THE PENNSYLVANIA STATE UNIVERSITY
DEPARTMENT OF ARCHITECTURAL ENGINEERING

WOOD REDESIGN OF THE ADDITION TO 11141 GEORGIA AVENUE, WHEATON, MD

SAMANTHA DEVRIES
SPRING 2015

A thesis
submitted in partial fulfillment
of the requirements
for a baccalaureate degree
in Architectural Engineering

Thesis Advisor: Thomas E. Boothby

Structural Option

11141 Georgia Avenue: High Rise Residential Apartments
Located in: Wheaton, MD

Building Statistics

Full Height: 158 Feet
Number of Stories: 14
Size: 179,760 GSF Square Feet
Cost: $44 Million (for the addition)
Construction Dates: February 2013 - August 2014
Project Delivery Method: Contractor at Risk

Project Team

Owner: ML Wheaton, LLC c/o Lower Enterprises
General Contractor and CM: Whiting-Turner
Architect: Bonstra Haresign Architects, LLP
Structure: Rathgeber/Goss Associates
Mechanical: Brothers Ductwork HVAC, Inc.
Plumbing: KNI Engineering, Inc.
Lighting Design: Gilmore Lighting Design


Photo of building from nearby parking garage roof: Photo taken by Samantha deVries

Structural Systems

- Original Concrete Building
  - Concrete moment frames
  - Concrete floor slabs
  - Spread footings and retaining walls
- New Addition
  - Steel moment frames
  - Lightweight composite floor joists with deck
- Loads
  - Original loads for office building
  - New live loads smaller for residential
- Renovation Work
  - New stairwell and elevator locations
  - New utility openings
  - Façade modifications


Photo of typical apartment: Photo courtesy of The George (Apartment)

Architecture

- 5 story 1960's office building
- 7 story addition
- High rise apartment building with one and two bedroom studios.

Construction

- Underpin Foundations
- Renovations work in existing building
- Construct addition directly above existing

Mechanical

- Cooling by rooftop chiller condensing units
- Units have occupant operable windows
- Heating by electrical heaters and heat pumps.

Electrical/Lighting

- Recessed lighting in apartments
- Pendant and wall mounted fixtures in lobbies
- 2 Main Power Distributers fed from a transformer
- One 1400 KVA and one 1750 KVA


Photo of rooftop terrace: Photo courtesy of The George (Apartment)

Samantha deVries: Structural Option
Advisor: Ali Said

Project Sponsor: Rathgeger/Goss Associates
https://www.engr.psu.edu/ae/thesis/portfolios/2015/sjd5225/deVries_AE_Thesis/Home.html

Executive Summary

The building located at 11141 Georgia Avenue in Wheaton, Maryland was recently renovated into an apartment building, finishing construction in August of 2014. The original building was a 7 story concrete office building. A 7 story addition was added on top of the existing structure in joist-framed steel, and the concrete portion of the building was renovated to meet new architectural needs.

The following report includes the methods and processes used in the analysis and redesign of the addition. Both the gravity and lateral systems were analyzed in the redesigned system. Also included is breadth work in the topics of construction management and mechanical.

In order to keep the addition lightweight to minimize effects on the existing system, wood was chosen for the redesign. Although wood construction does not currently meet code US for the 7 story addition, the report discusses the research regarding taller wood buildings and the use of wood in taller buildings in other countries. A purpose of the report was to discover whether or not a wood addition would be feasible in the case of 11141 Georgia Ave.

A panel product called Cross Laminated Timber was used for the floors, which spans a full bay between girders. Glulam is used for the girders and columns. The gravity system was designed for flexure, deflections, fire performance, and connections for use with dyrwall encapsulation. Fire performance calculations were completed with the assumption that all wood structural elements will be encapsulated with a single layer of drywall.

The lateral system included several concrete shear walls to resist wind loading, the controlling lateral case over seismic. ETABS was used to complete the lateral system design. Several methods were used to validate the model.

The topics of construction management and mechanical systems were also explored in this report. The construction breadth determined that the redesigned system is competitive with the existing system when considered both cost and schedule. The wood redesign cannot have enclosed spaces, and thus new mechanical system was chosen to improve the aesthetics of the equipment in each apartment.

After completing the wood redesign, it was found that the wood alternate is structurally feasible as an addition. There are both challenges and benefits to using wood in the addition. Although there are several challenges with regards to fire safety, research has shown that heavy timber can meet safety requirements. Finally, the wood redesign does not add too much cost considering it significantly reduces the schedule, and it is a very lightweight structure.

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Acknowledgments

I would like to thank the following people for helping me throughout my thesis work this year:

- The engineers at Rathgeber and Goss Associates for allowing me to use 11141 Georgia Avenue for my thesis, especially Michael Goss who provided building plans and answered many of my questions.
- My parents who have supported me and been there for me throughout all of my years at Penn State.
- The AE faculty for their time and advice, especially my advisor Dr. Thomas Boothby.
- All of my classmates and friends.

1 Introduction

1.1 Existing Building

11141 Georgia Ave is a high-rise residential apartment building. The original building, built in 1962, was a 5 story concrete office building with 2 basement levels. When the building changed owners, it was expanded to meet the needs of the new owner, rather than being torn down. Construction of a 7 story addition in steel framing on top of the existing building began in February of 2013 and was completed in August of 2014 at a cost of 44 million dollars for the addition.

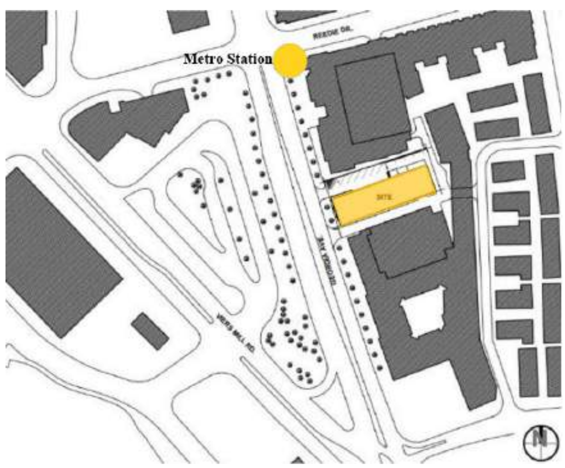


Figure 1.1: Building Location on Site, from Architectural drawings

The residential units are one and two-bedroom studio apartments. There is a rooftop terrace with a small wading pool, aesthetically pleasing views, and a penthouse lounge for residents of the building, which includes dining areas, kitchen space for events, a fitness center, and a game room. There is a location to store and repair bikes in the building, and the site is closely located to the Wheaton Metro Station, shown in figure 1.1. The building is located near the corner of Reedie Drive and Georgia Avenue in Wheaton, MD. Figure 1.2 provides a view of the building.

Figure 1.2: View of 11141 Georgia Ave

1.1.1 Structural Systems Overview

The original structure was built in concrete on spread footing foundations. The addition to the structure was built in steel. The foundations include spread footings and retaining walls, which required a few modifications due to layout changes. The original building is framed with structural two-way slabs and concrete columns. The original floor framing also required modifications to account for changes in the layout of stairwells and elevators, and the addition of other openings for new utilities, trash chutes, etc.

The new addition of 7 stories is framed in steel with columns that match the original building's concrete column grid. The floors are framed with W-shapes and composite floor joists, and the roof is framed with roof joists. The lateral system of the original building includes concrete perimeter moment frames. The steel addition uses steel moment frames to resist lateral loads. Many of the connections and joint details include tie-in to the original building. The following sections will cover the building's structural systems in further detail, covering the original building, its modifications, and the new addition's structure.

1.1.2 Foundations

The foundation system contains the original construction from the 1960's as well as some modifications to account for a modified layout.

Foundation System Prior to Addition

The original foundations of 11141 Georgia Ave were designed for 8000 psf allowable soil bearing stress from columns lines 1-5 and 4000 psf from column lines 6-12. The foundations consist of spread footings averaging 13 feet square with a pier, on top of which rests the structural column. Larger combined footings are used along column lines C and D.

The building is built on a slight hill , and therefore, there is a basement retaining wall in the basement structure along the north side of the building and between the levels.

Modifications to Foundations

Geotechnical exploration confirmed the 4000 psf and 8000 psf values from the original 1960's drawing set. Some existing footings required underpinning due to the addition of an elevator pit to accommodate 3 new elevators. The lowest basement level slab was filled in where the 2 original elevators were removed. The existing stairwell was removed, and 2 new stairwells were added. New foundations were added to support new CMU bearing walls around the slab edge at the new openings for the stairs and elevators.

1.1.3 Gravity System

The existing portion of the building is flat slab with drop panels construction. Due to differences in the occupancy type of the original building and the new structure, the gravity live loads are smaller. The original penthouse structure was also removed. Due to the new live loads, the removal of the penthouse, and the use of steel for the addition which is a significantly lighter material than concrete, very little work on the foundations was required for gravity loads despite the 7-story addition in steel. Modifications were required in the slab floors to accommodate layout changes. The addition was built out of steel to impose a lighter dead load on the original structure than if it were built out of concrete.

Original Concrete Structure

The original building is a concrete structure. The layout consists of a square column grid of 3 bays by 10 bays, each bay approximately 21'by 20', with a single row of 26'bays on the west end of the building. See figure 1.3 for a typical floor plan.

Level B1 has a 6 1/2" slab, the first floor has a 6 1/2" slab in the office area, and an 8" slab everywhere else, and all other floors (2nd to 5th) have a 6 1/2" slab. The roof has an 8" slab in the penthouse to support the mechanical equipment, and all other areas of the roof as well as the penthouse roof have the typical 6 1/2" slab. (See figure 1.4 for slab thicknesses). There are 7'x7'x4" drop panels typical at the columns.

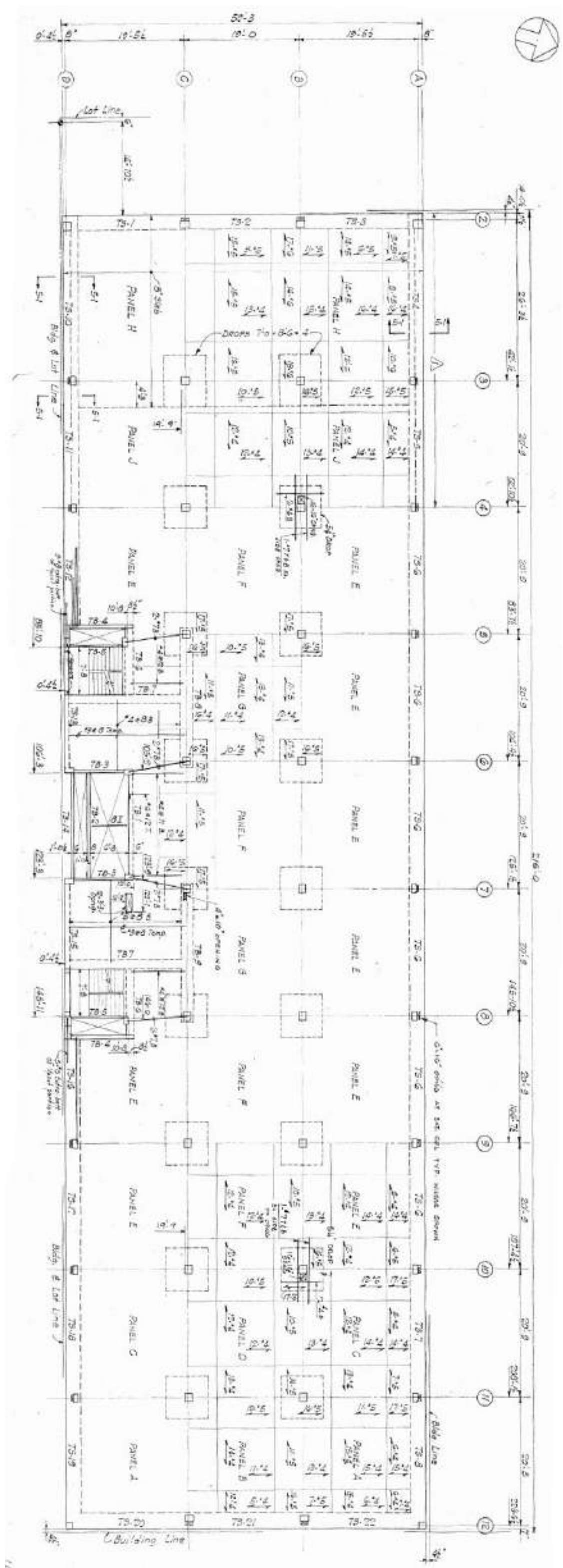


Figure 1.3: Typical Original Concrete Structure Floor Plan, From Existing Structural Drawings

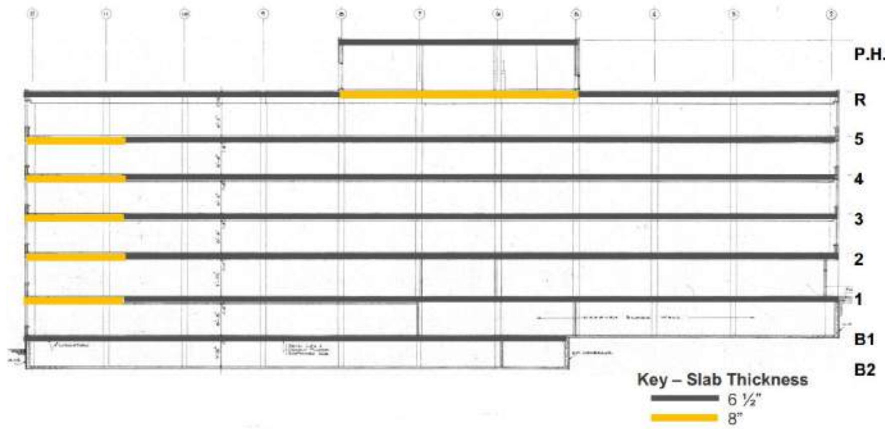


Figure 1.4: Section through existing building showing slab thicknesses, base section from Drawing A10

Concrete System Renovations

A few modifications were made to the slabs to accommodate layout changes and new openings. Typical on all floors were the demolition of slab to create new openings for new elevator and stairwell positions. A combination of load bearing CMU walls as shown in figure 1.5 and new steel W-shapes were used to support the slab edges around the new openings. Existing openings at the old elevator and stairwell were filled in with new slab. In spots where new openings were added in drop panels and close to columns, (such as the openings for trash chutes), carbon fiber reinforcement was added. Several new shaft openings were also cut in the slab more towards the inner portion of their respective bays.

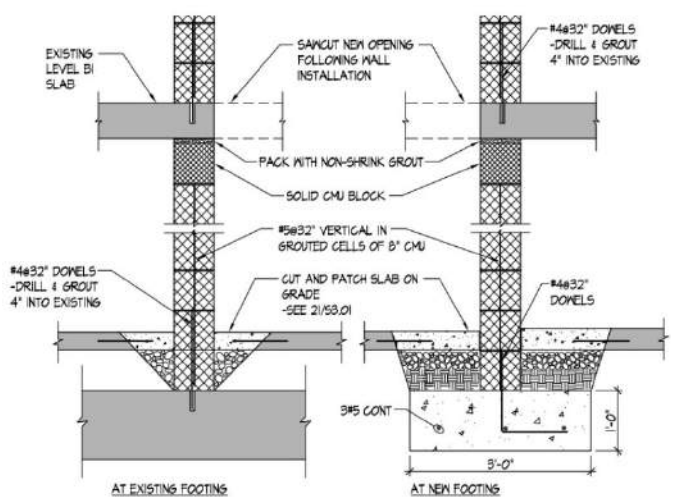


Figure 1.5: Section through new load bearing CMU Walls. Existing slab was cut to allow walls to bear on existing or new footings. From Drawing 1/S3.02

Steel Addition

The 7-story addition is framed in steel with the column layout of W-shapes directly matching the original concrete column layout. The typical girder size spanning south to north is a W10x33 due to the small bay size and lower residential live loads. The joists spanning east to west are typically 12" deep ecospan composite floor joists at 4' on center with W12 shapes typical along the column lines. The structural slab consists of a 1" steel deck with 2 1/2" of normal weight concrete topping for at total thickness of 3 1/2" reinforced with welded wire fabric.

1.1.4 Lateral System

This section will provide a brief overview of the existing lateral system. The original building's lateral system as well as the new addition's lateral system will be discussed in the following sections.

Original Concrete Lateral System

The original building resisted lateral loads through its concrete moment frame structure. The addition of multiple stories resulted in increased shear and wind loading on the existing building's concrete moment frames. However, the system is sufficiently stiff to resist the additional loads. CMU shear walls were added around the stair and elevator cores up to the top of the concrete portion of the building, but they are not nearly as stiff as the concrete frames and contribute very little to lateral resistance.

Steel Addition

The new steel frame addition has several moment frames which resist lateral loads. See Figure 1.6 for typical floor plan with highlighted locations of moment frames.

