49,500	93,000
32 oc	7/8	36,000	47,000	54,000	110,000	47,000	61,000	59,500	110,000
48 oc	1-1/8	50,500	65,500	76,000	155,000	65,500	85,000	83,500	155,000

1. Sheathing grades used for calculating  $G_v$  values for diaphragm and shear wall tables.

2.  $G_v t_v$  values for 3/8" panels with span rating of 24/0 used to estimate  $G_v$  values for 5/16" panels.

3. 5-ply applies to plywood with five or more layers. For 5-ply plywood with three layers, use  $G_v t_v$  values for 4-ply panels.

4. See Table 4.2.2C for relationship between span rating and nominal panel thickness.

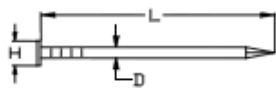
Table 29: SDPWS 2008 Table C4.2.2D: Fastener Slip,  $e_n$ **Table C4.2.2D Fastener Slip,  $e_n$  (in.)**

Sheathing	Fastener Size	Maximum Fastener Load ( $V_n$ ) (lb/fastener)	Fastener Slip, $e_n$ (in.)	
			Fabricated w/green (>19% m.c.) lumber	Fabricated w/dry ( $\leq$ 19% m.c.) lumber
Wood Structural Panel (WSP) or Particleboard <sup>1</sup>	6d common	180	$(V_n/434)^{2.314}$	$(V_n/456)^{3.144}$
	8d common	220	$(V_n/857)^{1.669}$	$(V_n/616)^{3.018}$
	10d common	260	$(V_n/977)^{1.894}$	$(V_n/769)^{3.276}$
Structural Fiberboard	All	-	-	0.07
Gypsum Board	All	-	-	0.03
Lumber	All	-	-	0.07

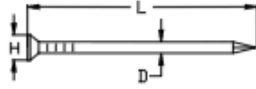
1. Slip values are based on plywood and OSB fastened to lumber with a specific gravity of 0.50 or greater. The slip shall be increased by 20 percent when plywood is not Structural I. Nail slip for common nails have been extended to galvanized box or galvanized casing nails of equivalent penny weight for purposes of calculating  $G_v$ .

Table 30: SDPWS 2008 Table A1: Dimensions of Standard Common, Box, and Sinker Nails

**Table A1 Standard Common, Box, and Sinker Nails<sup>1</sup>**



Common or Box



Sinker

D = diameter  
L = length  
H = head diameter

Type		Pennyweight										
		6d	7d	8d	10d	12d	16d	20d	30d	40d	50d	60d
Common	L	2"	2-1/4"	2-1/2"	3"	3-1/4"	3-1/2"	4"	4-1/2"	5"	5-1/2"	6"
	D	0.113"	0.113"	0.131"	0.148"	0.148"	0.162"	0.192"	0.207"	0.225"	0.244"	0.263"
	H	0.266"	0.266"	0.281"	0.312"	0.312"	0.344"	0.406"	0.438"	0.469"	0.5"	0.531"
Box	L	2"	2-1/4"	2-1/2"	3"	3-1/4"	3-1/2"	4"	4-1/2"	5"		
	D	0.099"	0.099"	0.113"	0.128"	0.128"	0.135"	0.148"	0.148"	0.162"		
	H	0.266"	0.266"	0.297"	0.312"	0.312"	0.344"	0.375"	0.375"	0.406"		
Sinker	L	1-7/8"	2-1/8"	2-3/8"	2-7/8"	3-1/8"	3-1/4"	3-3/4"	4-1/4"	4-3/4"		5-3/4"
	D	0.092"	0.099"	0.113"	0.12"	0.135"	0.148"	0.177"	0.192"	0.207"		0.244"
	H	0.234"	0.250"	0.266"	0.281"	0.312"	0.344"	0.375"	0.406"	0.438"		0.5"

1. Tolerances specified in ASTM F 1667. Typical shape of common, box, and sinker nails shown. See ASTM F1667 for other nail types.



