Rapidly Deployable Structures for Disaster Relief and Military Use

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Abstract

Rapidly deployable structures currently play a vital role in the recovery aspect of disaster-stricken areas and the safety of military personnel in hostile zones. More recently there has also been a need for the structures to deal with unforeseen disruptions caused by the COVID pandemic, such as COVID testing centers or backyard pop-up offices. These structures could be used in different environments and serve many other purposes. Along with the structural performance of these shelters, they need to integrate into systems set in place effortlessly. For that reason, four key factors were taken into consideration, namely (1) deployability (how difficult is it for the system to be fully deployed), (2) transportability (how difficult is it to ship these structures to the target locations), (3) cost, and (4) building envelope resistance (the resistance and durability of the building’s envelope to unforeseen hazards). This study aims to design a rapidly deployable structure for emergency response such as disaster relief with the potential for military applications. This research begins by covering existing deployable shelters used for disaster relief and military applications. Next, it presents a few alternative designs using different materials and different functionality. Then, it addresses how these new designs compare to the existing shelters in each category. Finally, it discusses tests performed on the folding mechanism, hinges, being used throughout the design of a deployable wood structure. The testing in this research shows a method that can be used to quantify the capacity of hinges. As a result of the study, new designs are shown to be competitive with the existing ones. The results of an experimental study of how hinges perform as structural elements will be discussed.
Dedication

This work is dedicated to, Kenzlie and Milo, who have been a constant source of support and encouragement throughout this research.

This work is also dedicated to my parents, Brian and Kelly, who have always loved me unconditionally and have raised me to be who I am today.

I would also like to thank my little brother, Kaden, and my grandparents for supporting me through this journey.

In loving memory of my Grandma
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Chapter 1: Introduction

1.1 Research Motivation and Objective

It is not always possible to know when disaster will strike, and planning for these events can be difficult. Disaster can take the shape of many forms, including but not limited to natural disasters, man-made disasters, war, and, more recently, pandemics. When these events happen, there is potential for people to be displaced from their homes and resources to be overwhelmed. This immediately brings the need for new structures, such as homes and medical facilities. Building such structures from the ground up can be time-consuming and intensive on labor and materials. Therefore, prefabricated, or deployable structures could be put into place instead. The need for deployable structures will always be relevant and could be immediately implemented into modern society. The research motive is to improve the current solutions used to aid those affected by disasters. This motivation led to the research objective, which is to design prefabricated, turnkey structures that can be used in (1) post-disaster and (2) military applications in hostile environments where rapid deployment of structures is critical.

1.2 Potential Applications of Deployable Structures

There are two main target sectors in which these deployable structures would be aimed, post-disaster and military applications. There are many other markets and uses for these deployable structures, such as residential and commercial consumers. As seen in recent years, the need for testing facilities and quarantine zones, brought out by Coronavirus (COVID-19), came unexpectedly. Natural disasters such as hurricanes, earthquakes, tornadoes, and wildfires, along with man-made disasters such as terrorism and climate change, can bring out the need for temporary shelters. Military operations also benefit from deployable structures. Military deployable structures can be found in new bases in expeditionary regions or pop-up command
centers. Residential uses for such deployable structures could be tiny homes, backyard offices, or even vacation homes. Commercial uses could be vendor booths, construction site offices, and any other scenario or event that could benefit from a non-permanent easy to deploy structure.

1.3 Material Selection for Deployable Structures

This research consisted of four deployable structure designs along with connection testing of hinges used throughout the expandable structures. The four designs utilized two different materials and two different deployment methods.

The two materials used were Cross-Laminated Timber (CLT) panels and light-frame construction materials (i.e., dimension lumber and structural panels). CLT is a wood panel that is made up of sawn lumber like a 2x6. These panels can be prefabricated into large pieces that can range up to 60 ft long and 10 ft wide (APA, n.d.). The second material used was light-frame prefabricated panels. Light-frame construction has been around for a long time and is generally seen in residential and small commercial structures. The benefits of each material type are discussed more in detail in section 2.5. However, in general, light-frame construction is typically more cost-effective and lighter than CLT construction. CLT construction typically produces a stronger and more resilient structure when compared to light-frame alternatives. CLT construction also allows for timber to be used in mid to high-rise buildings, whereas light-frame limits timber to low- to mid-rise structures.

1.4 Scope of Research

The original objective of this research was to design a CLT structure that could provide housing and space for medical treatment for those in need following a disaster. After seeing the potential for a similar shelter system in the military, the scope was widened to include military use. In this research, factors such as ease of deployment, transportability, cost, and building
envelope resistance will be used to design a competitive deployable structure. These structures will then be graded on how well they perform in each category. This will aid the end user in determining which structure best suits their needs. The other part of this research focused on the folding mechanism used throughout the design. The folding mechanism used throughout the design of the deployable expandable structures were hinges. Experimental testing was performed on different hinge varieties and sizes.

Deployability can be subjective; however, in this research, deployability will be a factor of weight, time to assemble or erect, and required tools/machinery. A calculation was performed to determine the approximate weight of each structure. Assembly time could not be directly measured as a full-scale model was not part of this research; however, an estimation for assembly time could be made based on structures of similar size and weight. If machinery or uncommon tools are needed for assembly, this will make it more difficult to deploy the structure. Also, if extensive work is needed to complete the structure once arriving onsite, the rating associated with the deployment will decline.

Transportability, in this research, is calculated as a function of weight and size as most shipping methods have limitations on weight and size. The idea behind the post-disaster shelters is that the structures would be shipped to locations suffering from some form of disaster. For this idea to work, the units must be able to make it to the location; therefore, unrealistic sizing or weight would not be acceptable.

Cost is an important factor because these units will be made in large quantities. Therefore, small savings in a single unit could lead to substantial savings in the long run. An estimated structure cost for each unit was made. The lower the cost of the structure, the better the rating in the cost category.
Building envelope resistance is the measure of a structure’s durability and strength. The factor used to grade building envelope resistance is force protection. Force protection is the goal of protecting a structure and its inhabitants from hostile threats and hazards.

Experimental testing was performed to obtain a general understanding of how hinges could perform as structural elements. The testing in this research was not intended to provide design values for these hinges, as many more tests would need to be conducted to understand the variability of these mechanisms. This testing could also be used as formwork for future testing on hinges.
Chapter 2: Literature Review

2.1 Types of Deployable Structures

Deployable shelter systems must be flexible and adaptable. They need to be available when needed and easily stored when not in use. Disaster relief can bring these structures into emergency use; however, the military uses these structure types more frequently. The military is continuously using deployable shelters, whether it be in hostile zones or on U.S. soil. Because of the greater need for these shelter systems in the military, they have set the foundation for deployable shelters.

A committee within the Department of Defense (DoD) has taken the lead on defining deployable shelters. This committee is known as the Joint Committee on Tactical Shelters (JOCOTAS). This committee was formed to be the governing body of tactical shelters for military applications. The term tactical shelter is another way of saying deployable shelter. The JOCOTAS describes a tactical shelter as “A mobile, transportable structure designed for a functional requirement. It provides an environmentally controllable space for human habitation and use, and/or permanent equipment storage or operation” (DSP, n.d.). The DSP lists three purposes for why the JOCOTAS was formed in 1975 being: “Prevent the duplication of Tactical Shelter Research and Deployment, Eliminate the proliferation of non-standard tactical shelters in the DoD inventory, and Maximize the usage of DoD Standard Family of Tactical Shelters” (U.S. Army Natick Soldier RD&E Center, 2012). By creating the JOCOTAS, the DoD was able to create a Standard Family of Tactical Shelters that allows the shelters to be useable by more than one military branch. For example, a shelter that is commonly deployed by the army is still usable for the Air Force branch because it is in the Standard Family and therefore meets certain criteria for both branches. The JOCOTAS also aims to eliminate the duplication of research and development for these tactical shelters, which helps push the needle forward towards improvements and
integration of new technology advancements. The DoD has four criteria that make a shelter to be recognized as part of the Tactical Shelter Family. These are, “Maximum standardization for DoD shelters and continue to meet the needs of the Service. Rapid worldwide deployment of a tactical force featuring ISO (International Organization for Standardization) handling, structural and dimensional standards, where applicable, for container vessel shipment. Lower long-term deployment costs by recovering deployed structures for future uses, eliminating wasted dollars on permanent construction which is later abandoned. Rugged qualified shelters that stand up to rough use in the field and climates all over the world” (U.S. Army Natick Soldier RD&E Center, 2012).

The deployable shelters designed in this research can either be used standalone or joined together through connecting corridors or passageways to make up larger community networks. The DoD lists some uses those previous tactical shelters held, such as field kitchens, field hospitals (aid stations, operating rooms, dental facilities), supply rooms, maintenance facilities, command posts, dining areas, and barracks. Being able to attach these structures allows for complete bases to be constructed of individual units. This makes an entire base quick to construct and deconstruct without much waste of resources.

The JOCOTAS has set forth three categories in which a tactical shelter can reside. These three categories are Rigid Wall Shelters, Soft Wall Shelters, and Hybrid Shelters. A Rigid Wall Shelter is a “pre-sized” structure that requires minimal site preparation and no specialized setup. A Soft Wall Shelter is a frame or air-supported fabric structure that is erected or assembled on site. Finally, a Hybrid Shelter is a combination of rigid and soft wall shelters. A great way to get a visualization of what these structures look like is to view units the military currently uses.

The military currently has both expandable and non-expandable rigid wall tactical shelters. This paragraph contains some examples of what is currently being used by the military. The first
shelter is called “ISO Shelter, Tactical, Non-expandable, S-781/G (60 Amp).” For easier reference, National Stock Number, which is an identifying number assigned to materials and items, will be used to identify a shelter. The National Stock Number (NSN) for the first unit is 5411-01-136-9827. This unit weighs 3900lbs and is 19’ 11” long, 8’ wide, and 8’ tall. The interior dimensions of this unit are 19’ 1” L x 7’ 7” W x 7’ 1” H. This unit does not have an erection/assembly time as it is fully functional in its shipped configuration. Lights, Circuit breakers, and leveling jacks are just a few of the key things included with this unit. The next shelter, NSN 5411-01-136-9838, is an expandable unit with interior dimensions (expanded) of 18’ 4” L x 21’ 6” W x 7’ 1” H. The nonexpanded exterior dimensions are 19’ 11” L x 8’ W x 8’ H. This unit has a weight of 6900lbs and an assembly time of only 30 minutes with four soldiers. These units are designed to have dimensions consistent with a standard ISO shipping container of 20’ L x 8’ W x 8’ H. This allows for easy and consistent transportation methods. These units are also made from steel which gives the units a rugged and durable finish. Along with traditional transportation methods, the military may also drop these units from aircraft with parachutes.

Figure 2. 1: Example of Non-expandable Rigid Wall Shelter (NSN 5411-01-136-9827)
The military also utilizes soft wall shelters as a means of less permanent shelter. The soft wall shelters are typically a stretched fabric supported by either a frame or pressurized air. An example of a frame-supported soft wall shelter is NSN 5419-01-465-3025EJ. The dimensions for this unit assembled are 20’ x 32.5’. The weight of this unit is 1250 lbs., and the erection time is less than an hour with four soldiers. An example of an air-supported soft wall shelter is the Airbeam tent NSN 8340-01-558-8707. This structure is supported by a series of Airbeams that are inflated onsite. Think of a bouncy house, but instead of the entire structure inflating, only a few beams inflate and hold up the rest of the structure. The weight of this structure is around 1000 lbs. With the supplied compressor, the erection time is only 15 minutes with four soldiers. One negative associated with the structure is that electricity is required for assembly. Although not a major issue, it is something to consider.
Before designing deployable structures, it is a clever idea to view what is already being used and what better source than the military. The military uses deployable structures on a regular basis and has spent a great deal of time determining what works and what does not. There were a few major conclusions made from this literature review. The first is that the military prefers to have options. In their fleet, they have durable structures made from steel but also have tents inflated with air. These two structures are different in size and material selection, but the overall purpose of each unit is the same, to provide shelter for military personnel. The second is that transportability needs to be a major consideration. The two rigid wall shelter examples conform to standard ISO sizes. The final observation that can be made from the military structures is that deployability is...
extremely important. The structures are designed so that they can be deployed quickly and easily by nonspecialized personnel. These considerations shaped how the structures in this research were designed.

2.2 Modular Construction

Modular construction is a construction process in which components or units are constructed off-site, typically in a factory setting. These pieces are then shipped to the construction site and connected to form complete structures. An early example of modern modular construction in the United States was kit homes. Sears, Roebuck, and Company were one of the most popular producers of these homes and sold them through their catalogs between 1906 and 1983 (Hunter, 2012). These kit homes were shipped out in easy-to-construct sections and were assembled on site. Kit homes were important because they showed countless Americans the many advantages of modular construction. With millions of Americans returning home from war, there was a rapid need for affordable housing. Prefabricated or modular homes became a primary method of supplying homes to veterans. More recently, modular construction has been seen in civilian markets such as residential, hospitality, education, healthcare, and many more. It is also seen in military operations for rapidly deployable structures. Modular construction can provide benefits in terms of cost, time, and quality (Lawson et al., 2014). However, there are some negatives, such as shipping and lack of flexibility, that can be deterrents. Although the benefits are quite desirable, modular construction still only makes up a small portion of the market share. Two possible reasons for this are that the construction world is slow to adopt changes and the perception that modular construction produces a lower quality product. As factory production technology improves, it is believed that modular construction will begin to gain traction in the U.S. construction market.
Understanding the benefits of modular construction make it a perfect choice for the deployable structures being designed in this research. Having identical modules that can be repeated easily in a factory setting should make the structures quicker to build, cheaper, and of better quality. When building a structure on site, factors such as labor quality as well as weather conditions can drastically change the performance of the structure. Weather can also prolong the time it takes for a structure to be ready to occupy. Constructing in a factor setting removes weather as a variable which can result in many of the benefits previously mentioned. Having indistinguishable and repeatable units should increase the efficiency and quality of the work.

2.3 Expandable Rigid Wall Structures

Expandable rigid wall structures are more involved when it comes to the design when compared to non-expandable structures. There are many ways to design an expandable structure, and most ideas are inspired by origami. Origami is the art of paper folding and shares a common interest in designing an expandable structure. With current engineering practice and knowledge, structures cannot be constructed solely of paper. However, many principles of origami can be used in the design of a foldable structure. Thin material like paper is easy to fold and crease, but structural building materials like wood and steel cannot be folded so easily. Because of this, careful consideration must be taken when deciding where to place creases or folds in a structure. There has been much research on the incorporation of origami in deployable structures. One of which is the Lever Shelter Module. The Lever Shelter Module is one of many design concepts created by researchers at the University of Notre Dame, designed for use in military applications (Quaglia et al., 2014). The designs incorporate counterweighting as a method of deploying heavier structures. The researchers used design criteria such as transportability, reconfigurability, minimizing the
number of moving parts, are ease of erection. Some advantages to the Lever Shelter Module are this it folds out from a small package and is highly modular and reconfigurable. Of the many concepts created, the Lever Shelter Module was a candidate for use on forward operating bases and has been selected by the U.S. Army Natick Research, Development and Engineering Center for further study. The materials used for the rigid walls are sandwich panels with a lightweight core made from foam insulation and two rigid faces made from fiberglass. This is similar to structural insulated panels, SIP, which uses insulation between two engineered wood panels. A major benefit of including insulation is that the structure should be more energy efficient. Although this is a large concern during typical civilian construction, it is an even greater concern in military construction in expeditionary regions. Keeping a power supply to a base requires a fuel source, and often, obtaining fuel in these regions can be dangerous and even deadly. Therefore, it is best to minimize fuel usage to keep a structure comfortable enough to live in. Figure 2.5 shows the stages of deployment of the Lever Shelter Module.

Figure 2. 5: Lever Shelter Module. From (Quaglia et al. 2014)
2.4 Additive Construction

The military is continuously pushing the envelope with advancements in shelter technology. Another form of construction being researched and implemented is called additive construction (AC). Additive manufacturing, or construction, is essentially 3D printing structures with concrete. Some advantages of AC are the potential for 40% cost savings as well as being able to utilize geometries that perform better structurally (Thrall & Dascarnio, 2021). Although this method does not deliver a turnkey product to the site, it constructs one onsite with incredible speed. A simple shelter can be printed in less than 24 hours. Also, continuous printing is possible and does not require human operation once the printing has begun. These structures are extremely rugged and should stand up to some of the toughest environments. Concrete also performs well against man-made threats such as ballistics and shrapnel. A drawback associated with AC is that the structures are permanent and not transportable. Therefore, once the unit is printed, it can only be used in that location. If the military group moves frequently and wants a re-deployable structure, additive construction would not be the best choice.

Figure 2. 6: 3D Printed Military Shelter constructed by U.S. Army Engineer Research and Development Center, Construction Engineer Research Laboratory (2017).
2.5 Cross-Laminated Timber

2.5.1 General Characteristics

Cross Laminated Timber (CLT) is a relatively new construction material in the U.S. However, CLT has been around since the 1990s in Austria and Germany (Karacabeyli & Douglas, 2013). In Europe, CLT has taken a foothold in the construction market and has been utilized in residential and non-residential applications. CLT is an engineered wood product; however, it does not directly compete with or compare well with light-frame wood construction. CLT’s main use is in mid-rise to high-rise buildings, whereas light-frame is mainly used in low-rise buildings. Typically, mid to high-rise buildings are constructed with steel or concrete. In 2015, CLT was first recognized in the U.S. building code and has become more accepted as a construction material in recent years. CLT is a wood panel made up of lumber boards stacked and glued together. The stacking configuration usually consists of an alternating pattern at 90 degrees to the adjacent layer. Layers typically come in odd numbers ranging from 3 to 9. Dimensions of these panels can vary vastly, but it is typical to see them from 2ft to 10ft wide and up to 60ft long. The thickness of the panels depends on the number of layers, but a standard 3-ply, ply meaning layers, panel is 4 1/8” thick and a 9-ply is 12 3/8” thick. By designing these panels with an odd number of layers, they generate a strong, or major axis. The strong axis is usually the exterior ply direction or the direction in which most layers run longitudinally. The major strength direction is normally orientated up and down for walls and in the major span direction for floors or roofs. Orientating the panels this way provides the maximum capacity of the system in the direction where it is needed most. Figure 2.7 shows a CLT panel with major and minor axis indicated. CLT has many general benefits, such
as strength-to-weight ratio, acoustic, fire, and seismic performance. It is also a quicker and more sustainable method of construction.

Sustainability is a major driving force behind the use of CLT in the U.S. and is highly responsible for the advancements of CLT in U.S. building codes. CLT is made almost entirely from wood which is a renewable resource. Modern timber harvesting is just like harvesting any other crop. When the crops (trees) are harvested, new crops are replanted, and thus the cycle continues. Also, since CLT is made from smaller pieces of lumber, the trees are not required to be large when they are harvested, giving a quicker turnaround time. Another way CLT is a more sustainable product is because there is less waste generated. The prefabricated nature of CLT allows for it to be precisely cut with a Computer Numerical Control (CNC) machine. This also reduces the construction time and required labor.

2.5.2 Ballistic and Blast Performance

Along with the general characteristics of CLT, there are also some attributes that are more focused on military applications. These include ballistic and blast performance. Ballistic and blast resistance of a structure is grouped within force protection which is a large contributor to the building envelope resistance category used in this research. Force protection parameters are mainly targeted toward military use applications; however, even in the civilian sector, extra protection is always a good thing. Force protection measures are important because of two main reasons, the
first being that in dangerous environments, the resistance of the structures may be used to protect
the lives of those occupying them. The second is that it gives peace of mind to those who use the
structures. This leads to a better work environment and, in turn, a more productive atmosphere.

Ballistic performance describes how well a material can withstand or resist penetration
from a projectile. Projectiles can be from either direct or indirect fires. Indirect fire could be
shrapnel or fragments from explosives such as rockets or mortars. Indirect fire weapons can be
devastating due to the large area affected by the debris. Direct fire could be bullets from small
arms such as handguns, rifles, and shotguns. There are two main utilized commercial standards for
rating ballistic performance in the U.S. They are Underwriters Laboratory (UL) Standard for
Bullet-Resisting Equipment, UL Standard 752, and National Institute of Justice (NIJ) Ballistic
Resistant Protective Materials, NIJ Standard 0108.01. Both of these standards follow the same
general concept, which is to provide a rating based on the ammunition type, bullet velocity, and
the number of shots a material can resist. Figure 2.8 is a table from NIJ 0108.01 showing the rating
levels.
The previously mentioned standards are used to rate the performance of materials. These standards do not predict how a specific material will perform. Instead, there are different standards that generate equations to predict the performance of a material based on certain criteria. Research conducted at the Georgia Institute of Technology by Kathryn Sanborn covers the models used to predict the ballistic performance of materials (Sanborn, 2018). Her research contained an experimental portion that conducted several ballistic tests on CLT specimens. The testing performed in her research was conducted with a 0.5-inch diameter steel ball projectile. Although
this projectile shape is not a common modern bullet shape, the research gives a good baseline and standard way of viewing the performance of CLT as a ballistic resisting material. The research recommends using a model referred to as the “THOR CLT model” to determine the required thickness of a CLT panel to stop a 0.5-inch diameter steel ball at a given striking velocity. The THOR CLT model is an alteration of the already existing THOR model described in here research. The THOR CLT model equation developed by Sanborn can be seen in equation 2.5.2-1.

\[ T_w = C_1 \times \left( \frac{v_s^a}{10^c \times \rho^d \times H^f} \right) \]  

(EQ 2.5.2-1)

Where:

- \( T_w \) = Thickness required to stop performance
- \( v_s \) = Striking velocity
- \( \rho \) = Density of wood
- \( H \) = Hardness of wood
- \( C_1, a, c, d, f \) = Constants

Along with ballistic performance, the blast performance of a structure can save lives. A major difference between ballistic and blast threats is the way in which they can be used. Ballistic threats have an extremely long effective range, while blasts are typically limited to shorter ranges. As shown previously, ballistic threat levels can stem from factors such as projectile velocity, weight, size, shape, and quantity of hits. Blast factors usually included the amount of explosive, usually expressed in weight, and how far away the target is at detonation. A common solution to blast protection is to increase the distance, known as standoff distance, in which the explosion can take place. This is often done by putting up fences at a specified offset from the base and having security watch the perimeter. Although this method is effective, securing a perimeter of a large
base could be labor and time intensive. Therefore, for a deployable structure used in military expeditionary structures, blast resistance is an important factor. Deployable structures are usually used as a temporary solution while the base is getting set up, and therefore the security measures are not yet available.

Blast performance is an important characteristic of any structure used by the military, especially when used in hostile environments. Blast performance describes how resistant a structure is to the pressures created through blasts. Standoff distance is typically the main way to protect a structure against blasts. However, sometimes this is not an option or is not effective. Potential bases in dense urban environments naturally have a smaller standoff distance as space is limited. Standoff distance is less effective when explosives are delivered by air or weapon. In these situations, having blast resistance structures could be the difference between life and death. Blast performance pairs well with ballistic performance as most explosives create shrapnel, which is fragments propelled by the explosives. The shrapnel can be resisted by a structure's ballistic protection, whereas the pressures can be resisted by the blast protection.

Blast testing on CLT structures was performed by Woodworks in 2016 and 2017. The weight of the explosives ranged from 32lbs to 199lbs. The standoff distance in the tests was 75ft. Pressures generated from blasts can be magnitudes higher than those from environmental loads. In test number three, the weight of the explosive was 199lbs, and the peak reflected pressure was 1900 psf. This is more than 10x the maximum pressure seen in the wind calculations. These tests reveal that the CLT structures perform relatively well and as expected in their elastic region. They even outperformed the expected results when testing was pushed outside the elastic limits (Weaver, 2017). For the purposes of this research, CLT blast performance will be a qualitative
measure rather than quantitative to display the predicted performance of each structure more easily.

2.6 Hinges

Many deployable structure designs feature some form of foldability because it is an efficient way to maximize usable space while reducing the footprint of the structure during shipment. Of the deployable structures, hinges are a common element used as the folding mechanism. This mechanism allows panels to rotate about an axis. This rotation can be used to design a structure that can be folded and unfolded. Although hinges are commonly found throughout structure designs, the capacity of hinges is unknown. Relevant strength performance of common hinges is not readily available. Sierra Pacific Engineering and Products provides a “Guide for selecting continuous hinges” (SPEP, 2012). This guide provides graphs that can be used to determine what size hinge is needed according to the expected load. Two graphs from this document can be seen below in Figures 2.9 and 2.10. The testing that was the basis of this guide was reportedly performed by the Illinois Institute of Technology (IIT) for the Builders Hardware Manufacturers Association (BHMA).

Testing was performed because it was unknown whether the failure mode would occur between the connection to the CLT panels or would occur within the hinge. Also, the effect of the pin and knuckle assembly on the connection was not understood. Finally, since this was the only piece of literature found about the strength of hinges, it was unsure if this design guide would accurately represent the capacity of the hinges used throughout this research.
Figure 2. 9: Vertical Load Hinge Graph from (SPEP, 2012)
Figure 2. 10: Horizontal Load Hinge Graph from (SPEP, 2012)
Chapter 3: Testing on Hinges

3.1 Overview of Tests

The objective of this research is to design prefabricated, turnkey structures that can be used for disaster relief and military applications. In the design chapter of this research, the structures are covered in more detail. For context, two of the structures being designed are expandable and feature hinges as the folding mechanism. Hinges are commonly used in deployable structure designs; however, the structural properties of hinges are much less known. Testing was conducted with the intent to determine how hinges behave as structural elements and to determine strengths that could be used throughout the designs for this research. Also, this testing could be used as a basis for future testing on hinges. Hopefully, more testing will be performed in the future that could be used to create a design guide that includes how to calculate hinges capacities as well as how they can be used efficiently throughout an expandable structure. Since there is a lack of testing and not a standardized testing method, these tests are not meant to provide the capacity of the hinges but rather a starting point that can be used in the structural designs.

There were a series of tests performed on hinges to determine approximate structural behavior. There were two hinge types that were used for the testing in this research, continuous style hinges, also known as piano hinges, and residential butt hinges. Table 3.1 shows the hinge types and properties. For simplicity of naming and referencing, each hinge type may either be referred to as hinge A or hinge B throughout this research. Figure 3.1 shows some key dimensions of each hinge type.
Table 3.1: Hinge Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Hinge Type</th>
<th>A</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leaf Thickness</td>
<td>Continuous</td>
<td>0.12&quot;</td>
<td>0.0855&quot;</td>
</tr>
<tr>
<td>Pin Diameter</td>
<td>Butt</td>
<td>3/8&quot;</td>
<td>15/64&quot;</td>
</tr>
<tr>
<td>Length</td>
<td></td>
<td>12&quot;</td>
<td>4&quot;</td>
</tr>
<tr>
<td>Width</td>
<td></td>
<td>5&quot;</td>
<td>4&quot;</td>
</tr>
<tr>
<td>Screw Diameter</td>
<td></td>
<td>0.25&quot;</td>
<td>0.137&quot;</td>
</tr>
<tr>
<td>Screw Length</td>
<td></td>
<td>3.5&quot;</td>
<td>1&quot;</td>
</tr>
</tbody>
</table>

Figure 3.1: Key Dimensions of the continuous hinge (left) and the butt hinge (right)

Each hinge type was tested in two directions, loading parallel and perpendicular to the pin. Parallel to the pin tests will be referred to as shear tests, and perpendicular to the pin test will be called tension tests. Figure 3.2 shows a conceptual drawing of the directions each hinge was tested in. Multi-directional strength properties are important because the deployment stage will induce forces on the hinges. By completing these tests, the hinges could be incorporated into the structure’s strength rather than ignored when in the deployed state. Along with the load tests performed on the hinges, there was also a material test performed on the leaf of each hinge. The material test provided key materialistic properties such as ultimate stress, yield stress, and modulus of elasticity of the steel used in the hinges. All these properties are important metrics when describing how a structural element performs. A summary of the load tests performed on the hinges can be found in Table 3.2.
Table 3.2: Summary of Tests Performed on Hinges

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Loading</th>
<th>Hinge Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>Shear</td>
<td>A</td>
</tr>
<tr>
<td>1B</td>
<td>Shear</td>
<td>B</td>
</tr>
<tr>
<td>2A</td>
<td>Tension</td>
<td>A</td>
</tr>
<tr>
<td>2B</td>
<td>Tension</td>
<td>B</td>
</tr>
</tbody>
</table>

3.2 Material Tests

Before being able to understand how the hinges perform as structural elements, the material properties are required. Since the butt hinge used in testing was designed for residential doors, important material properties were not provided to the consumer. A common way to determine the yield strength, ultimate strength, and modulus of elasticity of a material is to perform a tensile test.
Tensile testing was performed on a UTM machine that had grips that would attach to the specimen ends. Once the grips are tightly attached to the material specimen, an acceptable load rate must be set. The goal is to have a load rate that is not too slow or too fast. Once an acceptable load rate is set, the test is ready to be run. For both material tests, a small section of the hinge leaf was cut out and thinned out in the center. The resulting shape of the test specimen looked like a dog bone. By reducing the area in the center, the specimen is forced to fail at that location. This allows for an easier time calculating the failure plane area so that the stresses can be determined. A conceptual representation of a material test specimen can be seen in Figure 3.3. The equations for determining the yield and ultimate stress of the steels in each hinge are given by:

\[
\sigma_y = \frac{F_y}{A_f} \quad \text{(EQ 3.2-1)}
\]

\[
\sigma_u = \frac{F_u}{A_f} \quad \text{(EQ 3.2-2)}
\]

Where:

\( \sigma_y \) = Yield Stress

\( \sigma_u \) = Ultimate Stress

\( F_y \) = Yield Force

\( F_u \) = Ultimate Force

\( A_f \) = Area of Failure Plane
Strain gauges were used to measure the displacement over the failure section and were plotted with stress. The strain gauges had a limit of 3% elongation, which could not capture the complete material test. For that reason, the stress vs. strain graphs will be used to show the yield stress and the modulus of elasticity, E. The stress vs. time graphs will be used to show the ultimate stress observed in each hinge material. Stress vs. strain graphs for the continuous hinge material as well as the butt hinge material can be found in Figures 3.4 and 3.5, respectively. For clarity, the stress vs. strain graphs was cut off at the location the strain gauges reached their elongation limits. The stress vs. times graphs can be seen in Figures 3.6 and 3.7 for the continuous and butt hinge material tests, respectively.

![Figure 3.4: Continuous Hinge Material Test (Stress VS Strain)](image)
Figure 3. 5: Butt Hinge Material Test (Stress VS Strain)

Figure 3. 6: Continuous Hinge Material Test (Stress VS Time)
The modulus of elasticity was calculated to quantify how stiff the hinge material is. The equation for calculating the modulus of elasticity is given by:

\[ E = \frac{\sigma_E}{\varepsilon_E} \]  
(EQ 3.2-3)

Where:

\( E = \text{Modulus of Elasticity} \)

\( \sigma_E = \text{Closest Test Stress Value to Yield Stress} \)

\( \varepsilon_E = \text{Corresponding Strain to } \sigma_E \)

Table 3.3 shows the key material properties gathered from completing the material tests. The test results show that both hinge materials have similar ultimate stress and modulus of elasticity but different yield stress.
Table 3.3: Material Test Results

<table>
<thead>
<tr>
<th>Hinge</th>
<th>Yield Stress (ksi)</th>
<th>Ultimate Stress (ksi)</th>
<th>Modulus of Elasticity (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>42</td>
<td>83</td>
<td>32,161</td>
</tr>
<tr>
<td>B</td>
<td>80</td>
<td>87</td>
<td>36,989</td>
</tr>
</tbody>
</table>

3.3 Shear (Parallel to Pin) Test

Shear tests were performed on the continuous and butt hinge and are referred to as Test 1A and 1B, respectively. The shear tests were done to gather data on how these hinges would perform when loaded in the direction parallel to the pin. One way this type of loading could be observed in a structure is when shear is transferred from wall to floor panels. Both tests, 1A and 1B, were performed with cyclic loading. Cyclic loading is when a load is applied in one direction, then removed and applied in the opposite direction. The loading setup in both shear tests was set to follow CUREE standard loading protocol. Cyclic loading is often chosen over monotonic loading, single-direction loading without load removal, because it may represent loading caused by wind gusts or earthquake accelerations better. Figure 3.8 shows a conceptual drawing of the test setup for Test 1A. Figure 3.9 shows the testing setup as well as instrumentation for Tests 1A and 1B. 3-ply southern yellow pine CLT panels were used as the main member throughout these tests.
Figure 3. 8: Conceptual Test Setup for Test 1A
Each test measured the displacement of each string potentiometer as well as the force and displacement of the actuator. The string potentiometers were used to measure localized displacement, and the actuator was used to measure forces and displacements of the entire system. Figures 3.10 and 3.11 show the force vs. actuator displacement of Test 1A and 1B, respectively. These figures also show the expected values for design strength as well as measured stiffness. For the continuous hinge, Simpson Strong-Tie SDS screws were used to fasten the hinges to the CLT panels. The manufacturer provided the design ASD values, which are displayed in blue. The ASD value was then multiplied by the format conversion factor to show a non-factored LRFD value. A strength reduction factor was not used because the goal was to try to estimate the test peak value rather than a design value. The equation used to convert the ASD value to a non-factored LRFD equivalent is given by:

\[ \text{NFLRFD} = \text{ASD} \times K_{FZ} \]  

(EQ 3.3-1)
Where:

\( NFLRFD = \) non-factored \( LRFD \) Equivalent Value

\( ASD = \) Manufacturer Suggested ASD Value

\( K_{FZ} = \) Format Conversion Factor For Dowel – Type Fasteners

For the butt hinge, the expected values were calculated by using chapters 11 and 12 of the 2018 NDS. The shown Z value is the calculated ASD design strength. The calculation for NDS connection strengths can be found in the appendix. The test results were made to show the force vs. actuator displacement per hinge. For example, since Test 1A used two hinges, the force was divided by two, and Test 1B used four hinges, so the force was divided by four. The ASD and \( K_F \times ASD \) values shown on each graph are for four screws which was the number of screws in each hinge for both hinge A and B.

![Figure 3.10: Force VS Actuator Displacement for Continuous Hinge per Hinge](image-url)
From the above figures, the ultimate strength, yield strength, and initial stiffness of the connection can be determined. These key values can be found in Table 3.4. Since this was cyclic loading, there was a push and pull cycle. The push cycle occurred first, followed by the pull cycle. The ultimate strength was determined by identifying the peak force for the push and pull cycles. The yield strength was determined by offsetting the initial stiffness by 5% of the screw diameter (ASTM, 2018). The initial stiffness was determined by taking the slope of the initial part of the curve.

Table 3. 4: Test Results from Shear Tests

<table>
<thead>
<tr>
<th></th>
<th>Hinge A</th>
<th>Hinge B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Push</td>
<td>Pull</td>
</tr>
<tr>
<td>Ultimate Strength (kip)</td>
<td>9.06</td>
<td>6.72</td>
</tr>
<tr>
<td>Yield Strength (kip)</td>
<td>5.08</td>
<td>3.12</td>
</tr>
<tr>
<td>Initial Stiffness (kip/in)</td>
<td>35.62</td>
<td>35.68</td>
</tr>
</tbody>
</table>
The values provided by Simpson Strong-Tie underestimated the strength of the connection. This is most likely because the values provided by Simpson Strong-Tie were for wood main members with a specific gravity of at least 0.5, whereas the wood used during testing was Southern Yellow Pine and has a reference specific gravity of 0.55. Also, the steel tensile strength for the main member needed to be at least 45ksi, whereas the steel used in that test was around 83 ksi. Another possible and likely reason for underestimation is variability and safety factors. When reviewing the butt hinge test results, the expected strength of the connection, through NDS calculations, was close to the tested strength. Since the expected values were relatively close to the tested values, it is believed that the NDS equations can be used to estimate the strength of the connection. However, the calculated NDS for the butt hinge was much closer to the test value than for the continuous hinge. This means the factory of safety is less. There is one factor believed to be the cause of the reduced factor of safety. The factor is that the NDS equations are for calculating the capacity of the dowel connection for direct shear only. In Test 1A and 1B, the hinges experienced both direct shear and moment. As force is applied to the middle CLT panel, the force is transferred to the hinge leaf by the screws on that panel. The force must then go through the hinge and across the pin to the other hinge leaf. Finally, the force is transferred from the hinge leaf to the screws in the other CLT panel down to the support. Due to the hinges being in a 90-degree fold, the moment caused by the forces causes the hinge leaves to get pulled away from the CLT panel. This force is known as a pullout force experienced by the screws. The screws used in Test 1A were structural screws that have been engineered and designed to perform well in structural applications. This explains why the Simpson Strong-Tie screws were able to better resist the pullout forces from the test. The screws used in Test 1B were intended for use with a residential door and are not expected to need to resist the heavy structural loading seen in this test. For this
reason, the expected screw pullout strength is low. The moment created by the applied force is a function of force and distance. As the hinge length expands, the expected pullout moment is reduced. The continuous hinges were long and used structural screws, and the butt hinges were short and used non-structural screws. This combination is believed to be the reason for the reduction in factor of safety seen between both tests. The table below shows the average test peak strength to the $K_f \times ASD$ values for both hinges. It is assumed that the tests conducted in the research represent the mean strength of like hinges. LRFD capacities are based on nominal strengths. For this research, the nominal value is assumed as the point where 95% of samples pass and 5% fail, also referred to as the lower 5th percentile. Since only one shear test was performed on each hinge and there is no other hinge test information to use, the distribution of the capacity is unknown. For this research, an assumption of a normal distribution will be assumed. The coefficient of variation will also need to be assumed based on other wood connection tests. The coefficient of variation was assumed to be 0.15 (Stoner, 2020). This results in a nominal-to-mean ratio of 1.33. This shows that the NDS $K_f \times ASD$ value is not consistent with a lower 5th percentile as the ratio is only 1.0 and not 1.33. This also shows that the manufactured $ASD \times K_f$ for the continuous hinge is providing a more conservative value than the lower 5th percentile.

<table>
<thead>
<tr>
<th>Hinge</th>
<th>Test Peak Strength/$K_f \times ASD$ Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuous</td>
<td>1.4</td>
</tr>
<tr>
<td>Butt</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Failure was observed in both Test 1A and 1B and was documented with pictures. Both tests failed through a combination of failure modes because the system experienced forces other than direct shear only. As mentioned earlier, Test 1A was conducted with continuous hinges that were fastened by Simpson Strong-Tie SDS screws. These screws were designed for structural
applications and therefore have greater resistance to pullout forces. Since the SDS screws were strong against pullout, the next expected failure would be through shear. As expected, that is the main source of failure in Test 1A. Figure 3.12 shows the continuous hinge separated from the CLT panel due to shear failure in the screws. On the other hand, Test 1B used provided screws that are not expected to be used in structural applications. Therefore, the pullout strength was relatively low, and in turn, the main mode of failure observed in this test had a combination of screw pullout and shear, as shown in Figure 3.13.

Figure 3.12: Failure in Test 1A
3.4 Tension (Perpendicular to Pin) Test

Tension tests were performed on the continuous and butt hinge and are referred to as Test 2A and 2B, respectively. The tension tests were performed to gather data on how these hinges would perform when loaded in the direction perpendicular to the pin. Loading conditions that could cause these forces on the structure could be through deployment, raising panels into place, and forces that act normal to the panels, such as suction caused by wind. Both tests, 2A and 2B, were performed with monotonic loading, which is when a load is applied in one direction at a constant rate until failure. Monotonic loading was chosen for this direction because compression toward the pin would not provide useful or practical data.
As discussed previously, the shear tests performed on each hinge were set up and executed the same. Therefore, a direct comparison can be made between these tests. The tension tests did not use the same setup between hinges; therefore, the goal of this section is not to compare the results directly. Instead, the butt hinge was tested in an isolated state, without connections to CLT panels. The continuous hinge was tested as a complete connection as it may be constructed within the deployable dynamic unit. In summary, the continuous hinge was fastened to CLT panels, and test data was gathered to describe the connection system, whereas the butt hinge was tested by itself without any connection to CLT. Figure 3.14 shows the tension test setup and instrumentation.

String potentiometers (SP) were used to measure displacements during each test, and the load cell within the UTM machine was used to measure the applied force. In Test 2B, only one
string potentiometer was used because the only behavior the test was set to observe was the stiffness and strength of the knuckle and pin assembly. Test 2A utilized five string potentiometers because there were three behavior locations this test was used to capture. String potentiometers 1 and 5 were used to measure the displacement of the entire connection system, which captured the movement of CLT panel to CLT panel. String potentiometers 2 and 3 were used to measure the displacement between the CLT panel and the attached hinge leaf. Finally, string potentiometer 4 was used to measure the displacement occurring between each hinge leaf. Table 3.6 describes the measured displacement of each string potentiometer.

Table 3. 6: String Potentiometer Description for Test 2A

<table>
<thead>
<tr>
<th>SP #</th>
<th>Measured Displacement</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP1</td>
<td>CLT Panel to Panel</td>
</tr>
<tr>
<td>SP2</td>
<td>CLT Panel to Hinge Leaf</td>
</tr>
<tr>
<td>SP3</td>
<td>CLT Panel to Hinge Leaf</td>
</tr>
<tr>
<td>SP4</td>
<td>Hinge Leaf to Hinge Leaf</td>
</tr>
<tr>
<td>SP5</td>
<td>CLT Panel to Panel</td>
</tr>
</tbody>
</table>

Figure 3.15 shows the raw data obtained from Test 2A. For the purpose of describing the systematic behavior of the entire hinge connection assembly, string potentiometers 1 and 5 were averaged and replotted, as shown in Figure 3.16. From Figure 3.16, the ultimate strength, yield strength, and initial stiffness can be approximated. These values are shown in Table 3.7.
Figure 3. 15: Test 2A Results

Figure 3. 16: Force VS Averaged Displacement of SP1 and SP5 for Test 2A

Table 3. 7: Characteristic Properties Concluded from Test 2A

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate Strength (kip)</td>
<td>8.4</td>
</tr>
<tr>
<td>Yield Strength (kip)</td>
<td>3.6</td>
</tr>
<tr>
<td>Initial Stiffness (kip/in)</td>
<td>42.2</td>
</tr>
</tbody>
</table>
Figure 3.17 shows the test results of Test 2B. From this figure, the ultimate strength, yield strength, and initial stiffness can be approximated. These values are shown in Table 3.8. The yield strength was taken as the point where the initial slope changed. The previous method could not be used as there were no screws used.

![Figure 3. 17: Test 2B Results in Per 4” Hinge](image)

Table 3.8: Characteristic Properties Concluded from Test 2B

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate Strength (kip)</td>
<td>2.3</td>
</tr>
<tr>
<td>Yield Strength (kip)</td>
<td>2.1</td>
</tr>
<tr>
<td>Initial Stiffness (kip/in)</td>
<td>709.4</td>
</tr>
</tbody>
</table>

Since Test 2B did not contain a steel to wood connection and was solely focused on the butt hinge, made completely from steel, the initial stiffness was expected to be much greater than that observed in Test 2A. This shows that the connection to a wood member significantly reduced the stiffness of the system.
Failure in each test was observed and captured through pictures. In Test 2A, the CLT panel developed a small crack around the screw penetration near 7.1 kips. This crack continued to grow throughout the test until it reached a point where the test was ended. The initial crack can be seen in Figure 3.18, and the end of the test can be seen in Figure 3.19. This test was intended to show how this hinge would perform as used within the structure. It is important to mention that because of the geometry of the hinge, the required end distance associated with the screws was not met. In Test 2B, the failure occurred between the knuckle and pin assembly. As seen in Figures 3.20 and 3.21, the knuckles of the hinge gradually unwound from the pin.
Figure 3. 19: Test 2A Close to Test End

Figure 3. 20: Unwinding of Knuckles from the Pin
3.5 Conclusion of Testing

Experimental testing was performed on two hinge types and each hinge type’s leaf material. Hinge type A was a continuous hinge that was slightly larger than hinge type B in every dimension. Hinge type B was a standard off-the-shelf butt hinge that is typically seen in residential structures for interior doors. Material tests were performed on each hinge material to get the basic material properties such as ultimate stress, yield stress, and modulus of elasticity. Hinge type A was used in conjunction with Simpson Strong-Tie SDS screws, while hinge type B used screws that were provided in the packaging. The two directions that were tested are parallel and perpendicular to the pin. From Test 1B, it became evident that the NDS equations do not accurately represent the hinge connections. The NDS equations do not consider any pullout induced from the moments. The NDS equations also do not consider the countersunk holes in the leaf of the butt
hinge. However, the NDS equations can be used to get an estimate of the strength of the hinges. Test 2A shows that hinges would not be a good choice as a hold-down connection because of the small width of the hinge leaf and the linear screw pattern.

Results from the tests show there is strength much greater than the forces applied from deployment. Therefore, these hinges could be used as structural elements when paired with proper fasteners. As with any other structural element, the hinges must be properly designed, and the entire connection system must be considered. As with any material, there is variability. For that reason, the author of this research believes further testing is required before complete and definitive statements regarding these hinge strengths can be made. The scope of this testing was to obtain an initial understanding of the behavior of hinges as structural elements, not to determine their design strength values. This testing also provides a basis that can be used to conduct future testing of hinges.
Chapter 4: Design of Deployable Structures

4.1 Design Constraints/Considerations

The need for rapidly deployable structures can be seen in many different environments and cases. This played a large role in the design of the structures and was one of the first areas of concentration. On the civilian side of this research, the expected mode of shipment is by land through the use of semi-trucks. In order to make shipment of these units as easy and seamless as possible, ISO standard shipping sizes played a large role in the design footprint (ISO, 2020). A twenty-foot-long unit became the basis of design due to standardization and practicality. The most typical 20 ft long shipping container is 8 ft wide x 8.5 ft tall. Another standard-size shipping container is known as the “high cube” container. These containers are 20 ft long x 8 ft wide x 9.5 ft tall. By moving up to the high cube size, the structures become more livable and comfortable with more headroom. The dimensional selection and layouts associated with the static units presented in this research were developed with Clemson University’s School of Architecture and under the direct mentorship of Professor Dustin Albright. The original design in this research contained only the CLT static unit, but because of dimensional constraints due to shipping considerations, the dynamic unit was conceptualized. Table 4.1 shows the key dimensions of the static and dynamic structure types. The static unit does not fold; therefore, the folded dimensions are not applicable for this unit type.

Table 4. 1: Dimensions of the Static and Dynamic Structures

<table>
<thead>
<tr>
<th>Unit Type</th>
<th>Deployed</th>
<th>Folded</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Length</td>
<td>Width</td>
</tr>
<tr>
<td>Dynamic</td>
<td>20’</td>
<td>19' 5”</td>
</tr>
<tr>
<td>Static</td>
<td>20’</td>
<td>8’</td>
</tr>
</tbody>
</table>
With only around 160 sq. ft. of living space in the static unit, the idea for a foldable design came about. By designing a foldable unit, the usable space increased to around 390 sq. ft., over 2x what was seen in the static unit. Another major consideration made in the design of these structures was the weight. Two of the four key factors used in the design are greatly influenced by the weight of the structures, namely, deployability and transportability. Since these structures arrive mostly assembled, the weight is much larger compared to a system that is delivered in components and constructed on-site. Therefore, it is important to reduce weight where possible. The consideration of weight is what led to designing both the static and dynamic structures with an additional material type. The chosen additional material was light-frame wood construction. This construction material is relatively lightweight, cost-effective, and customizable.

Another benefit of reduced weight is the ease of deployability. In the dynamic units, the panels need to be rotated into place before the structure is used. By having lighter panels, the deployment time and equipment needed are reduced, making it more desirable and easier to use the system. Another major design consideration was the cost of building these structures. During the time this research was conducted, the cost of construction materials, including wood, increased exponentially, driving up the expected costs of building these structures. However, it is expected that with time these prices will come back down to around pre-pandemic prices. In general, it is expected that the light-frame units will cost less to produce than the CLT units because of the volume of raw materials used.

4.2 Folding/Deployment Mechanism

When designing the dynamic unit, one of the first parts that needed to be figured out was how these units would expand and contract. There are many different strategies and designs that have been used in the past to achieve this task. For this research, continuous hinges were used as
the folding mechanism. By choosing hinges, the method of deployment would be through foldability of panels about the axis the hinge lies. This deployment method had many impacts on the design of the structure. One of which is the dimensions of each panel. Getting the structure within the ISO shipping standard sizes while also maximizing useable space took careful planning.

**4.3 Design for Gravity Loads**

**4.3.1 Overview**

Gravity load design is the process of engineering a structure and the individual members to withstand forces brought on by gravity. There are three basic forces any member within a structure may experience, bending, shear, and axial. Bending and shear forces are most typically seen in elements that span horizontally, such as a beam. Axial loads are typically concentrated near vertical members such as a column. A good way to understand gravity loads is to pick a location on a structure and draw an arrow to the earth’s surface because that is the final location in which the force must end up. Starting at a roof joist, seen in light-frame construction, the joist must carry either roof or floor loads to a member that can bring these forces down to the foundation. The most common way for this to be done is for the joist to resist and transfer these forces by bending and shearing. Once this load reaches the supports of the joists, the supports then carry the load down to the foundation through axial forces. The process of load movement through a structure is known as load path. A simplified diagram of load path can be seen in Figure 4.1.
Light-Frame construction is a construction method using sawn or dimension lumber. These lumber members are small in terms of the complete structure. The members are spaced at a designed spacing so that load can be shared between them. This construction method is cost-effective and materially resourceful. By using smaller pieces of lumber, it is more environmentally friendly, as mentioned in the CLT literature review section.

Cross Laminated Timber design was performed using the 2018 National Design Specification (NDS) for Wood Construction, The U.S. Edition of the CLT Handbook, and SMARTLAM Cross-Laminated Timber (CLT) Specification Guide. In light-frame design, the load and capacity were determined for a single member, whereas, in CLT design, the design was performed on a per foot of board length basis, where applicable.

4.3.2 Demand
There are two things that must be solved for every structural engineering problem those being demand and capacity. The goal of every design is to have a capacity greater than the demand. The design portion of this research was completed using LRFD, Load and Resistance Factor Design. Therefore, ultimate loads refer to loads in their factored form. There are seven load combinations provided in ASCE 7-16 that were used to calculate ultimate loads (ASCE, 2017).

\[ 1.4 \times DL \]
\[ 1.2 \times DL + 1.6 \times LL + 0.5 \times (L_r or S or R) \]  
\[ 1.2 \times DL + 1.6 \times (L_r or S or R) + (LL or 0.5 \times W) \]  
\[ 1.2 \times DL + W + LL + 0.5 \times (L_r or S or R) \]  
\[ 0.9 \times DL + W \]  
\[ 1.2 \times DL + E + LL + 0.2 \times S \]  
\[ 0.9 \times DL + E \]  

(EQ 4.3.2-1)  
(EQ 4.3.2-2)  
(EQ 4.3.2-3)  
(EQ 4.3.2-4)  
(EQ 4.3.2-5)  
(EQ 4.3.2-6)  
(EQ 4.3.2-7)

Where:

DL = Dead Load

LL = Live Load

L_r = Roof Live Load

R = Rain Load

S = Snow Load

W = Wind Load

E = Earthquake Load

When a structural design is being performed, there are two limit states that are considered. Strength limit states are conditions that control the safety of the structure and are required to pass. Strength limit states are designed for the worst expected conditions and are the reason the load combination seen in EQ 4.3.2-1 through 4.3.2-7 are used. Serviceability limit states are conditions
that define the functional performance of the structure and do not necessarily have to pass; however, they should be met. Serviceability considerations are made on the assumption of an average day, not the worst. Therefore, load factors are not used when designing for serviceability (West, 2003). The most common serviceability consideration is deflection which was used in this research.

Determining the loads on a member, whether the member be a 2x4 sawn lumber board or an 8 ft wide CLT panel, is performed the same. The only change is the tributary area that affects each member. For this reason, loading such as bending, shear, and axial will not be shown for both materials. The complete designs for each unit can be found in the appendices. The equations and concepts discussed in the demand section can be used for both light-frame and CLT units.

**Bending**

Throughout the design of the light-frame structures, all spans were taken as a single span simply supported. This is typically done because it is easier to analyze by hand and delivers acceptable results. The equation for the ultimate bending moment of a single span simply supported member is given by:

\[ M_u = \frac{w_u \times L^2}{8} \]  

(EQ 4.3.2-8)

Where:

- \( M_u \) = *Ultimate Bending Moment*
- \( w_u \) = *Ultimate Distributed Load*
- \( L \) = *Span Length*

**Shear**

The equation for ultimate shear load in a single span, simply supported beam is given by:

\[ V_u = \frac{w_u \times L}{2} \]  

(EQ 4.3.2-9)
Where:

\( V_u = \text{Ultimate Shear Load} \)

**Axial**

For light-frame construction, walls are made up of 2x sawn lumber studs spaced at a design spacing. These studs were treated as individual columns that were responsible for supporting their own tributary area. For CLT construction, walls were designed on a linear foot basis. Therefore, the tributary area is for a foot-long section of wall length. The equation for calculating the ultimate axial force in a stud is given by:

\[
P_u = LC \times A_{trib}
\]  
(EQ 4.3.2-10)

Where:

\( P_u = \text{Ultimate Axial Load} \)

\( LC = \text{Controlling Load Case} \)

\( A_{trib} = \text{Tributary Area} \)

**Deflection**

Deflection demand is based on allowable deflection and differs depending on where the member is located. These deflection limits should be larger than the expected deflection of respective members. Deflection limits were referenced from Chapter 16 of the 2021 International Building Code and can be seen below (ICC, 2021):

\[
\Delta_{lim,R} = \frac{L}{120} \quad (\text{EQ 4.3.2-11})
\]

\[
\Delta_{lim,F} = \frac{L}{240} \quad (\text{EQ 4.3.2-12})
\]

Where:

\( \Delta_{lim,R} = \text{Deflection Limit for Roof Members Not Supporting Ceiling} \)

\( \Delta_{lim,F} = \text{Deflection Limit for Floor Members} \)

**4.3.3 Capacity**
Unlike demand, capacity is calculated differently between light-frame and CLT.

4.3.3.1 Bending: Light-Frame

As mentioned earlier, an engineer’s job is to design the structure so that the capacity is always greater than the load. Loads are based on judgment and usage categories; capacity is a function of material properties and member size. For light-frame construction, the method used throughout this research was to first calculate the adjusted material properties, then multiply by the appropriate member geometry to achieve a member capacity. All reference material properties for light-frame calculations were based on No. 2 Southern Yellow Pine. The reference values were obtained through the 2018 NDS. The equation used to calculate the adjusted material strength in bending is given by:

\[
F_b' = F_b \times C_m \times C_t \times C_{fu} \times C_i \times C_r \times K_{fb} \times \phi_b \times \lambda \quad \text{(EQ 4.3.3-1)}
\]

Where:

\( F_b' \) = Adjusted Design Value for Bending

\( F_b \) = Reference ASD Value for Bending

\( C_m \) = Wet Service Factor

\( C_t \) = Temperature Factor

\( C_L \) = Beam Stability Factor

\( C_{fu} \) = Flat Use Factor

\( C_i \) = Incising Factor

\( C_r \) = Repetitive Member Factor

\( K_{fb} \) = Format Conversion Factor for Bending

\( \phi_b \) = Resistance Factor for Bending

\( \lambda \) = Time Effect Factor
Once the adjusted material strength has been determined, the member capacity can be calculated by multiplying the material strength by the appropriate member geometry. The equation for calculating the design bending capacity of a member is given by:

\[ M_n' = f_b' \times S_x \]

(EQ 4.3.3-2)

Where:
\( M_n' = \text{Design Member Capacity for Bending} \)
\( S_x = \text{Section Modulus of the Member (X – Axis)} \)

### 4.3.3.2 Bending: CLT

Determining the capacity of a CLT panel is much different than that of a sawn lumber board. Sawn lumber is divided into grading, either visually graded or mechanically graded. Cross-laminated Timber also has grades, but each CLT manufacturer has its own grading scheme as well as reference design values. For this reason, the nearest CLT manufacturer to Clemson University was taken as the manufacturer whose grades and values are used in this research. At the time of writing, SmartLam has a CLT manufacturing plant in Dothan, Alabama. Locally, southern yellow pine is the most common lumber species for construction purposes, and so SmartLam’s SL-V3 CLT grade was chosen for this research. All reference design values for this research were pulled from SMARTLAM Cross-Laminated Timber (CLT) Specification Guide (SmartLam, 2020). These design values still need to be adjusted and converted into LRFD values so that they can be used throughout the design.

In most scenarios, roof panels will be supported on all sides. This raises some challenges as bending is typically considered one-way bending. For the purpose of analysis, panels will be treated as a unit width, 1 ft, beam that is simply supported in a single span. Because of this simplification, an assumption is being made that the panel will only bend, shear, and deflect as much as the major direction will allow (Raymond, 2019). Figure 4.2 shows one-way vs. two-way
bending of a plane (Civil Lead, n.d.). Since CLT can be manufactured in many different layups, 3-alt was used as the starting point in the design. 3-alt is the thinnest layup in SmartLam’s specification guide and refers to three layers of sawn lumber layered in an alternating pattern. The reference ASD values of SL-V3 3-alt can be seen in Table 4.2.

![Figure 4.2: (a) One-way bending deformation. (b) Two-way bending deformation. (Civil Lead, n.d.)](image)

<table>
<thead>
<tr>
<th>Grade</th>
<th>Layup</th>
<th>Thickness (in)</th>
<th>Weight (psf)</th>
<th>Major Strength Direction</th>
<th>Minor Strength Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$F_{bS_{eff,0}}$ (lbf-ft/ft)</td>
<td>$EI_{eff,0}$ (10^6 lbf-in^2/ft)</td>
</tr>
<tr>
<td>SL-V3</td>
<td>3-alt</td>
<td>4.125</td>
<td>13.5</td>
<td>1740</td>
<td>95</td>
</tr>
</tbody>
</table>

The equation to get the adjusted member strength for bending is given by:

$$F_{bS_{eff}'} = F_{bS_{eff}} \times C_M \times C_t \times C_L \times K_{Fb} \times \phi_b \times \lambda$$  \hspace{1cm} (EQ 4.3.3-3)

Where:

- $F_{bS_{eff}'} = Design\ Member\ Capacity\ in\ Bending$
- $F_{bS_{eff}} = Reference\ ASD\ Value\ for\ Bending$

### 4.3.3.3 Shear: Light-Frame
Like bending, the first step is to get the adjusted material property for shear, then this value will be multiplied by the appropriate geometry of the member to get a design capacity in shear. The equation to calculate the adjusted material shear capacity is given by:

\[ F_{v}' = F_v \times C_m \times C_t \times C_i \times K_{fv} \times \phi_v \times \lambda \]  

(EQ 4.3.3-4)

Where:

- \( F_{v}' \) = Adjusted Design Value for Shear (Parallel to Grain)
- \( F_v \) = Reference ASD Value for Shear (Parallel to Grain)
- \( K_{fv} \) = Format Conversion Factor for Shear
- \( \phi_v \) = Resistance Factor for Shear

The equation for calculating the capacity of the member in shear is given by:

\[ V_n' = F_{v}' \times A_v \]  

(EQ 4.3.3-5)

Where:

- \( V_n' \) = Design Member Capacity for Shear
- \( A_v \) = Shear Area

### 4.3.3.4 Shear: CLT

The equation used to calculate the adjusted member strength for rolling shear is given by:

\[ V_s' = V_s \times C_M \times C_t \times K_{fsv} \times \phi_v \times \lambda \]  

(EQ 4.3.3-6)

Where:

- \( V_s' \) = Design Member Capacity for Rolling Shear
- \( V_s \) = Reference ASD Value for Rolling Shear

### 4.3.3.5 Axial: Light-Frame

Axial loading of a stud in compression is referred to as compression parallel to grain because the grain direction of the stud runs vertically, up and down, the same direction as the load.
path. The equation to calculate the adjusted material strength for compression parallel to grain is given by:
\[
F'_c = F_c \times C_m \times C_t \times C_F \times C_p \times K_{F_c} \times \phi \times \lambda \quad \text{(EQ 4.3.3-7)}
\]
Where:
- \(F'_c\) = Adjusted Material Strength for Compression Parallel to Grain
- \(F_c\) = Reference ASD Value for Compression Parallel to Grain
- \(C_p\) = Column Stability Factor
- \(K_{F_c}\) = Format Conversion Factor for Compression Parallel to Grain
- \(\phi\) = Resistance Factor for Compression Parallel to Grain

Now that the adjusted material property has been computed, the member design strength can be calculated as shown below:
\[
P'_n = F'_c \times A_c \quad \text{(EQ 4.3.3-8)}
\]
Where:
- \(P'_n\) = Design Member Capacity in Compression
- \(A_c\) = Compression Area of the Member

**4.3.3.6 Axial: CLT**

The first step is to get the adjusted material strength for compression parallel to grain, this equation is given by:
\[
F'_c = F_c \times C_m \times C_t \times C_p \times K_{F_c} \times \phi \times \lambda \quad \text{(EQ 4.3.3-9)}
\]
Where:
- \(F'_c\) = Adjusted Design Value for Compression Parallel to Grain
- \(F_c\) = Reference ASD Value for Compression Parallel to Grain

The equation to calculate the design strength of the CLT panel for pure axial load is given by:
\[
P'_n = F'_c \times A_{cp} \quad \text{(EQ 4.3.3-10)}
\]
Where:

\( P_n' = \text{Design Member Capacity in Compression} \)

\( A_{cp} = \text{Area of Parallel Sawn Lumber Boards in Compression} \)

**4.3.3.7 Deflection: Light-Frame**

The equation to calculate the deflection of a single span, simply supported member is given by:

\[
\Delta = \frac{5 \times w \times L^4}{384 \times E \times I} \quad \text{(EQ 4.3.3-11)}
\]

Where:

\( \Delta = \text{The Expected Deflection for a Simply Supported Single Span} \)

\( w = \text{Unfactored Distributed Load} \)

\( E = \text{Modulus of Elasticity} \)

\( I = \text{Moment of Inertia} \)

**4.3.3.8 Deflection: CLT**

Deflection for CLT panels were separated by short and long-term effects. Short-term deflection is caused by non-permanent loading such as live load, LL. Long-term deflection is caused by permanent loading such as dead load, DL. The equation to calculate expected deflection is given by:

\[
\Delta = \frac{5 \times LL \times L^4}{384 \times EI_{app}} + K_{cr} \times \frac{5 \times DL \times L^4}{384 \times EI_{app}} \quad \text{(EQ 4.3.3-12)}
\]

Where:

\( EI_{app}' = \text{Apparent Bending Stiffness} \)

\( K_{cr} = \text{Time Dependent Deformation Factor} \)
4.4 Design for Lateral Loads

4.4.1 Overview

In addition to gravity loads, a structure must be able to withstand loads in the lateral direction, perpendicular to gravity loads. Lateral loading is usually considered for effects from wind and seismic. Both wind and seismic loads are location-dependent, and this research is not intended to design a structure for a specific site. For this research, there will be three levels of loading chosen for both wind and seismic. For wind loading, the conditions will be based on wind speed for risk category II. For wind loads, conditions of ‘High’, ‘Medium’, and ‘Low’ were used. These conditions cover just about every location in the U.S. For seismic loading, spectral accelerations for short period and 1 second period were assumed based on seismic zones. Conditions of ‘Very High’, ‘High’, and ‘Medium’ were used.

4.4.2 Wind Loads

Wind produces load on structures through positive and negative pressures. For this research, both the main wind force resisting system (MWFRS) and components and cladding (C&C) were considered in the design. The MWFRS is the structural system that transfers lateral loads to the foundation. In the case of the structures designed in this research, shear walls were used to fill this role. C&C are parts of the structure that receive direct wind pressure, such as sheathing. Wind loads are a factor of wind speed and geometry of the structure. Therefore, since the light-frame and CLT versions of each structure have the same shape and size, the design was performed for the structure type (i.e., static or dynamic) and not the material type. For determining wind loads, ASCE 7-16 directional procedure was followed.

The directional procedure follows a step-by-step process that will be outlined in this section. The first step is to determine the Risk Category of the structure. The definitions associated
with each Risk Category can be found in Table 1.5-1 of ASCE 7-16. The Risk Category chosen for these structures was Risk Category II. The second step is to determine the basic wind speed, \(V\). As mentioned earlier, conditions were used that corresponds to a wind speed shown in Table 4.3.

<table>
<thead>
<tr>
<th>Wind Speed Condition</th>
<th>Low</th>
<th>Medium</th>
<th>High</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind Speed (mph)</td>
<td>115</td>
<td>150</td>
<td>180</td>
</tr>
</tbody>
</table>

The third step is to determine wind load parameters. Most of these parameters are based on the location of the structure and since this research does not focus on a specific location, assumptions were made that are believed to represent the structures best. Further detail and explanation of each parameter is outline in Chapter 26 of ASCE 7-16.