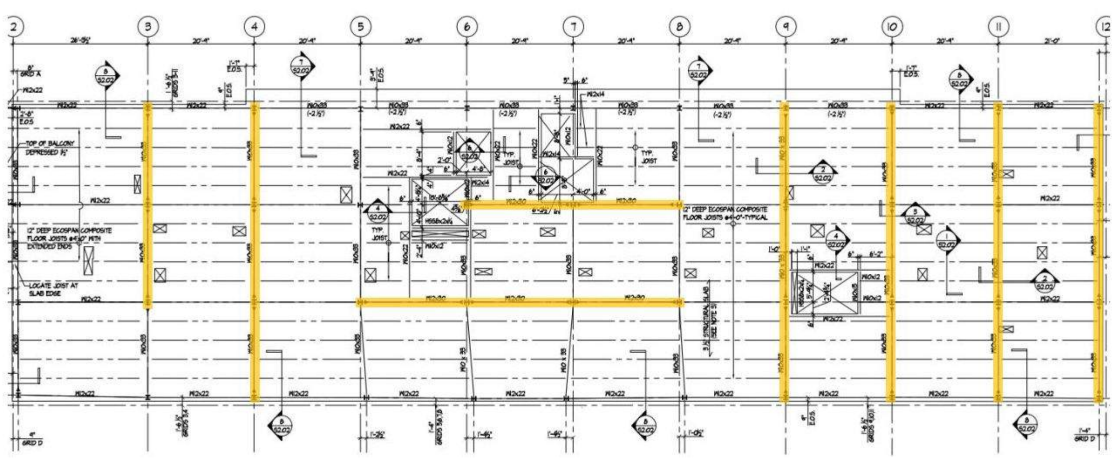


Figure 1.6: Moment Frames shown highlighted on typical floor plan. From Drawing S1.07

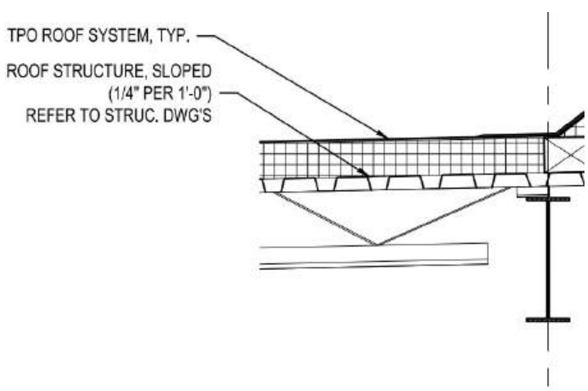
1.2 Load Analysis

The following section will discuss the loads determined for the existing building. Included is a summary of the gravity and lateral loads determined to be acting on the building. All loads were calculated using ASCE 7-05 (ASCE, 2005), since that was the version used for the existing building's design. Gravity load calculations are available in Appendix A, and lateral load calculations are available in Appendix B.

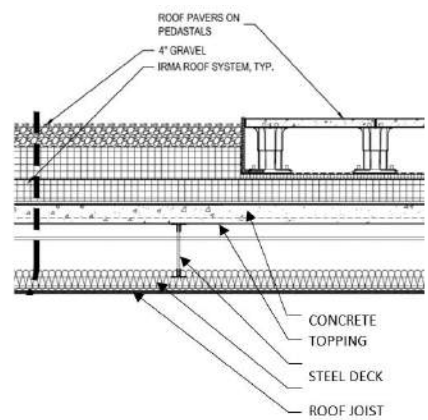
1.2.1 Gravity Loads

Roof Loads

The roof load calculation includes the roof dead loads, roof live loads, and snow loads. The loads calculated will also be compared to the loads used in the design of the building. Figure 1.7 (a) and figure 1.7 (b) shows the layers of roofing considered in the dead load calculations. Figure 1.8 shows the snow load diagram.



(a) *Section through penthouse roof. From 1/A4.09*



(b) *Section through roof at the 12th floor terrace level. From 3/A4.09*

Figure 1.7: Sections through Penthouse Roof and Outdoor Terrance Roof

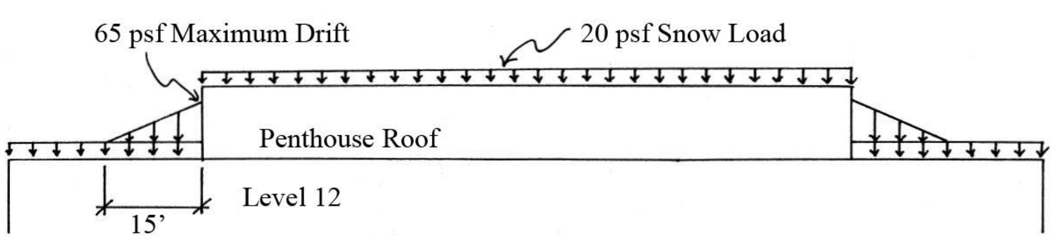
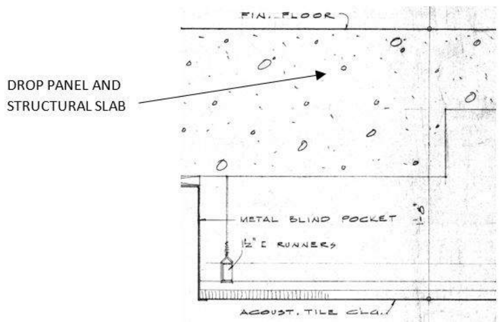


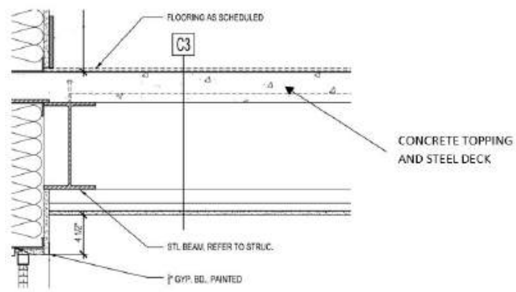
Figure 1.8: Snow Drift Diagram

Floor Loads

The floor load calculations will include both the dead and live loads for both the original concrete floors and the new addition's floors. Figure 1.9 a below shows a section through a typical concrete slab in the original building, and figure 1.9 b shows a section through a typical floor of the addition.



(a) *Section through typical floor in existing building. From A.12: Window and Wall Sections*

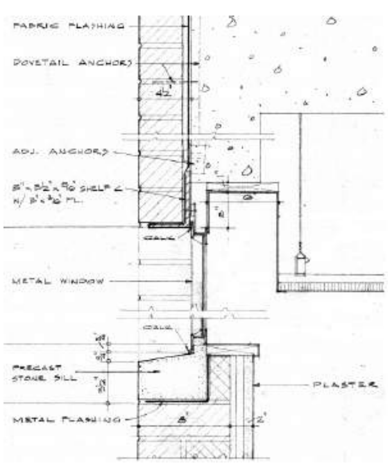


(b) *Section through typical floor in addition. From 10/A4.20*

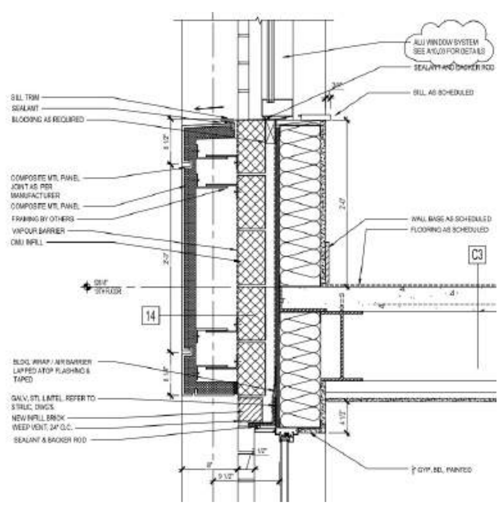
Figure 1.9: Sections through Original Concrete and New Steel Floors

Exterior Wall Loads

The exterior wall load calculations will produce a line load around the perimeter of the building for the original façade and the new façades. Figure 1.10 (a) is a typical section through the exterior wall in the original building, and figure 1.10 (b) is a section through a typical exterior wall in the addition.



(a) *Section through typical exterior wall in existing building. From A.12: Window and Wall Sections*



(b) *Section through typical exterior wall in addition. From 4A.21*

Figure 1.10: Exterior Wall Sections

Gravity Load Path

The exterior façade components, such as the brick or metal panels, rest on a steel angle at each level, and the gypsum board and insulation rests on the framed interior wall, which is attached to the brick or CMU. Therefore, the exterior wall loads act as a line load at each floor slab around the perimeter of the building. The load on the slab edge is then carried by the slab to the exterior columns, which then carry the load down to the foundations, followed by the soil.

Gravity Load Summary

All gravity loads, including dead, live, and snow, are summarized in table 1.1

Existing Gravity Loads			
Level	**Dead***	**Live**	**Snow**
Penthouse Roof	27 psf	30 psf	20 psf
12th Level	98 psf	100 psf	65 psf*
Typical Concrete Floor	105 psf	40 psf	N/A
Typical Steel Floor	75 psf	40 psf	N/A
Location	**Existing**	**Metal Panels**	**Brick**
Penthouse Roof	992 plf	443 plf	487 plf
*Value shown is maximum drift value and only occurs over portion of level next to penhouse walls.			

Table 1.1: Gravity Loads Summary

1.2.2 Lateral Loads

Wind Loads

Figure 1.11 shows a summary of the wind loads calculated for 11141 Georgia Ave according to ASCE 7-05: Section 6 using Method 2. Excel was utilized to program the equations for increased efficiency while working through the calculations. The spreadsheet output from excel showing the calculation process is included in Appendix B.

Seismic Loads

Figure 1.12 shows a summary of the seismic loads calculated for 11141 Georgia Ave according to ASCE 7-05: Chapters 11 and 12. Calculations of the seismic loads are provided in Appendix B.

Lateral Load Path

In the case of wind load, the load acts as a pressure in pounds per square foot. The façade carries the load to the backup wall and into the slab or floor system. From there it is distributed to the moment frames which carry the load down into the foundations and then the soil. The earthquake loads are a result of the building's own mass experiencing an acceleration caused by ground motion. The forces are again distributed into the lateral system and carried down to the ground.

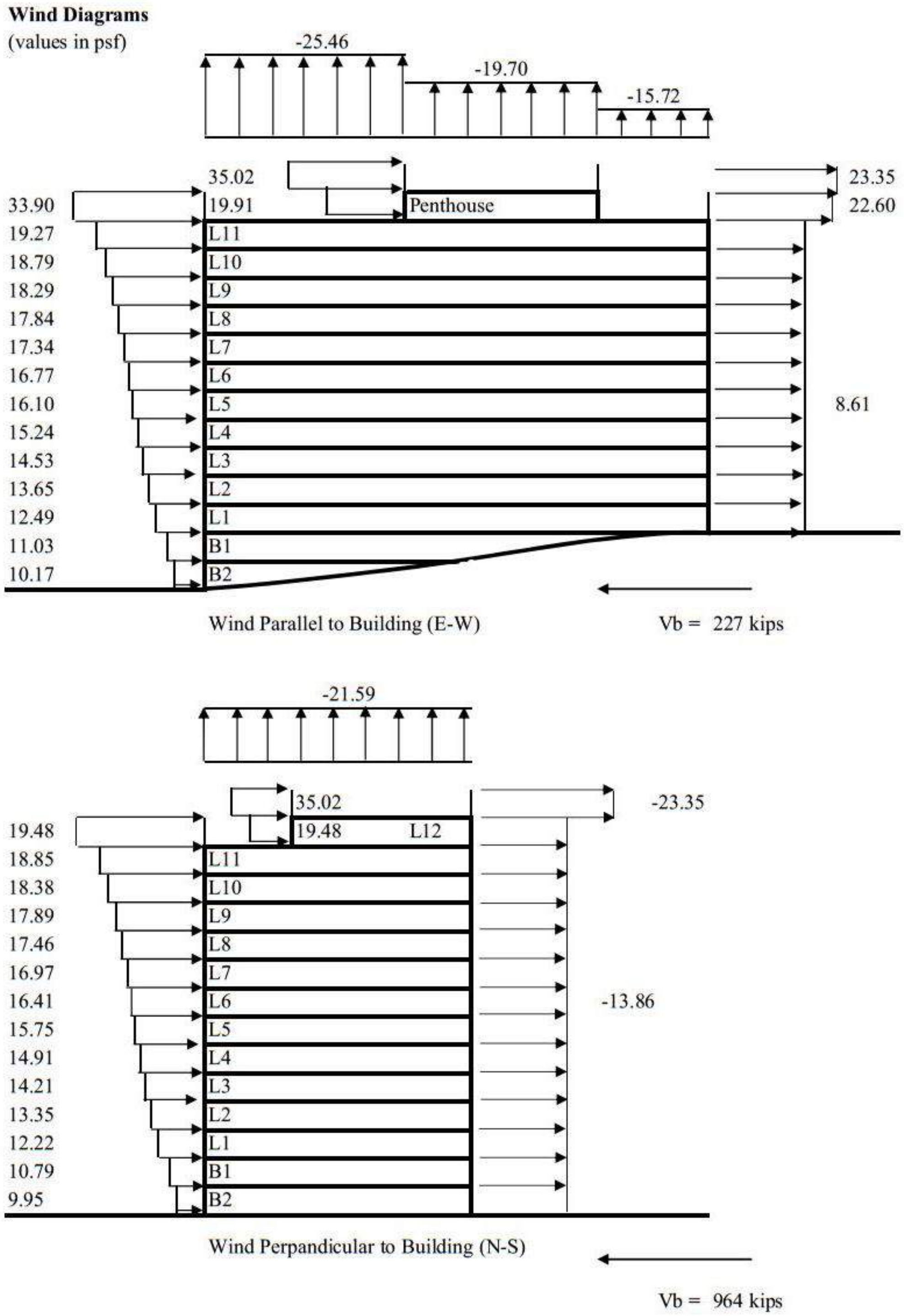


Figure 1.11: Wind Pressures Summary

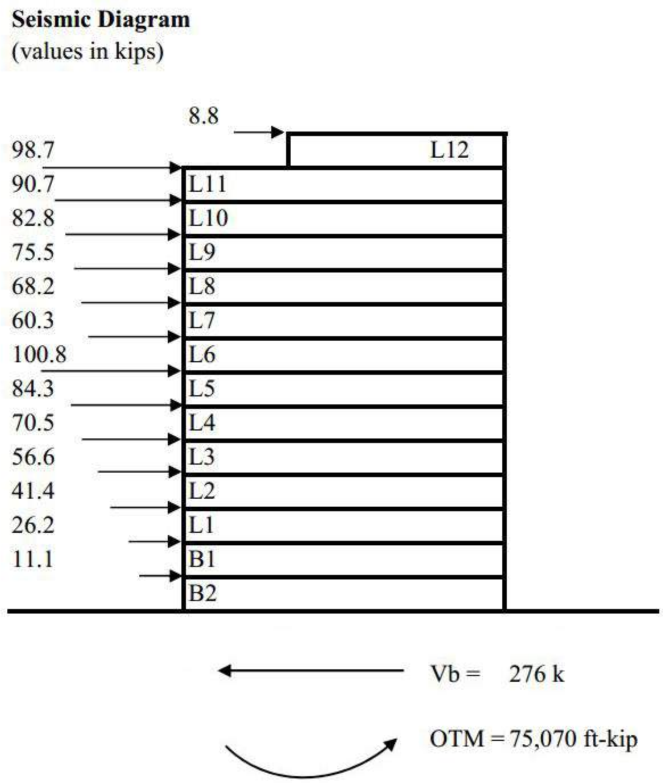


Figure 1.12: Summary of Seismic forces on building

1.3 Thesis Problem Statement

The newly completed seven story addition to 11141 Georgia Avenue is currently a steel framed system built over a 1960's concrete building. One of the benefits of the original design choice to retrofit an existing building for new use is that it is a much more affordable and sustainable alternative than tearing down the building and starting from new. Although building retrofit is not always a feasible option, in this case, the reuse was a good design alternative for 11141 Georgia Ave. Keeping the old building sacrificed some design freedom, but it also provided a sustainable design alternative while reducing construction costs and schedule time. Sustainability is an important factor moving forward in modern building practices, and because of this the proposed thesis work will maintain the original intent by looking at a sustainable light-weight framed addition alternative which also has the potential to be cost and schedule competitive.

To accomplish these goals, the work completed will include a study and analysis of an engineered wood structure as an alternative framing system redesign for the addition. Despite not currently meeting requirements of the International Building Code, there are several significant benefits to using wood; it is a sustainable and renewable material, it provides a lightweight alternative for the construction of a multi-story addition to an existing building, and it has the potential to be built on a faster schedule resulting in a reduction in schedule-related costs. This thesis will acknowledge the current code limitations on wood construction, however it will also study the feasibility of using wood as the primary structural material for the addition as well as whether or not it could plausibly meet the goals of the code.

1.3.1 Justification for Design Approach

Currently, heavy timber construction for a residential occupancy is limited to four stories in the US, and therefore the final wood re-design of 11141 Georgia Ave's addition will not be immediately applicable with regard to current codes. The current code limitations on heavy timber construction are founded on relative overall building fire-resistance and the concept of limiting a building's size and egress lengths based on the resistance of combustibility of the main structural material. However, there is research which has been carried out and which is ongoing that indicates that properly detailed and designed wood construction can meet fire-rating standards and life safety goals equal to steel and concrete construction for taller buildings than what is currently allowed by adopted codes in the US. Furthermore, other countries such as Canada and England have successfully built heavy timber buildings as tall as six and upwards of nine stories.

As previously mentioned, wood construction has several benefits which would make it a competitive alternative material not only for buildings taller than the four story limit, but specifically for 11141 Georgia Ave. First, the redesign in wood will be a lightweight alternative framing system. The existing steel addition floor structure is approximately 40 psf, while an initial estimate of wood framing weight is approximately 20 psf. Therefore, a heavy timber structure is an alternative that would put considerably less load and stress on the existing structure.

Wood buildings also show the potential to be built on quicker schedules. Since the structural elements in a heavy timber building are all prefabricated, the structure can be built very quickly, similarly to the schedule of a pre-cast concrete building. Therefore, a wood addition may be built more quickly than the current steel design, allowing a reduction in overall schedule and general conditions costs, as well as allowing the owner's use of the building earlier.

Finally, wood shows great potential to be a sustainable construction alternative. Certified forests in the US are using more sustainable forestry methods and are working to improve upon those methods. With the development of engineered glulam wood products, smaller trees can be used in constructing large structural members rather than cutting down old growth forests. While steel and concrete are produced from non-renewable resources, wood is the only renewable building material. Wood used in construction also has the ability to sequester carbon, effectively removing it from the atmosphere for the lifetime of the building and potentially longer depending how the wood is used at the end of the building's life. The study of a wood framing alternate will include a review of the sustainability benefits of wood construction and will discuss how a wood redesign of 11141 Georgia Ave's addition is a sustainable alternative design for the building. Because of the increasing need to reduce emissions and explore options for production practices which can be sustained moving forward, it is worthwhile to explore a wood design alternative in the context of a real building project.