## Appendix D: AutoCad and Revit Drawings



Figure 269: Architectural South Elevation



Figure 30: Structural South Elevation

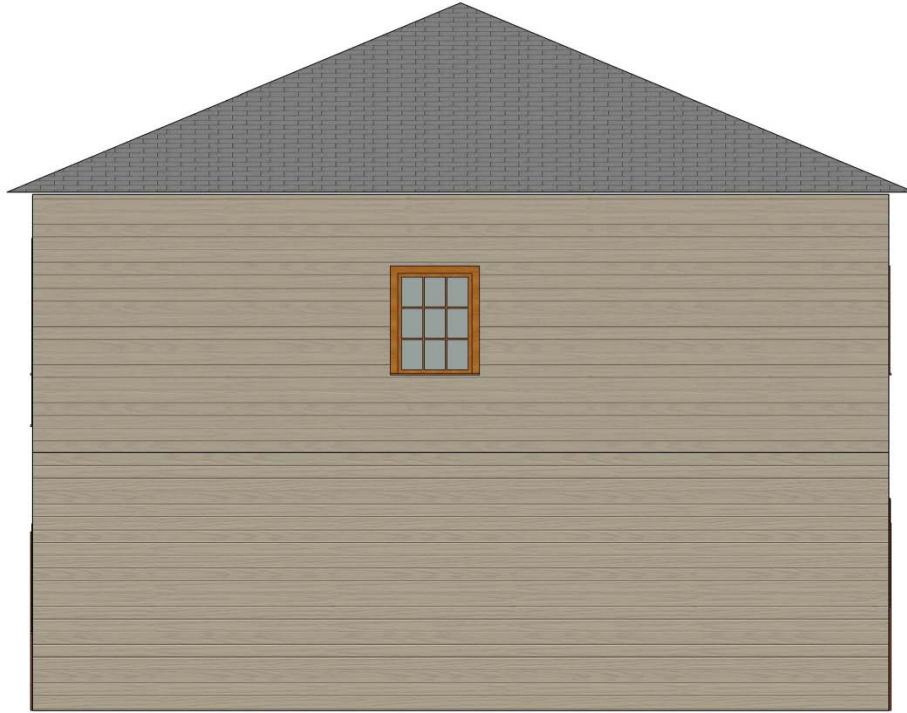


Figure 31: Architectural East Elevation



Figure 32: Structural East Elevation



Figure 33: Architectural North Elevation

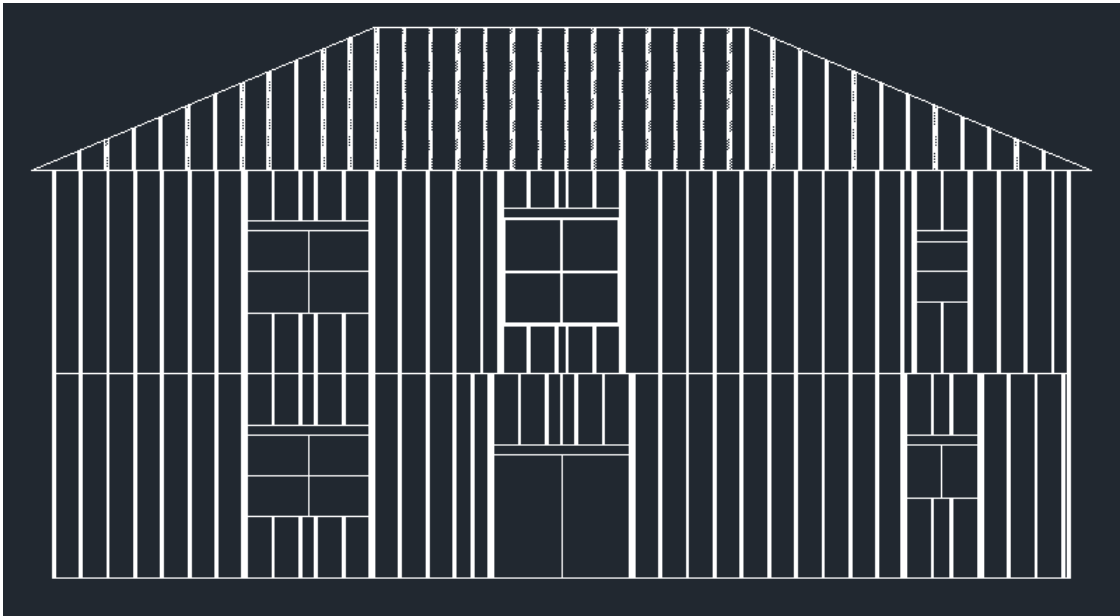


Figure 34: Structural North Elevation

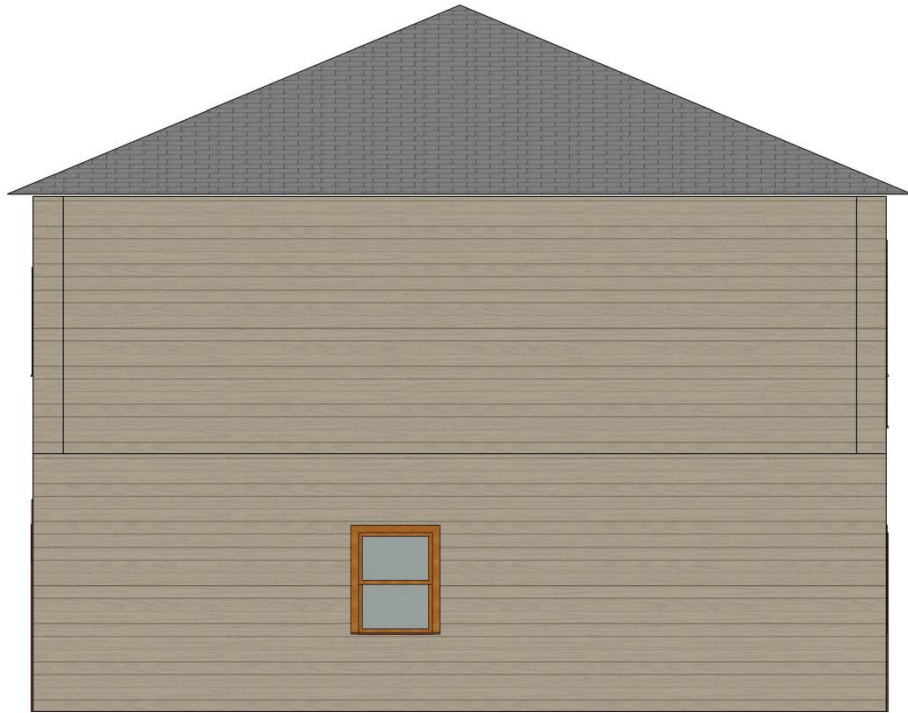


Figure 35: Architectural West Elevation



Figure 36: Structural West Elevation



Figure 37: 3D Rendering, Front



Figure 38: 3D Rendering, Back



Figure 39: First Floor Plan

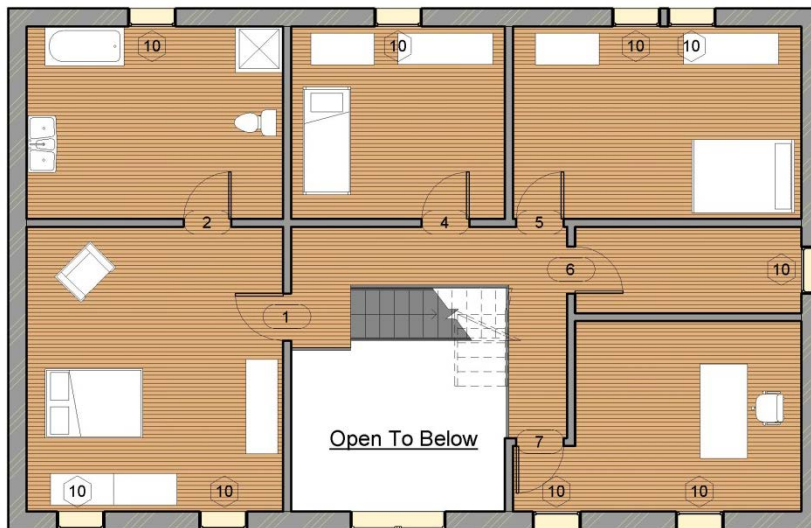


Figure 40: Second Floor Plan



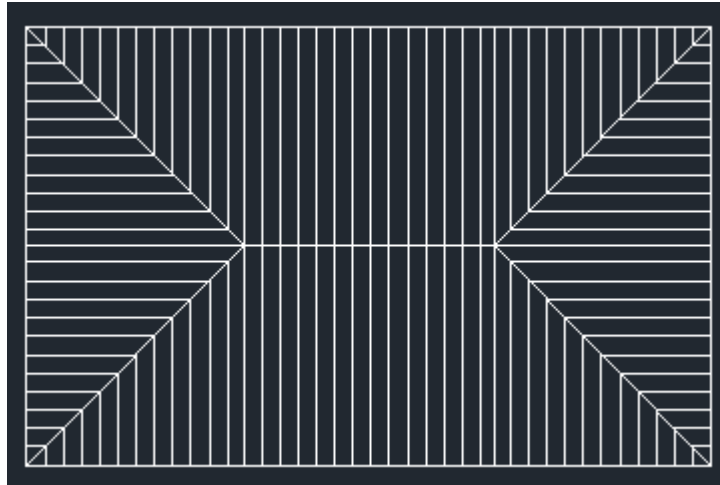


Figure 41: Roof Framing Plan

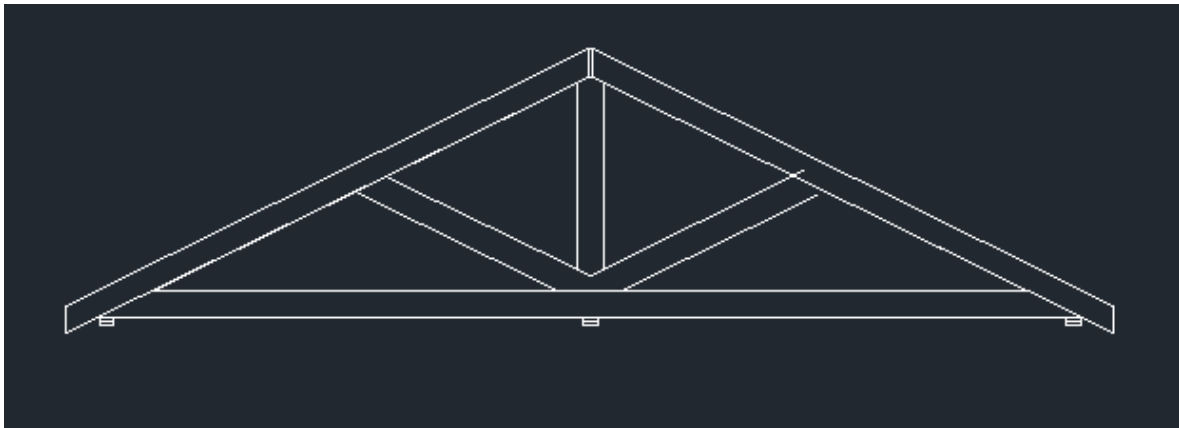


Figure 42: Roof Truss

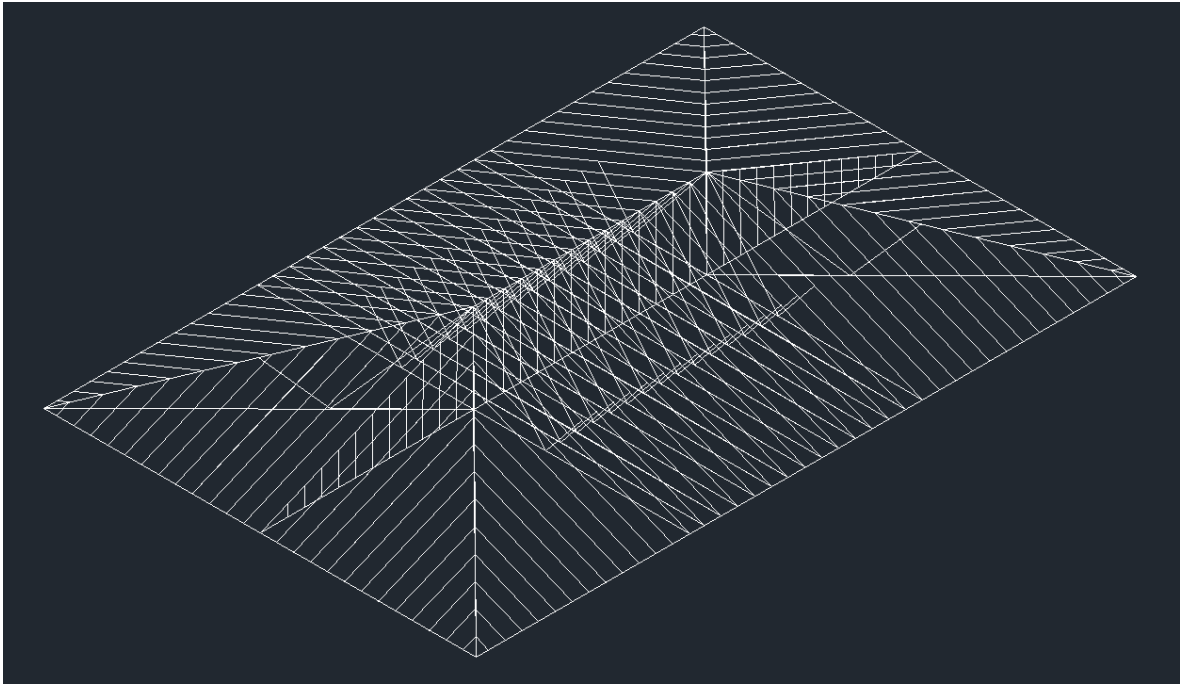


Figure 43: 3D Roof Structure



# Appendix E: Experimental Testing

## Testing Results Charts

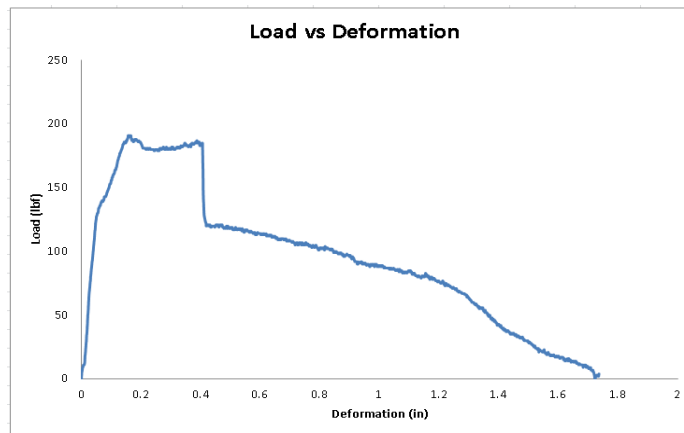


Figure 44: Toenail Test One, Load vs. Deformation

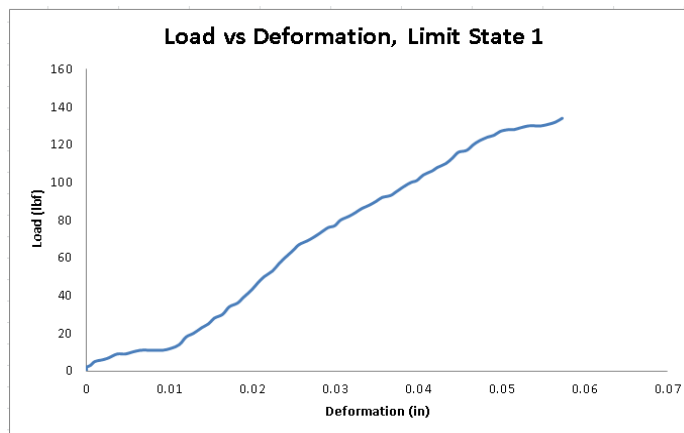


Figure 45: Toenail Test One, Load vs. Deformation, Limit State 1

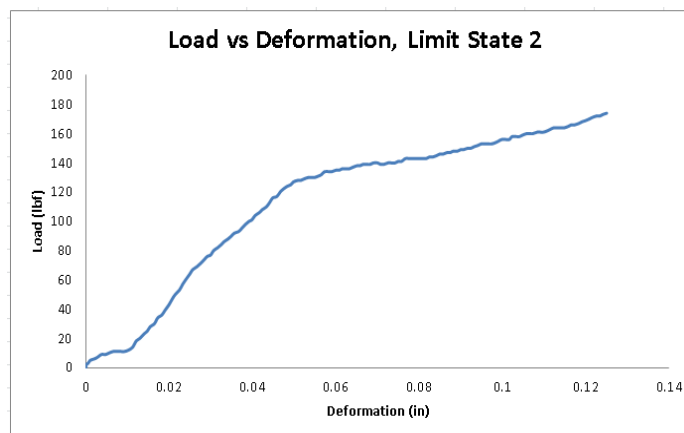


Figure 46: Toenail Test One, Load vs. Deformation, Limit State 2

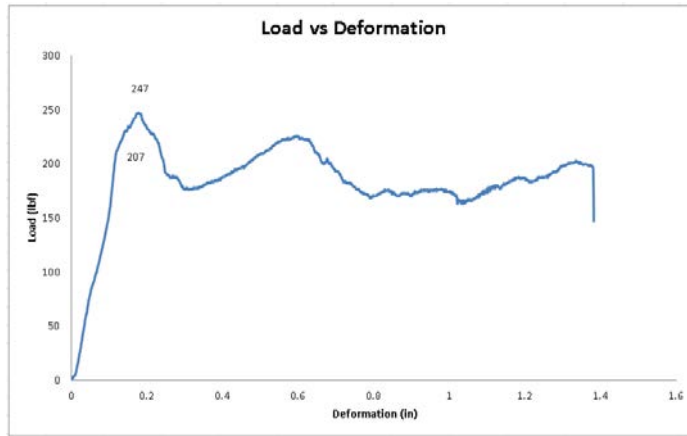


Figure 47: Toenail Test Two, Load vs. Deformation

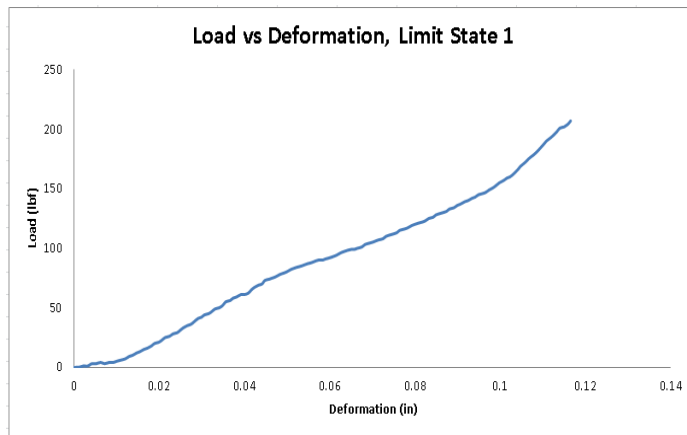


Figure 48: Toenail Test Two, Load vs. Deformation, Limit State 1

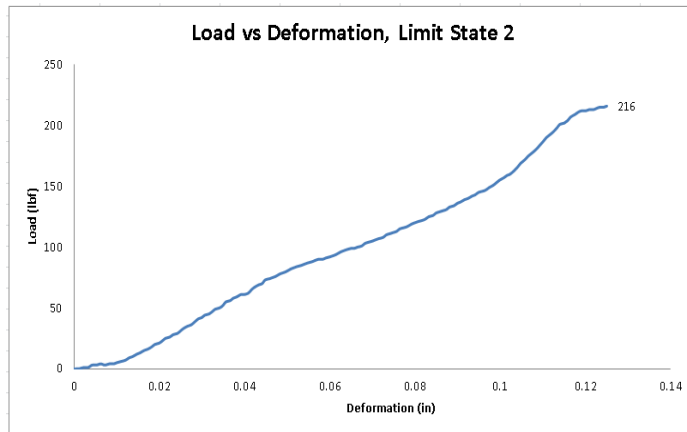