A brief overview and explanation for what was used in this research follows. For wind directionality factor, \(K_d\), the value of 0.85 was chosen because designs were performed for MWFRS and C&C. The exposure category was assumed to be category B because it best represents the expected locations these structures are intended to be used. The topographic factor, \(K_{zt}\), is for wind speed-up effects over hills, ridges, and escarpments. It was assumed these topographic features were not present and therefore a value of 1 was used. Ground elevation factor, \(K_e\), is a factor used to adjust for the density of air. Since this factor is a load reduction factor, a value of 1 was assumed for the designs. Gust-effect factor, \(G\), was taken as 0.85 since the structure is assumed to be rigid. This is because the structures are small and are believed to have a relatively low natural period. The final parameter is the internal pressure coefficient, \(G C_{pi}\), is based on the enclosure classification of the structure. It was assumed that the enclosure classification is partially open which corresponds to a value of +/- 0.18.
The fourth step is to determine the velocity pressure coefficient, $K_z$. This coefficient is based on the exposure category and the height of the structure. $K_z$ was taken as 0.57 as the structure height is less than 15 ft and exposure category B was assumed. The fifth step is to determine the velocity pressure, $q_z$. The equation for $q_z$ can be taken as:

$$q_z = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2$$  \hspace{1cm} \text{(EQ 4.4.2-1)}$$

The sixth step is to determine the external pressure coefficients, $C_p$. Figure 27.3-1 in ASCE 7-16 was used for determining the external pressure coefficients. The final step is to calculate the design wind pressures. The design wind pressures were calculated using the equations shown below.

$$p_{de} = q_z \times G \times C_p$$  \hspace{1cm} \text{(EQ 4.4.2-2)}$$

Where:

$p_{de}$ = Design Wind Pressure (External)
$q_z$ = Velocity Pressure
$G$ = Gust - Effect Factor
$C_p$ = External Pressure Coefficient

$$p_{di} = q_z \times GC_{pi}$$  \hspace{1cm} \text{(EQ 4.4.2-3)}$$

Where:

$p_{di}$ = Design Wind Pressure (Internal)
$q_z$ = Velocity Pressure
$GC_{pi}$ = Internal Pressure Coefficient

Now that the design wind pressures were determined, all that’s left is to use the geometry of the structures to determine the wind loads. The complete wind load calculations can be found in the appendix. The two main values desired from this procedure are the base shear and the overturning moment. From there the required tension force in the hold down and compression in the wall is determined. These values are shown in Table 4.4 below.
Table 4.4: MWFRS Wind Loads

<table>
<thead>
<tr>
<th>Wind Speed 115 mph</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Static Unit</td>
<td>Dynamic Unit</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Perpendicular to Ridge</td>
<td>Parallel to Ridge</td>
<td>Perpendicular to Ridge</td>
<td>Parallel to Ridge</td>
</tr>
<tr>
<td>Overturning Moment (kip*ft)</td>
<td>20.0</td>
<td>19.8</td>
<td>40.2</td>
<td>40.0</td>
</tr>
<tr>
<td>Base Shear (kip)</td>
<td>2.4</td>
<td>0.7</td>
<td>1.8</td>
<td>1.8</td>
</tr>
<tr>
<td>Required Hold Down Force (kip)</td>
<td>2.9</td>
<td>1.0</td>
<td>2.2</td>
<td>2.1</td>
</tr>
<tr>
<td>Required Compression Force (kip)</td>
<td>4.5</td>
<td>2.9</td>
<td>6.6</td>
<td>6.6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wind Speed 150 mph</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Static Unit</td>
<td>Dynamic Unit</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Perpendicular to Ridge</td>
<td>Parallel to Ridge</td>
<td>Perpendicular to Ridge</td>
<td>Parallel to Ridge</td>
</tr>
<tr>
<td>Overturning Moment (kip*ft)</td>
<td>34.0</td>
<td>33.6</td>
<td>68.1</td>
<td>67.9</td>
</tr>
<tr>
<td>Base Shear (kip)</td>
<td>4.1</td>
<td>1.2</td>
<td>3.1</td>
<td>3.0</td>
</tr>
<tr>
<td>Required Hold Down Force (kip)</td>
<td>4.9</td>
<td>1.8</td>
<td>3.7</td>
<td>3.6</td>
</tr>
<tr>
<td>Required Compression Force (kip)</td>
<td>6.5</td>
<td>3.6</td>
<td>8.1</td>
<td>8.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wind Speed 180 mph</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Static Unit</td>
<td>Dynamic Unit</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Perpendicular to Ridge</td>
<td>Parallel to Ridge</td>
<td>Perpendicular to Ridge</td>
<td>Parallel to Ridge</td>
</tr>
<tr>
<td>Overturning Moment (kip*ft)</td>
<td>49.8</td>
<td>49.1</td>
<td>99.8</td>
<td>99.5</td>
</tr>
<tr>
<td>Base Shear (kip)</td>
<td>6.0</td>
<td>1.7</td>
<td>4.5</td>
<td>4.4</td>
</tr>
<tr>
<td>Required Hold Down Force (kip)</td>
<td>7.1</td>
<td>2.6</td>
<td>5.4</td>
<td>5.2</td>
</tr>
<tr>
<td>Required Compression Force (kip)</td>
<td>8.8</td>
<td>4.4</td>
<td>9.8</td>
<td>9.7</td>
</tr>
</tbody>
</table>

The components and cladding calculations were performed to determine the bending and withdrawal strength of the structural sheathing and nailing. It is assumed that only the light-frame unit will require structural sheathing. Therefore, these calculations were only performed for those units. These calculations yielded the nailing schedule and sheathing thickness shown below in
Table 4.5. From the C&C calculations, it was determined the structural sheathing panels must be installed with the strong direction (usually the longer direction) perpendicular to the studs/joists. The panels must also have at least two continuous spans. Nail spacing in the field is typically less than edge spacing because shear is concentrated along panel edges. However, when considering withdrawal from negative wind pressures, the field nails have a greater tributary area. Therefore, it was decided to use equal nail spacing for both edge and field locations. The MWFRS calculations were used to determine the required and design shear strength of the structural sheathing. As shown, a sheathing thickness of 7/16 in. with 6 in. O.C. nailing for both edge and field works in almost every condition. The only scenario where sheathing thickness was increased was on the roof of the dynamic unit in high wind conditions.

<table>
<thead>
<tr>
<th>Wind Speed 115 mph</th>
<th>Wind Speed 150 mph</th>
<th>Wind Speed 180 mph</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Static Unit</td>
<td>Dynamic Unit</td>
</tr>
<tr>
<td><strong>Wall</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sheathing Thickness (in)</td>
<td>7/16</td>
<td>7/16</td>
</tr>
<tr>
<td>Edge Nail Spacing (in)</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Field Nail Spacing (in)</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td><strong>Roof</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sheathing Thickness (in)</td>
<td>7/16</td>
<td>7/16</td>
</tr>
<tr>
<td>Edge Nail Spacing (in)</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Field Nail Spacing (in)</td>
<td>6</td>
<td>6</td>
</tr>
</tbody>
</table>

### 4.4.3 Seismic Loads

Seismic loads differ from wind loads as there is not a physical push on the structure but rather a shake. Earthquakes are a result of movement along plate boundaries. There are three types of plate boundaries, namely transform, divergent, and convergent. These boundary types explain how the plates move relative to each other. Earthquake engineering is an elaborate profession that uses seismology and soil mechanics. The actual cause of earthquakes can be complex. However,
for the scope of this research, it can be explained as differential movement occurring between two plates that breaks the bond of interlocking rocks. When breaking these bonds, immense energy is released in the form of seismic waves. When these waves reach the site of a building, it excites the structure and creates forces that must be designed for. This excitation is commonly measured as an acceleration. As mentioned earlier, there are two accelerations used in determining the earthquake loads, $S_{ms}$, and $S_{m1}$. The values used for each acceleration in each seismic zone are given below in Table 4.6.

<table>
<thead>
<tr>
<th></th>
<th>Medium</th>
<th>High</th>
<th>Very High</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{ms}$ (g)</td>
<td>1</td>
<td>1.5</td>
<td>2.25</td>
</tr>
<tr>
<td>$S_{m1}$ (g)</td>
<td>0.6</td>
<td>0.9</td>
<td>1.35</td>
</tr>
</tbody>
</table>

For seismic loads, the equivalent lateral force (ELF) procedure was used to determine the forces produced by seismic activity. The first step of the ELF procedure is to calculate the weight of the structure. Even though the structures are only one-story, the design was performed as if the structures were two-story. An assumed first story of CMU block was added, which would result in a more conservative design. The next step is to determine the design spectral acceleration from the design response spectrum. The design response spectrum was created, and the natural period was used to determine the design spectral acceleration. An example design response spectrum can be seen in Figure 4.3. The approximate natural period of the structure was determined with equation 4.4.3-1. The upper limit of the natural period was also determined, and both periods were plotted on the design response spectrum. The equation for calculating the upper limit can be seen in equation 4.4.3-2.

$$T_n = C_t \times h_n^x$$  

(EQ 4.4.3-1)
Where:

\( T_n \) = Approximate Natural Period

\( C_t, x \) = Constants

\( h_n \) = Height of Structure

\[
T_u = T_n \times C_u \tag{EQ 4.4.3-2}
\]

Where:

\( T_u \) = Upper Limit of Natural Period

\( C_u \) = Constant

As shown above, the design spectral acceleration decreases in the region of low period. It is conservative and accepted to assume the acceleration in that area does not decrease and instead continues flat. This assumption was made in this research as shown in the calculations in the appendices.

Figure 4.3: Example Design Response Spectrum
The natural period can then be used in conjunction with the design response spectrum to determine the spectral response acceleration, $S_a$. Before being able to calculate the base shear, the seismic response coefficient, $C_s$, must be determined. The equation to determine the seismic response coefficient is shown as:

$$C_s = \frac{S_a}{R \cdot I_e}$$  \hspace{1cm} (EQ 4.4.3-3)

Where:

- $C_s = Seismic\ Response\ Coefficient$
- $S_a = Spectral\ Response\ Acceleration$
- $R = Response\ Modification\ Factor$
- $I_e = Importance\ Factor$

The response modification factor is based on the type of lateral system used in the structure. For the light-frame shear walls, a response modification factor of 6.5 was used. For the CLT shear walls, a response modification factor of 3 was used. Since CLT is a relatively new construction material, the response modification factor for CLT shear walls was not listed in ASCE 7-16. However, during the time of writing, ASCE 7-22 came out, and the response modification factor is listed there. The importance factor is based on the structure's risk category. Since the risk category used for design was II, the importance factor value of 1 was used.

Once the weight and seismic response coefficient is determined, the seismic base shear of the structure can be determined as follows:

$$V = C_s \times W$$  \hspace{1cm} (EQ 4.4.3-4)

Where:

- $V = Seismic\ Base\ Shear$
- $C_s = Seismic\ Response\ Coefficient$
\( W = \text{Weight of the Structure} \)

As mentioned previously, the structures were designed as two-story, therefore the vertical distribution of the forces must be determined as shown below:

\[
C_{vx} = \frac{w_x \times h_x^k}{\sum (w_i \times h_i^k)} \quad \text{(EQ 4.4.3-5)}
\]

\[
F_x = C_{vx} \times V \quad \text{(EQ 4.4.3-6)}
\]

Where:

\( C_{vx} = \text{Vertical Distribution Factor} \)

\( V = \text{Total Design Lateral Force or Shear at the Base} \)

\( w_i \) and \( w_x = \text{Portion of the Structures Weight at Each Level} \)

\( h_i \) and \( h_x = \text{Height from the Base to Each Level} \)

\( k = \text{Constant} \)

Now that the force at each level is computed, the overturning moment can be calculated. Figure 4.4 shows the forces being solved for in this process. The overturning moment can be calculated as:

\[
M_o = F_2 \times h \quad \text{(EQ 4.4.3-7)}
\]

Where:

\( M_o = \text{Overturning Moment} \)

\( F_2 = \text{Shear Force on the Second Floor} \)

\( h = \text{Height from Roof to Floor} \)
Now that all the loads have been determined, the next step is to determine the required strength of the hold downs and the shear resisting elements. The hold downs are required to resist the uplift produced from the overturning moment. To provide some tolerance in the placement of the hold downs 1 ft was subtracted from the moment arm. The equation to determine the required force in the hold down is given by:

\[ T = \frac{M_o}{L - 1\text{ft}} \]  

(EQ 4.4.3-8)

Where:

\( T = \text{Required Force the Hold Down Must Resist} \)

\( M_o = \text{Overturning Moment} \)

\( L = \text{Length of the Shear Wall} \)

The overturning moment causes not only tension on one side of the wall but also compression on the other side of the wall. When calculating the tension force, the dead and live loads were assumed
to be zero. This was a conservative measure to produce the maximum uplift. However, for the compression load, the dead and live load will be considered. The compression load can be determined by the equation below.

\[ C = T + 1.2 \times DL + LL \]  

(EQ 4.4.3-9)

Where:

\( T = \text{Tension} \)

\( DL = \text{Factored Dead Load} \)

\( LL = \text{Factored Live Load} \)

This compression force must be resisted by either the CLT panel or studs located at the edge of the wall. The tension force must be resisted by the hold downs and the studs at the wall end. The two main values desired from this procedure are the base shear and the overturning moment. From there, the required tension force in the hold down and compression in the wall is determined. These values are shown in Tables 4.7 through 4.10 below.

<table>
<thead>
<tr>
<th>CLT Dynamic Unit</th>
<th>Medium</th>
<th>High</th>
<th>Very High</th>
</tr>
</thead>
<tbody>
<tr>
<td>( S_{m1} ) (g)</td>
<td>0.6</td>
<td>0.9</td>
<td>1.35</td>
</tr>
<tr>
<td>( S_{m2} ) (g)</td>
<td>1</td>
<td>1.5</td>
<td>2.25</td>
</tr>
<tr>
<td>Overturning Moment (kip*ft)</td>
<td>36.8</td>
<td>55.3</td>
<td>82.9</td>
</tr>
<tr>
<td>Base Shear (kip)</td>
<td>4.8</td>
<td>7.2</td>
<td>10.8</td>
</tr>
<tr>
<td>Required Hold Down Force (kip)</td>
<td>2</td>
<td>3</td>
<td>4.5</td>
</tr>
<tr>
<td>Required Compression Force (kip)</td>
<td>6.4</td>
<td>7.4</td>
<td>8.9</td>
</tr>
</tbody>
</table>
### Table 4.8: LFRS Seismic Loads for CLT Static Unit

<table>
<thead>
<tr>
<th></th>
<th>CLT Static Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Medium</td>
</tr>
<tr>
<td>$S_m$ (g)</td>
<td>1</td>
</tr>
<tr>
<td>$S_{m1}$ (g)</td>
<td>0.6</td>
</tr>
<tr>
<td>Overturning Moment (kip*ft)</td>
<td>17.8</td>
</tr>
<tr>
<td>Base Shear (kip)</td>
<td>2.1</td>
</tr>
<tr>
<td>Required Hold Down Force (kip)</td>
<td>2.5</td>
</tr>
<tr>
<td>Required Compression Force (kip)</td>
<td>4.2</td>
</tr>
</tbody>
</table>

### Table 4.9: LFRS Seismic Loads for Light-Frame Dynamic Unit

<table>
<thead>
<tr>
<th></th>
<th>LF Dynamic Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Medium</td>
</tr>
<tr>
<td>$S_m$ (g)</td>
<td>1</td>
</tr>
<tr>
<td>$S_{m1}$ (g)</td>
<td>0.6</td>
</tr>
<tr>
<td>Overturning Moment (kip*ft)</td>
<td>9.4</td>
</tr>
<tr>
<td>Base Shear (kip)</td>
<td>1.2</td>
</tr>
<tr>
<td>Required Hold Down Force (kip)</td>
<td>0.5</td>
</tr>
<tr>
<td>Required Compression Force (kip)</td>
<td>4.6</td>
</tr>
</tbody>
</table>

### Table 4.10: LFRS Seismic Loads for Light-Frame Static Unit

<table>
<thead>
<tr>
<th></th>
<th>LF Static Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Medium</td>
</tr>
<tr>
<td>$S_m$ (g)</td>
<td>1.0</td>
</tr>
<tr>
<td>$S_{m1}$ (g)</td>
<td>0.6</td>
</tr>
<tr>
<td>Overturning Moment (kip*ft)</td>
<td>4.6</td>
</tr>
<tr>
<td>Base Shear (kip)</td>
<td>0.5</td>
</tr>
<tr>
<td>Required Hold Down Force (kip)</td>
<td>0.7</td>
</tr>
<tr>
<td>Required Compression Force (kip)</td>
<td>2.2</td>
</tr>
</tbody>
</table>

The shear resistance of the structural sheathing for both the roof diaphragm and shear walls were also determined and the design results can be seen below in Table 4.11.
4.4.4 Results from Lateral Loads

Lateral loads were determined for each structure with varying wind and seismic levels. Sections 4.4.2 and 4.4.3 covered how the loads were calculated and the resulting required forces from those loads. The actual members and connections that must resist those loads will be covered in this section. For axial loads, compression and tension, the CLT or studs will need to resist these forces. It was determined that the 3-ply CLT walls are strong enough for compression. For the light-frame walls, double studs at each end are required to resist the compression and tension forces. Tension for CLT elements is calculated the same as light-frame members; therefore, the two parallel boards in the CLT wall are strong enough for tension. The testing portion of this research resulted in strength values associated with each hinge type in each direction. It was also discussed that the hinges are believed to not make good hold downs for the shear walls. Therefore, the main strength direction that will be used in the design is the shear strength. Also, the continuous hinge type will be chosen as the hinge used in the actual structures. Therefore, the strength that will be used is the ultimate shear strength of the continuous hinge (type A). The test resulted in an ultimate strength, $V_n$, of 7.89 kip. A strength reduction factor $\phi_z$, as well as a nominal-to-mean ratio was used to determine the design strength, $V_n'$. The resulting design strength of the continuous hinge in shear was 3.85 kip. The demand base shear determined in sections 4.4.2 and 4.4.3 was
divided by the design strength to determine the length of hinge along the wall-to-floor and roof-to-wall connection needed to resist the base shear. The resulting hinge lengths for each design can be seen below in Tables 4.12 and Table 4.13.

Table 4.12: Required Length of Hinge from Wind Loads

<table>
<thead>
<tr>
<th>Wind Speed</th>
<th>Length of Hinge Required (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>115 mph</td>
<td>7.5 12.8 18.7</td>
</tr>
<tr>
<td>150 mph</td>
<td>5.6 9.7 14.0</td>
</tr>
<tr>
<td>180 mph</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.13: Required Length of Hinge from Seismic Loads

<table>
<thead>
<tr>
<th>Seismic Level</th>
<th>Length of Hinge Required (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medium</td>
<td>15.0 22.4 33.7</td>
</tr>
<tr>
<td>High</td>
<td>6.5 9.6 14.3</td>
</tr>
<tr>
<td>Very High</td>
<td>3.8 5.7 8.6</td>
</tr>
<tr>
<td>CLT Dynamic</td>
<td></td>
</tr>
<tr>
<td>CLT Static</td>
<td></td>
</tr>
<tr>
<td>LF Dynamic</td>
<td></td>
</tr>
<tr>
<td>LF Static</td>
<td></td>
</tr>
</tbody>
</table>

As shown above, the required length of the hinge never exceeds more than 3 ft. Although that may be all that is required for strength, the actual folding and unfolding of the unit will require more hinge length. During the gravity load calculations, it was determined that there would need to be at least 15 in of length of hinge to resist the shear between the two roof panels. Again, this does not seem like a practical amount of hinge length to easily unfold and fold the structure. Therefore, engineering judgment was used to determine a practical amount of hinge along the panel seams. Figure 4.5 shows the seams in which hinges are required for foldability. The other panel-to-panel connections are also important because they must transfer shear to the shear walls. All hinges will be assumed to be 1 ft long continuous hinges. For the other panel connections, it will be assumed that hinges are used there as well. The hinges in those locations will need to be connected to one
panel until the unit is unfolded. Then the hinge can be fastened to the other panel. The locations that do not require hinges for folding but are assumed to use them for shear resistance can be seen in Figure 4.6. The blue hinge location line in Figure 4.5 is hidden by the structure; however, it will be the same four hinges as shown in the orange hinge locations.

Figure 4. 5: Hinge Locations Required for Foldability and Shear
4.5 Performance Measures

This section covers the categories used throughout the design of these structures. These categories help wrap up the designs in a simple, more approachable way. By providing each structure design with a score in each category, the end user can quickly determine which structure will best suit their needs. Some of these categories could be subjective; however, the goal of the scores is to relate the structures relative to each other. Scoring in each category ranges from 1 to 4. A score of 1 means the structure did not perform relatively well in that category. A score of 2 or 3 means that the structure performed about on average in that category. A score of 4 means that the structure performed the best.
4.5.1 Deployability

Deployability is a measure of how easy it is for the structure to be deployed. Factors that affect deployability include weight, time to assemble/erect, and required tools. Deployability is an important metric as it relates to ease of use. The concept of disaster relief is to be able to provide assistance in a quick and easy manner. If a structure requires specialty tools or labor to be usable, it would not be considered especially deployable. Another reason deployability is important is that in remote areas, there may not be access to large machinery to lift the panels to unfold the unit. As a full-scale model was not in the scope of this research, assembly time could not be measured. However, estimation could be made from similar-sized structures used in the military. Also, an exact time deployment is not as important as the general time between each structure as this scoring system is only meant to compare relatively between each four structure designs. A score of 1 in deployability would be because a structure is lightweight, requires very little if any work upon arrival, and does not require special tools or machines.

The weight of the structures was estimated based on the weight and area of the panels. For light-frame panels, the estimated weight used is 7.5 psf which is slightly higher than the calculated weight. This can be broken down into the categories and weights seen below in Table 4.14. The weight of a CLT 3-ply SL-V3 is given as 13.5 psf by the manufacturer.

<table>
<thead>
<tr>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stud: 0.69</td>
</tr>
<tr>
<td>Sheathing: 1.90</td>
</tr>
<tr>
<td>Drywall: 1.80</td>
</tr>
<tr>
<td>Insulation: 0.50</td>
</tr>
<tr>
<td>T&amp;B Plates: 0.52</td>
</tr>
<tr>
<td>Total Wall: 5.40</td>
</tr>
</tbody>
</table>
The calculation for determining the weight of the structures is shown below. The estimated weight of each unit can be found in Table 4.15.

\[ W_s = W_p \times A_T \]  

(EQ 4.5.1-1)

Where:

\( W_s = Structure \ Weight \)

\( W_p = Weight \ of \ Panel \)

\( A_T = Total \ Area \ of \ Panels \)

<table>
<thead>
<tr>
<th>Weight (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLT Dynamic</td>
</tr>
<tr>
<td>CLT Static</td>
</tr>
<tr>
<td>LF Dynamic</td>
</tr>
<tr>
<td>LF Static</td>
</tr>
</tbody>
</table>

The dynamic units will require more work than the light-frame units because they will need to unfold and be finished on site. The tools required to finish these structures should be essential tools most erectors should already have. The CLT dynamic unit will require some type of machinery to help rotate the panels into place because they are heavier than a reasonable amount of people can lift.

The descriptions for scoring each factor that make up the deployability category can be found in Table 4.16. The scores of each unit in the deployability category can be seen below in Table 4.17. The score for the deployability category is calculated by taking the average of the scores that make up the category. The CLT dynamic unit got the worst score because it is the heaviest unit and requires machinery and finishing on site. The light-frame static unit received the best score because it is the lightest unit that requires minimal work on site.
### Table 4.16: Description of Deployability Factors Scoring

<table>
<thead>
<tr>
<th>Score</th>
<th>Weight</th>
<th>Deployment Time</th>
<th>Required Tools</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Describes a unit that weighs more than 20 kips</td>
<td>Describes a unit that requires longer than one day for complete deployment</td>
<td>Describes a unit that requires tools and machinery</td>
</tr>
<tr>
<td>2</td>
<td>Describes a unit that weighs more than 15 kips</td>
<td>Describes a unit that requires nearly a full day for complete deployment</td>
<td>Describes a unit that requires specialty tools</td>
</tr>
<tr>
<td>3</td>
<td>Describes a unit that weighs more than 10 kips</td>
<td>Describes a unit that requires nearly a few hours for complete deployment</td>
<td>Describes a unit that requires only standard tools</td>
</tr>
<tr>
<td>4</td>
<td>Describes a unit that weighs less than 10 kips</td>
<td>Describes a unit that requires less than a few hours for complete deployment</td>
<td>Describes a unit that requires no tools or machinery</td>
</tr>
</tbody>
</table>

### Table 4.17: Deployability Scores

<table>
<thead>
<tr>
<th></th>
<th>Weight</th>
<th>Deployment Time</th>
<th>Required Tools</th>
<th>Deployability</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLT Dynamic</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1.0</td>
</tr>
<tr>
<td>CLT Static</td>
<td>4</td>
<td>3</td>
<td>1</td>
<td>2.7</td>
</tr>
<tr>
<td>LF Dynamic</td>
<td>3</td>
<td>2</td>
<td>3</td>
<td>2.7</td>
</tr>
<tr>
<td>LF Static</td>
<td>4</td>
<td>3</td>
<td>3</td>
<td>3.3</td>
</tr>
</tbody>
</table>

#### 4.5.2 Transportability

The transportability category is a measure of how easy it is to transport these structures. Another consideration is how many structures can be shipped on one truck. Transportability is a function of the weight and size of the unit. All these units were designed to be within ISO standard shipping sizes. The maximum weight of a step deck trailer is 45 kips, and the maximum dimensions are about 50 ft long x 8.5 ft wide x 10.5 ft tall (Stream Logistics, n.d.). This limits both the static and dynamic units to two units per shipment. However, when looking at shipment in terms of square feet per shipment, the dynamic units clearly win. The descriptions for scoring each factor that make up the transportability category can be found in Table 4.18. The scores of each unit in
the transportability category can be seen below in Table 4.19. The score for the transportability
category is calculated by taking the average of the scores that make up the category. The light-
frame dynamic unit scored the best because it is relatively lightweight and can deliver more usable
square footage than its static unit counterpart. The CLT dynamic unit scored the worst because it
weighs the most.

Table 4. 18: Description of Transportability Factors for Scoring

<table>
<thead>
<tr>
<th>Score</th>
<th>Weight</th>
<th>Size</th>
<th>Shipment Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Describes a unit that weighs more than 20 kips</td>
<td>Describes a unit that is less than 100 sq. ft.</td>
<td>Describes a unit where one can be shipped on one truck</td>
</tr>
<tr>
<td>2</td>
<td>Describes a unit that weighs more than 15 kips</td>
<td>Describes a unit that is more than 100 sq. ft.</td>
<td>Describes a unit where two can be shipped on one truck</td>
</tr>
<tr>
<td>3</td>
<td>Describes a unit that weighs more than 10 kips</td>
<td>Describes a unit that is more than 200 sq. ft.</td>
<td>Describes a unit where three can be shipped on one truck</td>
</tr>
<tr>
<td>4</td>
<td>Describes a unit that weighs less than 10 kips</td>
<td>Describes a unit that is more than 300 sq. ft.</td>
<td>Describes a unit where four or more can be shipped on one truck</td>
</tr>
</tbody>
</table>

Table 4. 19: Transportability Scores

<table>
<thead>
<tr>
<th>CLT Dynamic</th>
<th>Weight</th>
<th>Size</th>
<th>Shipment Quantity</th>
<th>Transportability</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLT Static</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td>2.7</td>
</tr>
<tr>
<td>LF Dynamic</td>
<td>3</td>
<td>4</td>
<td>2</td>
<td>3.0</td>
</tr>
<tr>
<td>LF Static</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td>2.7</td>
</tr>
</tbody>
</table>

4.5.3 Cost

Cost is important because it shows the feasibility of implementing these structures on a
wide scale use. As with any product, the price can fluctuate depending on the materials used in
production. Also, the price can be reduced when built in large quantities. However, for this
research price of a single unit will be estimated at the current prices. The current price for 3-ply
southern yellow pine CLT is around $39 to $44 per sq. ft. (J. Gouge, personal communication,
January 19, 2022). For the CLT structures, a cost of $40 per sq. ft. was used as this is within the
expected price range. These prices do not include the assembly of the units. A rough breakdown of the light-frame panel cost per sq. ft. can be seen in Table 4.20. The cost used for the light-frame units was $7.5 per sq. ft. as this seemed like a reasonable cost that was conservative to the calculated values. The description for scoring the cost category can be found in Table 4.22. The scores of each unit in the cost category can be seen below in Table 4.23.

<table>
<thead>
<tr>
<th>Table 4. 20: Light-frame Panel Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cost ($/sq.ft.)</td>
</tr>
<tr>
<td>Stud</td>
</tr>
<tr>
<td>Sheathing</td>
</tr>
<tr>
<td>Drywall</td>
</tr>
<tr>
<td>Insulation</td>
</tr>
<tr>
<td>T&amp;B Plates</td>
</tr>
<tr>
<td>Total Wall</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 4. 21: Material and Complete Structure Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Cost ($)</td>
</tr>
<tr>
<td>CLT Dynamic</td>
</tr>
<tr>
<td>CLT Static</td>
</tr>
<tr>
<td>LF Dynamic</td>
</tr>
<tr>
<td>LF Static</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 4. 22: Description for Cost Scoring</th>
</tr>
</thead>
<tbody>
<tr>
<td>Score</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
</tbody>
</table>
Table 4. 23: Cost Scores

<table>
<thead>
<tr>
<th></th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLT Dynamic</td>
<td>1</td>
</tr>
<tr>
<td>CLT Static</td>
<td>1</td>
</tr>
<tr>
<td>LF Dynamic</td>
<td>4</td>
</tr>
<tr>
<td>LF Static</td>
<td>4</td>
</tr>
</tbody>
</table>

4.5.4 Building Envelope Resistance

Building envelope resistance is an important category for consideration in both civilian and military sectors. Some factors that could be used in this category include debris impact resistance, insect resistance, and fire resistance. Some factors that may play a larger role in military use are ballistic and blast resistance. The latter two factors will be used to create a score for each structure in this category.

The ballistic performance of the CLT panels was estimated using the THOR CLT model introduced earlier and shown in equation 2.5.2-1. The calculation was performed for a half-inch diameter steel ball projectile. The estimated striking velocity that can be resisted by a 3-ply southern yellow pine CLT panel is 1650 fps. Modern firearms do not use spherical steel balls as projectiles, and therefore these results are difficult to compare to real-world ammunition types. However, comparing the striking velocity to the velocities shown in Figure 2.8, it can be estimated that the CLT structures may show ballistic performance around the III-A NIJ rating. This roughly means that the CLT structures could be able to all common handgun bullets. Level III-A exceeds or meets most law enforcement agencies' requirements in the U.S. (T. Muszynski, personal communication, June 27, 2022). The light-frame walls are built up from $\frac{1}{2}$ in. dry wall and 7/16 in. OSB sheathing with studs spaced at 24 in. O.C. The light-frame walls will provide practically zero ballistic resistance. These walls will not even meet the lowest NIJ rating of level I.
The blast performance of CLT structures was discussed in the literature review chapter. CLT panels in the mentioned tests were able to withstand pressures up to 1900 psf. It would be difficult to compare the results of these tests to the structures designed in this research because blast resistance is heavily influenced by connections and structure penetrations. The blast performance of CLT structures is expected to perform much better than light-frame structures due to panel strength and composition. The light-frame units are expected to provide negligible blast resistance, whereas the CLT units are expected to provide substantial blast resistance. For scoring purposes, only scores 1 and 4 were used for blast resistance.

The descriptions for scoring each factor that make up the building envelope resistance category can be found in Table 4.24. The scores of each unit in the building envelope resistance category can be seen below in Table 4.25.

<table>
<thead>
<tr>
<th>Score</th>
<th>Ballistic Performance</th>
<th>Blast Performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Describes a unit that would perform worse than NIJ level I</td>
<td>Describes a unit that provides negligible blast resistance</td>
</tr>
<tr>
<td>2</td>
<td>Describes a unit that would perform to NIJ level I</td>
<td>Not Used</td>
</tr>
<tr>
<td>3</td>
<td>Describes a unit that would perform to NIJ level II</td>
<td>Not Used</td>
</tr>
<tr>
<td>4</td>
<td>Describes a unit that would perform to NIJ level III-A</td>
<td>Describes a unit that provides substantial blast resistance</td>
</tr>
</tbody>
</table>

Table 4.25: Building Envelope Resistance Scores

<table>
<thead>
<tr>
<th></th>
<th>Ballistic</th>
<th>Blast</th>
<th>Building Envelope Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLT Dynamic</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>CLT Static</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>LF Dynamic</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>LF Static</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>
4.5.5 Performance Measure Summary

Each structure was assigned scores in the categories of deployability, transportability, cost, and building envelope resistance. A radar chart shown in Figure 4.7 was made to show each performance category for all structure types together. This chart makes it easy to see which units perform best in each category and helps the end user determine which unit is right for their purposes. If a structure type has a higher score, it means that it performs comparatively well in that category.

![Figure 4.7: Radar Chart Showing the Results of the Performance Measures](image)

4.6 Structure Foundation

As with any structure, the design is only as strong as the foundation. The challenge with this research is that there is not a particular location for which these structures are designed around. Also, these units serve the purpose of being a rapidly deployable structure. Therefore, pouring a
concrete foundation for these units would not be an acceptable solution. The intent of this research is to provide a shelter system that can be deployed anywhere in the world and serve the purpose for which they are intended. With a wide range of site conditions, the foundation system chosen was concrete masonry units (CMU). By choosing CMUs, the erector of these structures will be able to provide a level foundation in a short period of time in almost any location. CMU, also known as cinder blocks, are also relatively affordable. Figure 4.8 shows the location of CMUs for both the static and dynamic units.

For all CMUs along the exterior of each structure, both cells must be filled with grout, and an anchor rod must be used in each cell to secure the structure down to the blocks.

Another consideration is how the forces will transfer from the CMU foundation to the ground. The following are a few options; however, the calculations for this consideration were considered outside the scope of this research. The first solution could be to use post-installed
adhesive anchors. This solution could only be used if the structures were placed on an existing concrete slab. This makes this solution rather limited; however, it is commonly used in the construction industry and has proven its performance as a reliable connection. Post-installed adhesive anchors can also be quick to implement. Another solution that could be used where concrete slabs are not available is diamond pier footings. These footings are precast concrete piers with galvanized steel pins that are driven into the ground. Figure 4.9 shows a diamond pier and its components. A few benefits of using a diamond pier footing include versatility in soil types and quick use as no time is needed for curing. Both foundations are viable solutions that could be used depending on the site conditions.

Figure 4.9: Diamond Pier Footing System
4.7 Summary of Design

The gravity design performed on all four units can be found in the appendices. In summary, light-frame units were designed with 2x4 and 2x6 No. 2 Southern Pine boards. The CLT units were designed with SL-V3 3-ply panels. Throughout the design, there were some assumptions made, such as how the floors will be supported. Since these units could be deployed in many different environments and conditions, it was assumed that the floor would be supported by CMU blocks. These allow the units to be leveled if a flat ground or slab is not available. While on the topic of foundations, the actual connection to the ground/foundation was not in the scope of this research; however possible solutions were mentioned. In the dynamic unit, roof panel 3 overhangs wall panel 4, because of this cantilever, there will need to be tension straps. The actual tension strap was not designed. Instead, the required load the tension strap must resist was calculated. It is believed the best method for the dynamic unit erection is to use temporary supports so that uplift does not occur. However, by providing tension straps, the units will still be functional if temporary support is not used there. These temporary supports would support the roof panel while the wall panels are being put into place. Temporary bracing can be 4x4 posts provided every 5 ft along the rim joist. The temporary bracing can be directly above the CMU locations where needed in the dynamic units. The temporary bracing can be shipped with these units as there would be extra space on the trailer. Non-structural components such as windows, doors, siding, and roofing were not selected or incorporated into this design and are considered to be outside the scope of this research. In addition to the items listed above, the waterproofing of these units was not part of this design. However, it is expected that the dynamic units would need to be finished with waterproofing measures once they arrive and are deployed on-site because of the seams between adjacent panels.
Chapter 5: Summary, Conclusions, and Recommendations

5.1 Summary of Study

This research consisted of three major sections, literature review, experimental testing, and design. The literature review section reviewed existing and innovative deployable structure systems being implemented in the military. It also covered force protection performance of cross-laminated timber such as ballistic and blast. The testing section of this research contained results and discussion from six tests. Two of the tests were performed to acquire the materialistic properties of the hinges' base metal. The other four tests were performed to determine the multidirectional strength of two different hinge types. These test results were then used to determine the applicability of hinges as a structural element in the units. The design chapter contains structural design and performance measure considerations for four types of rapidly deployable structures using different materials and deployment strategies. The structural design portion of this research contains gravity and lateral design for all four structure types. The lateral design was performed for three levels of each wind and seismic force. The four performance categories used in this research were deployability, transportability, cost, and building envelope resistance. By assigning scores to each unit for each category, the user of these structures can easily determine which unit type will work best in their situation. These units could be used in many different environments for multiple purposes; therefore, variety is important.

5.2 Conclusions

Although hinges are commonly implemented in deployable structure designs, the design strengths and testing on hinges are less known and studied. This lack of research led to the testing performed in this study. It was shown that NDS equations for dowel-type fasteners could be used as a rough estimate for hinge strength as the pin and knuckle assembly usually does not control.
This may not always be the case, and the need for testing the hinges planned for use is recommended. Showing that the NDS equations provide reasonable estimates opens up the possibility to explore and experiment with different size hinges based on the loads the structure is expected to face. It was determined that hinges do not perform well as a hold-down connection due to the width of the hinge leaf. A narrow hinge leaf does not provide enough end distance which can lead to cracking in the wood. It was also determined that the hinges provide adequate strength to be used as shear elements along diaphragm and shear wall connections. Each unit type was shown to have advantages over one another. By having both a static and dynamic unit, users can choose based on their needs. For example, the end user may prefer to have a static unit because it can be completely finished before shipment. Since the unit does not fold, all furnishings and finishes can be done before the end user receives the unit. Also, there is no additional work once the unit is set on a foundation because the unit does not need to be unfolded. A dynamic unit may be chosen because they provide a larger livable space that requires less space during storage and transportation. Both the static and dynamic units are designed in a way that multiple units can be placed next to each other to expand the usable space. The light-frame units will be lighter than their CLT counterparts allowing them to be transported and deployed more easily. Also, the light-frame units will have wall cavity space that can be filled with insulation to provide a more comfortable and energy-efficient shelter. Light-frame units are also easier to hide electrical and plumbing within the walls. Also, since light-frame construction uses less wood than CLT, the final costs of the light-frame units are expected to be less. The CLT units are much stronger and can be used in more extreme conditions. For example, CLT units should be able to provide greater resistance to threats such as blast and ballistic.
5.3 Recommendations

Recommendations for structure type vary depending on the expected use of the unit. For example, if the structure's main use is to provide value-oriented shelter for the civilian market, the light-frame options are believed to be the best choice. However, if the structure is expected to be used in harsh environments or expected to be at risk of blast or ballistic scenarios, the CLT options are believed to be the better choice.

Future research recommendations include further testing on hinges of varying types and sizes. Hinges are used frequently in deployable structure design, and without having accurate behavioral properties of the hinges, these structures could be unsafe. As mentioned, the testing in this research was not intended to determine design values for the hinges tested. Rather, it was intended to observe how the hinges perform as structural elements. Also, a similar structure design using SIPs could be a good alternative to both light-frame and CLT styles. Finally, the construction of a scaled version of these units would be beneficial to help determine more detail.
References


American Society of Civil Engineers (ASCE). (2017). Minimum design loads and associated criteria for buildings and other structures.

American Society of Civil Engineers (ASCE). (2022). Minimum design loads and associated criteria for buildings and other structures.


Hunter, R. L (2012). Mail-order homes: Sears homes and other kit houses. Shire Publications


Appendix A. Mathcad Calculations for Gravity Loads: Light-Frame Static Unit
Appendix A. Mathcad Calculations for Gravity Loads: Light-Frame Static Unit

This document contains the structural calculations for the Light-Frame Static Unit.

<table>
<thead>
<tr>
<th>Dimensions of Each Panel:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Panel:</td>
<td>Long Wall Panel:</td>
</tr>
<tr>
<td>Width = 8'</td>
<td>Height = 8' 10&quot;</td>
</tr>
<tr>
<td>Length = 20'</td>
<td>Length = 20'</td>
</tr>
<tr>
<td>Floor Panel:</td>
<td>Short Wall Panel:</td>
</tr>
<tr>
<td>Height: 8'</td>
<td>Height = 8' 10&quot;</td>
</tr>
<tr>
<td>Length = 20'</td>
<td>Width = 7' 4&quot;</td>
</tr>
</tbody>
</table>

Roof Panel: $E := 1400 \text{ ksi}$

$L := 8 \text{ ft}$ Length of roof joist
$DL := 20 \text{ psf}$ Dead Load
$LL := 20 \text{ psf}$ Live Load

Assume a spacing of joists:
$trib := 24 \text{ in}$

$w := (DL + LL) \cdot trib = 80 \text{ plf}$ unfactored
$w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 112 \text{ plf}$ factored

LRFD Factors:
$C_M := 1$
$C_t := 1$
$C_L := 1$ Floor sheathing nailed to joists, providing lateral stability
$C_F = ?$ Unknown
$C_{fu} := 1$
$C_i := 1$
$C_r := 1.15$

Deflection:

$$\Delta = \frac{5}{384} \cdot \frac{w \cdot L^4}{E' \cdot I}$$

$E' := E \cdot C_M \cdot C_t \cdot C_i = (1.4 \cdot 10^3) \text{ ksi}$

$$\Delta_{allow} := \frac{L}{180} = 0.533 \text{ in}$$

$$I_{req} := \frac{5}{384} \cdot \frac{w \cdot L^4}{\Delta_{allow} \cdot E'} = 9.874 \text{ in}^4$$

$I_{2x6} := 20.8 \text{ in}^4$ OK Moving on with 2x6 @ 24" o.c.
Appendix A. Mathcad Calculations for Gravity Loads: Light-Frame Static Unit

\[ trib = 24 \text{ in} \quad b = 1.5 \text{ in} \quad d = 5.5 \text{ in} \quad S_{xx, 2x6} := \frac{b \cdot d^2}{6} = 7.563 \text{ in}^3 \]

**Bending:** \[ F_b := 1000 \text{ psi} \]

\[ M_u := \frac{w_u \cdot L^2}{8} = 10.752 \text{ kip in} \quad \phi_b := 0.85 \quad K_{F_b} := 2.54 \quad \lambda := 0.8 \quad C_F := 1 \]

\[ F_b' := F_b \cdot \lambda \cdot \phi_b \cdot K_{F_b} \cdot C_M \cdot C_I \cdot C_L \cdot C_r \cdot C_{f_u} \cdot C_F = 1.986 \text{ ksi} \]

\[ M'_{n} := F_b' \cdot S_{xx, 2x6} = 15.021 \text{ kip in} \quad \text{OK} \]

**Shear:** \[ F_v := 175 \text{ psi} \]

\[ V_u := \frac{w_u \cdot L}{2} = 0.448 \text{ kip} \]

\[ \phi_v := 0.75 \quad K_{F_v} := 2.88 \quad \lambda = 0.8 \]

\[ F_v' := F_v \cdot C_M \cdot C_I \cdot K_{F_v} \cdot \phi_v \cdot \lambda = 302.4 \text{ psi} \]

\[ A_v := \frac{2}{3} \cdot b \cdot d = 5.5 \text{ in}^2 \]

\[ V_n' := F_v' \cdot A_v = 1.663 \text{ kip} \quad \text{OK} \]

**Rim Joist:**

Rim Joists will be supported along their full length. Therefore, I believe a single 2x6 would be adequate, however, for purposes of mounting hinges and other fastening hardware, I believe it would be best to use double 2x6.

**Summary of Roof Panel:**
- 2x6 Joist @ 24" o.c.
- Double 2x6 Rim Joist Supported along full length by exterior walls
Appendix A. Mathcad Calculations for Gravity Loads: Light-Frame Static Unit

Floor Panels:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L$</td>
<td>8 ft</td>
<td>Length of floor joist</td>
</tr>
<tr>
<td>$DL$</td>
<td>20 psf</td>
<td>Dead Load</td>
</tr>
<tr>
<td>$LL$</td>
<td>40 psf</td>
<td>Live Load</td>
</tr>
</tbody>
</table>

Assume a spacing of joists:

$trib := 16$ in

$w := (DL + LL) \cdot trib = 80$ plf unfactored

$w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 117.333$ plf factored

LRFD Factors:

$C_M := 1$

$C_t := 1$

$C_L := 1$  
Floor sheathing nailed to joists, providing lateral stability

$C_F := ?$  
Unknown

$C_{fu} := 1$

$C_i := 1$

$C_r := 1.15$

Deflection:

$\Delta = \frac{5}{384} \cdot \frac{w \cdot L^4}{E' \cdot I}$

$E' := E \cdot C_M \cdot C_t \cdot C_i = (1.4 \cdot 10^3) \text{ ksi}$

$\Delta_{allow} := \frac{L}{240} = 0.4 \text{ in}$

$I_{req} := \frac{5}{384} \cdot \frac{w \cdot L^4}{\Delta_{allow} \cdot E'} = 13.166 \text{ in}^4$

$I_{2x6} := 20.8 \text{ in}^4$  
OK  
Moving on with 2x6 @ 24" o.c.

$trib := 16$ in  
$b := 1.5$ in  
$d := 5.5$ in  
$S_{xx, 2x6} := \frac{b \cdot d^2}{6} = 7.563 \text{ in}^3$

Bending:

$F_b := 1000$ psi

$M_u := \frac{w_u \cdot L^2}{8} = 11.264 \text{ kip in}$

$\phi_b := 0.85$

$K_{Fb} := 2.54$

$\lambda := 0.8$

$C_F := 1$

$F_b' := F_b \cdot \lambda \cdot \phi_b \cdot K_{Fb} \cdot C_M \cdot C_t \cdot C_i \cdot C_L \cdot C_r \cdot C_{fu} \cdot C_F = 1.986 \text{ ksi}$

$M''_n := F_b' \cdot S_{xx, 2x6} = 15.021 \text{ kip in}$  
OK
Shear: \[ F_v := 175 \text{ psi} \]
\[ V_u := \frac{w_u \cdot L}{2} = 0.469 \text{ kip} \]
\[ \phi_v := 0.75 \quad K_{Fv} := 2.88 \quad \lambda = 0.8 \]
\[ F_v' := F_v \cdot C_M \cdot C_t \cdot C_i \cdot K_{Fv} \cdot \phi_v \cdot \lambda = 302.4 \text{ psi} \]
\[ A_v := \frac{2}{3} \cdot b \cdot d = 5.5 \text{ in}^2 \]
\[ V_n' := F_v' \cdot A_v = 1.663 \text{ kip} \quad \text{OK} \]

Rim Joist:

Assume we are supporting these units by CMU Blocks at some spacing. These CMU block would need to be placed along the rim joist line at spacing determined below.

\[ L = 5 \text{ ft} \quad \text{spacing of CMU Supports} \]
\[ trib = 4 \text{ ft} \quad \text{Half of panel width} \]
\[ w := (DL + LL) \cdot trib = 240 \text{ plf} \quad \text{unfactored} \]
\[ w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 352 \text{ plf} \quad \text{factored} \]

Deflection:
\[ \Delta = \frac{5}{384} \cdot \frac{w \cdot L^4}{E' \cdot I} \]
\[ E' := E \cdot C_M \cdot C_t \cdot C_i = (1.4 \cdot 10^3) \text{ ksi} \]
\[ \Delta_{allow} := \frac{L}{240} = 0.25 \text{ in} \]
\[ I_{req} := \frac{5}{384} \cdot \frac{w \cdot L^4}{\Delta_{allow} \cdot E'} = 9.643 \text{ in}^4 \]
\[ I_{2x6} = 20.8 \text{ in}^4 \]
\[ I_{prov} := I_{2x6} \cdot 2 = 41.6 \text{ in}^4 \quad \text{OK} \]

Moving on with Double 2x6 with CMU spaced at
\[ trib = 48 \text{ in} \quad b := 1.5 \text{ in} \quad d := 5.5 \text{ in} \]
\[ S_{prov} := \frac{2 \cdot b \cdot d^2}{6} = 15.125 \text{ in}^3 \]

Bending:
\[ F_b = 1000 \text{ psi} \]
\[ M_u := \frac{w_u \cdot L^2}{8} = 13.2 \text{ kip \cdot in} \]
\[ \phi_b = 0.85 \quad K_{Fb} = 2.54 \quad \lambda = 0.8 \quad C_F = 1 \]
\[ F_b' := F_b \cdot \lambda \cdot \phi_b \cdot K_{Fb} \cdot C_M \cdot C_t \cdot C_i \cdot C_L \cdot C_r \cdot C_f \cdot C_F = 1.986 \text{ ksi} \]
\[ M'_{n} := F_b' \cdot S_{prov} = 30.042 \text{ kip \cdot in} \quad \text{OK} \]
Appendix A. Mathcad Calculations for Gravity Loads: Light-Frame Static Unit

Shear: 

\[ V_u := \frac{w_u \cdot L}{2} = 0.88 \text{ kip} \]

\[ \phi_v = 0.75 \quad K_{F_v} = 2.88 \quad \lambda = 0.8 \]

\[ F_v' = F_v \cdot C_M \cdot C_t \cdot C_i \cdot K_{F_v} \cdot \phi_v \cdot \lambda = 302.4 \text{ psi} \]

\[ A_v := \frac{2}{3} \cdot 2 \cdot b \cdot d = 11 \text{ in}^2 \]

\[ V_n' = F_v' \cdot A_v = 3.326 \text{ kip} \]

Summary of Floor Panel 6:
- 2x6 Joist @ 16" o.c.
- Double 2x6 Rim Joist Supported every 5'
Appendix A. Mathcad Calculations for Gravity Loads: Light-Frame Static Unit

Long Wall Panel:

\[
LL = 40 \text{ psf} \\
DL = 20 \text{ psf} \\
trib := 4 \text{ ft} \quad \text{half width of panel 1} \\
width := 24 \text{ in} \quad \text{spacing of studs}
\]

\[
P_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib \cdot width = 0.704 \text{ kip}
\]

Try 2x4
\[
F_c := 1450 \text{ psi} \\
E_{\text{min}} := 510000 \text{ psi}
\]

* Sheathing will brace against weak axis buckling

\[
H := 8 \text{ ft} + 10 \text{ in}
\]

\[
d := 3.5 \text{ in} \\
b := 1.5 \text{ in}
\]

\[
\frac{H}{d} = 30.286 < 50 \text{ OKAY}
\]

\[
C_m := 1 \quad C_t := 1 \quad C_F := 1 \quad C_i := 1 \quad C_p \text{ unknown} \quad C_T := 1
\]

\[
K_{f,E_{\text{min}}} := 1.76 \quad \phi_{E_{\text{min}}} := 0.85 \quad K_{f,C} := 2.4 \quad \phi_c := 0.9 \quad \lambda = 0.8
\]

\[
E_{\text{min}}' = E_{\text{min}} \times [ - C_M \quad C_t \quad - \quad - \quad C_i \quad - \quad C_T \quad 1.76 \quad 0.85 \quad - ]
\]

\[
E_{\text{min}}' = E_{\text{min}} \cdot C_m \cdot C_t \cdot C_i \cdot C_T \cdot K_{f,E_{\text{min}}} \cdot \phi_{E_{\text{min}}} = 762.96 \text{ ksi}
\]

\[
F_{E,E} := \frac{0.822 \cdot E_{\text{min}}'}{(H^2/d)} = 0.684 \text{ ksi}
\]

\[
F_{c,\text{star}} := F_c \cdot C_m \cdot C_t \cdot C_i \cdot C_T \cdot K_{f,C} \cdot \phi_c \cdot \lambda = 2.506 \text{ ksi}
\]

Calculating \( C_p \):
\[
c := 0.8 \quad \text{for sawn lumber} \\
\alpha_c := \frac{F_{E,E}}{F_{c,\text{star}}} = 0.273
\]

\[
C_p := \frac{1 + \alpha_c}{2 \cdot c} - \sqrt{\left( \frac{1 + \alpha_c}{2 \cdot c} \right)^2 - \frac{\alpha_c}{c}} = 0.255 \quad \text{Not ideal}
\]
Appendix A. Mathcad Calculations for Gravity Loads: Light-Frame Static Unit

\[ F'_c := F_{c,\text{star}} \cdot C_p = 639.862 \text{ psi} \]

\[ A_c := b \cdot d = 5.25 \text{ in}^2 \]

\[ P' := A_c \cdot F'_c = 3.359 \text{ kip} \]

\[ P_u = 0.704 \text{ kip} \quad \text{okay} \]

If we put in 8' opening for door:
The studs around the opening will need to take more load:

\[ LL = 40 \text{ psf} \]
\[ DL = 20 \text{ psf} \]
\[ trib := 4 \text{ ft} \quad \text{half width of panel 1} \]
\[ width := 4 \text{ ft} \quad \text{half of opening} \]

\[ P_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib \cdot width = 1.408 \text{ kip} \]

Try 2x4
\[ F_c := 1450 \text{ psi} \]
\[ E_{\text{min}} := 510000 \text{ psi} \]

* Sheathing will brace against weak axis buckling

\[ H := 8 \text{ ft} + 10 \text{ in} \]
\[ d := 3.5 \text{ in} \]
\[ b := 1.5 \text{ in} \]

\[ \frac{H}{d} = 30.286 < 50 \text{ OKAY} \]

\[ C_m := 1 \quad C_t := 1 \quad C_F := 1 \quad C_i := 1 \quad C_p \quad \text{unknown} \quad C_T := 1 \]

\[ K_{f,E_{\text{min}}} := 1.76 \quad \phi_{E_{\text{min}}} := 0.85 \quad K_{f,C} := 2.4 \quad \phi_c := 0.9 \quad \lambda = 0.8 \]

\[ E'_{\text{min}} := E_{\text{min}} \cdot C_m \cdot C_t \cdot C_F \cdot C_i \cdot K_{f,E_{\text{min}}} \cdot \phi_{E_{\text{min}}} = 762.96 \text{ ksi} \]

\[ F_{cE} := \frac{0.822 \cdot E'_{\text{min}}}{\left( \frac{H}{d} \right)^2} = 0.684 \text{ ksi} \]

\[ F_{c,\text{star}} := F_c \cdot C_m \cdot C_t \cdot C_F \cdot C_i \cdot K_{f,C} \cdot \phi_c \cdot \lambda = 2.506 \text{ ksi} \]
calculating \( C_p \):
\[ C_p = \frac{1 + \alpha_c}{2 \cdot c} - \sqrt{\left( \frac{1 + \alpha_c}{2 \cdot c} \right)^2 - \frac{\alpha_c}{c}} = 0.255 \quad \text{Not ideal} \]

\[ F'_c := F_{c,\text{star}} \cdot C_p = 639.862 \, \text{psi} \]

\[ A_c := b \cdot d = 5.25 \, \text{in}^2 \]

\[ P' := A_c \cdot F'_c = 3.359 \, \text{kip} \]

\[ P_u = 1.408 \, \text{kip} \quad \text{okay} \]

---

Short Wall Panel:
* Assumed to not be load bearing for gravity. Therefore just use the same 2x4 @ 24” as long wall panel

Required Header on 8' Door Opening:
\[ L := 8 \, \text{ft} \quad \text{Length of roof joist} \]
\[ DL := 20 \, \text{psf} \quad \text{Dead Load} \]
\[ LL := 20 \, \text{psf} \quad \text{Live Load} \]

Assume a spacing of joists:
\[ \text{trib} := 4 \, \text{ft} \]
\[ w := (DL + LL) \cdot \text{trib} = 160 \, \text{plf} \quad \text{unfactored} \]
\[ w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot \text{trib} = 224 \, \text{plf} \quad \text{factored} \]

LRFD Factors:
\[ C_M := 1 \]
\[ C_t := 1 \]
\[ C_L := 1 \quad \text{Floor sheathing nailed to joists, providing lateral stability} \]
\[ C_f = ? \quad \text{Unknown} \]
\[ C_{fu} := 1 \]
\[ C_i := 1 \]
\[ C_r := 1.15 \]

Deflection:
\[ \Delta = \frac{5}{384} \cdot \frac{w \cdot L^4}{E' \cdot I} \]
\[ E' := E \cdot C_M \cdot C_t \cdot C_i = \left(1.4 \cdot 10^3\right) \, \text{ksi} \]
Appendix A. Mathcad Calculations for Gravity Loads: Light-Frame Static Unit

\[ \Delta_{allow} = \frac{L}{180} = 0.533 \text{ in} \]
\[ I_{req} = \frac{5}{384} \cdot \frac{w \cdot L^4}{\Delta_{allow} \cdot E'} = 19.749 \text{ in}^4 \]
\[ I_{2x6} = 20.8 \text{ in}^4 \quad \text{OK} \quad \text{Use double 2x6} \]
\[ trib = 48 \text{ in} \quad b = 1.5 \text{ in} \quad d = 5.5 \text{ in} \quad S_{xx,2x6} = \frac{b \cdot d^2}{6} = 7.563 \text{ in}^3 \quad S_{xx,prov} = 2 \cdot S_{xx,2x6} \]

Bending:
\[ F_b = 1000 \text{ psi} \]
\[ M_u = \frac{w_u \cdot L^2}{8} = 21.504 \text{ kip in} \]
\[ \phi_b = 0.85 \quad K_{Fb} = 2.54 \quad \lambda = 0.8 \quad C_F = 1 \]
\[ F_b' = F_b \cdot \lambda \cdot \phi_b \cdot K_{Fb} \cdot C_M \cdot C_t \cdot C_i \cdot C_L \cdot C_r \cdot C_{fu} \cdot C_F = 1.986 \text{ ksi} \]
\[ M'_n = F_b' \cdot S_{xx,prov} = 30.042 \text{ kip in} \quad \text{OK} \]

Shear:
\[ F_v = 175 \text{ psi} \]
\[ V_u = \frac{w_u \cdot L}{2} = 0.896 \text{ kip} \]
\[ \phi_v = 0.75 \quad K_{Fv} = 2.88 \quad \lambda = 0.8 \]
\[ F_v' = F_v \cdot C_M \cdot C_t \cdot C_i \cdot K_{Fv} \cdot \phi_v \cdot \lambda = 302.4 \text{ psi} \]
\[ A_v = \left( \frac{2}{3} \cdot b \cdot d \right) \cdot 2 = 11 \text{ in}^2 \]
\[ V_n = F_v' \cdot A_v = 3.326 \text{ kip} \quad \text{OK} \]

Two 2x6 nailed to each other for header on 8’ door opening
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

This document contains the structural calculations for the Light-Frame Dynamic Unit.

<table>
<thead>
<tr>
<th>Dimensions of Each Panel:</th>
<th></th>
<th></th>
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<th></th>
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</thead>
<tbody>
<tr>
<td>Panel 1:</td>
<td>Panel 3:</td>
<td>Panel 5:</td>
<td>Panel 7:</td>
<td></td>
</tr>
<tr>
<td>Width = 8'</td>
<td>Width = 6'</td>
<td>Width = 9' 3&quot;</td>
<td>Width = 4' 10&quot;</td>
<td></td>
</tr>
<tr>
<td>Length = 20'</td>
<td>Length = 20'</td>
<td>Height = 10'</td>
<td>Length = 20'</td>
<td></td>
</tr>
<tr>
<td>Panel 2:</td>
<td>Panel 4:</td>
<td>Panel 6:</td>
<td></td>
<td></td>
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<tr>
<td>Height: 7&quot;</td>
<td>Width = 3'</td>
<td>Width = 8'</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length = 20'</td>
<td>Height = 8' 6&quot;</td>
<td>Length = 20'</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Note: Using Southern Yellow Pine (SYP) No.2 for all calculations  \( E := 1400 \text{ ksi} \)
Floor Panels:

**Panel 6**

Panel 6 Joists

\[
\begin{align*}
L &:= 8 \text{ ft} & \text{Length of floor joist} \\
DL &:= 20 \text{ psf} & \text{Dead Load} \\
LL &:= 40 \text{ psf} & \text{Live Load}
\end{align*}
\]

Assume a spacing of joists:

\[
trib := 16 \text{ in}
\]

\[
w := (DL + LL) \cdot trib = 80 \text{ plf} \quad \text{unfactored}
\]

\[
w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 117.333 \text{ plf} \quad \text{factored}
\]

LRFD Factors:

\[
\begin{align*}
C_M &:= 1 \\
C_t &:= 1 \\
C_L &:= 1 & \text{Floor sheathing nailed to joists, providing lateral stability} \\
C_F &:= ? & \text{Unknown} \\
C_{fu} &:= 1 \\
C_i &:= 1 \\
C_r &:= 1.15
\end{align*}
\]

Deflection:

\[
\Delta = \frac{5w \cdot L^4}{384 E' I}
\]

\[
E' := E \cdot C_M \cdot C_t \cdot C_i = 1400 \text{ ksi}
\]

\[
\Delta_{allow} := \frac{L}{240} = 0.4 \text{ in}
\]

\[
I_{req} := \frac{5w \cdot L^4}{384 \Delta_{allow} E'} = 13.166 \text{ in}^4
\]

\[
I_{2\times6} := 20.8 \text{ in}^4 \quad \text{OK} \\
\text{Moving on with 2x6 @ 16'' o.c.}
\]

\[
trib = 16 \text{ in} \quad b := 1.5 \text{ in} \quad d := 5.5 \text{ in} \quad S_{xx,2\times6} := \frac{b \cdot d^2}{6} = 7.563 \text{ in}^3
\]

Bending:

\[
F_b := 1000 \text{ psi}
\]

\[
M_u := \frac{w_u \cdot L^2}{8} = 11.264 \text{ kip \cdot in} \quad \phi_b := 0.85 \quad K_{F_b} := 2.54 \quad \lambda := 0.8 \quad C_F := 1
\]

\[
F_b' := F_b \cdot \phi_b \cdot K_{F_b} \cdot C_M \cdot C_t \cdot C_i \cdot C_L \cdot C_r \cdot C_{fu} \cdot C_F = 1.986 \text{ ksi}
\]

\[
M'_n := F_b' \cdot S_{xx,2\times6} = 15.021 \text{ kip \cdot in} \quad \text{OK}
\]
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

**Shear:**  
\[ F_v := 175 \text{ psi} \]

\[ V_v := \frac{w_u \cdot L}{2} = 0.469 \text{ kip} \]

\[ \phi_v := 0.75 \quad K_{Fv} := 2.88 \quad \lambda = 0.8 \]

\[ F_v' := F_v \cdot C_M \cdot C_t \cdot C_i \cdot K_{Fv} \cdot \phi_v \cdot \lambda = 302.4 \text{ psi} \]

\[ A_v := \frac{2}{3} \cdot b \cdot d = 5.5 \text{ in}^2 \]

\[ V' := F_v' \cdot A_v = 1.663 \text{ kip} \quad \text{OK} \]

**Rim Joist:**

Assume we are supporting these units by CMU Blocks at some spacing. These CMU block would need to be placed along the rim joist line at spacing determined below.