1.3.2 Proposed Solution

The proposed new wood-framed building will include a design similar to the existing steel-framed addition. The 20'x20'bay size will be kept since a smaller bay will be beneficial for span when designing a wood framing system. Glulam structural elements will be used to achieve a heavy timber design, where the minimum beam or girder size and floor thickness for heavy timber is 6" wide by 10" deep and 4" thick respectively. Minimum column size for heavy timber is 8" by 8". The layout used will include glulam girders and an engineered structural panel product which spans between girders. Initial strength calculations predict that girders may be 16" deep or greater, and the largest columns will be about 15" square, noting that the final sizes must match available glulam sizes. The work for this thesis will include design of the primary structural elements for strength, deflection, and expected fire loadings.

A wood framed building will also require a different lateral system than the current steel moment frame system. Therefore, the elevator core shear walls, which currently extend only until the top of the original concrete building, will be carried through to the top floor. The existing CMU shear walls will be kept as they are currently. There will be CMU shear walls around the stairwells for improved fire and smoke safety in the egress route. The design work and research will look into the feasibility of wood shear walls elsewhere. A benefit of continuing the shear walls up in wood wherever possible is that it will add significantly less weight than continuing them in CMU from the concrete portion of the building, thus reducing the increased loads and foundation size due to the change in the lateral system for the addition.

1.3.3 Solution Method

Structural Depth

The design of the wood floor system for gravity loads will be based on design values from the CLT Handbook and the Engineered Wood Association design guides, as well as information from AE 401: Design of Steel and Wood Structures, BE 462: Design of Wood Structures, and any other structural wood design resources. The CMU shear wall design option will be based on the Masonry Building Code. The wood shear wall design alternative will use information from existing research on the topic as well as available design guides. Modeling of the structure will be completed in ETABS modeling software. The research methods for this thesis work will also include seeking the advice of professors as well as professionals who are currently researching the use of wood in tall buildings and those designing and implementing tall wood buildings.

Breadth Topics

Both breadth topic selections are a result of the selection of wood as a framing alternate and the effects of that decision on cost, schedule, and mechanical equipment. The breadth topics include a construction management and mechanical breadth.

Construction Management

In the construction management breadth, cost and schedule analysis will both be completed for the existing and new addition. The focus of the cost analysis will be specifically on the existing and new additions themselves, but will take into account any significant changes to foundations, renovations, and general conditions costs due to scheduling. The schedule analysis will determine scheduling differences between the designs and identify any significant changes. The goal of the cost and schedule analysis in this breadth is to determine approximately if the wood design alternative is feasibly and economically competitive with its equivalent mid-rise steel addition in the case of 11141 Georgia Avenue.

Mechanical

Since no concealed spaces are allowed in heavy timber construction, the ductwork, wiring, and other mechanical systems normally hidden above a drop ceiling will be exposed. This is an important difference between the proposed wood redesign and the existing steel structure with drop ceilings. Therefore, it is important for the mechanical equipment to be arranged aesthetically such that the apartments are just as appealing as in typical competing apartment buildings. The mechanical breadth will determine the changes that need to be made for aesthetic purposes and will look in detail at one instance of an equipment change in a typical apartment.

2 Heavy Timber Construction

2.1 Introduction

Heavy timber is a construction type that uses engineered wood products as the main structural elements of a structure. Heavy timber takes advantage of relatively recent innovations to create larger structural elements out of smaller lumber sizes. Currently, US building codes limit wood construction to about four stories due to the combustible nature of wood. However, timber is beginning to be considered over steel or concrete construction for its sustainability benefits, and potentially quicker schedule time and lower costs. Therefore, research is currently being conducted, especially in Canada and Europe to develop heavy timber as a construction alternative for taller buildings with equivalent fire safety measures and details. The following chapter provides background information on the use heavy timber in taller buildings, including a summary of current research and research that will still be required before taller timber buildings could be seriously considered by code writing bodies.

2.2 Heavy Timber Defined

There are currently two main types of wood construction; heavy timber and light frame construction, which are compared in figure 2.1. Light frame construction involves the use of smaller dimension lumber such as 2×4, 2×6, etc. to build up floor and wall framing systems, while heavy timber consists of large wood elements, such that floor decking is at least 3" thick, beams are 6"×10" or greater, and columns are at least 8"×8". This thesis focuses on engineered wood products since it is more economical and sustainable to create larger structural elements from smaller-cut trees rather than trying to find large pieces or sawn lumber or damaging old growth forests. (Green and Karsh, 2012)

Within the engineering wood products which make up heavy timber elements, there are three main types; cross laminated timber (CLT), glulam, and laminated veneer lumber (LVL). CLT includes several layers of dimensional lumber, with the layers perpendicular to each other and structurally glued together, as shown in figure 2.2 (a). Glulam is similar to CLT in that layers of dimensional lumber are glued together. However, the layers in glulam are all parallel to each other as shown in figure 2.2 (b). (FPInnovations and Council, 2013) Finally, LVL is made from peeled

(a) *CLT Building. Source: woodwindowstoday.blogspot.com*

(b) *Light-framed home. Source: arupconnect.com*

Figure 2.1: Heavy Timber versus Light Frame Wood Buildings Under Construction

veneer layers from a log which are then structurally glued together with the grains perpendicular from layer to layer. (Green and Karsh, 2012) CLT will be the structural product used in this thesis for the floor system because it is available in a panel product with a sufficiently wide dimension, has better dimensional stability due to the nature of cross lamination, and is more suitable for connection design than if glulam were used to create a panel product. Glulam will be used for girders and columns because of its availability for this purpose and its strength.

(a) *Cross Laminated Timber Panel. Source: archiexpo.com*

(b) *Glulam Beams. Source: timberfirst.wordpress.com*

Figure 2.2: Different Types of Engineered Wood Products

2.2.1 Benefits of Heavy Timber

Heavy Timber is not common for use in buildings taller than about six stories, but a variety of design firms, other countries such as England and Canada, research institutions, and manufacturers are increasingly interested in heavy timber because of its benefits. An engineered wood product may be chosen alternatively to another material for a number of reasons; it is a sustainable and renewable construction material, it can be cost and schedule competitive, elements can be pre-fabricated, it is a lightweight material which reduces required foundation sizes, and it can provide interesting design freedom.

Many of these benefits will be further studied in this thesis. The potential sustainability features of using wood will be covered in this chapter. The redesigned CLT addition will be analyzed for cost and schedule, and it will be compared to the existing addition to determine the effects of the wood system on cost and schedule for 11141 Georgia Ave specifically.

2.2.2 Challenges of Heavy Timber

Despite the benefits of using engineered wood products, there are several challenges to its use, especially in taller buildings. Challenges include fire-safety, public perception, code limitations, constructability knowledge, and several more.

Of these challenges, fire-safety and code limitations will be specifically addressed in this thesis. The addition redesign will be studied for its limitations with regards to the code, however it will also attempt to address the goals of the International Building Code. Furthermore, existing research on the fire-safety of taller heavy timber buildings will be reviewed.

2.3 Environmental Impact

One of the major reasons heavy timber is being explored as an alternate material for taller buildings is that it is a sustainable option. Although it would never completely replace steel or concrete, wood has a lower carbon footprint and is less energy intensive to produce than both steel and concrete. Therefore, it is worth exploring the feasibility of wood in taller buildings as a potential sustainable alternative for a greater variety of buildings.

2.3.1 Effects on Climate Change

Today, about 50 percent of the world population lives in cities, and it is expected that even higher percentages of the population will live in cities in the future (Green and Karsh, 2012). Furthermore, people spend most of their time in buildings. Because of this, it should be no surprise that the construction and use of buildings accounts for a large portion of energy consumption and greenhouse gas emission. Figure 2.3 shows that construction, electricity, and heating make up almost 40 percent of energy consumption. Therefore, any improvements in the sustainability of the construction industry will have a significant effect.

There are a two main approaches to improving sustainability and preventing climate change; reduce greenhouse emissions, or store excess greenhouse gasses. Building with wood contributes to both of these approaches. The forests grown to produce wood become carbon sinks, and the carbon stored in a tree will continue to be stored as it is used in a building. The manufacturing process of engineered wood products is also much less energy intensive than steel or concrete. Most importantly, while steel and concrete are produced from non-renewable resources, wood is a renewable resource. As long as forests are well managed and harvested sustainably, new wood material can continue to grow indefinitely (Green and Karsh, 2012). In the US, both sustainable forestry and clearcutting are practiced, but the building industry generally uses FSC-certified products, which must meet requirements for sustainable forestry practices. (Edward Allen, 2009)

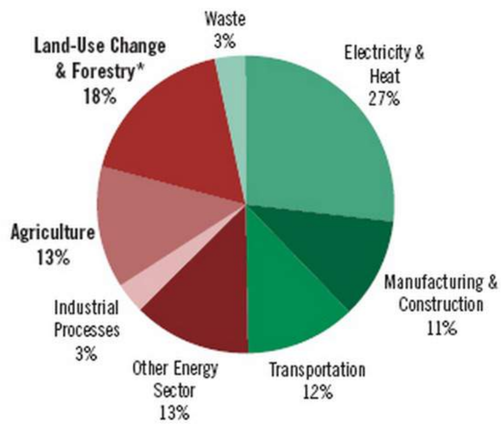


Figure 2.3: Percentage of energy consumption by sector. Source: e-education.psu.edu

The production, use, and growth of wood is ultimately a sustainable cycle rather than a one-way street, as shown in figure 2.4. This is not to say that there are not any effects on habitats and forests, but when done correctly, the harvested areas can recover quickly such that the overall long term habitat is not negatively effected.

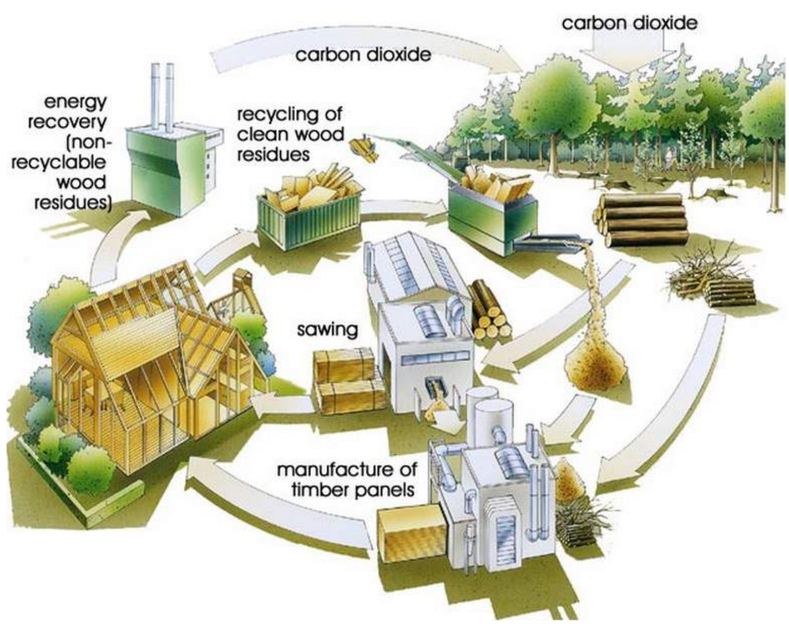


Figure 2.4: Production and Growth Cycle of Wood. Source: machielsbuildingsolutions.be

2.3.2 Life Cycle Analysis

When looking at the environmental effects of a buildings, the entire life of the building must be taken into account, including the production of its materials, its construction, the lifetime and durability of the building, and finally the demolition and disposal or reuse of its materials.

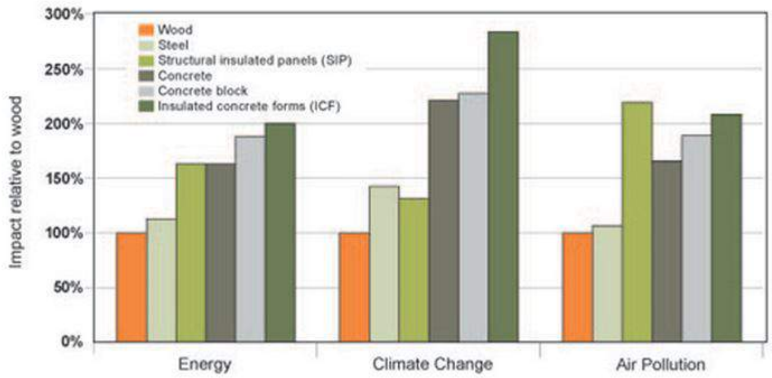


Figure 2.5: Environmental Impact Comparison Between Wood and Other Construction Materials. Source: naturallywood.com

A life cycle analysis of a heavy timber building will include wood growth and carbon storage, energy consumption during the harvesting process, shipping and manufacturing, delivery to site, construction, building lifespan, and end of life use of wood material. Whether the wood is reclaimed and reused, sent to a landfill, or used as fuel will significantly affect the net greenhouse gas balances of the full life cycle of the building. (Börjesson and Gustavsson, 2000)

A criticism of the sustainable nature of heavy timber is that the carbon it sequesters will only delay it's effect on carbon emissions as the sequestered carbon could potentially be released back into the atmosphere at the end of a building's life if the material is burned or sent to a landfill. This is an important point, and thus an effort should be made to design CLT structures such that the material may be reclaimed and reused at the end the building life. Fortunately, methods to accomplish this are being developed to achieve such a goal. The only case in which the net effect is a positive emission of greenhouse gasses at the end of the life span is when the lumber goes to a landfill and a large portion is decomposed (Börjesson and Gustavsson, 2000).

At the very least, the delay of carbon dioxide release will buy upward of a 100 years of time to make improvements to sustainability in the construction industry and other sectors, and the energy used during manufacturing is still lower than steel or concrete. Ultimately, life cycle analysis of wood structures determines that the net effect on greenhouse gasses over the life of a wood building will generally result in a reduction in emissions compared to steel and concrete construction. (Gustavsson and Sathre, 2006) In the residential sector, it is estimated that wood frame homes versus steel or concrete alternatives cause 20-50 percent less emissions during construction (Upton et al., 2008) Therefore, the use of heavy timber in taller buildings should not be dismissed as it has clear potential to be a sustainable alternative construction material in a greater variety of cases.

2.4 Fire-Safety

Fire-safety is a key consideration in modern building construction, in which the four main goals of fire-safety include protection of life, protection of building, protection of contents, and continuity of operation, with life safety ranking as the most important fire performance goal (Walter T. Grondzik, 2010). Heavy timber buildings are constructed out of wood, which is a combustible material, and therefore these buildings must be designed differently from steel and concrete buildings to achieve the same level of fire safety.

2.4.1 Heavy Timber Fire Resistance

In any building constructed, there will be both passive and active methods of fire-protection. Active methods involved the use of systems such as sprinklers and smoke detectors. This serves as the first line defense in most buildings. Passive methods serve as the last resort as far as keeping a building fire under control, and they take advantage of the inherent fire-resistance properties of the materials used in construction. This section will provide an overview of the passive fire-protection in heavy timber.

Passive fire-protection is dependent on the building materials themselves, and therefore will vary between construction material types. In steel construction, additional fire-resistant materials must be added to protect the steel since steel loses strength quickly in high temperatures. In contrast, concrete itself is resistant to fire and therefore provides its own fire protection. Wood is somewhat similar to steel in that it must be protected from fire. However, heavy timber bears similarity to concrete as its fire protection is inherent in the material itself. This behavior is shown in figure 2.6. When wood burns, it forms a char layer which then acts as an insulating layer for the rest of the wood. The next layer inward is a heated layer which doesn't burn when protected by the char, but which undergoes thermal decomposition. This second layer is called the pyrolysis zone. The innermost layer remains protected with all or most of its structural capacity. The wood elements in light frame construction are too small to retain an unheated wood inner layer, but engineered heavy timber products are large enough to retain a protected inner structural section. (Gerard et al., 2013)

Some of the main approaches to making taller heavy timber buildings safer in a fire include designing a sacrificial layer of wood to protect the structural wood required for a reduced load condition during a fire, redundant sprinkler and alarm systems, gypsum board encapsulation, compartmentalization design to prevent spread of smoke and fire, and use of non-combustible material for egress stairwells.

When designing heavy timber structural elements, the known char rate of wood can be used to design in a sacrificial layer such that the structural core of normal wood can continue to support the predicted loads during a fire. Initial research has shown that this method can be used to obtain a two-hour or greater fire-rating. Furthermore, the thickness of a floor slab for example will still most likely be governed by other existing strength or deflection requirements, such that the design of a sacrificial layer during reduced fire load conditions would tend to add insignificant additional thickness for the final design. (Green and Karsh, 2012)

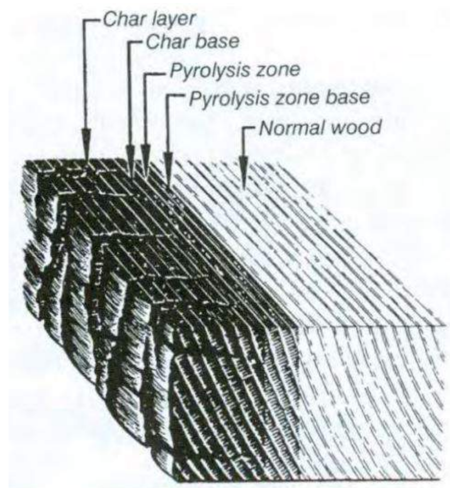


Figure 2.6: Layers in Burning CLT Panel. Source: FPRF

2.4.2 Code With Respect to Fire Safety

The International Building Code determines maximum allowable heights and areas based on building occupancy type and construction type. There are five construction types, which generally define how fire-resistant a building must be in order to fall within that category. Heavy timber and engineered wood are in category IV, which currently restricts the maximum number of stories for any heavy timber building to six stories. For residential occupancies, the number of stories is limited to four. The limit on the number of stories to achieve these goals varies from country to country, as shown in figure 2.7. Category IV includes heavy timber structures, with the important defining characteristic being that the main structural materials are combustible.