Figure 49: Toenail Test Two, Load vs. Deformation, Limit State 2

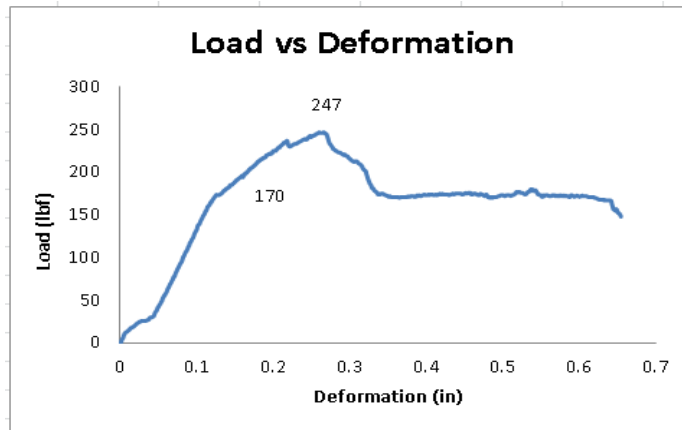


Figure 50: Toenail Test Three, Load vs. Deformation

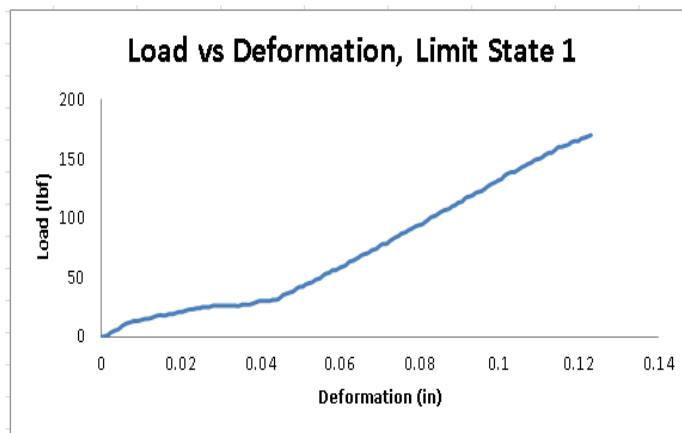


Figure 51: Toenail Test Three, Load vs. Deformation, Limit State 1

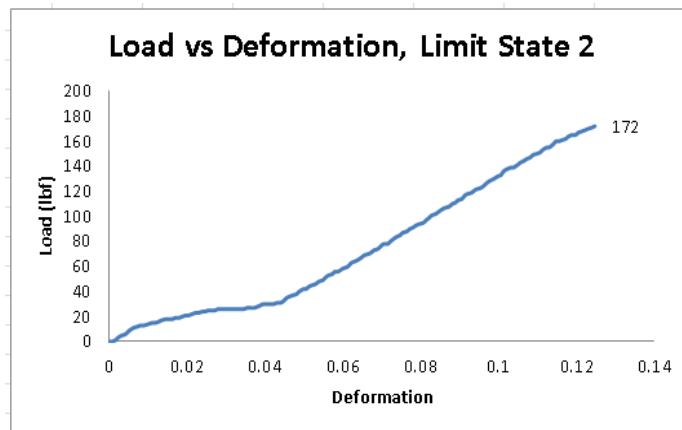


Figure 52: Toenail Test Three, Load vs. Deformation, Limit State 2

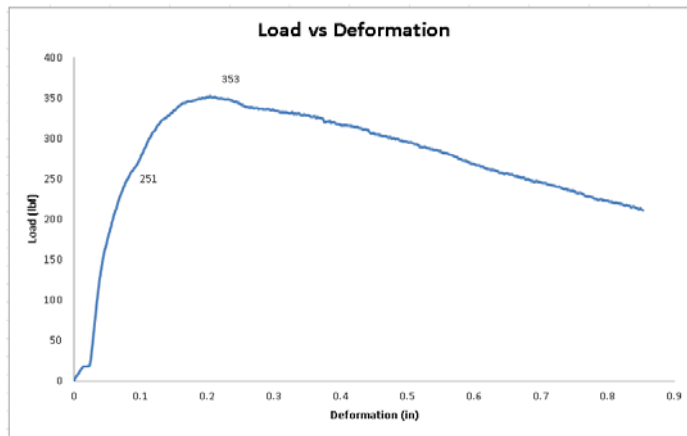


Figure 53: Toenail Test Four, Load vs. Deformation

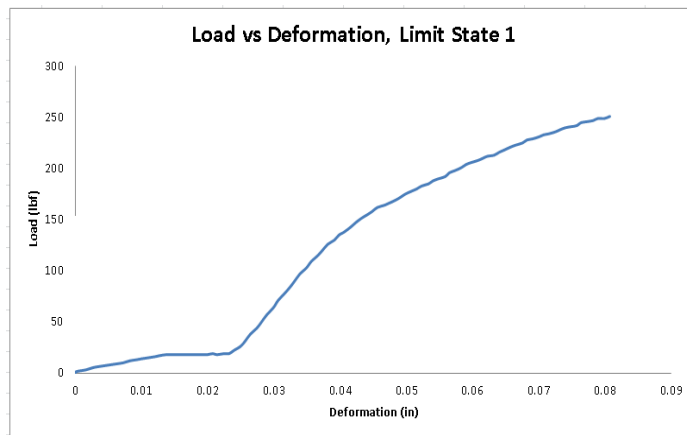


Figure 54: Toenail Test Four, Load vs. Deformation, Limit State 1

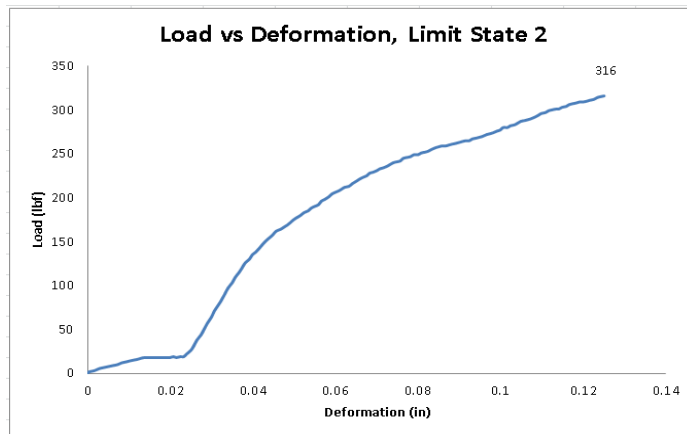


Figure 55: Toenail Test Four, Load vs. Deformation, Limit State 2

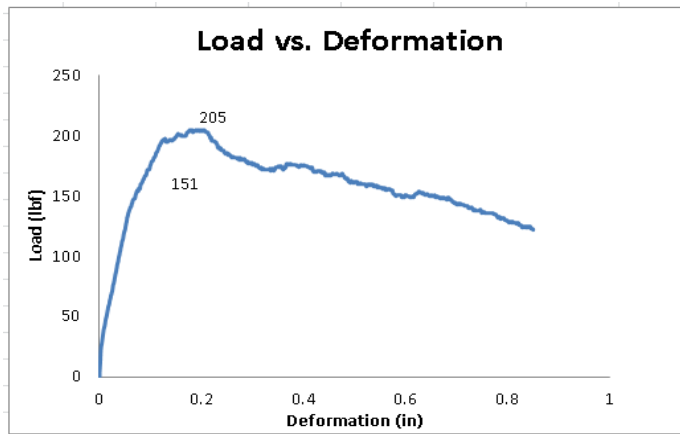


Figure 56: Toenail Test Five, Load vs. Deformation

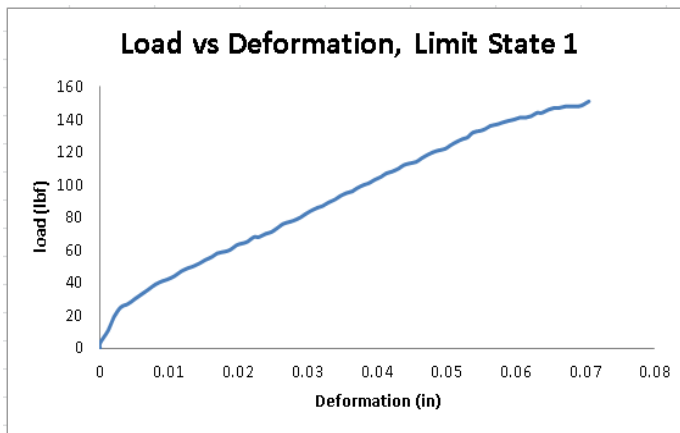


Figure 57: Toenail Test Five, Load vs. Deformation, Limit State 1

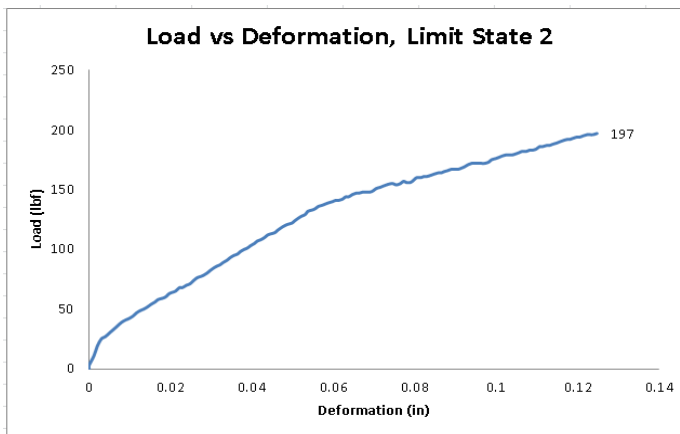


Figure 58: Toenail Test Five, Load vs. Deformation, Limit State 2

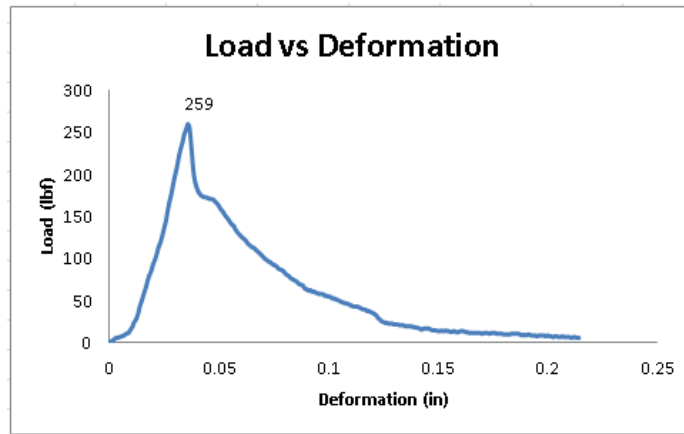


Figure 59: Liquid Nails Test One, Load vs. Deformation

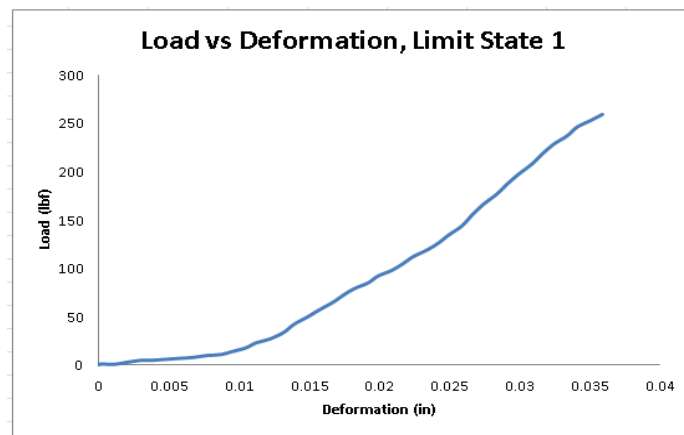


Figure 60: Liquid Nails Test One, Load vs. Deformation, Limit State 1

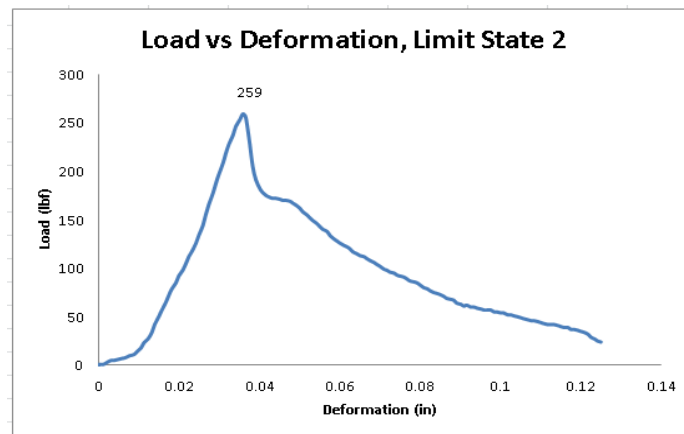


Figure 61: Liquid Nails Test One, Load vs. Deformation, Limit State 2

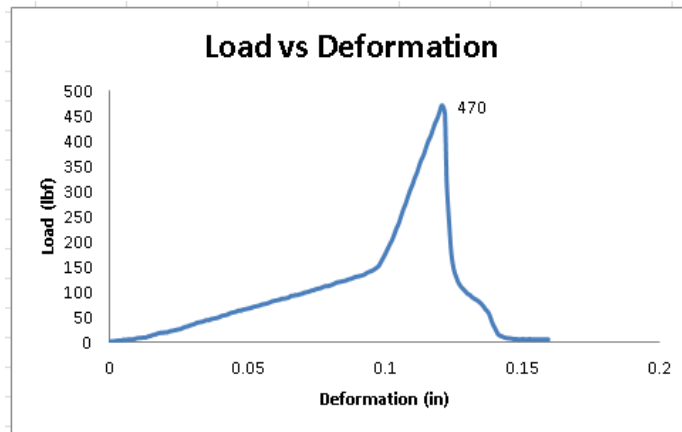


Figure 62: Liquid Nails Test Two, Load vs. Deformation

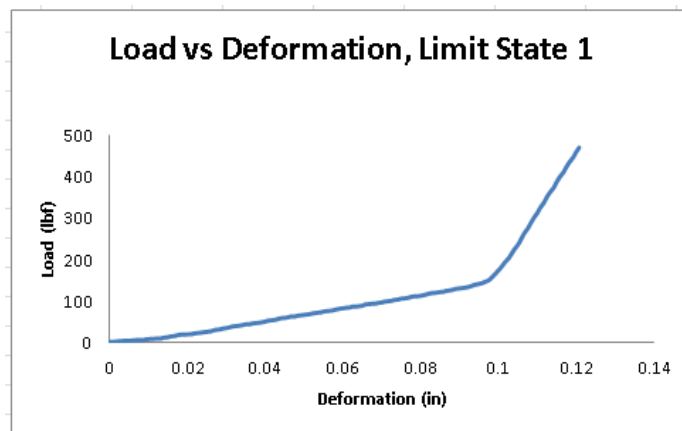