**Spacing:**

\[ L = 5 \text{ ft} \quad \text{spacing of CMU Supports} \]

\[ trib = 4 \text{ ft} \quad \text{Half of panel width} \]

\[ w := (DL + LL) \cdot trib = 240 \text{ plf} \quad \text{unfactored} \]

\[ w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 352 \text{ plf} \quad \text{factored} \]

**Deflection:**

\[ \Delta = \frac{5}{384} \cdot \frac{w \cdot L^4}{E' \cdot I} \]

\[ E' := E \cdot C_M \cdot C_t \cdot C_i = 1400 \text{ ksi} \]

\[ \Delta_{allow} := \frac{L}{240} = 0.25 \text{ in} \]

\[ I_{req} := \frac{5}{384} \cdot \frac{w \cdot L^4}{\Delta_{allow} \cdot E'} = 9.643 \text{ in}^4 \]

\[ I_{2x6} := 20.8 \text{ in}^4 \]

\[ I_{prov} := I_{2x6} \cdot 2 = 41.6 \text{ in}^4 \quad \text{OK} \quad \text{Moving on with Double 2x6 with CMU spaced at} \]

\[ trib = 48 \text{ in} \quad b := 1.5 \text{ in} \quad d := 5.5 \text{ in} \quad S_{prov} := \frac{2 \cdot b \cdot d^2}{6} = 15.125 \text{ in}^3 \]

**Bending:**

\[ F_b = 1000 \text{ psi} \]

\[ M_u := \frac{w_u \cdot L^2}{8} = 13.2 \text{ kip in} \quad \phi_b = 0.85 \quad K_{Fb} = 2.54 \quad \lambda = 0.8 \quad C_F = 1 \]

\[ F_b' := F_b \cdot \phi_b \cdot K_{Fb} \cdot C_M \cdot C_t \cdot C_i \cdot C_L \cdot C_r \cdot C_{fu} \cdot C_F = 1.986 \text{ ksi} \]

\[ M' := F_b' \cdot S_{prov} = 30.042 \text{ kip in} \quad \text{OK} \]
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

**Shear:**

\[ V_u := \frac{w_u \cdot L}{2} = 0.88 \text{ kip} \]

\[ \phi_v = 0.75 \quad K_{Fv} = 2.88 \quad \lambda = 0.8 \]

\[ F'_v := F_v \cdot C_M \cdot C_t \cdot C_i \cdot K_{Fv} \cdot \phi_v \cdot \lambda = 302.4 \text{ psi} \]

\[ A_v := \frac{2}{3} \cdot 2 \cdot b \cdot d = 11 \text{ in}^2 \]

\[ V'_n := F'_v \cdot A_v = 3.326 \text{ kip} \]

**Summary of Floor Panel 6:**
- 2x6 Joist @ 16" o.c.
- Double 2x6 Rim Joist Supported every 5'
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

Panel 7

Panel 7 Joists

\[ L := 5 \text{ ft} \quad \text{Length of floor joist} \]
\[ DL := 20 \text{ psf} \quad \text{Dead Load} \]
\[ LL := 40 \text{ psf} \quad \text{Live Load} \]

Assume a spacing of joists:
\[ trib := 24 \text{ in} \]
\[ w := (DL + LL) \cdot trib = 120 \text{ plf} \quad \text{unfactored} \]
\[ w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 176 \text{ plf} \quad \text{factored} \]

LRFD Factors:
\[ C_M := 1 \]
\[ C_t := 1 \]
\[ C_L := 1 \quad \text{Floor sheathing nailed to joists, providing lateral stability} \]
\[ C_f = ? \quad \text{Unknown} \]
\[ C_{fu} := 1 \]
\[ C_i := 1 \]
\[ C_r := 1.15 \]

Deflection:
\[ \Delta = \frac{5}{384} \cdot \frac{w \cdot L^4}{E' \cdot I} \]
\[ E' := E \cdot C_M \cdot C_t \cdot C_i = 1400 \text{ ksi} \]
\[ \Delta_{allow} := \frac{L}{240} = 0.25 \text{ in} \]
\[ I_{req} := \frac{5}{384} \cdot \frac{w \cdot L^4}{\Delta_{allow} \cdot E'} = 4.821 \text{ in}^4 \]
\[ I_{2x4} := 5.359 \text{ in}^4 \quad \text{OK} \quad \text{Moving on with 2x4 @ 24" o.c.} \]

Bending:
\[ F_b := 1100 \text{ psi} \]
\[ M_u := \frac{w_u \cdot L^2}{8} = 6.6 \text{ kip \cdot in} \]
\[ \phi_b := 0.85 \quad K_{Fb} := 2.54 \quad \lambda := 0.8 \quad C_F := 1 \]
\[ F_b' := F_b \cdot \phi_b \cdot K_{Fb} \cdot C_M \cdot C_t \cdot C_i \cdot C_L \cdot C_r \cdot C_{fu} \cdot C_F = 2.185 \text{ ksi} \]
\[ M'_{n} := F_b' \cdot S_{xx,2x4} = 6.691 \text{ kip \cdot in} \quad \text{OK} \]
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

**Shear:**  
\[ F_v := 175 \text{ psi} \]

\[ V_u := \frac{w_u \cdot L}{2} = 0.44 \text{ kip} \]

\[ \phi_v := 0.75 \quad K_{Fv} := 2.88 \quad \lambda = 0.8 \]

\[ F'_v := F_v \cdot C_M \cdot C_t \cdot C_i \cdot K_{Fv} \cdot \phi_v \cdot \lambda = 302.4 \text{ psi} \]

\[ A_v := \frac{2}{3} \cdot b \cdot d = 3.5 \text{ in}^2 \]

\[ V'_n := F'_v \cdot A_v = 1.058 \text{ kip} \quad \text{OK} \]

**Rim Joist:**

Assume we are supporting these units by CMU Blocks at some spacing. These CMU block would need to be placed along the rim joist line at spacing determined below.

\[ L = 5 \text{ ft} \quad \text{spacing of CMU Supports} \]

\[ trib = 2.5 \text{ ft} \quad \text{Half of panel width} \]

\[ w := (DL + LL) \cdot trib = 150 \text{ plf} \quad \text{unfactored} \]

\[ w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 220 \text{ plf} \quad \text{factored} \]

**Deflection:**

\[ \Delta = \frac{5}{384} \cdot \frac{w \cdot L^4}{E' \cdot I} \]

\[ E' := E \cdot C_M \cdot C_t \cdot C_i = 1400 \text{ ksi} \]

\[ \Delta_{allow} := \frac{L}{240} = 0.25 \text{ in} \]

\[ I_{req} := \frac{5}{384} \cdot \frac{w \cdot L^4}{\Delta_{allow} \cdot E'} = 6.027 \text{ in}^4 \]

\[ I_{2x4} = 5.359 \text{ in}^4 \]

\[ I_{prov} := I_{2x4} \cdot 2 = 10.718 \text{ in}^4 \quad \text{OK} \]

Moving on with Double 2x4 with CMU spaced at

\[ trib = 30 \text{ in} \quad b := 1.5 \text{ in} \quad d := 3.5 \text{ in} \quad S_{prov} := \frac{2 \cdot b \cdot d^2}{6} = 6.125 \text{ in}^3 \]

**Bending:**

\[ F_b = 1100 \text{ psi} \]

\[ M_u := \frac{w_u \cdot L^2}{8} = 8.25 \text{ kip \cdot in} \quad \phi_b = 0.85 \quad K_{Fb} = 2.54 \quad \lambda = 0.8 \quad C_F = 1 \]

\[ F'_b := F_b \cdot \lambda \cdot \phi_b \cdot K_{Fb} \cdot C_M \cdot C_t \cdot C_i \cdot C_L \cdot C_r \cdot C_{fu} \cdot C_F = 2.185 \text{ ksi} \]

\[ M'_{n} := F'_b \cdot S_{prov} = 13.383 \text{ kip \cdot in} \quad \text{OK} \]
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

**Shear:** \( F_v = 175 \text{ psi} \)

\[ V_u := \frac{w_u \cdot L}{2} = 0.55 \text{ kip} \]

\[ \phi_v = 0.75 \quad K_{Fv} = 2.88 \quad \lambda = 0.8 \]

\[ F'_v := F_v \cdot C_M \cdot C_I \cdot C_i \cdot K_{Fv} \cdot \phi_v \cdot \lambda = 302.4 \text{ psi} \]

\[ A_v := \frac{2}{3} \cdot 2 \cdot b \cdot d = 7 \text{ in}^2 \]

\[ V_n' := F'_v \cdot A_v = 2.117 \text{ kip} \]

Summary of Floor Panel 7:
- 2x4 Joist @ 24" o.c.
- Double 2x4 Rim Joist Supported every 5'
Roof Panels:

**Panel 1**

**Panel 1 Joists**

\[ L := 8 \text{ ft} \quad \text{Length of roof joist} \]
\[ DL := 20 \text{ psf} \quad \text{Dead Load} \]
\[ LL := 20 \text{ psf} \quad \text{Live Load} \]

Assume a spacing of joists:
\[ trib := 24 \text{ in} \]
\[ w := (DL + LL) \cdot trib = 80 \text{ plf} \quad \text{unfactored} \]
\[ w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 112 \text{ plf} \quad \text{factored} \]

**LRFD Factors:**
\[ C_M := 1 \]
\[ C_t := 1 \]
\[ C_L := 1 \quad \text{Floor sheathing nailed to joists, providing lateral stability} \]
\[ C_F = ? \quad \text{Unknown} \]
\[ C_{fu} := 1 \]
\[ C_i := 1 \]
\[ C_r := 1.15 \]

**Deflection:**
\[ \Delta = \frac{5}{384} \cdot \frac{w \cdot L^4}{E' \cdot I} \]
\[ E' := E \cdot C_M \cdot C_t \cdot C_i = 1400 \text{ ksi} \]
\[ \Delta_{allow} := \frac{L}{180} = 0.533 \text{ in} \]
\[ I_{req} := \frac{5}{384} \cdot \frac{w \cdot L^4}{\Delta_{allow} \cdot E'} = 9.874 \text{ in}^4 \]
\[ I_{2x6} = 20.8 \text{ in}^4 \quad \text{OK} \quad \text{Moving on with 2x6 @ 24” o.c.} \]

**Bending:**
\[ F_b := 1000 \text{ psi} \]
\[ M_u := \frac{w_u \cdot L^2}{8} = 10.752 \text{ kip \cdot in} \]
\[ \phi_b := 0.85 \quad \lambda := 0.8 \quad C_F := 1 \]
\[ F_b' := F_b \cdot \lambda \cdot \phi_b \cdot K_{fb} ' = 2.54 \quad C_F := 1.986 \text{ ksi} \]
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

\[ M'_n := F'_b \cdot S_{xx_{2x6}} = 15.021 \text{ kip \cdot in} \quad \text{OK} \]

Shear:
\[ F_v := 175 \text{ psi} \]
\[ V_u := \frac{w_u \cdot L}{2} = 0.448 \text{ kip} \]
\[ \phi_v := 0.75 \quad K_{Fv} := 2.88 \quad \lambda = 0.8 \]
\[ F_v' := F_v \cdot C_M \cdot C_t \cdot C_i \cdot K_{Fv} \cdot \phi_v \cdot \lambda = 302.4 \text{ psi} \]
\[ A_v := \frac{2}{3} \cdot b \cdot d = 5.5 \text{ in}^2 \]
\[ V'_n := F'_v \cdot A_v = 1.663 \text{ kip} \quad \text{OK} \]

Rim Joist:

Rim Joists will be supported along their full length. Therefore, I believe a single 2x6 would be adequate, however, for purposes of mounting hinges and other fastening hardware, I believe it would be best to use double 2x6.

Summary of Roof Panel 1:
- 2x6 Joist @ 24” o.c.
- Double 2x6 Rim Joist Supported along full length by either exterior walls or Panel 3
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

**Panel 3**

* Being design as 2 independent panels

\[ L := 10 \text{ ft} \]

Length of span, column at midspan

\[ DL := 20 \text{ psf} \]

Dead Load

\[ LL := 20 \text{ psf} \]

Live Load

* Notice P will cause uplift

*Outside Joist Will have wu and Pu
*Inside Joist only wu

\[ trib := 1.5 \text{ ft} \quad DL := 20 \text{ psf} \quad LL := 20 \text{ psf} \]

\[ w := (DL + LL) \cdot trib = 60 \text{ plf} \quad \text{unfactored} \]

\[ w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 84 \text{ plf} \quad \text{factored} \]

\[ P_{dep} := (DL + LL) \cdot 4 \text{ ft} = 160 \text{ plf} \]

\[ P_{u,dep} := (1.2 \cdot DL + 1.6 \cdot LL) \cdot 4 \text{ ft} = 224 \text{ plf} \quad \text{half width of panel 1 deployed} \]

\[ P_{nd} := DL \cdot 8 \text{ ft} = 160 \text{ plf} \]

\[ P_{u,nd} := 1.4 \cdot DL \cdot 8 \text{ ft} = 224 \text{ plf} \quad \text{undeployed state} \]

*Design Roof Joist as ss Beam

Roof Joist as SS Beam:

\[ L := 10 \text{ ft} \]

*Design the roof joist supports (edge joist) as cantilevered with point load causing uplift and need of tension strap at back span.

Deflection:

\[ \Delta = \frac{5}{384} \cdot \frac{(w + P_{dep}) \cdot L^4}{E' \cdot I} \]

\[ E' := E \cdot C_M \cdot C_t \cdot C_i = 1400 \text{ ksi} \]

\[ \Delta_{allow} := \frac{L}{180} = 0.667 \text{ in} \]

\[ I_{req} := \frac{5}{384} \cdot \frac{(w + P_{dep}) \cdot L^4}{\Delta_{allow} \cdot E'} = 53.036 \text{ in}^4 \quad n := 4 \]

\[ I_{2x6} = 20.8 \text{ in}^4 \quad \text{OK} \]

Moving on with 4 2x6

\[ trib = 18 \text{ in} \quad b := 1.5 \text{ in} \quad d := 5.5 \text{ in} \]

\[ I_{prov} := n \cdot I_{2x6} = 83.2 \text{ in}^4 \]

\[ S_{xx,2x6} := \frac{b \cdot d^2}{6} = 7.563 \text{ in}^3 \]

\[ S_{x,prov} := n \cdot S_{xx,2x6} = 30.25 \text{ in}^3 \]
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

Bending:

\[ F_b := 1000 \text{ psi} \]

\[ M_u := \left( w_u + P_{u, dep} \right) \cdot L^2 \cdot \frac{8}{2} = 46.2 \text{ kip \cdot in} \]

\[ \phi_b := 0.85 \quad K_{Fb} := 2.54 \quad \lambda := 0.8 \quad C_F := 1 \]

\[ F_b' := F_b \cdot \lambda \cdot \phi_b \cdot K_{Fb} \cdot C_M \cdot C_i \cdot C_L \cdot C_c \cdot C_f \cdot C_F = 1.986 \text{ ksi} \]

\[ M'_n := F_b' \cdot S_{x, prov} = 60.085 \text{ kip \cdot in} \quad \text{OK} \]

Shear:

\[ F_v := 175 \text{ psi} \]

\[ V_u := \frac{\left( w_u + P_{u, dep} \right) \cdot L}{2} = 1.54 \text{ kip} \]

\[ \phi_v := 0.75 \quad K_{Fv} := 2.88 \quad \lambda = 0.8 \]

\[ F_v' := F_v \cdot C_M \cdot C_i \cdot K_{Fv} \cdot \phi_v \cdot \lambda = 302.4 \text{ psi} \]

\[ A_v := \frac{2}{3} \cdot n \cdot b \cdot d = 22 \text{ in}^2 \]

\[ V_n' := F_v' \cdot A_v = 6.653 \text{ kip} \quad \text{OK} \]

Roof Joist Support (edge joists) (middle ones will control)

* should have uplift

\[ L = 3 \text{ ft} \quad L_c := 1.5 \text{ ft} \]

\[ \text{tribwidth} = 10 \text{ ft} \quad \text{10' of long roof joist} \]

\[ \text{tribwidth2} = \left( 1.5 \text{ ft} + 4 \text{ ft} \right) = 5.5 \text{ ft} \quad \text{half of panel 1 and half of panel 3} \]

\[ P_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot \text{tribwidth} \cdot \text{tribwidth2} = 3.08 \text{ kip} \]

Conservative and reasonable to assume no backspan loading

\[ \Sigma M_b = (P_u \cdot L_c) - (T \cdot (L - L_c)) \quad \text{equal length of backspan and cantilevered span} \]

Therefore the required tension strap capacity would be 3.1 kip

\[ n := 3 \quad S_{xx, 2\times6} = 7.563 \text{ in}^3 \]
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

Bending: \( F_b := 1000 \text{ psi} \)

\[
M_u := (P_u \cdot L_c) = 4.62 \text{ kip \cdot ft} \quad \phi_b := 0.85 \quad K_{F_b} := 2.54 \quad \lambda := 0.8 \quad C_F := 1
\]

\[
F_b' := F_b \cdot \lambda \cdot \phi_b \cdot K_{F_b} \cdot C_M \cdot C_t \cdot C_i \cdot C_L \cdot C_v \cdot C_{fu} \cdot C_F = 1.986 \text{ ksi}
\]

\[
M'_n := F_b' \cdot n \cdot S_{xx,2x6} = 45.064 \text{ kip \cdot in} \quad \text{OK}
\]

Shear: \( F_v := 175 \text{ psi} \)

\[
V_u := \frac{(P_u) \cdot L}{2} = 4.62 \text{ ft \cdot kip} \quad \text{Need 3 because of shear}
\]

\[
\phi_v := 0.75 \quad K_{F_v} := 2.88 \quad \lambda = 0.8
\]

\[
F_v' := F_v \cdot C_M \cdot C_t \cdot C_i \cdot K_{F_v} \cdot \phi_v \cdot \lambda = 302.4 \text{ psi}
\]

\[
A_v := \frac{2}{3} \cdot n \cdot b \cdot d = 16.5 \text{ in}^2
\]

\[
V'_n := F_v' \cdot A_v = 4.99 \text{ kip} \quad \text{OK}
\]
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

Wall Panels:

Panel 2:

\[ LL = 20 \text{ psf} \]
\[ DL = 20 \text{ psf} \]
\[ trib = 4 \text{ ft} \quad \text{half width of panel 1} \]
\[ width := 24 \text{ in} \quad \text{spacing of studs} \]

\[ P_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib \cdot width = 0.448 \text{ kip} \]

Try 2x4
\[ F_c := 1450 \text{ psi} \]
\[ E_{\text{min}} := 510000 \text{ psi} \]

* Sheathing will brace against weak axis buckling

\[ H := 7 \text{ ft} \]
\[ d := 3.5 \text{ in} \]
\[ b := 1.5 \text{ in} \]

\[ \frac{H}{d} = 24 < 50 \text{ OKAY} \]

\[ C_m := 1 \quad C_i := 1 \quad C_F := 1 \quad C_c := 1 \quad C_p \quad \text{unknown} \quad C_T := 1 \]

\[ K_{f, E_{\text{min}}} := 1.76 \quad \phi_{E_{\text{min}}} := 0.85 \quad K_{f, C} := 2.4 \quad \phi_c := 0.9 \quad \lambda := 0.8 \]

\[
\begin{array}{c|cccccc}
E_{\text{min}}' &=& C_M & C_t & - & - & - & C_i & - & - & C_T & - & 1.76 & 0.85 & - \\
E_{\text{min}}' &=& C_M & C_t & - & - & - & C_i & - & - & C_T & - & 1.76 & 0.85 & - \\
\end{array}
\]

\[ E_{\text{min}}' := E_{\text{min}} \cdot C_m \cdot C_t \cdot C_i \cdot C_T \cdot K_{f, E_{\text{min}}} \cdot \phi_{E_{\text{min}}} = 762.96 \text{ ksi} \]

\[ F_{cE} := \frac{0.822 \cdot E_{\text{min}}'}{d} = 1.089 \text{ ksi} \]

\[ F_{c, \text{star}} := F_c \cdot C_m \cdot C_t \cdot C_F \cdot C_i \cdot K_{F, C} \cdot \phi_c \cdot \lambda = 2.506 \text{ ksi} \]
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

calculating \( C_p \):
\[
c := 0.8 \quad \text{for sawn lumber} \quad \alpha_c := \frac{F_c E}{F_{c,\text{star}}} = 0.435
\]
\[
C_p := \frac{1 + \alpha_c}{2 \cdot c} - \sqrt{\left(\frac{1 + \alpha_c}{2 \cdot c}\right)^2 - \frac{\alpha_c}{c}} = 0.386 \quad \text{Not ideal}
\]
\[
F_c' := F_{c,\text{star}} \cdot C_p = 967.193 \text{ psi}
\]
\[
A_c := b \cdot d = 5.25 \text{ in}^2
\]
\[
P' := A_c \cdot F_c' = 5.078 \text{ kip}
\]
\[
P_u = 0.448 \text{ kip} \quad \text{okay}
\]
Wall Panel 4 Deployed

\[ LL = 20 \text{ psf} \]
\[ DL = 20 \text{ psf} \]
\[ trib := (4 \text{ ft} + 4 \text{ ft} + 6 \text{ ft}) \cdot 10 \text{ ft} = 140 \text{ ft}^2 \]

two halves of panel 1 and all panel 3

\[ P_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 7.84 \text{ kip} \]

Try 2x4
\[ F_c := 1450 \text{ psi} \]
\[ E_{\text{min}} := 510000 \text{ psi} \]

* Sheathing will brace against weak axis buckling

\[ H := 8.5 \text{ ft} \]
\[ d := 3.5 \text{ in} \]
\[ b := 1.5 \text{ in} \]

\[ \frac{H}{d} = 29.143 < 50 \text{ OKAY} \]

\[ C_m := 1 \quad C_t := 1 \quad C_F := 1 \quad C_i := 1 \quad C_p \: \text{ unknown} \quad C_T := 1 \]

\[ K_{f,E_{\text{min}}} := 1.76 \quad \phi_{E_{\text{min}}} := 0.85 \quad K_{f,C} := 2.4 \quad \phi_c := 0.9 \quad \lambda = 0.8 \]

\[ E'_{\text{min}} := E_{\text{min}} \cdot C_m \cdot C_t \cdot C_i \cdot C_T \cdot K_{f,E_{\text{min}}} \cdot \phi_{E_{\text{min}}} = 762.96 \text{ ksi} \]

\[ F_{cE} := 0.822 \cdot E'_{\text{min}} = 0.738 \text{ ksi} \]

\[ F_{c,\text{star}} := F_c \cdot C_m \cdot C_t \cdot C_F \cdot C_i \cdot K_{f,C} \cdot \phi_c \cdot \lambda = 2.506 \text{ ksi} \]

calculating \( Cp \):
\[ c := 0.8 \quad \text{for sawn lumber} \quad \alpha_c := \frac{F_{cE}}{F_{c,\text{star}}} = 0.295 \]

\[ C_p := \frac{1 + \alpha_c}{2 \cdot c} - \sqrt{\left(\frac{1 + \alpha_c}{2 \cdot c}\right)^2 - \frac{\alpha_c}{c}} = 0.274 \quad \text{Not ideal} \]

\[ F' := F_{c,\text{star}} \cdot C_p = 686.598 \text{ psi} \]

\[ A_c := b \cdot d = 5.25 \text{ in}^2 \]
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

\[ P' := A_c \cdot F'_c = 3.605 \text{kip} \quad \text{okay} \]

\[ P'_n := P' \cdot 3 = 10.814 \text{kip} \quad \text{three studs per 3' long panel (12" spacing)} \]

Okay

Wall Panel 4 Folded

\[ LL = 20 \text{ psf} \]
\[ DL = 20 \text{ psf} \]
\[ trib := (8 \text{ ft} + 8 \text{ ft} + 6 \text{ ft}) \cdot 10 \text{ ft} = 220 \text{ ft}^2 \quad \text{two of panel 1 and all panel 3} \]

\[ P_u := (1.4 \cdot DL) \cdot trib = 6.16 \text{kip} \quad \text{Deployed state controls} \]
\[ \quad \text{No live load during deployment} \]
Appendix B. Mathcad Calculations for Gravity Loads: Light-Frame Dynamic Unit

Shear Force to Hinge along roof Panel 1 and 3:

\[
DL := 20 \text{ psf} \\
LL := 20 \text{ psf}
\]

\[
A_t := \frac{8 \text{ ft} \cdot 20 \text{ ft}}{2} = 80 \text{ ft}^2
\]

\[w_{u1} := (1.2 \cdot DL + 1.6 \cdot LL) \cdot A_t = 4.48 \text{ kip}\]

or during deployment:

\[
A_t := 8 \text{ ft} \cdot 20 \text{ ft}
\]

\[w_{u2} := (1.2 \cdot DL) \cdot A_t = 3.84 \text{ kip}\]

\[w_u := \max (w_{u1}, w_{u2}) = 4.48 \text{ kip}\]

\[V_n := 8.373 \text{ klf}\]

\[V_n' := 0.65 \cdot V_n = 5.442 \text{ klf}\]

only need one foot of hinge
Appendix C. Mathcad Calculations for Gravity Loads: CLT Static

Unit
Appendix C. Mathcad Calculations for Gravity Loads: CLT Static Unit

This document contains the structural calculations for the CLT Static Unit.

Dimensions of Each Panel:
- **Roof Panel**: Width = 8', Length = 20'
- **Floor Panel**: Height = 8', Length = 20'
- **Long Wall Panel**: Height = 8' 10", Length = 20'
- **Short Wall Panel**: Height = 8' 10", Width = 4' 6"
- **Medium Wall Panel**: Height = 8' 10", Length = 8' 6"

### SL-V3

<table>
<thead>
<tr>
<th>Lumber Species</th>
<th>Longitudinal Layers: Visually graded No. 2 Southern Pine</th>
<th>Transverse Layers: Visually graded No. 3 Southern Pine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (@ 12% MC)</td>
<td>39 lb/ft³</td>
<td>39 lb/ft³</td>
</tr>
<tr>
<td>Specific Gravity, G</td>
<td>0.55</td>
<td>0.55</td>
</tr>
<tr>
<td>$F_{ci}$</td>
<td>410 psi</td>
<td>410 psi</td>
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</table>

### Allowable Design Properties for SL-V3 Lumber Laminations

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<tr>
<th>Major Strength Direction</th>
<th>Minor Strength Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{b,0}$ (psi)</td>
<td>$E_0$ (10⁶ psi)</td>
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<tr>
<td>---------------------</td>
<td>---------------------</td>
</tr>
<tr>
<td>750</td>
<td>1.4</td>
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</table>

### Southern Pine

#### SL-V3

<table>
<thead>
<tr>
<th>Layup</th>
<th>Thickness (in)</th>
<th>Weight (lbs)</th>
<th>$F_{S,0}$ (lbf/ft²)</th>
<th>$E_{L,0}$ (10⁶ lbf/in²)</th>
<th>$G_{A,0}$ (10⁵ lbf/in²)</th>
<th>$V_{L,0}$ (lbf/ft)</th>
<th>$F_{S,0,90}$ (lbf/ft²)</th>
<th>$E_{L,0,90}$ (10⁶ lbf/in²)</th>
<th>$G_{A,0,90}$ (10⁵ lbf/in²)</th>
<th>$V_{L,0,90}$ (lbf/ft)</th>
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</thead>
<tbody>
<tr>
<td>3-alt</td>
<td>4 1/8</td>
<td>13.5</td>
<td>1,740</td>
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<td>140</td>
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<td>0.52</td>
<td>605</td>
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<tr>
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<td>18.0</td>
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<td>2,420</td>
<td>565</td>
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<td>1,210</td>
</tr>
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<td>6 1/2</td>
<td>22.5</td>
<td>4,000</td>
<td>363</td>
<td>0.98</td>
<td>3,025</td>
<td>1,230</td>
<td>88</td>
<td>1.0</td>
<td>1,820</td>
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<tr>
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<td>4,980</td>
<td>451</td>
<td>1.0</td>
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<td>0.62</td>
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<td>1,820</td>
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<tr>
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<td>4,225</td>
</tr>
<tr>
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<td>40.5</td>
<td>14,130</td>
<td>2,304</td>
<td>2.0</td>
<td>5,450</td>
<td>2,835</td>
<td>338</td>
<td>1.6</td>
<td>3,025</td>
</tr>
</tbody>
</table>
## Appendix C. Mathcad Calculations for Gravity Loads: CLT Static Unit

### Material Properties:

<table>
<thead>
<tr>
<th>Major:</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_b S_{eff} ) := 1740 ( \frac{lb \cdot ft}{ft} )</td>
</tr>
<tr>
<td>Minor:</td>
</tr>
<tr>
<td>( F_b S_{eff.90} ) := 140 ( \frac{lb \cdot ft}{ft} )</td>
</tr>
</tbody>
</table>

\[ SLV3 - 3 - alt \]

\[ thk := 4.125 \text{ in} \]

\[ EI_{eff} := 95 \times 10^6 \frac{lb \cdot in^2}{ft} \]

\[ EI_{eff.90} := 3.4 \times 10^6 \frac{lb \cdot in^2}{ft} \]

\[ GA_{eff} := 0.49 \times 10^6 \frac{lb}{ft} \]

\[ GA_{eff.90} := 0.52 \times 10^6 \frac{lb}{ft} \]

\[ V_s := 1820 \frac{lb}{ft} \]

\[ V_{s.90} := 605 \frac{lb}{ft} \]

### Apparent Bending Stiffness:

\[ K_s := 11.5 \quad \text{Single span} \]

\[ L := 8 \text{ ft} \]

\[ EI_{app} := \frac{EI_{eff}}{1 + \frac{K_s \cdot EI_{eff}}{GA_{eff} \cdot L^2}} \]

\[ EI_{app.90} := \frac{EI_{eff.90}}{1 + \frac{K_s \cdot EI_{eff.90}}{GA_{eff.90} \cdot L^2}} \]

\[ EI_{app} = (7.649 \times 10^7) \frac{lb \cdot in^2}{ft} \]

\[ EI_{app.90} = (3.372 \times 10^6) \frac{lb \cdot in^2}{ft} \]

### Roof Panel:

\[ L := 8 \text{ ft} \]

\[ DL := 23.5 \text{ psf} \quad \text{Dead Load} \]

\[ LL := 20 \text{ psf} \quad \text{Live Load} \]

\[ trib := 1 \text{ ft} \quad \text{per foot basis} \]

\[ w := (DL + LL) \cdot trib = 43.5 \text{ plf} \quad \text{unfactored} \]

\[ w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 60.2 \text{ plf} \quad \text{factored} \]

### Bending:

\[ w_u = 60.2 \text{ plf} \]

\[ M_u := \frac{w_u \cdot L^2}{8} = 481.6 \frac{lb \cdot ft}{8} \]

\[ C_m := 1 \]

\[ C_t := 1 \]

\[ \phi_b := 0.85 \]

\[ \lambda := 0.8 \]

\[ F_b S_{eff} := F_b S_{eff} \cdot K_{fb} \cdot \phi_b \cdot C_m \cdot C_t \cdot C_L = 3005.328 \frac{lb \cdot ft}{ft} \]
Appendix C. Mathcad Calculations for Gravity Loads: CLT Static Unit

Rolling Shear Design:

\[ K_{fv} := 2 \quad \phi_v := 0.75 \]

\[ V'_s := K_{fv} \cdot \phi_v \cdot C_m \cdot C_t \cdot V_s = 2730 \frac{\text{lb}}{\text{ft}} \]

OKAY

Deflection Check:

\[ EI'_{\text{app}} := EI_{\text{app}} \cdot C_m \cdot C_t = (7.649 \cdot 10^7) \frac{1}{\text{ft}} \cdot \frac{\text{lb}}{\text{in}^2} \]

\[ K_c := 2 \]

\[ \Delta_{ST} := \frac{5 \cdot LL \cdot \text{trib} \cdot L^4}{384 \cdot EI'_{\text{app}} \cdot 1 \text{ ft}} = 0.024 \text{ in} \]

\[ \Delta_{LT} := \frac{5 \cdot DL \cdot \text{trib} \cdot L^4}{384 \cdot EI'_{\text{app}} \cdot 1 \text{ ft}} = 0.028 \text{ in} \]

\[ \Delta_T := \Delta_{ST} + K_c \cdot \Delta_{LT} = 0.081 \text{ in} \]

\[ \frac{L}{180} = 0.533 \text{ in} \quad \text{Okay} \]

Floor Panel:

\[ L := 8 \text{ ft} \quad \text{Length of roof joist} \]

\[ DL := 23.5 \text{ psf} \quad \text{Dead Load} \]

\[ LL := 40 \text{ psf} \quad \text{Live Load} \]

\[ \text{trib} := 1 \text{ ft} \quad \text{per foot basis} \]

\[ w := (DL + LL) \cdot \text{trib} = 63.5 \text{ plf} \quad \text{unfactored} \]

\[ w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot \text{trib} = 92.2 \text{ plf} \quad \text{factored} \]

Bending:

\[ w_u = 92.2 \text{ plf} \]

\[ M_u := \frac{w_u \cdot L^2}{8} = 737.6 \text{ lb} \cdot \text{ft} \]

\[ C_m := 1 \]

\[ C_t := 1 \]

\[ C_L := 1 \]

\[ K_{fb} := 2.54 \quad \phi_b := 0.85 \quad \lambda := 0.8 \]

\[ F'_b S_{\text{eff}} := F_b S_{\text{eff}} \cdot \lambda \cdot K_{fb} \cdot \phi_b \cdot C_m \cdot C_t \cdot C_L = 3005.328 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \]

Rolling Shear Design:

\[ K_{fv} := 2 \quad \phi_v := 0.75 \]

\[ V_u := \frac{w_u \cdot L}{2} = 368.8 \text{ lb} \]
Appendix C. Mathcad Calculations for Gravity Loads: CLT Static Unit

\[ V_s := K_{fv} \cdot \phi_v \cdot C_m \cdot C_t \cdot V_s = 2730 \, \text{lbf/ft} \]

OKAY

**Deflection Check:**

\[ EI'_{app} := EI_{app} \cdot C_m \cdot C_t = \left(7.649 \times 10^7\right) \frac{1}{1 \, \text{ft}} \cdot \text{lbf \cdot in}^2 \]

\[ K_{cr} := 2 \]

\[ \Delta_{ST} := \frac{5 \cdot LL \cdot trib \cdot L^4}{384 \cdot EI'_{app} \cdot 1 \, \text{ft}} = 0.048 \, \text{in} \]

\[ \Delta_{LT} := \frac{5 \cdot DL \cdot trib \cdot L^4}{384 \cdot EI'_{app} \cdot 1 \, \text{ft}} = 0.028 \, \text{in} \]

\[ \Delta_T := \Delta_{ST} + K_{cr} \cdot \Delta_{LT} = 0.105 \, \text{in} \]

\[ \frac{L}{180} = 0.533 \, \text{in} \]

Wall Panel:

For gravity design so assuming axial load only:

Loading:

\[ trib := \frac{8 \, \text{ft}}{2} = 4 \, \text{ft} \]

\[ DL := 23.5 \, \text{psf} \]

\[ LL := 20 \, \text{psf} \]

\[ w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 240.8 \, \text{plf} \]

Capacity:

\[ F_c := 1250 \, \text{psi} \]

\[ F' = F_c \cdot C_m \cdot C_t \cdot C_p \]

\[ EI'_{app_{min}} := EI'_{app} \cdot 0.5184 = \left(3.965 \times 10^7\right) \frac{\text{lbf \cdot in}^2}{1 \, \text{ft}} \]

\[ l_c := 8 \, \text{ft} + 10 \, \text{in} \]

\[ P_{ce} := \frac{\pi^2 \cdot EI'_{app_{min}}}{l_c^2} = \left(3.483 \times 10^4\right) \, \text{plf} \]

\[ A_c := 2 \cdot \left(1 + \frac{3}{8}\right) \, \text{in} = 2.75 \, \text{in} \]

leave in per foot of wall length

\[ P_{star} := F_c \cdot C_m \cdot C_t \cdot A_c = \left(4.125 \times 10^4\right) \, \text{plf} \]
Appendix C. Mathcad Calculations for Gravity Loads: CLT Static Unit

\[ \alpha_c := \frac{P_{ce}}{P_{star}} = 0.844 \]

\[ c := 0.9 \]

\[ C_p := \frac{1 + \alpha_c}{2 \cdot c} - \frac{1 + \alpha_c}{2 \cdot c}^2 = 0.69 \]

\[ F'_c := F_c \cdot C_m \cdot C_t \cdot C_p \cdot A_c = 28480.261 \quad \text{plf} \]

Okay
Appendix D. Mathcad Calculations for Gravity Loads: CLT

Dynamic Unit
Appendix D. Mathcad Calculations for Gravity Loads: CLT Dynamic Unit

This document contains the structural calculations for the CLT Dynamic Unit.

Dimensions of Each Panel:

Panel 1:  
Width = 8'  
Length = 20'

Panel 2:  
Height: 7'  
Length = 20'

Panel 3:  
Width = 6'  
Length = 20'

Panel 4:  
Width = 3'  
Height = 8' 6"

Panel 5:  
Width = 9' 3"  
Height = 10'

Panel 6:  
Width = 8'  
Length = 20'

Panel 7:  
Width = 4' 10"  
Length = 20'

* Note: SLV3 used for design

Panels 1, 3, 6, and 7 are bending elements

Panels 2, 4, 5 are compression elements

Going to span Panel 1 and 6 the short direction

**Material Properties:**

**SLV3-3-alt:**  
\[ \text{thk}_3 := 4.125 \text{ in} \]

Major:

\[ F_{b}S_{eff,3} := 1740 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}} \]

\[ EI_{eff,3} := 95 \cdot 10^{6} \frac{\text{lbf} \cdot \text{in}^2}{\text{ft}} \]

\[ GA_{eff,3} := 0.49 \cdot 10^{6} \frac{\text{lbf}}{\text{ft}} \]

\[ V_{s,3} := 1820 \frac{\text{lbf}}{\text{ft}} \]

Minor:

\[ F_{b}S_{eff,90,3} := 140 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}} \]

\[ EI_{eff,90,3} := 3.4 \cdot 10^{6} \frac{\text{lbf} \cdot \text{in}^2}{\text{ft}} \]

\[ GA_{eff,90,3} := 0.52 \cdot 10^{6} \frac{\text{lbf}}{\text{ft}} \]

\[ V_{s,90,3} := 605 \frac{\text{lbf}}{\text{ft}} \]
Appendix D. Mathcad Calculations for Gravity Loads: CLT Dynamic Unit

**SLV3-5-alt:**  
\[ \text{thk}_5 = 5.5 \text{ in} \]

**Major:**  
\[ F_b S_{\text{eff},5} := 4000 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}} \]

\[ E I_{\text{eff},5} := 363 \cdot 10^6 \frac{\text{lbf} \cdot \text{in}^2}{\text{ft}} \]

\[ G A_{\text{eff},5} := 0.98 \cdot 10^6 \frac{\text{lbf}}{\text{ft}} \]

\[ V_{s,5} := 3025 \frac{\text{lbf}}{\text{ft}} \]

**Minor:**  
\[ F_b S_{\text{eff},90.5} := 1230 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}} \]

\[ E I_{\text{eff},90.5} := 88 \cdot 10^6 \frac{\text{lbf} \cdot \text{in}^2}{\text{ft}} \]

\[ G A_{\text{eff},90.5} := 1.0 \cdot 10^6 \frac{\text{lbf}}{\text{ft}} \]

\[ V_{s,90.5} := 1820 \frac{\text{lbf}}{\text{ft}} \]

<table>
<thead>
<tr>
<th>Southern Pine</th>
<th>Layup</th>
<th>Thickness (in)</th>
<th>Weight (psf)</th>
<th>( F_b S_{\text{eff}} ) (lbf-ft/ft)</th>
<th>( E I_{\text{eff}} ) (10^6 lbf-in^2/ft)</th>
<th>( G A_{\text{eff}} ) (10^6 lbf-in^2/ft)</th>
<th>( V_{s,0} ) (lbf/ft)</th>
<th>( F_b S_{\text{eff},90} ) (lbf-ft/ft)</th>
<th>( E I_{\text{eff},90} ) (10^6 lbf-in^2/ft)</th>
<th>( G A_{\text{eff},90} ) (10^6 lbf-in^2/ft)</th>
<th>( V_{s,90} ) (lbf/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-Alt</td>
<td>4 1/8</td>
<td>13.5</td>
<td>1,740</td>
<td>95</td>
<td>0.49</td>
<td>1,820</td>
<td>140</td>
<td>3.4</td>
<td>0.52</td>
<td>605</td>
<td>4 1/8</td>
</tr>
<tr>
<td>4-Maxx</td>
<td>5 1/2</td>
<td>18.0</td>
<td>2,825</td>
<td>205</td>
<td>0.58</td>
<td>2,420</td>
<td>565</td>
<td>27</td>
<td>1.1</td>
<td>1,210</td>
<td>5 1/2</td>
</tr>
<tr>
<td>5-Alt</td>
<td>6 3/8</td>
<td>22.5</td>
<td>4,000</td>
<td>363</td>
<td>0.98</td>
<td>3,025</td>
<td>1,230</td>
<td>88</td>
<td>1.0</td>
<td>1,820</td>
<td>6 3/8</td>
</tr>
<tr>
<td>5-Maxx</td>
<td>6 7/8</td>
<td>22.5</td>
<td>4,980</td>
<td>451</td>
<td>1.0</td>
<td>4,225</td>
<td>1,230</td>
<td>88</td>
<td>1.1</td>
<td>1,820</td>
<td>6 7/8</td>
</tr>
<tr>
<td>7-Alt</td>
<td>9 1/8</td>
<td>31.5</td>
<td>7,100</td>
<td>899</td>
<td>1.5</td>
<td>4,225</td>
<td>2,825</td>
<td>338</td>
<td>1.6</td>
<td>3,025</td>
<td>9 1/8</td>
</tr>
<tr>
<td>7-Maxx</td>
<td>9 7/8</td>
<td>31.5</td>
<td>9,120</td>
<td>1,157</td>
<td>1.5</td>
<td>4,225</td>
<td>1,230</td>
<td>88</td>
<td>1.1</td>
<td>1,820</td>
<td>9 7/8</td>
</tr>
<tr>
<td>9-Alt</td>
<td>12 3/8</td>
<td>40.5</td>
<td>11,000</td>
<td>1,793</td>
<td>2.0</td>
<td>5,450</td>
<td>5,025</td>
<td>837</td>
<td>2.1</td>
<td>4,225</td>
<td>12 3/8</td>
</tr>
<tr>
<td>9-Maxx</td>
<td>12 3/8</td>
<td>40.5</td>
<td>14,130</td>
<td>2,304</td>
<td>2.0</td>
<td>5,450</td>
<td>2,835</td>
<td>338</td>
<td>1.6</td>
<td>3,025</td>
<td>12 3/8</td>
</tr>
</tbody>
</table>

**Apparent Bending Stiffness:**

\[ K_s := 11.5 \]

3-ply Panel 1

\[ L_{3,p1} := 8 \text{ ft} \]

\[ L_{3,p3} := 10 \text{ ft} \]

\[ E I_{\text{app},3,p1} := \frac{E I_{\text{eff},3}}{1 + \frac{K_s \cdot E I_{\text{eff},3}}{G A_{\text{eff},3} \cdot L_{3,p1}^2}} \]

\[ E I_{\text{app},3,p1} = 76494078.462 \frac{\text{lbf} \cdot \text{in}^2}{\text{ft}} \]

\[ E I_{\text{app},90,3,p1} := \frac{E I_{\text{eff},90,3}}{1 + \frac{K_s \cdot E I_{\text{eff},90,3}}{G A_{\text{eff},90,3} \cdot L_{3,p1}^2}} \]

\[ E I_{\text{app},90,3,p1} = 3372484.28 \frac{\text{lbf} \cdot \text{in}^2}{\text{ft}} \]

3-ply Panel 3

\[ E I_{\text{app},3,p3} := \frac{E I_{\text{eff},3}}{1 + \frac{K_s \cdot E I_{\text{eff},3}}{G A_{\text{eff},3} \cdot L_{3,p3}^2}} \]

\[ E I_{\text{app},90,3,p3} := \frac{E I_{\text{eff},90,3}}{1 + \frac{K_s \cdot E I_{\text{eff},90,3}}{G A_{\text{eff},90,3} \cdot L_{3,p3}^2}} \]

Non-Commercial Use Only
## Appendix D. Mathcad Calculations for Gravity Loads: CLT Dynamic Unit

### Roof Panel # 1:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>(L)</td>
<td>8 ft</td>
</tr>
<tr>
<td>(DL)</td>
<td>23.5 psf</td>
</tr>
<tr>
<td>(LL)</td>
<td>20 psf</td>
</tr>
<tr>
<td>(trib)</td>
<td>1 ft</td>
</tr>
<tr>
<td>(w)</td>
<td>((DL + LL) \cdot trib = 43.5) plf</td>
</tr>
<tr>
<td>(w_u)</td>
<td>((1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 60.2) plf</td>
</tr>
</tbody>
</table>

**Dead Load SW 3-Ply = 13.5 psf**

**Live Load**

\[
w = (DL + LL) \cdot trib = 43.5\text{ plf}
\]

unfactored

\[
w_u = (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 60.2\text{ plf}
\]

factored

### Bending:

\[
w_u = 60.2\text{ plf}
\]

\[
M_u := \frac{w_u \cdot L^2}{8} = 481.6\text{ lbf ft}
\]

\[
K_{fb} = 2.54\quad \phi_b = 0.85\quad \lambda = 0.8
\]

\[
F'_{bs_{eff}} := F_b s_{eff,3} \cdot \lambda \cdot K_{fb} \cdot \phi_b \cdot C_m \cdot C_t \cdot C_L = 3005.328\text{ lbf ft}
\]

### Rolling Shear Design:

\[
K_{fv} = 2\quad \phi_v = 0.75
\]

\[
V_u := \frac{w_u \cdot L}{2} = 240.8\text{ lbf}
\]

\[
V_s := K_{fv} \cdot \phi_v \cdot C_m \cdot C_t \cdot V_{s,3} = 2730\text{ lbf ft}
\]

**OKAY**

### Deflection Check:

\[
EI'_{app} := EI'_{app,3,p1} \cdot C_m \cdot C_t = 76494078.462\text{ lbf in}^2\text{ ft}
\]

\[
\Delta_{ST} := \frac{5 \cdot LL \cdot trib \cdot L^4}{384 \cdot EI'_{app} \cdot 1\text{ ft}} = 0.024\text{ in}
\]

\[
\Delta_{LT} := \frac{5 \cdot DL \cdot trib \cdot L^4}{384 \cdot EI'_{app} \cdot 1\text{ ft}} = 0.028\text{ in}
\]

\[
\Delta_T := \Delta_{ST} + K_{cr} \cdot \Delta_{LT} = 0.081\text{ in}
\]

\[
\frac{L}{180} = 0.533\text{ in}\quad \text{Okay}
\]
Appendix D. Mathcad Calculations for Gravity Loads: CLT Dynamic Unit

### Roof Panel # 3:

<table>
<thead>
<tr>
<th></th>
<th>3-ply, 2-span continuous</th>
<th>Two-halves of 3' width each</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L$</td>
<td>10 ft</td>
<td></td>
</tr>
<tr>
<td>$DL$</td>
<td>23.5 psf</td>
<td>Dead Load SW 3-Ply = 13.5 psf</td>
</tr>
<tr>
<td>$LL$</td>
<td>20 psf</td>
<td>Live Load</td>
</tr>
<tr>
<td>$trib$</td>
<td>3 ft</td>
<td>3' width of panel</td>
</tr>
</tbody>
</table>

\[
w_{u,1} := ((1.2 \cdot DL + 1.6 \cdot LL) \cdot 4 \text{ ft}) + (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 421.4 \text{ plf}
\]

\[
w_{u,2} := (1.4 \cdot DL) \cdot 8 \text{ ft} + (1.4 \cdot DL) \cdot trib = 361.9 \text{ plf}
\]

\[
w_{u,3} := ((1.2 \cdot DL + 1.6 \cdot LL) \cdot trib) + (1.4 \cdot DL) \cdot 8 \text{ ft} = 443.8 \text{ plf}
\]

\[
w_u = \max (w_{u,1}, w_{u,2}, w_{u,3}) = 443.8 \text{ plf}
\]

**Bending:**

\[
w_u = 443.8 \text{ plf}
\]

\[
M_{u,1} := \frac{w_u \cdot L^2}{8} = 5547.5 \text{ lbf} \cdot \text{ft}
\]

\[
M_{u,2} := \frac{9 \cdot w_u \cdot L^2}{128} = 3120.469 \text{ lbf} \cdot \text{ft}
\]

\[
M_u := \max (M_{u,1}, M_{u,2}) = 5547.5 \text{ lbf} \cdot \text{ft}
\]

\[
K_{fb} := 2.54 \quad \phi_b := 0.85
\]

\[
F' S_{eff} := F_b S_{eff,3} \cdot \lambda \cdot K_{fb} \cdot \phi_b \cdot C_m \cdot C_t \cdot C_L \cdot 3 \text{ ft} = 9015.984 \text{ lbf} \cdot \text{ft}
\]

**Rolling Shear Design:**

\[
K_{fb} := 2 \quad \phi_u := 0.75
\]

\[
V_u := \frac{5 \cdot w_u \cdot L}{8} = 2773.75 \text{ lbf}
\]

\[
V'_s := K_{fb} \cdot \phi_v \cdot C_m \cdot C_t \cdot V_{s,3} \cdot 3 \text{ ft} = 8190 \text{ lbf}
\]

**OKAY**

**Deflection Check:**

\[
EI'_{app} := EI_{app,3,p} \cdot C_m \cdot C_t = 82262993.189 \text{ lbf} \cdot \text{in}^2 \quad \text{Ker} := 2
\]

\[
\Delta_{ST} := \frac{((LL \cdot trib) + (LL \cdot 4 \text{ ft})) \cdot L^4}{185 \cdot EI'_{app} \cdot 3 \text{ ft}} = 0.053 \text{ in}
\]

\[
\Delta_{LT} := \frac{((DL \cdot trib) + (DL \cdot 4 \text{ ft})) \cdot L^4}{185 \cdot EI'_{app} \cdot 3 \text{ ft}} = 0.062 \text{ in}
\]

\[
\Delta_T := \Delta_{ST} + \text{Ker} \cdot \Delta_{LT} = 0.178 \text{ in} \quad \frac{L}{180} = 0.667 \text{ in} \quad \text{Okay}
\]
Appendix D. Mathcad Calculations for Gravity Loads: CLT Dynamic Unit

Required force in tension strap between wall panel 4 and roof panel 3:

\[
trib := 10 \text{ ft} \\
DL = 23.5 \text{ psf} \\
w_u := (1.4 \cdot DL \cdot 8 \text{ ft} \cdot trib) = 2.632 \text{ kip} \quad \text{along roof panel 1 and 3 connection}
\]

\[
w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot 10 \text{ ft} \cdot (1.5 \text{ ft} + 4 \text{ ft}) = 3.311 \text{ kip}
\]

Tension strap must take 3.5 kip
Appendix D. Mathcad Calculations for Gravity Loads: CLT Dynamic Unit

**Floor Panel # 6:**

$L := 8 \text{ ft}$  
$DL := 23.5 \text{ psf}$  
$LL := 40 \text{ psf}$  
$\text{trib} := 1 \text{ ft}$  

Dead Load SW 3-Ply = 13.5 psf  
Live Load  

$w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot \text{trib} = 92.2 \text{ plf}$

**Bending:**

$w_u = 92.2 \text{ plf}$  
$C_m := 1$  
$C_t := 1$  
$C_L := 1$  
$\lambda := 0.8$

$M_{u.1} := \frac{w_u \cdot L^2}{8} = 737.6 \text{ lbf} \cdot \text{ft}$  
$K_{fb} := 2.54$  
$\phi_b := 0.85$

$F'_b S_{eff} := F_b S_{eff.3} \cdot \lambda \cdot K_{fb} \cdot \phi_b \cdot C_m \cdot C_t \cdot C_L = 3005.328 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$

**Rolling Shear Design:**

$K_{fb} := 2$  
$\phi_v := 0.75$

$V_u := \frac{5 \cdot w_u \cdot L}{8} = 461 \text{ lbf}$

$V_s := K_{fv} \cdot \phi_v \cdot C_m \cdot C_t \cdot V_{s.3} = 2730 \frac{\text{lbf}}{\text{ft}}$

**Deflection Check:**

$EI'_{app} := EI_{app.3,P3} \cdot C_m \cdot C_t = 82262993.189 \frac{1 \text{ ft} \cdot \text{lbf} \cdot \text{in}^2}{\text{ft}}$  
$K_{cr} := 2$

$\Delta_{ST} := \frac{5 \cdot (LL \cdot \text{trib}) \cdot L^4}{384 \cdot EI'_{app} \cdot 1 \text{ ft}} = 0.045 \text{ in}$  
$\Delta_{LT} := \frac{5 \cdot (DL \cdot \text{trib}) \cdot L^4}{384 \cdot EI'_{app} \cdot 1 \text{ ft}} = 0.026 \text{ in}$

$\Delta_T := \Delta_{ST} + K_{cr} \cdot \Delta_{LT} = 0.097 \text{ in}$  
$L = 0.533 \text{ in}$  
Okay
Floor Panel # 7:

\[ L := 5 \text{ ft} \]
\[ DL := 23.5 \text{ psf} \]
\[ LL := 40 \text{ psf} \]
\[ trib := 1 \text{ ft} \]

Dead Load SW 3-Ply = 13.5 psf
Live Load per floor basis

\[ w_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 92.2 \text{ plf} \]

Bending:
\[ w_u = 92.2 \text{ plf} \]
\[ C_m := 1 \]
\[ C_t := 1 \]
\[ C_L := 1 \]
\[ \lambda := 0.8 \]
\[ K_{fb} := 2.54 \]
\[ \phi_b := 0.85 \]
\[ F'_b S_{eff} := F_b S_{eff,3} \cdot \lambda \cdot K_{fb} \cdot \phi_b \cdot C_m \cdot C_t \cdot C_L = 3005.328 \frac{lb \cdot ft}{ft} \]

Rolling Shear Design:
\[ K_{fv} := 2 \]
\[ \phi_v := 0.75 \]
\[ V_u := \frac{5 \cdot w_u \cdot L}{8} = 288.125 \text{ lbf} \]
\[ V'_s := K_{fv} \cdot \phi_v \cdot C_m \cdot C_t \cdot V_{s,3} = 2730 \frac{lb \cdot ft}{ft} \]

OKAY

Deflection Check:
\[ EI'_{app} := EI'_{app,3,3} \cdot C_m \cdot C_t = 82262993.189 \frac{1 \text{ ft} \cdot lb \cdot in^2}{\text{ft}} \]
\[ K_{cr} := 2 \]
\[ \Delta_{ST} := \frac{5 \cdot (LL \cdot trib) \cdot L^4}{384 \cdot EI'_{app} \cdot 1 \text{ ft}} = 0.007 \text{ in} \]
\[ \Delta_{LT} := \frac{5 \cdot (DL \cdot trib) \cdot L^4}{384 \cdot EI'_{app} \cdot 1 \text{ ft}} = 0.004 \text{ in} \]
\[ \Delta_T := \Delta_{ST} + K_{cr} \cdot \Delta_{LT} = 0.015 \text{ in} \]
\[ \frac{L}{180} = 0.333 \text{ in} \]

Okay
Appendix D. Mathcad Calculations for Gravity Loads: CLT Dynamic Unit

**Wall Panel 2:**

For gravity design so assuming axial load only:

**Loading:**

\[ trib := \frac{8 \text{ ft}}{2} = 4 \text{ ft} \]

\[ DL := 23.5 \text{ psf} \]

\[ LL := 20 \text{ psf} \]

\[ w_\text{u} := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 240.8 \text{ plf} \]

**Capacity:**

\[ F_c := 1250 \text{ psi} \]

\[ F' := F_c \cdot C_m \cdot C_t \cdot C_p \]

\[ EI'_{\text{app, min}} := EI'_{\text{app}} \cdot 0.5184 = 42645135.669 \text{ lbf \cdot in}^2 \text{ ft} \]

\[ l_e := 7 \text{ ft} \]

\[ P_{ce} := \frac{P_{ce}^2}{EI'_{\text{app, min}}} = 59650.031 \text{ plf} \]

\[ A_c := 2 \left( 1 + \frac{3}{8} \right) \text{ in} = 2.75 \text{ in} \]

leave in per foot of wall length

\[ P_{star} := F_c \cdot C_m \cdot C_t \cdot A_c = 41250 \text{ plf} \]

\[ \alpha_c := P_{ce} \cdot P_{star} = 1.446 \]

\[ c := 0.9 \]

\[ C_p := \frac{1 + \alpha_c}{2 \cdot c} - \left( \frac{1 + \alpha_c}{2 \cdot c} \right)^2 - \frac{\alpha_c}{c} = 0.869 \]

\[ F' := F_c \cdot C_m \cdot C_t \cdot C_p \cdot A_c = 35849.943 \text{ plf} \]

Okay

---

2.2.2 Column Stability Factor, \( C_p \)

The column stability factor accounts for tendency of a column to buckle. Since CLT is a plate element, buckling only needs to be checked in the out-of-plane direction. Derived from the NDS, the formula for the column stability factor for CLT is as follows:

\[ C_p = \frac{1 + \frac{P_p}{P_e}}{2} - \frac{1 + \frac{P_p}{P_e}}{2} \cdot \frac{P_p}{P_e} \]

where:

\[ P_e := \text{Composite compression design capacity} \]

\[ P_p := \text{Applicable adjustment factors except } C_p \]

\[ P_p := \frac{E'_{\text{app, min}}}{I} \]

(see Section 2.2.3).

2.2.3 Minimum Apparent Bending Stiffness, \( E_{\text{app}} \)

The apparent bending stiffness, \( E_{\text{app}} \), should be determined using Equation [9]. The following equation can be used to adjust the average \( E_{\text{app}} \) to a minimum value, \( E_{\text{app, min}} \), for use in column buckling design:

\[ E_{\text{app, min}} = 0.5184 E_{\text{app}} \]
Wall Panel 4:

For gravity design so assuming axial load only:

\[
trib := (4 \text{ ft} + 6 \text{ ft} + 4 \text{ ft}) \cdot 10 \text{ ft} = 140 \text{ ft}^2
\]

\[
DL := 23.5 \text{ psf}
\]

\[
LL := 20 \text{ psf}
\]

\[
P_u := (1.2 \cdot DL + 1.6 \cdot LL) \cdot trib = 8.428 \text{ kip}
\]

Capacity:

\[
F_c := 1250 \text{ psi}
\]

\[
F'_c = F_c \cdot C_m \cdot C_t \cdot C_p
\]

\[
EI'_{app \_min} := EI'_{app} \cdot 0.5184 = 42645135.669 \text{ lbf \cdot in}^2 / \text{ ft} \quad l_c := 8 \text{ ft} + 6 \text{ in}
\]

\[
P_{ce} := \frac{\pi^2 \cdot EI'_{app \_min}}{l_c^2} = 40454.692 \text{ plf}
\]

\[
A_c := 2 \cdot \left(1 + \frac{3}{8}\right) \text{ in} = 2.75 \text{ in} \quad \text{leave in per foot of wall length}
\]

\[
P_{star} := F_c \cdot C_m \cdot C_t \cdot A_c = 41250 \text{ plf}
\]

\[
\alpha_c := \frac{P_{ce}}{P_{star}} = 0.981
\]

\[
c := 0.9
\]

\[
C_p := \frac{1 + \alpha_c}{2 \cdot c} - \sqrt{\left(\frac{1 + \alpha_c}{2 \cdot c}\right)^2 - \frac{\alpha_c}{c}} = 0.752
\]

\[
F'_{c} := F_c \cdot C_m \cdot C_t \cdot c \cdot P_{star} = 93.094 \text{ kip}
\]

Okay
Appendix D. Mathcad Calculations for Gravity Loads: CLT Dynamic Unit

Shear Force to Hinge along roof Panel 1 and 3:

\[ DL := 23.5 \text{ psf} \]
\[ LL := 20 \text{ psf} \]

\[ A_t := \frac{8 \text{ ft} \cdot 20 \text{ ft}}{2} = 80 \text{ ft}^2 \]

\[ w_{u1} := (1.2 \cdot DL + 1.6 \cdot LL) \cdot A_t = 4.816 \text{ kip} \]

or during deployment:

\[ A_t := 8 \text{ ft} \cdot 20 \text{ ft} \]

\[ w_{u2} := (1.2 \cdot DL) \cdot A_t = 4.512 \text{ kip} \]

\[ w_u := \max(w_{u1}, w_{u2}) = 4.816 \text{ kip} \]

\[ V_n := 8.373 \text{ klf} \]

\[ V'_n := 0.65 \cdot V_n = 5.442 \text{ klf} \]

only need one foot of hinge
Appendix E. Mathcad Calculations for Wind Loads: Static Units
Appendix E. Mathcad Calculations for Wind Loads: Static Units

115 MPH:
Assumptions:
- Partially Open enclosure classification
- 3:12 Roof Pitch

Using Chapter 26 and 27 of ASCE 7-16

| Step 1: Determine Risk Category of building; see Table 1.5-1. |
| Step 2: Determine the basic wind speed, \( V \), for the applicable Risk Category; see Figs. 26.5-1 and 26.5-2. |
| Step 3: Determine wind load parameters: |
  | Wind directionality factor, \( K_d \); see Section 26.6 and Table 26.6-1. |
  | Exposure category; see Section 26.7. |
  | Topographic factor, \( K_t \); see Section 26.8 and Table in Fig. 26.8-1. |
  | Ground elevation factor, \( K_e \); see Section 26.9 |
  | Gust-effect factor, \( G \) or \( G_e \); see Section 26.11. |
  | Enclosure classification; see Section 26.12. |
  | Internal pressure coefficient, (GC_p); see Section 26.13 and Table 26.13-1. |
| Step 4: Determine velocity pressure exposure coefficient, \( K_z \); see Table 26.10-1. |
| Step 5: Determine velocity pressure \( q_z \) or \( q_z \), Eq. (26.10-1). |
| Step 6: Determine external pressure coefficient, \( C_p \) or \( C_k \): |
  | Fig. 27.3-1 for walls and flat, gable, hip, monoslope, or mansard roofs. |
  | Fig. 27.3-2 for domed roofs. |
  | Fig. 27.3-3 for arched roofs. |
  | Fig. 27.3-4 for monoslope roof, open building. |
  | Fig. 27.3-5 for pitched roof, open building. |
  | Fig. 27.3-6 for trenched roof, open building. |
  | Fig. 27.3-7 for along-ridge/valley wind load case for monoslope, pitched, or trenched roof, open building. |
| Step 7: Calculate wind pressure, \( p \), on each building surface: |
  | Eq. (27.3-1) for rigid and flexible buildings. |
  | Eq. (27.3-2) for open buildings. |

**Notation**

- \( B \): Horizontal dimension of building, in ft (m), measured normal to wind direction.
- \( L \): Horizontal dimension of building, in ft (m), measured parallel to wind direction.
- \( h \): Mean roof height, in ft (m), except that eave height shall be used for \( 0 \leq 10 \) degrees.
- \( z \): Height above ground, in ft (m).
- \( G \): Gust-effect factor.
- \( q_z \): Velocity pressure, in lb/ft^2 (N/m^2) evaluated at respective height.
- \( \theta \): Angle of plane of roof from horizontal, in degrees.

Step 1: Risk Category II
Step 2: Basic Wind Speed = 115 mph \( V := 115 \)
Step 3:
- Wind directionality \( K_d := 0.85 \)
- Exposure Category B
- Topographic factor \( K_t := 1 \)
- Ground elevation factor \( K_e := 1 \)
- Gust-effect factor \( G := 0.85 \)
- Enclosure classification partially open
- Internal pressure coefficient \( GC_{pi} := 0.18 \)
Step 4: Velocity pressure exposure coefficient \( K_z := 0.57 \)
Step 5: Velocity pressure \( q_z := (0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2) \cdot psf = 16.403 \text{ psf} \)
Use: \( q_z := 16.5 \text{ psf} \) Everywhere
Step 6: External pressure coefficients
Appendix E. Mathcad Calculations for Wind Loads: Static Units

**Wind Perpendicular to Ridge:**

\[ L := 8 \text{ ft} \quad h := 9.5 \text{ ft} \]
\[ B := 20 \text{ ft} \]
\[ \frac{L}{B} = 0.4 \]

Wall Pressure Coefficients:

<table>
<thead>
<tr>
<th>Surface</th>
<th>( L/B )</th>
<th>( C_p )</th>
<th>Use With</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>All values</td>
<td>0.8</td>
<td>( q_z )</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>0-1</td>
<td>-0.5</td>
<td>( q_z )</td>
</tr>
<tr>
<td>Sidewall</td>
<td>( \geq 2 )</td>
<td>-0.7</td>
<td>( q_z )</td>
</tr>
</tbody>
</table>

Wall Pressure Coefficients, \( C_p \) for use with \( q_z \):

<table>
<thead>
<tr>
<th>Wind Direction</th>
<th>Angle (degrees)</th>
<th>( L/B )</th>
<th>( C_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>( \leq 25 )</td>
<td>0.5</td>
<td>-0.2</td>
</tr>
<tr>
<td>( \leq 25 )</td>
<td>( \leq 15 )</td>
<td>0.5</td>
<td>-0.2</td>
</tr>
<tr>
<td>( \leq 15 )</td>
<td>( \leq 10 )</td>
<td>0.5</td>
<td>-0.2</td>
</tr>
<tr>
<td>( \leq 10 )</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Roof Pressure Coefficients, \( C_p \) for use with \( q_z \):

<table>
<thead>
<tr>
<th>Wind Direction</th>
<th>Angle (degrees)</th>
<th>( L )</th>
<th>( h )</th>
<th>( C_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal to Ridge for ( \leq 25 )</td>
<td>( \leq 15 )</td>
<td>0 to ( \frac{h}{2} )</td>
<td>0 to ( \frac{h}{2} )</td>
<td></td>
</tr>
<tr>
<td>Normal to Ridge for ( \leq 25 )</td>
<td>( \leq 15 )</td>
<td>0 to ( \frac{h}{2} )</td>
<td>0 to ( \frac{h}{2} )</td>
<td></td>
</tr>
<tr>
<td>Normal to Ridge for ( \leq 25 )</td>
<td>( \leq 15 )</td>
<td>0 to ( \frac{h}{2} )</td>
<td>0 to ( \frac{h}{2} )</td>
<td></td>
</tr>
</tbody>
</table>

\( C_p \) for roof slopes greater than 45°, use \( C_p = 0.8 \).