Country	Applicable Building Code	Maximum # of Stories	
		Sprinklered	Non-Sprinklered
Australia	2013 Building Code of Australia (BCA)	3	3
Austria	Austrian Building Codes	8 (*72 feet [22m])	4
Canada	2010 National Building Code of Canada (NBCC)	4	3
Germany	2012 Federal Building Code	8 (*59 feet [18m])	5
Sweden	2013 Planning and Building Act	8	2
United Kingdom	2010 Building Regulations	8	6
United States	2013 International Building Code (IBC)	5**	4**
	2012 National Fire Protection Association (NFPA) 5000	6**	5**

*Indicates a height limit in addition to a maximum story limit
**Number of Heavy Timber stories permitted

Figure 2.7: Review of Max Code story limit in various countries. Source: Fire Protection Research Foundation (Gerard et al., 2013)

Fire Safety Goals

The International Building Code limits the number of stories in a building with the idea that fire-safety is affected by the type of construction. The intent of the code is to limit area, height, and number of stories as a method to improve the level of a building's fire-safety with increasing building structure combustibility. However, the fire safety goals which are the intent behind the code choices, are important considerations. Proponents of taller wood buildings argue that as long as the goals of the code with regard to fire-safety are met, then buildings should be allowed to be constructed taller. As mentioned before, the main fire safety goals include protection of life, protection of building, protection of contents, and continuity of operation. (Walter T. Grondzik, 2010)

Life safety is regarded as the most important of goals, and thus the code includes a lot of provisions aimed at protecting the occupants while they are leaving the building during an emergency. Some of the unique challenges a heavy timber building poses towards meeting this goal include smoke control due to the combustible nature of the structure itself, fire performance of egress routes which may or may not include wood shear walls around a core, and the typical enclosed spaces found in steel and concrete buildings which would allow the quick and quiet spread of fire and smoke. Life safety was the primary goal under consideration when making fire performance-related design decisions throughout this thesis.

Protection of the building is another goal which has unique challenges for a heavy timber building as opposed to steel and concrete buildings. If the structure is ignited, it will combust, unlike concrete which is fire-proof, and steel, which just loses strength. However, since heavy timber is difficult to ignite due to its mass, small fires which can be suppressed by sprinkler systems will not pose significant damage threats. Once a fire becomes large enough in a building that several structural members require complete replacement, it is likely that life safety and preventing a progressive collapse will become the main goal. Most of the time, the mechanical and sprinkler systems will prevent large fires, thus limiting damage to a level which can be easily repaired.

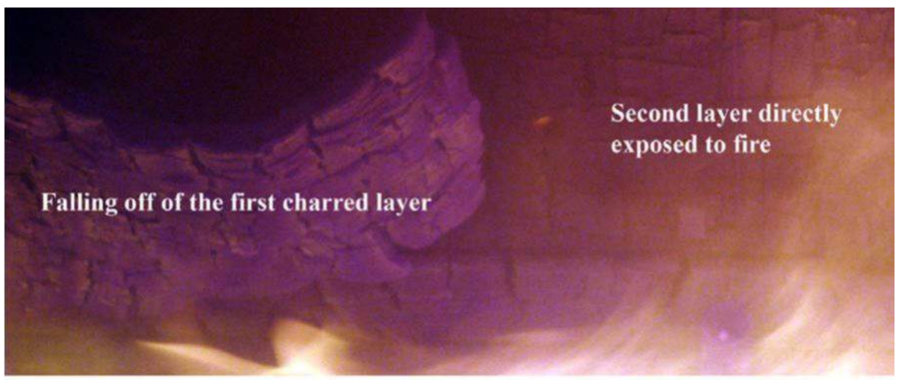
The goal of protecting contents will most likely be similar in a heavy timber building as in other buildings. The method to protect the objects within a building will primarily be through the sprinkler systems. As long as the sprinklers are functioning properly, they can be extremely effective at suppressing, and even putting out, a fire just as it is starting. The final goal continuity of operation, and as with the goal of protecting the building contents, the sprinkler system will effectively suppress a fire, thus limiting its effect to a single room. This system, along with a fire alarm system, will typically prevent serious fires from occurring. Thus, although a heavy timber building behaves differently than steel or concrete, the fire protection methods in place to meet the goal of operational continuity are the same.

2.4.3 Topics requiring further study and research

Although heavy timber is fairly well understood with regards to how it burns and behaves in a fire, there are other details which pose fire-safety challenges and which require further study before taller wood buildings can be considered and eventually accepted by the code. Some of the main areas that require further research include CLT delamination and char fall-off, penetrations for services, timber façades, and protection of egress routes. (Gerard et al., 2013)

CLT Delamination and Char Fall-Off

CLT delamination and char fall-off occurs uniquely in CLT engineered wood products. Delamination may occur once the char layer reaches an interface between layers, and the car could fall off in pieces, causing the panel to burn more quickly, and contribute more burning load to a compartment fire. Figure 2.8 shows a piece of a lamination falling during fire testing. This behavior affects the overall fire-rating of a floor or wall assembly and is being studied to determine conditions under which char fall-off may occur. (Gerard et al., 2013)



Falling off of charred layers (Frangi et. al, 2009)

Figure 2.8: Falling delaminated lamination during CLT fire testing. (Frangi and Jobstl, 2009)

Service Penetrations

Openings for mechanical, electrical, and other service equipment must be just as fire-resistant as the floor or wall assembly to prevent the spread of fire to other spaces. This is an approach to keeping the fire relatively contained within a single compartment of the building for as long as possible. In most cases, a fire-rated caulk or other firestop system may be used to fire-proof the penetrations for services. However, in heavy timber buildings, since the wood panel material in a floor or wall is combustible, the wood surrounding the opening and its seal will char, thereby compromising the area around the openings and potentially allowing smoke through the charred areas. A potential solution is to extend the firestop system material into the panel to protect an extended circumference of the opening. This method is promising, however it poses constructability challenges and still requires further fire-testing and research.

A final challenge for openings is to educate building owners about fire concerns related to creating new openings to move or add services in the future. It is possible that during the lifetime of a building, an owner may, for example, move outlet receptacles and create a new opening which is not properly fire-protected. Not only is the education of the initial owner important, but the transfer of information to educate any future building owners is important as well. (Gerard et al., 2013)

Façades or Exterior Walls

If the building is completely constructed out of timber, including timber exterior walls and/or façades, then fire spread via the exterior is a concern. If the façade catches fire, then the flames can spread to upper stories through the exterior of the building. Some options include using fire-retardants, however the use of timber in façades is fairly new and not well understood. Therefore, the redesign of 11141 Georgia Ave will not include a timber façade, but will keep the original brick façade. (Gerard et al., 2013)

Protection of Egress Routes

Finally, one of the main goals of fire-safety is to provide occupants with adequate time and an available egress route to exit the building safely. The stairwells in a multi-story building are critical for egress, and therefore should have higher standards of protection. This is related to the four story limit for heavy timbers, since four stories is the maximum height at which a ladder rescue is feasible. Timber panel shear walls are potentially structurally feasible, however, when those shear walls exist in a stairwell core, smoke production is a concern for egress. Since wood is combustible, as soon as it is exposed to fire within a stair well, it will burn and produce smoke. In the redesign for Georgia Ave, wood shear walls will be use where possible since this is the prevailing practice for heavy timber residential construction in the US. However, since the timber portion of the building is well beyond the reach of ladder rescue, a masonry or concrete shear wall will be used around the stairwells to increase the building's egress safety. (Gerard et al., 2013)

2.5 Additional Considerations

There are other considerations which come with choosing heavy timber and engineered products for taller buildings. This includes vibration performance, sound insulation, building envelope detailing, keeping wood panels relatively well protected from extended water exposure during construction, and fire-safety during construction. (FPInnovations and Council, 2013) These consideration will be discussed briefly here, but most will not be fully explored in the redesign work of the 11141 Georgia Ave addition.

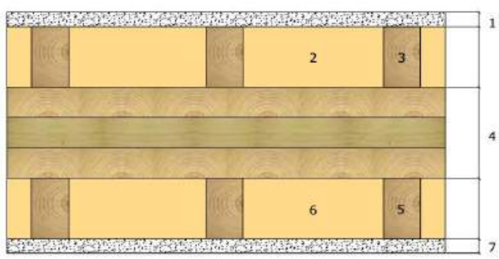
2.5.1 Vibration Performance

Vibration performance should be considered in CLT buildings because they tend to have a critical damping ratio less than typical lightweight wood joist floors. Therefore, it is more difficult to control vibrations in CLT floors. Wood in general is more susceptible to vibrations than other materials due to its lightweight nature. However, there is a design method presented in the CLT Handbook which uses a CLT floor's mass to control the vibration response. In a residential occupancy such as 11141 Georiga Ave, vibration performance is not as critical because people tend not to do work that requires them to focus on a computer for awhile such as they would at work in an office.

2.5.2 Sound Insulation

Sound insulation is an important factor in building design, especially for residential occupancies as in the case of 11141 Georiga Ave. Poor sound insulation can result in unpleasant living conditions and distractions to occupants. The IBC requires a minimum Sound Transmission Class (STC) of 50 for walls and floors and an Impact Insulation Class (IIC) of 50 for Floors. For a 5-layer CLT panel, the STC is 39, which could be easily improved to meet code by adding drywall or other methods. The IIC of a 5-layer panel is 24, which is fairly poor. Providing a floor system which meets IIC requirements will likely require some type of floor topping. (FPInnovations and Council, 2013)

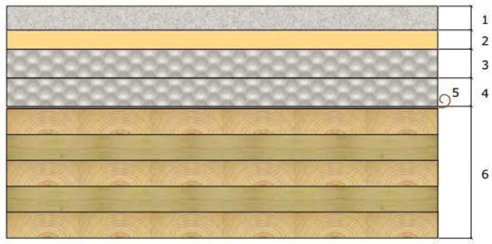
Ultimately, wood floors and walls create the challenge of meeting acoustic-related code requirements. Since the wood products themselves don't provide the required STC and IIC values, acoustic-insulating materials must be added to meet code. Therefore, if a thick floor topping is required, the floor to ceiling heights will be affected, and possibly the height of the structure as well, if the floor to floor height needs to be increased. Figure 2.9 shows a potential wall assembly which meets code, while figure 2.10 shows a potential floor assembly which meets code.



Assembly Description from Top to Bottom (6.3)		STC
1	Gypsum board of 5/8 in. (15 mm)	58 or above depending on CLT thickness
2	Mineral wool of about 2.36 in. (~ 60 mm)	
3	Lumber studs of 2 in. x 3 in. (38 mm x 63 mm) at least 16 in. (400 mm) o.c.	
4	3-layer CLT panel of 3 3/4 in. ~ 4 1/2 in. (95 mm ~ 115 mm)	
5	Mineral wool of about 2.36 in. (~ 60 mm)	
6	Lumber studs of 2 in. x 3 in. (38 mm x 63 mm) at least 16 in. (400 mm) o.c., attached to CLT and gypsum boards	
7	Gypsum board of 5/8 in. (15 mm)	

Figure 2.9: Acoustic Wall Assembly. Source: CLT Handbook.

The floor assembly requires an additional 4" topping below the floor covering. This is significantly thick as the original floor finish topping in the addition was only about an inch thick, and will be taken into account in determining the final floor to ceiling heights resulting from the gravity system redesign.



	Assembly Description from Top to Bottom (7.1)	STC	IIC
1	Gypsum fiberboard FERMACELL of 1.0 in. (25 mm)	62	59
2	Sub-floor ISOVER EP3 of 0.79 in. (20 mm)		
3	Honeycomb acoustic infill FERMACELL of 1.18 in. (30 mm)		
4	Honeycomb acoustic infill FERMACELL of 1.18 in. (30 mm)		
5	Kraft paper underlayment		
6	5-layer CLT panel of 5 5/16 in. (135 mm)		

Figure 2.10: Acoustic Floor Assembly. Source: CLT Handbook.

2.5.3 Envelope Design

All building envelopes must keep the structure and interior of a building dry. Therefore, the requirements for wood are not much different from steel or concrete despite public perception. Although wood rots when exposed to moisture for lengths of time, concrete and steel are degraded by water as well. There are however, some envelope considers unique to wood design. Since wood is organic, a main issue is termites, as they can eat out the inside of a wooden element without leaving many exterior signs of damage. A method to protect a building from termites is to build a concrete podium as a base above ground level. Since the 11141 Georgia Ave addition is built above a multi-story addition, termites will not be a problem for the redesigned wood addition. A benefit of using wood as part of the exterior wall is that it has low thermal conductivity, and therefore provides some amount of insulation value for an enclosure, adding to the energy efficiency of the building. (FPInnovations and Council, 2013)

2.5.4 Construction Challenges

All materials have special considerations during a building's construction phase. This is because the building is unfinished and does not have all of the protection or support systems that complete its design. Two challenges unique to heavy timber construction, or wood construction in general, include protection from moisture and fire.

Since CLT and other wood products do not do well when exposed to moisture for an extended time, it is important to protect the wood from moisture during construction before the enclosure has been built. A potential method of protection is to erect a temporary tarping system, which would most likely be attached to the scaffolding around and over the building (Figure 2.11). This will serve as a temporary envelope to protect the wood during construction. (FPInnovations and Council, 2013)

A review of heavy timber fire incidents from the Fire Protection Research Foundation indicates that many of the most serious fires in heavy timber buildings occurred during construction or

Figure 2.11: Moisture Protection of CLT During Construction. Source: CLT Handbook

renovations. While the building is under construction, it does not yet have sprinklers, alarm systems, protective gypsum board, or any of the other methods used for fire protection. Therefore, the building is at more risk to burn down completely should a portion be ignited. Additionally, the tools used during construction and renovation produce a lot of heat, and therefore care must be taken to prevent ignition caused by construction activity. Finally, if there are other buildings close by, a fire during construction could put surrounding structures at risk as well. In this case, additional precautions must be taken to prevent fire during construction. (Gerard et al., 2013)

2.6 Literature Review

The following section reviews current literature and research towards the feasibility of taller wood construction as a sustainable, fire-safe, and cost competitive structural system alternative.

CLT Handbook (US Edition)

Cross laminated timber is a product which is being used for taller wood construction in countries such as Canada and England. Within the last five years, a few CLT manufacturers have started up in the US, and as a result, a CLT Handbook has been made available to aid with the design of CLT buildings. The Handbook was published in 2013 through the combined efforts of FPInnovations and the Binational Softwood Lumber Council.

The handbook includes information on design for gravity and lateral loads, connections, vibration and sound insulation performance, construction management considerations, fire-performance, and more. Material design values provided in the CLT Handbook were used in the structural system redesign. (FPInnovations and Council, 2013)

Timber Tower Research Project

Skidmore, Owings, and Merrill completed a conceptual project in May 2013 in which they developed a structural system for taller timber buildings and applied the system to a prototype building design based on an existing concrete building. The goal of the project was to create a building that was as sustainable as possible while also being cost competitive with other types of modern construction. The scope of the project includes the structural design, architectural design, building services design, an embodied carbon analysis, recommendations, and more.

The benchmark building chosen for comparison was the Dewitt-Chestnut Apartment building located in Chicago. The structure is a concrete flat plate system with gravity columns and a tube frame around the perimeter for the lateral system. The structural system used was a "concrete jointed timber frame." This system includes the use of mass timber products for the main elements such as floors, columns, and shear walls. The connections are made through the use of reinforcement and concrete joints. There are also perimeter concrete beams, with the floors spanning from the core to the perimeter beams. Some structural issues came up during the design process. Since the wood prototype is much lighter than the concrete benchmark, net uplift due to wind loading became an issue. Furthermore, the lateral system required more lateral elements in the prototype to achieve the same stiffness as in the benchmark. SOM designed the prototype to address these issues.

The design project also looked at design for fire and determined that the most practical approach would be to follow a performance based design method since taller timber buildings of this construction type do not quite fit within the framework of the code. SOM decided that the project should meet the goals of the code, which include protecting the safety of occupants and fire fighters during a fire as well as preventing a progressive collapse or major failure of the structure. Many principles for fire design were created, including the concept of using fewer larger structural elements in the design rather than multiple smaller elements. For example, a thicker floor without ribs would be preferred over a thin floor with beams. Ultimately, SOM recommends completing a full building fire performance analysis for final design rather than just looking at assemblies fire ratings alone. (SOM, 2013)

The Case for Tall Wood Buildings

A prototype building design approximately 70 feet square was created in order to investigate the feasibility of wood in taller buildings. Structurally, the prototype building used a "strong column/weak beam approach," where the shear walls serve as the strong column and steel W-shapes served as the weak beam, adding ductility to the structure. In buildings 12 stories or less, lateral forces could be adequately resisted by a wood shear wall core alone. As the building gets taller, additional shear walls and then perimeter moment frames are required when a wood lateral system is still used. Therefore, as a wood building gets taller and keeps wood as the lateral system, the overall freedom of architectural design may become greatly reduced.

The report also looked at fire performance and determined that for the prototype building, the char method alone could be used as the passive fire protection. In this case, the design was controlled by deflections and vibration performance rather than by fire performance. The report acknowledges however that most early CLT buildings will use the encapsulation method until more research is done. Other topics regarding the feasibility of taller wood buildings were explored, and the report ended with several recommended studies which would further the research. (Green and Karsh, 2012)

Fire Safety Challenges of Tall Wood Buildings

In 2013, the Fire Protection Research Foundation compiled current knowledge on the topic of tall wood building fire safety and identified gaps in knowledge which require further research and study.

A review of major fire incidents revealed that the worst fires in wood buildings occurred in a few typical cases. The highest risk generally appeared in light frame wood buildings. There is a higher risk of fire during construction prior to the installation of fire protection systems. The construction phase also tends to pose a risk due to the tools and equipment used in the process. Also, fires occurred often in buildings with concealed spaces, which allowed routes for rapid fire and smoke spread throughout a building. In heavy timber, large fires have been less likely to happen because of the inherent fire resistance of the large sizes and surface area. Ultimately, lessons learned previously, and knowledge of fire behavior can be implemented to protect taller wood buildings against fire.