Figure 63: Liquid Nails Test Two, Load vs. Deformation, Limit State 1

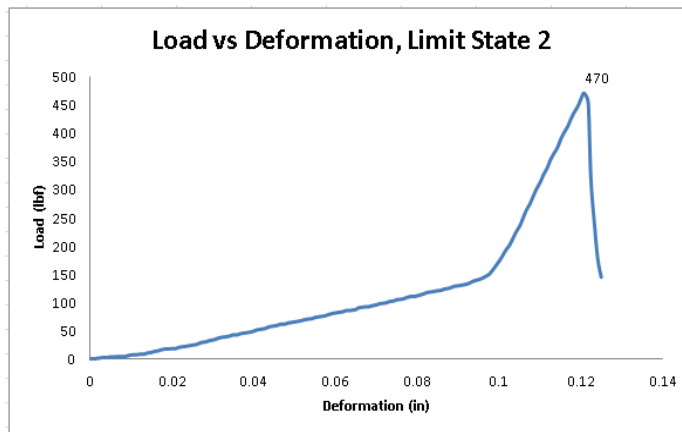


Figure 64: Liquid Nails Test Two, Load vs. Deformation, Limit State 2

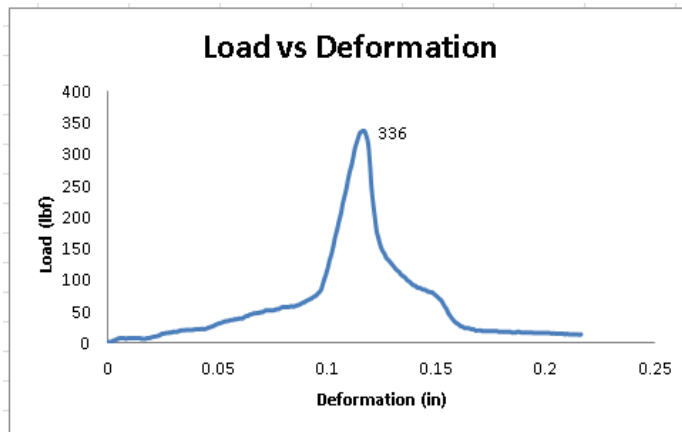


Figure 65: Liquid Nails Test Three, Load vs. Deformation

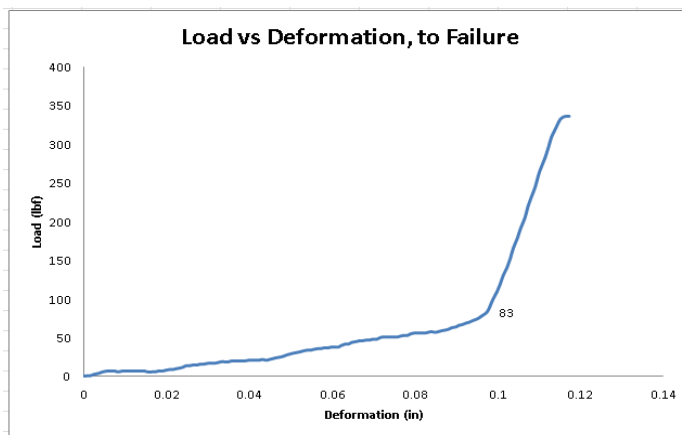


Figure 66: Liquid Nails Test Three, Load vs. Deformation, Limit State 1

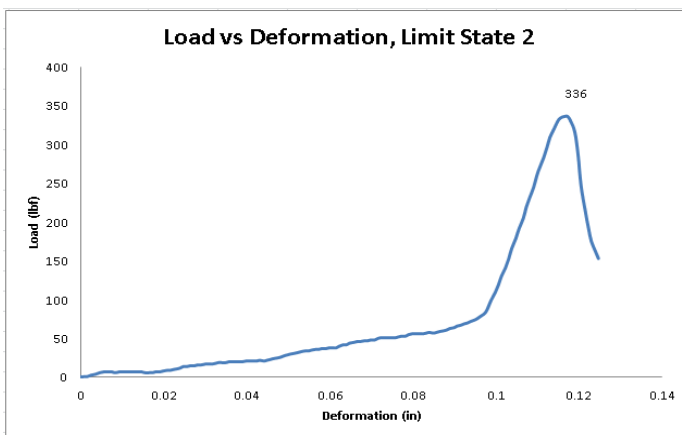


Figure 67: Liquid Nails Test Three, Load vs. Deformation, Limit State 2



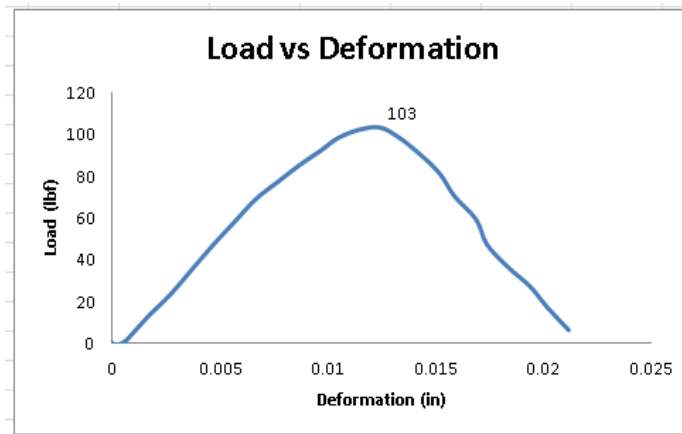


Figure 68: Liquid Nails Test Four, Load vs. Deformation

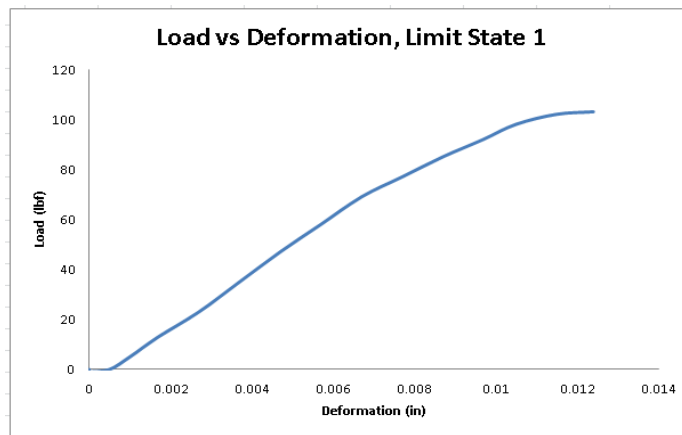


Figure 69: Liquid Nails Test Four, Load vs. Deformation, Limit State 1

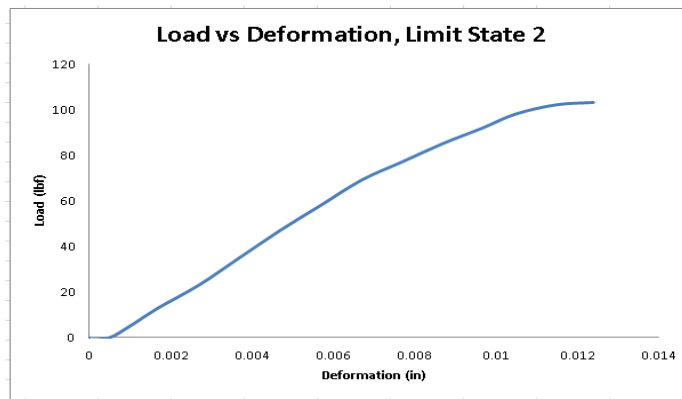
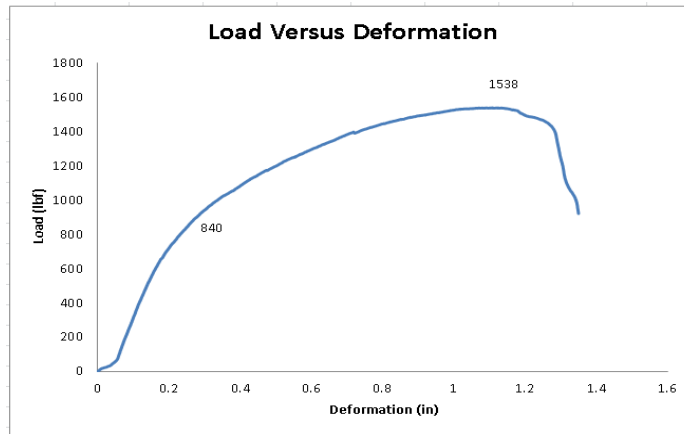
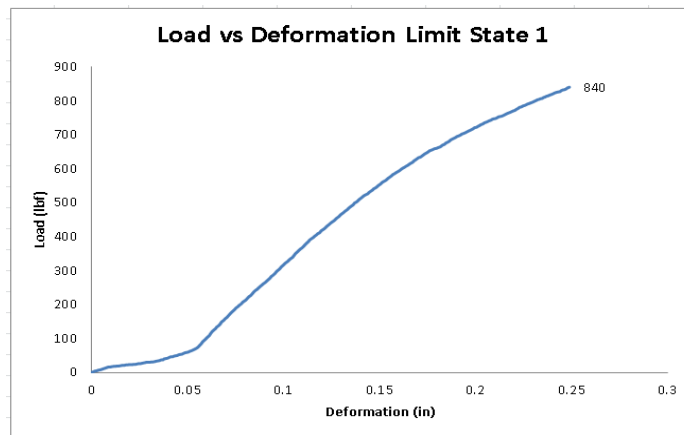


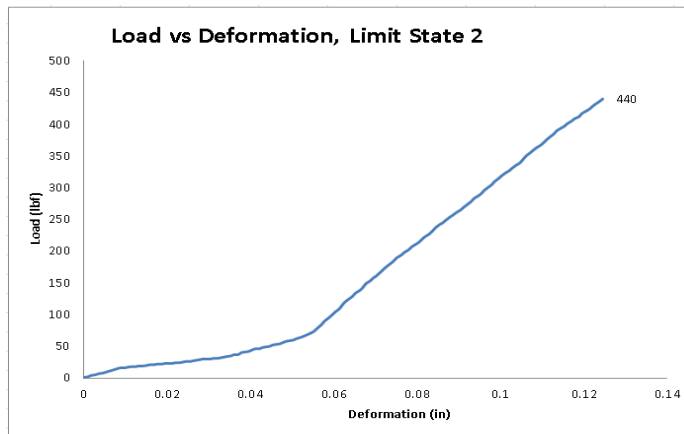
Figure 70: Liquid Nails Test Four, Load vs. Deformation, Limit State 2



**Figure 71: Toenail with Hurricane Strap Test One, Load vs. Deformation**



**Figure 72: Toenail with Hurricane Strap Test One, Load vs. Deformation, Limit State 1**



**Figure 73: Toenail with Hurricane Strap Test One, Load vs. Deformation, Limit State 2**

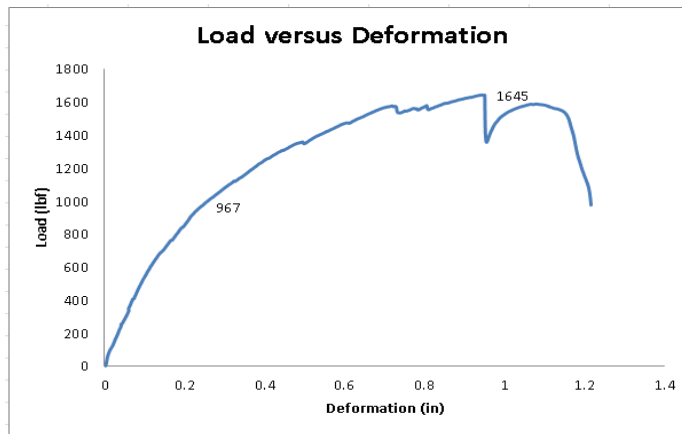


Figure 74: Toenail with Hurricane Strap Test Two, Load vs. Deformation

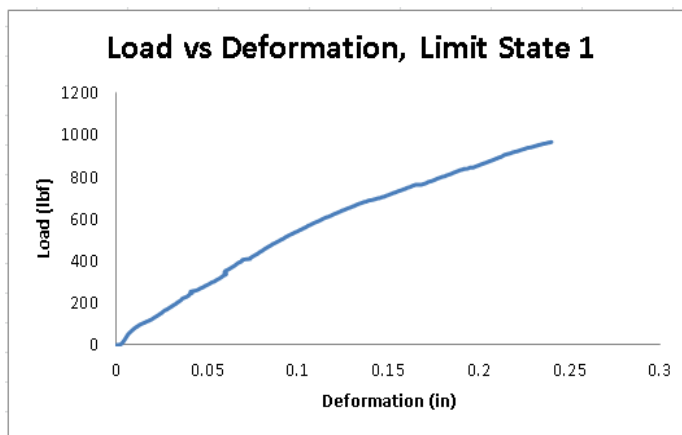


Figure 75: Toenail with Hurricane Strap Test Two, Load vs. Deformation, Limit State 1