Roof Pressure Coefficients:

\[ \frac{h}{L} = 1.188 \]
\[ C_{p_{r\_perp}} = -1.3 \]

Windward Wall \( C_{p_{ww}} = 0.8 \)
Leeward Wall \( C_{p_{lw\_perp}} = -0.5 \)
Sidewall \( C_{p_{sw}} = -0.7 \)

Wind pressure:

\[ P_{\text{ww} \_\text{ext}} := q_z \cdot G \cdot C_{p_{ww}} = 11.22 \text{ psf} \]
\[ P_{\text{lw} \_\text{ext}} := q_z \cdot G \cdot C_{p_{lw\_perp}} = -7.013 \text{ psf} \]
\[ P_{\text{r}\_\text{ext}} := q_z \cdot G \cdot C_{p_{r\_perp}} = -18.233 \text{ psf} \]
\[ P_{\text{int}} := q_z \cdot G C_{p_{i}} = 2.97 \text{ psf} \]

Roof pressure:

\[ P_r := P_{\text{r}\_\text{ext}} - 1 \cdot P_{\text{int}} = -21.203 \text{ psf} \]
Appendix E. Mathcad Calculations for WindLoads: Static Units

\textbf{Forces:}

\textbf{Windward Wall:}

\[ P_{ww\text{-}ext} = 11.22 \text{ psf} \]

\[ F_{ww} = P_{ww\text{-}ext} \cdot \left( \frac{h}{2} \right) = 53.295 \text{ plf} \quad \text{Line load along top and bottom of wall} \]

\textbf{Leeward Wall:}

\[ P_{lw\text{-}ext} = -7.013 \text{ psf} \]

\[ F_{lw} := (P_{lw\text{-}ext} - P_{\text{int}}) \cdot \left( \frac{h}{2} \right) = -47.417 \text{ plf} \quad \text{Line load along top and bottom of wall} \]

\textbf{Roof:}

\[ P_r = -21.203 \text{ psf} \]

\textbf{x-comp:}

\[ P_{rx} := P_r \cdot \sin(14 \deg) = -5.129 \text{ psf} \]

\textbf{y-comp:}

\[ P_{ry} := P_r \cdot \cos(14 \deg) = -20.573 \text{ psf} \]

Each shear wall Perp to ridge

\[ F_{\text{top}} := \frac{1}{2} (F_{ww} - F_{lw}) \cdot \left( \frac{B}{2} \right) = 1.417 \text{ kip} \]

\[ F_{\text{bot}} := \frac{1}{2} (F_{ww} - F_{lw}) \cdot \left( \frac{B}{2} \right) = 1.007 \text{ kip} \]

\[ F_{r\text{-}y} = P_{ry} \cdot \left( \frac{B}{2} \right) = -205.727 \text{ plf} \]

\textbf{Shear at base of wall:}

\[ V_b := F_{\text{top}} + F_{\text{bot}} = 2.425 \text{ kip} \]
Load Case:

\[1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R)\]  

(EQ 4.3.2-4)

Overturning Moments:

- Overturning moment from Ftop:
  \[M_{F,\text{top}} := F_{\text{top}} \cdot h = 13.466 \text{ kip} \cdot \text{ft} \]

- Overturning moment from Roof Uplift:
  \[M_{F,\text{up}} := -F_{r_y} \cdot L \cdot \frac{L}{2} = 6.583 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:

- Resisting moment:
  \[M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 15.424 \text{ kip} \cdot \text{ft} \]

Required force in hold down:

\[T := \frac{M_{F,\text{top}} + M_{F,\text{up}} - M_r}{L - 1 \text{ ft}} = 0.661 \text{ kip} \]

Required Compression Force:

\[C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 4.517 \text{ kip} \]

Required Studs for compression:

- CLT Capacity:
  \[F' = 35849 \text{ plf} = 35.849 \text{ klf} \]
  Okay

- Light-Frame Capacity:
  \[P' = 5 \text{ kip} \]

\[C = 0.903 \]

Need at least 2 studs in corner for compression

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[\phi = 0.65 \text{ Connections} \]

\[V'_n = 3.85 \text{ klf} \]

For Base shear we would need:

\[\frac{V}{V'_n} = 0.03 \frac{1}{\text{lbf} \cdot \text{ft}} \text{ of Hinge along wall} \]
# Appendix E. Mathcad Calculations for Wind Loads: Static Units

### Load Case:

\[ 0.9 \times DL + W \quad (EQ \ 4.3.2-5) \]

### Overturning Moments:

- Overturning moment from Ftop:
  \[ M_{F,\text{top}} := F_{\text{top}} \cdot h = 13.466 \text{ kip} \cdot \text{ft} \]
- Overturning moment from Roof Uplift:
  \[ M_{F,\text{up}} := -F_{r,y} \cdot L \cdot \frac{L}{2} = 6.583 \text{ kip} \cdot \text{ft} \]

### Required force in hold down:

\[ T := \frac{M_{F,\text{top}} + M_{F,\text{up}}}{L - 1 \text{ ft}} = 2.864 \text{ kip} \]

### Required Compression Force:

\[ C := T = 2.864 \text{ kip} \]

### Required Studs for compression:

- CLT Capacity:
  \[ F' := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]
- Light-Frame Capacity:
  \[ P' := 5 \text{ kip} \]

\[ \frac{C}{P'} = 0.573 \quad \text{Need at least 1 studs in corner for compression} \]

### Required hinge length for base shear:

- Ultimate Strength of Hinge A in Shear
  \[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 0.03 \cdot \frac{1}{lbf} \cdot \text{ft} \quad \text{of Hinge along wall} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

<table>
<thead>
<tr>
<th>Wood Sheathing Nailing to Resist Shear: (LF UNITS)</th>
<th>( \phi_D := 0.8 )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Shearwall:</strong></td>
<td></td>
</tr>
<tr>
<td>( V := \frac{F_{top}}{L \cdot \phi_D} = 221.479 \text{ plf} )</td>
<td></td>
</tr>
<tr>
<td>( V_w := 715 \text{ plf} )</td>
<td></td>
</tr>
<tr>
<td>Use 8d nails with 6&quot; spacing along edge</td>
<td></td>
</tr>
<tr>
<td><strong>Diaphragm:</strong></td>
<td></td>
</tr>
<tr>
<td>( V_d := V = 221.479 \text{ plf} )</td>
<td></td>
</tr>
<tr>
<td>( V_{wd} := 670 \text{ plf} ) okay</td>
<td></td>
</tr>
</tbody>
</table>
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Wind Parallel To Ridge:
\[ L := 20 \text{ ft} \quad h := 9.5 \text{ ft} \]
\[ B := 8 \text{ ft} \]
\[ \frac{L}{B} = 2.5 \]

Wall Pressure Coefficients:

<table>
<thead>
<tr>
<th>Surface</th>
<th>Use With</th>
<th>( C_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>All values</td>
<td>0.8</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>All values</td>
<td>0.7</td>
</tr>
<tr>
<td>Sidewall</td>
<td>All values</td>
<td>-0.3</td>
</tr>
</tbody>
</table>

Wall Pressure Coefficients, \( C_p \) for use with \( q_a \):

Windward Wall  
Leeward Wall  
Sidewall

Roof Pressure Coefficients:

<table>
<thead>
<tr>
<th>Surface</th>
<th>Use With</th>
<th>( C_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal to Ridge for ( \theta &lt; 10^\circ )</td>
<td>All values</td>
<td>0.475</td>
</tr>
<tr>
<td>to Ridge for ( \theta &gt; 10^\circ )</td>
<td>All values</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Roof pressure:
\[ P_r := P_{r, ext} + -1 \cdot P_{int} = -15.593 \text{ psf} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Forces:

Windward Wall:

\[ P_{ww_{ext}} = 11.22 \text{ psf} \]

\[ F_{ww} = P_{ww_{ext}} \cdot \left( \frac{h}{2} \right) = 53.295 \text{ plf} \]

Line load along top and bottom of wall

Leeward Wall:

\[ P_{lw_{ext}} = -4.208 \text{ psf} \]

\[ F_{lw} = (P_{lw_{ext}} - P_{int}) \cdot \left( \frac{h}{2} \right) = -34.093 \text{ plf} \]

Line load along top and bottom of wall

Roof:

\[ P_{ry} = -20.573 \text{ psf} \]

Each shear wall Par to ridge

- Half of top of wall (ww & lw) line load

\[ F_{top} := (F_{ww} - F_{lw}) \cdot \left( \frac{B}{2} \right) = 0.35 \text{ kip} \]

- Half of bot of wall (ww & lw) line load

\[ F_{bot} := (F_{ww} - F_{lw}) \cdot \left( \frac{B}{2} \right) = 0.35 \text{ kip} \]

- Uplift line load

\[ F_{r,y} := P_{ry} \cdot \left( \frac{B}{2} \right) = -82.291 \text{ plf} \]

Shear at base of wall:

\[ V_{b} := F_{top} + F_{bot} = 0.699 \text{ kip} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Load Case:

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \]  
(EQ 4.3.2-4)

Overturning Moments:

\[ \begin{align*} 
L &= 20 \text{ ft} \\
B &= 8 \text{ ft} \\
DL_r &= 23.5 \text{ psf} \\
LL_r &= 20 \text{ psf} \\
Trib &= \frac{B}{2} 
\end{align*} \]

Overturning moment from Ftop:

\[ M_{F,\text{top}} = F_{\text{top}} \cdot h = 3.321 \text{ kip } \cdot \text{ft} \]

Overturning moment from Roof Uplift:

\[ M_{F,\text{up}} = -F_{r,y} \cdot L \cdot \frac{L}{2} = 16.458 \text{ kip } \cdot \text{ft} \]

Resisting moment from Dead and Live:

\[ M_r = (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 38.56 \text{ kip } \cdot \text{ft} \]

**Required force in hold down:**

\[ T = \frac{M_{F,\text{top}} + M_{F,\text{up}} - M_r}{L - 1 \text{ ft}} = -0.988 \text{ kip} \]

**Required Compression Force:**

\[ C = T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 2.868 \text{ kip} \]

**Required Studs for compression:**

CLT Capacity:
\[ F'_c = 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

Light-Frame Capacity:
\[ P' = 5 \text{ kip} \]

\[ \frac{C}{P'} = 0.574 \quad \text{Need at least 1 studs in corner for compression} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear
\[ \phi = 0.65 \quad \text{Connections} \]

\[ V'_n = 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 0.058 \text{ \frac{1}{ft} } \text{ of Hinge along wall} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Load Case:

\[ 0.9 \times DL + W \]  
\[ (EQ \ 4.3.2-5) \]

Overturning Moments:

\[ L = 20 \ ft \]
\[ B = 8 \ ft \]
\[ DL_r := 23.5 \ psf \]
\[ LL_r := 20 \ psf \]
\[ Trib := \frac{B}{2} \]

Overturning moment from Ftop:

\[ MF_{top} := F_{top} \cdot h = 3.321 \ kip \cdot ft \]

Overturning moment from Roof Uplift:

\[ MF_{up} := -F_{r_{y}} \cdot L \cdot \frac{L}{2} = 16.458 \ kip \cdot ft \]

**Required force in hold down:**

\[ T := \frac{MF_{top} + MF_{up}}{L - 1 \ ft} = 1.041 \ kip \]
\[ MF_{top} + MF_{up} = 19.779 \ kip \cdot ft \]

**Required Compression Force:**

\[ C := T = 1.041 \ kip \]

**Required Studs for compression:**

CLT Capacity:

\[ F'_{c} := 35849 \ plf = 35.849 \ klf \]

Light-Frame Capacity:

\[ P' := 5 \ kip \]

\[ C = 0.208 \]

Need at least 1 studs in corner for compression

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \]

\[ V'_{n} := 3.85 \ klf \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 0.058 \frac{1}{ft} \] of Hinge along wall
Appendix E. Mathcad Calculations for Wind Loads: Static Units

<table>
<thead>
<tr>
<th>Wood Sheathing Nailing to Resist Shear: (LF UNITS)</th>
<th></th>
<th>( \phi_D := 0.8 )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Shearwall:</strong></td>
<td>( V := \frac{F_{top}}{L \cdot \phi_D} = 21.847 \text{ plf} )</td>
<td></td>
</tr>
<tr>
<td>Sheathing:</td>
<td>( V := \frac{F_{top}}{L \cdot \phi_D} = 21.847 \text{ plf} )</td>
<td></td>
</tr>
<tr>
<td>Structural 1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7/16&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>24/16 Span Rating</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( V_w := 715 \text{ plf} )</td>
<td></td>
<td>Use 8d nails with 6&quot; spacing along edge</td>
</tr>
<tr>
<td><strong>Diaphragm:</strong></td>
<td>( V_d := V = 21.847 \text{ plf} )</td>
<td></td>
</tr>
<tr>
<td>( V_{wd} := 670 \text{ plf} ) okay</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Appendix E. Mathcad Calculations for Wind Loads: Static Units

**Components and Cladding (C&C):**

**Table 30.3-1 Steps to Determine C&C Wind Loads for Enclosed and Partially Enclosed Low-Rise Buildings**

1. **Step 1:** Risk Category II
2. **Step 2:** Basic Wind Speed = 115 mph  \( V := 115 \)
3. **Step 3:**
   - Wind Directionality  \( K_d := 0.85 \)
   - Exposure Category B
   - Topographic Factor  \( K_{zt} := 1 \)
   - Ground Elevation Factor  \( K_e := 1 \)
   - Enclosure Classification Partially Open
   - Internal Pressure Coefficient  \( GC_{pi} := 0.18 \)
4. **Step 4:** Velocity pressure exposure coefficient
5. **Step 5:** Velocity pressure
6. **Step 6:** External pressure coefficient

**Walls:**

**Diagram**

![Elevation Diagram](image)

**Notation**

- \( a := 0.1 \) min of least horizontal dimension or 0.46, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
- \( h \) = Mean roof height, in ft (m), except that raved height shall be used for 0 ≤ 10°.
- \( \theta \) = Angle of plane of roof from horizontal, in degrees.

\[
\begin{align*}
a & := \min (0.1 \cdot \min (B, L), 0.4 \cdot h_{\text{mean}}) = 0.8 \text{ ft} \\
a_{\text{min}} & := \max (0.04 \cdot \min (B, L), 3 \text{ ft}) = 3 \text{ ft} \\
a & := \max (a, a_{\text{min}}) = 3 \text{ ft} \\
\theta & := 3 \text{ deg}
\end{align*}
\]

**Perpendicular to Ridge:**

\[
\begin{align*}
A_{5_{\text{perp}}} & := a \cdot z = 31.5 \text{ ft}^2 \\
A_{4_{\text{perp}}} & := (L - 2 \cdot a) \cdot z = 147 \text{ ft}^2
\end{align*}
\]

**Parallel to Ridge:**

\[
\begin{align*}
A_{5_{\text{par}}} & := a \cdot z = 31.5 \text{ ft}^2 \\
A_{4_{\text{par}}} & := (B - 2 \cdot a) \cdot z = 21 \text{ ft}^2
\end{align*}
\]

\( h := 9.5 \text{ ft} \)

\( z := 10.5 \text{ ft} \)

\( B := 8 \text{ ft} \)

\( L := 20 \text{ ft} \)

\( h_{\text{mean}} := \frac{h + z}{2} = 10 \text{ ft} \)
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Positive:

\[ GC_{p,5\_pos} := 1 \]
\[ GC_{p,4\_pos} := 1 \]

Negative:

\[ GC_{p,5\_neg} := -1.4 \]
\[ GC_{p,4\_neg} := -1.1 \]

Design Pressures:

\[ p_{5\_pos} := q_h \cdot (GC_{p,5\_pos} + GC_{pi}) = 19.47 \text{ psf} \]
\[ p_{4\_pos} := q_h \cdot (GC_{p,4\_pos} + GC_{pi}) = 19.47 \text{ psf} \]
\[ p_{5\_neg} := q_h \cdot (GC_{p,5\_neg} - GC_{pi}) = -26.07 \text{ psf} \]
\[ p_{4\_neg} := q_h \cdot (GC_{p,4\_neg} - GC_{pi}) = -21.12 \text{ psf} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

**Roof:**

**Diagrams**

**Notation**
- \( a \) = 10% of least horizontal dimension or 0.4\( h \), whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
- \( h \) = Mean roof height, in ft (m).
- \( W \) = Building width, in ft (m).
- \( \theta \) = Angle of plane of roof from horizontal, in degrees.

**External Pressure Coefficients**

**Notes**
1. Vertical scale denotes \((GC_p)\) to be used with \( q_h \).
2. Horizontal scale denotes effective wind area \( A \), in \( \text{ft}^2 \) (\( \text{m}^2 \)).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
Appendix E. Mathcad Calculations for Wind Loads: Static Units

<table>
<thead>
<tr>
<th>Area:</th>
<th>( B = 8 \text{ ft} )</th>
<th>( L = 20 \text{ ft} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A_3 := (2 \cdot a) \cdot (4 \cdot a) = 72 \text{ ft}^2 )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( A_2 := 2 \cdot (B - (2 \cdot a)) \cdot a + (L \cdot a) = 72 \text{ ft}^2 )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( A_1 := 0 \text{ ft}^2 )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Positive:
\( GC_{p,3_{\text{pos}}} := 0.4 \)
\( GC_{p,2_{\text{pos}}} := 0.4 \)

Negative:
\( GC_{p,3_{\text{neg}}} := -2.2 \)
\( GC_{p,2_{\text{neg}}} := -1.6 \)

Design Pressures:
\( p_{3_{\text{pos}}} := q_h \cdot (GC_{p,3_{\text{pos}}} + GC_{pi}) = 9.57 \text{ psf} \)
\( p_{2_{\text{pos}}} := q_h \cdot (GC_{p,2_{\text{pos}}} + GC_{pi}) = 9.57 \text{ psf} \)

\( p_{3_{\text{neg}}} := q_h \cdot (GC_{p,3_{\text{neg}}} + GC_{pi}) = -33.33 \text{ psf} \)
\( p_{2_{\text{neg}}} := q_h \cdot (GC_{p,2_{\text{neg}}} + GC_{pi}) = -23.43 \text{ psf} \)
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Sheathing Design: (For Light-Frame Unit)

Wall:

Bending Resistance:
- 24" spacing of joists and studs
- 24/16 span rating 7/16" thick

\[ p_{\text{max}} := \max (p_{5 \text{ pos}}, p_{4 \text{ pos}}, p_{5 \text{ neg}}, p_{4 \text{ neg}}) = 19.47 \text{ psf} \]
\[ p_{\text{min}} := \min (p_{5 \text{ pos}}, p_{4 \text{ pos}}, p_{5 \text{ neg}}, p_{4 \text{ neg}}) = -26.07 \text{ psf} \]

<table>
<thead>
<tr>
<th>Sheathing Type</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in.)</th>
<th>Perpendicular to Supports</th>
<th>Parallel to Supports</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Maximum Stud Spacing (in.)</td>
<td>Actual Stud Spacing (in.)</td>
</tr>
<tr>
<td>Wood Structural Panels</td>
<td>24/0</td>
<td>3/8</td>
<td>24</td>
<td>425</td>
</tr>
<tr>
<td>(Sheathing Grades, C-C, C-D, C-C Plugged, OSB)</td>
<td>24/16</td>
<td>7/16</td>
<td>24</td>
<td>540</td>
</tr>
<tr>
<td></td>
<td>32/16</td>
<td>15/32</td>
<td>24</td>
<td>625</td>
</tr>
<tr>
<td></td>
<td>40/20</td>
<td>19/32</td>
<td>24</td>
<td>955</td>
</tr>
<tr>
<td></td>
<td>48/24</td>
<td>23/32</td>
<td>24</td>
<td>1160</td>
</tr>
<tr>
<td>Particleboard Sheathing</td>
<td>3/8</td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(M-S Exterior Glue)</td>
<td>1/2</td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Particleboard Panel Siding</td>
<td>5/8</td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(M-S Exterior Glue)</td>
<td>3/4</td>
<td>24</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hardboard Siding (Direct to</td>
<td>7/16</td>
<td>16</td>
<td>24</td>
<td>460</td>
</tr>
<tr>
<td>Studs)</td>
<td>Lap Siding</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shiplap Edge Panel</td>
<td>7/16</td>
<td>24</td>
<td>460</td>
</tr>
<tr>
<td></td>
<td>Square Edge Panel</td>
<td>7/16</td>
<td>24</td>
<td>460</td>
</tr>
<tr>
<td>Cellulosic Fiberboard Sheathing</td>
<td>Regular</td>
<td>1/2</td>
<td>16</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td>Structural</td>
<td>1/2</td>
<td>16</td>
<td>135</td>
</tr>
<tr>
<td></td>
<td>Structural</td>
<td>25/32</td>
<td>16</td>
<td>165</td>
</tr>
</tbody>
</table>

1. Nominal capacities shall be adjusted in accordance with Section 3.2.1 to determine ASD uniform load capacity and LRFD uniform resistances.
2. Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood with 3 or more plies.
3. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
4. Wood structural panels shall conform to the requirements for its type in DOC PS 1 or PS 2. Particleboard sheathing shall conform to ANSI A208.1. Hardboard panel and siding shall conform to the requirements of ANSI/CPA A135.6. Cellulosic fiberboard sheathing shall conform to ASTM C 208.
5. Tabulated values are for maximum bending loads from wind. Loads are limited by bending or shear stress assuming a 2-span continuous condition. Where panels are continuous over 3 or more spans the tabulated values shall be permitted to be increased in accordance with the ASD/LRFD Manual for Engineered Wood Construction.
6. Strength axis is defined as the axis parallel to the face and back orientation of the flake or the grain (veneer), which is generally the long panel direction, unless otherwise marked.

Must Span strength direction perpendicular to studs
Must have at least two spans continuous

\[ \phi_b := 0.85 \]
\[ R := 135 \text{ psf} \cdot \phi_b = 114.75 \text{ psf} > \text{ max pressure} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Withdrawal Resistance:

\[ N_s := \frac{4 \text{ ft}}{\frac{6 \text{ in}}{1}} = 8 \]

number of nails per stud

\[ N := N_s \cdot 4 = 32 \]

nails per panel

\[ p_{\text{min}} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -834.24 \text{ lbf} \]

\[ W := \frac{41 \text{ lbf}}{\text{ in}} \quad L_d := 2.5 \text{ in} \quad L_m := 2.5 \text{ in} - \frac{7}{16} \text{ in} = 2.063 \text{ in} \]

\[ W_{sd} := W \cdot L_m = 84.563 \text{ lbf} \quad K_{fw} := 3.32 \quad \phi_w := 0.65 \]

\[ W'_{sd} := W_{sd} \cdot K_{fw} \cdot \phi_w = 182.486 \text{ lbf} \]

\[ W'_{sd} \cdot N = 5839.548 \text{ lbf} \quad \text{Capacity} > \text{Load} \]

Roof:

Bending Resistance:

---

Table 3.2.2 Nominal Uniform Load Capacities (psf) for Roof Sheathing Resisting Out-of-Plane Wind Loads\(^{1,6}\)

<table>
<thead>
<tr>
<th>Sheathing Type(^1)</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in.)</th>
<th>Strength Axis(^1) Applied Perpendicular to Supports</th>
<th>Strength Axis(^1) Applied Parallel to Supports</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Rafter/Truss Spacing (in.)</td>
<td>Rafter/Truss Spacing (in.)</td>
</tr>
<tr>
<td>Wood Structural Panels</td>
<td>24/0</td>
<td>3/8</td>
<td>425</td>
<td>240</td>
</tr>
<tr>
<td>(Sheathing Grades, C-C</td>
<td>24/16</td>
<td>7/16</td>
<td>540</td>
<td>305</td>
</tr>
<tr>
<td>C-D, C-C Plugged, OSB</td>
<td>32/16</td>
<td>15/32</td>
<td>625</td>
<td>355</td>
</tr>
<tr>
<td>40/29</td>
<td>19/32</td>
<td>595</td>
<td>595</td>
<td>415</td>
</tr>
<tr>
<td>49/24</td>
<td>23/32</td>
<td>1160(^{1})</td>
<td>840(^{1})</td>
<td>615(^{1})</td>
</tr>
<tr>
<td>Wood Structural Panels</td>
<td>16 o.c.</td>
<td>19/32</td>
<td>705</td>
<td>395</td>
</tr>
<tr>
<td>(Single Floor Grades</td>
<td>20 o.c.</td>
<td>19/32</td>
<td>615</td>
<td>455</td>
</tr>
<tr>
<td>24 o.c.</td>
<td>23/32</td>
<td>1160(^{1})</td>
<td>705</td>
<td>455(^{1})</td>
</tr>
<tr>
<td>32 o.c.</td>
<td>7/8</td>
<td>1395(^{1})</td>
<td>1005(^{1})</td>
<td>695(^{1})</td>
</tr>
<tr>
<td>48 o.c.</td>
<td>1/8</td>
<td>1735(^{1})</td>
<td>1235(^{1})</td>
<td>1000(^{1})</td>
</tr>
</tbody>
</table>

---

1. Nominal capacities shall be adjusted in accordance with Section 3.2.3 to determine ASD uniform load capacity and LRFD uniform resistance.
2. Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
3. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
4. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
5. Wood structural panels shall conform to the requirements for its type in DOC P6 1 or P6 2.
6. Tabulated values are for maximum bending loads from wind. Loads are limited by bending or shear stress assuming a 2-span continuous condition. Where spans are continuous over 3 or more spans, the tabulated values shall be permitted to be increased in accordance with the ASD-LRFD Manual for Engineered Wood Construction.
7. Strength axis is defined as the axis parallel to the face and back orientation of the flake(s) or the grain (veneer), which is generally the long panel direction, unless otherwise marked.
Appendix E. Mathcad Calculations for Wind Loads: Static Units

\[
p_{3, \text{pos}} = 9.57 \text{ psf} \\
p_{2, \text{pos}} = 9.57 \text{ psf} \\
p_{\text{max}} := \max (p_{3, \text{pos}}, p_{2, \text{pos}}) = 9.57 \text{ psf}
\]

\[
p_{3, \text{neg}} = -33.33 \text{ psf} \\
p_{2, \text{neg}} = -23.43 \text{ psf} \\
p_{\text{min}} := \min (p_{3, \text{neg}}, p_{2, \text{neg}}) = -33.33 \text{ psf}
\]

Must Span strength direction perpendicular to studs
Must have at least two spans continuous
\[
\phi_{b} := 0.85
\]

\[
R := 135 \text{ psf} \cdot \phi_{b} = 114.75 \text{ psf} > \text{max pressure}
\]

Withdrawal Resistance:
\[
N_{s} := \frac{4 \text{ ft}}{6 \text{ in}} = 8 \quad \text{number of nails per stud}
\]
\[
N := N_{s} \cdot 4 = 32 \quad \text{nails per panel}
\]
\[
p_{\text{min}} = -33.33 \text{ psf}
\]
\[
p_{\text{min}} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -1066.56 \text{ lbf}
\]
\[
W := 41 \frac{\text{lbf}}{\text{in}} \quad L_{d} := 2.5 \text{ in} \quad L_{m} := 2.5 \text{ in} - \frac{1}{2} \text{ in} = 2 \text{ in}
\]
\[
W_{sd} := W \cdot L_{m} = 82 \text{ lbf} \quad K_{fw} := 3.32 \quad \phi_{w} := 0.65
\]
\[
W'_{sd} := W_{sd} \cdot K_{fw} \cdot \phi_{w} = 176.956 \text{ lbf}
\]
\[
W_{sd} \cdot N = 5662.592 \text{ lbf} \quad \text{Capacity > Load}
\]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

**150 MPH:**
Assumptions:
- Partially Open enclosure classification
- 3:12 Roof Pitch

Using Chapter 26 and 27 of ASCE 7-16

<table>
<thead>
<tr>
<th>Step</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Determine Risk Category of building; see Table 1.5-1.</td>
</tr>
<tr>
<td>2</td>
<td>Determine the basic wind speed, ( V ), for the applicable Risk Category; see Figs. 26.5-1 and 26.5-2.</td>
</tr>
<tr>
<td>3</td>
<td>Determine wind load parameters:</td>
</tr>
<tr>
<td></td>
<td>- Wind directionality factor, ( K_d ); see Section 26.6 and Table 26.6-1.</td>
</tr>
<tr>
<td></td>
<td>- Exposure category; see Section 26.7.</td>
</tr>
<tr>
<td></td>
<td>- Topographic factor, ( K_{zt} ); see Fig. 26.8-1.</td>
</tr>
<tr>
<td></td>
<td>- Ground elevation factor, ( K_e ); see Section 26.9.</td>
</tr>
<tr>
<td></td>
<td>- Gust-effect factor, ( G ) or ( G_z ); see Section 26.11.</td>
</tr>
<tr>
<td></td>
<td>- Enclosure classification; see Section 26.12.</td>
</tr>
<tr>
<td></td>
<td>- Internal pressure coefficient, ( GC_{pi} ); see Table 26.13 and Table 26.13-1.</td>
</tr>
<tr>
<td>4</td>
<td>Determine velocity pressure exposure coefficient, ( K_e ) or ( K_{zt} ); see Table 26.10-1.</td>
</tr>
<tr>
<td>5</td>
<td>Determine velocity pressure, ( q_z ) or ( q_z ), Eq. (26.10-1).</td>
</tr>
<tr>
<td>6</td>
<td>Determine external pressure coefficient, ( C_p ) or ( C_N ):</td>
</tr>
<tr>
<td></td>
<td>- Eq. (27.3-1) for rigid and flexible buildings,</td>
</tr>
<tr>
<td></td>
<td>- Eq. (27.3-2) for open buildings.</td>
</tr>
</tbody>
</table>

**Step 1:** Risk Category II  
**Step 2:** Basic Wind Speed = 150 mph \( V := 150 \)  
**Step 3:**  
- Wind directionality factor \( K_d := 0.85 \)  
- Exposure Category B  
- Topographic factor \( K_{zt} := 1 \)  
- Ground elevation factor \( K_e := 1 \)  
- Gust-effect factor \( G := 0.85 \)  
- Enclosure classification partially open  
- Internal pressure coefficient \( GC_{pi} := 0.18 \)

**Step 4:** Velocity pressure exposure coefficient \( K_e := 0.57 \)  
**Step 5:** Velocity pressure \( q_z := \left( 0.00256 \cdot K_e \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 \right) \cdot psf = 27.907 \text{ psf} \)  
Use: \( q_z := 28 \text{ psf} \) Everywhere  
**Step 6:** External pressure coefficients

---

Notation
- \( B \): Horizontal dimension of building, in ft (m), measured normal to wind direction.  
- \( L \): Horizontal dimension of building, in ft (m), measured parallel to wind direction.  
- \( h \): Mean roof height, in ft (m), except that eave height shall be used for \( 0 \leq 10 \) degrees.  
- \( z \): Height above ground, in ft (m).  
- \( G \): Gust-effect factor.  
- \( q_z, q_e \): Velocity pressure, in lb/ft\(^2\) (N/m\(^2\)), evaluated at respective height.  
- \( \theta \): Angle of plane of roof from horizontal, in degrees.
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Wind Perpendicular to Ridge:

\[ L := 8 \text{ ft} \quad h := 9.5 \text{ ft} \]
\[ B := 20 \text{ ft} \]
\[ \frac{L}{B} = 0.4 \]

Wall Pressure Coefficients:

<table>
<thead>
<tr>
<th>Surface</th>
<th>L/B</th>
<th>Cp</th>
<th>Use With</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>0-1</td>
<td>0.8</td>
<td>( q_w )</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>2+</td>
<td>-0.3</td>
<td>( q_a )</td>
</tr>
<tr>
<td>Sidewall</td>
<td>All values</td>
<td>-0.7</td>
<td>( q_s )</td>
</tr>
</tbody>
</table>

Roof Pressure Coefficients:

<table>
<thead>
<tr>
<th>Angle (degrees)</th>
<th>Cp_r_perp</th>
</tr>
</thead>
<tbody>
<tr>
<td>10°</td>
<td>-1.3</td>
</tr>
</tbody>
</table>

\[ h \]

\[ P_{ww \text{ext}} := q_z \cdot G \cdot C_{p_{ww}} = 19.04 \text{ psf} \]
\[ P_{lw \text{ext}} := q_z \cdot G \cdot C_{p_{lw\_perp}} = -11.9 \text{ psf} \]
\[ P_{r \text{ext}} := q_z \cdot G \cdot C_{p_{r\_perp}} = -30.94 \text{ psf} \]
\[ P_{int} := q_z \cdot G \cdot C_{pi} = 5.04 \text{ psf} \]

roof pressure:  \[ P_r := P_{r\text{ext}} + -1 \cdot P_{int} = -35.98 \text{ psf} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

**Forces:**

*Windward Wall:*

\[ P_{ww\text{-ext}} = 19.04 \text{ psf} \]

\[ F_{ww} = P_{ww\text{-ext}} \cdot \left(\frac{h}{2}\right) = 90.44 \text{ plf} \]

Line load along top and bottom of wall

*Leeward Wall:*

\[ P_{lw\text{-ext}} = -11.9 \text{ psf} \]

\[ F_{lw} = (P_{lw\text{-ext}} - P_{int}) \cdot \left(\frac{h}{2}\right) = -80.465 \text{ plf} \]

Line load along top and bottom of wall

*Roof:*

\[ P_r = -35.98 \text{ psf} \]

x-comp:

\[ P_{rx} = P_r \cdot \sin(14 \text{ deg}) = -8.704 \text{ psf} \]

y-comp:

\[ P_{ry} = P_r \cdot \cos(14 \text{ deg}) = -34.911 \text{ psf} \]

Each shear wall Perp to ridge

Ftop:
- Half of top of wall (ww & lw) line load
- Half of roof x comp

\[ F_{top} = (F_{ww} - F_{lw} - P_{rx} \cdot L) \cdot \left(\frac{B}{2}\right) = 2.405 \text{ kip} \]

Fbot:
- Half of bot of wall (ww & lw) line load

\[ F_{bot} = (F_{ww} - F_{lw}) \cdot \left(\frac{B}{2}\right) = 1.709 \text{ kip} \]

Fry:
- uplift line load

\[ F_{r,y} = P_{ry} \cdot \left(\frac{B}{2}\right) = -349.112 \text{ plf} \]

Shear at base of wall:

\[ V_b := F_{top} + F_{bot} = 4.114 \text{ kip} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Load Case:

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \]  \hspace{1cm} (EQ 4.3.2-4)

Overturning Moments:

\[ L = 8 \text{ ft} \quad B = 20 \text{ ft} \]

Overturning moment from Ftop:

\[ M_{F_{top}} := F_{top} \cdot h = 22.851 \text{ kip} \cdot \text{ft} \]

Overturning moment from Roof Uplift:

\[ M_{F_{up}} := -F_{r-y} \cdot L \cdot \frac{L}{2} = 11.172 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:

\[ M_{r} := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 15.424 \text{ kip} \cdot \text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F_{top}} + M_{F_{up}} - M_{r}}{L - 1 \text{ ft}} = 2.657 \text{ kip} \]

Required Compression Force:

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 6.513 \text{ kip} \]

Required Studs for compression:

CLT Capacity:

\[ F'_{c} := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ C = 1.303 \quad \text{Need at least 2 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 0.039 \frac{1 \text{ lbf}}{\text{ft}} \text{ of Hinge along wall} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Load Case: 

\[ 0.9 \times DL + W \quad (EQ \ 4.3.2-5) \]

Overturning Moments: 

\[ L = 8 \ ft \quad B = 20 \ ft \]

\[ DL_r := 23.5 \ psf \]

\[ LL_r := 20 \ psf \]

\[ Trib := \frac{B}{2} \]

Overturning moment from Ftop: 

\[ M_{F_{top}} := F_{top} \cdot h = 22.851 \ kip \cdot ft \]

Overturning moment from Roof Uplift: 

\[ M_{F_{up}} := -F_{r_{y}} \cdot L \cdot \frac{L}{2} = 11.172 \ kip \cdot ft \]

Required force in hold down: 

\[ T := \frac{M_{F_{top}} + M_{F_{up}}}{L - 1 \ ft} = 4.86 \ kip \]

\[ M_{F_{top}} + M_{F_{up}} = 34.023 \ kip \cdot ft \]

Required Compression Force: 

\[ C := T = 4.86 \ kip \]

Required Studs for compression: 

CLT Capacity: 

\[ F' := 35849 \ plf = 35.849 \ klf \quad \text{Okay} \]

Light-Frame Capacity: 

\[ P' := 5 \ kip \]

\[ C = 0.972 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear: 

Ultimate Strength of Hinge A in Shear 

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_{n} := 3.85 \ klf \]

For Base shear we would need: 

\[ \frac{V}{V'_{n}} = 0.039 \ \frac{1}{lb} \cdot ft \quad \text{of Hinge along wall} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

<table>
<thead>
<tr>
<th>Shearwall:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$V := \frac{F_{top}}{L \cdot \phi_D} = 375.843 \text{ plf}$</td>
<td>$\phi_D := 0.8$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sheathing:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural 1</td>
<td></td>
</tr>
<tr>
<td>7/16&quot;</td>
<td></td>
</tr>
<tr>
<td>24/16 Span Rating</td>
<td></td>
</tr>
</tbody>
</table>

$V_w := 715 \text{ plf}$
Use 8d nails with 6" spacing along edge

<table>
<thead>
<tr>
<th>Diaphragm:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_d := V = 375.843 \text{ plf}$</td>
<td></td>
</tr>
<tr>
<td>$V_{wd} := 670 \text{ plf}$</td>
<td>okay</td>
</tr>
</tbody>
</table>
Appendix E. Mathcad Calculations for Wind Loads: Static Units

**Wind Parallel To Ridge:**

\[
L := 20 \text{ ft} \quad h := 9.5 \text{ ft} \\
B := 8 \text{ ft} \\
\frac{L}{B} = 2.5
\]

**Wall Pressure Coefficients:**

<table>
<thead>
<tr>
<th>Surface</th>
<th>(L/B)</th>
<th>(C_p)</th>
<th>Use With</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>All values</td>
<td>0.8</td>
<td>(q_z)</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>2</td>
<td>-0.3</td>
<td>(q_a)</td>
</tr>
<tr>
<td>Sidewall</td>
<td>All values</td>
<td>-0.7</td>
<td>(q_s)</td>
</tr>
</tbody>
</table>

**Roof Pressure Coefficients, \(C_p\) for use with \(q_a\):**

<table>
<thead>
<tr>
<th>Wind Direction</th>
<th>Angle (degrees)</th>
<th>(C_p)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal to Ridge for (h/L)</td>
<td>0 to (h/2)</td>
<td>0.0</td>
</tr>
<tr>
<td>Normal to Ridge for (h/L)</td>
<td>(h/2) to 0</td>
<td>0.0</td>
</tr>
<tr>
<td>Normal to Ridge for (h/L)</td>
<td>0 to (h/2)</td>
<td>0.0</td>
</tr>
<tr>
<td>Normal to Ridge for (h/L)</td>
<td>(h/2) to 0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

**Non-Commercial Use Only**

**Roof Pressure Coefficients:**

\[
h = \frac{0.475}{L} \\
C_{p_{r\_perp}} = -0.9
\]

**Wall Pressure Coefficients:**

<table>
<thead>
<tr>
<th>Surface</th>
<th>(C_{p_{ww}})</th>
<th>(C_{p_{lw_perp}})</th>
<th>(C_{p_{sw}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward Wall</td>
<td>0.8</td>
<td>-0.3</td>
<td></td>
</tr>
<tr>
<td>Leeward</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sidewall</td>
<td></td>
<td></td>
<td>-0.7</td>
</tr>
</tbody>
</table>

**Calculation:**

\[
\begin{align*}
P_{ww\_ext} &= q_z \cdot G \cdot C_{p_{ww}} = 19.04 \text{ psf} \\
P_{lw\_ext} &= q_z \cdot G \cdot C_{p_{lw\_perp}} = -7.14 \text{ psf} \\
P_{r\_ext} &= q_z \cdot G \cdot C_{p_{r\_perp}} = -21.42 \text{ psf} \\
P_{\text{int}} &= q_z \cdot G \cdot C_{pi} = 5.04 \text{ psf} \\
\text{roof pressure: } P_r &= P_{r\_ext} + -1 \cdot P_{\text{int}} = -26.46 \text{ psf}
\end{align*}
\]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Forces:

Windward Wall:

\[ P_{ww\_{ext}} = 19.04 \text{ psf} \]
\[ F_{ww} = P_{ww\_{ext}} \cdot \left( \frac{h}{2} \right) = 90.44 \text{ plf} \]

Line load along top and bottom of wall

Leeward Wall:

\[ P_{lw\_{ext}} = -7.14 \text{ psf} \]
\[ F_{lw} := (P_{lw\_{ext}} - P_{int}) \cdot \left( \frac{h}{2} \right) = -57.855 \text{ plf} \]

Line load along top and bottom of wall

Roof:

\[ P_{ry} = -34.911 \text{ psf} \]

Each shear wall Par to ridge

- Half of top of wall (ww & lw) line load

\[ F_{top} := (F_{ww} - F_{lw}) \cdot \left( \frac{B}{2} \right) = 0.593 \text{ kip} \]

- Half of bot of wall (ww & lw) line load

\[ F_{bot} := (F_{ww} - F_{lw}) \cdot \left( \frac{B}{2} \right) = 0.593 \text{ kip} \]

- Uplift line load

\[ F_{r,y} := P_{ry} \cdot \left( \frac{B}{2} \right) = -139.645 \text{ plf} \]

Shear at base of wall:

\[ V_b := F_{top} + F_{bot} = 1.186 \text{ kip} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Load Case:

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \]  \hspace{1cm} (EQ 4.3.2-4)

Overturning Moments:

Overturning moment from Ftop:

\[ M_{F_{\text{top}}} := F_{\text{top}} \cdot h = 5.635 \text{ kip} \cdot \text{ft} \]

Overturning moment from Roof Uplift:

\[ M_{F_{\text{up}}} := -F_{r_y} \cdot L \cdot \frac{L}{2} = 27.929 \text{ kip} \cdot \text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F_{\text{top}}} + M_{F_{\text{up}}} - M_r}{L - 1 \text{ ft}} = -0.263 \text{ kip} \]

Required Compression Force:

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 3.593 \text{ kip} \]

Required Studs for compression:

CLT Capacity:

\[ F'_{c} := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ C = 0.719 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi = 0.65 \quad \text{Connections} \]

\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 0.098 \frac{1}{\text{ft}} \cdot \text{ft} \quad \text{of Hinge along wall} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Load Case:

0.9 × DL + W \hspace{1cm} (EQ 4.3.2-5)

Overturning Moments:

\[ L = 20 \text{ ft} \quad B = 8 \text{ ft} \]

Overturning moment from Ftop:

\[ M_{F, top} := F_{top} \cdot h = 5.635 \text{ kip} \cdot \text{ft} \]

Overturning moment from Roof Uplift:

\[ M_{F, up} := -F_{r,y} \cdot L \cdot \frac{L}{2} = 27.929 \text{ kip} \cdot \text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F, top} + M_{F, up}}{L - 1 \text{ ft}} = 1.767 \text{ kip} \quad M_{F, top} + M_{F, up} = 33.564 \text{ kip} \cdot \text{ft} \]

Required Compression Force:

\[ C := T = 1.767 \text{ kip} \]

Required Studs for compression:

CLT Capacity:

\[ F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ C = 0.353 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 0.098 \frac{1}{\text{ft}} \text{ of Hinge along wall} \]
### Appendix E. Mathcad Calculations for Wind Loads: Static Units

<table>
<thead>
<tr>
<th>Component</th>
<th>Calculation</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Sheathing Nailing to Resist Shear: (LF UNITS)</td>
<td>$ϕ_D := 0.8$</td>
<td></td>
</tr>
<tr>
<td>Shearwall:</td>
<td>$V := \frac{F_{top}}{L \cdot ϕ_D} = 37.074 \text{ plf}$</td>
<td>Sheathing: Structural 1, 7/16&quot; 24/16 Span Rating</td>
</tr>
<tr>
<td></td>
<td>$V_w := 715 \text{ plf}$</td>
<td>Use 8d nails with 6&quot; spacing along edge</td>
</tr>
<tr>
<td>Diaphragm:</td>
<td>$V_d := V = 37.074 \text{ plf}$</td>
<td>$V_{wd} := 670 \text{ plf}$ okay</td>
</tr>
</tbody>
</table>
Components and Cladding (C&C):

| Step 1: Determine risk category; see Table 1.5-1. |
| Step 2: Determine the basic wind speed, V, for applicable risk category; see Figs. 26.5-1 and 26.5-2. |
| Step 3: Determine wind load parameters: |
| - Wind directionality factor, $K_d$; see Section 26.6 and Table 26.6-1. |
| - Exposure category B, C, or D; see Section 26.7. |
| - Topographic factor, $K_z$; see Section 26.8 and Fig. 26.8-1. |
| - Ground elevation factor, $K_e$; Section 26.9 and Table 26.9-1. |
| - Enclosure classification; see Section 26.12. |
| - Internal pressure coefficient, ($GC_{pi}$); see Section 26.13 and Table 26.13-1. |

Step 4: Determine velocity pressure exposure coefficient, $K_h$; see Table 26.10-1.

Step 5: Determine velocity pressure, $q_h$, Eq. (26.10-1).

Step 6: Determine external pressure coefficient, ($GC_p$):
- Walls; see Fig. 30.3-1.
- Flat roofs, gable roofs, hip roofs; see Fig. 30.3-2.
- Stepped roofs; see Fig. 30.3-3.
- Multi-span gable roofs; see Fig. 30.3-4.
- Mansard roofs; see Fig. 30.3-5.
- Sawtooth roofs; see Fig. 30.3-6.
- Dormer roofs; see Fig. 30.3-7.
- Archid roofs; see Fig. 27.3-3, Note 4.

Step 7: Calculate wind pressure, $p_c$; Eq. (30.3-1).
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Positive:
\[ GC_{p,5\_pos} := 1 \]
\[ GC_{p,4\_pos} := 1 \]

Negative:
\[ GC_{p,5\_neg} := -1.4 \]
\[ GC_{p,4\_neg} := -1.1 \]

Design Pressures:
\[ p_{5\_pos} := q_h \cdot (GC_{p,5\_pos} + GC_{pi}) = 48.38 \text{ psf} \]
\[ p_{4\_pos} := q_h \cdot (GC_{p,4\_pos} + GC_{pi}) = 48.38 \text{ psf} \]
\[ p_{5\_neg} := q_h \cdot (GC_{p,5\_neg} - GC_{pi}) = -64.78 \text{ psf} \]
\[ p_{4\_neg} := q_h \cdot (GC_{p,4\_neg} - GC_{pi}) = -52.48 \text{ psf} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Roof:

*Diagrams*

**Notation**

- \( a = 10\% \) of least horizontal dimension or \( 0.4h \), whichever is smaller, but not less than either \( 4\% \) of least horizontal dimension or 3 ft (0.9 m).
- \( h = \) Mean roof height, in ft (m).
- \( W = \) Building width, in ft (m).
- \( \theta = \) Angle of plane of roof from horizontal, in degrees.

**External Pressure Coefficients**

**Notes**

1. Vertical scale denotes \((GC_\rho)\) to be used with \( q_\rho \).
2. Horizontal scale denotes effective wind area \( A_e \), in \( \text{ft}^2 (\text{m}^2) \).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
# Appendix E. Mathcad Calculations for Wind Loads: Static Units

<table>
<thead>
<tr>
<th>Area:</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_3 := (2 \cdot a) \cdot (4 \cdot a) = 72 \text{ ft}^2$</td>
</tr>
<tr>
<td>$A_2 := 2 \cdot (B - (2 \cdot a)) \cdot a + (L \cdot a) = 72 \text{ ft}^2$</td>
</tr>
<tr>
<td>$A_1 := 0 \text{ ft}^2$</td>
</tr>
</tbody>
</table>

**Positive:**
- $GC_{p,3\_pos} := 0.4$
- $GC_{p,2\_pos} := 0.4$

**Negative:**
- $GC_{p,3\_neg} := -2.2$
- $GC_{p,2\_neg} := -1.6$

**Design Pressures:**
- $p_{3\_pos} := q_h \cdot (GC_{p,3\_pos} + GC_{pi}) = 23.78 \text{ psf}$
- $p_{2\_pos} := q_h \cdot (GC_{p,2\_pos} + GC_{pi}) = 23.78 \text{ psf}$
- $p_{3\_neg} := q_h \cdot (GC_{p,3\_neg} + GC_{pi}) = -82.82 \text{ psf}$
- $p_{2\_neg} := q_h \cdot (GC_{p,2\_neg} + GC_{pi}) = -58.22 \text{ psf}$
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Sheathing Design: (For Light-Frame Unit)

Wall: $p_{5,\text{pos}} = 48.38\text{ psf}$

Bending Resistance: $l_b = 24\text{ in}$

- 24" spacing of joists and studs
- 24/16 span rating 7/16" thick

$p_{\text{max}} := \max (p_{5,\text{pos}}, p_{4,\text{pos}}, p_{5,\text{neg}}, p_{4,\text{neg}}) = 48.38\text{ psf}$

$p_{\text{min}} := \min (p_{5,\text{pos}}, p_{4,\text{pos}}, p_{5,\text{neg}}, p_{4,\text{neg}}) = -64.78\text{ psf}$

<table>
<thead>
<tr>
<th>Sheathing Type</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in.)</th>
<th>Perpendicular to Supports</th>
<th>Parallel to Supports</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Maximum Stud Spacing</td>
<td>Maximum Actual Stud Spacing</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(in.)</td>
<td>(in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>12</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Nominal Uniform Loads (psf)</td>
<td>Actual Uniform Loads (psf)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>12</td>
<td>15</td>
</tr>
<tr>
<td>Wood Structural Panels (Sheathing Grades, C-C, C-D, C-C Plugged, OSB)²</td>
<td>24/0</td>
<td>3/8</td>
<td>24</td>
<td>425</td>
</tr>
<tr>
<td></td>
<td>24/16</td>
<td>7/16</td>
<td>24</td>
<td>540</td>
</tr>
<tr>
<td></td>
<td>32/16</td>
<td>15/32</td>
<td>24</td>
<td>625</td>
</tr>
<tr>
<td></td>
<td>40/20</td>
<td>19/32</td>
<td>24</td>
<td>955</td>
</tr>
<tr>
<td></td>
<td>48/24</td>
<td>23/32</td>
<td>24</td>
<td>1160²</td>
</tr>
<tr>
<td>Particleboard Sheathing (M-S Exterior Glue)</td>
<td>3/8</td>
<td>1/2</td>
<td>16</td>
<td>(contact manufactur)</td>
</tr>
<tr>
<td></td>
<td>5/8</td>
<td>3/4</td>
<td>16</td>
<td>(contact manufactur)</td>
</tr>
<tr>
<td>Hardboard Siding (Direct to Studs)</td>
<td>Lap Siding</td>
<td>7/16</td>
<td>16</td>
<td>460</td>
</tr>
<tr>
<td></td>
<td>Shiplap Edge Panel Siding</td>
<td>7/16</td>
<td>16</td>
<td>460</td>
</tr>
<tr>
<td></td>
<td>Square Edge Panel Siding</td>
<td>7/16</td>
<td>16</td>
<td>460</td>
</tr>
<tr>
<td>Cellulosic Fiberboard Sheathing</td>
<td>Regular</td>
<td>1/2</td>
<td>16</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td>Structural</td>
<td>1/2</td>
<td>16</td>
<td>135</td>
</tr>
<tr>
<td></td>
<td>Structural</td>
<td>25/32</td>
<td>16</td>
<td>165</td>
</tr>
</tbody>
</table>

1. Nominal capacities shall be adjusted in accordance with Section 3.2.1 to determine ASD uniform load capacity and LRFD uniform resistances.
2. Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood with 3 or more plies.
3. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
4. Wood structural panels shall conform to the requirements for its type in DOC PS 1 or PS 2. Particleboard sheathing shall conform to ANSI A208.1. Hardboard panel and siding shall conform to the requirements of ANSI/CPA A135.6. Cellulosic fiberboard sheathing shall conform to ASTM C 208.
5. Tabulated values are for maximum bending loads from wind. Loads are limited by bending or shear stress assuming a 2-span continuous condition. Where panels are continuous over 3 or more spans the tabulated values shall be permitted to be increased in accordance with the ASD/LRFD Manual for Engineered Wood Construction.
6. Strength axis is defined as the axis parallel to the face and back orientation of the flake or the grain (veneer), which is generally the long panel direction, unless otherwise marked.

Must Span strength direction perpendicular to studs
Must have at least two spans continuous

$\phi_b := 0.85$

$R := 135\text{ psf} \cdot \phi_b = 114.75\text{ psf} > \text{max pressure}$
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Withdrawal Resistance:

\[ N_s = \frac{4 \text{ ft}}{6 \text{ in}} = 8 \]  
number of nials per stud

\[ N = N_s \cdot 4 = 32 \]  
nails per panel

\[ p_{min} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -2072.96 \text{ lbf} \]

\[ W := \frac{41 \text{ lbf}}{\text{in}} \quad L_d := 2.5 \text{ in} \quad L_m := 2.5 \text{ in} - \frac{7}{16} \text{ in} = 2.063 \text{ in} \]

\[ W_{sd} := W \cdot L_m = 84.563 \text{ lbf} \quad K_{fw} := 3.32 \quad \phi_w := 0.65 \]

\[ W'_{sd} := W_{sd} \cdot K_{fw} \cdot \phi_w = 182.486 \text{ lbf} \]

\[ W'_{sd} \cdot N = 5839.548 \text{ lbf} \quad \text{Capacity > Load} \]

Roof:

Bending Resistance:

Table 3.2.2 Nominal Uniform Load Capacities (psf) for Roof Sheathing Resisting Out-of-Plane Wind Loads

<table>
<thead>
<tr>
<th>Sheathing Type</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in)</th>
<th>Strength Axis' Applied Perpendicular to Supports</th>
<th>Strength Axis' Applied Parallel to Supports</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Structural Panels</td>
<td>24/0</td>
<td>3/8</td>
<td>425</td>
<td>240</td>
</tr>
<tr>
<td>(Sheathing Grades, C-C, C-D, C-C Plugged, OSB)</td>
<td>24/16</td>
<td>7/16</td>
<td>540</td>
<td>305</td>
</tr>
<tr>
<td></td>
<td>32/16</td>
<td>15/32</td>
<td>625</td>
<td>355</td>
</tr>
<tr>
<td></td>
<td>49/29</td>
<td>19/32</td>
<td>955</td>
<td>595</td>
</tr>
<tr>
<td></td>
<td>49/24</td>
<td>23/32</td>
<td>1160</td>
<td>840</td>
</tr>
<tr>
<td>Wood Structural Panels</td>
<td>16 o.c.</td>
<td>19/32</td>
<td>705</td>
<td>395</td>
</tr>
<tr>
<td>(Single Floor Grades, Underlayment, C-C Plugged)</td>
<td>20 o.c.</td>
<td>19/32</td>
<td>815</td>
<td>455</td>
</tr>
<tr>
<td></td>
<td>24 o.c.</td>
<td>23/32</td>
<td>915</td>
<td>670</td>
</tr>
<tr>
<td></td>
<td>32 o.c.</td>
<td>7/8</td>
<td>1385</td>
<td>1000</td>
</tr>
<tr>
<td></td>
<td>48 o.c.</td>
<td>1-1/8</td>
<td>1750</td>
<td>1385</td>
</tr>
</tbody>
</table>

1. Nominal capacities shall be adjusted in accordance with Section 3.2.3 to determine ASD uniform load capacity and LRFD uniform resistances.
2. Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
3. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
4. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
5. Wood structural panels shall conform to the requirements for its type in DOC PB 1 or PB 2.
6. Tabulated values are for maximum bending loads from wind. Loads are limited by bending or shear stress assuming a 2-span continuous condition. Where panels are continuous over 3 or more spans, the tabulated values shall be permitted to be increased in accordance with the ASD-LRFD Manual for Engineered Wood Construction.
7. Strength axis is defined as the axis parallel to the face and back orientation of the sheathing or the grain (vesse), which is generally the long panel direction, unless otherwise marked.
Appendix E. Mathcad Calculations for Wind Loads: Static Units

\[ p_{3, \text{pos}} = 23.78 \text{ psf} \]
\[ p_{2, \text{pos}} = 23.78 \text{ psf} \]

\[ p_{\text{max}} := \text{max} \ (p_{3, \text{pos}}, p_{2, \text{pos}}) = 23.78 \text{ psf} \]

\[ p_{3, \text{neg}} = -82.82 \text{ psf} \]
\[ p_{2, \text{neg}} = -58.22 \text{ psf} \]

\[ p_{\text{min}} := \text{min} \ (p_{3, \text{neg}}, p_{2, \text{neg}}) = -82.82 \text{ psf} \]

**Must Span strength direction perpendicular to studs**
**Must have at least two spans continuous**

\[ \phi_b := 0.85 \]
\[ R := 135 \text{ psf} \cdot \phi_b = 114.75 \text{ psf} \quad > \text{max pressure} \]

**Withdrawal Resistance:**

\[ N_s := \frac{4 \text{ ft}}{6 \text{ in}} = 8 \quad \text{number of nails per stud} \]
\[ N := N_s \cdot 4 = 32 \quad \text{nails per panel} \]
\[ p_{\text{min}} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -2650.24 \text{ lbf} \]
\[ W := 41 \frac{\text{lbf}}{\text{in}} \quad L_d := 2.5 \text{ in} \quad L_m := 2.5 \text{ in} - \frac{1}{2} \text{ in} = 2 \text{ in} \]
\[ W_{8d} := W \cdot L_m = 82 \text{ lbf} \quad K_{fw} := 3.32 \quad \phi_w := 0.65 \]
\[ W'_{8d} := W_{8d} \cdot K_{fw} \cdot \phi_w = 176.956 \text{ lbf} \]
\[ W'_{8d} \cdot N = 5662.592 \text{ lbf} \quad \text{Capacity} > \text{Load} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

180 MPH:
Assumptions:
- Partially Open enclosure classification
- 3:12 Roof Pitch

Using Chapter 26 and 27 of ASCE 7-16

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Determine Risk Category of building; see Table 1.5-1.</td>
</tr>
<tr>
<td>2</td>
<td>Determine the basic wind speed, ( V ), for the applicable Risk Category; see Figs. 26.5-1 and 26.5-2.</td>
</tr>
<tr>
<td>3</td>
<td>Determine wind load parameters:</td>
</tr>
<tr>
<td></td>
<td>- Wind directionality factor, ( K_d ); see Section 26.6 and Table 26.6-1.</td>
</tr>
<tr>
<td></td>
<td>- Exposure category; see Section 26.7.</td>
</tr>
<tr>
<td></td>
<td>- Topographic factor, ( K_{zt} ); see Section 26.8 and Table in Fig. 26.8-1.</td>
</tr>
<tr>
<td></td>
<td>- Ground elevation factor, ( K_e ); see Section 26.9</td>
</tr>
<tr>
<td></td>
<td>- Gust-effect factor, ( G ) or ( G_e ); see Section 26.11.</td>
</tr>
<tr>
<td></td>
<td>- Enclosure classification; see Section 26.12.</td>
</tr>
<tr>
<td></td>
<td>- Internal pressure coefficient, ( GC_{pi} ); see Section 26.13 and Table 26.13-1.</td>
</tr>
<tr>
<td>4</td>
<td>Determine velocity pressure exposure coefficient, ( K_p ) or ( K_e ); see Table 26.10-1.</td>
</tr>
<tr>
<td>5</td>
<td>Determine velocity pressure ( q ) or ( q_z ), Eq. (26.10-1).</td>
</tr>
<tr>
<td>6</td>
<td>Determine external pressure coefficient, ( C_p ) or ( C_e ):</td>
</tr>
<tr>
<td></td>
<td>- Fig. 27.3-1 for walls and flat, gable, hip, monoslope, or mansard roofs.</td>
</tr>
<tr>
<td></td>
<td>- Fig. 27.3-2 for domed roofs.</td>
</tr>
<tr>
<td></td>
<td>- Fig. 27.3-3 for arched roofs.</td>
</tr>
<tr>
<td></td>
<td>- Fig. 27.3-4 for monoslope roof, open building.</td>
</tr>
<tr>
<td></td>
<td>- Fig. 27.3-5 for pitched roof, open building.</td>
</tr>
<tr>
<td></td>
<td>- Fig. 27.3-6 for roughed roof, open building.</td>
</tr>
<tr>
<td></td>
<td>- Fig. 27.3-7 for along-ridge/valley wind load case for monoslope pitched, or roughed roof, open building.</td>
</tr>
<tr>
<td>7</td>
<td>Calculate wind pressure, ( p_z ) on each building surface:</td>
</tr>
<tr>
<td></td>
<td>- Eq. (27.3-1) for rigid and flexible buildings.</td>
</tr>
<tr>
<td></td>
<td>- Eq. (27.3-2) for open buildings.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Risk Category II</td>
</tr>
<tr>
<td>2</td>
<td>Basic Wind Speed = 180 mph ( V := 180 )</td>
</tr>
<tr>
<td>3</td>
<td>Wind directionality ( K_d := 0.85 )</td>
</tr>
<tr>
<td></td>
<td>Exposure Category B</td>
</tr>
<tr>
<td></td>
<td>Topographic factor ( K_{zt} := 1 )</td>
</tr>
<tr>
<td></td>
<td>Ground elevation factor ( K_e := 1 )</td>
</tr>
<tr>
<td></td>
<td>Gust-effect factor ( G := 0.85 )</td>
</tr>
<tr>
<td></td>
<td>Enclosure classification partially open</td>
</tr>
<tr>
<td></td>
<td>Internal pressure coefficient ( GC_{pi} := 0.18 )</td>
</tr>
<tr>
<td>4</td>
<td>Velocity pressure exposure coefficient ( K_p := 0.57 )</td>
</tr>
<tr>
<td>5</td>
<td>Velocity pressure ( q_z := (0.00256 \cdot K_p \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2) \cdot \text{psf} = 40.186 \text{ psf} )</td>
</tr>
<tr>
<td>6</td>
<td>External pressure coefficients</td>
</tr>
</tbody>
</table>

Notation:
- \( B \): Horizontal dimension of building, in ft (m), measured normal to wind direction.
- \( L \): Horizontal dimension of building, in ft (m), measured parallel to wind direction.
- \( h \): Mean roof height, in ft (m), except that curve height shall be used for 0 \( \leq \) 10 degrees.
- \( z \): Height above ground, in ft (m).
- \( G \): Gust-effect factor.
- \( q_z, q_{z0} \): Velocity pressure, in lb/ft\(^2\) (N/m\(^2\)), evaluated at respective height.
- \( \theta \): Angle of plane of roof from horizontal, in degrees.
Appendix E. Mathcad Calculations for Wind Loads: Static Units

### Wind Perpendicular to Ridge:

\[ L := 8 \text{ ft} \quad h := 9.5 \text{ ft} \quad B := 20 \text{ ft} \]

\[ \frac{L}{B} = 0.4 \]

**Wall Pressure Coefficients:**

<table>
<thead>
<tr>
<th>Surface</th>
<th>( L/B )</th>
<th>( C_p )</th>
<th>Use With</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>All values</td>
<td>0.8</td>
<td>( q_w )</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>-0.5</td>
<td>( q_w )</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>2</td>
<td>-0.3</td>
<td>( q_w )</td>
</tr>
<tr>
<td></td>
<td>2/4</td>
<td>-0.2</td>
<td>( q_w )</td>
</tr>
<tr>
<td>Sidewall</td>
<td>All values</td>
<td>-0.7</td>
<td>( q_w )</td>
</tr>
</tbody>
</table>

**Roof Pressure Coefficients:**

\[ \frac{h}{L} = 1.188 \]

\[ C_{p_{r\_perp}} := -1.3 \]

**Windward Wall**

\[ C_{p_{ww}} := 0.8 \]

**Leeward Wall**

\[ C_{p_{lw\_perp}} := -0.5 \]

\[ C_{p_{sw}} := -0.7 \]

---

**Example Calculations:**

**Windward Wall**

\[ P_{ww\_ext} := q_z \cdot G \cdot C_{p_{ww}} = 27.88 \text{ psf} \]

**Leeward Wall**

\[ P_{lw\_ext} := q_z \cdot G \cdot C_{p_{lw\_perp}} = -17.425 \text{ psf} \]

\[ P_{r\_ext} := q_z \cdot G \cdot C_{p_{r\_perp}} = -45.305 \text{ psf} \]

**Internal Pressure**

\[ P_{int} := q_z \cdot G \cdot C_{p_{int}} = 7.38 \text{ psf} \]

**Roof Pressure**

\[ P_r := P_{r\_ext} + 1 \cdot P_{int} = -52.685 \text{ psf} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

**Forces:**

**Windward Wall:**

\[ P_{ww\_ext} = 27.88 \text{ psf} \]

\[ F_{ww} = P_{ww\_ext} \cdot \left( \frac{h}{2} \right) = 132.43 \text{ plf} \quad \text{Line load along top and bottom of wall} \]

**Leeward Wall:**

\[ P_{lw\_ext} = -17.425 \text{ psf} \]

\[ F_{lw} := (P_{lw\_ext} - P_{int}) \cdot \left( \frac{h}{2} \right) = -117.824 \text{ plf} \quad \text{Line load along top and bottom of wall} \]

**Roof:**

\[ P_r = -52.685 \text{ psf} \]

x-comp:

\[ P_{rx} := P_r \cdot \sin(14 \text{ deg}) = -12.746 \text{ psf} \]

y-comp:

\[ P_{ry} := P_r \cdot \cos(14 \text{ deg}) = -51.12 \text{ psf} \]

Each shear wall Perp to ridge

- **Ftop:** - Half of top of wall (ww & lw) line load
  - Half of roof x comp
  \[ F_{top} := (F_{ww} - F_{lw} - (P_{rx} \cdot L)) \cdot \left( \frac{B}{2} \right) = 3.522 \text{ kip} \]

- **Fbot:** - Half of bot of wall (ww & lw) line load
  \[ F_{bot} := (F_{ww} - F_{lw}) \cdot \left( \frac{B}{2} \right) = 2.503 \text{ kip} \]

- **Fry:** - uplift line load
  \[ F_{r\_y} := P_{ry} \cdot \left( \frac{B}{2} \right) = -511.2 \text{ plf} \]

Shear at base of wall:

\[ V_b := F_{top} + F_{bot} = 6.025 \text{ kip} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Load Case:

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \quad (EQ \ 4.3.2-4) \]

Overturning Moments:

\[ L = 8 \text{ ft} \quad B = 20 \text{ ft} \]

Overturning moment from Ftop:

\[ M_{F\text{,top}} := F_{\text{top}} \cdot h = 33.461 \text{ kip} \cdot \text{ft} \]

Overturning moment from Roof Uplift:

\[ M_{F\text{,up}} := -F_{r\text{-y}} \cdot L \cdot \frac{L}{2} = 16.358 \text{ kip} \cdot \text{ft} \]

Overturning moment from Tributary:

\[ M_{\text{Trib}} := \frac{B}{2} \]

Resisting moment from Dead and Live:

\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot \frac{L}{2} = 15.424 \text{ kip} \cdot \text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F\text{,top}} + M_{F\text{,up}} - M_r}{L - 1 \text{ ft}} = 4.914 \text{ kip} \]

Required Compression Force:

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} = 8.77 \text{ kip} \]

Required Studs for compression:

CLT Capacity:

\[ F'_{c} = 35.849 \text{ plf} = 35.849 \text{ klf} \]

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ \frac{C}{P'} = 1.754 \quad \text{Need at least 2 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 0.047 \quad \frac{1 \text{ lbf}}{\text{ft}} \] of Hinge along wall
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Load Case:

\[ 0.9 \times DL + W \]  \hspace{1cm}  \text{(EQ 4.3.2-5)}

**Overturning Moments:**

\[
\begin{align*}
L &= 8 \text{ ft} \\
B &= 20 \text{ ft} \\
DL_r &= 23.5 \text{ psf} \\
LL_r &= 20 \text{ psf} \\
Trib &= \frac{B}{2} \\
\end{align*}
\]

Overturning moment from Ftop:

\[ M_{F,\text{top}} := F_{\text{top}} \cdot h = 33.461 \text{ kip} \cdot \text{ft} \]

Overturning moment from Roof Uplift:

\[ M_{F,\text{up}} := -F_{r,y} \cdot L \cdot \frac{L}{2} = 16.358 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**

\[
T := \frac{M_{F,\text{top}} + M_{F,\text{up}}}{L - 1 \text{ ft}} = 7.117 \text{ kip} \\
\]

\[ T = \frac{2.144 \text{ kip}}{3.32} \]

\[ M_{F,\text{top}} + M_{F,\text{up}} = 49.819 \text{ kip} \cdot \text{ft} \]

\[ F_t := 675 \text{ psi} \]

\[ C_M := 1 \quad C_t := 1 \quad C_i := 1 \quad K_{Ft} := 2.7 \quad \phi_t := 0.8 \quad \lambda := 1 \]

\[ F' := F_t \cdot C_M \cdot C_t \cdot C_i \cdot K_{Ft} \cdot \phi_t \cdot \lambda = (1.458 \cdot 10^3) \text{ psi} \]

\[ D_d := 0.25 \text{ in} \quad \text{Dowel Diameter} \]

\[ A_n := 2 \cdot 1.5 \text{ in} \cdot (3.5 \text{ in} - D_d) = 9.75 \text{ in}^2 \]

\[ T'_{\text{n}} := F' \cdot A_n = 14.216 \text{ kip} \]