The study identified some gaps in knowledge of taller wood buildings related to; effects of structural loading on fire performance, full system fire testing, CLT delamination and char fall-off, pipe and service penetrations, fire spread through timber façades, and protection of egress routes. (Gerard et al., 2013)

Greenhouse Gas Balances in Building Construction

Pal Borjesson and Leif Gustavsson co-published a paper in 1999 which compared green house gas emissions of multi-story concrete and wood buildings in both a life cycle analysis and a land-use analysis. The study found that the initial energy used to manufacture the building materials is approximately 60-80 percent higher in a concrete building than a wood building. The net greenhouse gas emissions (GHG) in a wood building were determined to vary greatly depending on end of life use. If the wood is reused, then the net GHG emissions are negative. However, if the wood goes to a landfill, it will produce biogasses such as methane as it decomposes, thus causing net positive GHG emissions. If those biogasses are used to replace fossil fuels, however, then the GHG emissions are negligible.

The concrete analysis in the paper considered the absorption of carbon back into the concrete over its lifetime through the carbonisation process. When this occurs, the net GHG of a multi-story concrete structure is approximately the same as when the wood at the end of the building life decomposes in a landfill. The paper came to the conclusion that the entire life cycle of a building plays an important role in the overall sustainability of a building. Wood is most beneficial at the beginning of the cycle during the manufacturing process, but requires care at the end of life to make sure it is disposed of or reused in a sustainable manner. (Börjesson and Gustavsson, 2000)

The Greenhouse Gas and Energy Impacts of Using Wood Instead of Alternatives

In 2007, Brad Upton, Reid Miner, Mike Spinney, and Linda Heath published a paper on the GHG and energy impacts of using wood versus alternatives in US residential construction. The study found that houses built using wood require about 15 percent less energy than concrete or steel homes for non-heating and cooling energy requirements. When looking at a 100 year period of time, the wood homes were found to perform even better, with a 20 to 50 percent decrease in emissions. The important difference is that steel and concrete materials required the use of fossil fuels in the manufacturing process, which account for much higher greenhouse gas emissions. (Upton et al., 2008)

Developing Hybrid Timber Construction For Sustainable Tall Buildings

Carsten Hein wrote an article for the Council of Tall Buildings and Urban Habitat (CTBUH) Journal in 2014 about hybrid timber construction in sustainable tall buildings. The article recognizes that wood has great potential for use in taller buildings, and discusses its use in composite system design. The article reviewed a proposed concept for a 20-story tower using a timber-concrete-composite (TCC) floor system. An 8-story prototype using the TCC floor system was built and tested in 2011. The floor uses a concrete slab with glulam beams. A sound-absorbing floor was included in the project to test acoustical properties. The floor met acoustical requirements, but may not have been the most cost-effective solution.

Fire testing was carried out and the floor system achieved 90 minutes of fire resistance in a test by the PAVUS Test Institute in the Czech Republic. The building design also included a concrete core for lateral stability and to provide a main fire egress route. A cost analysis was performed, and the project was about 105 to 110 percent the cost of a typical office building. The project also determined the sustainability benefits of the building and found that the concept reduced the building's embodied carbon by about 50 percent. Ultimately, it was found that a hybrid timber building is feasible and can meet various performance requirements. (Hein, 2014)

2.7 Application to 11141 Georgia Ave

The selection of heavy timber for the redesign of the 11141 Georgia Ave addition requires a variety of considerations related to building outside the code. The code limits residential heavy timber to 4 stories, but the addition is 7 stories and located on top of an existing concrete building. The fire-safety goals behind that limitation will be considered in the redesign.

Structural

Factors which will primarily affect the structural redesign include increasing structural element sizes for better performance during fire, the wall type surrounding stairwell egress routes, and floor and wall system design for sound insulation.

The design of the floor gravity system elements, in addition to being designed for typical loads such as flexural, and deflections, and will be designed for fire conditions. As mentioned previously, a sacrificial layer can be designed such that the remaining protected wood is still able to carry the required loads for the duration of occupant egress. Also, a topping may be required for sound insulation, causing an increase in design loads as well as affect the floor to ceiling height. These will both be considered in the addition redesign.

Construction Management

Any heavy timber building will have many effects on the construction management process. The redesign work will primarily consider schedule and cost differences between the existing addition and the redesign. Several research studies have concluded that CLT construction can be cost and schedule competitive with other methods of construction. This thesis work will act as a feasibility study and determine whether or not in the case of the Georgia Ave addition CLT construction is cost and schedule competitive with the existing addition.

Mechanical

Many of the more severe fires in heavy timber structures have occurred when concealed spaces exist. A fire can spread quickly and quietly through concealed spaces, bypassing the sprinkler and alarm systems that would have prevented damage in the event that the fire started in an occupied space. Therefore, modern heavy timber buildings are not permitted to have concealed spaces, and the mechanical equipment will be exposed. The mechanical breadth will therefore consider the aesthetic placement and selection of mechanical equipment, will the goal to make the apartments aesthetically competitive with the original design.

Drywall Encapsulation

A significant design decision which affects many aspects of the redesign is weather to leave the wood exposed or to use the drywall encapsulation method. Based on the pros and cons in various categories shown in table 2.1, the drywall encapsulation method was chosen for this project. The factor given the most weight in the decision was fire-performance. Although it may be possible to achieve equivalent fire safety standards with an exposed structure, it is more likely that the first tall wood buildings in the US will be required to have some level of drywall encapsulation to meet stricture fire safety standards.

Pros and Cons of Encapsulation Method Versus Exposed Wood			
Category		**Encapsulation**	**Char Only (Wood Exposed)**
Construction	**Pros**	-Can cover mechanical work up -Construction of structure doesn't have to look as neat	-Simple construction -Connections can be simple if architect allows partially exposed connections
	Cons	-More time intensive -Connections must be more carefully detailed so that drywall can be installed	-Wood and other materials must be kept clean and neat during entire process since it will be exposed
Sound	**Pros**	-Increases STC and IIC values and overall sound insulation	-
	Cons	-	-A thicker topping for better sound insulation may be required
Economy of Cost and Structure	**Pros**	-Fire protection provided by drywall helps to reduce structure sizes, weight, and cost -When a single layer of drywall in used, sizes tend to be controlled by deflections or strength rather than the fire performance design check	-Drywall material and installation costs would be eliminated for the addition of the redesign
	Cons	-	-Structural elements may be larger since initial 30 minutes of drywall protection is not there -Larger structural elements will cost and weigh more
Architectural	**Pros**	-Interior finish will match original concrete portion -Increases freedom of interior design	-Look of wood adds warmth to space -May be desirable to some architects and occupants -Creates interest in wood as a building material for taller buildings -Exposed structure can be educational about how larger wood structures work
	Cons	-Not as unique as wood	-Could be viewed as "matchbox" building depending on public perception -Tenants may not be as comfortable living there if concerned about fire performance since this type of construction is not widely known or used in US
Fire Performance	**Pros**	-Gypsum provides time during which structure does not contribute to fuel load -Increases fire rating	-
	Cons	-Connections must still be carefully designed to prevent smoke and fire movement	-Exposed wood has potential to contribute to fuel load and produce smoke sooner during a fire -Connections must be more carefully sealed against fire and smoke

Table 2.1: Pros and Cons of leaving wood structure exposed versus using the drywall encapsulation method

2.8 Conclusions

The concept of using heavy timber in taller structures is relatively new, and therefore there are many potential benefits and challenges that are still being studied and explored. The decision to use heavy timber for the redesign of the 11141 Georgia Ave addition has many implications for the design work. Since the addition is three stories higher than the limit for heavy timber in residential and built on top of a multi-story building, extra care must be taken to ensure that the structure meets the goals of the code. Ultimately, the use of heavy timber significantly effects the structural depth work as well as both the construction management and mechanical breadths.

3 Structural Redesign

3.1 Introduction

The existing steel addition of 11141 Georgia Ave has been redesigned using engineered wood products. Both the gravity and lateral systems were redesigned, and the methods and results of the redesign are presented in this chapter.

3.2 Gravity System

The gravity system was designed using hand calculations. A typical bay layout uses a multi-ply CLT panel to span the full 20.75’ typical bay in the east-west direction. The panels are supported by glulam girders which span along the width of the building similar to the building shown in Figure 3.1. The girders frame into glulam columns which follow the same layout as the existing steel addition’s columns. The following sections describe the calculation process and assumptions used to design the gravity elements. Values and design procedures come from the 2015 NDS (American Wood Council, 2015), the Glulam Specification (APA - The Engineered Wood Association, 2008), and the Glulam Column Guide (APA - The Engineered Wood Association, 2009).

3.2.1 CLT Floor Panel Design

The floors were designed using a Cross Laminated Timber panel product, as described previously in chapter 2. Although the panels have two-way properties, the maximum deliverable panel width is shorter than the bay width. Therefore, it was assumed that each panel spans one-way from girder to girder. Most panels span 20.75’, and therefore the typical panel will be kept the same throughout a floor, including around stair and elevator cores, for uniform thickness. The only place where a different design will be used is between grids 2 and 3 where the span is approximately 26’. The floor panels were designed for strength, deflections, and fire performance, the processes for which are described in the following sections. Initial hand calculations and excel calculation tables can be found in Appendix A for reference.

Figure 3.1: Image of Structural System Used with CLT. Source: City of Melbourne online site

Load Calculations and Assumptions

The CLT floor panel design was completed for three floor types; the typical level, level 12, and the penthouse roof. The load assumptions for each level type are shown in table 3.1. The dead loads shown are based on including the self-weight of either a 5-ply or 7-ply CLT panel, and in non-typical cases, the dead load has been adjusted for other panel thicknesses as required. The 12th level snow load shown is slightly below the maximum drift value because even in the bay in which drift occurs, the overall average drift in psf would be lower than that at the maximum.

Gravity Loads (psf)			
Level	**Dead***	**Live**	**Snow**
Typical Level	36	40	0
12th Level	40	100	55
Penthouse Roof	36	30	20
* Difference in dead loads is due to thicker CLT Panel on 12th Level			

Table 3.1: Load Assumptions

Design for Bending Strength

The CLT panels were designed for strength assuming a simply supported span. The maximum moment for a panel was found based on the applicable load cases for the given level using equation 3.1 for a 1' unit strip width of panel, treating it like a simply supported beam. Then, rearranging equation 3.2 into equation 3.3, the calculated moment can be used to choose a panel size based on the required section modulus.

$$M = \frac{wl^2}{8} \tag{3.1}$$

$$S_{required} = \frac{M}{F_b} \tag{3.2}$$

$$M \leq F_b \times S_{required} \tag{3.3}$$

The F_b values used for design came from the US edition of the CLT Handbook (FPInnovations and Council, 2013). A table showing values for F_b and F_b*$S_{required}$ for various material grades and panel thickness can be found in Appendix C. Anytime an allowable stress F_b is used in this and the following design sections, the following factors are assumed:

$C_M = 1.0$
$C_T = 1.0$
$C_i = 1.0$
$C_D = 0.9$ for dead load, 1.0 for live, 1.15 for snow, 1.25 for construction, 1.6 for wind
C_F or C_V will be determined as required.

Deflection Check and Serviceability

Deflections were limited to l/360 for service live loads and l/240 for service dead plus live load. The following equation for a simply supported beam was used with a 1' strip unit width of the CLT panel. The value for EI was found using a table from the CLT Handbook, which can be found in Appendix C. A factor of $K_{cr} = 2.0$ was applied to dead load due to long term deflection and creep of wood per the National Design Specification for Wood Construction.

$$\Delta = \frac{5wl^4}{384EI} \tag{3.4}$$

Additionally for serviceability, design for vibrations was checked. Table 3.2 provides recommended CLT spans to limit vibrations. The 12th level is considered an assembly space and has a 7-ply panel which meets the recommended span limit. Although the 5-ply panels on other levels do not meet the limit of 18 feet, they only span 20 feet, which is close to the limit. Furthermore, the spaces on the levels with 5-ply panels are not expected to be as sensitive to vibrations as an office environment for example. Therefore, the 20 foot span may be considered acceptable in this case.

Type of CLT	Thickness (in.)	Vibration Controlled Span, L (ft.)	Equivalent UDL Criterion
5-layer (5s)	5 1/2	15.6	Span/417
5-layer (5s)	7 3/16	18.0	Span/497
7-layer (7ss)	9	23.0	Span/606

Table 3.2: Table showing recommended spans for various CLT thicknesses to limit vibration. Source: CLT Handbook

To help improve vibration performance, single bay CLT spans will be used. In the CLT handbook, continuous spans are not recommended across units in multi-family because of flank transmission. Furthermore, the additional stiffness provided by continuous spans would limit deflections, but it has not been shown to improve vibration performance since the mass is the same. Therefore, the single span panels will help to isolate the apartments from other units, thus decreasing the vibrations observed.

Fire Performance

The design for fire performance combines both the encapsulation and char methods. Although the research being done by design firms and other associations show that it is possible to design completely exposed CLT structures for adequate fire performance, this redesign chose a more conservative approach. The wood elements will be encapsulated in a layer of drywall to reduce the depth of charring during two hours as well as increase the likeliness of US code official acceptance since the 1st tall wood buildings accepted will most likely have drywall encapsulated wood.

The drywall will be installed up against the CLT panel such that there are no concealed spaces. Each layer of drywall provides an additional 30 minutes of fire protection. Table 3.3 shows the total effective char in a CLT panel for varying periods of time. The portion of the panel within the effective char layer is assumed to no longer add any capacity to the section. The CLT panel design began with the assumption that a single layer of drywall protects the panel, and therefore the effective char used for design will be taken after 90 minutes as 2.5", since the drywall provides the initial 30 minutes.

Required Fire Resistance	Effective Charring Rate, β_{eff} (in./hr)	Visual Char Layer Thickness (in.)	Zero-strength Layer (in.)	Effective Char Layer Thickness, a_{char} (in.)
45 min (¾-h)	1.90	1.19	0.24	1.42
60 min (1-h)	1.80	1.50	0.30	1.80
90 min (1½-h)	1.67	2.09	0.42	2.50
120 min (2-h)	1.58	2.64	0.53	3.16

Table 3.3: Table showing effective char thickness for varying duration. Source: CLT Handbook

The calculation was completed using the following methods. First, reduced loads were found since after two hours, it is assumed that many people will have left the building by then and live loads will be much lower. It was also assumed that the 12th level would no longer be acting as an assembly space two hours into a serious fire, and so it was treated like a residence space. Furthermore, the structure itself and its contents will have burned away after two hours, and therefore the dead load would be reduced. The controlling case used for fire performance design is dead plus live.

$$(D+L)_{reduced} = (0.75D) + (0.4L) \tag{3.5}$$

$$M = \frac{(D+L)_{reduced}\, l^2}{8} \tag{3.6}$$

Next, the residual panel thickness was found using the effective char depth. Since the effective char depth is 2.5 inches, and 2 plies of a panel are 2.75 inches, approximately 2 plies no longer contribute to strength. Therefore the effective residual structural panel thickness was taken to by 2 plies less than the original thickness.

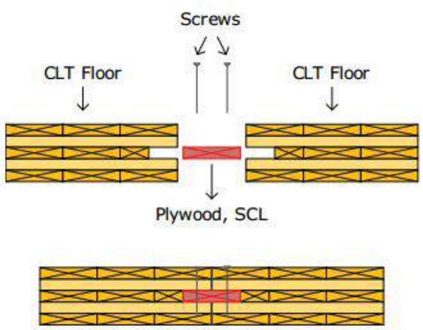
$$\#plies_{residual} = \#plies_{original} - 2 \tag{3.7}$$

The $F_b S_{effective}$ used in this stage of the design will be that off the residual panel thickness. Similar to strength design, the calculated moment must be check against this value.

$$M_{fire} \leq (F_b \times S_{required})_{resid.} \tag{3.8}$$

Panel Connections

The CLT floor panels require a connection between panels in the direction of the span. This connection detail allows the panels to act together better and prevent differential deflections from panel to panel due to variations in loading. Figure 3.2 shows 2 spline options to create this connection.



(a) *Single Spline Connection. Source: CLT Handbook*

(b) *Double Spline Connection. Source: CLT Handbook*

Figure 3.2: Spline connection options between panels

3.2.2 Glulam Girder Design

The CLT floor panels span into the girders, which were designed using a glulam engineered wood product. It should be noted that a fire-rating calculation exists for glulam, however, the code limits the use of that calculation to one hour even if the equation gives higher values. However, the NDS provides values for char thickness, which will be used in the design. At 90 minutes, there is an effective char thickness of 2.5".

The girders will be an inverted T-shape to provide a base for the floor panels to frame into. In the typical detail where brackets are provided, to support a girder framing into a rectangular beam or a wall, the connection may be relatively large and bulky. In this design, since it is important for the drywall to fully encapsulate the wood tight to the surface, this design should be avoided. Furthermore, a goal is to limit the depth of the structure, so it is not acceptable to simply frame the panels on top of the girders. The inverted T connection detail will also provide protection for steel fasteners since they will be buried in lumber, as shown in figure 3.3. The connection on the right in figure 3.3 is a proposed connection in which the T-girder flange provides 4" of bearing for the CLT panels framing into each side. Self-tapping screws or another fastener type can be used to connect the elements. Connection design is a topic within CLT buildings which requires more study depending on the requirements of the connection.