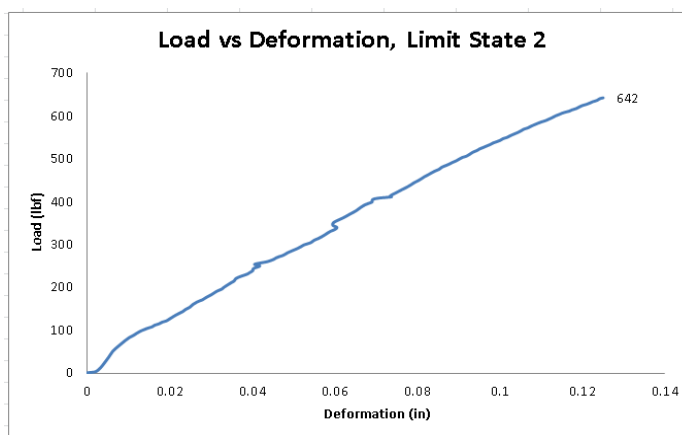


Figure 76: Toenail with Hurricane Strap Test Two, Load vs. Deformation, Limit State 2

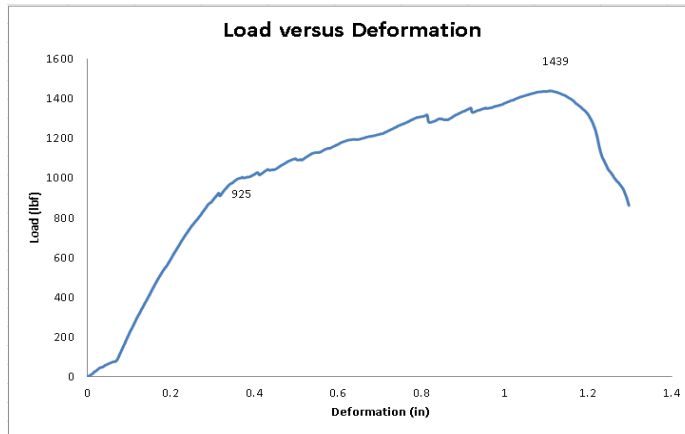


Figure 77: Toenail with Hurricane Strap Test Three, Load vs. Deformation

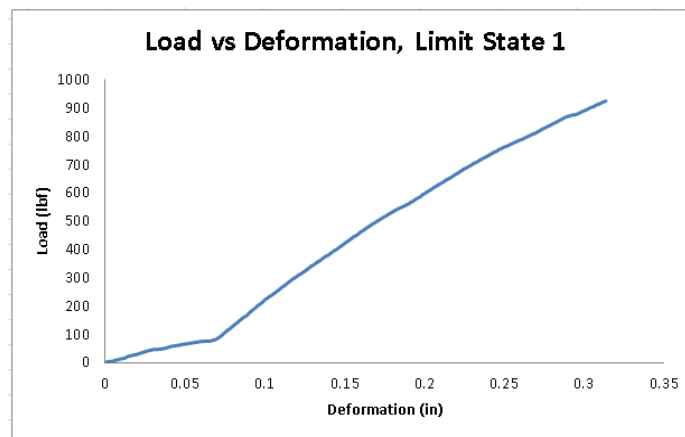


Figure 78: Toenail with Hurricane Strap Test Three, Load vs. Deformation, Limit State 1

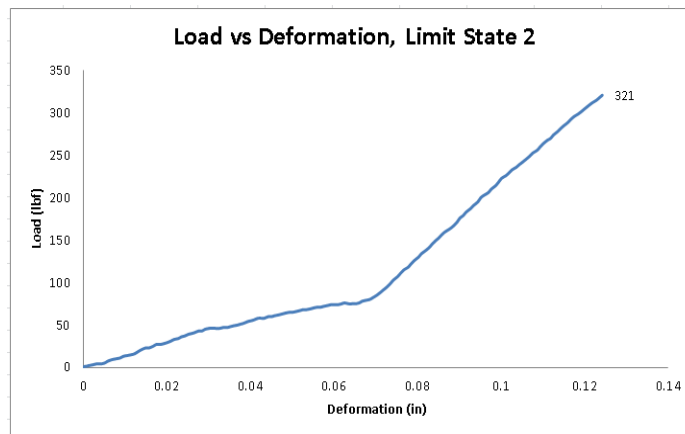


Figure 79: Toenail with Hurricane Strap Test Three, Load vs. Deformation, Limit State 2

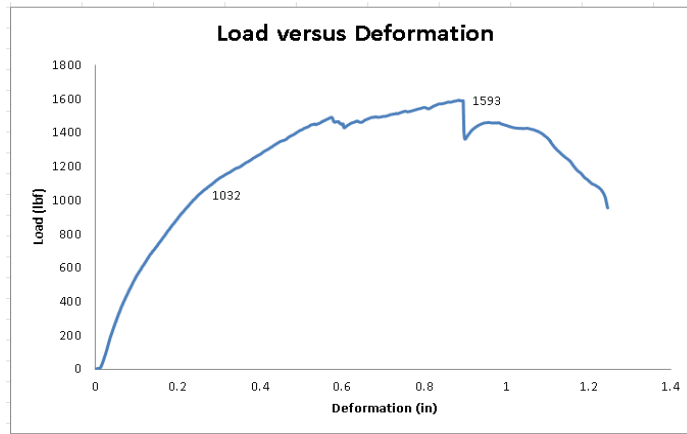


Figure 80: Toenail with Hurricane Strap Test Four, Load vs. Deformation

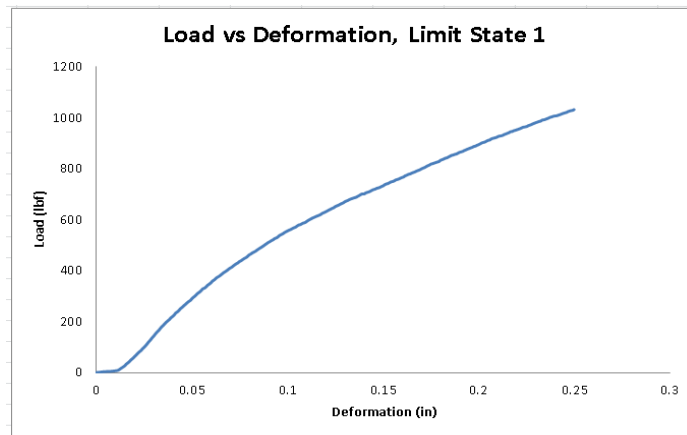


Figure 81: Toenail with Hurricane Strap Test Four, Load vs. Deformation, Limit State 1

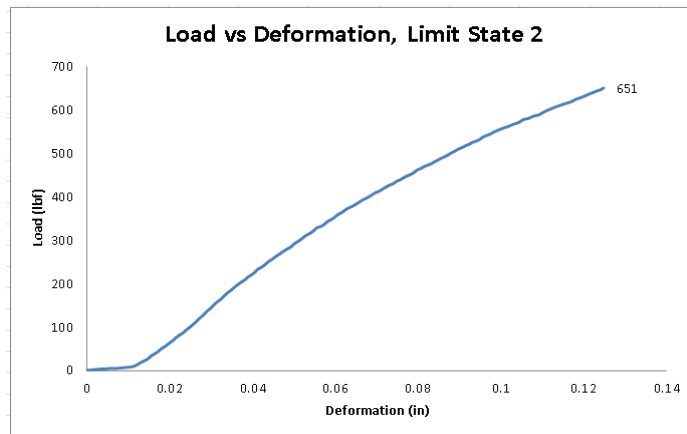


Figure 82: Toenail with Hurricane Strap Test Four, Load vs. Deformation, Limit State 2

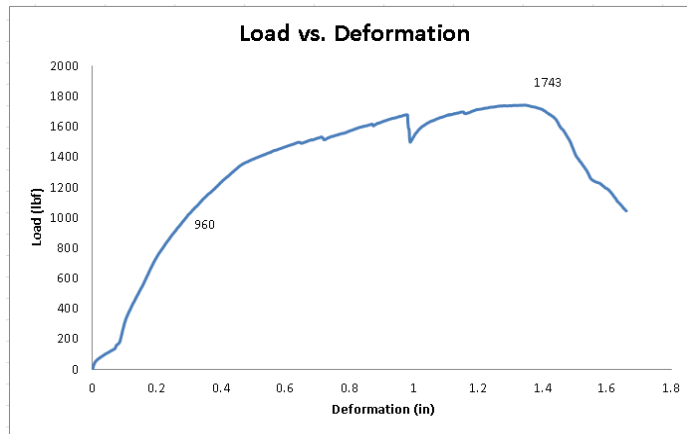


Figure 83: Toenail with Hurricane Strap Test Five, Load vs. Deformation

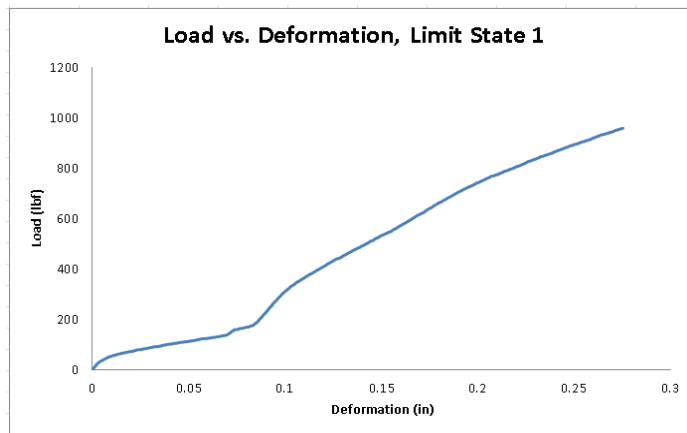


Figure 84: Toenail with Hurricane Strap Test Five, Load vs. Deformation, Limit State 1

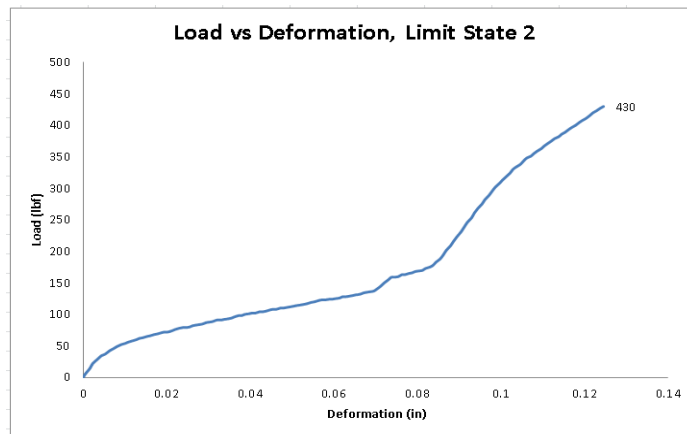


Figure 85: Toenail with Hurricane Straps Test Five, Load vs. Deformation, Limit State 2

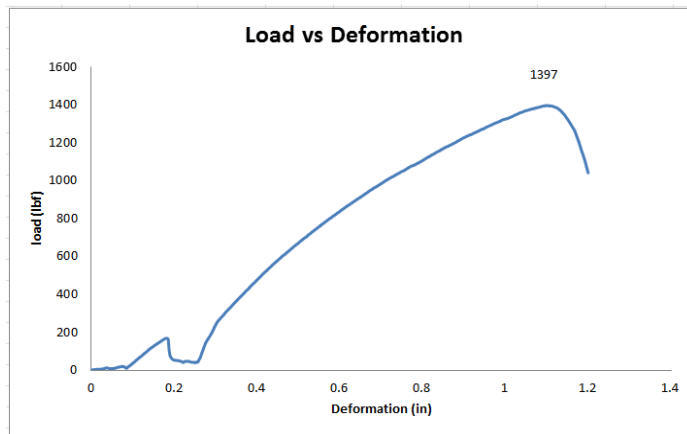


Figure 86: Liquid Nails with Hurricane Strap Test One, Load vs. Deformation

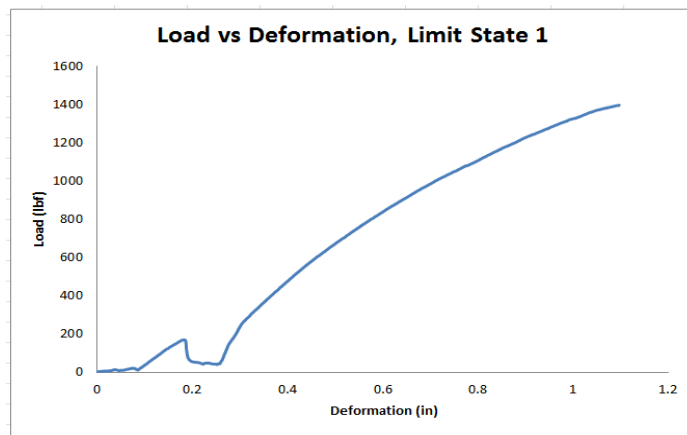


Figure 87: Liquid Nails with Hurricane Strap Test One, Load vs. Deformation, Limit State 1