Two studs is good

**Required Compression Force:**

\[ C := T = 7.117 \text{ kip} \]

**Required Studs for compression:**

CLT Capacity:

\[ F'_{c} := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \quad \frac{C}{P'} = 1.423 \]

Need at least 2 studs in corner for compression

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_{\text{n}} := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_{\text{n}}} = 0.047 \frac{1 \text{ lbf} \cdot \text{ft}}{lbf \cdot \text{ft}} \text{ of Hinge along wall} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Wood Sheathing Nailing to Resist Shear: (LF UNITS) \[ \phi_D := 0.8 \]

Shearwall:

\[
V := \frac{F_{top}}{L \cdot \phi_D} = 550.342 \text{ plf}
\]

Sheathing:

Structural 1
7/16"
24/16 Span Rating

\[ V_w := 715 \text{ plf} \]
Use 8d nails with 6" spacing along edge

Diaphragm:

\[ V_d := V = 550.342 \text{ plf} \]
\[ V_{wd} := 670 \text{ plf} \quad \text{okay} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

**Wind Parallel To Ridge:**

\[ L := 20 \text{ ft} \quad h := 9.5 \text{ ft} \quad \frac{L}{B} = 2.5 \]

**Wall Pressure Coefficients:**

<table>
<thead>
<tr>
<th>Surface</th>
<th>( L/B )</th>
<th>( C_p )</th>
<th>Use With</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>All values</td>
<td>0.8</td>
<td>( q_a )</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>-0.5</td>
<td>( q_a )</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>2</td>
<td>-0.3</td>
<td>( q_a )</td>
</tr>
<tr>
<td></td>
<td>≥4</td>
<td>-0.2</td>
<td>( q_a )</td>
</tr>
<tr>
<td>Sidewall</td>
<td>All values</td>
<td>-0.7</td>
<td>( q_a )</td>
</tr>
</tbody>
</table>

**Roof Pressure Coefficients:**

**Windward Wall**

\( C_{p_{ww}} := 0.8 \)

**Leeward Wall**

\( C_{p_{lw\_perp}} := -0.3 \)

**Sidewall**

\( C_{p_{sw}} := -0.7 \)

**Roof Pressure Coefficients:**

\( h := 0.475 \)

\( \frac{h}{L} = 0.475 \)

\( C_{p_{r\_perp}} := -0.9 \)

\[ P_{ww\_ext} := q_z \cdot G \cdot C_{p_{ww}} = 27.88 \text{ psf} \]

\[ P_{lw\_ext} := q_z \cdot G \cdot C_{p_{lw\_perp}} = -10.455 \text{ psf} \]

\[ P_{r\_ext} := q_z \cdot G \cdot C_{p_{r\_perp}} = -31.365 \text{ psf} \]

\[ P_{int} := q_z \cdot G C_{pi} = 7.38 \text{ psf} \]

**roof pressure:**

\[ P_r := P_{r\_ext} + -1 \cdot P_{int} = -38.745 \text{ psf} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

**Forces:**

Windward Wall:

\[ P_{ww_{ext}} = 27.88 \text{ psf} \]

\[ F_{ww} = P_{ww_{ext}} \cdot \left( \frac{h}{2} \right) = 132.43 \text{ plf} \]

Line load along top and bottom of wall

Leeward Wall:

\[ P_{lw_{ext}} = -10.455 \text{ psf} \]

\[ F_{lw} = (P_{lw_{ext}} - P_{int}) \cdot \left( \frac{h}{2} \right) = -84.716 \text{ plf} \]

Line load along top and bottom of wall

Roof:

\[ P_{ry} = -51.12 \text{ psf} \]

Each shear wall Par to ridge

\[ F_{top} = \left( F_{ww} - F_{lw} \right) \cdot \left( \frac{B}{2} \right) = 0.869 \text{ kip} \]

- Half of top of wall (ww & lw) line load

\[ F_{bot} = \left( F_{ww} - F_{lw} \right) \cdot \left( \frac{B}{2} \right) = 0.869 \text{ kip} \]

- Half of bot of wall (ww & lw) line load

\[ F_{ry} = P_{ry} \cdot \left( \frac{B}{2} \right) = -204.48 \text{ plf} \]

- Uplift line load

Shear at base of wall:

\[ V_b = F_{top} + F_{bot} = 1.737 \text{ kip} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Load Case:

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \quad (EQ \ 4.3.2-4) \]

Overturning Moments:

\[ L = 20 \text{ ft} \quad B = 8 \text{ ft} \]

Overturning moment from Ftop:

\[ M_{F,\text{top}} := F_{\text{top}} \cdot h = 8.252 \text{ kip} \cdot \text{ft} \]

Overturning moment from Roof Uplift:

\[ M_{F,\text{up}} := -F_{r,y} \cdot L \cdot \frac{L}{2} = 40.896 \text{ kip} \cdot \text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F,\text{top}} + M_{F,\text{up}} - M_r}{L - 1 \text{ ft}} = 0.557 \text{ kip} \]

Required Compression Force:

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 4.413 \text{ kip} \]

Required Studs for compression:

CLT Capacity:

\[ F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ C = 0.883 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 0.143 \cdot \frac{1}{\text{ft}} \cdot \text{ft} \quad \text{of Hinge along wall} \]
## Appendix E. Mathcad Calculations for Wind Loads: Static Units

**Load Case:**

\[0.9 \times DL + W\]  \hfill (EQ 4.3.2-5)

### Overturning Moments:

- **Overturning moment from Ftop:**
  \[M_{F,\text{top}} := F_{\text{top}} \cdot h = 8.252 \, \text{kip} \cdot \text{ft}\]

- **Overturning moment from Roof Uplift:**
  \[M_{F,\text{up}} := \frac{-F_{r,y} \cdot L}{2} = 40.896 \, \text{kip} \cdot \text{ft}\]

### Required force in hold down:

\[T := \frac{M_{F,\text{top}} + M_{F,\text{up}}}{L-1} = 2.587 \, \text{kip}\]

### Required Compression Force:

\[C := T = 2.587 \, \text{kip}\]

### Required Studs for compression:

- **CLT Capacity:**
  \[F'_{c} := 35849 \, \text{plf} = 35.849 \, \text{klf}\]  \hfill Okay

- **Light-Frame Capacity:**
  \[P' := 5 \, \text{kip}\]

\[\frac{C}{P'} = 0.517\]  \hfill Need at least 1 studs in corner for compression

### Required hinge length for base shear:

- **Ultimate Strength of Hinge A in Shear Connections:**
  \[\phi := 0.65\]

\[V'_{n} := 3.85 \, \text{klf}\]

For Base shear we would need:

\[\frac{V}{V'_{n}} = 0.143 \, \frac{1}{\text{ft}} \]  \hfill of Hinge along wall
Appendix E. Mathcad Calculations for Wind Loads: Static Units

<table>
<thead>
<tr>
<th>Wood Sheathing Nailing to Resist Shear: (LF UNITS)</th>
<th>$\phi_D := 0.8$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Shearwall:</strong></td>
<td></td>
</tr>
<tr>
<td>$V := \frac{F_{top}}{L \cdot \phi_D} = 54.287 \text{ plf}$</td>
<td></td>
</tr>
<tr>
<td><strong>Sheathing:</strong></td>
<td></td>
</tr>
<tr>
<td>Structural 1</td>
<td></td>
</tr>
<tr>
<td>7/16&quot;</td>
<td></td>
</tr>
<tr>
<td>24/16 Span Rating</td>
<td></td>
</tr>
<tr>
<td>$V_w := 715 \text{ plf}$</td>
<td></td>
</tr>
<tr>
<td>Use 8d nails woth 6&quot; spacing along edge</td>
<td></td>
</tr>
<tr>
<td><strong>Diaphragm:</strong></td>
<td></td>
</tr>
<tr>
<td>$V_d := V = 54.287 \text{ plf}$</td>
<td></td>
</tr>
<tr>
<td>$V_{wd} := 670 \text{ plf}$</td>
<td>okay</td>
</tr>
</tbody>
</table>
## Components and Cladding (C&C):

Table 30.3-1: Steps to Determine C&C Wind Loads for Enclosed and Partially Enclosed Low-Rise Buildings

**Step 1:** Determine risk category; see Table 1.5-1.
**Step 2:** Determine the basic wind speed, \( V \), for applicable risk category; see Figs. 26.5-1 and 26.5-2.
**Step 3:** Determine wind load parameters:
- Wind directionality factor, \( K_d \), see Section 26.6 and Table 26.6-1.
- Exposure category B, C, or D; see Section 26.7.
- Topographic factor, \( K_t \); see Section 26.8 and Fig. 26.8-1.
- Ground elevation factor, \( K_e \); see Section 25.9 and Table 26.9-1.
- Enclosure classification; see Table 26.12.
- Internal pressure coefficient, \( GC_{pi} \), see Section 26.13 and Table 26.13-1.
**Step 4:** Determine velocity pressure exposure coefficient, \( K_h \); see Table 26.10-1.
**Step 5:** Determine velocity pressure, \( q_h \), Eq. (26.10-1).
**Step 6:** Determine external pressure coefficient, \( GC_p \):
- Walls; see Fig. 30.3-1.
- Flat roofs, gable roofs, hip roofs; see Fig. 30.3-2.
- Stepped roofs; see Fig. 30.3-3.
- Multislope gable roofs; see Fig. 30.3-4.
- Multislope roofs; see Fig. 30.3-5.
- Sawtooth roofs; see Fig. 30.3-6.
- Dormer roofs; see Fig. 30.3-7.
- Atrium roofs; see Fig. 27.3-3, Note 4.
**Step 7:** Calculate wind pressure, \( p_c \); Eq. (30.3-1).

### Walls:

- **Diagram**

#### Notation
- \( a = 10\% \) of least horizontal dimension or 0.46, whichever is smaller, but not less than either 4\% of least horizontal dimension or 3 ft (0.9 m).
- **Exception:** For buildings with \( \theta = 0^\circ \) to \( 7^\circ \) and a least horizontal dimension greater than 300 ft (90 m), the dimension \( a \) should be limited to a maximum of 0.8\%.
- \( h = \) Mean roof height, in ft (m), except that rafter height shall be used for \( \theta \leq 10^\circ \).
- \( \theta = \) Angle of plane of roof from horizontal, in degrees.

\[
\begin{align*}
a &:= min (0.1 \cdot min (B, L), 0.4 \cdot h_{mean}) = 0.8 \text{ ft} \\
a_{min} &:= max (0.04 \cdot min (B, L), 3 \text{ ft}) = 3 \text{ ft} \\
a &:= max (a, a_{min}) = 3 \text{ ft} \\
\theta &:= 3 \text{ deg} \\

Perpendicular to Ridge: & Parallel to Ridge: & \text{*smaller area worse CGp} \\
A_{5,\text{perp}} &:= a \cdot z = 31.5 \text{ ft}^2 & A_{5,\text{par}} &:= z \cdot a = 31.5 \text{ ft}^2 \\
A_{4,\text{perp}} &:= (L - 2 \cdot a) \cdot z = 147 \text{ ft}^2 & A_{4,\text{par}} &:= (B - 2 \cdot a) \cdot z = 21 \text{ ft}^2
\end{align*}
\]

### Calculations

#### Step 1: Risk Category II

#### Step 2: Basic Wind Speed = 180 mph  \( V := 180 \)

#### Step 3:

- **Wind Directionality** \( K_d := 0.85 \)
- **Exposure Category B**
- **Topographic Factor** \( K_t := 1 \)
- **Ground Elevation Factor** \( K_e := 1 \)
- **Enclosure Classification Partially Open**
- **Internal Pressure Coefficient** \( GC_{pi} := 0.18 \)

#### Step 4: Velocity pressure exposure coefficient \( K_h := 0.57 \)

#### Step 5: Velocity pressure

\[
q_h := (0.00256 \cdot K_h \cdot K_t \cdot K_d \cdot K_e \cdot V^2) \cdot psf = 40.186 \text{ psf} \\
q_h := 41 \text{ psf}
\]

#### Step 6: External pressure coefficient

\[
h = 9.5 \text{ ft} \\
z = 10.5 \text{ ft} \\
B = 8 \text{ ft} \\
L = 20 \text{ ft} \\

h_{mean} := \frac{h + z}{2} = 10 \text{ ft}
\]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Positive:
\[ GC_{p\_5\_pos} := 1 \]
\[ GC_{p\_4\_pos} := 1 \]

Negative:
\[ GC_{p\_5\_neg} := -1.4 \]
\[ GC_{p\_4\_neg} := -1.1 \]

Design Pressures:
\[ p_{5\_pos} := q_h \cdot (GC_{p\_5\_pos} + GC_{pi}) = 48.38 \text{ psf} \]
\[ p_{4\_pos} := q_h \cdot (GC_{p\_4\_pos} + GC_{pi}) = 48.38 \text{ psf} \]
\[ p_{5\_neg} := q_h \cdot (GC_{p\_5\_neg} - GC_{pi}) = -64.78 \text{ psf} \]
\[ p_{4\_neg} := q_h \cdot (GC_{p\_4\_neg} - GC_{pi}) = -52.48 \text{ psf} \]
Appendix E. Mathcad Calculations for Wind Loads: Static Units

Roof:

**Diagrams**

**Notation**

- $a = 10\%$ of least horizontal dimension or 0.4$h$, whichever is smaller, but not less than either 4\% of least horizontal dimension or 3 ft (0.9 m).
- $h = \text{Mean roof height, in ft (m)}$.
- $W = \text{Building width, in ft (m)}$.
- $\theta = \text{Angle of plane of roof from horizontal, in degrees}$.

**External Pressure Coefficients**

**Notes**

1. Vertical scale denotes $(GC_p)$ to be used with $q_h$.
2. Horizontal scale denotes effective wind area $A_e$, in ft$^2$ (m$^2$).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
# Appendix E. Mathcad Calculations for Wind Loads: Static Units

<table>
<thead>
<tr>
<th>Area: $A_3 := (2 \cdot a) \cdot (4 \cdot a) = 72 \text{ ft}^2$</th>
<th>$B = 8 \text{ ft}$</th>
<th>$L = 20 \text{ ft}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_2 := 2 \cdot (B - (2 \cdot a)) \cdot a + (L \cdot a) = 72 \text{ ft}^2$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$A_1 := 0 \text{ ft}^2$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive: $GC_{p.3_pos} := 0.4$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$GC_{p.2_pos} := 0.4$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative: $GC_{p.3_neg} := -2.2$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$GC_{p.2_neg} := -1.6$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Design Pressures:**

| $p_{3\_pos} := q_h \cdot (GC_{p.3\_pos} + GC_{pi}) = 23.78 \text{ psf}$ |                   |                   |
| $p_{2\_pos} := q_h \cdot (GC_{p.2\_pos} + GC_{pi}) = 23.78 \text{ psf}$ |                   |                   |
| $p_{3\_neg} := q_h \cdot (GC_{p.3\_neg} + GC_{pi}) = -82.82 \text{ psf}$ |                   |                   |
| $p_{2\_neg} := q_h \cdot (GC_{p.2\_neg} + GC_{pi}) = -58.22 \text{ psf}$ |                   |                   |
Appendix E. Mathcad Calculations for Wind Loads: Static Units

### Sheathing Design: (For Light-Frame Unit)

Wall:

Bending Resistance:
- 24” spacing of joists and studs
- 24/16 span rating 7/16” thick

\[
\begin{align*}
p_{5\text{, pos}} &= 48.38 \text{ psf} \\
p_{4\text{, pos}} &= 48.38 \text{ psf} \\
p_{5\text{, neg}} &= -64.78 \text{ psf} \\
p_{4\text{, neg}} &= -52.48 \text{ psf}
\end{align*}
\]

\[
l_b = 24 \text{ in}
\]

\[
p_{\text{max}} := \max (p_{5\text{, pos}}, p_{4\text{, pos}}, p_{5\text{, neg}}, p_{4\text{, neg}}) = 48.38 \text{ psf}
\]

\[
p_{\text{min}} := \min (p_{5\text{, pos}}, p_{4\text{, pos}}, p_{5\text{, neg}}, p_{4\text{, neg}}) = -64.78 \text{ psf}
\]

### Table 3.2.1 Nominal Uniform Load Capacities (psf) for Wall Sheathing Resisting Out-of-Plane Wind Loads

<table>
<thead>
<tr>
<th>Sheathing Type</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in)</th>
<th>Perpendicular to Supports</th>
<th>Parallel to Supports</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Maximum Stud Spacing (in.)</td>
<td>Actual Stud Spacing</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>12</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Nominal Uniform Loads (psf)</td>
<td>(in.)</td>
</tr>
<tr>
<td>Wood Structural Panels</td>
<td>24/0</td>
<td>3/8</td>
<td>24</td>
<td>425</td>
</tr>
<tr>
<td>(Sheathing Grades, C-C, C-D, C, C, Plugged, OSB)</td>
<td>24/16</td>
<td>7/16</td>
<td>24</td>
<td>540</td>
</tr>
<tr>
<td></td>
<td>15/32</td>
<td>24</td>
<td>625</td>
<td>355</td>
</tr>
<tr>
<td></td>
<td>40/20</td>
<td>19/32</td>
<td>24</td>
<td>955</td>
</tr>
<tr>
<td></td>
<td>48/24</td>
<td>23/32</td>
<td>24</td>
<td>1160</td>
</tr>
<tr>
<td>Particleboard Sheathing (M-S Exterior Glue)</td>
<td>3/8</td>
<td>16</td>
<td>16</td>
<td>(contact manufacturer)</td>
</tr>
<tr>
<td></td>
<td>1/2</td>
<td>16</td>
<td>16</td>
<td>(contact manufacturer)</td>
</tr>
<tr>
<td>Particleboard Panel Siding (M-S Exterior Glue)</td>
<td>5/8</td>
<td>16</td>
<td>16</td>
<td>(contact manufacturer)</td>
</tr>
<tr>
<td></td>
<td>3/4</td>
<td>16</td>
<td>16</td>
<td>(contact manufacturer)</td>
</tr>
<tr>
<td>Hardboard Siding (Direct to Studs)</td>
<td>7/16</td>
<td>16</td>
<td>16</td>
<td>460</td>
</tr>
<tr>
<td></td>
<td>7/16</td>
<td>16</td>
<td>16</td>
<td>460</td>
</tr>
<tr>
<td></td>
<td>7/16</td>
<td>16</td>
<td>16</td>
<td>460</td>
</tr>
<tr>
<td>Cellulosic Fiberboard Sheathing</td>
<td>Regular</td>
<td>1/2</td>
<td>16</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td>Structural</td>
<td>1/2</td>
<td>16</td>
<td>135</td>
</tr>
<tr>
<td></td>
<td>Structural</td>
<td>25/32</td>
<td>16</td>
<td>165</td>
</tr>
</tbody>
</table>

1. Nominal capacities shall be adjusted in accordance with Section 3.2.1 to determine ASD uniform load capacity and LRFD uniform resistance.
2. Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood or 3 or more plies.
3. Tabulated values are based on the lesser of nominal values for either OSB or plywood or 4 or more plies.
4. Wood structural panels shall conform to the requirements for its type in DOC PS 1 or PS 2. Particleboard sheathing shall conform to ANSI A208.1. Hardboard panel and siding shall conform to the requirements of ANSI/CPA A135.6. Cellulosic fiberboard sheathing shall conform to ASTM C 208.
5. Tabulated values are for maximum bending loads from wind. Loads are limited by bending or shear stress assuming a 2-span continuous condition. Where panels are continuous over 3 or more spans the tabulated values shall be permitted to be increased in accordance with the ASD/LRFD Manual for Engineered Wood Construction.
6. Strength axis is defined as the axis parallel to the face and back orientation of the flakes or the grain (veneer), which is generally the long panel direction, unless otherwise marked.

**Must Span strength direction perpendicular to studs**

**Must have at least two spans continuous**

\[
\phi_b := 0.85
\]

\[
R := 135 \text{ psf} \cdot \phi_b = 114.75 \text{ psf} > \text{ max pressure}
\]
Withdrawal Resistance:

\[ N_s = \frac{4 \text{ ft}}{6 \text{ in}} = 8 \quad \text{number of nails per stud} \]
\[ N = N_s \cdot 4 = 32 \quad \text{nails per panel} \]
\[ p_{\text{min}} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -2072.96 \text{ lbf} \]
\[ W := 41 \frac{\text{lbf}}{\text{in}} \quad L_d := 2.5 \text{ in} \quad L_m := 2.5 \text{ in} - \frac{7}{16} \text{ in} = 2.063 \text{ in} \]
\[ W_{sd} := W \cdot L_m = 84.563 \text{ lbf} \quad K_{fw} := 3.32 \quad \phi_w := 0.65 \]
\[ W'_{sd} := W_{sd} \cdot K_{fw} \cdot \phi_w = 182.486 \text{ lbf} \]
\[ W''_{sd} \cdot N = 5839.548 \text{ lbf} \quad \text{Capacity > Load} \]

Roof:

Bending Resistance:

<table>
<thead>
<tr>
<th>Sheathing Type(^1)</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in)</th>
<th>Strength Axis(^2) Applied Perpendicular to Supports</th>
<th>Strength Axis(^2) Applied Parallel to Supports</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Rafter/Truss Spacing (in)</td>
<td>Rafter/Truss Spacing (in)</td>
</tr>
<tr>
<td>Wood Structural Panels</td>
<td>Sheathing Grades, C-C</td>
<td>24/0</td>
<td>3/8</td>
<td>425</td>
</tr>
<tr>
<td></td>
<td>C-D, C-C Plugged, OSB</td>
<td>24/16</td>
<td>7/16</td>
<td>540</td>
</tr>
<tr>
<td></td>
<td>32/16</td>
<td>15/32</td>
<td>625</td>
<td>355</td>
</tr>
<tr>
<td></td>
<td>40/29</td>
<td>19/32</td>
<td>955</td>
<td>595</td>
</tr>
<tr>
<td></td>
<td>49/24</td>
<td>23/32</td>
<td>1160</td>
<td>840</td>
</tr>
<tr>
<td>Wood Structural Panels</td>
<td>(Single Floor Grades, C-C Plugged)</td>
<td>16 o.c.</td>
<td>19/32</td>
<td>705</td>
</tr>
<tr>
<td></td>
<td>20 o.c.</td>
<td>19/32</td>
<td>815</td>
<td>455</td>
</tr>
<tr>
<td></td>
<td>24 o.c.</td>
<td>23/32</td>
<td>1160</td>
<td>870</td>
</tr>
<tr>
<td></td>
<td>32 o.c.</td>
<td>7/8</td>
<td>1395</td>
<td>1000</td>
</tr>
<tr>
<td></td>
<td>48 o.c.</td>
<td>1-1/8</td>
<td>1750</td>
<td>1395</td>
</tr>
</tbody>
</table>

1. Nominal capacities shall be adjusted in accordance with Section 3.2.3 to determine ASD uniform load capacity and LRFD uniform resistances.
2. Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
3. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
4. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
5. Tabulated values are for maximum bending loads from wind. Loads are limited by bending or shear stress assuming a 2-span continuous condition. Where panels are continuous over 3 or more spans, the tabulated values shall be permitted to be increased in accordance with the ASD-LRFD Manual for Engineered Wood Construction.
6. Strength axis is defined as the axis parallel to the face and back orientation of the flakes or the grain (vesse), which is generally the long panel direction, unless otherwise marked.
Appendix E. Mathcad Calculations for Wind Loads: Static Units

\[ p_{3,\text{pos}} = 23.78 \text{ psf} \]
\[ p_{2,\text{pos}} = 23.78 \text{ psf} \]

\[ p_{\text{max}} := \max (p_{3,\text{pos}}, p_{2,\text{pos}}) = 23.78 \text{ psf} \]

\[ p_{3,\text{neg}} = -82.82 \text{ psf} \]
\[ p_{2,\text{neg}} = -58.22 \text{ psf} \]

\[ p_{\text{min}} := \min (p_{3,\text{neg}}, p_{2,\text{neg}}) = -82.82 \text{ psf} \]

**Must Span strength direction perpendicular to studs**
**Must have at least two spans continuous**

\[ \phi_{b} := 0.85 \]
\[ R := 135 \text{ psf} \cdot \phi_{b} = 114.75 \text{ psf} > \text{max pressure} \]

Withdrawal Resistance:

\[ N_{s} := \frac{4 \text{ ft}}{6 \text{ in}} = 8 \text{ number of nails per stud} \]

\[ N := N_{s} \cdot 4 = 32 \text{ nails per panel} \]
\[ p_{\text{min}} = -82.82 \text{ psf} \]
\[ p_{\text{min}} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -2650.24 \text{ lbf} \]
\[ W := 41 \text{ lbf} \text{ in} \]
\[ L_{d} := 2.5 \text{ in} \]
\[ L_{m} := 2.5 \text{ in} - \frac{1}{2} \text{ in} = 2 \text{ in} \]

\[ W_{\text{sd}} := W \cdot L_{m} = 82 \text{ lbf} \]
\[ K_{fw} := 3.32 \]
\[ \phi_{w} := 0.65 \]
\[ W'_{\text{sd}} := W_{\text{sd}} \cdot K_{fw} \cdot \phi_{w} = 176.956 \text{ lbf} \]
\[ W'_{\text{sd}} \cdot N = 5662.592 \text{ lbf} \]

Capacity > Load
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

115 MPH:

Assumptions:
- Partially Open Enclosure Classification
- Roof Slope 3 Degrees

Step 1: Risk Category II
Step 2: Basic Wind Speed = 115 mph $V := 115$
Step 3:

<table>
<thead>
<tr>
<th>Assumption</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind Directionality Factor, $K_d$</td>
<td>0.85</td>
</tr>
<tr>
<td>Exposure Category B</td>
<td></td>
</tr>
<tr>
<td>Topographic Factor, $K_{zt}$</td>
<td>1</td>
</tr>
<tr>
<td>Ground Elevation Factor, $K_e$</td>
<td>1</td>
</tr>
<tr>
<td>Gust-effect Factor, $G$</td>
<td>0.85</td>
</tr>
<tr>
<td>Enclosure Classification Partially Open</td>
<td></td>
</tr>
<tr>
<td>Internal Pressure Coefficient, $GC_{pi}$</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Step 4: Velocity pressure exposure coefficient $K_z := 0.57$

Step 5: Velocity pressure $q_z := (0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2) \cdot psf = 16.403 \text{ psf}$

Use: $q_z := 16.5 \text{ psf}$

Step 6: External pressure coefficient

Table 27.2-1 Steps to Determine MWFRS Wind Loads for Enclosed, Partially Enclosed, and Open Buildings of All Heights

Notation
- $B$ = Horizontal dimension of building, in ft (m), measured normal to wind direction.
- $L$ = Horizontal dimension of building, in ft (m), measured parallel to wind direction.
- $h$ = Mean roof height, in ft (m), except that eave height shall be used for $0 \leq 10$ degrees.
- $z$ = Height above ground, in ft (m).
- $G$ = Gust-effect factor.
- $q_z$, $q_h$ = Velocity pressure, in lb/ft$^2$ (N/m$^2$), evaluated at respective height.
- $\theta$ = Angle of plane of roof from horizontal, in degrees.
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

**Wind Perpendicular to Ridge:**

\[ L := 19.5 \text{ ft} \quad h := 8.5 \text{ ft} \]
\[ B := 20 \text{ ft} \quad z := 7.5 \text{ ft} \]

Height to peak of roof

\[ \frac{L}{B} = 0.975 \]

Height to eave

**Wall Pressure Coefficients:**

<table>
<thead>
<tr>
<th>Surface</th>
<th>( L/B )</th>
<th>( C_p )</th>
<th>Use With</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>All values</td>
<td>0.8</td>
<td>( q_r )</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>0.1</td>
<td>-0.5</td>
<td>( q_a )</td>
</tr>
<tr>
<td>Sidewall</td>
<td>2</td>
<td>-0.3</td>
<td>( q_s )</td>
</tr>
</tbody>
</table>

**Roof Pressure Coefficients:**

\[ C_{p_{r\_perp}} := -0.9 \]

\[ \frac{h}{L} = 0.436 \]

\[ C_{p_{r\_perp}} := -0.9 \]

\[ P_{r\_ext} := q_z \cdot G \cdot C_{p_{r\_perp}} = 12.623 \text{ psf} \]

\[ P_{r\_int} := q_z \cdot G \cdot C_{p_{r\_perp}} = 2.97 \text{ psf} \]

\[ P_r := P_{r\_ext} + \frac{1}{2} P_{r\_int} = 15.593 \text{ psf} \]

*Roof pressures will cancel in X-direction*
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Forces:

Windward Wall:

\[ P_{ww\_ext} = 11.22 \text{ psf} \]

\[ F_{ww} = P_{ww\_ext} \cdot \left( \frac{h}{2} \right) = 47.685 \text{ plf} \]

Leeward Wall:

\[ P_{lw\_ext} = -7.013 \text{ psf} \]

\[ P_{int} = 2.97 \text{ psf} \]

\[ F_{lw} = (P_{lw\_ext} - P_{int}) \cdot \left( \frac{h}{2} \right) = -42.426 \text{ plf} \]

Roof:

\[ P_{r} = -15.593 \text{ psf} \]

x-comp:

\[ P_{rx} := P_{r} \cdot \sin(3 \text{ deg}) = -0.816 \text{ psf} \]

y-comp:

\[ P_{ry} := P_{r} \cdot \cos(3 \text{ deg}) = -15.571 \text{ psf} \]

Each shear wall Perp to ridge

Ftop:

- Half of top of wall (ww & lw) line load

\[ F_{top} := \left( F_{ww} - F_{lw} \right) \cdot \left( \frac{B}{2} \right) = 0.901 \text{ kip} \]

Fbot:

- Half of bot of wall (ww & lw) line load

\[ F_{bot} := \left( F_{ww} - F_{lw} \right) \cdot \left( \frac{B}{2} \right) = 0.901 \text{ kip} \]

Fry:

- Uplift line load

\[ F_{r\_y} := P_{ry} \cdot \left( \frac{B}{2} \right) = -155.711 \text{ plf} \]

Shear at base of wall:

\[ V_{b} := F_{top} + F_{bot} = 1.8 \text{ kip} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Load Case:

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \]  \quad \text{(EQ 4.3.2-4)}

Overturning Moments:

\[ L = 5.944 \text{ m} \quad B = 6.096 \text{ m} \]

Overturning moment from Ftop:

\[ M_{F,\text{top}} := F_{\text{top}} \cdot z = 6.758 \text{ kip} \cdot \text{ft} \]

Overturning moment from Roof Uplift:

\[ M_{F,\text{up}} := -F_{r,y} \cdot 22 \text{ ft} \cdot \frac{L}{2} = 33.4 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:

\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot \frac{L}{2} = 91.64 \text{ kip} \cdot \text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F,\text{top}} + M_{F,\text{up}} - M_r}{L - 1 \text{ ft}} = -2.783 \text{ kip} \]

Required Compression Force:

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 6.616 \text{ kip} \]

Required Studs for compression:

CLT Capacity:

\[ F'_{c} := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ C = 1.323 \quad \text{Need at least 2 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 0.007 \cdot \frac{1}{N} \text{ ft} \quad \text{of Hinge along wall} \]
## Load Case:

$$0.9 \times DL + W \quad (EQ \ 4.3.2-5)$$

### Overturning Moments:

$$L = 5.944 \ m \quad B = 6.096 \ m$$

- Overturning moment from Ftop:
  $$M_{F,\text{top}} := F_{\text{top}} \cdot z = 6.758 \ kip \cdot ft$$

- Overturning moment from Roof Uplift:
  $$M_{F,\text{up}} := -F_{r,y} \cdot 22 \ ft \cdot \frac{L}{2} = 33.4 \ kip \cdot ft$$

#### Required force in hold down:

$$T := \frac{M_{F,\text{top}} + M_{F,\text{up}}}{L - 1 \ ft} = 2.171 \ kip$$

#### Required Compression Force:

$$C := T = 2.171 \ kip$$

### Required Studs for compression:

- CLT Capacity:
  $$F'_{c} := 35849 \ plf = 35.849 \ klf \quad \text{Okay}$$

- Light-Frame Capacity:
  $$P' := 5 \ kip$$

$$\frac{C}{P'} = 0.434 \quad \text{Need at least 1 studs in corner for compression}$$

### Required hinge length for base shear:

- Ultimate Strength of Hinge A in Shear
  $$\phi := 0.65 \quad \text{Connections}$$

  $$V'_{n} := 3.85 \ klf$$

For Base shear we would need:

$$\frac{V}{V'_{n}} = 0.007 \ \frac{1}{N} \ ft \quad \text{of Hinge along wall}$$
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Wood Sheathing Nailing to Resist Shear: (LF UNITS) \( \phi_D := 0.8 \)

Shearwall:
\[
V := \frac{F_{\text{top}}}{L \cdot \phi_D} = 57.763 \text{ plf}
\]

Sheathing:
Structural 1
7/16''
24/16 Span Rating

### Table 4.3A Nominal Unit Shear Capacity

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Fastener Penetration in Framing Member or Blocking (in.)</th>
<th>Fastener Type &amp; Size</th>
<th>B Wind</th>
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<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>( V_0 ) (plf)</td>
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<tr>
<td>Wood Structural Panels - Structural</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>560</td>
</tr>
<tr>
<td>Structural 1</td>
<td>7/16</td>
<td>1-3/8</td>
<td>8d</td>
<td>645</td>
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<tr>
<td></td>
<td>19/32</td>
<td>1-3/8</td>
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<td>715</td>
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<tr>
<td></td>
<td>19/32</td>
<td>1-1/2</td>
<td>10d</td>
<td>765</td>
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<tr>
<td>7/16''</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>505</td>
</tr>
<tr>
<td></td>
<td>7/16</td>
<td>1-3/8</td>
<td>8d</td>
<td>615</td>
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<td></td>
<td>19/32</td>
<td>1-3/8</td>
<td>8d</td>
<td>670</td>
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<td></td>
<td>19/32</td>
<td>1-1/2</td>
<td>10d</td>
<td>730</td>
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<tr>
<td>Plywood Siding</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>390</td>
</tr>
<tr>
<td></td>
<td>3/8</td>
<td>1-3/8</td>
<td>8d</td>
<td>450</td>
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<tr>
<td>Particleboard Sheathing - (M-S &quot;Exterior Glue&quot; and M-2 &quot;Exterior Glue&quot;)</td>
<td>3/8</td>
<td>1-1/4</td>
<td>6d</td>
<td>335</td>
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<tr>
<td></td>
<td>3/8</td>
<td>1/2</td>
<td>8d</td>
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<td>5/16</td>
<td>1/2</td>
<td>10d</td>
<td>502</td>
</tr>
<tr>
<td>Structural Fiberboard Sheathing</td>
<td>1/2</td>
<td>1/2</td>
<td>10d</td>
<td>560</td>
</tr>
<tr>
<td></td>
<td>25/32</td>
<td>11 ga galv. roofing nail (0.120'' x 1-1/2'' long x 7/16'' head)</td>
<td>11 ga galv. roofing nail (0.120'' x 1-3/4'' long x 3/8'' head)</td>
<td>745</td>
</tr>
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<td></td>
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<td></td>
<td>475</td>
</tr>
</tbody>
</table>

\( V_{ud} := 715 \text{ plf} \)

Use 8d nails with 6" spacing along edge

Diaphragm:
\( V_d := V = 57.763 \text{ plf} \)
\( V_{ud} := 670 \text{ plf} \) okay
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

### Wind Parallel To Ridge:

\[ L := 20 \text{ ft} \quad h := 8.5 \text{ ft} \quad B := 19.5 \text{ ft} \quad z := 7.5 \text{ ft} \]

- Height to peak & used for wall height entire length
- Moment arm for overturning moment from F top

### Wall Pressure Coefficients:

\[ \frac{L}{B} = 1.026 \]

#### Windward Wall

- \( C_{p_{ww}} := 0.8 \)

#### Leeward Wall

- \( C_{p_{lw\_perm}} := -0.5 \)

#### Sidewall

- \( C_{p_{sw}} := -0.7 \)

### Roof Pressure Coefficients:

\[ \frac{h}{L} = 0.425 \]

- \( C_{p_{r\_perm}} := -0.9 \)

### Roof Pressure Coefficients:

\[ P_{ww\_ext} = q_z \cdot G \cdot C_{p_{ww}} = 11.22 \text{ psf} \]

\[ P_{lw\_ext} = q_z \cdot G \cdot C_{p_{lw\_perm}} = -7.013 \text{ psf} \]

\[ P_{r\_ext} = q_z \cdot G \cdot C_{p_{r\_perm}} = -12.623 \text{ psf} \]

\[ P_{int} = q_z \cdot G C_{p_{i}} = 2.97 \text{ psf} \]

- Roof pressure: \( P_r := P_{r\_ext} + -1 \cdot P_{int} = -15.593 \text{ psf} \)
Forces:

Windward Wall:

\[ P_{ww_{ext}} = 11.22 \text{ psf} \]

\[ F_{ww} := P_{ww_{ext}} \cdot \left( \frac{h}{2} \right) = 47.685 \text{ plf} \]

Line load along top and bottom of wall

Leeward Wall:

\[ P_{lw_{ext}} = -7.013 \text{ psf} \]

\[ F_{lw} := (P_{lw_{ext}} - P_{int}) \cdot \left( \frac{h}{2} \right) = -42.426 \text{ plf} \]

Line load along top and bottom of wall

Roof:

\[ P_{ry} := P_r = -15.593 \text{ psf} \]

Each shear wall Par to ridge

- \( F_{top} := \frac{F_{ww} - F_{lw}}{2} \cdot \left( \frac{B}{2} \right) = 0.879 \text{ kip} \)

- \( F_{bot} := \frac{F_{ww} - F_{lw}}{2} \cdot \left( \frac{B}{2} \right) = 0.879 \text{ kip} \)

Fry:

- \( F_{r,y} := P_{ry} \cdot \left( \frac{B}{2} \right) = -152.027 \text{ plf} \)

Shear at base of wall:

\[ V_b := F_{top} + F_{bot} = 1.757 \text{ kip} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Load Case:

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \]  

Overturning Moments:
\[ L = 6.096 \, m \quad B = 5.944 \, m \]
\[ DL_r = 23.5 \, psf \]
\[ LL_r = 20 \, psf \]
\[ Trib = \frac{B}{2} \]
\[ M_F_{top} := F_{top} \cdot z = 6.589 \, kip \cdot ft \]
\[ M_F_{up} := -F_{r, y} \cdot 22 \, ft \cdot \frac{L}{2} = 33.446 \, kip \cdot ft \]
\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 93.99 \, kip \cdot ft \]

Required force in hold down:
\[ T := \frac{M_F_{top} + M_F_{up} - M_r}{L - 1 \, ft} = -2.84 \, kip \]

Required Compression Force:
\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 6.559 \, kip \]

Required Studs for compression:
CLT Capacity:
\[ F'_{c} = 35849 \, plf = 35.849 \, klf \]
Okay
Light-Frame Capacity:
\[ P' := 5 \, kip \]
\[ C = 1.312 \quad \text{Need at least 2 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_n := 3.85 \, klf \]

For Base shear we would need:
\[ \frac{V}{V'_n} = 0.049 \, \frac{1}{m} \cdot ft \quad \text{of Hinge along wall} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Load Case:

\[ 0.9 \times DL + W \]  \hspace{1cm} (EQ 4.3.2-5)

Overturning Moments:

\[ L = 6.096 \ m \quad B = 5.944 \ m \]

Overturning moment from Ftop:

\[ M_{F,\text{top}} := F_{\text{top}} \cdot z = 6.589 \ \text{kip} \cdot \text{ft} \]

Overturning moment from Roof Uplift:

\[ M_{F,\text{up}} := F_{\text{r,y}} \cdot 22 \ \text{ft} \cdot \frac{L}{2} = 33.446 \ \text{kip} \cdot \text{ft} \]

**Required force in hold down:**

\[ T := \frac{M_{F,\text{top}} + M_{F,\text{up}}}{L - 1 \ \text{ft}} = 2.107 \ \text{kip} \]

\[ M_{F,\text{top}} + M_{F,\text{up}} = 40.035 \ \text{kip} \cdot \text{ft} \]

**Required Compression Force:**

\[ C := T = 2.107 \ \text{kip} \]

**Required Studs for compression:**

CLT Capacity:

\[ F' := 35849 \ \text{plf} = 35.849 \ \text{klf} \quad \text{Okay} \]

Light-Frame Capacity:

\[ P' := 5 \ \text{kip} \]

\[ C = 0.421 \quad \text{Need at least 1 studs in corner for compression} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V' := 3.85 \ \text{klf} \]

For Base shear we would need:

\[ \frac{V}{V'} = 0.049 \ \text{ft} \quad \text{of Hinge along wall} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Wood Sheathing Nailing to Resist Shear: (LF UNITS) \( \phi_D := 0.8 \)

Shearwall:

\[
V := \frac{F_{top}}{L \cdot \phi_D} = 54.911 \text{ plf}
\]

Sheathing:
Structural 1
7/16"
24/16 Span Rating

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Fastener Penetration in Framing Member or Blocking (in.)</th>
<th>Fastener Type &amp; Size</th>
<th>B Wind Panel Edge Fastener Spacing (in.) 6</th>
<th>4</th>
<th>3</th>
<th>2</th>
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</thead>
<tbody>
<tr>
<td>Wood Structural Panels - Structural</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>560</td>
<td>840</td>
<td>1090</td>
<td>1430</td>
</tr>
<tr>
<td></td>
<td>7/16</td>
<td>1-3/8</td>
<td>6d</td>
<td>645</td>
<td>1010</td>
<td>1290</td>
<td>1710</td>
</tr>
<tr>
<td></td>
<td>1 1/2</td>
<td>10d</td>
<td>1532</td>
<td>950</td>
<td>1430</td>
<td>1860</td>
<td>2435</td>
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<tr>
<td>Wood Structural Panels - Sheathing</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>505</td>
<td>755</td>
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<td>7/16</td>
<td>1-3/8</td>
<td>6d</td>
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<td>895</td>
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<td>1405</td>
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<tr>
<td></td>
<td>1 1/2</td>
<td>10d</td>
<td>1532</td>
<td>730</td>
<td>1065</td>
<td>1370</td>
<td>1790</td>
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<td>1-1/4</td>
<td>6d</td>
<td>870</td>
<td>1290</td>
<td>1680</td>
<td>2155</td>
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<td>1-3/8</td>
<td>6d</td>
<td>950</td>
<td>1430</td>
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<td>2435</td>
</tr>
<tr>
<td>Particleboard Sheathing - (M.S &quot;Exterior Glue&quot; and M.2 &quot;Exterior Glue&quot;)</td>
<td>3/8</td>
<td>1-1/4</td>
<td>6d</td>
<td>335</td>
<td>505</td>
<td>645</td>
<td>840</td>
</tr>
<tr>
<td></td>
<td>1/2</td>
<td>6d</td>
<td>365</td>
<td>530</td>
<td>670</td>
<td>880</td>
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<tr>
<td></td>
<td>1/2</td>
<td>10d</td>
<td>390</td>
<td>590</td>
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<td>980</td>
<td></td>
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<tr>
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<td>11/2</td>
<td>10d</td>
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<td>770</td>
<td>1010</td>
<td>1290</td>
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<tr>
<td></td>
<td>25/32</td>
<td>11 ga. galv. roofing nail (0.120&quot; x 1-1/2&quot; long x 7/16&quot; head)</td>
<td>475</td>
<td>645</td>
<td>730</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>11 ga. galv. roofing nail (0.120&quot; x 1-3/4&quot; long x 3/8&quot; head)</td>
<td>475</td>
<td>645</td>
<td>730</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ V_{w} := 715 \text{ plf} \]

Use 8d nails woth 6" spacing along edge

Diaphragm:

\[ V_{d} := V = 54.911 \text{ plf} \]

\[ V_{ud} := 670 \text{ plf} \quad \text{okay} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Components and Cladding (C&C):

Table 30.3-1 Steps to Determine C&C Wind Loads for Enclosed and Partially Enclosed Low-Rise Buildings

| Step 1: Determine risk category; see Table 1.5.1. |
| Step 2: Determine the basic wind speed, V, for applicable risk category; see Figs. 26.5-1 and 26.5-2. |
| Step 3: Determine wind load parameters: |
| – Wind directionality factor, \( K_d \); see Section 26.6 and Table 26.6-1. |
| – Exposure category B, C, or D; see Section 26.7. |
| – Topographic factor, \( K_z \); see Section 26.8 and Fig. 26.8-1. |
| – Ground elevation factor, \( K_e \); Section 26.9 and Table 26.9-1. |
| – Enclosure classification; see Section 26.12. |
| – Internal pressure coefficient, \( GC_{pi} \); see Section 26.13 and Table 26.13-1. |

Step 4: Determine velocity pressure exposure coefficient, \( K_h \); see Table 26.10-1.

Step 5: Determine velocity pressure, \( q_h \), Eq. (26.10-1).

Step 6: Determine external pressure coefficient, \( GC_p \):

- Walls; see Fig. 30.3-1.
- Flat roofs, gable roofs, hip roofs; see Fig. 30.3-2.
- Stepped roofs; see Fig. 30.3-3.
- Multispan gable roofs; see Fig. 30.3-4.
- Monoslope roofs; see Fig. 30.3-5.
- Sawtooth roofs; see Fig. 30.3-6.
- Domed roofs; see Fig. 30.3-7.
- Arched roofs; see Fig. 27.3-3, Not 4.

Step 7: Calculate wind pressure, \( p \); Eq. (30.3-1).

Walls:

Diagram

Notation

\( a = 10\% \) of least horizontal dimension or 0.4b, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

Exception: For buildings with \( \theta = 0^\circ \) to \( 7^\circ \) and a least horizontal dimension greater than 300 ft (90 m), dimension \( a \) shall be limited to a maximum of 0.8b.

\( h = \) Mean roof height, in ft (m), except that eave height shall be used for \( \theta \leq 10^\circ. \)

\( \theta = \) Angle of plane of roof from horizontal, in degrees.

\( \theta ^\circ = 3 \) deg

Perpendicular to Ridge:

Perpendicular to Ridge: *smaller area worse CGp

\( A_{\text{5_perp}} := a \cdot z = 2.09 \ m^2 \)

\( A_{\text{4_perp}} := (L - 2 \cdot a) \cdot z = 9.755 \ m^2 \)

Parallel to Ridge:

Parallel to Ridge:

\( A_{\text{5_par}} := z \cdot a = 2.09 \ m^2 \)

\( A_{\text{4_par}} := (B - 2 \cdot a) \cdot z = 9.406 \ m^2 \)

Step 1: Risk Category II

Step 2: Basic Wind Speed = 115 mph \( V := 115 \)

Step 3:

- Wind Directionality \( K_d := 0.85 \)
- Exposure Category B
- Topographic Factor \( K_z := 1 \)
- Ground Elevation Factor \( K_e := 1 \)
- Enclosure Classification Partially Open

Internal Pressure Coefficient \( GC_{pi} := 0.18 \)

Step 4: Velocity pressure exposure coefficient \( K_h := 0.57 \)

Step 5: Velocity pressure

\( q_h := (0.00256 \cdot K_h \cdot K_z \cdot K_d \cdot K_e \cdot V^2) \cdot psf = 16.403 \ psf \)

\( q_h := 16.5 \ psf \)

Step 6: External pressure coefficient

\( h = 2.591 \ m \)

\( z = 2.286 \ m \)

\( B = 5.944 \ m \)

\( L = 6.096 \ m \)

\( h_{\text{mean}} := \frac{h + z}{2} = 2.438 \ m \)
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

### Notes
1. Vertical scale denotes \((GC_p)\) to be used with \(q_h\).
2. Horizontal scale denotes effective wind area, in \(\text{ft}^2\) (\(\text{m}^2\)).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of \((GC_p)\) for walls shall be reduced by 10% when \(\theta \leq 10^\circ\).

#### Positive:
\[
GC_{p, 5\_pos} := 1 \cdot 0.9 = 0.9
\]
\[
GC_{p, 4\_pos} := 0.9 \cdot 0.9 = 0.81
\]

#### Negative:
\[
GC_{p, 5\_neg} := -1.4 \cdot 0.9 = -1.26
\]
\[
GC_{p, 4\_neg} := -1 \cdot 0.9 = -0.9
\]

#### Design Pressures:
\[
p_{5\_pos} := q_h \cdot (GC_{p, 5\_pos} + GC_{pi}) = 17.82 \text{ psf}
\]
\[
p_{4\_pos} := q_h \cdot (GC_{p, 4\_pos} + GC_{pi}) = 16.335 \text{ psf}
\]
\[
p_{5\_neg} := q_h \cdot (GC_{p, 5\_neg} - GC_{pi}) = -23.76 \text{ psf}
\]
\[
p_{4\_neg} := q_h \cdot (GC_{p, 4\_neg} - GC_{pi}) = -17.82 \text{ psf}
\]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Roof:

**Diagram**

**Notation**

\[
\sigma = 10\% \text{ of least horizontal dimension or } 0.4h, \text{ whichever is smaller, but not less than either } 4\% \text{ of least horizontal dimension or } 3 \text{ ft (0.9 m)}.
\]

**Exception:** For buildings with \( \theta = 0^\circ \) to \( 7^\circ \) and a least horizontal dimension greater than 300 ft (90 m), dimension \( \sigma \) shall be limited to a maximum of 0.8h.

\( h = \text{Mean roof height, in ft (m), except that eave height shall be used for } \theta \leq 10^\circ. \)

\( \theta = \text{Angle of plane of roof from horizontal, in degrees.} \)

**External Pressure Coefficients**

**Notes**

1. Vertical scale denotes \((GC_p)\) to be used with \(q_v\).
2. Horizontal scale denotes effective wind area, in \(ft^2 (m^2)\).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. If a parapet equal to or higher than 3 ft (0.9 m) is provided around the perimeter of the roof with \( \theta \leq 7^\circ \), the negative values of \((GC_p)\) in Zone 3 shall be equal to those for Zone 2, and positive values of \((GC_p)\) in Zones 2 and 3 shall be set equal to those for wall Zones 4 and 5, respectively, in Fig. 30.3-1.
6. Values of \((GC_p)\) for roof overhangs include pressure contributions from both upper and lower surfaces.
7. If overhangs exist, the lesser horizontal dimension of the building shall not include any overhang dimension, but the edge distance, \(a\), shall be measured from the outside edge of the overhang.
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Area:
\[ A_3 := ((0.6 \cdot h) \cdot (0.2 \cdot h) \cdot 2) - (0.2 \cdot h)^2 = 1.342 \ m^2 \]
\[ A_2 := (2 \cdot B \cdot 0.6 \cdot h) + (2 \cdot L \cdot 0.6 \cdot h) - 4 \cdot (0.6 \cdot h)^2 - 4 \cdot A_3 = 22.395 \ m^2 \]
\[ A_{1,prime} := (B - 2.4 \cdot h) \cdot (L - 2.4 \cdot h) = 0.033 \ m^2 \]
\[ A_1 := (B \cdot L) - A_{1,prime} - A_2 - 4 \cdot A_3 = 8.434 \ m^2 \]

Positive:
\[ GC_{p,3\_pos} := 0.3 \]
\[ GC_{p,2\_pos} := 0.2 \]
\[ GC_{p,1\prime\_pos} := 0.2 \]
\[ GC_{p,1\_pos} := 0.3 \]

Negative:
\[ GC_{p,3\_neg} := -3.2 \]
\[ GC_{p,2\_neg} := -1.6 \]
\[ GC_{p,1\prime\_neg} := -0.9 \]
\[ GC_{p,1\_neg} := -1.4 \]

Design Pressures:
\[ p_{3\_pos} := q_h \cdot (GC_{p,3\_pos} + GC_{pi}) = 7.92 \ \text{psf} \]
\[ p_{2\_pos} := q_h \cdot (GC_{p,2\_pos} + GC_{pi}) = 6.27 \ \text{psf} \]
\[ p_{1\prime\_pos} := q_h \cdot (GC_{p,1\prime\_pos} + GC_{pi}) = 6.27 \ \text{psf} \]
\[ p_{1\_pos} := q_h \cdot (GC_{p,1\_pos} + GC_{pi}) = 7.92 \ \text{psf} \]
\[ p_{3\_neg} := q_h \cdot (GC_{p,3\_neg} + GC_{pi}) = -49.83 \ \text{psf} \]
\[ p_{2\_neg} := q_h \cdot (GC_{p,2\_neg} + GC_{pi}) = -23.43 \ \text{psf} \]
\[ p_{1\prime\_neg} := q_h \cdot (GC_{p,1\prime\_neg} + GC_{pi}) = -11.88 \ \text{psf} \]
\[ p_{1\_neg} := q_h \cdot (GC_{p,1\_neg} + GC_{pi}) = -20.13 \ \text{psf} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Sheathing Design: (For Light-Frame Unit)

Wall:

<table>
<thead>
<tr>
<th>Sheathing Type</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in.)</th>
<th>Perpendicular to Supports</th>
<th>Parallel to Supports</th>
</tr>
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<tbody>
<tr>
<td></td>
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<td></td>
</tr>
<tr>
<td>Wood Structural Panels</td>
<td>24/0</td>
<td>3/8</td>
<td>24</td>
<td>425</td>
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<tr>
<td>(Sheathing Grades, C-C, C-D, C-C Plugged, OSB)</td>
<td>24/16</td>
<td>7/16</td>
<td>24</td>
<td>540</td>
</tr>
<tr>
<td></td>
<td>15/32</td>
<td>24</td>
<td>625</td>
<td>355</td>
</tr>
<tr>
<td></td>
<td>19/32</td>
<td>24</td>
<td>955</td>
<td>595</td>
</tr>
<tr>
<td></td>
<td>23/32</td>
<td>24</td>
<td>1160</td>
<td>840</td>
</tr>
<tr>
<td>Particleboard Sheathing (M-S Exterior Glue)</td>
<td>3/8</td>
<td>16</td>
<td>24</td>
<td>(contact manufacturer)</td>
</tr>
<tr>
<td></td>
<td>1/2</td>
<td>16</td>
<td>(contact manufacturer)</td>
<td></td>
</tr>
<tr>
<td>Particleboard Panel Siding (M-S Exterior Glue)</td>
<td>5/8</td>
<td>16</td>
<td>24</td>
<td>(contact manufacturer)</td>
</tr>
<tr>
<td></td>
<td>3/4</td>
<td>16</td>
<td>(contact manufacturer)</td>
<td></td>
</tr>
<tr>
<td>Hardboard Siding (Direct to Studs)</td>
<td>7/16</td>
<td>16</td>
<td>24</td>
<td>460</td>
</tr>
<tr>
<td>(Lap Siding)</td>
<td>7/16</td>
<td>16</td>
<td>24</td>
<td>460</td>
</tr>
<tr>
<td>(Shiplap Edge Panel Siding)</td>
<td>7/16</td>
<td>16</td>
<td>24</td>
<td>460</td>
</tr>
<tr>
<td>(Square Edge Panel Siding)</td>
<td>7/16</td>
<td>16</td>
<td>24</td>
<td>460</td>
</tr>
<tr>
<td>Cellulose Fiberboard Sheathing</td>
<td>Regular</td>
<td>1/2</td>
<td>16</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td>Structural</td>
<td>1/2</td>
<td>16</td>
<td>135</td>
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<tr>
<td></td>
<td>Structural</td>
<td>25/32</td>
<td>16</td>
<td>165</td>
</tr>
</tbody>
</table>

1. Nominal capacities shall be adjusted in accordance with Section 3.2.1 to determine ASD uniform load capacity and LRFD uniform resistance.
2. Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood with 3 or more plies.
3. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
4. Wood structural panels shall conform to the requirements for its type in DOC PS 1 or PS 5. Particleboard sheathing shall conform to ANSI A208.1. Hardboard panel and siding shall conform to the requirements of ANSI/CPA A315.6. Cellulose fiberboard sheathing shall conform to ASTM C 208.
5. Tabulated values for maximum bending loads are from wind. Loads are limited by bending or shear stress assuming a 2-span continuous condition. Where panels are continuous over 3 or more spans the tabulated values shall be permitted to be increased in accordance with the ASD/LRFD Manual for Engineered Wood Construction.
6. Strength axis is defined as the axis parallel to the face and back orientation of the flake or the grain (veneer), which is generally the long panel direction, unless otherwise marked.

Must Span strength direction perpendicular to studs
Must have at least two spans continuous

\[
\phi_b := 0.85
\]

\[
R := 135 \text{ psf} \cdot \phi_b = 114.75 \text{ psf} > \text{ max pressure}
\]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Withdrawal Resistance:

\[ N_s := \frac{4 \text{ ft}}{6 \text{ in}} = 8 \]

number of nails per stud

\[ N := N_s \cdot 4 = 32 \]

nails per panel

\[ p_{\text{min}} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -760.32 \text{ lbf} \]

\[ W := 41 \text{ lbf} \quad L_d := 2.5 \text{ in} \quad L_m := 2.5 \text{ in} - \frac{7}{16} \text{ in} = 2.063 \text{ in} \]

\[ W_{\text{sd}} := W \cdot L_m = 84.563 \text{ lbf} \quad K_{fw} := 3.32 \quad \phi_w := 0.65 \]

\[ W'_{\text{sd}} := W_{\text{sd}} \cdot K_{fw} \cdot \phi_w = 811.738 \text{ N} \]

\[ W'_{\text{sd}} \cdot N = 5839.548 \text{ lbf} \quad \text{Capacity > Load} \]

---

Roof:

Bending Resistance:

---

Table 3.2.2 Nominal Uniform Load Capacities (psf) for Roof Sheathing Resisting Out-of-Plane Wind Loads

<table>
<thead>
<tr>
<th>Sheathing Type¹</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in.)</th>
<th>Strength Axis² Applied Perpendicular to Supports</th>
<th>Strength Axis² Applied Parallel to Supports</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rafter/Truss Spacing (in.)</td>
<td>Rafter/Truss Spacing (in.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wood Structural Panels (Sheathing Grades, C-C)</td>
<td>24/0</td>
<td>3/8</td>
<td>425</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>24/16</td>
<td>7/16</td>
<td>540</td>
<td>305</td>
</tr>
<tr>
<td></td>
<td>32/16</td>
<td>15/32</td>
<td>625</td>
<td>555</td>
</tr>
<tr>
<td></td>
<td>40/29</td>
<td>19/32</td>
<td>955</td>
<td>595</td>
</tr>
<tr>
<td></td>
<td>49/24</td>
<td>23/32</td>
<td>1160</td>
<td>840</td>
</tr>
<tr>
<td>Wood Structural Panels (Single Floor Grades, Underlayment, C-C Plugged)</td>
<td>16 o.c.</td>
<td>19/32</td>
<td>705</td>
<td>395</td>
</tr>
<tr>
<td></td>
<td>20 o.c.</td>
<td>19/32</td>
<td>815</td>
<td>455</td>
</tr>
<tr>
<td></td>
<td>24 o.c.</td>
<td>23/32</td>
<td>1160</td>
<td>670</td>
</tr>
<tr>
<td></td>
<td>32 o.c.</td>
<td>23/32</td>
<td>1160</td>
<td>670</td>
</tr>
<tr>
<td></td>
<td>48 o.c.</td>
<td>1-1/8</td>
<td>1750²</td>
<td>1250²</td>
</tr>
</tbody>
</table>

¹ Nominal capacities shall be adjusted in accordance with Section 3.2.3 to determine ASD uniform load capacity and LRFD uniform resistance.

² Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood with 3 or more plies.

³ Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.

⁴ Tabulated values are based on the lesser of nominal values for either OSB or plywood with 5 or more plies.

⁵ Wood structural panels shall conform to the requirements for its type in DOC PS 1 or PS 2.

⁶ Tabulated values are for maximum bending loads from wind. Loads are limited by: bending or shear stress assuming a 2-span continuous condition. Where panels are continuous over 3 or more spans, the tabulated values shall be permitted to be increased in accordance with the ASD-LRFD Manual for Engineered Wood Construction.

⁷ Strength axis is defined as the axis parallel to the face and back orientation of the flake or the grain (vesse), which is generally the long panel direction, unless otherwise marked.
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

\[ p_{3\_pos} = 7.92 \text{ psf} \]
\[ p_{2\_pos} = 6.27 \text{ psf} \]
\[ p_{1\_prime\_pos} = 6.27 \text{ psf} \]
\[ p_{1\_pos} = 7.92 \text{ psf} \]

\[ p_{\max} := \max (p_{3\_pos}, p_{2\_pos}, p_{1\_pos}, p_{1\_prime\_pos}) = 7.92 \text{ psf} \]

\[ p_{3\_neg} = -49.83 \text{ psf} \]
\[ p_{2\_neg} = -23.43 \text{ psf} \]
\[ p_{1\_prime\_neg} = -11.88 \text{ psf} \]
\[ p_{1\_neg} = -20.13 \text{ psf} \]

\[ p_{\min} := \min (p_{3\_neg}, p_{2\_neg}, p_{1\_neg}, p_{1\_prime\_neg}) = -49.83 \text{ psf} \]

Must Span strength direction perpendicular to studs
Must have at least two spans continuous
\[ \phi_b := 0.85 \]
\[ R := 135 \text{ psf} \cdot \phi_b = 114.75 \text{ psf} \quad > \text{max pressure} \]

Withdrawal Resistance:
\[ N_s := \frac{4 \text{ ft}}{6 \text{ in}} = 8 \quad \text{number of nails per stud} \]
\[ N := N_s \cdot 4 = 32 \quad \text{nails per panel} \]
\[ p_{\min} = -49.83 \text{ psf} \]
\[ p_{\min} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -1594.56 \text{ lbf} \]
\[ W := 41 \frac{\text{lb}}{\text{in}} \quad L_d := 2.5 \text{ in} \quad L_m := 2.5 \text{ in} - \frac{1}{2} \text{ in} = 2 \text{ in} \]
\[ W_{8d} := W \cdot L_m = 82 \text{ lbf} \quad K_{fw} = 3.32 \quad \phi_w := 0.65 \]
\[ W'_{8d} := W_{8d} \cdot K_{fw} \cdot \phi_w = 787.14 \text{ N} \]
\[ W_{8d} \cdot N = 5662.592 \text{ lbf} \quad \text{Capacity > Load} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

150 MPH:

Assumptions:
- Partially Open Enclosure Classification
- Roof Slope 3 Degrees

---

Table 27.2-1 Steps to Determine MWFIRS Wind Loads for Enclosed, Partially Enclosed, and Open Buildings of All Heights

<table>
<thead>
<tr>
<th>Step 1</th>
<th>Step 2</th>
<th>Step 3</th>
<th>Step 4</th>
<th>Step 5</th>
<th>Step 6</th>
</tr>
</thead>
</table>
| Determine Risk Category of building; see Table 1.5-1. | Determine the basic wind speed, \( V \), for the applicable Risk Category; see Figs. 26.5-1 and 26.5-2. | Determine wind load parameters:  
  - Wind directionality factor, \( K_d \); see Section 26.6 and Table 26.6-1.  
  - Exposure category, see Section 26.7.  
  - Topographic factor, \( K_z \); see Section 26.8 and Table 26.8-1.  
  - Ground elevation factor, \( K_e \); see Section 26.9.  
  - Gust-effect factor, \( G \) or \( G_j \); see Section 26.11.  
  - Enclosure classification, see Section 26.12.  
  - Internal pressure coefficient, \( GC_{pi} \); see Section 26.13 and Table 26.13-1. | Determine velocity pressure exposure coefficient, \( K_z \) or \( K_t \); see Table 26.10-1. | Determine velocity pressure \( q_z \) or \( q_t \), Eq. (26.10-1). | Determine external pressure coefficient, \( C_p \) or \( C_q \):  
  - Fig. 27.3-1 for walls and flat, gable, hip, monoslope, or mansard roofs.  
  - Fig. 27.3-2 for domed roofs.  
  - Fig. 27.3-3 for arched roofs.  
  - Fig. 27.3-4 for monoslope roof, open building.  
  - Fig. 27.3-5 for pitched roof, open building.  
  - Fig. 27.3-6 for gabled roof, open building.  
  - Fig. 27.3-7 for along-ridge/gable valley wind load case for monoslope, pitched, or gabled roof, open building. | Calculate wind pressure, \( p \), on each building surface:  
  - Eq. (27.3-1) for rigid and flexible buildings.  
  - Eq. (27.3-2) for open buildings. |

---

Step 1: Risk Category II

Step 2: Basic Wind Speed = 150 mph  \( V := 150 \)

Step 3:

- Wind Directionality  \( K_d := 0.85 \)
- Exposure Category B  
  - Topographic Factor  \( K_z := 1 \)
  - Ground Elevation Factor  \( K_e := 1 \)
  - Gust-effect Factor  \( G := 0.85 \)
- Enclosure Classification Partially Open
- Internal Pressure Coefficient  \( GC_{pi} := 0.18 \)

Step 4: Velocity pressure exposure coefficient  \( K_z := 0.57 \)

Step 5: Velocity pressure  
\[
q_z := \left(0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2\right) \cdot \text{psf} = 27.907 \text{ psf}
\]

Use:  \( q_z := 28 \text{ psf} \)

Step 6: External pressure coefficient

---

Notation

- \( B \): Horizontal dimension of building, in ft \((m)\), measured normal to wind direction.
- \( L \): Horizontal dimension of building, in ft \((m)\), measured parallel to wind direction.
- \( h \): Mean roof height, in ft \((m)\), except that eave height shall be used for \( 0 \leq 10 \text{ degrees} \).
- \( z \): Height above ground, in ft \((m)\).
- \( G \): Gust-effect factor.
- \( q_z \): Velocity pressure, in lb/ft\(^2\) \((N/m^2)\), evaluated at respective height.
- \( \theta \): Angle of plane of roof from horizontal, in degrees.