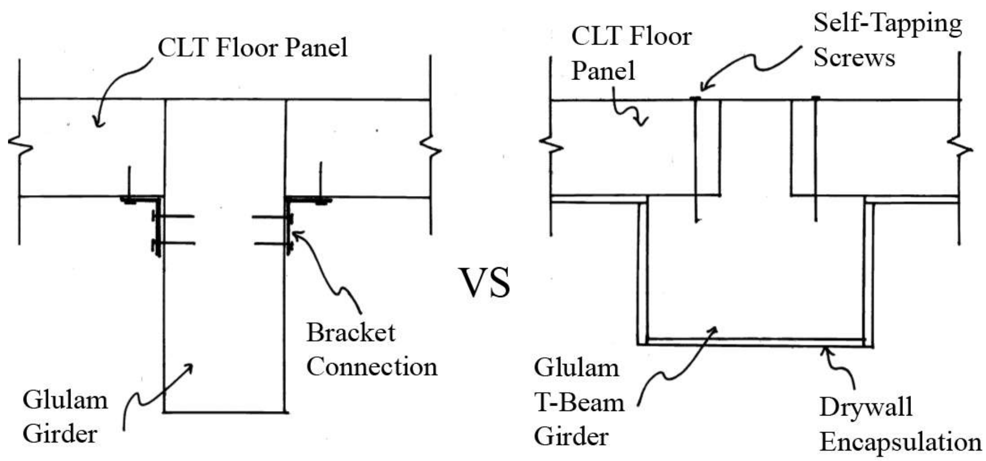


Figure 3.3: Connection options for rectangular girder versus inverted T

The girders have been designed for strength, deflections, and fire performance at each level type. Hand calculations for typical girders and excel calculations for both typical and non-typical girders are available in Appendix A.

Load Calculations and Assumptions

The load assumptions for the girder design are the same as they were for the CLT floor panel design. The only difference here is that the self-weight of the girder must be added to the dead load. based on an approximate initial guess of a 12×20" girder, 50 plf was used for the girder self-weight in the design calculations. All final girders sizes weighed less than this, and therefore it was a conservative design choice and does not need to be rechecked.

Design for Strength

The glulam girders were designed for strength assuming a simply supported span. The design for strength is very similar to the CLT panel since the panel was designed per 1' strip and treated like a beam. The maximum moment in a girder was found based on the applicable load cases for the given level using equation 3.9. Then, a girder size can be chosen based on the required section modulus and available typical sizes. The strength calculations used F_c=2400 psi and E'= 1.8×10^6 psi, as found in the Engineered Wood Association's Glulam Design Guide for Douglas-Fir.

$$M = \frac{wl^2}{8} \tag{3.9}$$

$$S_{required} = \frac{M}{F_b C v} \tag{3.10}$$

$$S_{actual} \leq S_{required} \tag{3.11}$$

In addition, the girder design requires the use of a volume factor, C_V:

$$C_V = \frac{12}{d}^{(1/10)} \times \frac{5.125}{b}^{(1/10)} \times \frac{21}{L}^{(1/10)} \tag{3.12}$$

Since the girder is an inverted T-shape, the flanges contribute to the girder's strength capacity. Originally, the calculations for strength did not include the flanges for simplicity, especially since deflections or fire performance usually controlled. However, there were significant advantages to taking the time to calculate the section properties for the T-shapes and this ultimately helped the efficiency of the design. The F_b values used for design came from the APA Engineered Wood Association Glulam Design Guide.

Deflection Check

As in the strength check section, the deflection check here is very similar to the check for the CLT panel. Deflections were limited to l/360 for service live loads and l/240 for service dead plus live load. The following equation for a simply supported beam was used to check a girder for deflection. The value for EI was found using a table from The Engineered Wood Association Glulam Design Guide, which can be found in Appendix C. A factor of $K_{cr} = 1.5$ was applied to dead load due to long term deflection and creep of wood per the National Design Specification for Wood Construction.

$$\Delta = \frac{5wl^4}{384EI} \tag{3.13}$$

Fire Performance

The design for fire performance is again similar to the design for CLT and combines both the encapsulation and char methods. The beam will be encapsulated in a layer of drywall as shown in figure 3.4, just as the CLT panel was, with the drywall flush against the wood leaving no concealed space. The design again began with the assumption that there will be a single layer of drywall, which provides 30 minutes of protection, and that the effective char depth for design is 2.5 inches after 2 hours. The flanges are wide enough in the design that there will be at least a small portion of bearing area left to support the panels temporarily. In the case of a fire so severe that this degree of char occurs, the building would require major renovation or demolition, and thus long term carrying capacity of the connection after this amount of charring is not considered in the connection design. The portion of the beam within the effective char layer is assumed to no longer add any capacity to the girder.

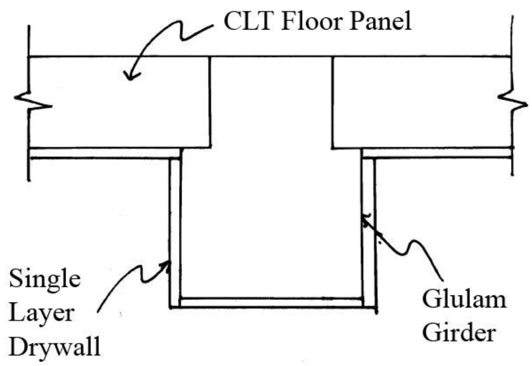


Figure 3.4: Drywall Encapsulation Method

The calculation was completed using the following methods. The loading assumptions during a fire are the same as for the CLT panel design calculations. The controlling case is again dead plus live, and the reduced loads and associated moment must be calculated.

$$(D + L)_{reduced} = (0.75D) + (0.4L) \tag{3.14}$$

$$M = \frac{(D + L)_{reduced} \times l^2}{8} \tag{3.15}$$

Next, the residual girder size was found using the effective char depth. Since 3 sides are exposed to the fire now, more of the section is lost to fire than in the CLT design, as shown in figure 3.5. Since the effective char depth is 2.5 inches, this thickness must be removed from three sides of the beam to calculate the strength performance during a fire. Since the top web portion of the inverted T-beam is protected by the surrounding CLT panel as shown in figure 3.6, the full web thickness could be used in the residual section property calculations.

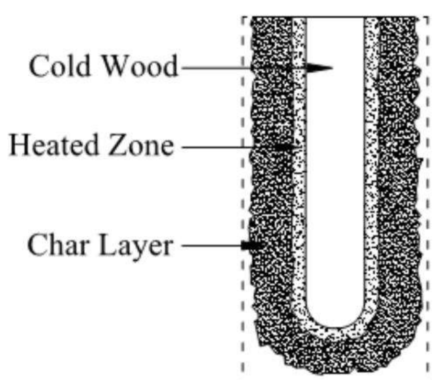


Figure 3.5: Diagram of charring which occurs in beams. Source: FPRF

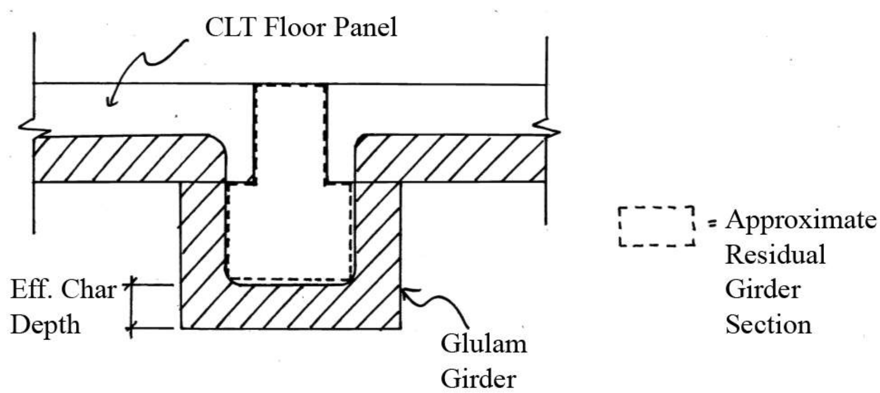


Figure 3.6: Char pattern occurring at Panel to Girder connection

$$\text{Width}_{\text{residual}} = \text{Width}_{\text{original}} - 5'' \tag{3.16}$$

$$\text{Depth}_{\text{residual}} = \text{Depth}_{\text{original}} - 2.5'' \tag{3.17}$$

The residual depth and width were used to find the section modulus, which was then compared to the required section modulus.

$$S_{required} = \frac{M}{F_b C_V C_D} \tag{3.18}$$

Here the duration factor, C_D, was assumed to be 1.6 since the duration of a fire is similar to that of a wind or seismic loading.

$$S_{actual} \leq S_{required} \tag{3.19}$$

Finally, the actual and required section modulus can be compared to choose or confirm a size.

3.2.3 Glulam Column Design

The glulam columns in the addition follow the same layout as in the steel design. Initial calculations have shown that the new wood framed system is lighter than the original steel addition, therefore the effect of the redesigned system on the existing foundations and columns will not be checked, since the existing design already showed that the additional weight of the addition did not over stress the existing columns.

Hand calculations were completed for a typical interior and exterior column at the base of the addition to find initial sizes and determine the calculation method to be used. These hand calculations are available in Appendix A for reference. An excel spreadsheet was programmed for use to design the columns for each level. For simplification, the columns in the building were grouped into 8 column types. The excel column design sheet can be found in the appendix.

Load Calculations and Assumptions

Load combinations which included dead, live, and snow loads were all checked in the initial hand calculations of typical members in Appendix A, and the controlling load combination was dead plus live, and was therefore used throughout the calculations. The same dead and live load assumptions as before are used here with the addition of the column self-weight. Live load reduction was not utilized, resulting in a conservative design.

Design for Compressive Strength

The compressive strength calculations began with F_c=1950 psi and E'= 1.6×10^6 psi, as found in the Engineered Wood Association's Glulam Column Design Guide for Douglas-Fir.

Next the factors were assumed to be the same as in the glulam design, including the C_v factor. The following equations were then used to determine F'$_c$:

First, F_c must be modified by all factors. Here, the only factor not equal to 1.0 is C_v.

$$F_c^* = C_V F_c \tag{3.20}$$

Next, E'$_{min}$ must be found, where COV_E = 0.1 for glue laminated timber of 5 or more laminations.

$$E'_{min} = E'(1 - 1.645 COV_E(1.05))/1.66 \tag{3.21}$$

The C_p factor equation accounts for the slenderness effect of a column, where c = 0.9 for glue laminated timber.

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} \tag{3.22}$$

The final F'$_c$ value is obtained by multiplying F_c^* by the slenderness factor.

$$F'_c = F_c^* \times C_p \tag{3.23}$$

The actual stress based on the applied loads and column area can then be calculated and compared to F'$_c$.

$$f_c = \frac{P}{A} \tag{3.24}$$

$$f_c \leq F'_c \tag{3.25}$$

Fire Performance

The design for fire performance of the columns takes into account that all 4 sides will char in the fire, resulting in greater section loss than in the panels or girders. Just as before, the calculations began with the assumption that there will be a single layer of drywall and the char depth after 2 hours will be 2.5". The previous calculations will still be used with just a few exceptions. Since a fire occurs over a relatively short period of time, the duration factor, C_D, was assumed to be 1.6 as before. Therefore;

$$F_c^* = C_V C_D F_c \tag{3.26}$$

Additionally, the load will be reduced using the same assumptions as in the panel and glulam design methods, and the residual area will be calculated based on the effective char depth.

$$A_{residual} = (b - 5") \times (d - 5") \tag{3.27}$$

$$f_c = \frac{P_{reduced}}{A_{residual}} \tag{3.28}$$

3.2.4 Typical Opening Design

There are several openings in the floor for mechanical openings and shafts. Therefore, there must be some support provided for the CLT spanning to the opening. A typical opening detail was designed in which the side of the opening perpendicular to the panel's span is supported by a steel angle, and is then welded to a steel plate support which is attached to the top of the panel. See figure 3.7 for an image of this typical detail.

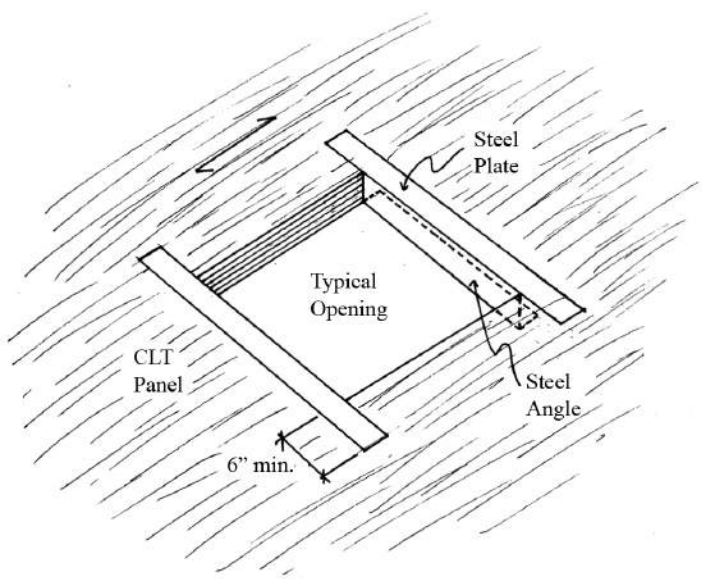


Figure 3.7: Image of Typical Opening and Support

The calculation process includes finding the loads and maximum moment applied to the angle. Based on using A36 steel, the required section modulus was found and a typical L8x6x3/8 was chosen using the Steel Manual (AISC, 2010). This size is for the typical 6.875" panel thickness such that the angle extends upwards enough for a welded connection to the flat plate, as shown in figure 3.8. It is okay for the angle to extend further than the plate since it is at a floor edge and each opening is surrounded by a partition wall.

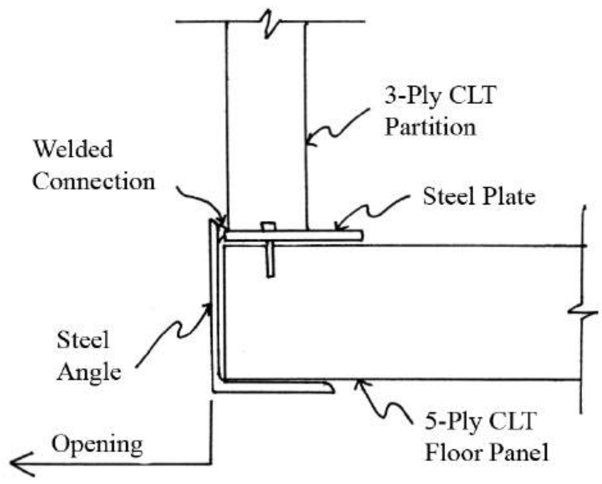


Figure 3.8: Cross-sectional cut through typical opening detail

Note that this is an approximate and somewhat conceptual detail just to show that it is possible to strengthen the openings without providing additional glulam beams or bearing walls to support the opening, both of which are not economical and would be time-consuming to construct.

3.2.5 Concrete Bearing Walls

The concrete shear walls in the concrete portion of the existing design will now have additional weight due to the additional height of the shear wall extending into the addition redesign. Some of the gravity loads will also frame into a few of the shear walls, adding to the total bearing load. Therefore, critical concrete shear wall conditions must be checked for the new design conditions. Since the shear walls must be designed primarily for lateral load, the concrete wall design, including bearing checks, will be covered in the lateral system section.

3.2.6 Gravity System Conclusions and Design Summary

The gravity redesign included design for the CLT floor panels, glulam girders, and glulam columns for the typical level, level 12, and the penthouse roof, as well as a typical opening detail. In the floor system design, size choices were mainly controlled by deflections, connection design, and sometimes by strength. Due to the types of beam and column cross sections used in this case, the fire performance design method never controlled the design. In the design of a CLT building, fire performance should always be checked, although it will tend not to control the final selected member sizes due the the nature of the heavy timber design. Table 3.4 presents a summary of the final designed sizes.