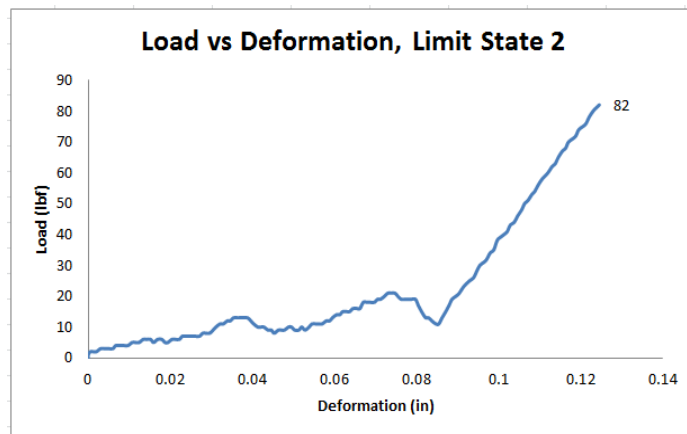


Figure 88: Liquid Nails with Hurricane Strap Test One, Load vs. Deformation, Limit State 2

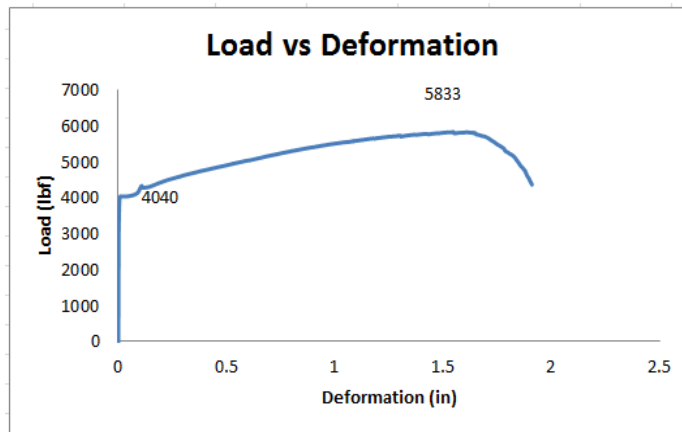


Figure 89: Liquid Nails with Hurricane Strap Test Two, Load vs. Deformation

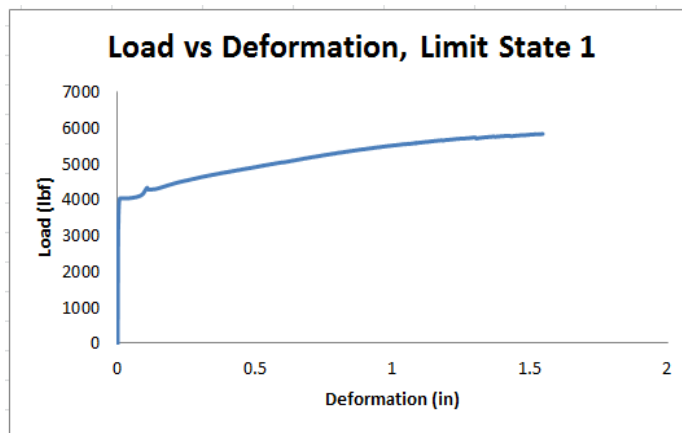


Figure 90: Liquid Nails with Hurricane Strap Test Two, Load vs. Deformation, Limit State 1

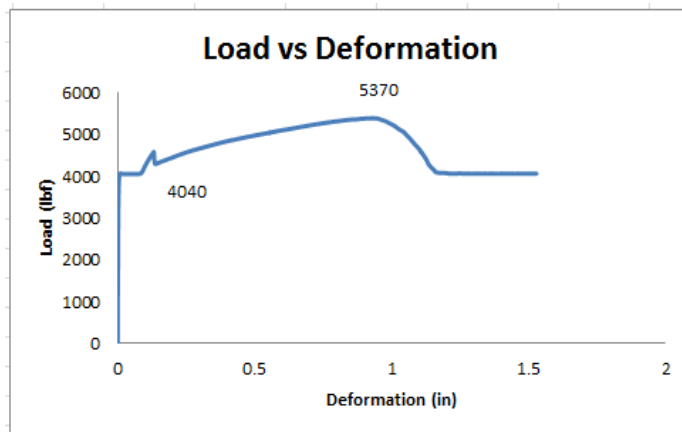


Figure 91: Liquid Nails with Hurricane Strap Test Two, Load vs. Deformation, Limit State 2



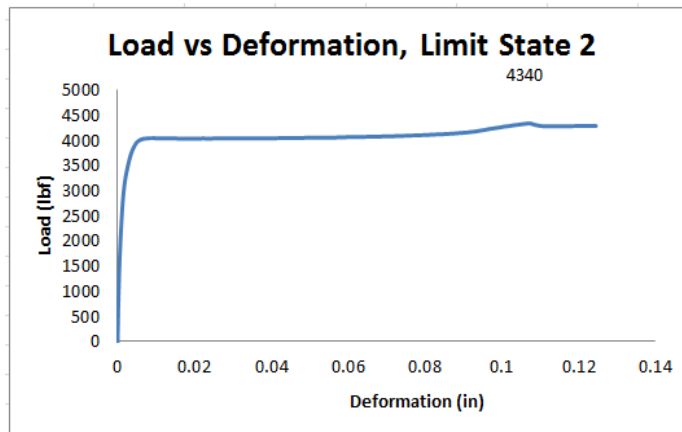


Figure 92: Liquid Nails with Hurricane Strap Test Three, Load vs. Deformation

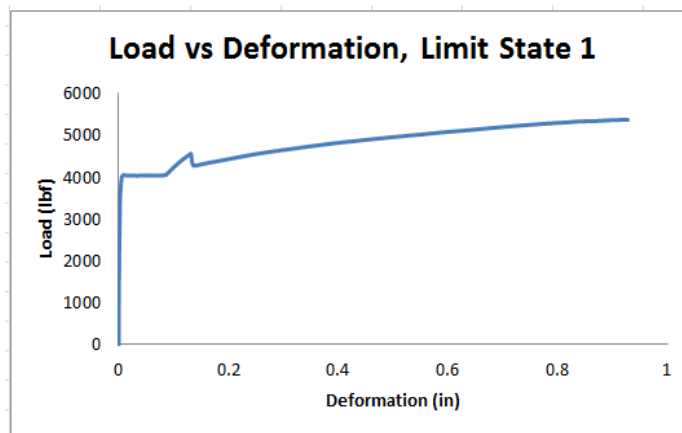


Figure 93: Liquid Nails with Hurricane Strap Test Three, Load vs. Deformation, Limit State 1

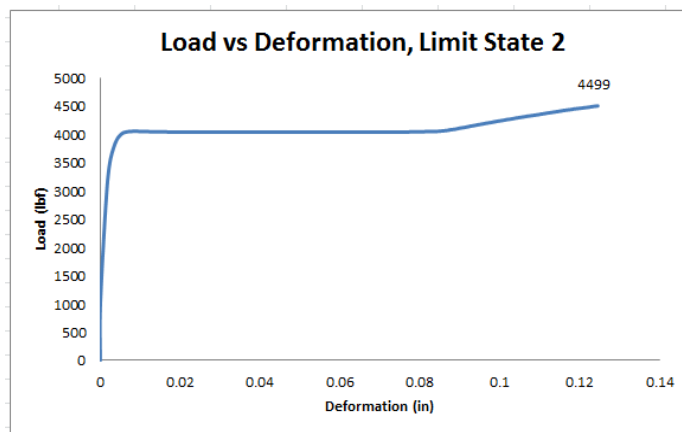


Figure 94: Liquid Nails with Hurricane Strap Test Three, Load vs. Deformation. Limit State 2

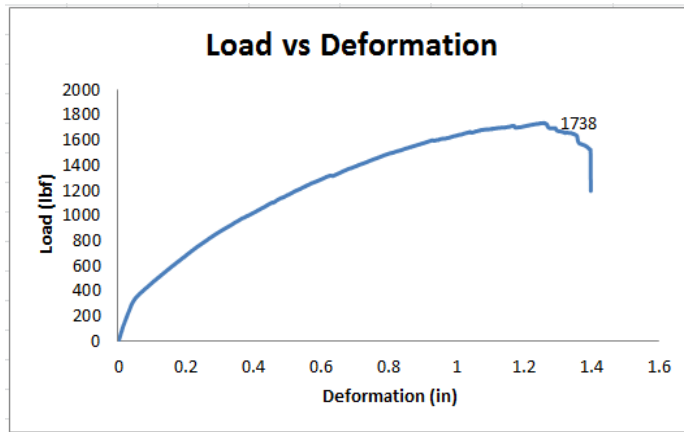


Figure 95: Liquid Nails with Hurricane Strap Test Four, Load vs. Deformation

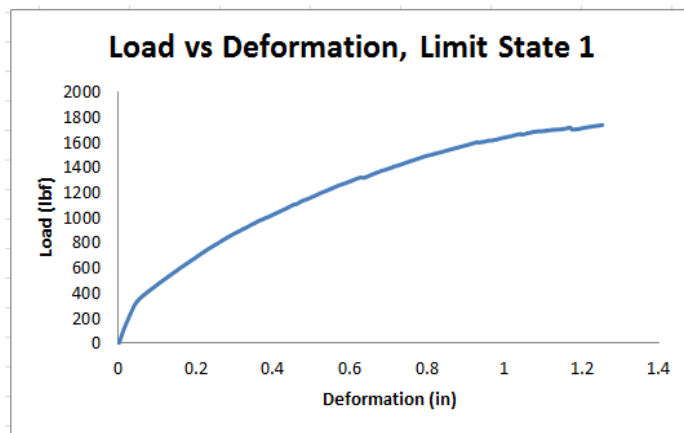


Figure 96: Liquid Nails with Hurricane Strap Test Four, Load vs. Deformation, Limit State 1

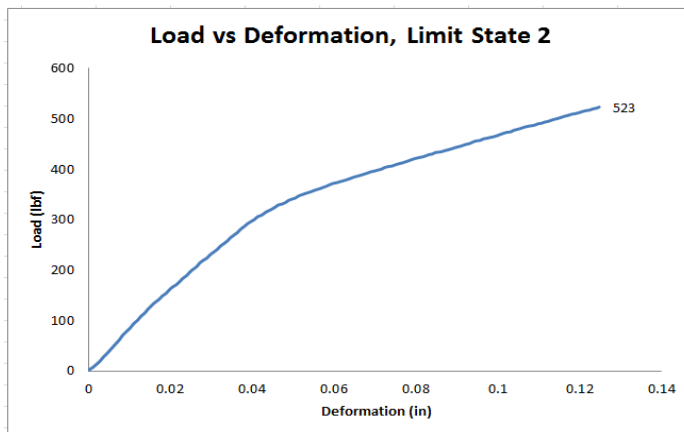


Figure 97: Liquid Nails with Hurricane Strap Test Four, Load vs. Deformation, Limit State 2

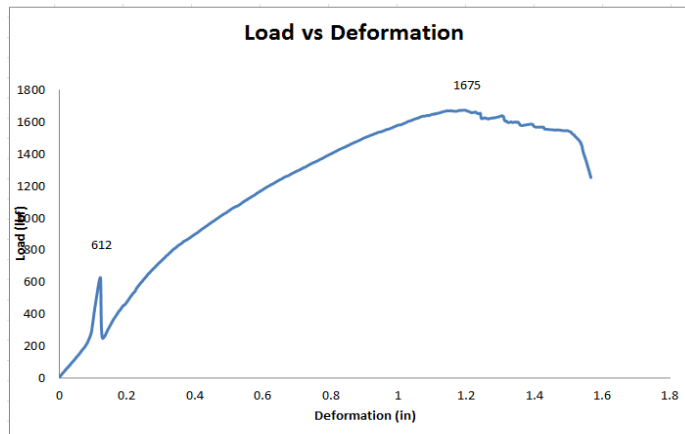


Figure 98: Liquid Nails with Hurricane Strap Test Five, Load vs. Deformation

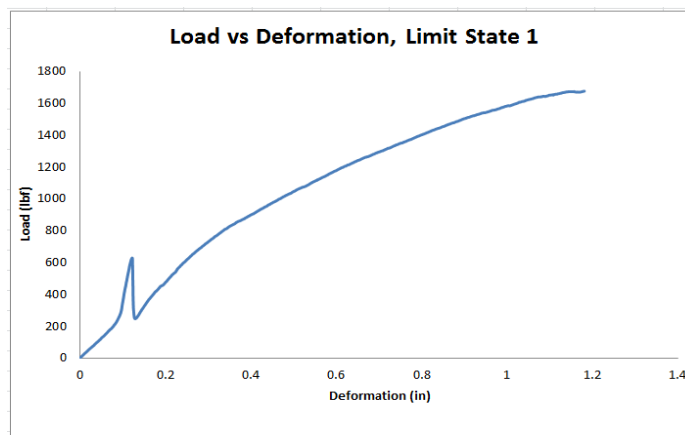


Figure 99: Liquid Nails with Hurricane Strap Test Five, Load vs. Deformation, Limit State 1