---

Notation

- \( B \): Horizontal dimension of building, in ft \((m)\), measured normal to wind direction.
- \( L \): Horizontal dimension of building, in ft \((m)\), measured parallel to wind direction.
- \( h \): Mean roof height, in ft \((m)\), except that eave height shall be used for \( 0 \leq 10 \text{ degrees} \).
- \( z \): Height above ground, in ft \((m)\).
- \( G \): Gust-effect factor.
- \( q_z \): Velocity pressure, in lb/ft\(^2\) \((N/m^2)\), evaluated at respective height.
- \( \theta \): Angle of plane of roof from horizontal, in degrees.
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Wind Perpendicular to Ridge:

\[ L := 19.5 \, \text{ft} \quad h := 8.5 \, \text{ft} \quad B := 20 \, \text{ft} \quad z := 7.5 \, \text{ft} \]

Height to peak of roof

\[ \frac{L}{B} = 0.975 \]

Height to eave

Wall Pressure Coefficients:

<table>
<thead>
<tr>
<th>Surface</th>
<th>( L/B )</th>
<th>( C_p )</th>
<th>Use With</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>All values</td>
<td>0.8</td>
<td>( q_a )</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>-0.5</td>
<td>( q_a )</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>2</td>
<td>-0.3</td>
<td>( q_a )</td>
</tr>
<tr>
<td>Sidewall</td>
<td>All values</td>
<td>-0.7</td>
<td>( q_a )</td>
</tr>
</tbody>
</table>

Roof Pressure Coefficients, \( C_p \) for use with \( q_a \):

| Wind Direction | \( L/H \) | 0 to \( H/2 \) | \( H/2 \) to 0 | 0 to 20 | \( H/2 \) to 20 | \( H/2 \) to 30 | \( H/2 \) to 40 | 0 to \( H/2 \) | \( H/2 \) to 20 | \( H/2 \) to 30 | 4 to 60 | 60 to \( H \) | \( L/H \) |
|----------------|----------|---------------|---------------|--------|----------------|----------------|---------------|---------------|----------------|----------------|----------------|--------|---------------|----------|
| Normal to Ridge| 0.05     | 0 to \( H/2 \) | 0 to \( H/2 \) | 0 to 20 | 0 to 30 | 0 to 40 | 0 to 60 | 0 to \( H/2 \) | 0 to \( H/2 \) | 0 to 20 | 0 to 30 | 0 to 40 | 0 to 60 | 0.05 |
| 0.05 to \( H \) | 0 to \( H/2 \) | 0 to \( H/2 \) | 0 to 20 | 0 to 30 | 0 to 40 | 0 to 60 | 0 to \( H/2 \) | 0 to \( H/2 \) | 0 to 20 | 0 to 30 | 0 to 40 | 0 to 60 | 0.05 |
| \( \geq 0.05 \) | 0 to \( H/2 \) | 0 to \( H/2 \) | 0 to 20 | 0 to 30 | 0 to 40 | 0 to 60 | 0 to \( H/2 \) | 0 to \( H/2 \) | 0 to 20 | 0 to 30 | 0 to 40 | 0 to 60 | 0.05 |

Roof Pressure Coefficients:

\[ h = 0.436 \]

\[ \frac{L}{H} = 0.975 \]

\[ C_{p_{r, \text{perp}}} = -0.9 \]

\[ P_{\text{ww,ext}} := q_z \cdot G \cdot C_{p_{\text{ww}}} = 19.04 \, \text{psf} \]

\[ P_{\text{lw,ext}} := q_z \cdot G \cdot C_{p_{\text{lw,perp}}} = -11.9 \, \text{psf} \]

\[ P_{\text{r,ext}} := q_z \cdot G \cdot C_{p_{r, \text{perp}}} = -21.42 \, \text{psf} \]

\[ P_{\text{int}} := q_z \cdot G \cdot C_{p_{\text{int}}} = 5.04 \, \text{psf} \]

roof pressure:

\[ P_r := P_{\text{r,ext}} + -1 \cdot P_{\text{int}} = -26.46 \, \text{psf} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

**Forces:**

**Windward Wall:**

\[ P_{ww_{ext}} = 19.04 \text{ psf} \]

\[ F_{ww} = P_{ww_{ext}} \cdot \left( \frac{h}{2} \right) = 80.92 \text{ plf} \quad \text{Line load along top and bottom of wall} \]

**Leeward Wall:**

\[ P_{lw_{ext}} = -11.9 \text{ psf} \]

\[ P_{int} = 5.04 \text{ psf} \]

\[ F_{lw} = (P_{lw_{ext}} - P_{int}) \cdot \left( \frac{h}{2} \right) = -71.995 \text{ plf} \quad \text{Line load along top and bottom of wall} \]

**Roof:**

\[ P_r = -26.46 \text{ psf} \]

**x-comp:**

\[ P_{rx} := P_r \cdot \sin(3 \text{ deg}) = -1.385 \text{ psf} \]

**y-comp:**

\[ P_{ry} := P_r \cdot \cos(3 \text{ deg}) = -26.424 \text{ psf} \]

Each shear wall Perp to ridge

- **Ftop:**
  - Half of top of wall (ww & lw) line load

\[ F_{top} := (F_{ww} - F_{lw}) \cdot \left( \frac{B}{2} \right) = 1.529 \text{ kip} \]

- **Fbot:**
  - Half of bot of wall (ww & lw) line load

\[ F_{bot} := (F_{ww} - F_{lw}) \cdot \left( \frac{B}{2} \right) = 1.529 \text{ kip} \]

- **Fry:**
  - uplift line load

\[ F_{r,y} := P_{ry} \cdot \left( \frac{B}{2} \right) = -264.237 \text{ plf} \]

Shear at base of wall:

\[ V_b := F_{top} + F_{bot} = 3.1 \text{ kip} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Load Case:

\[ 1.2 \times DL + W + LL + 0.5 \times (L_F \text{ or } S \text{ or } R) \]  
\[ (EQ \ 4.3.2-4) \]

**Overturning Moments:** \[ L = 5.944 \ m \quad B = 6.096 \ m \]
\[ DL_r := 23.5 \ psf \]
\[ LL_r := 20 \ psf \quad Trib := \frac{B}{2} \]

Overturning moment from Ftop:
\[ M_{F, \text{top}} := F_{\text{top}} \cdot z = 11.469 \ kip \cdot ft \]

Overturning moment from Roof Uplift:
\[ M_{F, \text{up}} := -F_{r,y} \cdot 22 \ ft \cdot \frac{L}{2} = 56.679 \ kip \cdot ft \]

Overturning moment from Roof Uplift:
\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot \frac{L}{2} = 91.64 \ kip \cdot ft \]

**Required force in hold down:**
\[ T := \frac{M_{F, \text{top}} + M_{F, \text{up}} - M_r}{L - 1 \ ft} = -1.27 \ kip \]

**Required Compression Force:**
\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 8.129 \ kip \]

**Required Studs for compression:**
CLT Capacity:
\[ F'_{c} := 35849 \ plf = 35.849 \ klf \quad \text{Okay} \]
Light-Frame Capacity:
\[ P' := 5 \ kip \]
\[ \frac{C}{P'} = 1.626 \quad \text{Need at least 2 studs in corner for compression} \]

**Required hinge length for base shear:**
Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_{n} := 3.85 \ klf \]

For Base shear we would need:
\[ \frac{V}{V'_{n}} = 0.009 \frac{1}{N} \ ft \quad \text{of Hinge along wall} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Load Case:

\[ 0.9 \times DL + W \quad (EQ \ 4.3.2-5) \]

Overturning Moments:

\[ L = 5.944 \, m \quad \quad B = 6.096 \, m \]

Overturning moment from Ftop:
\[ M_{F,\text{top}} := F_{\text{top}} \cdot z = 11.469 \, \text{kip}\cdot\text{ft} \]

Overturning moment from Roof Uplift:
\[ M_{F,\text{up}} := -F_{r,y} \cdot 22 \, \text{ft} \cdot \frac{L}{2} = 56.679 \, \text{kip}\cdot\text{ft} \]

Required force in hold down:
\[ T := \frac{M_{F,\text{top}} + M_{F,\text{up}}}{L - 1 \, \text{ft}} = 3.684 \, \text{kip} \]

Required Compression Force:
\[ C := T = 3.684 \, \text{kip} \]

Required Studs for compression:
CLT Capacity:
\[ F' := 35849 \, \text{plf} = 35.849 \, \text{klf} \quad \text{Okay} \]

Light-Frame Capacity:
\[ P' := 5 \, \text{kip} \]

\[ C = 0.737 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_{n} := 3.85 \, \text{klf} \]

For Base shear we would need:
\[ \frac{V}{V'_{n}} = 0.009 \frac{1}{N} \, \text{ft} \quad \text{of Hinge along wall} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Wood Sheathing Nailing to Resist Shear: (LF UNITS)

\[ \phi d = 0.8 \]

Shearwall:

\[ V := \frac{F_{top}}{L \cdot \phi d} = 98.022 \text{ plf} \]

Sheathing:

Structural 1

7/16"

24/16 Span Rating

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Fastener Penetration in Framing Member or Blocking (in.)</th>
<th>Fastener Type &amp; Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Structural Panels - Structural 1</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
</tr>
<tr>
<td>Wood Structural Panels - Sheathing</td>
<td>7/16</td>
<td>1-3/8</td>
<td>8d</td>
</tr>
<tr>
<td>Plywood Siding</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
</tr>
<tr>
<td>Particleboard Sheathing - M's &quot;Exterior Glue&quot; and M-2 &quot;Exterior Glue&quot;</td>
<td>5/32</td>
<td>1/2</td>
<td>8d</td>
</tr>
<tr>
<td>Structural Fiberboard Sheathing</td>
<td>5/32</td>
<td>1/2</td>
<td>10d</td>
</tr>
</tbody>
</table>

Table 4.3A Nominal Unit Shear Capacity

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Fastener Penetration in Framing Member or Blocking (in.)</th>
<th>Fastener Type &amp; Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Structural Panels - Structural 1</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
</tr>
<tr>
<td>Wood Structural Panels - Sheathing</td>
<td>7/16</td>
<td>1-3/8</td>
<td>8d</td>
</tr>
<tr>
<td>Plywood Siding</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
</tr>
<tr>
<td>Particleboard Sheathing - M's &quot;Exterior Glue&quot; and M-2 &quot;Exterior Glue&quot;</td>
<td>5/32</td>
<td>1/2</td>
<td>8d</td>
</tr>
<tr>
<td>Structural Fiberboard Sheathing</td>
<td>5/32</td>
<td>1/2</td>
<td>10d</td>
</tr>
</tbody>
</table>

\[ V_{wd} := 715 \text{ plf} \]

Use 8d nails with 6" spacing along edge

Diaphragm:

\[ V_{ud} := V = 98.022 \text{ plf} \]

\[ V_{wd} := 670 \text{ plf} \]

okay
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

**Wind Parallel To Ridge:**

\[ L := 20 \text{ ft} \quad h := 8.5 \text{ ft} \]
\[ B := 19.5 \text{ ft} \quad z := 7.5 \text{ ft} \]

- Height to peak & used for wall height entire length
- Moment arm for overturning moment from F top

### Wall Pressure Coefficients:

\[ \frac{L}{B} = 1.026 \]

#### Wall Pressure Coefficients, \( C_p \)

<table>
<thead>
<tr>
<th>Surface</th>
<th>( L/B )</th>
<th>( C_p )</th>
<th>Use With</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>All values</td>
<td>0.8</td>
<td>( q_x )</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>-0.5</td>
<td>( q_x )</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>2</td>
<td>-0.3</td>
<td>( q_x )</td>
</tr>
<tr>
<td>Sidewall</td>
<td>All values</td>
<td>-0.7</td>
<td>( q_x )</td>
</tr>
</tbody>
</table>

#### Roof Pressure Coefficients, \( C_p \) for use with \( q_x \)

<table>
<thead>
<tr>
<th>Wind Direction</th>
<th>Angle (°)</th>
<th>Normal to Ridge</th>
<th>Horizontal Distance from Windward Edge</th>
<th>( C_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10</td>
<td>15</td>
<td>20</td>
<td>25</td>
</tr>
<tr>
<td>Perpendicular</td>
<td>0</td>
<td>0.8</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.8</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>0.8</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>0.8</td>
<td>0.9</td>
<td>0.9</td>
</tr>
</tbody>
</table>

\*Value is provided for interpolation purposes.

\*Value can be reduced linearly with area over which it is applicable as follows:

\*For roof slopes greater than 30°, use \( C_p = 0.8 \).

#### Roof Pressure Coefficients:

\[ h = 0.425 \]
\[ \frac{h}{L} = 0.425 \]

\[ C_{p_r \text{-perp}} = -0.9 \]

---

**Windward Wall**

\[ C_{p_{ww}} := 0.8 \]

**Leeward Wall**

\[ C_{p_{lw \text{-perp}}} := -0.5 \]

\[ C_{p_{sw}} := -0.7 \]

---

**Wall Pressure Coefficients:**

- Windward Wall
- Leeward Wall
- Sidewall

---

**Windward Wall**

\[ P_{ww\text{-ext}} := q_z \cdot G \cdot C_{p_{ww}} = 19.04 \text{ psf} \]

**Leeward Wall**

\[ P_{lw\text{-ext}} := q_z \cdot G \cdot C_{p_{lw \text{-perp}}} = -11.9 \text{ psf} \]

**Sidewall**

\[ P_{r\text{-ext}} := q_z \cdot G \cdot C_{p_{sw}} = -21.42 \text{ psf} \]

**Interior Pressure**

\[ P_{int} := q_z \cdot G C_{pi} = 5.04 \text{ psf} \]

**Roof Pressure**

\[ P_{r} := P_{r\text{-ext}} + -1 \cdot P_{int} = -26.46 \text{ psf} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

**Forces:**

Windward Wall:

\[ P_{ww, ext} = 19.04 \text{ psf} \]

\[ F_{ww} = P_{ww, ext} \cdot \left( \frac{h}{2} \right) = 80.92 \text{ plf} \]

Line load along top and bottom of wall

Leeward Wall:

\[ P_{lw, ext} = -11.9 \text{ psf} \]

\[ F_{lw} := \left( P_{lw, ext} - P_{int} \right) \cdot \left( \frac{h}{2} \right) = -71.995 \text{ plf} \]

Line load along top and bottom of wall

Roof:

\[ P_{ry} := P_r = -26.46 \text{ psf} \]

Each shear wall Par to ridge

\[ F_{top} := \left( F_{ww} - F_{lw} \right) \cdot \left( \frac{B}{2} \right) = 1.491 \text{ kip} \]

- Half of top of wall (ww & lw) line load

\[ F_{bot} := \left( F_{ww} - F_{lw} \right) \cdot \left( \frac{B}{2} \right) = 1.491 \text{ kip} \]

- Half of bot of wall (ww & lw) line load

Fry:

- uplift line load \[ F_{ry, y} := P_{ry} \cdot \left( \frac{B}{2} \right) = -257.985 \text{ plf} \]

Shear at base of wall:

\[ V_b := F_{top} + F_{bot} = 2.982 \text{ kip} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Load Case:

\[1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S) \text{ or } R\]  \hspace{1cm} (EQ 4.3.2-4)

Overturning Moments:

\[L = 6.096 \text{ m} \quad B = 5.944 \text{ m}\]
\[DL_r := 23.5 \text{ psf} \quad LL_r := 20 \text{ psf} \quad Trib := \frac{B}{2}\]

Overturning moment from Ftop:

\[M_{F,\text{top}} := F_{\text{top}} \cdot z = 11.182 \text{ kip} \cdot \text{ft}\]

Overturning moment from Roof Uplift:

\[M_{F,\text{up}} := -F_{r_y} \cdot 22\text{ ft} \cdot \frac{L}{2} = 56.757 \text{ kip} \cdot \text{ft}\]

Resisting moment from Dead and Live:

\[M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 93.99 \text{ kip} \cdot \text{ft}\]

Required force in hold down:

\[T := \frac{M_{F,\text{top}} + M_{F,\text{up}} - M_r}{L - 1 \text{ ft}} = -1.371 \text{ kip}\]

Required Compression Force:

\[C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 8.028 \text{ kip}\]

Required Studs for compression:

CLT Capacity:

\[F'_{c} := 35849 \text{ psf} = 35.849 \text{ klf} \quad \text{Okay}\]

Light-Frame Capacity:

\[P' := 5 \text{ kip}\]

\[
\frac{C}{P'} = 1.606 \quad \text{Need at least 2 studs in corner for compression}\]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[\phi := 0.65 \quad \text{Connections}\]

\[V'_{n} := 3.85 \text{ klf}\]

For Base shear we would need:

\[
\frac{V}{V'_{n}} = 0.084 \frac{1}{m} \cdot \text{ft} \quad \text{of Hinge along wall}\]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Load Case:

$0.9 \times DL + W$ \hspace{1cm} (EQ 4.3.2-5)

Overturning Moments:

$L = 6.096 \text{ m} \quad B = 5.944 \text{ m}$

Overturning moment from Ftop:

$M_{F_{\text{top}}} := F_{\text{top}} \cdot z = 11.182 \text{ kip} \cdot \text{ft}$

Overturning moment from Roof Uplift:

$M_{F_{\text{up}}} := -F_{r-y} \cdot 22 \text{ ft} \cdot \frac{L}{2} = 56.757 \text{ kip} \cdot \text{ft}$

Required force in hold down:

$T := \frac{M_{F_{\text{top}}} + M_{F_{\text{up}}}}{L - 1 \text{ ft}} = 3.576 \text{ kip}$ \hspace{1cm} $M_{F_{\text{top}}} + M_{F_{\text{up}}} = 67.939 \text{ kip} \cdot \text{ft}$

Required Compression Force:

$C := T = 3.576 \text{ kip}$

Required Studs for compression:

CLT Capacity:

$F'_{c} := 35849 \text{ plf} = 35.849 \text{ klf}$ \hspace{1cm} Okay

Light-Frame Capacity:

$P' := 5 \text{ kip}$

$C = 0.715$ \hspace{1cm} Need at least 1 studs in corner for compression

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

$\phi := 0.65$ \hspace{1cm} Connections

$V'_{n} := 3.85 \text{ klf}$

For Base shear we would need:

$\frac{V}{V'_{n}} = 0.084 \text{ ft} \cdot \frac{1}{m} \text{ of Hinge along wall}$
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Wood Sheathing Nailing to Resist Shear: (LF UNITS) \[ \phi_D := 0.8 \]

Shearwall:

\[ V := \frac{F_{top}}{L \cdot \phi_D} = 93.183 \text{ plf} \]

Sheathing:

Structural 1

7/16”

24/16 Span Rating

\[ \text{Diaphragm:} \]

\[ V_{ud} := 715 \text{ plf} \]

Use 8d nails with 6” spacing along edge

\[ V_{ud} := V = 93.183 \text{ plf} \]

\[ V_{ud} := 670 \text{ plf} \quad \text{okay} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Components and Cladding (C&C):

Table 30.3-1 Steps to Determine C&C Wind Loads for Enclosed and Partially Enclosed Low-Rise Buildings

Step 1: Risk Category II
Step 2: Determine the basic wind speed, \( V = 150 \) mph
Step 3:
- Wind Directionality: \( K_d = 0.85 \)
- Exposure Category B
- Topographic Factor: \( K_{zt} = 1 \)
- Ground Elevation Factor: \( K_e = 1 \)
- Enclosure Classification: Partially Open
- Internal Pressure Coefficient: \( GC_{pi} = 0.18 \)

Step 4: Velocity pressure exposure coefficient: \( K_h = 0.57 \)

Step 5: Velocity pressure
\[
q_h = (0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2) \cdot psf = 27.907 \text{ psf}
\]
\( q_h = 28 \text{ psf} \)

Step 6: External pressure coefficient

Walls:

Diagram

Notation
\( a = 10\% \) of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

Exception: For buildings with \( \theta = 0^\circ \) to \( 7^\circ \) and a least horizontal dimension greater than 300 ft (90 m), dimension \( a \) shall be limited to a maximum of 0.8h.

\( h = \text{Mean roof height, in ft (m), except that eave height shall be used for } \theta \leq 10^\circ. \)

\( \theta = \text{Angle of plane of roof from horizontal, in degrees.} \)

\[
a = \min \left( 0.1 \cdot \min (B, L), 0.4 \cdot h_{mean} \right) = 0.594 \text{ m}
\]

\[
a_{min} = \max (0.04 \cdot \min (B, L), 3 \text{ ft}) = 0.914 \text{ m}
\]

\[
a = \max (a, a_{min}) = 0.914 \text{ m}
\]

\( \theta = 3 \text{ deg} \)

Perpendicular to Ridge:
\[
A_{5,\text{per}} = a \cdot z = 2.09 \text{ m}^2
\]

Parallel to Ridge:
\[
A_{5,\text{par}} = z \cdot a = 2.09 \text{ m}^2
\]

*smaller area worse CGp

\[
A_{4,\text{per}} = (L - 2 \cdot a) \cdot z = 9.755 \text{ m}^2
\]

\[
A_{4,\text{par}} = (B - 2 \cdot a) \cdot z = 9.406 \text{ m}^2
\]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

**Notes**
1. Vertical scale denotes (GCp) to be used with qh.
2. Horizontal scale denotes effective wind area, in ft² (m²).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of (GCp) for walls shall be reduced by 10% when θ ≤ 10°.

**Positive:**
\[ GC_{p,5\_pos} := 1 \cdot 0.9 = 0.9 \]
\[ GC_{p,4\_pos} := 0.9 \cdot 0.9 = 0.81 \]

**Negative:**
\[ GC_{p,5\_neg} := -1.4 \cdot 0.9 = -1.26 \]
\[ GC_{p,4\_neg} := -1 \cdot 0.9 = -0.9 \]

**Design Pressures:**
\[ p_{5\_pos} := qh \cdot (GC_{p,5\_pos} + GC_{pi}) = 30.24 \text{ psf} \]
\[ p_{4\_pos} := qh \cdot (GC_{p,4\_pos} + GC_{pi}) = 27.72 \text{ psf} \]
\[ p_{5\_neg} := qh \cdot (GC_{p,5\_neg} - GC_{pi}) = -40.32 \text{ psf} \]
\[ p_{4\_neg} := qh \cdot (GC_{p,4\_neg} - GC_{pi}) = -30.24 \text{ psf} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Roof:

**Diagram**

**Notation**

\( a = 10\% \) of least horizontal dimension or 0.4\( h \), whichever is smaller, but not less than either 4\% of least horizontal dimension or 3 ft (0.9 m).

**Exception:** For buildings with \( \theta = 0^\circ \) to 7\(^\circ\) and a least horizontal dimension greater than 300 ft (90 m), dimension \( a \) shall be limited to a maximum of 0.8\( h \).

\( h \) = Mean roof height, in ft (m), except that eave height shall be used for \( \theta \leq 10^\circ \).

\( \theta \) = Angle of plane of roof from horizontal, in degrees.

**External Pressure Coefficients**

**Notes**

1. Vertical scale denotes \( (GC_P) \) to be used with \( q_p \).
2. Horizontal scale denotes effective wind area, in \( ft^2 \) (m²).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. If a parapet equal to or higher than 3 ft (0.9 m) is provided around the perimeter of the roof with \( \theta \leq 7^\circ \), the negative values of \( (GC_P) \) in Zone 3 shall be equal to those for Zone 2, and positive values of \( (GC_P) \) in Zones 2 and 3 shall be set equal to those for wall Zones 4 and 5, respectively, in Fig. 30.3-1.
6. Values of \( (GC_P) \) for roof overhangs include pressure contributions from both upper and lower surfaces.
7. If overhangs exist, the lesser horizontal dimension of the building shall not include any overhang dimension, but the edge distance, \( a \), shall be measured from the outside edge of the overhang.
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

\[
A_3 := ((0.6 \cdot h) \cdot (0.2 \cdot h) \cdot 2) - (0.2 \cdot h)^2 = 1.342 \ m^2
\]

\[
A_2 := (2 \cdot B \cdot 0.6 \cdot h) + (2 \cdot L \cdot 0.6 \cdot h) - 4 \cdot (0.6 \ h)^2 - 4 \cdot A_3 = 22.395 \ m^2
\]

\[
A_{1,prime} := (B - 2.4 \cdot h) \cdot (L - 2.4 \cdot h) = 0.033 \ m^2
\]

\[
A_1 := (B \cdot L) - A_{1,prime} - A_2 - 4 \cdot A_3 = 8.434 \ m^2
\]

Positive:
\[
GC_{p,3\_pos} := 0.3
\]
\[
GC_{p,2\_pos} := 0.2
\]
\[
GC_{p,1\_prime\_pos} := 0.2
\]
\[
GC_{p,1\_pos} := 0.3
\]

Negative:
\[
GC_{p,3\_neg} := -3.2
\]
\[
GC_{p,2\_neg} := -1.6
\]
\[
GC_{p,1\_prime\_neg} := -0.9
\]
\[
GC_{p,1\_neg} := -1.4
\]

Design Pressures:
\[
p_{3\_pos} := q_h \cdot (GC_{p,3\_pos} + GC_{pi}) = 13.44 \ psf
\]
\[
p_{2\_pos} := q_h \cdot (GC_{p,2\_pos} + GC_{pi}) = 10.64 \ psf
\]
\[
p_{1\_prime\_pos} := q_h \cdot (GC_{p,1\_prime\_pos} + GC_{pi}) = 10.64 \ psf
\]
\[
p_{1\_pos} := q_h \cdot (GC_{p,1\_pos} + GC_{pi}) = 13.44 \ psf
\]
\[
p_{3\_neg} := q_h \cdot (GC_{p,3\_neg} + GC_{pi}) = -84.56 \ psf
\]
\[
p_{2\_neg} := q_h \cdot (GC_{p,2\_neg} + GC_{pi}) = -39.76 \ psf
\]
\[
p_{1\_prime\_neg} := q_h \cdot (GC_{p,1\_prime\_neg} + GC_{pi}) = -20.16 \ psf
\]
\[
p_{1\_neg} := q_h \cdot (GC_{p,1\_neg} + GC_{pi}) = -34.16 \ psf
\]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

### Sheathing Design: (For Light-Frame Unit)

**Wall:**

**Bending Resistance:**
- 24" spacing of joists and studs
- 24/16 span rating 7/16" thick

\[
\begin{align*}
p_{5,\text{pos}} &= 30.24 \text{ psf} \\
p_{4,\text{pos}} &= 27.72 \text{ psf} \\
p_{5,\text{neg}} &= -40.32 \text{ psf} \\
p_{4,\text{neg}} &= -30.24 \text{ psf}
\end{align*}
\]

\[
l_b = 24 \text{ in}
\]

\[
p_{\text{max}} = \max (p_{5,\text{pos}}, p_{4,\text{pos}}, p_{5,\text{neg}}, p_{4,\text{neg}}) = 30.24 \text{ psf}
\]

\[
p_{\text{min}} = \min (p_{5,\text{pos}}, p_{4,\text{pos}}, p_{5,\text{neg}}, p_{4,\text{neg}}) = -40.32 \text{ psf}
\]

### Table 3.2.1 Nominal Uniform Load Capacities (psf) for Wall Sheathing Resisting Out-of-Plane Wind Loads\(^a\)

<table>
<thead>
<tr>
<th>Sheathing Type(^b)</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in.)</th>
<th>Perpendicular to Supports</th>
<th>Strength Axis(^a)</th>
<th>Parallel to Supports</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Maximum Stud Spacing (in.)</td>
<td>Actual Stud Spacing</td>
<td>Maximum Stud Spacing (in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>12</td>
<td>15</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Nominal Uniform Loads (psf)</td>
<td>Nominal Uniform Loads (psf)</td>
<td></td>
</tr>
<tr>
<td>Wood Structural Panels (Sheathing Grades, C-C, C-D, C-C Plugged, OSB)(^c)</td>
<td>24/0</td>
<td>3/8</td>
<td>24</td>
<td>425</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>24/16</td>
<td>7/16</td>
<td>24</td>
<td>540</td>
<td>305</td>
</tr>
<tr>
<td></td>
<td>15/32</td>
<td>24</td>
<td>625</td>
<td>355</td>
<td>155</td>
</tr>
<tr>
<td></td>
<td>19/32</td>
<td>24</td>
<td>955</td>
<td>595</td>
<td>265</td>
</tr>
<tr>
<td></td>
<td>23/32</td>
<td>24</td>
<td>1160(^d)</td>
<td>840(^d)</td>
<td>395(^d)</td>
</tr>
<tr>
<td>Particleboard Sheathing (M-S Exterior Glue)</td>
<td>3/8</td>
<td>16</td>
<td>(contact manufacturer)</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1/2</td>
<td>16</td>
<td>(contact manufacturer)</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>Particleboard Panel Siding (M-S Exterior Glue)</td>
<td>5/8</td>
<td>16</td>
<td>(contact manufacturer)</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3/4</td>
<td>16</td>
<td>(contact manufacturer)</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>Hardboard Siding (Direct to Studs)</td>
<td>Lap Siding</td>
<td>7/16</td>
<td>16</td>
<td>460</td>
<td>260</td>
</tr>
<tr>
<td></td>
<td>Shiplap Edge Panel Siding</td>
<td>7/16</td>
<td>16</td>
<td>460</td>
<td>260</td>
</tr>
<tr>
<td></td>
<td>Square Edge Panel Siding</td>
<td>7/16</td>
<td>16</td>
<td>460</td>
<td>260</td>
</tr>
<tr>
<td>Cellulosic Fiberboard Sheathing</td>
<td>Regular</td>
<td>1/2</td>
<td>16</td>
<td>90</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>Structural</td>
<td>1/2</td>
<td>16</td>
<td>135</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td>Structural</td>
<td>25/32</td>
<td>16</td>
<td>165</td>
<td>90</td>
</tr>
</tbody>
</table>

Notes:
1. Nominal capacities shall be adjusted in accordance with Section 3.2.1 to determine ASD uniform load capacity and LRFD uniform resistances.
2. Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood with 3 or more plies.
3. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
4. Wood structural panels shall conform to the requirements for its type in DOC PS 1 or PS 2. Particleboard sheathing shall conform to ANSI A208.1. Hardboard panel and siding shall conform to the requirements of ANSI/CPA A135.6. Cellulosic fiberboard sheathing shall conform to ASTM C 208.
5. Tabulated values are for maximum bending loads from wind. Loads are limited by bending or shear stress assuming a 2-span continuous condition. Where panels are continuous over 3 or more spans the tabulated values shall be permitted to be increased in accordance with the ASD/LRFD Manual for Engineered Wood Construction.
6. Strength axis is defined as the axis parallel to the face and back orientation of the flake or the grain (veneer), which is generally the long panel direction, unless otherwise marked.

**Must Span strength direction perpendicular to studs**

**Must have at least two spans continuous**

\[
\phi_b = 0.85
\]

\[
R = 135 \text{ psf} \cdot \phi_b = 114.75 \text{ psf} > \text{max pressure}
\]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Withdrawal Resistance:

\[ N_s = \frac{4 \text{ ft}}{6 \text{ in}} = 8 \] number of nails per stud

\[ N = N_s \cdot 4 = 32 \] nails per panel

\[ p_{min} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -1290.24 \text{ lbf} \]

\[ W := \frac{41 \text{ lbf}}{\text{in}} \quad L_d := 2.5 \text{ in} \quad L_m := 2.5 \text{ in} - \frac{7}{16} \text{ in} = 2.063 \text{ in} \]

\[ W_{sd} := W \cdot L_m = 84.563 \text{ lbf} \quad K_{fu} := 3.32 \quad \phi_w := 0.65 \]

\[ W_{sd} := W_{sd} \cdot K_{fu} \cdot \phi_w = 811.738 \text{ N} \]

\[ W'_{sd} \cdot N = 5839.548 \text{ lbf} \quad \text{Capacity} > \text{Load} \]

Roof:

Bending Resistance:

![Diagram of roof structure]

Table 3.2.2 Nominal Uniform Load Capacities (psf) for Roof Sheathing Resisting Out-of-Plane Wind Loads

<table>
<thead>
<tr>
<th>Sheathing Type 1</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in.)</th>
<th>Nominal Uniform Loads (psf)</th>
<th>Nominal Uniform Loads (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Structural Panels</td>
<td>24/0</td>
<td>3/8</td>
<td>425</td>
<td>425</td>
</tr>
<tr>
<td>(Sheathing Grades, C-C)</td>
<td>24/16</td>
<td>7/16</td>
<td>540</td>
<td>540</td>
</tr>
<tr>
<td>C-D, C-C Plugged, OSB</td>
<td>32/16</td>
<td>15/32</td>
<td>625</td>
<td>625</td>
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<tr>
<td>40/20</td>
<td>19/32</td>
<td>955</td>
<td>955</td>
<td></td>
</tr>
<tr>
<td>49/24</td>
<td>23/32</td>
<td>1160</td>
<td>1160</td>
<td></td>
</tr>
<tr>
<td>60/40</td>
<td>29/32</td>
<td>1250</td>
<td>1250</td>
<td></td>
</tr>
<tr>
<td>80/50</td>
<td>35/32</td>
<td>1330</td>
<td>1330</td>
<td></td>
</tr>
<tr>
<td>96/60</td>
<td>41/32</td>
<td>1410</td>
<td>1410</td>
<td></td>
</tr>
</tbody>
</table>

1. Nominal capacities shall be adjusted in accordance with Section 3.2.3 to determine ASD uniform load capacity and LRFD uniform resistances.
2. Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
3. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 3 or more plies.
4. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 3 or more plies.
5. Wood structural panels shall conform to the requirements for its type in DOC P8 1 or P8 2.
6. Tabulated values are for maximum bending loads from wind. Loads are limited by bending or shear stress assuming a 2-span continuous condition. Where panels are continuous over 3 or more spans, the tabulated values shall be permitted to be increased in accordance with the ASD-LRFD Manual for Engineered Wood Construction.
7. Strength axis is defined as the axis parallel to the face and back orientation of the flake or the grain (veneer), which is generally the long panel direction, unless otherwise marked.
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

\[
p_{3\text{ pos}} = 13.44 \text{ psf} \\
p_{2\text{ pos}} = 10.64 \text{ psf} \\
p_{1\text{ prime pos}} = 10.64 \text{ psf} \\
p_{1\text{ pos}} = 13.44 \text{ psf} \\
\]

\[
P_{\text{max}} := \max (p_{3\text{ pos}}, p_{2\text{ pos}}, p_{1\text{ pos}}, p_{1\text{ prime pos}}) = 13.44 \text{ psf} \\
\]

\[
p_{3\text{ neg}} = -84.56 \text{ psf} \\
p_{2\text{ neg}} = -39.76 \text{ psf} \\
p_{1\text{ prime neg}} = -20.16 \text{ psf} \\
p_{1\text{ neg}} = -34.16 \text{ psf} \\
\]

\[
P_{\text{min}} := \min (p_{3\text{ neg}}, p_{2\text{ neg}}, p_{1\text{ neg}}, p_{1\text{ prime neg}}) = -84.56 \text{ psf} \\
\]

**Must Span strength direction perpendicular to studs**  
**Must have at least two spans continuous**  
\[
\phi_b := 0.85 \\
R := 135 \text{ psf} \cdot \phi_b = 114.75 \text{ psf} > \text{ max pressure} \\
\]

**Withdrawal Resistance:**  
\[
N_s := \frac{4 \text{ ft}}{6 \text{ in}} = 8 \quad \text{number of nails per stud} \\
N := N_s \cdot 4 = 32 \quad \text{nails per panel} \\
p_{\text{min}} = -84.56 \text{ psf} \\
p_{\text{min}} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -2705.92 \text{ lbf} \\
\]

\[
W := 41 \frac{\text{lbf}}{\text{in}} \\
L_d := 2.5 \text{ in} \\
L_m := 2.5 \text{ in} - \frac{1}{2} \text{ in} = 2 \text{ in} \\
\]

\[
W_{\text{8d}} := W \cdot L_m = 82 \text{ lbf} \\
K_{\text{fw}} := 3.32 \quad \phi_w := 0.65 \\
W'_{\text{8d}} := W_{\text{8d}} \cdot K_{\text{fw}} \cdot \phi_w = 787.14 \text{ N} \\
W_{\text{8d}} \cdot N = 5662.592 \text{ lbf} \quad \text{Capacity > Load} \\
\]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

180 MPH:

Assumptions:
- Partially Open Enclosure Classification
- Roof Slope 3 Degrees

Step 1: Risk Category II
Step 2: Basic Wind Speed = 180 mph  \( V := 180 \)
Step 3:
- Wind Directionality factor, \( K_d := 0.85 \)
- Exposure Category B
- Topographic Factor \( K_z := 1 \)
- Ground Elevation Factor \( K_e := 1 \)
- Gust-effect Factor \( G := 0.85 \)
- Enclosure Classification Partially Open
- Internal Pressure Coefficient \( GC_{pi} := 0.18 \)

Step 4: Velocity pressure exposure coefficient \( K_z := 0.57 \)
Step 5: Velocity pressure
\[
q_z := \left(0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2\right) \cdot psf = 40.186 \text{ psf}
\]
Use: \( q_z := 41 \text{ psf} \)
Step 6: External pressure coefficient

### Notation

- \( B \) = Horizontal dimension of building, in ft (m), measured normal to wind direction.
- \( L \) = Horizontal dimension of building, in ft (m), measured parallel to wind direction.
- \( h \) = Mean roof height, in ft (m), except that eave height shall be used for \( 0 \leq 10 \) degrees.
- \( z \) = Height above ground, in ft (m).
- \( G \) = Gust-effect factor.
- \( q_z, q_0 \) = Velocity pressure, in lb/ft\(^2\) (N/m\(^2\)), evaluated at respective height.
- \( \theta \) = Angle of plane of roof from horizontal, in degrees.
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Wind Perpendicular to Ridge:
\[ L := 19.5 \text{ ft} \quad h := 8.5 \text{ ft} \quad \text{Height to peak of roof} \]
\[ B := 20 \text{ ft} \quad z := 7.5 \text{ ft} \quad \text{Height to eave} \]

\[ \frac{L}{B} = 0.975 \]

Wall Pressure Coefficients:

<table>
<thead>
<tr>
<th>Surface</th>
<th>( L/B )</th>
<th>( C_p )</th>
<th>Use With</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>All values</td>
<td>0.8</td>
<td>( q_w )</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>-0.5</td>
<td>( q_w )</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>2</td>
<td>-0.3</td>
<td>( q_w )</td>
</tr>
<tr>
<td>Sidewall</td>
<td>All values</td>
<td>-0.7</td>
<td>( q_w )</td>
</tr>
</tbody>
</table>

Roof Pressure Coefficients, \( C_p \) for use with \( q_s \):

<table>
<thead>
<tr>
<th>Wind Direction</th>
<th>( h )</th>
<th>( C_p )</th>
<th>Use With</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal to Ridge</td>
<td>( 0 ) to ( 15^\circ )</td>
<td>0 to ( 1/2 )</td>
<td>( q_s )</td>
</tr>
<tr>
<td>Normal</td>
<td>( 0 ) to ( 15^\circ )</td>
<td>0 to ( 1/2 )</td>
<td>( q_s )</td>
</tr>
<tr>
<td>0 to ( 15^\circ )</td>
<td>( 1/2 ) to ( 20^\circ )</td>
<td>( -0.5 )</td>
<td>( q_s )</td>
</tr>
<tr>
<td>0 to ( 15^\circ )</td>
<td>( 20^\circ ) to ( 30^\circ )</td>
<td>( -0.6 )</td>
<td>( q_s )</td>
</tr>
<tr>
<td>0 to ( 15^\circ )</td>
<td>( 30^\circ ) to ( 45^\circ )</td>
<td>( -0.7 )</td>
<td>( q_s )</td>
</tr>
</tbody>
</table>

*Value is provided for interpolation purposes.

\[ h \]
\[ \frac{L}{B} = 0.436 \]

\[ C_{p_{r, \perp}} = -0.9 \]

Roof pressures will cancel in X-direction

\[ P_{ww, ext} := q_s \cdot G \cdot C_{p_{ww}} = 27.88 \text{ psf} \]
\[ P_{lw, ext} := q_s \cdot G \cdot C_{p_{lw, \perp}} = -17.425 \text{ psf} \]
\[ P_{r, ext} := q_s \cdot G \cdot C_{p_{r, \perp}} = -31.365 \text{ psf} \]
\[ P_{int} := q_s \cdot G \cdot C_{p_{int}} = 7.38 \text{ psf} \]

roof pressure:
\[ P_r := P_{r, ext} - 1 \cdot P_{int} = -38.745 \text{ psf} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

**Forces:**

Windward Wall:

\[ P_{ww\_ext} = 27.88 \text{ psf} \]

\[ F_{ww} = P_{ww\_ext} \cdot \left( \frac{h}{2} \right) = 118.49 \text{ plf} \]

Line load along top and bottom of wall

Leeward Wall:

\[ P_{lw\_ext} = -17.425 \text{ psf} \quad P_{int} = 7.38 \text{ psf} \]

\[ F_{lw} := (P_{lw\_ext} - P_{int}) \cdot \left( \frac{h}{2} \right) = -105.421 \text{ plf} \]

Line load along top and bottom of wall

Roof:

\[ P_r = -38.745 \text{ psf} \]

x-comp:

\[ P_{rx} := P_r \cdot \sin(3 \text{ deg}) = -2.028 \text{ psf} \]

y-comp:

\[ P_{ry} := P_r \cdot \cos(3 \text{ deg}) = -38.692 \text{ psf} \]

Each shear wall Perp to ridge

Shear at base of wall:

\[ V_b := F_{top} + F_{bot} = 4.5 \text{ kip} \]

\[ h = 2.591 \text{ m} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Load Case:

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \quad (EQ \ 4.3.2-4) \]

Overturning Moments:

- Overturning moment from Ftop:
  \[ M_{F_{\text{top}}} := F_{\text{top}} \cdot z = 16.793 \text{ kip}\cdot\text{ft} \]

- Overturning moment from Roof Uplift:
  \[ M_{F_{\text{up}}} := -F_{r_y} \cdot 22 \text{ ft} \cdot \frac{L}{2} = 82.994 \text{ kip}\cdot\text{ft} \]

- Resisting moment from Dead and Live:
  \[ M_r := (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot \frac{L}{2} = 91.64 \text{ kip}\cdot\text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F_{\text{top}}} + M_{F_{\text{up}}} - M_r}{L - 1 \text{ ft}} = 0.44 \text{ kip} \]

Required Compression Force:

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot L = 9.839 \text{ kip} \]

Required Studs for compression:

- CLT Capacity:
  \[ F' := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

- Light-Frame Capacity:
  \[ P' := 5 \text{ kip} \]

\[ \frac{C}{P'} = 1.968 \quad \text{Need at least 2 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \text{ Connections} \]

\[ V' := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V_n} = 0.011 \frac{1}{N} \cdot \text{ft} \quad \text{of Hinge along wall} \]
## Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

**Load Case:**

\[
\mathbf{0.9} \times \mathbf{DL + W} \quad \text{(EQ 4.3.2-5)}
\]

**Overturning Moments:**

\[
L = 5.944 \text{ m} \quad B = 6.096 \text{ m}
\]

Overturning moment from Ftop:

\[
M_{F,\text{top}} := F_{\text{top}} \cdot z = 16.793 \text{ kip} \cdot \text{ft}
\]

Overturning moment from Trib:

\[
Trib := B \quad \text{and} \quad DL_r := 23.5 \text{ psf}
\]

Overturning moment from Ftop:

\[
M_{F,\text{top}} := 23.5 \text{ psf} \quad \text{and} \quad LL_r := 20 \text{ psf}
\]

**Required force in hold down:**

\[
T := \frac{M_{F,\text{top}} + M_{F,\text{up}}}{L - 1 \text{ ft}} = 5.394 \text{ kip}
\]

**Required Compression Force:**

\[
C := T = 5.394 \text{ kip}
\]

**Required Studs for compression:**

- CLT Capacity:
  \[F' := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay}\]
- Light-Frame Capacity:
  \[P' := 5 \text{ kip}\]

\[
\frac{C}{P'} = 1.079 \quad \text{Need at least 2 studs in corner for compression}
\]

**Required hinge length for base shear:**

- Ultimate Strength of Hinge A in Shear
  \[\phi := 0.65 \quad \text{Connections}\]

\[
V'_{n} := 3.85 \text{ klf}
\]

For Base shear we would need:

\[
\frac{V}{V'_{n}} = 0.011 \frac{1}{N} \text{ ft} \quad \text{of Hinge along wall}
\]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Wood Sheathing Nailing to Resist Shear: (LF UNITS)  \( \phi_D := 0.8 \)

Shearwall:

\[
V := \frac{F_{top}}{L \cdot \phi_D} = 143.533 \text{ plf}
\]

Sheathing:
- Structural 1
  - 7/16" 24/16 Span Rating

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Fastener Penetration in Framing Member or Blocking (in.)</th>
<th>Fastener Type &amp; Size</th>
<th>Panel Edge Fastener Spacing (in.)</th>
<th>( V_{top} ) (plf)</th>
<th>( V_{top} ) (plf)</th>
<th>( V_{top} ) (plf)</th>
<th>( V_{top} ) (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Structural Panel</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>645</td>
<td>645</td>
<td>645</td>
<td>645</td>
<td>645</td>
</tr>
<tr>
<td>Wood Structural Panel -</td>
<td>3/8</td>
<td>1-1/4</td>
<td>8d</td>
<td>715</td>
<td>715</td>
<td>715</td>
<td>715</td>
<td>715</td>
</tr>
<tr>
<td>Wood Structural Panel -</td>
<td>7/16</td>
<td>1-3/8</td>
<td>8d</td>
<td>765</td>
<td>765</td>
<td>765</td>
<td>765</td>
<td>765</td>
</tr>
<tr>
<td>Wood Structural Panel -</td>
<td>15/32</td>
<td>1-1/2</td>
<td>8d</td>
<td>950</td>
<td>950</td>
<td>950</td>
<td>950</td>
<td>950</td>
</tr>
<tr>
<td>Wood Structural Panel</td>
<td>5/16</td>
<td>1-1/4</td>
<td>10d</td>
<td>615</td>
<td>615</td>
<td>615</td>
<td>615</td>
<td>615</td>
</tr>
<tr>
<td>Wood Structural Panel -</td>
<td>3/8</td>
<td>1-1/4</td>
<td>10d</td>
<td>670</td>
<td>670</td>
<td>670</td>
<td>670</td>
<td>670</td>
</tr>
<tr>
<td>Wood Structural Panel -</td>
<td>7/16</td>
<td>1-3/8</td>
<td>10d</td>
<td>730</td>
<td>730</td>
<td>730</td>
<td>730</td>
<td>730</td>
</tr>
<tr>
<td>Wood Structural Panel -</td>
<td>15/32</td>
<td>1-1/2</td>
<td>10d</td>
<td>870</td>
<td>870</td>
<td>870</td>
<td>870</td>
<td>870</td>
</tr>
<tr>
<td>Plywood Siding</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>390</td>
<td>390</td>
<td>390</td>
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<td>390</td>
</tr>
<tr>
<td>Plywood Siding</td>
<td>3/8</td>
<td>1-1/4</td>
<td>8d</td>
<td>450</td>
<td>450</td>
<td>450</td>
<td>450</td>
<td>450</td>
</tr>
</tbody>
</table>

Use 8d nails with 6" spacing along edge

Diaphragm:

\[
V_{wd} := V = 143.533 \text{ plf}
\]

\[
V_{wd} := 670 \text{ plf} \quad \text{okay}
\]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Wind Parallel To Ridge:

\[ L := 20 \text{ ft} \quad h := 8.5 \text{ ft} \quad B := 19.5 \text{ ft} \quad z := 7.5 \text{ ft} \]

Height to peak & used for wall height entire length

\[ \frac{L}{B} = 1.026 \]

Moment arm for overturning moment from F top

Wall Pressure Coefficients:

Wall Pressure Coefficients, \( C_p \)

<table>
<thead>
<tr>
<th>Surface</th>
<th>( \frac{L}{B} )</th>
<th>( C_p )</th>
<th>Use With</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>All values</td>
<td>0.8</td>
<td>( q_z )</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>0.1</td>
<td>-0.5</td>
<td>( q_z )</td>
</tr>
<tr>
<td>Sidewall</td>
<td>2</td>
<td>-0.3</td>
<td>( q_z )</td>
</tr>
</tbody>
</table>

Windward Wall \( C_{p_{ww}} := 0.8 \)

Leeward \( C_{p_{lw_{perp}}} := -0.5 \)

Sidewall \( C_{p_{sw}} := -0.7 \)

Roof Pressure Coefficients:

Roof Pressure Coefficients:

\[ \frac{h}{L} = 0.425 \]

\[ C_{p_{r_{perp}}} := -0.9 \]

Wall Pressure:

\[ P_{ww_{ext}} := q_z \cdot G \cdot C_{p_{ww}} = 27.88 \text{ psf} \]

\[ P_{lw_{ext}} := q_z \cdot G \cdot C_{p_{lw_{perp}}} = -17.425 \text{ psf} \]

\[ P_{r_{ext}} := q_z \cdot G \cdot C_{p_{r_{perp}}} = -31.365 \text{ psf} \]

\[ P_{int} := q_z \cdot G \cdot C_{pi} = 7.38 \text{ psf} \]

\[ P_r := P_{r_{ext}} + -1 \cdot P_{int} = -38.745 \text{ psf} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

**Forces:**

Windward Wall:

\[ P_{ww,ext} = 27.88 \text{ psf} \]

\[ F_{ww} := P_{ww,ext} \cdot \left( \frac{h}{2} \right) = 118.49 \text{ plf} \]  
Line load along top and bottom of wall

Leeward Wall:

\[ P_{lw,ext} = -17.425 \text{ psf} \]

\[ F_{lw} := (P_{lw,ext} - P_{int}) \cdot \left( \frac{h}{2} \right) = -105.421 \text{ plf} \]  
Line load along top and bottom of wall

Roof:

\[ P_{ry} := P_{r} = -38.745 \text{ psf} \]

Each shear wall Par to ridge

\[ F_{top} := \frac{F_{ww} + F_{lw}}{2} = 2.183 \text{ kip} \]

- Half of top of wall (ww & lw) line load

\[ F_{bot} := \frac{F_{ww} - F_{lw}}{2} = 2.183 \text{ kip} \]

- Half of bot of wall (ww & lw) line load

\[ F_{ry} := P_{ry} \cdot \left( \frac{B}{2} \right) = -377.764 \text{ plf} \]

- Uplift line load

Shear at base of wall:

\[ V_b := F_{top} + F_{bot} = 4.366 \text{ kip} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Load Case:

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \quad \text{(EQ 4.3.2-4)} \]

Overturning Moments:

\[ L = 6.096 \, m \quad B = 5.944 \, m \]

\[ DL_r := 23.5 \, psf \]

\[ LL_r := 20 \, psf \]

\[ Trib := B / 2 \]

Overturning moment from Ftop:

\[ M_{F, \text{top}} := F_{\text{top}} \cdot z = 16.374 \, kip \cdot ft \]

Overturning moment from Roof Uplift:

\[ M_{F, \text{up}} := -F_{r,y} \cdot 22 \, ft \cdot \frac{L}{2} = 83.108 \, kip \cdot ft \]

Required force in hold down:

\[ T := \frac{M_{F, \text{top}} + M_{F, \text{up}} - M_r}{L - 1 \, ft} = 0.289 \, kip \]

Required Compression Force:

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 9.688 \, kip \]

Required Studs for compression:

CLT Capacity:

\[ F' := 35849 \, plf = 35.849 \, klf \quad \text{Okay} \]

Light-Frame Capacity:

\[ P' := 5 \, kip \]

\[ C = 1.938 \quad \text{Need at least 2 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V' := 3.85 \, klf \]

For Base shear we would need:

\[ \frac{V}{V'} = 0.122 \, \frac{1}{m} \cdot ft \quad \text{of Hinge along wall} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Load Case:

\[ 0.9 \times DL + W \]  

(EQ 4.3.2-5)

Overturning Moments:  
\[ L = 6.096 \text{ m} \quad B = 5.944 \text{ m} \]
\[ DL_r := 23.5 \text{ psf} \]
\[ LL_r := 20 \text{ psf} \]
\[ Trib := \frac{B}{2} \]

Overturning moment from Ftop:
\[ M_{F, top} := F_{top} \cdot z = 16.374 \text{ kip} \cdot \text{ft} \]

Overturning moment from Roof Uplift:
\[ M_{F, up} := -F_{r-y} \cdot 22 \text{ ft} \cdot \frac{L}{2} = 83.108 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**
\[ T := \frac{M_{F, top} + M_{F, up}}{L - 1 \text{ ft}} = 5.236 \text{ kip} \]
\[ M_{F, top} + M_{F, up} = 99.482 \text{ kip} \cdot \text{ft} \]

**Required Compression Force:**
\[ C := T = 5.236 \text{ kip} \]

**Required Studs for compression:**
CLT Capacity:
\[ F' := 35849 \text{ plf} = 35.849 \text{ klf} \]
Okay
Light-Frame Capacity:
\[ P' := 5 \text{ kip} \]
\[ C \quad P' = 1.047 \quad \text{Need at least 2 studs in corner for compression} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V' := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V'_n} = 0.122 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Wood Sheathing Nailing to Resist Shear: (LF UNITS)
\[ V := \frac{F_{top}}{L \cdot \phi_D} = 136.446 \text{ plf} \]

Shearwall:
\[ \phi_D := 0.8 \]

Sheathing:
Structural 1
7/16''
24/16 Span Rating

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Fastener Penetration In Framing Member or Blocking (in.)</th>
<th>Fastener Type &amp; Size</th>
<th>Panel Edge Fastener Spacing (in.)</th>
<th>Wind</th>
<th>Sheathing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td>6</td>
<td>4</td>
</tr>
<tr>
<td>Wood Structural Panels - Structural</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>560</td>
<td>840</td>
<td>1090</td>
</tr>
<tr>
<td></td>
<td>3/8</td>
<td>1-1/4</td>
<td>8d</td>
<td>645</td>
<td>1010</td>
<td>1390</td>
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<tr>
<td></td>
<td>7/16</td>
<td>1-3/8</td>
<td></td>
<td>715</td>
<td>1185</td>
<td>1415</td>
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<tr>
<td></td>
<td>19/32</td>
<td>1-3/8</td>
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<td>785</td>
<td>1265</td>
<td>1540</td>
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<tr>
<td></td>
<td>15/32</td>
<td>1-1/2</td>
<td>100</td>
<td>950</td>
<td>1430</td>
<td>1860</td>
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<td>1-1/4</td>
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<td>505</td>
<td>755</td>
<td>980</td>
</tr>
<tr>
<td></td>
<td>3/8</td>
<td>1-1/4</td>
<td>8d</td>
<td>560</td>
<td>840</td>
<td>1090</td>
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<tr>
<td></td>
<td>7/16</td>
<td>1-3/8</td>
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<td>615</td>
<td>895</td>
<td>1150</td>
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<td></td>
<td>19/32</td>
<td>1-3/8</td>
<td></td>
<td>670</td>
<td>990</td>
<td>1260</td>
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<td></td>
<td>15/32</td>
<td>1-1/2</td>
<td>100</td>
<td>730</td>
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<td>Plywood Siding</td>
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<td>390</td>
<td>590</td>
<td>770</td>
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<tr>
<td></td>
<td>3/8</td>
<td>1-1/4</td>
<td>8d</td>
<td>450</td>
<td>670</td>
<td>870</td>
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<tr>
<td>Particleboard Sheathing - (M-S &quot;Exterior Glue&quot; and M-2 &quot;Exterior Glue&quot;)</td>
<td>3/8</td>
<td>1-1/4</td>
<td>6d</td>
<td>335</td>
<td>505</td>
<td>645</td>
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<tr>
<td></td>
<td>1/2</td>
<td>1-1/4</td>
<td>8d</td>
<td>365</td>
<td>530</td>
<td>670</td>
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<tr>
<td></td>
<td>3/8</td>
<td>1-3/8</td>
<td></td>
<td>390</td>
<td>590</td>
<td>755</td>
</tr>
<tr>
<td></td>
<td>1/2</td>
<td>1-3/8</td>
<td>100</td>
<td>520</td>
<td>779</td>
<td>1016</td>
</tr>
<tr>
<td></td>
<td>1/2</td>
<td>1-1/2</td>
<td></td>
<td>560</td>
<td>855</td>
<td>1195</td>
</tr>
<tr>
<td>Structural Fiberboard Sheathing</td>
<td>25/32</td>
<td></td>
<td></td>
<td>475</td>
<td>645</td>
<td>730</td>
</tr>
</tbody>
</table>

\[ V_{td} := 715 \text{ plf} \]
Use 8d nails with 6" spacing along edge

Diaphragm:
\[ V_d := V = 136.446 \text{ plf} \]
\[ V_{wd} := 670 \text{ plf} \]
\[ V_{wd} \] okay
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Components and Cladding (C&C):

Step 1: Risk Category II
Step 2: Basic Wind Speed = 180 mph $V := 180$
Step 3:
- Wind Directionality $K_d := 0.85$
- Exposure Category B
- Topographic Factor $K_{zt} := 1$
- Ground Elevation Factor $K_e := 1$
- Enclosure Classification Partially Open
- Internal Pressure Coefficient $GC_{pi} := 0.18$
Step 4: Velocity pressure exposure coefficient $K_h := 0.57$
Step 5: Velocity pressure $q_h := (0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2) \cdot psf = 40.186 \text{ psf}$
Step 6: External pressure coefficient

Walls:

Diagram

\[
\begin{align*}
\theta &:= 3 \text{ deg} \\
A_{5\text{perp}} &:= a \cdot z = 2.09 \text{ m}^2 \\
A_{5\text{par}} &:= z \cdot a = 2.09 \text{ m}^2 \\
A_{4\text{perp}} &:= (L - 2 \cdot a) \cdot z = 9.755 \text{ m}^2 \\
A_{4\text{par}} &:= (B - 2 \cdot a) \cdot z = 9.406 \text{ m}^2
\end{align*}
\]

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Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

**External Pressure Coefficient, \((GC_p)\) - Walls**

<table>
<thead>
<tr>
<th>Effective Wind Area, (\text{ft}^2 \ (m^2))</th>
<th>(GC_p)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>+1.8</td>
</tr>
<tr>
<td>500</td>
<td>+0.7</td>
</tr>
<tr>
<td>0.1</td>
<td>+0.81</td>
</tr>
<tr>
<td>0.3</td>
<td>+0.9</td>
</tr>
<tr>
<td>0.9</td>
<td>+1.1</td>
</tr>
<tr>
<td>(1.63)</td>
<td>+1.4</td>
</tr>
<tr>
<td>(2.63)</td>
<td>+1.4</td>
</tr>
</tbody>
</table>

**Notes**

1. Vertical scale denotes \((GC_p)\) to be used with \(q_h\).
2. Horizontal scale denotes effective wind area, in \(\text{ft}^2 \ (m^2)\).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of \((GC_p)\) for walls shall be reduced by 10% when \(\theta \leq 10^\circ\).

**Positive:**

\[ GC_{p,5\_pos} := 1 \cdot 0.9 = 0.9 \]
\[ GC_{p,4\_pos} := 0.9 \cdot 0.9 = 0.81 \]

**Negative:**

\[ GC_{p,5\_neg} := -1.4 \cdot 0.9 = -1.26 \]
\[ GC_{p,4\_neg} := -1 \cdot 0.9 = -0.9 \]

**Design Pressures:**

\[ p_{5\_pos} := q_h \cdot (GC_{p,5\_pos} + GC_{pi}) = 44.28 \text{ psf} \]
\[ p_{4\_pos} := q_h \cdot (GC_{p,4\_pos} + GC_{pi}) = 40.59 \text{ psf} \]
\[ p_{5\_neg} := q_h \cdot (GC_{p,5\_neg} - GC_{pi}) = -59.04 \text{ psf} \]
\[ p_{4\_neg} := q_h \cdot (GC_{p,4\_neg} - GC_{pi}) = -44.28 \text{ psf} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Roof:

**Diagrams**

**Notation**

- $B =$ Horizontal dimension of building measured normal to wind direction, in ft (m).
- $h =$ Eave height shall be used for $\theta = 10^\circ$.
- $\theta =$ Angle of plane of roof from horizontal, in degrees.

**External Pressure Coefficients**

**Notes**

1. Vertical scale denotes $(G_{Cp})$ to be used with $\varphi_r$.
2. Horizontal scale denotes effective wind area, in $ft^2$ (m²).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. If a parapet equal to or higher than 3 ft (0.9 m) is provided around the perimeter of the roof with $\theta \leq 7^\circ$, the negative values of $(G_{Cp})$ in Zone 3 shall be equal to those for Zone 2, and positive values of $(G_{Cp})$ in Zones 2 and 3 shall be set equal to those for wall Zones 4 and 5, respectively, in Fig. 30.3-1.
6. Values of $(G_{Cp})$ for roof overhangs include pressure contributions from both upper and lower surfaces.
7. If overhangs exist, the lesser horizontal dimension of the building shall not include any overhang dimension, but the edge distance, $a$, shall be measured from the outside edge of the overhang.
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

\[ A_3 := ((0.6 \cdot h) \cdot (0.2 \cdot h) \cdot 2) - (0.2 \cdot h)^2 = 1.342 \text{ m}^2 \]

\[ A_2 := (2 \cdot B \cdot 0.6 \cdot h) + (2 \cdot L \cdot 0.6 \cdot h) - 4 \cdot (0.6 \cdot h)^2 - 4 \cdot A_3 = 22.395 \text{ m}^2 \]

\[ A_{1,\text{prime}} := (B - 2.4 \cdot h) \cdot (L - 2.4 \cdot h) = 0.033 \text{ m}^2 \]

\[ A_1 := (B \cdot L) - A_{1,\text{prime}} - A_2 - 4 \cdot A_3 = 8.434 \text{ m}^2 \]

Positive:
\[ GC_{p,3,\text{pos}} := 0.3 \]
\[ GC_{p,2,\text{pos}} := 0.2 \]
\[ GC_{p,1,\text{prime, pos}} := 0.2 \]
\[ GC_{p,1,\text{pos}} := 0.3 \]

Negative:
\[ GC_{p,3,\text{neg}} := -3.2 \]
\[ GC_{p,2,\text{neg}} := -1.6 \]
\[ GC_{p,1,\text{prime, neg}} := -0.9 \]
\[ GC_{p,1,\text{neg}} := -1.4 \]

Design Pressures:
\[ p_{3,\text{pos}} := q_h \cdot (GC_{p,3,\text{pos}} + GC_{pi}) = 19.68 \text{ psf} \]
\[ p_{2,\text{pos}} := q_h \cdot (GC_{p,2,\text{pos}} + GC_{pi}) = 15.58 \text{ psf} \]
\[ p_{1,\text{prime, pos}} := q_h \cdot (GC_{p,1,\text{prime, pos}} + GC_{pi}) = 15.58 \text{ psf} \]
\[ p_{1,\text{pos}} := q_h \cdot (GC_{p,1,\text{pos}} + GC_{pi}) = 19.68 \text{ psf} \]

\[ p_{3,\text{neg}} := q_h \cdot (GC_{p,3,\text{neg}} + GC_{pi}) = -123.82 \text{ psf} \]
\[ p_{2,\text{neg}} := q_h \cdot (GC_{p,2,\text{neg}} + GC_{pi}) = -58.22 \text{ psf} \]
\[ p_{1,\text{prime, neg}} := q_h \cdot (GC_{p,1,\text{prime, neg}} + GC_{pi}) = -29.52 \text{ psf} \]
\[ p_{1,\text{neg}} := q_h \cdot (GC_{p,1,\text{neg}} + GC_{pi}) = -50.02 \text{ psf} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Sheathing Design: (For Light-Frame Unit)

Wall:
Bending Resistance:
- 24" spacing of joists and studs
- 24/16 span rating 7/16" thick

\[ p_{5, pos} = 44.28 \text{ psf} \]
\[ p_{4, pos} = 40.59 \text{ psf} \]
\[ p_{5, neg} = -59.04 \text{ psf} \]
\[ p_{4, neg} = -44.28 \text{ psf} \]

\[ l_b := 24 \text{ in} \]

\[ p_{\text{max}} := \max (p_{5, pos}, p_{4, pos}, p_{5, neg}, p_{4, neg}) = 44.28 \text{ psf} \]
\[ p_{\text{min}} := \min (p_{5, pos}, p_{4, pos}, p_{5, neg}, p_{4, neg}) = -59.04 \text{ psf} \]

Must Span strength direction perpendicular to studs
Must have at least two spans continuous

\[ \phi_b := 0.85 \]
\[ R := 135 \text{ psf} \cdot \phi_b = 114.75 \text{ psf} > \text{max pressure} \]
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

Withdrawal Resistance:

\[
N_s := \frac{4 \text{ ft}}{6 \text{ in}} = 8 \quad \text{number of nials per stud}
\]

\[
N := N_s \cdot 4 = 32 \quad \text{nails per panel}
\]

\[
p_{\text{min}} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -1889.28 \text{ lbf}
\]

\[
W := 41 \text{ lbf} \quad L_d := 2.5 \text{ in} \quad L_m := 2.5 \text{ in} - \frac{7}{16} \text{ in} = 2.063 \text{ in}
\]

\[
W_{\text{sd}} := W \cdot L_m = 84.563 \text{ lbf} \quad K_{\text{fw}} := 3.32 \quad \phi_w := 0.65
\]

\[
W'_{\text{sd}} := W_{\text{sd}} \cdot K_{\text{fw}} \cdot \phi_w = 811.738 \text{ N}
\]

\[
W'_{\text{sd}} \cdot N = 5839.548 \text{ lbf} \quad \text{Capacity > Load}
\]

Roof:

Bending Resistance:

![Diagram of roof structure]

Table 3.2.2 Nominal Uniform Load Capacities (psf) for Roof Sheathing Resisting Out-of-Plane Wind Loads

<table>
<thead>
<tr>
<th>Sheathing Type (^1)</th>
<th>Span Rating or Grade</th>
<th>Minimum Thickness (in.)</th>
<th>Strength Axis (^2) Applied Perpendicular to Supports</th>
<th>Strength Axis (^2) Applied Parallel to Supports</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Rafter/Truss Spacing (in.)</td>
<td>Rafter/Truss Spacing (in.)</td>
</tr>
<tr>
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<td></td>
<td>12 16 19.2 24 32 48</td>
<td>12 16 24</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Nominal Uniform Loads (psf)</td>
<td>Nominal Uniform Loads (psf)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wood Structural Panels</th>
<th>Wood Structural Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Sheathing Grades, C-C)</td>
<td>(Sheathing Grades, C-C)</td>
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<tr>
<td>24/0</td>
<td>24/0</td>
</tr>
<tr>
<td>24/16</td>
<td>24/16</td>
</tr>
<tr>
<td>32/16</td>
<td>32/16</td>
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<tr>
<td>40/24</td>
<td>40/24</td>
</tr>
<tr>
<td>48/24</td>
<td>48/24</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>C-D, C-C Plugged, OSB</th>
<th>C-D, C-C Plugged, OSB</th>
</tr>
</thead>
<tbody>
<tr>
<td>16 19/32</td>
<td>16 19/32</td>
</tr>
<tr>
<td>20 19/32</td>
<td>20 19/32</td>
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<tr>
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<td>28 19/32</td>
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<td>32 19/32</td>
<td>32 19/32</td>
</tr>
<tr>
<td>40 19/32</td>
<td>40 19/32</td>
</tr>
</tbody>
</table>

\(1\) Nominal capacities shall be adjusted in accordance with Section 3.2.3 to determine ASD uniform load capacity and LRFD uniform resistances.