Design Summary				
Structural Floor Elements				
Element	**Typical Level**	**12th Level**	**Penthouse Roof**	**# Gyp Layers**
Typ. CLT Floor Panel	5-ply	7-ply	5-ply	1
Typ. Glulam Girder*	15" deep	18" deep	15" deep	1
Floor Panel (26' bay)	7-ply	9-ply	7-ply	1
Typical Column	**Interior**		**Exterior**	-
At addition base	12" x 12 3/8"		12" x 12 3/8"	1

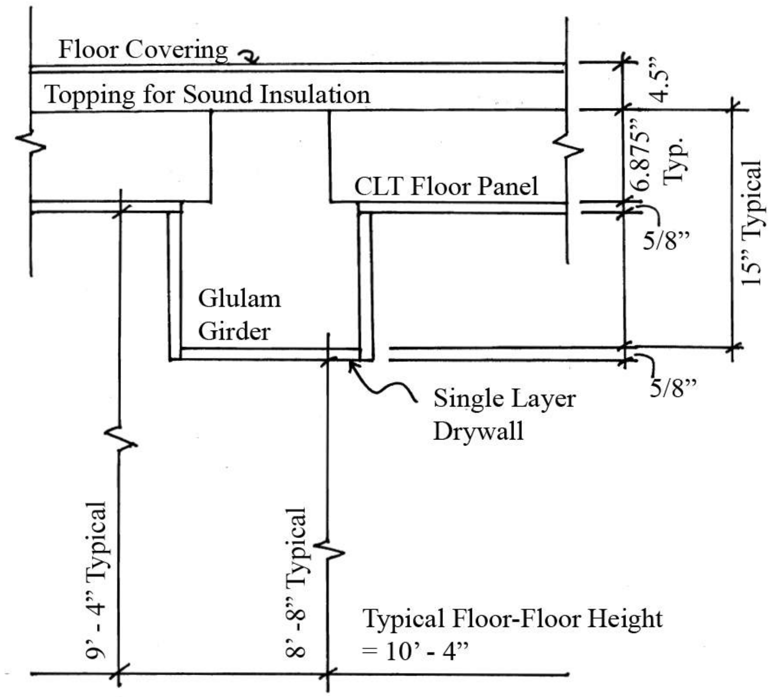
Non-Typical Girders*					
Girder Type		**Typical Level**	**12th Level at Parapet**	**12th Level at Penthouse**	**Penthouse Roof**
Perimeter Girders	**E-W dir.**	15"	13"	15"	13"
	E side	19 1/2"	19 1/2"	25 1/2"	18"
	W side	19 1/2"	21"	25 1/2"	18"

Non-Typical Columns and Columns at Other Levels**				
Column Type	**Levels 7&8**	**Levels 9&10**	**Levels 11&12**	**Penthouse**
Typ. Int.	12 3/8"	10 1/2"	8 1/2"	8 1/2"
Typ. Ext.	12 3/8"	10 1/2"	8 1/2"	-
A	12 3/8"	10 1/2"	8 1/2"	8 1/2"
B	10 1/2"	8 1/2"	8 1/2"	-
C	10 1/2"	10 1/2"	8 1/2"	8 1/2"
D	10 1/2"	8 1/2"	8 1/2"	8 1/2"
E	12 3/8"	10 1/2"	8 1/2"	-
F	12 3/8"	10 1/2"	8 1/2"	-
*Note: All girders have bf = 12" and bw = 4" for improved connection constructability				
**All columns have bw = 12 to match the girder flange width				

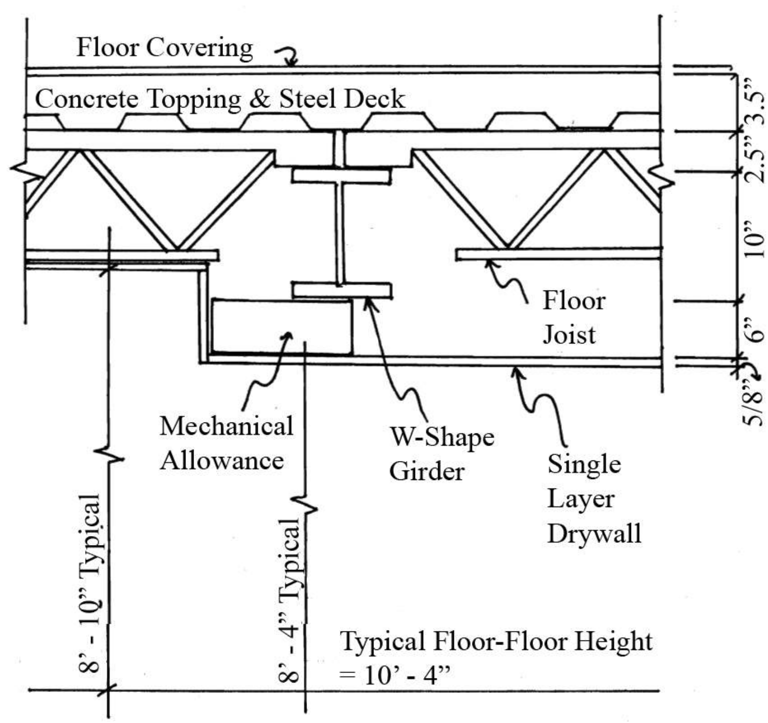
Table 3.4: Gravity System Design Summary

The girder and column design ultimately required that they have the same width for drywall encapsulation and a clean connection appearance. Therefore, after initial design calculations, the final calculations were based on using a 12" width for both girders and columns. The column design excel sheet can be found in the appendix and shows that more efficient members could be used in the upper levels. However, the difference is that rather than using an 6.75" × 12" column for strength and fire performance, a 8.5" × 12" columns was used for easier connection detailing. Finally, the typical opening design used an L8×6×3/8 angle.

With the conclusion of the gravity system design, figure 3.9 (a) shows a typical cross section through the redesigned floor to floor height, and figure 3.9 (b) provides the existing design cross section for comparison. The new system typically has taller ceiling heights, even with the increased floor topping thickness for sound insulation. Part of the reason for this is that the mechanical equipment is exposed, and therefore a drop ceiling is not required.



(a) *Gravity System Redesign*



(b) *Existing Gravity System*

Figure 3.9: Floor to Floor height cross section comparison

3.3 Lateral System

The lateral system was predominately designed through the use of 3D modeling in ETABS structural analysis software. The following sections describe the methods and assumptions used to model the lateral system, as well as the design decisions and process for the lateral system design.

Only the addition of the building was modeled for the redesign of the addition. The analysis of existing building showed that the concrete portion was sufficiently stiff to resist additional wind loading due to the increased height of the building. It is not necessary to include the lower portion in the lateral modeling of the redesign because the results of both portions could be superimposed. Modeling only the addition also keeps the model in ETABS simpler.

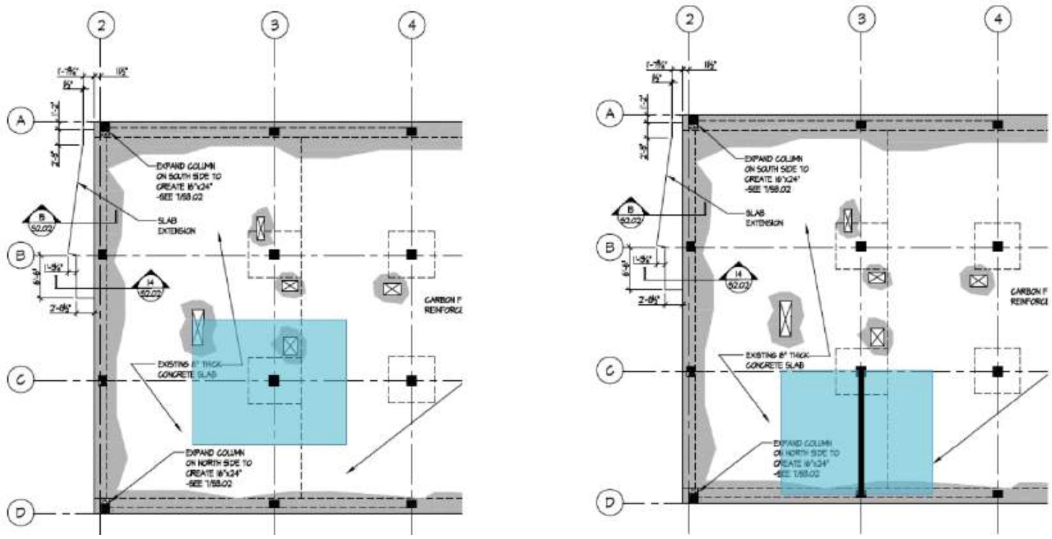
Because of the choice of wood as the structural material, the lateral system requires significant changes. In the existing steel addition, the lateral system includes several moment frames. However, in wood construction, it would be very difficult and expensive to create moment connections. Furthermore, it would be especially difficult to use the drywall encapsulation method around bulky moment connections. Therefore, a different lateral system will be used entirely. Braced frames were considered as it is an approach that has been used in other CLT buildings. The main factors against using braced frames include difficulties with the encapsulation method in avoiding concealed spaces and architectural conflict with the windows and corridors. Therefore shear walls were used in the new lateral system. The shear walls around the core area were continued upwards through the building, and a few shear walls were added to the ends of the building.

3.3.1 Modeling Approach and Assumptions

The lateral system model included only the lateral system components, leaving the gravity system out of the model for simplicity. Therefore, the shear walls and floor diaphragms were the elements included in the ETABS model. Although the wood floor panels are fairly thick compared to a joist framed system, their stiffness is nowhere near that of a rigid concrete floor, and thus the floor diaphragm was modeled as semi-rigid.

Although only the addition was considered, the shear walls extending through the original concrete building must meet the requirements as well. However, the concrete moment frame structure is so stiff that much of the lateral load would be distributed into the moment frames. Furthermore, the original portion of the building experiences very low drift, and thus it can be assumed that the shear walls at these levels are braced well enough at each level that the wall is controlled by compression since it is not free to deflect enough to experience potentially controlling flexural or shear forces. The shear walls typically have the same tributary area as the concrete columns in the original portion of the building as shown in figure 3.10. Thus, since the concrete columns have already been proven to be adequate for the loads, and the shear walls have a greater cross-sectional area, it can be assumed that the shear walls are adequate for the load bearing on them. This assumption further justifies the decision to model only the addition.

The approach used to create a shear wall layout was to first check the addition for drift requirements. Masonry shear walls were initially used around the stair cores and one of the elevators for the fire-resistance of egress routes. Initially, wood shear walls were added to the



(a) *Tributary acting on typical column* (b) *Tributary acting on typical shear wall*

Figure 3.10: Comparison of tributary areas for both concrete column and shear wall

building for additional lateral support. The walls would have been masonry in the concrete portion, switching over to wood in the addition to decrease gravity loads. Initial modeling showed that wood shear walls in the addition is not a realistic option because the lengthiness of the building causes very large wind loads in the north-south direction. The number of wood shear walls required in the case of this addition becomes uneconomical. Therefore, masonry shear walls were initially used in the redesign of the lateral system. However, the amount of reinforcing required in the ends of the walls for flexure was too large, and the final design used concrete shear walls.

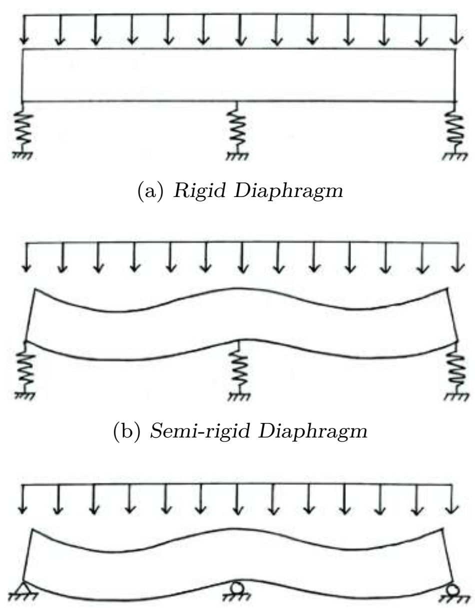
Several assumptions were made in the modeling process to approximate the behavior of a wood heavy timber building with concrete shear walls. The assumptions used include the following:

- All bases are pinned
- Concrete shear walls are modeled as thin shells
- The concrete was modeled as 4000 psi normal weight concrete
- A cracking modifier of 0.7 for f11 and f22 was included in the wall properties
- Since the shear walls are assumed to not take out-of-plane bending, m11, m12, and m22 were set to 0.1
- CLT panels are modeled as an isotropic material with properties based on the CLT handbook
- The floors are modeled as semi rigid diaphragms and a thin shelled element
- Story elevations were entered based on an elevation of zero feet at level B1

- Since wind loads controlled over seismic, wind loads were the main consideration
- Gravity loads were left out of the model and checked elsewhere by hand or by inspection
- Automatic wind loads in ETABS based on ASCE 7-05 were used for analysis and design of the wood addition
- Walls and floors were auto-meshed to have element sizes of 4’ by 4’ or less

3.3.2 Model Behavior

Because of the semi-rigid nature of the wood floor, the model of the redesigned wood addition behaves differently than both the existing steel addition and the original concrete building. Looking at the shear walls, they experience a drift typical to other buildings. The floors themselves, however, deflect in the plane of the diaphragm under lateral load between the lateral resisting elements just as a beam deflects between its supports. The behavior is similar to a flexible beam on spring supports as modeled in figure 3.11 (b). The rigid diaphragm behaves like a rigid beam on spring supports and distributes loads to lateral elements based on stiffness. The flexible diaphragm behaves like a flexible beam on rigid supports, and distributes loads based on tributary area. The semi-rigid diaphragm demonstrates a behavior which is between a rigid and flexible diaphragm, distributing load partially by stiffness and partially by tributary area.



(a) *Rigid Diaphragm*

(b) *Semi-rigid Diaphragm*

(c) *Flexible Diaphragm*

Figure 3.11: Diaphragm Behavior Based on Rigidity

Additionally, the building is rather long in the X direction, resulting in fairly large wind loads in the Y direction. Figure 3.12 shows the assumed X and Y directions of the building. Many taller CLT and wood buildings in other countries are approximately square in shape, and are therefore less likely to encounter the same challenges encountered in the design of the 11141 Georgia Ave addition. The addition model showed no problem with deflections in the X direction, where the building is only 60 feet wide. In the Y direction, the wind pressures are acting on a 214 foot width of wall, and thus the deflections requirements could not be met with wood shear walls even though other CLT buildings have worked with wood shear walls.

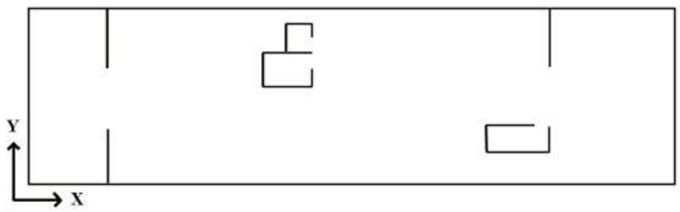


Figure 3.12: Simplified Plan Showing X and Y Directions

The behavior of the semi-rigid diaphragm means that ideally the shear walls should be somewhat evenly spaced such that the floors do not experience too much deflection within their plane between lateral resisting elements.

3.3.3 Shear Wall Design

The shear wall design began with design of all the walls in ETABS, and the model is shown in figure 3.13. Once the walls were designed using software, The wall on column line 3 between grids A and B was designed by hand as a spot check for the lateral system design. The results and comparisons between both software and hand methods are included in the following sections. More detailed calculation and loading information is available in Appendix C. The following sections also include additional service checks for the shear wall design.

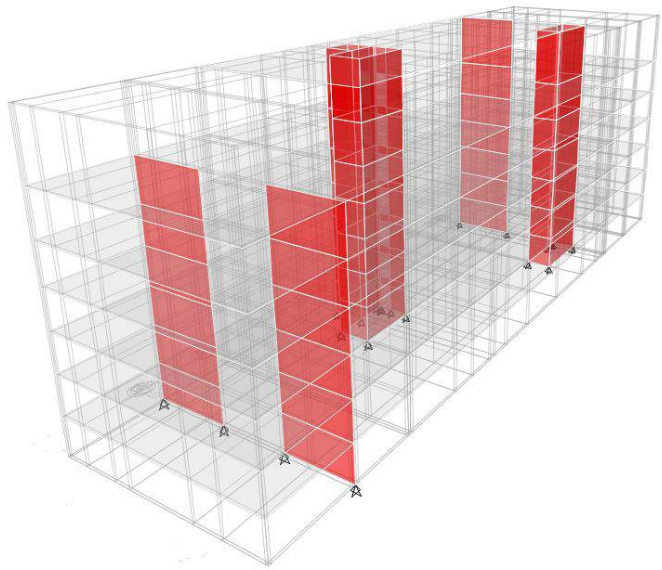


Figure 3.13: ETABS model 3-D View

ETABS Shear Wall Design

With the completion of the modeling, analysis, and initial layout of the lateral system redesign, the design was run in ETABS. Detailing was also run through ETABS using code limits on spacing and steel area as a starting point. The lateral forces from the wind load applied at each level is shown in figure 3.14.

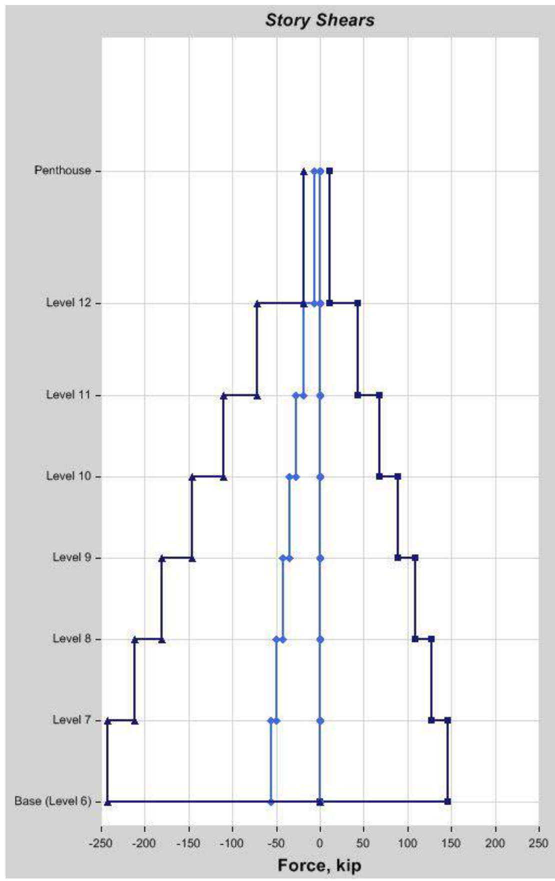


Figure 3.14: Lateral Forces Applied to Diaphragms due to Wind Load

The ideal reinforcing in the walls was based on code minimums, reflecting the expectation that the shear walls will be controlled by flexure rather than shear. The reinforcement that worked best in the ETABS design included number 4's for the horizontal and vertical reinforcing. In the ends of the walls, reinforcement was required for flexure, but 4's didn't work at the lower levels. Therefore, the smallest bars that could be considered were number 6's since a difference of at least two bar sizes is required within the same concrete element for easier inspection. The number 6's worked, and various amounts of reinforcing was required in the ends, with the most at the base level, and very little at the top levels. This is reasonable because the flexural forces will increase going from top to bottom in the building.

The 8" concrete wall was found to be adequate for the design with the inclusion of the reinforcement. The benefit of the 8" wall working for the design is that it is fairly thin and can fit easily within the desired wall thicknesses and meet architectural design requirements. Figure 3.15 shows a schematic drawing of a sample wall's required reinforcement at the 7th floor, which is the critical level for the addition. This shear wall is located on column line 3 between grids A and B, and will be checked by hand as well to spot check the software design.

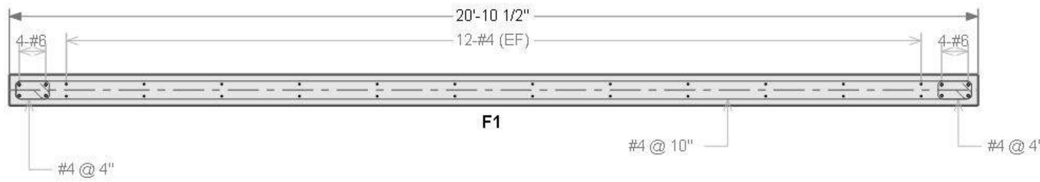


Figure 3.15: Schematic drawing of typical required shear wall reinforcement

Hand Calculation Wall Design Spot Check

Once the shear walls were designed for lateral forces, in ETABS, the wall on column line 3 between grids A and B was designed by hand as a spot check. The hand calculation for this spot check can be found in Appendix B. The hand check found that the wall was controlled by flexure and did not require any reinforcing for shear other than the code minimums because the 8" wall provided sufficient shear area. The calculated minimum reinforcing included 2 curtains of horizontal number 4's at 10" on center, and 2 curtains of vertical number 4's at 18" on center, which matched exactly the reinforcing provided in the walls in ETABS. The required end reinforcing for flexure required 4 number 6's at level 7, which also met the reinforcing from ETABS for the spot checked wall.