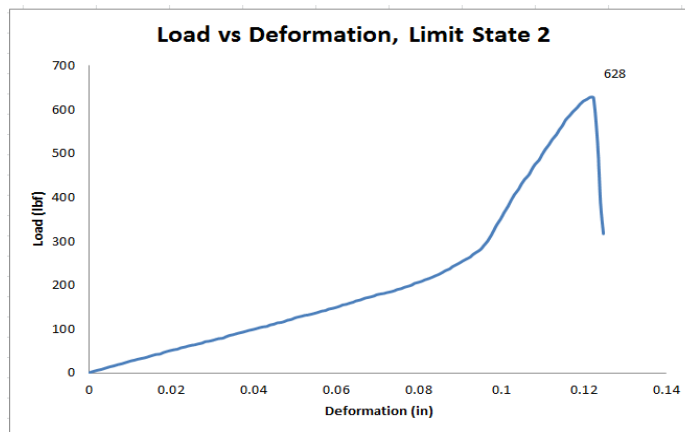


Figure 100: Liquid nails with Hurricane Strap Test Five, Load vs. Deformation, Limit State 2

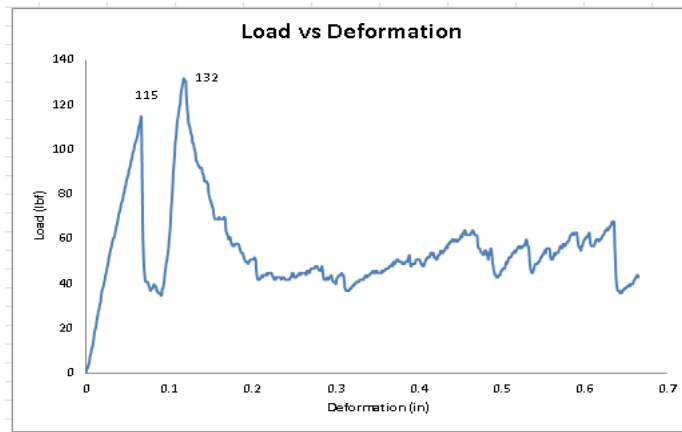


Figure 101: Kevlar Strap Test One, Load vs. Deformation

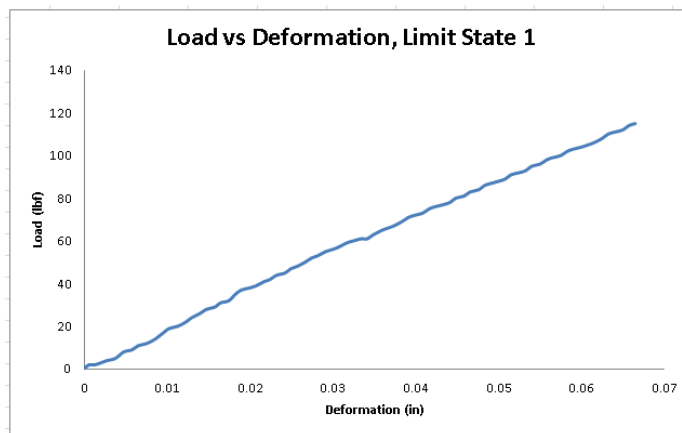


Figure 102: Kevlar Strap Test One, Load vs. Deformation, Limit State 1

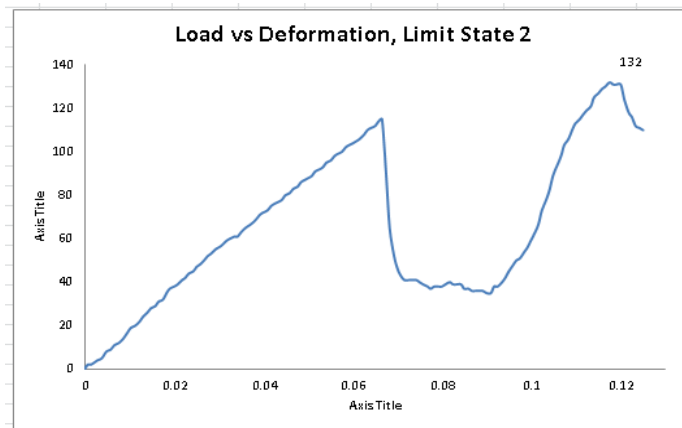


Figure 103: Kevlar Strap Test One, Load vs. Deformation, Limit State 2

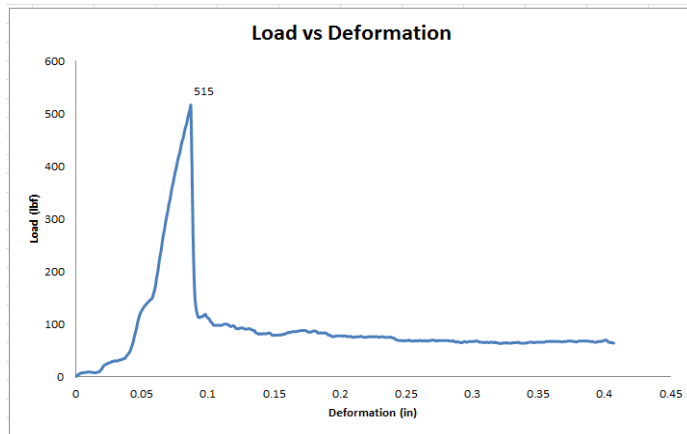


Figure 104: Kevlar Strap Test Two, Load vs. Deformation

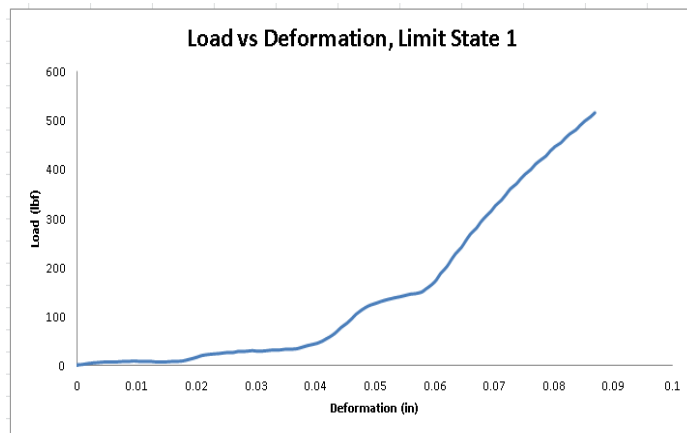


Figure 105: Kevlar Strap Test Two, Load vs. Deformation, Limit State 1

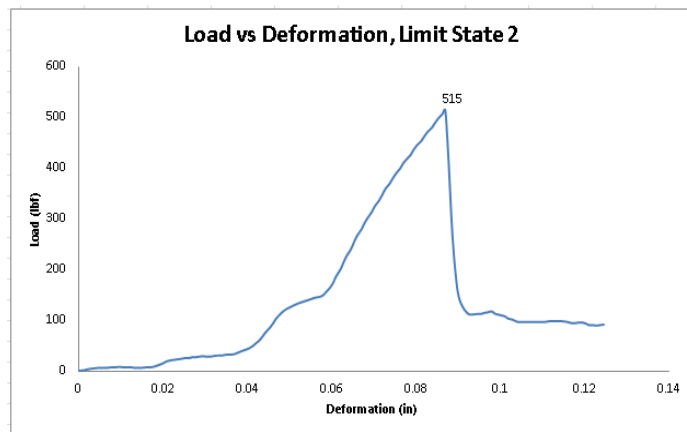


Figure 106: Kevlar Strap Test Two, Load vs. Deformation, Limit State 2

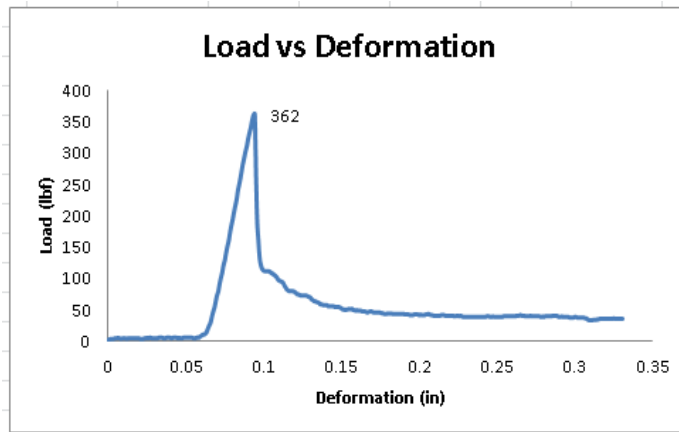


Figure 107: Kevlar Strap Test Three, Load vs. Deformation

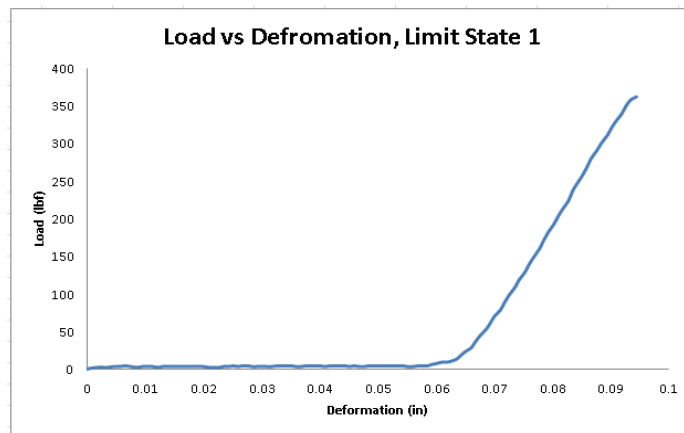


Figure 108: Kevlar Strap Test Three, Load vs. Deformation, Limit State 1

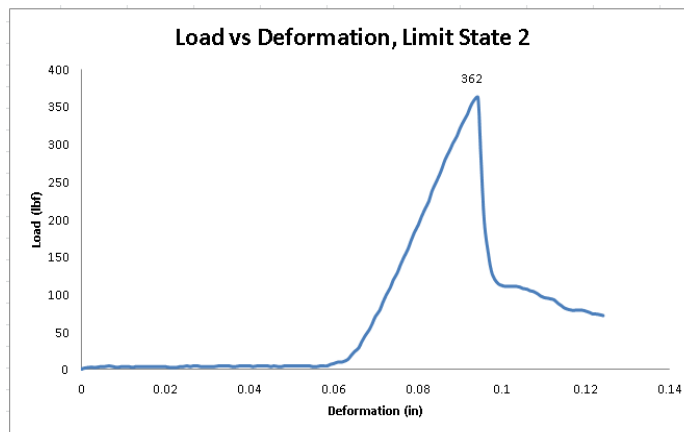


Figure 109: Kevlar Strap Test Three, Load vs. Deformation, Limit State 2

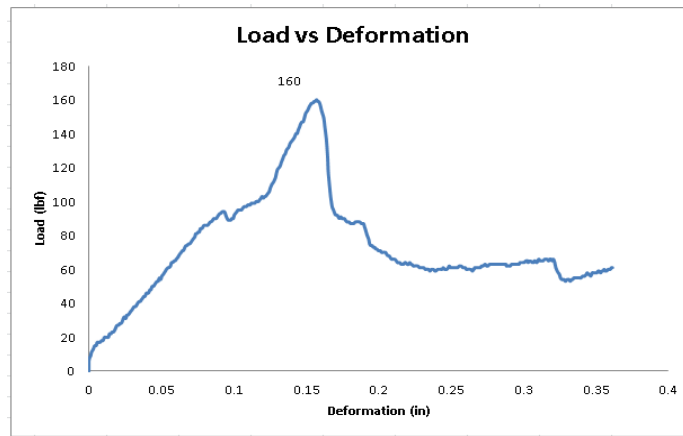


Figure 110: Kevlar Strap Test Four, Load vs. Deformation

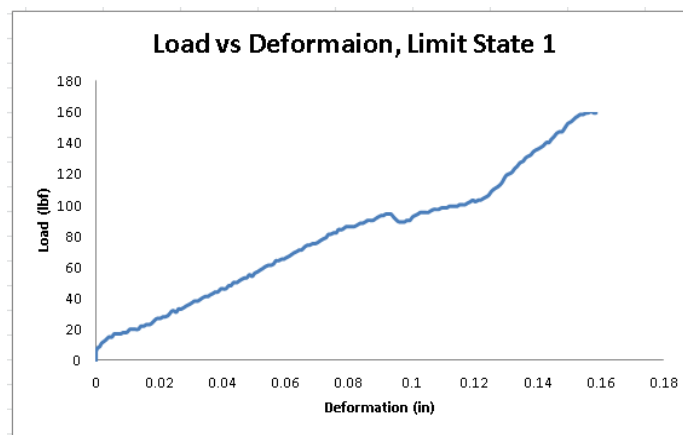


Figure 111: Kevlar Strap Test Four, Load vs. Deformation, Limit State 1

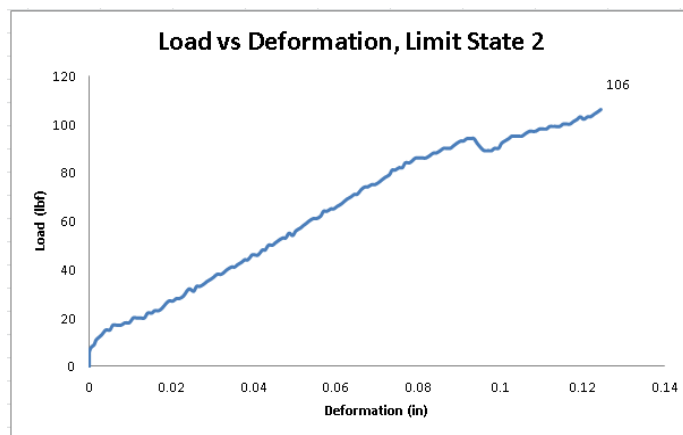


Figure 112: Kevlar Strap Test Four, Load vs. Deformation, Limit State 2

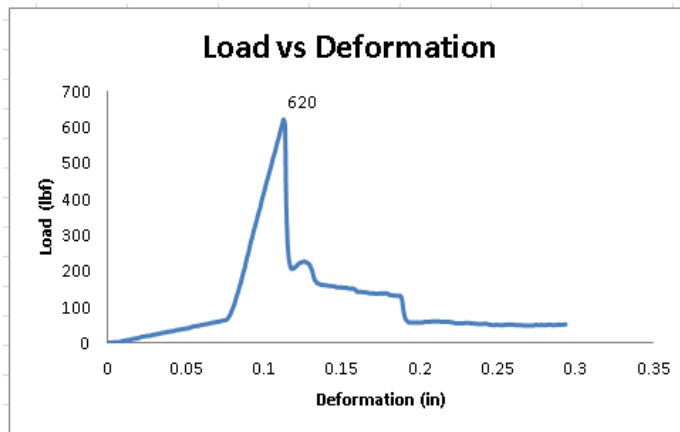


Figure 1127: Kevlar Strap Test Five, Load vs. Deformation

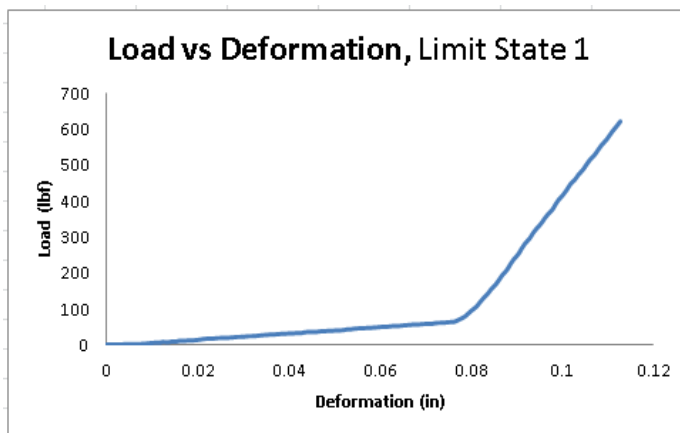


Figure 114: Kevlar Strap Test Five, Load vs. Deformation, Limit State 1

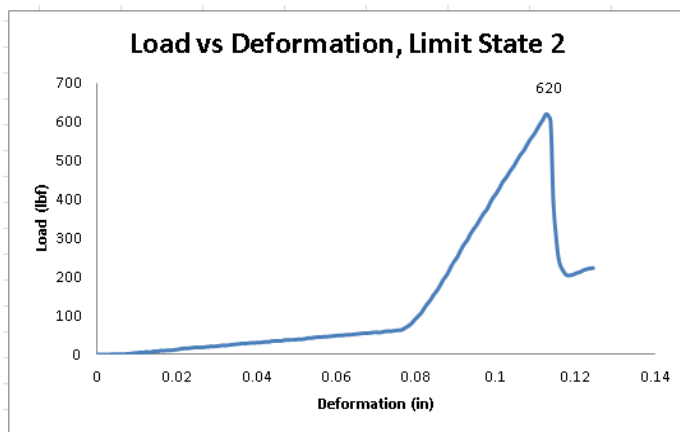


Figure 115: Kevlar Strap Test Five, Load vs. Deformation, Limit State 2



## Testing Results Tables

**Table 31: Connection Comparison, Costs, Strength, and Rank**

	Time Loss	Material Cost	Labor Cost	Total Cost	Strength Limit State 1	Strength Limit State 2	Rank
Toenail	0	\$24.78	\$294.00	\$318.78	182.60	215.00	3
Toenail & Strap	0	\$99.12	\$596.90	\$696.02	944.80	496.80	1
Liquid Nails	1 day	\$16.03	\$739.45	\$755.48	292.00	355.00	4
Liquid Nails & Strap	1 day	\$90.37	\$1,069.08	\$1,159.45	3202.60	2014.40	2
Kevlar	1 day	\$396.82	\$739.45	\$1,136.27	354.40	347.00	5

**Table 32: Toenail Cost Analysis**

Toenail				
Materials	Cost (\$)	# per connection	# of connections	Cost (\$)
16d Nails	\$0.07	3	118	\$24.78
Time				
hrs/ connection	# of connections	Total Hours		
0.03	118	3.89		
Wage/ hr	Total Hours	Labor Costs		
\$75.50	3.89	\$294.00		
<b>Total Cost</b>	<b>\$ 318.78</b>			

**Table 33: Liquid Nail Cost Analysis**

Liquid Nails				
Materials	Cost (\$)	# per connection	# of connections	Cost (\$)