\(2\) Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.

\(3\) Tabulated values are based on the lesser of nominal values for either OSB or plywood with 3 or more plies.

\(4\) Wood structural panels shall conform to the requirements for its type in DOC PS 1 or PS 2.

\(5\) Strength axis is defined as the axis parallel to the face and back orientation of the flake or the grain (veseer), which is generally the long panel direction, unless otherwise marked.
Appendix F. Mathcad Calculations for Wind Loads: Dynamic Units

\[ p_{3, \text{pos}} = 19.68 \text{ psf} \]
\[ p_{2, \text{pos}} = 15.58 \text{ psf} \]
\[ p_{1, \text{prime, pos}} = 15.58 \text{ psf} \]
\[ p_{1, \text{pos}} = 19.68 \text{ psf} \]

\[ p_{\text{max}} := \max (p_{3, \text{pos}}, p_{2, \text{pos}}, p_{1, \text{pos}}, p_{1, \text{prime, pos}}) = 19.68 \text{ psf} \]

\[ p_{3, \text{neg}} = -123.82 \text{ psf} \]
\[ p_{2, \text{neg}} = -58.22 \text{ psf} \]
\[ p_{1, \text{prime, neg}} = -29.52 \text{ psf} \]
\[ p_{1, \text{neg}} = -50.02 \text{ psf} \]

\[ p_{\text{min}} := \min (p_{3, \text{neg}}, p_{2, \text{neg}}, p_{1, \text{neg}}, p_{1, \text{prime, neg}}) = -123.82 \text{ psf} \]

**Must Span strength direction perpendicular to studs**
**Must have at least two spans continuous**

\[ \phi_b := 0.85 \]
\[ R := 135 \text{ psf} \cdot \phi_b = 114.75 \text{ psf} \quad < \text{max pressure} \]

Need to move up to

- 0.5"
- 32/16 span rating

for roof

\[ R := 155 \text{ psf} \cdot \phi_b = 131.75 \text{ psf} \quad > \text{max pressure} \]

Withdrawal Resistance:

\[ N_s := \frac{4 \text{ ft}}{6 \text{ in}} = 8 \quad \text{number of nials per stud} \]
\[ N := N_s \cdot 4 = 32 \quad \text{nails per panel} \]
\[ p_{\text{min}} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = -3962.24 \text{ lbf} \]
\[ W := 41 \text{ lbf in} \]
\[ L_d := 2.5 \text{ in} \]
\[ L_m := 2.5 \text{ in} - \frac{1}{2} \text{ in} = 2 \text{ in} \]
\[ W_{8d} := W \cdot L_m = 82 \text{ lbf} \quad K_{fw} := 3.32 \quad \phi_w := 0.65 \]
\[ W'_{8d} := W_{8d} \cdot K_{fw} \cdot \phi_w = 787.14 \text{ N} \]
\[ W_{8d} \cdot N = 5662.592 \text{ lbf} \quad \text{Capacity} > \text{Load} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

### Medium Seismic Condition:

**Weight:**

\[ LF = 7.5 \text{ psf} \quad \text{Weight of Light-Frame wall} \]

Square footage of wall:

\[ A := (2 \cdot 8 \cdot 20 + 8.5 \cdot 9 + 4.5 \cdot 9 + 7.5 \cdot 9 + 20 \cdot 9) \cdot ft^2 = 684.5 \text{ ft}^2 \]

\[ W := LF \cdot A = 5.134 \text{ kip} \quad W_2 := \frac{W}{2} = 2.567 \text{ kip} \quad W_1 := \frac{W}{2} = 2.567 \text{ kip} \]

**Ground Motion:**

\[ S_{MS} := 1 \quad g \]

\[ S_{M1} := 0.6 \quad g \]

\[ S_{DS} := \frac{2}{3} \cdot S_{MS} = 0.667 \]

\[ S_{D1} := \frac{2}{3} \cdot S_{M1} = 0.4 \]

**Design Response Spectrum:**

\[ T_n = C_t \cdot h_n^x \quad \text{Approximate Period} \]

\[ C_t := 0.02 \]

\[ h_n := \frac{h_1 + h_2}{ft} \quad \text{Unit Height + Potential CMU Height} \]

\[ x := 0.75 \quad \text{Table 12.8-2 ASCE 7-16} \]

\[ T_n := (C_t \cdot h_n^x) \cdot s = 0.117 \text{ s} \]

\[ C_u := 1.4 \]

\[ C_u T_a := C_u \cdot T_n = 0.163 \text{ s} \quad \text{Upper limit for period} \]

\[ T_s := \left( \frac{S_{D1}}{S_{DS}} \right) \cdot s = 0.6 \text{ s} \]

\[ T_0 := 0.2 \cdot T_s = 0.12 \text{ s} \quad T_L := 8 \text{ s} \]

\[ T := 0 \text{ s}, 0.01 \text{ s}, 0.10 \text{ s} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

\[ S_a(T) := \begin{cases} 
\text{if } T \leq T_0 \\
S_{DS} \cdot \left(0.4 + \left(0.6 \cdot \frac{T}{T_0}\right)\right) \\
S_{DS} \\
S_{DS} \cdot \frac{s}{T} \\
S_{DS} \cdot \frac{T_L \cdot s}{T^2}
\end{cases} \]

\( S_a(T) \)

Conservative to use \( S_{DS} \)

\( R := 6.5 \quad I_e := 1 \)

\[ C_s := \min \left\{ \frac{S_{DS}}{R}, \frac{S_{D1} \cdot s}{I_e \cdot R}, \frac{0.44 \cdot S_{DS} \cdot I_e + 0.01, 0.5 \cdot S_{M1}}{R} \right\} = 0.103 \]

\( C_{s, min} := \max \left( 0.44 \cdot S_{DS} \cdot I_e, 0.01, 0.5 \cdot S_{M1} \right) = 0.046 \)

\[ C_s := \max (C_s, C_{s, min}) = 0.103 \quad C_s := \frac{S_{DS}}{R} = 0.103 \]

\( V := C_s \cdot W = 0.527 \text{ kip} \)
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Vertical Force Distribution:

\[ F_x = C_{vx} \cdot V \]

\[ C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k} \]

\[ k(T) = \begin{cases} 1 & \text{if } T_n \leq 0.5 \text{ s} \\ 2 & \text{else if } T_n \geq 2.5 \text{ s} \\ 1 + \frac{(T_n - 0.5 \text{ s})}{(2.5 \text{ s} - 0.5 \text{ s})} \cdot (2 - 1) & \text{else if } 0.5 \text{ s} < T_n < 2.5 \text{ s} \end{cases} \]

\[ k := k(T_n) = 1 \]

\[ C_{v2} := \frac{W_2 \cdot (h_1 + h_2)^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_1 + h_2)^k)} = 0.91 \]

\[ C_{v1} := \frac{W_1 \cdot h_1^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_1 + h_2)^k)} = 0.09 \]

\[ F_2 := C_{v2} \cdot V = 0.481 \text{ kip} \]

\[ F_1 := C_{v1} \cdot V = 0.046 \text{ kip} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Load Case: Direction 1

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S) \text{ or } R \]  
\[ (EQ \ 4.3.2-A) \]

Overturning Moments:
\[
L := 20 \ \text{ft} \quad B := 8 \ \text{ft} \\
DL_r := 20 \ \text{psf} \quad LL_r := 20 \ \text{psf} \quad Trib := \frac{B}{2} 
\]

Overturning moment from F2:
\[ M_{F,\text{top}} := F_2 \cdot h_2 = 4.567 \ \text{kip}\cdot\text{ft} \]

Resisting moment from Dead and Live:
\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 35.2 \ \text{kip}\cdot\text{ft} \]

Required force in hold down:
\[ T := \frac{M_{F,\text{top}} - M_r}{L - 1 \ \text{ft}} = -1.612 \ \text{kip} \]

Required Compression Force:
\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 1.908 \ \text{kip} \]

Required Studs for compression:

Light-Frame Capacity:
\[ P' := 5 \ \text{kip} \]
\[ \frac{C}{P'} = 0.382 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V_n' := 3.85 \ \text{klf} \]

For Base shear we would need:
\[ \frac{V}{V_n'} = 0.137 \ \text{ft} \quad \text{of Hinge along wall} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Load Case: Direction 1

\[ 0.9 \times DL + W \]  \hspace{1cm} (EQ 4.3.2-5)

Overturning Moments:
\[
L = 6.096 \text{ m} \quad B = 2.438 \text{ m}
\]
\[
DL_r := 20 \text{ psf} \\
LL_r := 20 \text{ psf} \\
Trib := \frac{B}{2}
\]

Overturning moment from Ftop:
\[
M_{F_{\text{top}}} := F_2 \cdot h_2 = 4.567 \text{ kip} \cdot \text{ft}
\]

Required force in hold down:
\[
T := \frac{M_{F_{\text{top}}}}{L - 1 \text{ ft}} = 0.24 \text{ kip}
\]

Required Compression Force:
\[
C := T = 0.24 \text{ kip}
\]

Required Studs for compression:

Light-Frame Capacity:
\[
P' := 5 \text{ kip}
\]

\[
C = 0.048 \quad \frac{C}{P'} = 0.048 \quad \text{Need at least 1 studs in corner for compression}
\]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear
\[
\phi := 0.65 \quad \text{Connections}
\]

\[
V_n' := 3.85 \text{ klf}
\]

For Base shear we would need:
\[
\frac{V}{V_n'} = 0.137 \text{ ft} \quad \text{of Hinge along wall}
\]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Load Case: Direction 2

\[ 1.2 \times DL + W + LL + 0.5 \times (L_F \text{ or } S \text{ or } R) \]  

(\text{EQ } 4.3.2-4)

Overturning Moments: 

\[ L := 8 \text{ ft} \quad B := 20 \text{ ft} \]

Overturning moment from F2:

\[ M_{F_{\text{top}}} := F_2 \times h_2 = 4.567 \text{ kip} \times \text{ft} \]

Resisting moment from Dead and Live:

\[ M_r := (1.2 \times DL_r + LL_r) \times Trib \times L \times \frac{L}{2} = 14.08 \text{ kip} \times \text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F_{\text{top}}} - M_r}{L - 1 \text{ ft}} = -1.359 \text{ kip} \]

Required Compression Force:

\[ C := T + (1.2 \times DL_r + LL_r) \times Trib \times L = 2.161 \text{ kip} \]

Required Studs for compression:

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ \frac{C}{P'} = 0.432 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 0.137 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Load Case: Direction 2

\[ 0.9 \times DL + W \quad (EQ \ 4.3.2-5) \]

Overturning Moments:
\[ L = 2.438 \ m \quad B = 6.096 \ m \]
\[ DL_r := 20 \ psf \]
\[ LL_r := 20 \ psf \]
\[ Trib := \frac{B}{2} \]

Overturning moment from Ftop:
\[ M_{F, top} := F_2 \cdot h_2 = 4.567 \ kip \cdot ft \]

Required force in hold down:
\[ T := \frac{M_{F, top}}{L - 1 \ ft} = 0.652 \ kip \]
\[ M_{F, top} = 4.567 \ kip \cdot ft \]

Required Compression Force:
\[ C := T = 0.652 \ kip \]

Required Studs for compression:

Light-Frame Capacity:
\[ P' := 5 \ kip \]
\[ C \quad P' = 0.13 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_n := 3.85 \ klf \]

For Base shear we would need:
\[ \frac{V}{V'_n} = 0.137 \ ft \quad \text{of Hinge along wall} \]
## High Seismic Condition:

**Weight:**

\[ LF := 7.5 \text{ psf} \]

Weight of Light-Frame wall

Square footage of wall:

\[ A := (2 \cdot 8 \cdot 20 + 8.5 \cdot 9 + 4.5 \cdot 9 + 7.5 \cdot 9 + 20 \cdot 9) \cdot ft^2 = 684.5 \text{ ft}^2 \]

\[ W := LF \cdot A = 5.134 \text{ kip} \]

\[ W_2 := \frac{W}{2} = 2.567 \text{ kip} \]

\[ W_1 := \frac{W}{2} = 2.567 \text{ kip} \]

**Ground Motion:**

\[ S_{MS} := 1.5 \text{ g} \]

\[ S_{M1} := 0.9 \text{ g} \]

\[ S_{DS} := \frac{2}{3} \cdot S_{MS} = 1 \]

\[ S_{D1} := \frac{2}{3} \cdot S_{M1} = 0.6 \]

**Design Response Spectrum:**

\[ T_n := C_t \cdot h_n^x \]

Approximate Period

\[ C_t := 0.02 \]

\[ h_n := \frac{h_1 + h_2}{ft} \]

Unit Height + Potential CMU Height

\[ x := 0.75 \]

Table 12.8-2 ASCE 7-16

\[ T_n := \left(C_t \cdot h_n^x\right) \cdot s = 0.117 \text{ s} \]

\[ C_u := 1.4 \]

\[ C_u T_n := C_u \cdot T_n = 0.163 \text{ s} \]

Upper limit for period

\[ T_s := \left(S_{D1} \cdot S_{DS}\right) \cdot s = 0.6 \text{ s} \]

\[ T_0 := 0.2 \cdot T_s = 0.12 \text{ s} \]

\[ T_L := 8 \text{ s} \]

\[ T := 0 \text{ s} \text{,} 0.01 \text{ s} \text{,} \ldots \text{,} 10 \text{ s} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

\[ S_a(T) := \begin{cases} 
\text{if } T \leq T_0 & S_{DS} \cdot \left( 0.4 + 0.6 \cdot \frac{T}{T_0} \right) \\
\text{if } T_0 < T \leq T_s & S_{DS} \\
\text{if } T_s < T \leq T_L & S_{D1} \cdot \frac{s}{T} \\
\text{else if } T > T_L & \frac{S_{D1} \cdot T_L \cdot s}{T^2} 
\end{cases} \]

Conservative to use \( S_{DS} \)

\[ R := 6.5 \quad I_e := 1 \]

\[ C_s := \min \left( \frac{S_{DS}}{R \cdot \frac{I_e}{T_n}}, \frac{S_{D1} \cdot s}{T \cdot \frac{R}{I_e}} \right) = 0.154 \]

\[ C_{s,min} := \max \left( 0.044 \cdot S_{DS} \cdot I_e, 0.01, \frac{0.5 \cdot S_{M1}}{R \cdot \frac{I_e}{I_e}} \right) = 0.069 \]

\[ C_s := \max (C_s, C_{s,min}) = 0.154 \]

\[ V := C_s \cdot W = 0.79 \text{ kip} \]

\[ S_a(T) \]

\( T \) (s)

\( T_n \) (s)

\( C_u T_a \) (s)

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Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Vertical Force Distribution:

\[ F_x = C_{vx} \cdot V \]

\[ C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k} \]

\[ k(T) := \begin{cases} 
1 & \text{if } T_n \leq 0.5 \text{ s} \\
2 & \text{else if } T_n \geq 2.5 \text{ s} \\
1 + \frac{(T_n - 0.5 \text{ s})}{(2.5 \text{ s} - 0.5 \text{ s})} \cdot (2 - 1) & \text{else if } 0.5 \text{ s} < T_n < 2.5 \text{ s} 
\end{cases} \]

\[ k := k(T_n) = 1 \]

\[ C_{v2} := \frac{W_2 \cdot (h_1 + h_2)^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_1 + h_2)^k)} = 0.91 \]

\[ C_{v1} := \frac{W_1 \cdot h_1^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_1 + h_2)^k)} = 0.09 \]

\[ F_2 := C_{v2} \cdot V = 0.721 \text{ kip} \]

\[ F_1 := C_{v1} \cdot V = 0.069 \text{ kip} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Load Case: Direction 1

\[ 1.2 \times DL + W + LL + 0.5 \times (L_T \text{ or } S \text{ or } R) \]
\[ \text{(EQ 4.3.2-4)} \]

**Overturning Moments:**

\[ L := 20 \text{ ft} \]
\[ B := 8 \text{ ft} \]
\[ DL_r := 20 \text{ psf} \]
\[ LL_r := 20 \text{ psf} \]
\[ Trib := \frac{B}{2} \]

Overturning moment from F2:
\[ M_{F,\text{top}} := F_2 \cdot h_2 = 6.851 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:
\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot \frac{L}{2} = 35.2 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**
\[ T := \frac{M_{F,\text{top}} - M_r}{L - 1 \text{ ft}} = -1.492 \text{ kip} \]

**Required Compression Force:**
\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 2.028 \text{ kip} \]

**Required Studs for compression:**

Light-Frame Capacity:
\[ P' := 5 \text{ kip} \]

\[ \frac{C}{P'} = 0.406 \quad \text{Need at least 1 studs in corner for compression} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V'_{n}} = 0.205 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

<table>
<thead>
<tr>
<th>Load Case: Direction 1</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>0.9 × DL + W</strong></td>
</tr>
<tr>
<td>(EQ 4.3.2-5)</td>
</tr>
<tr>
<td><strong>Overturning Moments:</strong></td>
</tr>
<tr>
<td>( L = 6.096 , m )</td>
</tr>
<tr>
<td>( B = 2.438 , m )</td>
</tr>
<tr>
<td>( DL_r := 20 , psf )</td>
</tr>
<tr>
<td>( LL_r := 20 , psf )</td>
</tr>
<tr>
<td>( Trib := \frac{B}{2} )</td>
</tr>
</tbody>
</table>

| **Overturning moment from Ftop:** |
| \( M_{F,\text{top}} := F_2 \cdot h_2 = 6.851 \, kip \cdot ft \) |

| **Required force in hold down:** |
| \( T := \frac{M_{F,\text{top}}}{L - 1 \, ft} = 0.361 \, kip \) |
| \( M_{F,\text{top}} = 6.851 \, kip \cdot ft \) |

| **Required Compression Force:** |
| \( C := T = 0.361 \, kip \) |

| **Required Studs for compression:** |
| Light-Frame Capacity: |
| \( P' := 5 \, kip \) |
| \( \frac{C}{P'} = 0.072 \) Need at least 1 studs in corner for compression |

| **Required hinge length for base shear:** |
| Ultimate Strength of Hinge A in Shear |
| \( \phi := 0.65 \) Connections |
| \( V'_n := 3.85 \, klf \) |
| For Base shear we would need: |
| \( \frac{V}{V'_n} = 0.205 \, ft \) of Hinge along wall |
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Load Case: Direction 2

\[
1.2 \times DL + W + LL + 0.5 \times (L_f \text{ or } S \text{ or } R) \quad (EQ \ 4.3.2-4)
\]

**Overturning Moments:**

- Overturning moment from F2:
  \[
  M_{F\_top} := F_2 \cdot h_2 = 6.851 \text{ kip} \cdot \text{ft}
  \]

Resisting moment from Dead and Live:

- Resisting moment:
  \[
  M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 14.08 \text{ kip} \cdot \text{ft}
  \]

**Required force in hold down:**

\[
T := \frac{M_{F\_top} - M_r}{L - 1 \text{ ft}} = -1.033 \text{ kip}
\]

**Required Compression Force:**

\[
C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 2.487 \text{ kip}
\]

**Required Studs for compression:**

Light-Frame Capacity:

\[
P' := 5 \text{ kip}
\]

\[
\frac{C}{P'} = 0.497 \quad \text{Need at least 1 studs in corner for compression}
\]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[
\phi := 0.65 \quad \text{Connections}
\]

\[
V'_{n} := 3.85 \text{ klf}
\]

For Base shear we would need:

\[
\frac{V}{V'_n} = 0.205 \text{ ft} \quad \text{of Hinge along wall}
\]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Load Case: Direction 2

**0.9 \times DL + W** \hspace{1cm} (EQ 4.3.2-5)

Overturning Moments:

\[
\begin{align*}
L &= 2.438 \text{ m} \\
B &= 6.096 \text{ m}
\end{align*}
\]

Overturning moment from Ftop:

\[
M_{F,\text{top}} := F_2 \cdot h_2 = 6.851 \text{ kip} \cdot \text{ft}
\]

**Required force in hold down:**

\[
T := \frac{M_{F,\text{top}}}{L - 1 \text{ ft}} = 0.979 \text{ kip}
\]

**Required Compression Force:**

\[
C := T = 0.979 \text{ kip}
\]

**Required Studs for compression:**

Light-Frame Capacity:

\[
P' := 5 \text{ kip}
\]

\[
C = 0.196 \hspace{1cm} \frac{C}{P'} = 0.196 \hspace{1cm} \text{Need at least 1 studs in corner for compression}
\]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[
\phi := 0.65 \hspace{1cm} \text{Connections}
\]

\[
V_n' := 3.85 \text{ klf}
\]

For Base shear we would need:

\[
\frac{V}{V_n'} = 0.205 \text{ ft} \hspace{1cm} \text{of Hinge along wall}
\]
**Very High Seismic Condition:**

Weight:

\[ LF = 7.5 \text{ psf} \]

Weight of Light-Frame wall

\[ W := LF \cdot A = 5.134 \text{ kip} \]

Square footage of wall:

\[ A := (2 \cdot 8 \cdot 20 + 8.5 \cdot 9 + 4.5 \cdot 9 + 7.5 \cdot 9 + 20 \cdot 9) \cdot ft^2 = 684.5 \text{ ft}^2 \]

\[ W := LF \cdot A = 5.134 \text{ kip} \quad W_2 := \frac{W}{2} = 2.567 \text{ kip} \quad W_1 := \frac{W}{2} = 2.567 \text{ kip} \]

Ground Motion:

\[ S_{MS} := 2.25 \ G \]

\[ S_{M1} := 1.35 \ G \]

\[ S_{DS} := \frac{2}{3} \cdot S_{MS} = 1.5 \]

\[ S_{D1} := \frac{2}{3} \cdot S_{M1} = 0.9 \]

Design Response Spectrum:

\[ T_n = C_t \cdot h_n \]

Approximate Period

\[ C_t := 0.02 \]

\[ h_n := \frac{h_1 + h_2}{ft} \]

Unit Height + Potential CMU Height

\[ x := 0.75 \]

Table 12.8-2 ASCE 7-16

\[ T_n := \left(C_t \cdot h_n^x \right) s = 0.117 \text{ s} \]

\[ C_t := 1.4 \]

\[ C_a T_a := C_u \cdot T_n = 0.163 \text{ s} \]

Upper limit for period

\[ T_s := \left(\frac{S_{D1}}{S_{DS}}\right) \cdot s = 0.6 \text{ s} \]

\[ T_0 := 0.2 \cdot T_s = 0.12 \text{ s} \]

\[ T_L := 8 \text{ s} \]

\[ T := 0 \text{ s}, 0.01 \text{ s} \ldots 10 \text{ s} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

\[ S_a(T) := \begin{cases} 
  \text{if } T \leq T_0 \\
  \left| S_{DS} \cdot \left( 0.4 + \left( 0.6 \cdot \frac{T}{T_0} \right) \right) \right| \\
  \text{else if } T_0 < T \leq T_s \\
  S_{DS} \\
  \text{else if } T_s < T \leq T_L \\
  S_{D1} \cdot \frac{s}{T} \\
  \text{else if } T > T_L \\
  S_{D1} \cdot \frac{T_L \cdot s}{T^2} 
\end{cases} \]

Conservative to use \( S_{DS} \)

\[ R := 6.5 \quad I_e := 1 \]

\[ C_s := \min \left\{ \frac{S_{DS}}{R}, \frac{S_{D1} \cdot s}{I_e \cdot T_n} \right\} = 0.231 \]

\[ C_{s,\text{min}} := \max \left\{ 0.044 \cdot S_{DS} \cdot I_e, 0.01, \frac{0.5 \cdot S_{M1}}{R \cdot I_e} \right\} = 0.104 \]

\[ C_s := \max (C_s, C_{s,\text{min}}) = 0.231 \]

\[ C_s := \frac{S_{DS}}{R} = 0.231 \]

\[ V := C_s \cdot W = 1.185 \text{ kip} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Vertical Force Distribution:
\[ F_x = C_{vx} \cdot V \]

\[ C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k} \]

\[ k(T) := \begin{cases} 1 & \text{if } T_n \leq 0.5 \text{ s} \\ 2 & \text{else if } T_n \geq 2.5 \text{ s} \\ 1 + \frac{(T_n - 0.5 \text{ s})}{(2.5 \text{ s} - 0.5 \text{ s})} & \text{else if } 0.5 \text{ s} < T_n < 2.5 \text{ s} \end{cases} \]

\[ k := k(T_n) = 1 \]

\[ C_{v2} := \frac{W_2 \cdot (h_1 + h_2)^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_1 + h_2)^k)} = 0.91 \]

\[ C_{v1} := \frac{W_1 \cdot h_1^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_1 + h_2)^k)} = 0.09 \]

\[ F_2 := C_{v2} \cdot V = 1.082 \text{ kip} \]

\[ F_1 := C_{v1} \cdot V = 0.103 \text{ kip} \]
Load Case: Direction 1

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \]  \hspace{1cm} (EQ 4.3.2-4)

Overturning Moments:

- Overturning moment from F2:
  \[ M_{F,\text{top}} := F_2 \cdot h_2 = 10.276 \text{ kip} \cdot ft \]

Resisting moment from Dead and Live:

- \[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 35.2 \text{ kip} \cdot ft \]

**Required force in hold down:**

\[ T := \frac{M_{F,\text{top}} - M_r}{L - 1 \text{ ft}} = -1.312 \text{ kip} \]

**Required Compression Force:**

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 2.208 \text{ kip} \]

**Required Studs for compression:**

- Light-Frame Capacity:
  \[ P' := 5 \text{ kip} \]

  \[ \frac{C}{P'} = 0.442 \quad \text{Need at least 1 studs in corner for compression} \]

**Required hinge length for base shear:**

- Ultimate Strength of Hinge A in Shear
  \[ \phi := 0.65 \quad \text{Connections} \]

  \[ V'_{n} := 3.85 \text{ klf} \]

  For Base shear we would need:

  \[ \frac{V}{V'_{n}} = 0.308 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Load Case: Direction 1

\[ 0.9 \times DL + W \] \quad (EQ 4.3.2-5)

Overturning Moments:
\[ L = 6.096 \text{ m} \quad B = 2.438 \text{ m} \]
\[ DL_r := 20 \text{ psf} \]
\[ LL_r := 20 \text{ psf} \]
\[ Trib := \frac{B}{2} \]

Overturning moment from Ftop:
\[ M_{F,\text{top}} := F_2 \cdot h_2 = 10.276 \text{ kip} \cdot \text{ft} \]

Required force in hold down:
\[ T := \frac{M_{F,\text{top}}}{L - 1 \text{ ft}} = 0.541 \text{ kip} \]
\[ M_{F,\text{top}} = 10.276 \text{ kip} \cdot \text{ft} \]

Required Compression Force:
\[ C := T = 0.541 \text{ kip} \]

Required Studs for compression:

Light-Frame Capacity:
\[ P' := 5 \text{ kip} \]
\[ \frac{C}{P'} = 0.108 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V'_{n}} = 0.308 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Load Case: Direction 2

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \]  \hspace{1cm} (EQ 4.3.2-4)

Overturning Moments:

- \( L := 8 \text{ ft} \)
- \( B := 20 \text{ ft} \)
- \( DL_r := 20 \text{ psf} \)
- \( LL_r := 20 \text{ psf} \)
- \( \text{Trib} := \frac{B}{2} \)

Overturning moment from F2:

\[ M_{F_{\text{top}}} := F_2 \cdot h_2 = 10.276 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:

\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot L \cdot \frac{L}{2} = 14.08 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**

\[ T := \frac{M_{F_{\text{top}}} - M_r}{L - 1 \text{ ft}} = -0.543 \text{ kip} \]

**Required Compression Force:**

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot L = 2.977 \text{ kip} \]

**Required Studs for compression:**

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ \frac{C}{P'} = 0.595 \quad \text{Need at least 1 studs in corner for compression} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\( \phi := 0.65 \quad \text{Connections} \)

\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 0.308 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix G. Mathcad Calculations for Seismic Loads: Light-Frame Static Unit

Load Case: Direction 2

\[ 0.9 \times DL + W \quad (EQ \ 4.3.2-5) \]

Overturning Moments:
\[ L = 2.438 \ m \quad B = 6.096 \ m \]
\[ DL_r := 20 \ \text{psf} \]
\[ LL_r := 20 \ \text{psf} \]
\[ Trib := \frac{B}{2} \]

Overturning moment from Ftop:
\[ M_{F,\text{top}} := F_2 \cdot h_2 = 10.276 \ \text{kip} \cdot ft \]

Required force in hold down:
\[ T := \frac{M_{F,\text{top}}}{L - 1 \ ft} = 1.468 \ \text{kip} \quad M_{F,\text{top}} = 10.276 \ \text{kip} \cdot ft \]

Required Compression Force:
\[ C := T = 1.468 \ \text{kip} \]

Required Studs for compression:

Light-Frame Capacity:
\[ P' := 5 \ \text{kip} \]
\[ \frac{C}{P'} = 0.294 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_n := 3.85 \ \text{klf} \]

For Base shear we would need:
\[ \frac{V}{V'_n} = 0.308 \ \text{ft} \quad \text{of Hinge along wall} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit
Medium Seismic Condition:

Weight:

\[ W := 11.974 \text{ kip} \]

Effective seismic weight:

\[ W_2 := \frac{W}{2} = 5.987 \text{ kip} \quad W_1 := \frac{W}{2} = 5.987 \text{ kip} \]

Ground Motion:

\[ h_1 := 1 \text{ ft} \quad h_2 := 8.5 \text{ ft} \]

\[ S_{MS} := 1 \quad g \]

\[ S_{M1} := 0.6 \quad g \]

\[ S_{DS} := \frac{2}{3} \cdot S_{MS} = 0.667 \]

\[ S_{D1} := \frac{2}{3} \cdot S_{M1} = 0.4 \]

Design Response Spectrum:

\[ T_n = C_t \cdot h_n^x \quad \text{Approximate Period} \]

\[ C_t := 0.02 \]

\[ h_n := \frac{h_1 + h_2}{\text{ft}} \quad \text{Unit Height + Potential CMU Height} \]

\[ x := 0.75 \]

Table 12.8-2 ASCE 7-16

\[ T_n := \left( C_t \cdot h_n^x \right) \cdot s = 0.108 \text{ s} \]

\[ C_a := 1.4 \]

\[ C_a T_a := C_a \cdot T_n = 0.152 \text{ s} \quad \text{Upper limit for period} \]

\[ T_s := \left( \frac{S_{D1}}{S_{DS}} \right) \cdot s = 0.6 \text{ s} \]

\[ T_0 := 0.2 \cdot T_s = 0.12 \text{ s} \]

\[ T_L := 8 \text{ s} \]

\[ T := 0, 0.01, 0.02, \ldots, 10 \text{ s} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

\[ S_a(T) := \begin{cases} \frac{S_{DS} \cdot \left(0.4 + \left(0.6 \cdot \frac{T}{T_0}\right)\right)}{T} & \text{if } T \leq T_0 \\ \frac{S_{DS}}{T} & \text{if } T_0 < T \leq T_s \\ S_{DS} & \text{if } T_s < T \leq T_L \\ \frac{S_{DL} \cdot \frac{s}{T}}{T^2} & \text{if } T > T_L \end{cases} \]

Conservative to use \( S_{DS} \)

\[ C_s := \min \left( \frac{S_{DS}}{R}, \frac{S_{DL} \cdot s}{I_e} \right) = 0.103 \]

\[ C_{s, min} := \max \left( 0.044 \cdot S_{DS} \cdot I_e, 0.01, \frac{0.5 \cdot S_{ML}}{R} \right) = 0.046 \]

\[ V := C_s \cdot W = 1.228 \text{ kip} \]

\[ C_s := \frac{S_{DS}}{R} = 0.103 \]

\[ C_s := \max \left( C_s, C_{s, min} \right) = 0.103 \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Vertical Force Distribution:

\[ F_x = C_{vx} \cdot V \]

\[ C_{vx} = \frac{\sum_{i=1}^{n} w_i \cdot h_i^k}{w_x \cdot h_x^k} \]

\[ k(T) := \begin{cases} 1 & \text{if } T_n \leq 0.5 \text{ s} \\ 2 & \text{if } 0.5 \text{ s} < T_n < 2.5 \text{ s} \\ 1 + \frac{(T_n - 0.5 \text{ s})}{(2.5 - 0.5 \text{ s})} \cdot (2 - 1) & \text{if } T_n \geq 2.5 \text{ s} \end{cases} \]

\[ k := k(T_n) = 1 \]

\[ C_{v2} := \frac{W_2 \cdot (h_2 + h_1)^k}{\left( W_1 \cdot (h_1)^k \right) + \left( W_2 \cdot (h_2 + h_1)^k \right)} = 0.9 \]

\[ C_{v1} := \frac{W_1 \cdot (h_1)^k}{\left( W_1 \cdot (h_1)^k \right) + \left( W_2 \cdot (h_2 + h_1)^k \right)} = 0.1 \]

\[ F_2 := C_{v2} \cdot V = 1.111 \text{ kip} \]

\[ F_1 := C_{v1} \cdot V = 0.117 \text{ kip} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Load Case: Direction 1

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \]  \hspace{1cm} (EQ 4.3.2-4)

<table>
<thead>
<tr>
<th>Overturning Moments:</th>
<th>( L := 20 \text{ ft} )</th>
<th>( B := 19.5 \text{ ft} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( DL_r := 20 \text{ psf} )</td>
<td>( LL_r := 20 \text{ psf} )</td>
<td>( \text{Trib} := \frac{B}{2} )</td>
</tr>
</tbody>
</table>

Overturning moment from F2:

\[ M_{F,\text{top}} := F_2 \cdot h_2 = 9.445 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:

\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot L \cdot \frac{L}{2} = 85.8 \text{ kip} \cdot \text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F,\text{top}} - M_r}{L - 1} \text{ ft} = -4.019 \text{ kip} \]

Required Compression Force:

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot L = 4.561 \text{ kip} \]

Required Studs for compression:

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ \frac{C}{P'} = 0.912 \]  \hspace{1cm} Need at least 1 studs in corner for compression

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \]  \hspace{1cm} Connections

\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 0.319 \text{ ft} \]  \hspace{1cm} of Hinge along wall
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Load Case: Direction 1

\[ 0.9 \times DL + W \]  \hspace{1cm} (EQ 4.3.2-5)

Overturning Moments:

\[ L = 6.096 \text{ m} \quad B = 5.944 \text{ m} \]
\[ DL_r := 20 \text{ psf} \]
\[ LL_r := 20 \text{ psf} \]
\[ Trib := \frac{B}{2} \]

Overturning moment from Ftop:
\[ M_{F,\text{top}} := F_2 \cdot h_2 = 9.445 \text{ kip} \cdot \text{ft} \]

Required force in hold down:
\[ T := \frac{M_{F,\text{top}}}{L - 1 \text{ ft}} = 0.497 \text{ kip} \]
\[ M_{F,\text{top}} = 9.445 \text{ kip} \cdot \text{ft} \]

Required Compression Force:
\[ C := T = 0.497 \text{ kip} \]

Required Studs for compression:

Light-Frame Capacity:
\[ P' := 5 \text{ kip} \]
\[ C \]
\[ P' = 0.099 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V'_n} = 0.319 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Load Case: Direction 2

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \quad \text{(EQ 4.3.2-4)} \]

Overturning Moments:

\[
L := 19.5 \text{ ft} \quad B := 20 \text{ ft} \\
DL_r := 20 \text{ psf} \\
LL_r := 20 \text{ psf} \\
\text{Trib} := \frac{B}{2}
\]

Overturning moment from F2:

\[
M_{F,\text{top}} := F_2 \cdot h_2 = 9.445 \text{ kip} \cdot \text{ft}
\]

Resisting moment from Dead and Live:

\[
M_r := (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot \frac{L}{2} = 83.655 \text{ kip} \cdot \text{ft}
\]

**Required force in hold down:**

\[
T := \frac{M_{F,\text{top}} - M_r}{L - 1} = -4.011 \text{ kip}
\]

**Required Compression Force:**

\[
C := T + (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot L = 4.569 \text{ kip}
\]

**Required Studs for compression:**

Light-Frame Capacity:

\[
P' := 5 \text{ kip}
\]

\[
\frac{C}{P'} = 0.914 \quad \text{Need at least 1 studs in corner for compression}
\]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[
\phi := 0.65 \quad \text{Connections}
\]

\[
V'_{n} := 3.85 \text{ klf}
\]

For Base shear we would need:

\[
\frac{V}{V'_{n}} = 0.319 \text{ ft} \quad \text{of Hinge along wall}
\]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Load Case: Direction 2

\[ 0.9 \times DL + W \]  \hspace{2cm} (EQ 4.3.2-5)

Overturning Moments:

\[ L = 5.944 \text{ m} \quad B = 6.096 \text{ m} \]

Overturning moment from Ftop:

\[ M_{F_{\text{top}}} := F_2 \cdot h_2 = 9.445 \text{ kip \cdot ft} \]

Required force in hold down:

\[ T := \frac{M_{F_{\text{top}}}}{L - 1 \text{ ft}} = 0.511 \text{ kip} \]

Required Compression Force:

\[ C := T = 0.511 \text{ kip} \]

Required Studs for compression:

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ \frac{C}{P'} = 0.102 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 0.319 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

**High Seismic Condition:**

Weight:

\[ W := 11.974 \text{ kip} \]

Effective seismic weight:

\[ W_2 := \frac{W}{2} = 5.987 \text{ kip} \quad W_1 := \frac{W}{2} = 5.987 \text{ kip} \]

Ground Motion:

\[ h_1 := 1 \text{ ft} \quad h_2 := 8.5 \text{ ft} \]

\[ S_{MS} := 1.5 \quad g \]

\[ S_{M1} := 0.9 \quad g \]

\[ S_{DS} := \frac{2}{3} \cdot S_{MS} = 1 \]

\[ S_{D1} := \frac{2}{3} \cdot S_{M1} = 0.6 \]

Design Response Spectrum:

\[ T_n = C_t \cdot h_n^x \quad \text{Approximate Period} \]

\[ C_t := 0.02 \]

\[ h_n := \frac{h_1 + h_2}{\text{ft}} \quad \text{Unit Height + Potential CMU Height} \]

\[ x := 0.75 \quad \text{Table 12.8-2 ASCE 7-16} \]

\[ T_n := \left( C_t \cdot h_n^x \right) \cdot s = 0.108 \text{ s} \]

\[ C_u := 1.4 \]

\[ C_u T_a := C_u \cdot T_n = 0.152 \text{ s} \quad \text{Upper limit for period} \]

\[ T_s := \left( S_{D1} \right) \cdot \left( S_{DS} \right) \cdot s = 0.6 \text{ s} \]

\[ T_0 := 0.2 \cdot T_s = 0.12 \text{ s} \]

\[ T_L := 8 \text{ s} \]

\[ T := 0 \text{ s} \quad .01 \text{ s} \quad .10 \text{ s} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

$S_a(T) := \begin{cases} \text{if } T \leq T_0 \\
S_{DS} \cdot \left(0.4 + \left(0.6 \cdot \frac{T}{T_0}\right)\right) \\
S_{DS} \\
S_{DS} \cdot s \\\n\frac{S_{DS} \cdot T \cdot s}{T^2} \end{cases}$

Conservative to use $S_{DS}$

$R := 6.5 \quad I_e := 1$

$C_s := \min \left(\frac{S_{DS}}{R}, \frac{S_{DS} \cdot s}{I_e \cdot T^2} \right) = 0.154$

$C_{s,min} := \max \left(0.044 \cdot S_{DS} \cdot I_e, 0.01 \cdot \frac{0.5 \cdot S_{M1}}{R \cdot I_e} \right) = 0.069$

$C_s := \max \left(C_s, C_{s,min}\right) = 0.154 \quad C_s := \frac{S_{DS}}{R} = 0.154$

$V := C_s \cdot W = 1.842 \text{ kip}$
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Vertical Force Distribution:

\[ F_x = C_{vx} \cdot V \]

\[ C_{vx} = \frac{\sum_{i=1}^{n} w_i \cdot h_i^k}{w_x \cdot h_x^k} \]

\[ k(T) := \begin{cases} 
1 & \text{if } T_n \leq 0.5 \text{ s} \\
2 & \text{else if } T_n \geq 2.5 \text{ s} \\
1 + \frac{(T_n - 0.5 \text{ s}) \cdot (2 - 1)}{(2.5 \text{ s} - 0.5 \text{ s})} & \text{else if } 0.5 \text{ s} < T_n < 2.5 \text{ s} 
\end{cases} \]

\[ k := k(T_n) = 1 \]

\[ C_{v2} := \frac{W_2 \cdot (h_2 + h_1)^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_2 + h_1)^k)} = 0.9 \]

\[ C_{v1} := \frac{W_1 \cdot h_1^k}{(W_1 \cdot (h_1^k) + (W_2 \cdot (h_2 + h_1)^k)} = 0.1 \]

\[ F_2 := C_{v2} \cdot V = 1.667 \text{ kip} \]

\[ F_1 := C_{v1} \cdot V = 0.175 \text{ kip} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

<table>
<thead>
<tr>
<th>Load Case: Direction 1</th>
<th>( 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) ) ((EQ 4.3.2-4))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overturning Moments:</td>
<td>( L := 20 \text{ ft} ) ( B := 19.5 \text{ ft} )</td>
</tr>
<tr>
<td>Overturning moment from F2:</td>
<td>( DL_r := 20 \text{ psf} ) ( LL_r := 20 \text{ psf} ) ( Trib := \frac{B}{2} )</td>
</tr>
<tr>
<td>( M_{F_{-top}} := F_2 \cdot h_2 = 14.167 \text{ kip} \cdot \text{ft} )</td>
<td></td>
</tr>
<tr>
<td>Resisting moment from Dead and Live:</td>
<td>( M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 85.8 \text{ kip} \cdot \text{ft} )</td>
</tr>
<tr>
<td>( T := \frac{M_{F_{-top}} - M_r}{L - 1 \text{ ft}} = -3.77 \text{ kip} )</td>
<td></td>
</tr>
<tr>
<td>( C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 4.81 \text{ kip} )</td>
<td></td>
</tr>
<tr>
<td>Required Studs for compression:</td>
<td>( Light-Frame \text{ Capacity:} ) ( P' := 5 \text{ kip} )</td>
</tr>
<tr>
<td>( \frac{C}{P'} = 0.962 ) Need at least 1 studs in corner for compression</td>
<td></td>
</tr>
<tr>
<td>( \text{Required hinge length for base shear:} )</td>
<td>( \text{Ultimate Strength of Hinge A in Shear} ) ( \phi := 0.65 ) Connections</td>
</tr>
<tr>
<td>( V'_{n} := 3.85 \text{ klf} )</td>
<td></td>
</tr>
<tr>
<td>( \frac{V}{V'_{n}} = 0.478 \text{ ft} ) of Hinge along wall</td>
<td></td>
</tr>
</tbody>
</table>
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Load Case: Direction 1

\[ 0.9 \times DL + W \]  \hspace{1cm} (EQ 4.3.2-5)

Overturning Moments:

\[ L = 6.096 \text{ m} \hspace{1cm} B = 5.944 \text{ m} \]

Overturning moment from Ftop:

\[ M_{F,\text{top}} := F_2 \cdot h_2 = 14.167 \text{ kip} \cdot \text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F,\text{top}}}{L - 1 \text{ ft}} = 0.746 \text{ kip} \]

\[ M_{F,\text{top}} = 14.167 \text{ kip} \cdot \text{ft} \]

Required Compression Force:

\[ C := T = 0.746 \text{ kip} \]

Required Studs for compression:

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ \frac{C}{P'} = 0.149 \quad \text{Need at least 1 studs in corner for compression} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 0.478 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Load Case: Direction 2

\[
1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \quad (Eq. 4.3.2-4)
\]

**Overturning Moments:**

- \( L := 19.5 \text{ ft} \)
- \( B := 20 \text{ ft} \)
- \( DL_r := 20 \text{ psf} \)
- \( LL_r := 20 \text{ psf} \)
- \( Trib := \frac{B}{2} \)

Overturning moment from F2:

\[
M_{F,\text{top}} := F_2 \cdot h_2 = 14.167 \text{ kip \cdot ft}
\]

Resisting moment from Dead and Live:

\[
M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 83.655 \text{ kip \cdot ft}
\]

**Required force in hold down:**

\[
T := \frac{M_{F,\text{top}} - M_r}{L - 1 \text{ ft}} = -3.756 \text{ kip}
\]

**Required Compression Force:**

\[
C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 4.824 \text{ kip}
\]

**Required Studs for compression:**

Light-Frame Capacity:

\[
P' := 5 \text{ kip}
\]

\[
C = 0.965 \quad P' = 0.965 \quad \text{Need at least 1 studs in corner for compression}
\]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[
\phi := 0.65 \quad \text{Connections}
\]

\[
V'_{n} := 3.85 \text{ klf}
\]

For Base shear we would need:

\[
\frac{V}{V'_{n}} = 0.478 \text{ ft} \quad \text{of Hinge along wall}
\]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Load Case: Direction 2

\[ 0.9 \times DL + W \quad (EQ \ 4.3.2-5) \]

<table>
<thead>
<tr>
<th>Overturning Moments:</th>
<th>[ L = 5.944 \text{ m} ]</th>
<th>[ B = 6.096 \text{ m} ]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overturning moment from Ftop:</td>
<td>[ DL_r := 20 \text{ psf} ]</td>
<td>[ LL_r := 20 \text{ psf} ]</td>
</tr>
<tr>
<td>[ M_{F_{top}} := F_2 \cdot h_2 = 14.167 \text{ kip \cdot ft} ]</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Required force in hold down:**

\[ T := \frac{M_{F_{top}}}{L - 1 \text{ ft}} = 0.766 \text{ kip} \]

\[ M_{F_{top}} = 14.167 \text{ kip \cdot ft} \]

**Required Compression Force:**

\[ C := T = 0.766 \text{ kip} \]

**Required Studs for compression:**

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ \frac{C}{P'} = 0.153 \quad \text{Need at least 1 studs in corner for compression} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V_n' := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V_n'} = 0.478 \text{ ft} \quad \text{of Hinge along wall} \]
# Very High Seismic Condition:

**Weight:**

\[ W := 11.974\text{ kip} \]

Effective seismic weight:

\[ W_2 := \frac{W}{2} = 5.987\text{ kip} \quad W_1 := \frac{W}{2} = 5.987\text{ kip} \]

**Ground Motion:**

\[ S_{MS} := 2.25\text{ g} \]
\[ S_{M1} := 1.35\text{ g} \]
\[ S_{DS} := \frac{2}{3}S_{MS} = 1.5 \]
\[ S_{D1} := \frac{2}{3}S_{M1} = 0.9 \]

**Design Response Spectrum:**

\[ T_n = C_t \cdot h_n^x \]

Approximate Period

\[ C_t := 0.02 \]

\[ h_n := \frac{h_1 + h_2}{\text{ft}} \quad \text{Unit Height + Potential CMU Height} \]

\[ x := 0.75 \quad \text{Table 12.8-2 ASCE 7-16} \]

\[ T_n := \left(C_t \cdot h_n^x\right) s = 0.108\text{ s} \]
\[ C_u := 1.4 \]

\[ T_s := \left(S_{D1}\right) \cdot \left(S_{DS}\right) s = 0.6\text{ s} \]

\[ C_u T_a := C_u \cdot T_n = 0.152\text{ s} \quad \text{Upper limit for period} \]

\[ T_0 := 0.2 \cdot T_s = 0.12\text{ s} \quad T_L := 8\text{ s} \]

\[ T := 0\text{ s}, 0.01\text{ s}, \ldots, 10\text{ s} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

\[ S_a(T) := \begin{cases} 
\text{if } T \leq T_0 \\
S_{DS} \cdot \left( 0.4 + \left( \frac{0.6 \cdot T}{T_0} \right) \right) \\
\text{else if } T_0 < T \leq T_s \\
S_{DS} \\
\text{else if } T_s < T \leq T_L \\
S_{D1} \cdot \frac{s}{T} \\
\text{else if } T > T_L \\
S_{D1} \cdot \frac{T_L \cdot s}{T^2} 
\end{cases} \]

Conservative to use \( S_{DS} \)

\[ C_s := \min \left( \frac{S_{DS}}{R}, \frac{S_{D1} \cdot s}{I_c} \right) = 0.231 \]

\[ C_{s,min} := \max \left( 0.044 \cdot S_{DS} \cdot I_e, 0.01, \frac{0.5 \cdot S_{MI}}{R} \right) = 0.104 \]

\[ C_s := \max \left( C_s, C_{s,min} \right) = 0.231 \]

\[ V := C_s \cdot W = 2.763 \text{ kip} \]

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Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Vertical Force Distribution:

\[ F_x = C_{vx} \cdot V \]

\[ C_{vx} = \frac{w_x \cdot h_{x_k}}{\sum_{i=1}^{n} w_i \cdot h_{i_k}} \]

\[ k(T) := \begin{cases} 
1 & \text{if } T_n \leq 0.5 \text{ s} \\
1 + \frac{(T_n - 0.5 \text{ s})}{(2.5 \text{ s} - 0.5 \text{ s})} \cdot (2 - 1) & \text{else if } 0.5 \text{ s} < T_n < 2.5 \text{ s} \\
2 & \text{else if } T_n \geq 2.5 \text{ s} 
\end{cases} \]

\[ k := k(T_n) = 1 \]

\[ C_{v2} := \frac{W_2 \cdot (h_2 + h_1)^k}{\left(W_1 \cdot h_1^k\right) + \left(W_2 \cdot (h_2 + h_1)^k\right)} = 0.9 \]

\[ C_{v1} := \frac{W_1 \cdot h_1^k}{\left(W_1 \cdot (h_1)^k\right) + \left(W_2 \cdot (h_2 + h_1)^k\right)} = 0.1 \]

\[ F_2 := C_{v2} \cdot V = 2.5 \text{ kip} \]

\[ F_1 := C_{v1} \cdot V = 0.263 \text{ kip} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Load Case: Direction 1

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \quad (EQ \ 4.3.2-4) \]

Overturning Moments:

\[ \begin{align*}
L := & 20 \text{ ft} & B := 19.5 \text{ ft} \\
DL_r := & 20 \text{ psf} & LL_r := 20 \text{ psf} & Trib := \frac{B}{2}
\end{align*} \]

Overturning moment from F2:

\[ M_{F,\text{top}} := F_2 \cdot h_2 = 21.251 \ \text{kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:

\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 85.8 \ \text{kip} \cdot \text{ft} \]

**Required force in hold down:**

\[ T := \frac{M_{F,\text{top}} - M_r}{L - 1 \ \text{ft}} = -3.397 \ \text{kip} \]

**Required Compression Force:**

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 5.183 \ \text{kip} \]

**Required Studs for compression:**

Light-Frame Capacity:

\[ P' := 5 \ \text{kip} \]

\[ \frac{C}{P'} = 1.037 \quad \text{Need at least 1 studs in corner for compression} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_n := 3.85 \ \text{klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 0.718 \ \text{ft} \quad \text{of Hinge along wall} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Load Case: Direction 1

\[ 0.9 \times DL + W \]  

\( (EQ \ 4.3.2-5) \)

**Overturning Moments:**  
\[ L = 6.096 \ m \quad B = 5.944 \ m \]

\( DL_r := 20 \ \text{psf} \quad LL_r := 20 \ \text{psf} \quad Trib := \frac{B}{2} \)

**Overturning moment from Ftop:**  
\[ M_{F, top} := F_2 \cdot h_2 = 21.251 \ \text{kip ft} \]

**Required force in hold down:**  
\[ T := \frac{M_{F, top}}{L - 1 \ ft} = 1.118 \ \text{kip} \quad M_{F, top} = 21.251 \ \text{kip ft} \]

**Required Compression Force:**  
\[ C := T = 1.118 \ \text{kip} \]

**Required Studs for compression:**

Light-Frame Capacity:
\[ P' := 5 \ \text{kip} \]
\[ \frac{C}{P'} = 0.224 \quad \text{Need at least 1 studs in corner for compression} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear  
\[ \phi := 0.65 \ \text{Connections} \]

\[ V'_{n} := 3.85 \ \text{klf} \]

For Base shear we would need:
\[ \frac{V}{V'_{n}} = 0.718 \ \text{ft} \quad \text{of Hinge along wall} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Load Case: Direction 2

\[ 1.2 \times DL + W + LL + 0.5 \times (L_T \text{ or } S \text{ or } R) \quad (EQ\ 4.3.2-4) \]

Overturning Moments:

<table>
<thead>
<tr>
<th>Overturning moment from F2:</th>
<th>( M_{F,\text{top}} := F_2 \cdot h_2 = 21.251 \text{ kip} \cdot \text{ft} )</th>
</tr>
</thead>
</table>

Overturning Moments:

<table>
<thead>
<tr>
<th>L := 19.5 ( \text{ft} )</th>
<th>B := 20 ( \text{ft} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( DL_r := 20 \text{ psf} )</td>
<td>( LL_r := 20 \text{ psf} )</td>
</tr>
<tr>
<td>Trib := ( \frac{B}{2} )</td>
<td></td>
</tr>
</tbody>
</table>

Resisting moment from Dead and Live:

\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 83.655 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**

\[ T := \frac{M_{F,\text{top}} - M_r}{L - 1} \text{ ft} = -3.373 \text{ kip} \]

**Required Compression Force:**

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 5.207 \text{ kip} \]

**Required Studs for compression:**

Light-Frame Capacity:

\[ P' := 5 \text{ kip} \]

\[ \frac{C}{P'} = 1.041 \quad \text{Need at least 1 studs in corner for compression} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 0.718 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Load Case: Direction 2

\[ 0.9 \times DL + W \]  \hspace{1cm} (EQ 4.3.2-5)

Overturning Moments:

\[ L = 5.944 \, m \quad B = 6.096 \, m \]

Overturning moment from Ftop:

\[ M_{F,\text{top}} := F_2 \cdot h_2 = 21.251 \, kip \cdot ft \]

Required force in hold down:

\[ T := \frac{M_{F,\text{top}}}{L - 1} \, ft = 1.149 \, kip \]

Required Compression Force:

\[ C := T = 1.149 \, kip \]

Required Studs for compression:

Light-Frame Capacity:

\[ P' := 5 \, kip \]

\[ \frac{C}{P'} = 0.23 \]

Need at least 1 studs in corner for compression

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_{n} := 3.85 \, klf \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 0.718 \, ft \]

of Hinge along wall
Appendix H. Mathcad Calculations for Seismic Loads: Light-Frame Dynamic Unit

Sheathing:
Diaphragm: \( \phi_D = 0.8 \)

\( F_2 = 2.5 \text{ kip} \)
\( L = 5.944 \text{ m} \)

\[ V_{dia} = \frac{F_2}{L \cdot \phi_D} = 160.261 \text{ plf} \]
\( V_n = 360 \text{ plf} \) Okay

Shea wall:

\[ V_{sw} = \frac{F_2}{L \cdot \phi_D} = 160.261 \text{ plf} \]
\( V_n = 510 \text{ plf} \) Okay

Use 7/16" Thickness Structural 1 Sheathing with 6" OC nail spacing along boundary and edge of panels and 6" OC in the Field

13.2 \( \text{psi} = 1900.8 \text{ psf} \)
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

**Medium Seismic Conditions:**

Weight:

\[ CLT := 13.5 \text{ psf} \]

Weight of SL-V3 3-Ply

\[ A := (2 \cdot 8 \cdot 20 + 8.5 \cdot 9 + 4.5 \cdot 9 + 7.5 \cdot 9 + 20 \cdot 9) \cdot ft^2 = 684.5 \text{ ft}^2 \]

\[ W := CLT \cdot A = 9.241 \text{ kip} \quad W_2 := \frac{W}{2} = 4.62 \text{ kip} \quad W_1 := \frac{W}{2} = 4.62 \text{ kip} \]

Ground Motion:

\[ h_2 := 9.5 \text{ ft} \quad h_1 := 1 \text{ ft} \]

\[ S_{MS} := 1 \text{ g} \quad S_{M1} := 0.6 \text{ g} \]

\[ S_{DS} := \frac{2}{3} \cdot S_{MS} = 0.667 \]

\[ S_{D1} := \frac{2}{3} \cdot S_{M1} = 0.4 \]

Design Response Spectrum:

\[ T_n := C_t \cdot h_n^x \]  
Approximate Period

\[ C_t := 0.02 \]

\[ h_n := \frac{h_1 + h_2}{ft} \]

Unit Height + Potential CMU Height

\[ x := 0.75 \]  
Table 12.8-2 ASCE 7-16

\[ T_n := \left(C_t \cdot h_n^x\right) s = 0.117 \text{ s} \]

\[ C_u := 1.4 \]

\[ C_a T_a := C_u \cdot T_n = 0.163 \text{ s} \]  
Upper limit for period

\[ T_s := \left(S_{D1} \right) s = 0.6 \text{ s} \]

\[ S_{DS} \]

\[ T_0 := 0.2 \cdot T_s = 0.12 \text{ s} \]

\[ T_L := 8 \text{ s} \]

\[ T := 0 \text{ s}, 0.01 \text{ s}..10 \text{ s} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

\[ S_a(T) := \begin{cases} 
S_{DS} & \text{if } T \leq T_0 \\
S_{DS} \cdot \left(0.4 + \left(0.6 \cdot \frac{T}{T_0}\right)\right) & \text{else if } T_0 < T \leq T_s \\
S_{DS} & \text{else if } T_s < T \leq T_L \\
S_{D1} \cdot \frac{s}{T} & \text{else if } T > T_L \\
S_{D1} \cdot \frac{T_L \cdot s}{T^2} & \text{else if } T > T_L 
\end{cases} \]

Conservative to use \( S_{DS} \)

\[ C_s := \min \left( \frac{S_{DS}}{R}, \frac{S_{D1} \cdot s}{I_e \cdot T_n} \right) = 0.222 \]

\[ C_{s, min} := \max \left( 0.044 \cdot S_{DS} \cdot I_e, 0.01, \frac{0.5 \cdot S_{M1}}{R / I_e} \right) = 0.1 \]

\[ C_s := \max \left( C_s, C_{s, min} \right) = 0.222 \]

\[ V := C_s \cdot W = 2.054 \text{ kip} \]
### Vertical Force Distribution:

\[ F_x = C_{vx} \cdot V \]

\[ C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k} \]

\[ k(T) := \begin{cases} 
1 & \text{if } T_n \leq 0.5 \text{ s} \\
2 & \text{else if } T_n \geq 2.5 \text{ s} \\
1 + \frac{(T_n - 0.5 \text{ s})}{(2.5 \text{ s} - 0.5 \text{ s})} \cdot (2 - 1) & \text{else if } 0.5 \text{ s} < T_n < 2.5 \text{ s} 
\end{cases} \]

\[ k := k(T_n) = 1 \]

\[ C_{v2} := \frac{W_2 \cdot (h_1 + h_2)^k}{\left( W_1 \cdot h_1^k \right) + \left( W_2 \cdot (h_1 + h_2)^k \right)} = 0.91 \]

\[ C_{v1} := \frac{W_1 \cdot h_1^k}{\left( W_1 \cdot h_1^k \right) + \left( W_2 \cdot (h_1 + h_2)^k \right)} = 0.09 \]

\[ F_2 := C_{v2} \cdot V = 1.875 \text{ kip} \]

\[ F_1 := C_{v1} \cdot V = 0.179 \text{ kip} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

Load Case: Direction 1

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \]  \hspace{1cm} (EQ 4.3.2-4)

**Overturning Moments:**

\[ L := 20 \text{ ft} \quad B := 8 \text{ ft} \]

\[ DL_r := 23.5 \text{ psf} \]

\[ LL_r := 20 \text{ psf} \]

\[ \text{Trib} := \frac{B}{2} \]

Overturning moment from F2:

\[ M_{F,\text{top}} := F_2 \cdot h_2 = 17.812 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:

\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot L \cdot \frac{L}{2} = 38.56 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**

\[ T := \frac{M_{F,\text{top}} - M_r}{L - 1 \text{ ft}} = -1.092 \text{ kip} \]

**Required Compression Force:**

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot L = 2.764 \text{ kip} \]

**Required Studs for compression:**

CLT Capacity:

\[ F_{c}' := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]

\[ V_n' := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V_n'} = 0.533 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

Load Case: Direction 1

\[ 0.9 \times DL + W \]  
\[ (EQ \ 4.3.2-5) \]

Overturning Moments:

\[ L = 6.096 \ m \quad B = 2.438 \ m \]
\[ DL_r := 23.5 \ psf \]
\[ LL_r := 20 \ psf \]
\[ Trib := \frac{B}{2} \]

Overturning moment from Ftop:

\[ M_{F_{top}} := F_2 \cdot h_2 = 17.812 \ \text{kip} \cdot \text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F_{top}}}{L - 1 \ ft} = 0.937 \ \text{kip} \]
\[ M_{F_{top}} = 17.812 \ \text{kip} \cdot \text{ft} \]

Required Compression Force:

\[ C := T = 0.937 \ \text{kip} \]

Required Studs for compression:

CLT Capacity:

\[ F'_c := 35849 \ \text{plf} = 35.849 \ \text{klf} \]

Okay

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_n := 3.85 \ \text{klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 0.533 \ ft \quad \text{of Hinge along wall} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

**Load Case: Direction 2**

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \quad (EQ \ 4.3.2 - 4) \]

**Overturning Moments:**

\[ L := 8 \text{ ft} \quad B := 20 \text{ ft} \]

Overturning moment from F2:

\[ M_{F, \text{top}} := F_2 \cdot h_2 = 17.812 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:

\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot \frac{L}{2} = 15.424 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**

\[ T := \frac{M_{F, \text{top}} - M_r}{L - 1 \text{ ft}} = 0.341 \text{ kip} \]

**Required Compression Force:**

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 4.197 \text{ kip} \]

**Required Studs for compression:**

CLT Capacity:

\[ F' = 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V' = 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 0.533 \text{ ft} \quad \text{of Hinge along wall} \]
## Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

**Load Case: Direction 2**

\[ 0.9 \times DL + W \]  \hspace{2cm} \text{(EQ 4.3.2-5)}

<table>
<thead>
<tr>
<th>Overturning Moments:</th>
<th>( L = 2.438 \text{ m} )</th>
<th>( B = 6.096 \text{ m} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overturning moment from Ftop:</td>
<td>( DL_r := 23.5 \text{ psf} )</td>
<td>( LL_r := 20 \text{ psf} )</td>
</tr>
</tbody>
</table>

\[ M_{F_{\text{top}}} := F_2 \cdot h_2 = 17.812 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**

\[ T := \frac{M_{F_{\text{top}}}}{L - 1 \text{ ft}} = 2.545 \text{ kip} \]

\[ M_{F_{\text{top}}} = 17.812 \text{ kip} \cdot \text{ft} \]

**Required Compression Force:**

\[ C := T = 2.545 \text{ kip} \]

**Required Studs for compression:**

CLT Capacity:

\[ F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \]

Okay

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \text{ Connections} \]

\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 0.533 \text{ ft} \text{ of Hinge along wall} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

**High Seismic Condition:**

**Weight:**

\[ CLT := 13.5 \text{ psf} \]

Weight of SL-V3 3-Ply

\[ A := (2 \cdot 8.2 \cdot 20 + 8.5 \cdot 9 + 4.5 \cdot 9 + 7.5 \cdot 9 + 20 \cdot 9) \cdot ft^2 = 684.5 \text{ ft}^2 \]

\[ W := CLT \cdot A = 9.241 \text{ kip} \]

\[ W_2 := \frac{W}{2} = 4.62 \text{ kip} \]

\[ W_1 := \frac{W}{2} = 4.62 \text{ kip} \]

**Ground Motion:**

\[ h_2 := 9.5 \text{ ft} \]

\[ h_1 := 1 \text{ ft} \]

\[ S_{MS} := 1.5 \text{ g} \]

\[ S_{M1} := 0.9 \text{ g} \]

\[ S_{DS} := \frac{2}{3} \cdot S_{MS} = 1 \]

\[ S_{D1} := \frac{2}{3} \cdot S_{M1} = 0.6 \]

**Design Response Spectrum:**

\[ T_n = C_t \cdot h_n^x \]

Approximate Period

\[ C_t := 0.02 \]

\[ h_n := \frac{h_1 + h_2}{ft} \]

Unit Height + Potential CMU Height

\[ x := 0.75 \]

Table 12.8-2 ASCE 7-16

\[ T_n := \left( C_t \cdot h_n^x \right) s = 0.117 \text{ s} \]

\[ C_a := 1.4 \]

\[ C_a T_a := C_a \cdot T_n = 0.163 \text{ s} \]

Upper limit for period

\[ T_s := \left( \frac{S_{D1}}{S_{DS}} \right) s = 0.6 \text{ s} \]

\[ T_0 := 0.2 \cdot T_s = 0.12 \text{ s} \]

\[ T_L := 8 \text{ s} \]

\[ T := 0 \text{ s}, 0.01 \text{ s} \ldots 10 \text{ s} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

\[
S_a(T) := \begin{cases} 
\text{if } T \leq T_0 & S_{DS} \cdot \left(0.4 + \left(0.6 \cdot \frac{T}{T_0}\right)\right) \\
\text{else if } T_0 < T \leq T_s & S_{DS} \\
\text{else if } T_s < T \leq T_L & \frac{S_{D_1} \cdot s}{T} \\
\text{else if } T > T_L & \frac{S_{D_1} \cdot T_L \cdot s}{T^2}
\end{cases}
\]

Conservative to use \(S_{DS}\)

\[
R = 3 \quad I_e = 1
\]

\[
C_s := \min \left\{ \frac{S_{DS}}{R}, \frac{S_{D_1} \cdot s}{I_e \cdot R} \right\} = 0.333
\]

\[
C_{s,\min} := \max \left\{ 0.044 \cdot S_{DS} \cdot I_e, 0.01, \frac{0.5 \cdot S_{M_1}}{R \cdot I_e} \right\} = 0.15
\]

\[
C_s := \max \left( C_s, C_{s,\min} \right) = 0.333
\]

\[
V := C_s \cdot W = 3.08 \text{ kip}
\]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

Vertical Force Distribution:

\[
F_x = C_{vx} \cdot V
\]

\[
C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k}
\]

\[
k(T) := \begin{cases} 
1 & \text{if } T_n \leq 0.5 \ s \\
2 & \text{else if } T_n \geq 2.5 \ s \\
1 + \frac{(T_n - 0.5 \ s)}{(2.5 \ s - 0.5 \ s)} \cdot (2 - 1) & \text{else if } 0.5 \ s < T_n < 2.5 \ s
\end{cases}
\]

\[
k := k(T_n) = 1
\]

\[
C_{v2} := \frac{W_2 \cdot (h_1 + h_2)^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_1 + h_2)^k)} = 0.91
\]

\[
C_{v1} := \frac{W_1 \cdot h_1^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_1 + h_2)^k)} = 0.09
\]

\[
F_2 := C_{v2} \cdot V = 2.812 \ \text{kip}
\]

\[
F_1 := C_{v1} \cdot V = 0.268 \ \text{kip}
\]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

Load Case: Direction 1

\[ 1.2 \times DL + W + LL + 0.5 \times (L_T \text{ or } S \text{ or } R) \]  \hspace{1cm} (EQ 4.3.2-4)

Overturning Moments:

\[
L := 20 \text{ ft} \hspace{1cm} B := 8 \text{ ft} \\
DL_r := 23.5 \text{ psf} \hspace{1cm} LL_r := 20 \text{ psf} \hspace{1cm} Trib := \frac{B}{2}
\]

Overturning moment from F2:
\[ M_{F, top} := F_2 \cdot h_2 = 26.718 \text{ kip ft} \]

Resisting moment from Dead and Live:
\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 38.56 \text{ kip ft} \]

**Required force in hold down:**

\[ T := \frac{M_{F, top} - M_r}{L - 1 \text{ ft}} = -0.623 \text{ kip} \]

**Required Compression Force:**

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 3.233 \text{ kip} \]

**Required Studs for compression:**

CLT Capacity:
\[ F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \hspace{1cm} \text{Okay} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \hspace{1cm} \text{Connections} \]
\[ V_n' := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V_n'} = 0.8 \text{ ft} \hspace{1cm} \text{of Hinge along wall} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

<table>
<thead>
<tr>
<th>Load Case: Direction 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9 × DL + W (EQ 4.3.2-5)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Overturning Moments:</th>
<th>( L = 6.096 \text{ m} )</th>
<th>( B = 2.438 \text{ m} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( DL_r := 23.5 \text{ psf} )</td>
<td>( LL_r := 20 \text{ psf} )</td>
<td></td>
</tr>
<tr>
<td>( Trib := \frac{B}{2} )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Overturning moment from Ftop:
\[ M_{F, top} := F_2 \cdot h_2 = 26.718 \text{ kip} \cdot \text{ft} \]

Required force in hold down:
\[ T := \frac{M_{F, top}}{L - 1 \text{ ft}} = 1.406 \text{ kip} \]
\[ M_{F, top} = 26.718 \text{ kip} \cdot \text{ft} \]

Required Compression Force:
\[ C := T = 1.406 \text{ kip} \]

Required Studs for compression:
CLT Capacity:
\[ F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \]
Okay

Required hinge length for base shear:
Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V'_n} = 0.8 \text{ ft} \]
\[ \text{of Hinge along wall} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

Load Case: Direction 2

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \]  
\[ \text{(EQ 4.3.2-4)} \]

Overturning Moments: 
\[ L := 8 \text{ ft} \quad B := 20 \text{ ft} \]
\[ DL_r := 23.5 \text{ psf} \]
\[ LL_r := 20 \text{ psf} \]
\[ Trib := \frac{B}{2} \]

Overturning moment from F2:
\[ MF_{top} := F_2 \cdot h_2 = 26.718 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:
\[ Mr := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 15.424 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**
\[ T := \frac{MF_{top} - Mr}{L - 1 \text{ ft}} = 1.613 \text{ kip} \]

**Required Compression Force:**
\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 5.469 \text{ kip} \]

**Required Studs for compression:**

CLT Capacity:
\[ F_c' := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V_n' := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V_n'} = 0.8 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

Load Case: Direction 2

\[ 0.9 \times DL + W \quad (EQ\ 4.3.2-5) \]

Overturning Moments:

\[ L = 2.438 \text{ m} \quad B = 6.096 \text{ m} \]
\[ DL_r := 23.5 \text{ psf} \quad LL_r := 20 \text{ psf} \quad Trib := \frac{B}{2} \]

Overturning moment from Ftop:
\[ M_{F,\text{top}} := F_2 \cdot h_2 = 26.718 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**
\[ T := \frac{M_{F,\text{top}}}{L - 1 \text{ ft}} = 3.817 \text{ kip} \]
\[ M_{F,\text{top}} = 26.718 \text{ kip} \cdot \text{ft} \]

**Required Compression Force:**
\[ C := T = 3.817 \text{ kip} \]

**Required Studs for compression:**
CLT Capacity:
\[ F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V'_n} = 0.8 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

**Very High Seismic Condition:**

Weight:

\[ CLT := 13.5 \text{ psf} \]

Weight of SL-V3 3-Ply

\[ A := (2 \cdot 8 \cdot 20 + 8.5 \cdot 9 + 4.5 \cdot 9 + 7.5 \cdot 9 + 20 \cdot 9) \cdot ft^2 = 684.5 \text{ ft}^2 \]

\[ W := CLT \cdot A = 9.241 \text{ kip} \]

\[ W_2 := \frac{W}{2} = 4.62 \text{ kip} \]

\[ W_1 := \frac{W}{2} = 4.62 \text{ kip} \]

Ground Motion:

\[ h_2 := 9.5 \text{ ft} \]

\[ h_1 := 1 \text{ ft} \]

\[ S_{MS} := 2.25 \text{ g} \]

\[ S_{M1} := 1.35 \text{ g} \]

\[ S_{DS} := \frac{2}{3} \cdot S_{MS} = 1.5 \]

\[ S_{D1} := \frac{2}{3} \cdot S_{M1} = 0.9 \]

Design Response Spectrum:

\[ T_n := C_t \cdot h_n^x \]

Approximate Period

\[ C_t := 0.02 \]

\[ h_n := \frac{h_1 + h_2}{ft} \]

Unit Height + Potential CMU Height

\[ x := 0.75 \]

Table 12.8-2 ASCE 7-16

\[ T_n := \left( C_t \cdot h_n^x \right) s = 0.117 \text{ s} \]

\[ C_u := 1.4 \]

\[ C_aT_a := C_u \cdot T_n = 0.163 \text{ s} \]

Upper limit for period

\[ T_s := \left( \frac{S_{D1}}{S_{DS}} \right) s = 0.6 \text{ s} \]

\[ T_0 := 0.2 \cdot T_s = 0.12 \text{ s} \]

\[ T_L := 8 \text{ s} \]

\[ T := 0 \text{ s}, 0.01 \text{ s} .. 10 \text{ s} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

\[
S_a(T) := \begin{cases} 
\text{if } T \leq T_0 & S_{DS} \\
\text{else if } T_0 < T \leq T_s & S_{DS} \cdot \left(0.4 + \left(0.6 \cdot \frac{T}{T_0}\right)\right) \\
\text{else if } T_s < T \leq T_L & S_{DS} \\
\text{else if } T > T_L & S_{DS} \cdot \frac{T_{1} \cdot s}{T^2} 
\end{cases}
\]

Conservative to use \( S_{DS} \)

\[ R := 3 \quad I_e := 1 \]

\[ C_s := \min \left( S_{DS}, \frac{S_{DS}}{R}, \frac{S_{DS} \cdot I_e}{R} \right) = 0.5 \]

\[ C_{s, \text{min}} := \max \left( 0.044 \cdot S_{DS} \cdot I_e, 0.01, 0.5 \cdot S_{M1} \right) = 0.225 \]

\[ C_s := \max (C_s, C_{s, \text{min}}) = 0.5 \]

\[ C_s := S_{DS} \frac{R}{I_e} = 0.5 \]

\[ V := C_s \cdot W = 4.62 \text{ kip} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

Vertical Force Distribution:

\[ F_x = C_{vx} \cdot V \]

\[ C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k} \]

\[ k(T) := \begin{cases} 
1 & \text{if } T_n \leq 0.5 \text{ s} \\
2 & \text{if } T_n \geq 2.5 \text{ s} \\
1 + \frac{(T_n - 0.5 \text{ s})}{(2.5 \text{ s} - 0.5 \text{ s})} \cdot (2 - 1) & \text{if } 0.5 \text{ s} < T_n < 2.5 \text{ s} 
\end{cases} \]

\[ k := k(T_n) = 1 \]

\[ C_{v2} := \frac{W_2 \cdot (h_1 + h_2)^k}{\left(W_1 \cdot h_1^k\right) + \left(W_2 \cdot (h_1 + h_2)^k\right)} = 0.91 \]

\[ C_{v1} := \frac{W_1 \cdot h_1^k}{\left(W_1 \cdot h_1^k\right) + \left(W_2 \cdot (h_1 + h_2)^k\right)} = 0.09 \]

\[ F_2 := C_{v2} \cdot V = 4.219 \text{ kip} \]

\[ F_1 := C_{v1} \cdot V = 0.402 \text{ kip} \]
Load Case: Direction 1

\[ 1.2 \times DL + W + LL + 0.5 \times (L_{r} \text{ or } S \text{ or } R) \quad (EQ \ 4.3.2-4) \]

- **Overturning Moments:**
  - \( DL_r := 23.5 \text{ psf} \)
  - \( LL_r := 20 \text{ psf} \)
  - \( Trib := \frac{B}{2} \)
  - \( L := 20 \text{ ft} \)
  - \( B := 8 \text{ ft} \)
  - \( M_{F\_top} := F_2 \cdot h_2 = 40.077 \text{ kip} \cdot \text{ft} \)

- **Resisting moment from Dead and Live:**
  - \( M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 38.56 \text{ kip} \cdot \text{ft} \)

- **Required force in hold down:**
  - \( T := \frac{M_{F\_top} - M_r}{L - 1 \text{ ft}} = 0.08 \text{ kip} \)

- **Required Compression Force:**
  - \( C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 3.936 \text{ kip} \)

- **Required Studs for compression:**
  - CLT Capacity:
  - \( F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \)

- **Required hinge length for base shear:**
  - Ultimate Strength of Hinge A in Shear
  - \( \phi := 0.65 \) Connections
  - \( V'_n := 3.85 \text{ klf} \)
  - For Base shear we would need:
    - \( \frac{V}{V'_n} = 1.2 \text{ ft} \) of Hinge along wall
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

Load Case: Direction 1

\[ 0.9 \times DL + W \] (EQ 4.3.2-5)

Overturning Moments:
\[ L = 6.096 \text{ m} \quad B = 2.438 \text{ m} \]
\[ DL_r := 23.5 \text{ psf} \]
\[ LL_r := 20 \text{ psf} \]
\[ Trib := \frac{B}{2} \]

Overturning moment from Ftop:
\[ M_{F, top} := F_2 \cdot h_2 = 40.077 \text{ kip} \cdot \text{ft} \]

Required force in hold down:
\[ T := \frac{M_{F, top}}{L - 1 \text{ ft}} = 2.109 \text{ kip} \]
\[ M_{F, top} = 40.077 \text{ kip} \cdot \text{ft} \]

Required Compression Force:
\[ C := T = 2.109 \text{ kip} \]

Required Studs for compression:
CLT Capacity:
\[ F'_{c} := 35849 \text{ plf} = 35.849 \text{ klf} \]
Okay

Required hinge length for base shear:
Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V'_{n}} = 1.2 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

Load Case: Direction 2

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \quad \text{(EQ 4.3.2-4)} \]

Overturning Moments:

Overturning moment from F2:
\[ M_{F,\text{top}} := F_2 \cdot h_2 = 40.077 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:
\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 15.424 \text{ kip} \cdot \text{ft} \]

\textbf{Required force in hold down:}
\[ T := \frac{M_{F,\text{top}} - M_r}{L - 1 \text{ ft}} = 3.522 \text{ kip} \]

\textbf{Required Compression Force:}
\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 7.378 \text{ kip} \]

\textbf{Required Studs for compression:}
CLT Capacity:
\[ F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

\textbf{Required hinge length for base shear:}

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V'_{n}} = 1.2 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix I. Mathcad Calculations for Seismic Loads: CLT Static Unit

Load Case: Direction 2

\[ 0.9 \times DL + W \] 

\( (EQ 4.3.2-5) \)

Overturning Moments: 
\[ L = 2.438 \text{ m} \quad B = 6.096 \text{ m} \]
\[ DL_r := 23.5 \text{ psf} \]
\[ LL_r := 20 \text{ psf} \]
\[ Trib := \frac{B}{2} \]

Overturning moment from Ftop:
\[ M_{F\text{top}} := F_2 \cdot h_2 = 40.077 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**
\[ T := \frac{M_{F\text{top}}}{L - 1 \text{ ft}} = 5.725 \text{ kip} \]

\[ M_{F\text{top}} = 40.077 \text{ kip} \cdot \text{ft} \]

**Required Compression Force:**
\[ C := T = 5.725 \text{ kip} \]

**Required Studs for compression:**

CLT Capacity:
\[ F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \]
Okay

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V'_n} = 1.2 \text{ ft} \] of Hinge along wall
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

**Medium Seismic Condition:**

Weight:
\[ W := 21.56 \text{ kip} \]

Effective seismic weights:
\[ W_2 := \frac{W}{2} = 10.78 \text{ kip} \]
Half of weight goes to roof
\[ W_1 := \frac{W}{2} = 10.78 \text{ kip} \]
Half of weight goes to floor
\[ h_2 := 8.5 \text{ ft} \quad h_1 := 1 \text{ ft} \]

Ground Motion:
\[ S_{MS} := 1 \quad g \]
\[ S_{M1} := 0.6 \quad g \]
\[ S_{DS} := \frac{2}{3} \cdot S_{MS} = 0.667 \]
\[ S_{D1} := \frac{2}{3} \cdot S_{M1} = 0.4 \]

Design Response Spectrum:
\[ T_n = C_t \cdot h_n^x \quad \text{Approximate Period} \]
\[ C_t := 0.02 \]
\[ h_n := \frac{h_1 + h_2}{ft} \quad \text{Unit Height + Potential CMU Height} \]
\[ x := 0.75 \quad \text{Table 12.8-2 ASCE 7-16} \]
\[ T_n := (C_t \cdot h_n^x) \quad s = 0.108 \text{ s} \]
\[ C_a := 1.4 \]
\[ C_aT_n := C_a \cdot T_n = 0.152 \text{ s} \]
\[ T_s := \left( \frac{S_{D1}}{S_{DS}} \right) \cdot s = 0.6 \text{ s} \]
\[ T_0 := 0.2 \cdot T_s = 0.12 \text{ s} \quad T_L := 8 \text{ s} \]
\[ T := 0 \text{ s}, 0.01 \text{ s} \ldots 10 \text{ s} \]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

\[ S_a(T) := \begin{cases} 
\text{if } T \leq T_0 & |S_a(\frac{T}{T_0})| \\
\text{if } T_0 < T \leq T_s & |S_{DS} \cdot \left(0.4 + 0.6 \cdot \frac{T}{T_0}\right)| \\
\text{else if } T_s < T \leq T_L & |S_{DS}| \\
\text{else if } T > T_L & \frac{|S_{DS} \cdot T_L \cdot S_{DS}}{T^2}| \\
\end{cases} \]

Conservative to use \( S_{DS} \)

\[ C_s := \min \left( \frac{S_{DS}}{R}, \frac{S_{DS} \cdot R}{T_n \cdot I_e} \right) = 0.222 \]

\[ C_{s,\min} := \max \left( 0.044 \cdot S_{DS} \cdot I_e, 0.01, \frac{0.5 \cdot S_{DS}}{I_e} \right) = 0.1 \]

\[ C_s := \max (C_s, C_{s,\min}) = 0.222 \]

\[ C_s := \frac{S_{DS}}{R} = 0.222 \]

\[ V := C_s \cdot W = 4.791 \text{ kip} \]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

Vertical Force Distribution:
\[ F_x = C_{vx} \cdot V \]

\[ C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k} \]

\[ k(T) := \begin{cases} 1 & \text{if } T_n \leq 0.5 \text{ s} \\ 2 & \text{else if } T_n \geq 2.5 \text{ s} \\ 1 + \frac{(T_n - 0.5 \text{ s}) \cdot (2 - 1)}{(2.5 \text{ s} - 0.5 \text{ s})} & \text{else if } 0.5 \text{ s} < T_n < 2.5 \text{ s} \end{cases} \]

\[ k := k(T_n) = 1 \]

\[ C_{v2} := \frac{W_2 \cdot (h_2 + h_1)^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_2 + h_1)^k)} = 0.9 \]

\[ C_{v1} := \frac{W_1 \cdot h_1^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_2 + h_1)^k)} = 0.1 \]

\[ F_2 := C_{v2} \cdot V = 4.335 \text{ kip} \]

\[ F_1 := C_{v1} \cdot V = 0.456 \text{ kip} \]

h₁ = 0.305 m

h₂ = 2.591 m
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

Load Case: Direction 1

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \quad (EQ \ 4.3.2-4) \]

Overturning Moments:

\[ L := 20 \text{ ft} \quad B := 19.5 \text{ ft} \]
\[ DL_r := 23.5 \text{ psf} \quad LL_r := 20 \text{ psf} \quad Trib := \frac{B}{2} \]

Overturning moment from F2:

\[ M_{F,\text{top}} := F_2 \cdot h_2 = 36.846 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:

\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 93.99 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**

\[ T := \frac{M_{F,\text{top}} - M_r}{L - 1 \text{ ft}} = -3.008 \text{ kip} \]

**Required Compression Force:**

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 6.391 \text{ kip} \]

**Required Studs for compression:**

CLT Capacity:

\[ F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 1.244 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

Load Case: Direction 1

\[ 0.9 \times DL + W \quad (EQ \ 4.3.2-5) \]

<table>
<thead>
<tr>
<th>Overturning Moments:</th>
<th>( L = 6.096 \text{ m} )</th>
<th>( B = 5.944 \text{ m} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overturning moment from Ftop:</td>
<td>( DL_r := 23.5 \text{ psf} )</td>
<td>( LL_r := 20 \text{ psf} )</td>
</tr>
<tr>
<td>( M_{F,\text{top}} := F_2 \cdot h_2 = 36.846 \text{ kip} \cdot \text{ft} )</td>
<td>( Trib := \frac{B}{2} )</td>
<td></td>
</tr>
</tbody>
</table>

**Required force in hold down:**

\[
T := \frac{M_{F,\text{top}}}{L - 1 \text{ ft}} = 1.939 \text{ kip} \quad M_{F,\text{top}} = 36.846 \text{ kip} \cdot \text{ft}
\]

**Required Compression Force:**

\[
C := T = 1.939 \text{ kip}
\]

**Required Studs for compression:**

CLT Capacity:

\[
F_c' := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay}
\]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[
\phi := 0.65 \quad \text{Connections}
\]

\[
V_n' := 3.85 \text{ klf}
\]

For Base shear we would need:

\[
\frac{V}{V_n'} = 1.244 \text{ ft} \quad \text{of Hinge along wall}
\]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

Load Case: Direction 2

\[ 1.2 \times DL + W + LL + 0.5 \times (L_f \text{ or } S \text{ or } R) \]  \hspace{1cm} (EQ 4.3.2-4)

**Overturning Moments:**

\[ L := 19.5 \text{ ft} \quad B := 20 \text{ ft} \]

\[ DL_r := 23.5 \text{ psf} \]

\[ LL_r := 20 \text{ psf} \]

\[ Trib := \frac{B}{2} \]

Overturning moment from F2:

\[ MF_{top} := F_2 \cdot h_2 = 36.846 \text{ kip} \cdot \text{ft} \]

Resisting moment from Dead and Live:

\[ MR := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 91.64 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**

\[ T := \frac{MF_{top} - MR}{L - 1 \text{ ft}} = -2.962 \text{ kip} \]

**Required Compression Force:**

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 6.437 \text{ kip} \]

**Required Studs for compression:**

CLT Capacity:

\[ F'_{c} := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_{n}} = 1.244 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

Load Case: Direction 2

\[ 0.9 \times DL + W \]  \hspace{1cm} (EQ 4.3.2-5)

Overturning Moments:
- \( L = 5.944 \, m \)
- \( B = 6.096 \, m \)
- \( DL_r = 23.5 \, psf \)
- \( LL_r = 20 \, psf \)
- \( Trib = \frac{B}{2} \)

Overturning moment from Ftop:
- \( M_{F,\text{top}} := F_2 \cdot h_2 = 36.846 \, kip \cdot ft \)

**Required force in hold down:**

\[ T := \frac{M_{F,\text{top}}}{L - 1 \, ft} = 1.992 \, kip \]

\[ M_{F,\text{top}} = 36.846 \, kip \cdot ft \]

**Required Compression Force:**

\[ C := T = 1.992 \, kip \]

**Required Studs for compression:**

CLT Capacity:
- \( F'_c = 35849 \, plf = 35.849 \, klf \)
- Okay

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear
- \( \phi = 0.65 \)
- Connections

\[ V'_n := 3.85 \, klf \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 1.244 \, ft \]

of Hinge along wall
High Seismic Condition:

Weight: 
\[ W := 21.56 \text{ kip} \]

Effective seismic weights:
\[ W_2 := \frac{W}{2} = 10.78 \text{ kip} \]
Half of weight goes to roof
\[ W_1 := \frac{W}{2} = 10.78 \text{ kip} \]
Half of weight goes to floor

\[ h_2 := 8.5 \text{ ft} \quad h_1 := 1 \text{ ft} \]

Ground Motion:
\[ S_{MS} := 1.5 \quad \text{g} \]
\[ S_{M1} := 0.9 \quad \text{g} \]
\[ S_{DS} := \frac{2}{3} S_{MS} = 1 \]
\[ S_{D1} := \frac{2}{3} S_{M1} = 0.6 \]

Design Response Spectrum:
\[ T_n = C_t \cdot h_n^x \quad \text{Approximate Period} \]
\[ C_t := 0.02 \]
\[ h_n := \frac{h_1 + h_2}{\text{ft}} \quad \text{Unit Height + Potential CMU Height} \]
\[ x := 0.75 \quad \text{Table 12.8-2 ASCE 7-16} \]
\[ T_n := (C_t \cdot h_n^x) \quad s = 0.108 \text{ s} \]
\[ C_u := 1.4 \]
\[ C_u T_a := C_u \cdot T_n = 0.152 \text{ s} \]

Upper limit for period
\[ T_a := \frac{S_{D1}}{S_{DS}} \cdot s = 0.6 \text{ s} \]
\[ T_0 := 0.2 \cdot T_a = 0.12 \text{ s} \]
\[ T_L := 8 \text{ s} \]
\[ T := 0 \text{ s}, 0.01 \text{ s}, \ldots, 10 \text{ s} \]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

\[
S_a(T) := \begin{cases} 
  S_{DS} \cdot \left(0.4 + \left(0.6 \cdot \frac{T}{T_0}\right)\right) & \text{if } T \leq T_0 \\
  S_{DS} & \text{else if } T_0 < T \leq T_s \\
  S_{DS} & \text{else if } T_s < T \leq T_L \\
  \frac{S_{D1} \cdot s}{T} & \text{else if } T > T_L \\
  \frac{S_{D1} \cdot T_L \cdot s}{T^2} & 
\end{cases}
\]

Conservative to use \( S_{DS} \)

\[
C_s := \min \left( \frac{S_{DS}}{R}, \frac{S_{D1} \cdot s}{I_e \cdot T_n \cdot \frac{R}{I_e}} \right) = 0.333
\]

\[
C_{s,min} := \max \left( 0.044 \cdot S_{DS} \cdot I_e, 0.01, \frac{0.5 \cdot S_{M1}}{R / I_e} \right) = 0.15
\]

\[
C_s := \max (C_s, C_{s,min}) = 0.333
\]

\[
V := C_s \cdot W = 7.187 \text{ kip}
\]
Vertical Force Distribution:

\[ F_x = C_{vx} \cdot V \]

\[ C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k} \]

\[ k(T) := \begin{cases} 
1 & \text{if } T_n \leq 0.5 \text{ s} \\
2 & \text{else if } T_n \geq 2.5 \text{ s} \\
1 + \frac{(T_n - 0.5 \text{ s})}{(2.5 \text{ s} - 0.5 \text{ s})} \cdot (2 - 1) & \text{else if } 0.5 \text{ s} < T_n < 2.5 \text{ s} 
\end{cases} \]

\[ k := k(T_n) = 1 \]

\[ h_1 = 0.305 \text{ m} \]
\[ h_2 = 2.591 \text{ m} \]

\[ C_{v2} := \frac{W_2 \cdot (h_2 + h_1)^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_2 + h_1)^k)} = 0.9 \]

\[ C_{v1} := \frac{W_1 \cdot h_1^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_2 + h_1)^k)} = 0.1 \]

\[ F_2 := C_{v2} \cdot V = 6.502 \text{ kip} \]

\[ F_1 := C_{v1} \cdot V = 0.684 \text{ kip} \]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

Load Case: Direction 1

1. \(2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R)\)  

Overturning Moments:  
\[
L := 20 \text{ ft} \quad B := 19.5 \text{ ft}
\]
\[
DL_r := 23.5 \text{ psf} \quad LL_r := 20 \text{ psf} \quad Trib := \frac{B}{2}
\]

Overturning moment from \(F_2\):
\[
M_{F,\text{top}} := F_2 \cdot h_2 = 55.269 \text{ kip} \cdot \text{ft}
\]

Resisting moment from Dead and Live:
\[
M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 93.99 \text{ kip} \cdot \text{ft}
\]

Required force in hold down:
\[
T := \frac{M_{F,\text{top}} - M_r}{L - 1 \text{ ft}} = -2.038 \text{ kip}
\]

Required Compression Force:
\[
C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 7.361 \text{ kip}
\]

Required Studs for compression:
CLT Capacity:
\[
F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay}
\]

Required hinge length for base shear:

Ultimate Strength of Hinge \(A\) in Shear
\[
\phi := 0.65 \quad \text{Connections}
\]
\[
V'_n := 3.85 \text{ klf}
\]

For Base shear we would need:
\[
\frac{V}{V'_n} = 1.867 \text{ ft} \quad \text{of Hinge along wall}
\]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

Load Case: Direction 1

\[ 0.9 \times DL + W \quad (EQ 4.3.2-5) \]

Overturning Moments:

\[ L = 6.096 \text{ m} \quad B = 5.944 \text{ m} \]

\[ DL_r := 23.5 \text{ psf} \]

\[ LL_r := 20 \text{ psf} \]

\[ Trib := \frac{B}{2} \]

Overturning moment from Ftop:

\[ M_{F, top} := F_2 \cdot h_2 = 55.269 \text{ kip} \cdot \text{ft} \]

Required force in hold down:

\[ T := \frac{M_{F, top}}{L - 1 \text{ ft}} = 2.909 \text{ kip} \]

Required Compression Force:

\[ C := T = 2.909 \text{ kip} \]

Required Studs for compression:

CLT Capacity:

\[ F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear:

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 1.867 \text{ ft} \quad \text{of Hinge along wall} \]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

Load Case: Direction 2

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \quad (EQ \ 4.3.2-4) \]

Overturning Moments: \[ L := 19.5 \text{ ft} \quad B := 20 \text{ ft} \]

Overturning moment from F2:
\[ M_{F_{top}} := F_2 \cdot h_2 = 55.269 \text{ kip}\cdot \text{ft} \]

Resisting moment from Dead and Live:
\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot L \cdot \frac{L}{2} = 91.64 \text{ kip}\cdot \text{ft} \]

**Required force in hold down:**
\[ T := \frac{M_{F_{top}} - M_r}{L - 1 \text{ ft}} = -1.966 \text{ kip} \]

**Required Compression Force:**
\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot \text{Trib} \cdot L = 7.433 \text{ kip} \]

**Required Studs for compression:**
CLT Capacity:
\[ F'_{c} := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad \text{Connections} \]
\[ V'_{n} := 3.85 \text{ klf} \]

For Base shear we would need:
\[ \frac{V}{V'_{n}} = 1.867 \text{ ft} \quad \text{of Hinge along wall} \]
## Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

### Load Case: Direction 2

\[ \mathbf{0.9 \times DL + W} \quad (EQ \ 4.3.2-5) \]

<table>
<thead>
<tr>
<th>Overturning Moments:</th>
<th>( L = 5.944 \ m )</th>
<th>( B = 6.096 \ m )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overturning moment from Ftop:</td>
<td>( DL_r := 23.5 \ psf )</td>
<td>( LL_r := 20 \ psf )</td>
</tr>
</tbody>
</table>

\[ M_{F,\text{top}} := F_2 \cdot h_2 = 55.269 \ \text{kip} \cdot \text{ft} \]

### Required force in hold down:

\[ T := \frac{M_{F,\text{top}}}{L - 1 \ \text{ft}} = 2.988 \ \text{kip} \quad M_{F,\text{top}} = 55.269 \ \text{kip} \cdot \text{ft} \]

### Required Compression Force:

\[ C := T = 2.988 \ \text{kip} \]

### Required Studs for compression:

CLT Capacity:

\[ F'_c := 35849 \ \text{plf} = 35.849 \ \text{klf} \quad \text{Okay} \]

### Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_n := 3.85 \ \text{klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 1.867 \ \text{ft} \quad \text{of Hinge along wall} \]
Very High Seismic Condition:

Weight:
\[ W = 21.56 \text{ kip} \]

Effective seismic weights:
\[ W_2 := \frac{W}{2} = 10.78 \text{ kip} \]
Half of weight goes to roof

\[ W_1 := \frac{W}{2} = 10.78 \text{ kip} \]
Half of weight goes to floor

\[ h_2 := 8.5 \text{ ft} \]
\[ h_1 := 1 \text{ ft} \]

Ground Motion:
\[ S_{MS} := 2.25 \text{ g} \]
\[ S_{M1} := 1.35 \text{ g} \]

\[ S_{DS} := \frac{2}{3} S_{MS} = 1.5 \]

\[ S_{D1} := \frac{2}{3} S_{M1} = 0.9 \]

Design Response Spectrum:
\[ T_n = C_t \cdot h_n^x \]
Approximate Period

\[ C_t := 0.02 \]

\[ h_n := \frac{h_1 + h_2}{ft} \]
Unit Height + Potential CMU Height

\[ x := 0.75 \]
Table 12.8-2 ASCE 7-16

\[ T_n := (C_t \cdot h_n^x) \text{ s} = 0.108 \text{ s} \]
\[ C_u := 1.4 \]
\[ C_u T_n := C_u \cdot T_n = 0.152 \text{ s} \]
Upper limit for period

\[ T_s := \left( \frac{S_{D1}}{S_{DS}} \right) \cdot s = 0.6 \text{ s} \]

\[ T_0 := 0.2 \cdot T_s = 0.12 \text{ s} \]
\[ T_L := 8 \text{ s} \]

\[ T := 0 \text{ s}, 0.01 \text{ s}..., 10 \text{ s} \]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

\[ S_a(T) := \begin{cases} 
  & \text{if } T \leq T_0 \\
  & \left| S_{DS} \cdot \left( 0.4 + \left( 0.6 \cdot \frac{T}{T_0} \right) \right) \right| \\
  & \text{else if } T_0 < T \leq T_s \\
  & \left| S_{DS} \right| \\
  & \text{else if } T_s < T \leq T_L \\
  & \left| S_{D1} \cdot S \right| \\
  & \text{else if } T > T_L \\
  & \left| \frac{S_{D1} \cdot T_L \cdot s}{T^2} \right| 
\end{cases} \]

Conservative to use \( S_{DS} \)

\[ R := 3 \quad I_e := 1 \]

\[ C_s := \min \left( \frac{S_{DS}}{R}, \frac{S_{D1} \cdot s}{I_e} \cdot \frac{R}{T \cdot I_e} \right) = 0.5 \]

\[ C_{s,\text{min}} := \max \left( 0.044 \cdot S_{DS} \cdot I_e \cdot 0.01, \frac{0.5 \cdot S_{M1}}{R \cdot I_e} \right) = 0.225 \]

\[ C_s := \max \left( C_s, C_{s,\text{min}} \right) = 0.5 \]

\[ V := C_s \cdot W = 10.78 \text{ kip} \]

\[ C_s := \frac{S_{DS}}{R} = 0.5 \]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

Vertical Force Distribution:
\[ F_x = C_{vx} \cdot V \]
\[ C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k} \]
\[ k(T) := \begin{align*}
  &\text{if } T_n \leq 0.5 \text{ s} \\
  &1 \\
  &\text{else if } T_n \geq 2.5 \text{ s} \\
  &2 \\
  &\text{else if } 0.5 \text{ s} < T_n < 2.5 \text{ s} \\
  &1 + \frac{(T_n - 0.5 \text{ s})}{(2.5 \text{ s} - 0.5 \text{ s})} \cdot (2 - 1)
\end{align*} \]
\[ k := k(T_n) = 1 \]
\[ h_1 = 0.305 \text{ m} \]
\[ h_2 = 2.591 \text{ m} \]
\[ C_{v2} := \frac{W_2 \cdot (h_2 + h_1)^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_2 + h_1)^k)} = 0.9 \]
\[ C_{v1} := \frac{W_1 \cdot h_1^k}{(W_1 \cdot h_1^k) + (W_2 \cdot (h_2 + h_1)^k)} = 0.1 \]
\[ F_2 := C_{v2} \cdot V = 9.753 \text{kip} \]
\[ F_1 := C_{v1} \cdot V = 1.027 \text{kip} \]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

Load Case: Direction 1

1. \(2 \times DL + W + LL + 0.5 \times (L_{f} \ or \ S \ or \ R)\) \hspace{1cm} (EQ 4.3.2-4)

Overturning Moments:

\[ L := 20 \ ft \quad B := 19.5 \ ft \]
\[ DL_{r} := 23.5 \ psf \quad LL_{r} := 20 \ psf \]
\[ Trib := \frac{B}{2} \]

Overturning moment from F2:
\[ M_{F, top} := F_{2} \cdot h_{2} = 82.903 \ kip \cdot ft \]

Resisting moment from Dead and Live:
\[ M_{r} := (1.2 \cdot DL_{r} + LL_{r}) \cdot Trib \cdot L \cdot \frac{L}{2} = 93.99 \ kip \cdot ft \]

**Required force in hold down:**

\[ T := \frac{M_{F, top} - M_{r}}{L - 1 \ ft} = -0.584 \ kip \]

**Required Compression Force:**

\[ C := T + (1.2 \cdot DL_{r} + LL_{r}) \cdot Trib \cdot L = 8.815 \ kip \]

**Required Studs for compression:**

CLT Capacity:
\[ F'_{c} := 35849 \ plf = 35.849 \ klf \]
Okay

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear
\[ \phi := 0.65 \quad Connections \]
\[ V'_{n} := 3.85 \ klf \]

For Base shear we would need:
\[ \frac{V}{V'_{n}} = 2.8 \ ft \quad \text{of Hinge along wall} \]
Appendix J. Mathcad Calculations for Seismic Loads: CLT Dynamic Unit

Load Case: Direction 1

\[ 0.9 \times DL + W \]  \hspace{1cm} (EQ 4.3.2-5)

**Overturning Moments:**

- \( L = 6.096 \ m \)
- \( B = 5.944 \ m \)
- \( DL_r := 23.5 \ \text{psf} \)
- \( LL_r := 20 \ \text{psf} \)
- \( Trib := \frac{B}{2} \)

Overturning moment from Ftop:

\[ M_{F,\text{top}} := F_2 \cdot h_2 = 82.903 \ \text{kip} \cdot \text{ft} \]

\[ T := \frac{M_{F,\text{top}}}{L - 1 \ \text{ft}} = 4.363 \ \text{kip} \]

**Required force in hold down:**

\[ T := \frac{M_{F,\text{top}}}{L - 1 \ \text{ft}} = 4.363 \ \text{kip} \]

**Required Compression Force:**

\[ C := T = 4.363 \ \text{kip} \]

**Required Studs for compression:**

CLT Capacity:

\[ F'_c := 35849 \ \text{plf} = 35.849 \ \text{klf} \]

Okay

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \]

Connections

\[ V' := 3.85 \ \text{klf} \]

For Base shear we would need:

\[ \frac{V}{V'} = 2.8 \ \text{ft} \]

of Hinge along wall
Load Case: Direction 2

\[ 1.2 \times DL + W + LL + 0.5 \times (L_r \text{ or } S \text{ or } R) \]  
\[(EQ\ 4.3.2-4)\]

**Overturning Moments:**

\[ L := 19.5 \text{ ft} \quad B := 20 \text{ ft} \]
\[ DL_r := 23.5 \text{ psf} \]
\[ LL_r := 20 \text{ psf} \]
\[ Trib := \frac{B}{2} \]

**Overturning moment from F2:**

\[ M_{F,\text{top}} := F_2 \cdot h_2 = 82.903 \text{ kip} \cdot \text{ft} \]

**Resisting moment from Dead and Live:**

\[ M_r := (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L \cdot \frac{L}{2} = 91.64 \text{ kip} \cdot \text{ft} \]

**Required force in hold down:**

\[ T := \frac{M_{F,\text{top}} - M_r}{L - 1 \text{ ft}} = -0.472 \text{ kip} \]

**Required Compression Force:**

\[ C := T + (1.2 \cdot DL_r + LL_r) \cdot Trib \cdot L = 8.927 \text{ kip} \]

**Required Studs for compression:**

CLT Capacity:

\[ F'_c := 35849 \text{ plf} = 35.849 \text{ klf} \quad \text{Okay} \]

**Required hinge length for base shear:**

Ultimate Strength of Hinge A in Shear

\[ \phi := 0.65 \quad \text{Connections} \]

\[ V'_n := 3.85 \text{ klf} \]

For Base shear we would need:

\[ \frac{V}{V'_n} = 2.8 \text{ ft} \quad \text{of Hinge along wall} \]
Load Case: Direction 2

$$0.9 \times DL + W \quad (EQ \ 4.3.2-5)$$

Overturning Moments:

$$L = 5.944 \ m \quad B = 6.096 \ m$$

$$DL_r := 23.5 \ \text{psf} \quad LL_r := 20 \ \text{psf}$$

Overturning moment from Ftop:

$$M_{F, top} := F_2 \cdot h_2 = 82.903 \ \text{kip} \cdot \text{ft}$$

Required force in hold down:

$$T := \frac{M_{F, top}}{L - 1 \ \text{ft}} = 4.481 \ \text{kip} \quad M_{F, top} = 82.903 \ \text{kip} \cdot \text{ft}$$

Required Compression Force:

$$C := T = 4.481 \ \text{kip}$$

Required Studs for compression:

CLT Capacity:

$$F'_c := 35849 \ \text{plf} = 35.849 \ \text{klf} \quad \text{Okay}$$

Required hinge length for base shear:

Ultimate Strength of Hinge A in Shear

$$\phi := 0.65 \quad \text{Connections}$$

$$V'_n := 3.85 \ \text{klf}$$

For Base shear we would need:

$$\frac{V}{V'_n} = 2.8 \ \text{ft} \quad \text{of Hinge along wall}$$
Appendix K. Mathcad Calculations for Tests 1A and 1B Z and Z’ Values
Appendix K. Mathcad Calculations for Tests 1A and 1B Z and Z’ Values

Test 1A (Continuous Hinge):

Design Values:

\[ D := .25 \text{ in} \]
\[ F_{yb} := 172000 \text{ psi} \] *From Simpson Strong Tie
\[ K_D := \left(10 \cdot \frac{D}{\text{in}}\right) + 0.5 \]
\[ R_d := K_D = 3 \]
\[ F_{em} := 5550 \text{ psi} \]
\[ F_u := 75 \text{ ksi} \]
\[ F_{es} := \frac{2 \cdot F_u}{1.6} = 93.75 \text{ ksi} \]
\[ R_e := \frac{F_{em}}{F_{es}} = 0.059 \]
\[ l_s := 0.12 \text{ in} \]
\[ l_m := 3.5 \text{ in} - l_s \]
\[ R_t := \frac{l_m}{l_s} = 28.167 \text{ *3.5" screw} \]
\[ k_1 := \sqrt{R_e + 2 R_e^2 \left(1 + R_t + R_t^2\right) + R_t^2 R_e^3} - (R_e \cdot (1 + R_t)) \]
\[ k_2 := -1 + \sqrt{\frac{2 (1 + R_e) + 2 F_{yb} \cdot (1 + 2 R_e) D^2}{3 F_{em} \cdot l_s^2}} \]
\[ k_3 := -1 + \sqrt{\frac{2 (1 + R_e) + 2 F_{yb} \cdot (2 + R_e) D^2}{3 F_{em} \cdot l_s^2}} \]

Im:
\[ Z_{im} := \frac{D \cdot l_m \cdot F_{em}}{R_d} = \left(1.563 \cdot 10^3\right) \text{ lbf} \]

Is:
\[ Z_{is} := \frac{D \cdot l_s \cdot F_{es}}{R_d} = 937.5 \text{ lbf} \]

II:
\[ Z_2 := \frac{k_1 \cdot D \cdot l_s \cdot F_{es}}{R_d} = 637.807 \text{ lbf} \]

IIIm:
\[ Z_{3m} := \frac{k_2 \cdot D \cdot l_m \cdot F_{em}}{(1 + 2 R_e) R_d} = 696.459 \text{ lbf} \]

IIIs:
\[ Z_{3s} := \frac{k_3 \cdot D \cdot l_s \cdot F_{em}}{(2 + R_e) R_d} = 373.212 \text{ lbf} \]
Appendix K. Mathcad Calculations for Tests 1A and 1B Z and Z’ Values

IV:

\[
Z_4 := \frac{D^2}{R_d} \cdot \sqrt{\frac{2 F_{em} \cdot F_{yb}}{3 \left(1 + R_e\right)}} = 510.664 \text{ lbf}
\]

\[
Z := \min (Z_{1m}, Z_{1s}, Z_2, Z_{3m}, Z_{3s}, Z_4) = 373.212 \text{ lbf} \quad Z \cdot 4 = 1.493 \text{ kip}
\]

\[
Z' = Z \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta \cdot C_{eg} \cdot C_{di} \cdot C_{tn} \cdot 3.32 \cdot 0.65 \cdot \lambda \quad C_M := 1 \quad C_t := 1
\]

\[
C_g := 1 \quad C_\Delta := 1
\]

Assuming C_{di}, C_{tn}, and C_{eg} are 1

\[
C_{di} := 1 \quad C_{tn} := 1 \quad C_{eg} := 1 \quad \lambda := 1 \quad K_f := 3.32
\]

\[
Z' := Z \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta \cdot C_{eg} \cdot C_{di} \cdot C_{tn} \cdot 3.32 \cdot 1 \cdot \lambda = (1.239 \cdot 10^3) \text{ lbf}
\]

\[
Z' \cdot 4 = 4.956 \text{ kip}
\]

Simpson Gives:

\[
Z := 420 \text{ lbf} \quad Z \cdot 4 = 1.68 \text{ kip}
\]

\[
Z \cdot K_f = 1.394 \text{ kip} \quad Z \cdot K_f \cdot 4 = 5.578 \text{ kip}
\]

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From Testing:

Ultimate := 9050 \text{ lbf}
Yield := 6450 \text{ lbf}

Stiffness_i := \frac{4580.69 \text{ lbf}}{0.134133 \text{ in}} = 34.15 \text{ kip/in}
Appendix K. Mathcad Calculations for Tests 1A and 1B Z and Z’ Values

Test 1B (Mortise Hinge):

Design Values:

\[ D := .137 \text{ in} \]

\[ F_{yb} := 80000 \text{ psi} \]

\[ K_D := 2.2 \quad R_d := K_D = 2.2 \]

\[ F_{em} := 5550 \text{ psi} \quad F_u := 70 \text{ ksi} \quad F_{es} := \frac{2 \cdot F_u}{1.6} = 87500 \text{ psi} \]

\[ R_e := \frac{F_{em}}{F_{es}} = 0.063 \]

\[ l_s := 0.0855 \text{ in} \quad l_m := 1 \text{ in} \quad l_t := \frac{l_m}{l_s} = 10.696 \quad *1" \text{ screw} \]

\[ k_1 := \sqrt{R_e + 2 \frac{R_e}{R_t} \left(1 + R_t \right)^2 + \frac{R_e}{R_t} \left(R_t \right)^3 - (R_e \cdot \left(1 + R_t \right)) \frac{1}{\left(1 + R_e \right)}} = 0.292 \]

\[ k_2 := -1 + \sqrt{2 \frac{1 + R_e}{\left(1 + R_t \right)^2} + \frac{2 F_{yb} \cdot (1 + R_e)}{3 F_{em} \cdot \frac{D^2}{l_s^2}}} = 0.539 \]

\[ k_3 := -1 + \sqrt{2 \frac{1 + R_e}{R_e} + \frac{2 F_{yb} \cdot (2 + R_e)}{3 F_{em} \cdot \frac{D^2}{l_s^2}}} = 8.189 \]

Im:

\[ Z_{im} := \frac{D \cdot l_m \cdot F_{em}}{R_d} = 316.064 \text{ lbf} \]

Is:

\[ Z_{is} := \frac{D \cdot l_s \cdot F_{es}}{R_d} = 465.878 \text{ lbf} \]

II:

\[ Z_2 := k_1 \cdot \frac{D \cdot l_s \cdot F_{es}}{R_d} = 135.987 \text{ lbf} \]

IIIIm:

\[ Z_{3m} := \frac{k_2 \cdot D \cdot l_m \cdot F_{em}}{(1 + 2 R_e) R_d} = 151.304 \text{ lbf} \]

IIIs:

\[ Z_{3s} := \frac{k_3 \cdot D \cdot l_s \cdot F_{em}}{(2 + R_e) R_d} = 117.276 \text{ lbf} \]
Appendix K. Mathcad Calculations for Tests 1A and 1B Z and Z’ Values

IV:

\[ Z_4 = \frac{D^2}{R_d} \cdot \frac{2 F_{em} \cdot F_{yb}}{3 \left(1 + R_c\right)} = 142.334 \text{ lbf} \]

\[ Z = \min (Z_{1m}, Z_{1s}, Z_2, Z_{3m}, Z_{3s}, Z_4) = 117.276 \text{ lbf} \]

\[ Z' = Z \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta \cdot C_{eg} \cdot C_{di} \cdot C_{tn} \cdot 3.32 \cdot 0.65 \cdot \lambda \quad C_M := 1 \quad C_t := 1 \]

\[ C_g := 1 \quad C_\Delta := 1 \]

Assuming C_{di}, C_{tn}, and C_{eg} are 1

\[ C_{di} := 1 \quad C_{tn} := 1 \quad C_{eg} := 1 \quad \lambda := 1 \]

\[ Z' := Z \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta \cdot C_{eg} \cdot C_{di} \cdot C_{tn} \cdot 3.32 \cdot 1 \cdot 1 = 389.357 \text{ lbf} \]

\[ Z_{adj} := Z' \cdot 4 = 1.557 \text{ kip} \quad \text{four screws per leaf} \]

\[ Z_{adj} := 3.115 \text{ kip} \]

\[ Z' \cdot 4 = 3.115 \text{ kip} \]

\[ Z' \cdot 4 = 4.672 \text{ kip} \]

From Test Data:

\[ \frac{Stiffness}{0.0626576 \text{ in}} = \frac{2322.49 \text{ kip}}{3.707 \cdot 10^4 \text{ in}} = 3.707 \cdot 10^4 \text{ kip/in} \]

\[ \frac{Ultimate}{Yield} := 5.1 \text{ kip} \]

\[ \frac{Yield}{Yield} := 2.57 \text{ kip} \]
Appendix L. Foundation Calculations
Appendix L. Foundation Calculations

This document contains calculations for a possible foundation system.

This foundation system consists of the structures being supported and elevated off the ground by CMU blocks.

It will be assumed that the CMU blocks are placed every 5' along:
- Rim Joist span for light-frame units
- Minor direction for CLT units
and directly under the 3 interior column supports.

The maximum base shear from all lateral load conditions:
\[ V_b := 10.8 \text{ kip} \quad \text{CLT - Very High Seismic Base Shear (highest)} \]

\[ V_n = 4 \cdot A_{nv} \cdot \sqrt{f'_m} \]

\[ A_{nv} := 91.5 \cdot \text{in}^2 = 91.5 \text{ in}^2 \]

Where \( M_e/(V_u d_o) \geq 1.0 \)

\[ V_n \leq 4A_{mv} \sqrt{f'_m} \quad \text{(Equation 3-22)} \]

Assume

\[ f'_m := 1600 \text{ psi} \]

\[ V_n := 4 \cdot A_{nv} \cdot \text{psi} \cdot \sqrt{f'_m \text{ psi}} = 14.64 \text{ kip} \quad \phi := 0.8 \]

\[ \phi V_n := \phi \cdot V_n = 11.712 \text{ kip} \]

Use spaced every 5' along:
- Rim Joist span for light-frame units
- Minor direction for CLT units

Required Shear Strength in Anchor Rod:

\[ \frac{V_b}{4} = 2.7 \text{ kip} \]

\[ K_{f_r} := 3.32 \]

\[ \phi_z := 0.65 \]
Appendix L. Foundation Calculations

### Table 1—IBC and IRC Allowable Tension and Shear Loads for Titen HD Screw Anchors Installed in the Face of Fully Grouted CMU Masonry Construction

<table>
<thead>
<tr>
<th>ANCHOR MATERIAL</th>
<th>ANCHOR DIA.² (in.)</th>
<th>DRILL BIT DIA. (in.)</th>
<th>MIN. EMB.² (in.)</th>
<th>ANCHOR LOCATION¹ (in.)</th>
<th>ALLOWABLE LOADS BASED ON ANCHORS INSTALLED AT DISTANCES ≥ CRITICAL EDGE DISTANCE, cₘₐₓ, AND CRITICAL SPACING, sₘₐₓ, (lbf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Edge Distance</td>
<td>Spacing</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Critical, (c_{crit})</td>
<td>Minimum, (c_{min})</td>
</tr>
<tr>
<td>CARBON STEEL</td>
<td>¼&quot;</td>
<td>¼&quot;</td>
<td>2(\frac{1}{2})&quot;</td>
<td>4</td>
<td>1(\frac{3}{4})</td>
</tr>
<tr>
<td></td>
<td>½&quot;</td>
<td>½&quot;</td>
<td>2(\frac{1}{4})&quot;</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>½&quot;</td>
<td>½&quot;</td>
<td>3(\frac{1}{2})&quot;</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>¼&quot;</td>
<td>¼&quot;</td>
<td>4(\frac{1}{2})&quot;</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>STAINLESS STEEL</td>
<td>¼&quot;</td>
<td>¼&quot;</td>
<td>5(\frac{1}{2})&quot;</td>
<td>12</td>
<td>24</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, 1 psi = 6.89 kPa, 1 lbf = 4.48 N.

Allowable loads for anchors installed in the face or end of fully grouted CMU masonry walls subjected to combined shear and tension forces must be determined by the following equation:

\[
\frac{P_s}{P_t} + \frac{V_s}{V_t} \leq 1.0
\]

where:
- \(P_t\) = Applied service tension load.
- \(P_s\) = Allowable service tension load.
- \(V_s\) = Applied service shear load.
- \(V_t\) = Allowable service shear load.

Given:
- \(P_t := 665 \text{ lbf} \cdot K_{fz} \cdot \phi_z = 1435.07 \text{ lbf}\)
- \(V_t := 990 \text{ lbf} \cdot K_{fz} \cdot \phi_z = 2136.42 \text{ lbf}\)
- \(V_s := V_b = 10800 \text{ lbf}\)
- \(P_s := 4.5 \text{ kip} = 4500 \text{ lbf}\)

CLT - Very High Seismic (highest)

\[n := 10\] would need 10 anchor rods to resist shear force.

Therefore would need at least 5 CMU blocks spaced evenly under shear wall. Grout filled both cells and anchor rod in both cells.

*Double blocks at corner
Appendix L. Foundation Calculations

Gravity Load:

\[
f'_{m} := 1600 \text{ psi} \quad h := 8 \text{ in} \quad \phi_c := 0.6 \quad r := 2.2 \text{ in}
\]

\[
P_n := \phi_c \cdot 0.8 \cdot (0.8 \cdot f'_{m} \cdot A_{nv}) \cdot \left(1 - \left(\frac{h}{140 \cdot r}\right)^2\right) = 56.18 \text{ kip}
\]

\[
\frac{h}{r} = 3.636
\]

use total area as concrete. will provide lower design strength

### Table 8a: Horizontal Section Properties (Masonry Spanning Vertically)

<table>
<thead>
<tr>
<th>Unit</th>
<th>Grout spacing (in.)</th>
<th>Mortar bedding</th>
<th>Net cross-sectional properties</th>
<th>Average cross-sectional properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>(A_c \text{ (in.}^2/\text{ft}))</td>
<td>(A_{avg} \text{ (in.}^2/\text{ft}))</td>
</tr>
<tr>
<td>Hollow</td>
<td>No grout</td>
<td>Face shell</td>
<td>48.0</td>
<td>56.2</td>
</tr>
<tr>
<td>Hollow</td>
<td>No grout</td>
<td>Full</td>
<td>56.2</td>
<td>56.2</td>
</tr>
<tr>
<td>Hollow</td>
<td>100% solid/solidly grouted</td>
<td>Full</td>
<td>91.5</td>
<td>91.5</td>
</tr>
<tr>
<td>Hollow</td>
<td>16</td>
<td>Face shell</td>
<td>70.6</td>
<td>73.3</td>
</tr>
<tr>
<td>Hollow</td>
<td>24</td>
<td>Face shell</td>
<td>63.1</td>
<td>67.6</td>
</tr>
<tr>
<td>Hollow</td>
<td>32</td>
<td>Face shell</td>
<td>59.3</td>
<td>64.7</td>
</tr>
<tr>
<td>Hollow</td>
<td>40</td>
<td>Face shell</td>
<td>57.0</td>
<td>63.0</td>
</tr>
<tr>
<td>Hollow</td>
<td>48</td>
<td>Face shell</td>
<td>55.5</td>
<td>61.9</td>
</tr>
<tr>
<td>Hollow</td>
<td>72</td>
<td>Face shell</td>
<td>53.0</td>
<td>60.0</td>
</tr>
<tr>
<td>Hollow</td>
<td>96</td>
<td>Face shell</td>
<td>51.8</td>
<td>59.0</td>
</tr>
<tr>
<td>Hollow</td>
<td>120</td>
<td>Face shell</td>
<td>51.0</td>
<td>58.4</td>
</tr>
</tbody>
</table>

### Table 8b: Vertical Section Properties (Masonry Spanning Horizontally)

<table>
<thead>
<tr>
<th>Unit</th>
<th>Grout spacing (in.)</th>
<th>Mortar bedding</th>
<th>Net cross-sectional properties</th>
<th>Average cross-sectional properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>(A_c \text{ (in.}^2/\text{ft}))</td>
<td>(A_{avg} \text{ (in.}^2/\text{ft}))</td>
</tr>
<tr>
<td>Hollow</td>
<td>No grout</td>
<td>Face shell</td>
<td>48.0</td>
<td>55.4</td>
</tr>
<tr>
<td>Hollow</td>
<td>No grout</td>
<td>Full</td>
<td>48.0</td>
<td>56.2</td>
</tr>
<tr>
<td>Hollow</td>
<td>100% solid/solidly grouted</td>
<td>Full</td>
<td>91.5</td>
<td>91.5</td>
</tr>
<tr>
<td>Hollow</td>
<td>16</td>
<td>Face shell</td>
<td>69.8</td>
<td>77.1</td>
</tr>
<tr>
<td>Hollow</td>
<td>24</td>
<td>Face shell</td>
<td>62.5</td>
<td>69.9</td>
</tr>
<tr>
<td>Hollow</td>
<td>32</td>
<td>Face shell</td>
<td>58.9</td>
<td>66.3</td>
</tr>
<tr>
<td>Hollow</td>
<td>40</td>
<td>Face shell</td>
<td>56.7</td>
<td>64.1</td>
</tr>
<tr>
<td>Hollow</td>
<td>48</td>
<td>Face shell</td>
<td>55.3</td>
<td>62.6</td>
</tr>
<tr>
<td>Hollow</td>
<td>96</td>
<td>Face shell</td>
<td>51.6</td>
<td>59.0</td>
</tr>
<tr>
<td>Hollow</td>
<td>120</td>
<td>Face shell</td>
<td>50.9</td>
<td>58.3</td>
</tr>
</tbody>
</table>

\(\text{Table 8–8-inch (203-mm) Single Wythe Walls, 2 in. (51 mm) Face Shells (nonstan}\)
Appendix M. Ballistic Calculations
Appendix M. Ballistic Calculations

This document contains the calculations for determining the ballistic resistance of a CLT Panel.

From Kathryn Sanborn Research:

\[ T_w = C_1 \times \left( \frac{v_s^a}{10^c \times \rho^d \times H^f} \right) \]  
\hspace{1cm} (EQ 2.5.2-1)

Where:
\[ T_w = \text{Thickness required to stop perforation} \]
\[ v_s = \text{Striking velocity} \]
\[ \rho = \text{Density of wood} \]
\[ H = \text{Hardness of wood} \]
\[ C_1, a, c, d, f = \text{Constants} \]

Constants From:

\[ C_1 := 164.3 \]
\[ a := 1.493 \]
\[ c := 4.022 \]
\[ d := 1.373 \]
\[ f := 0.102 \]

\[ H := 656 \text{ lbf} \]
\[ \rho := 34.2 \text{ pcf} \]

\[ v_s := 1650 \text{ fps} \]

\[ T_w := C_1 \cdot \left( \frac{v_s^a}{10^c \cdot \rho^d \cdot H^f} \right) = 4.016 \]

Therefore, a 3-Ply CLT panel with a thickness of 4.125" should be able to stop a half inch diameter steel ball with a striking velocity of 1650 fps. This is made off the assumptions of hardness and density listed above.
Appendix N. Cost Calculations
Appendix N. Cost Calculations

This document contains the cost calculations for each structure type.

<table>
<thead>
<tr>
<th>Panel Square Footage:</th>
<th>$ := 1 USD Dollar</th>
</tr>
</thead>
</table>

Static Unit:

\[ A_s := 685 \text{ ft}^2 \]

Dynamic Unit:

\[ A_d := 1597 \text{ ft}^2 \]

Cost of Panel:

**Light-Frame:**

\[ C_{lf} := 7.5 \frac{\text{ $}}{\text{ft}^2} \]

**CLT:**

\[ C_{clt} := 40 \frac{\text{ $}}{\text{ft}^2} \]

Material Cost:

**CLT Dynamic:**

\[ C_{clt_{dyn}} := A_d \cdot C_{clt} = 63880 \text{ $} \]

**CLT Static:**

\[ C_{clt_{sta}} := A_s \cdot C_{clt} = 27400 \text{ $} \]

**LF Dynamic:**

\[ C_{lf_{dyn}} := A_d \cdot C_{lf} = 11978 \text{ $} \]

**LF Static:**

\[ C_{lf_{sta}} := A_s \cdot C_{lf} = 5138 \text{ $} \]

Total Structure (Framing = 22% LF and 43% CLT)

\[ CLT_{DYN} := \frac{C_{clt_{dyn}}}{0.43} = 148558 \text{ $} \]

\[ CLT_{STA} := \frac{C_{clt_{sta}}}{0.43} = 63721 \text{ $} \]

\[ LF_{DYN} := \frac{C_{lf_{dyn}}}{0.22} = 54443 \text{ $} \]
Appendix N. Cost Calculations

\[ LF_{STA} = \frac{C_{lf \_sta}}{0.22} = 23352 \ $ \]
Appendix O. Structure Weight Calculations
Appendix O. Structure Weight Calculations

**Dynamic Units:**
Light frame unit Weight calculations:

- **stud** := \( \frac{11}{8} \text{ ft} \cdot 2 \text{ psf} = 0.005 \text{ psi} \)  
  2x4 at 24" oc

- **sheathing** := \( \frac{\text{59 lbf}}{4 \text{ ft} \cdot 8 \text{ ft}} = 1.844 \text{ psf} \)  
  Zip R-6 sheathing

- **gypsum** := \( \frac{\text{57 lbf}}{4 \text{ ft} \cdot 8 \text{ ft}} = 1.781 \text{ psf} \)  
  1/2" gypsum board

- **insulation** := 0.5 \text{ psf}  
  
- **plates** := \( \frac{3 \cdot 11}{8 \cdot 8} \text{ psf} = 0.516 \text{ psf} \)

- **wall** := **stud** + **sheathing** + **gypsum** + **insulation** + **plates** = 5.328 \text{ psf}

Assume all light frame panels weigh 6 \text{ psf} \hspace{1cm} LF := 7.5 \text{ psf}

**CLT unit weight calculations:**

\( CLT := 13.5 \text{ psf} \)

Dimensions of Each Panel:

- Panel 1:
  - Width = 8'
  - Length = 20'
- Panel 2:
  - Height: 7'
  - Length = 20'
- Panel 3:
  - Width = 6'
  - Length = 20'
- Panel 4:
  - Width = 3'
  - Height = 8.5'
- Panel 5:
  - Width = 9.3'
  - Height = 10'
- Panel 6:
  - Width = 8'
  - Length = 20'
- Panel 7:
  - Height = 8.6'
  - Length = 20'

\[
\begin{align*}
AP1 & := 8 \text{ ft} \cdot 20 \text{ ft} = 160 \text{ ft}^2 \\
AP2 & := 7 \text{ ft} \cdot 20 \text{ ft} = 140 \text{ ft}^2 \\
AP3 & := 6 \text{ ft} \cdot 20 \text{ ft} = 120 \text{ ft}^2 \\
AP4 & := 3 \text{ ft} \cdot 8.5 \text{ ft} = 25.5 \text{ ft}^2 \\
AP5 & := 9.5 \text{ ft} \cdot 10 \text{ ft} = 95 \text{ ft}^2 \\
AP6 & := 8 \text{ ft} \cdot 20 \text{ ft} = 160 \text{ ft}^2 \\
AP7 & := 5 \text{ ft} \cdot 20 \text{ ft} = 100 \text{ ft}^2
\end{align*}
\]

Unit has the following:

- 2 P1
- 2 P2
- 1 P3
- 3 P4
- 4 P5
- 2 P6
- 1 P7

\[
Total_{\text{area}} := 2 \cdot AP1 + 2 \cdot AP2 + AP3 + 3 \cdot AP4 + 4 \cdot AP5 + 2 \cdot AP6 + 1 \cdot AP7 = 1596.5 \text{ ft}^2
\]

\[
CLT_{\text{total}} := CLT \cdot Total_{\text{area}} = 21.553 \text{ kip}
\]

\[
LF_{\text{total}} := LF \cdot Total_{\text{area}} = 11.974 \text{ kip}
\]
Appendix O. Structure Weight Calculations

**Static Units:**

**LF Static**

\[ LF = 7.5 \text{ psf} \]  
Weight of Light-Frame wall

Square footage of wall:

\[ A = (2 \cdot 8 \cdot 20 + 8.5 \cdot 9 + 4.5 \cdot 9 + 7.5 \cdot 9 + 20 \cdot 9) \cdot ft^2 = 684.5 \text{ ft}^2 \]

\[ W = LF \cdot A = 5.134 \text{ kip} \]

**CLT Static**

\[ CLT = 13.5 \text{ psf} \]  
Weight of SL-V3 3-Ply

\[ A = (2 \cdot 8 \cdot 20 + 8.5 \cdot 9 + 4.5 \cdot 9 + 7.5 \cdot 9 + 20 \cdot 9) \cdot ft^2 = 684.5 \text{ ft}^2 \]

\[ W = CLT \cdot A = 9.241 \text{ kip} \]
Appendix P. Dynamic Unit Folding Sequence (CLT Style)
Appendix Q. Dynamic Unit Floor Plan Option
Appendix R. Static Unit (CLT Style)