Liquid Nails	\$2.47	0.055	118	\$16.03
Time				
hrs/ connection	# of connections	Total Hours		
0.08	118	9.79		
Wage/ hr	Total Hours	Labor Costs		
\$75.50	9.79	\$739.45		
<b>Total Cost</b>	<b>\$755.48</b>			

**Table 34: Toenail and Hurricane Strap Cost Analysis**

Toenail & Hurricane Strap				
Materials	Cost (\$)	# per connection	# of connections	Cost (\$)
16d Nails	\$0.07	3	118	\$24.78
Usp Hurricane Strap	\$0.33	1	118	\$38.94
8d USP Nails	\$0.03	10	118	\$35.40
			Total Materials	\$99.12
Time				
hrs/ connection	# of connections	Total Hours		
0.07	118	7.91		
Wage/ hr	Total Hours	Labor Costs		
\$75.50	7.91	\$596.90		
<b>Total Cost</b>		<b>\$696.02</b>		

**Table 35: Liquid Nail and Hurricane Strap Cost Analysis**

Liquid Nails and Hurricane Strap				
Materials	Cost (\$)	# per connection	# of connections	Cost (\$)
Liquid Nails	\$2.47	0.06	118	\$16.03
Usp Hurricane Strap	\$0.33	1	118	\$38.94
8d USP Nails	\$0.03	10	118	\$35.40
			Total Materials	\$90.37
Time				
hrs/ connection	# of connections	Total Hours		
0.12	118	14.16		
Wage/ hr	Total Hours	Labor Costs		
\$75.50	14.16	\$1,069.08		
<b>Total Cost</b>		<b>\$1,159.45</b>		

**Table 36: Kevlar Cost Analysis**

Kevlar				
Materials	Cost (\$)	# per connection	# of connections	Cost (\$)
Kevlar	\$43.72	0.04	118	\$221.84
Adhesive	\$114.07	0.01	118	\$174.98
			Total Materials	\$396.82
Time				
hrs/ connection	# of connections	Total Hours		
0.08	118	9.79		
Wage/ hr	Total Hours	Labor Costs		
\$75.50	9.79	\$739.45		
<b>Total Cost</b>		<b>\$1,136.27</b>		

**Table 37: Cost Analysis of All Materials**

<b>Material Costs</b>			
<b>8d Nails</b>		<b>16d Nails</b>	
111 nails / lb		54 nails / lb	
5550 nails / box		200 nails / box	
\$187.63 / box		\$14.77 / box	
\$0.03	Price per Nail	\$0.07	Price per Nail
<b>Hurricane Strap</b>		<b>Liquid Nails</b>	
100 / box		\$2.47	Price per Tube
\$33.44 / box			
\$0.33	Price per Strap		
<b>Kevlar</b>		<b>Fiberglass Resin</b>	
\$43.72	Price per Yard	\$114.07	Price per Gallon

**Appendix F: Testing Photos**



**Figure 116: Top Plate Section**



**Figure 117: Toenail Connection Specimen**



**Figure 118: Toenail and Hurricane Strap Connection Specimens**



**Figure 119: Liquid Nail Connection Specimen**





**Figure 120: Liquid Nail and Hurricane Strap Connection Specimen**



**Figure 121: Tinius Olsen Universal Testing Machine**

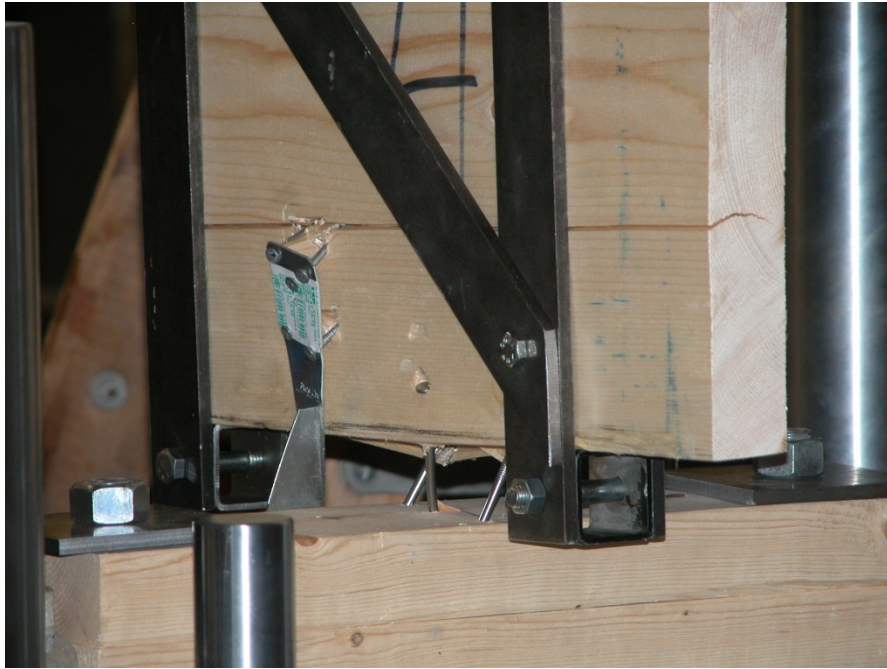


**Figure 122: Loading the Specimen in the Test Machine**

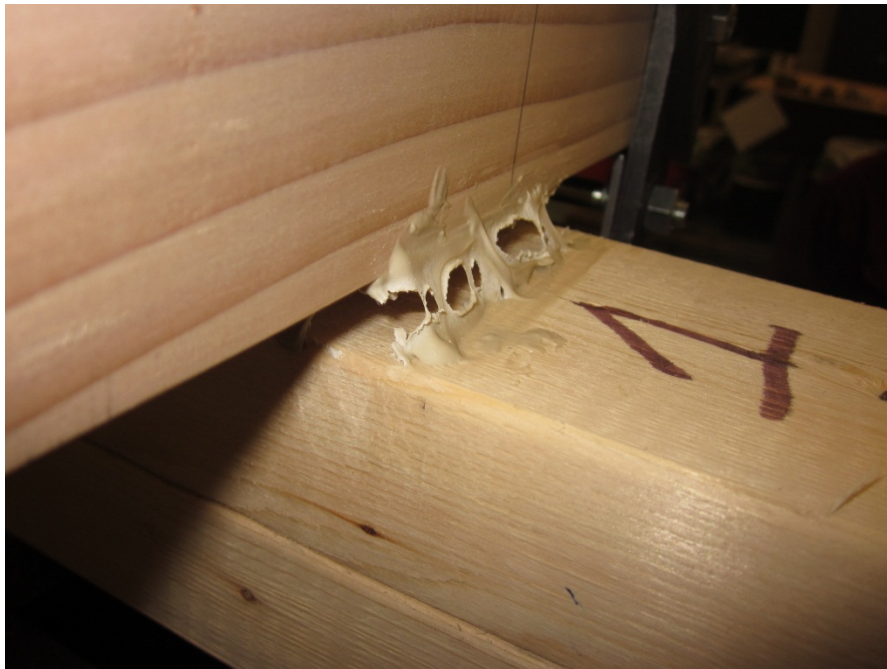


**Figure 123: Deformed Toenail Connection Specimen**





**Figure 124: Deformed Toenail and Hurricane Strap Connection Specimen**



**Figure 125: Deformed Liquid Nail Connection Specimen**





**Figure 126: Deformed Liquid Nail and Hurricane Strap Connection Specimen**