Building Overturning Moment Check

The overturning moment in a wood building should generally be checked. Since wood is much lighter than the other conventional building materials, it has the benefit of requiring smaller foundations for smaller gravity loads. A caveat to this is that as a wood building gets taller and has more wind load, the uplift could potentially grow larger than the weight of the building. Thus, the foundations in taller wood buildings should be designed for uplift, and overturning moment should be checked. However, in the case of 11141 Georgia Ave, there is the benefit of the original concrete portion of the building and its mass. Despite the lightweight wood addition, the apartment building is not expected to have any trouble resisting the overturning moment. An overturning hand check for the full building verified this expectation and is available in Appendix B.

Drift Check

The lateral system was checked against allowable drift. An allowable drift of h/400, or 0.0025, was used in the design. Since the diaphragm is semi-rigid, the lateral elements may not deflect equally. Therefore, the drift check was completed for each lateral element at its top elevation, as shown in Table 3.5. The design drift is well below the allowable, and therefore meets the requirements.

Drift Check at Shear Walls							
Shear Wall	Height (ft)	X-direction		Y-direction		Allow. Drift	Drift Check
		Disp.	Drift	Displ.	Drift		
Elev/ Stair Core	80	0.135	0.000141	0.542	0.000565	0.0025	ok
Stair Core	80	0.123	0.000128	0.238	0.000248	0.0025	ok
Grid 3 AB	63.3	0.078	0.000103	0.087	0.000115	0.0025	ok
Grid 3 CD	63.3	0.11	0.000145	0.066	0.000087	0.0025	ok
Grid 10	63.3	0.105	0.000138	0.265	0.000349	0.0025	ok

Table 3.5: Drift Values and Checks at Lateral Force Resisting Elements

Drift was also checked for the total diaphragm and lateral system deflection. Drift was checked at the top of the building for h/400, both at the top of the penthouse and the top of the 12th level as shown in Table 3.6. The load combination used for drift is based on the commentary on appendix C in ASCE 7-05 on serviceability, which allows the use of D + 0.5L + 0.7W since full wind load is considered to be overly conservative. The commentary also recommends checking interstory drift against 3/8" to prevent damage to non-structural elements. This check is shown in Table 3.7.

Overal Drift Check (Including Diphragm Deflection					
Level	Height (ft)	Disp.	Drift	Allow.	Check
Penthouse	80	0.42	0.00044	0.0025	ok
Level 12	63.33	1.5	0.00197	0.0025	ok

Table 3.6: Drift Values and Checks at for Total Drift including diaphragm deflection

Interstory Drift				
Level	Height (ft)	Disp.	Allow. (in)	Check
Penthouse	80	0.16	0.375	ok
Level 12	63.33	0.37	0.375	ok
Level 11	51.33	0.13	0.375	ok
Level 10	41.33	0.07	0.375	ok
Level 9	31	0.08	0.375	ok
Level 8	20	0.18	0.375	ok
Level 7	10.33	0.66	0.375	No Good

Table 3.7: Interstory Drift check for Cladding and non-structural elements

The transition from the original building with a rigid diaphragm to the flexible diaphragm is a concern, because the combined interstory drift is larger than the limit. The location of concern is only on east side of the building in y direction where the distance between lateral elements and the end of building is just over 40 feet. Although the model shows excessive deflection of the diaphragm here, it was assumed that the mass of the masonry backup in the exterior wall would prevent in-place diaphragm deflection that large. Therefore, the addition meets drift and interstory deflection requirements.

In-Plane Floor Deflection Check

Due to the semi-rigid behavior of the floor as previously described, loads distributed by the façade to the diaphragms cause in-plane deflections. Since the in-plane deflections were fairly large, they were checked against allowable in-plane deflection to determine if the placement of the lateral elements is sufficient to prevent large in-plane deflections of the floor. The deflections were considered only due to wind in the Y-direction because this is the controlling case. The limit used was the analogous beam live load deflection of L/360, where L is the distance between lateral elements for the portion of the floor under consideration. Figure 3.16 provides the considered floor sections and their respective length.

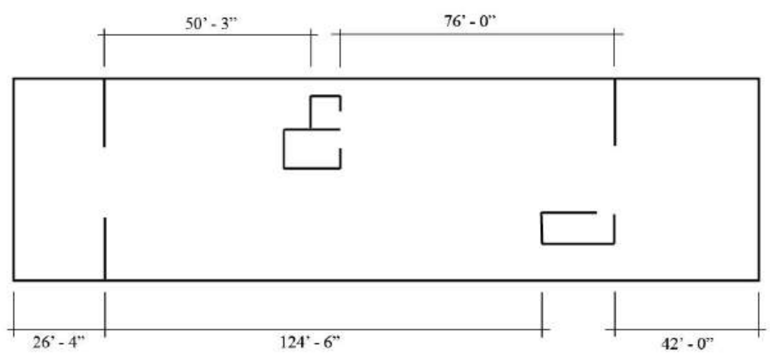


Figure 3.16: Assumed Floor Lengths for In-Plane Deflection Calculations

The 12th level saw the highest in-plane deflections, and was used in this check as the worst case level. The maximum displacement in the y direction was found for each floor section. The average deflection of the lateral element "supports" was then subtracted from the floor displacement to obtain the effective total displacement of the floor in a given floor section relative to its supports. The effective displacement was then compared to L/360, as shown in table 3.8.

In-Plane Floor Deflection Checks						
Location	"Length"	Max Displ.	Avg. lat. Disp.	Eff. Disp.	allowable disp.	OK?
Grid 2-3	26.3	0.901	0.078	0.823	0.877	ok
Grid 3-5	50.25	0.270	0.220	0.050	1.675	ok
Grid 3-9	124.5	0.26	0.108	0.152	4.150	ok
Grid 6a-10	76	0.178	0.225	-0.047	2.533	ok
Grid 10-12	42	1.58	0.190	1.39	1.400	ok

Table 3.8: In-plane deflection of the floor diaphragm

The interior floors were considerably within the allowable limits. These sections behaved like a beam with two fixed end supports, and therefore, the deflections were fairly low. The ends of the building behaved like a cantilever with a fixed support, and saw much larger in-plane deflections. Because of this, the end deflections fell just within the limits. The cantilever and beam diagrams used for comparing behavior are shown in figure 3.17. The deflections are based on service load conditions, and therefore it is acceptable that there is very little left over allowable deflection. The shear wall placement is adequate at limiting the in-plane deflection of the floor.

(a) *Fixed Cantilever Analysis Diagrams*

(b) *Fixed-Fixed Beam Analysis Diagrams*

Figure 3.17: Beam Diagrams from the NDS

3.3.4 Lateral System Conclusions and Design Summary

The lateral system was modeled and designed primarily with ETABS software, with some hand spot checks to validate the model. The system was designed for both strength and serviceability, with service requirements controlling the design. The semi-rigid behavior of the diaphragm controlled much of the layout and system requirements. A summary of the typical required reinforcing in the shear walls in provided in Table 3.9.

Typical Required Reinforcement in Wall Ends or Corners*					
Level	Grid 10	Elev Stair Core	Stair Core	Grid 3, Wall AB	Grid 3, Wall CD
PH	(4) #4	(4) #4	(6) #6	-	-
12	(4) #6	(4) #4	(6) #6	(4) #6	(4) #6
11	(4) #6	(4) #4	(6) #6	(4) #6	(6) #6
10	(4) #6	(4) #4	(6) #6	(4) #6	(8) #6
9	(6) #6	(4) #4	(6) #6	(4) #6	(8) #6
8	(6) #6	(4) #4	(6) #6	(4) #6	(8) #6
7	(8) #6	(4) #4	(6) #6	(4) #6	(10) #6
*All walls typically have 2 curtains of #4's at 18" o.c. vertical and 10" o.c. horizontal in the field of the wall					

Table 3.9: Required Reinforcement in shear walls

3.4 Structural Redesign Conclusions

The redesign of the addition to 11141 Georgia Ave included the design of a wood gravity system and a concrete shear wall lateral system. The gravity system design included the design of glulam columns and girders as well as cross laminated timber floor panels which spanned approximately 20' between girders. The gravity system elements were checked for strength, deflections, and fire performance, with deflections controlling most sizing requirements. The design was also based on the drywall encapsulation method for improved fire-performance, which required simple connections. The connection concepts significantly effected the girder design.

The lateral system redesign investigated the possibility of using wood shear walls in some places, but due to several factors, concrete walls were used for all shear walls. A typical 8" concrete wall was adequate with 2 curtains of number 4 rebar and number 6's at the ends for flexure. The lateral layout was controlled by the semi-rigid behavior of the wood floor diaphragm. Structural plans are shown in Figures

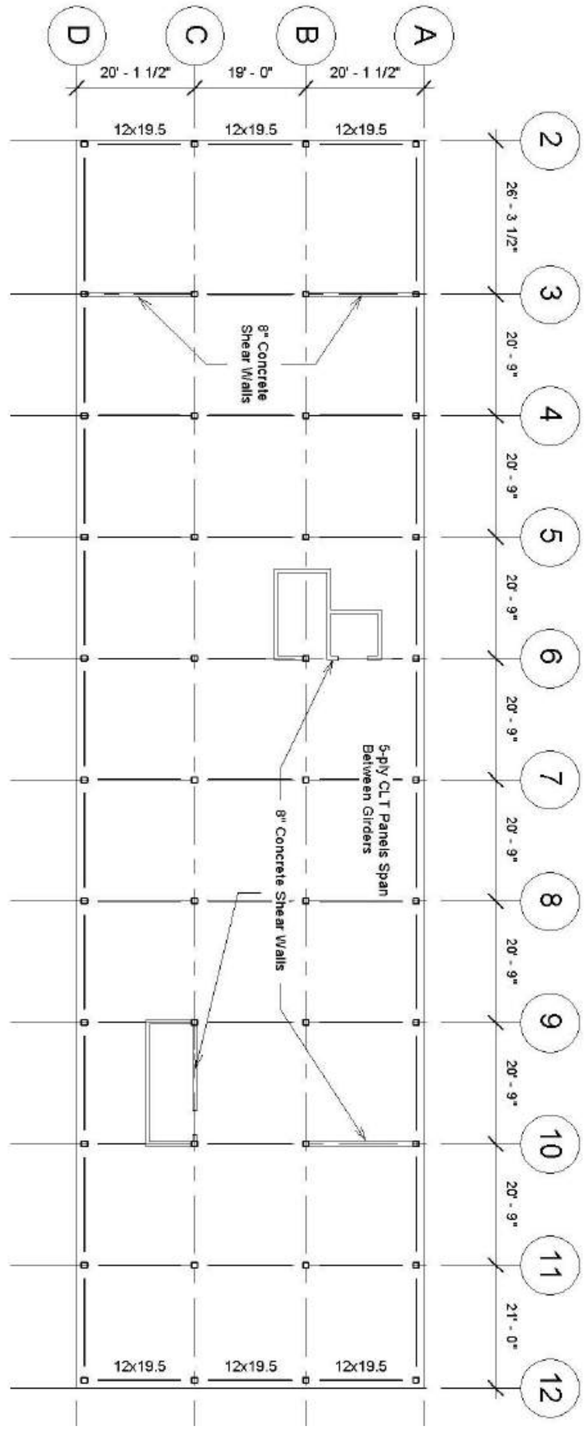


Figure 3.18: Typical Level Structural Plan. Unless otherwise noted, girders are 15" deep typ.

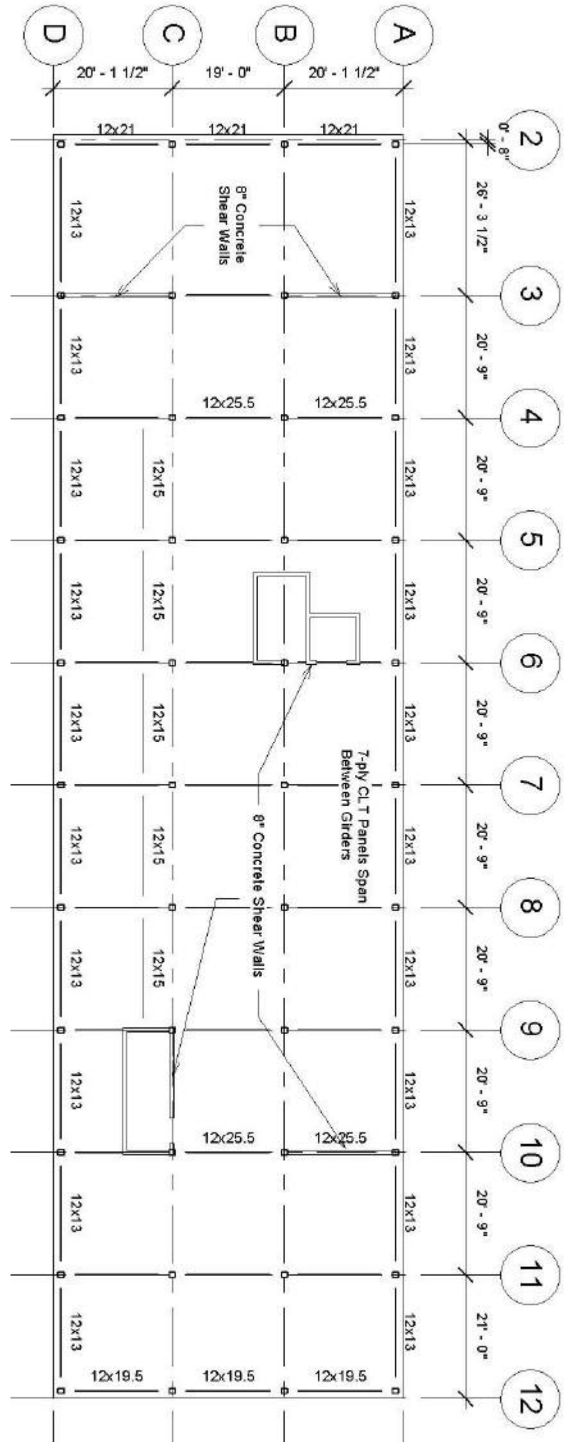


Figure 3.19: Typical Level Structural Plan. Unless otherwise noted, girders are 18" deep typ.

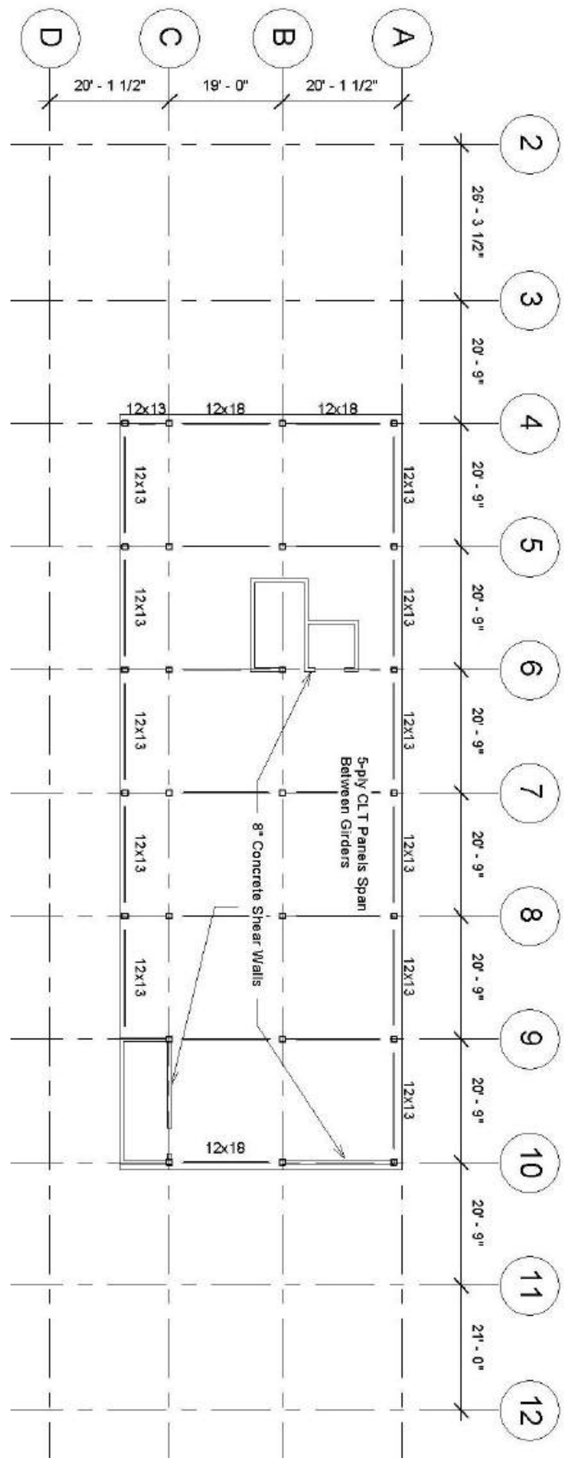


Figure 3.20: Typical Level Structural Plan. Unless otherwise noted, girders are 15" deep typ.

4 Construction Breadth

4.1 Introduction

In the construction management breadth, a cost and schedule analysis will be completed for the existing and new addition. The schedule analysis will help determine scheduling differences between the methods in order to identify any significant changes in schedule. The cost analysis will provide a basis for investigating the economic feasibility of a wood-framed addition compared to the costs of the existing steel addition. The focus of the cost analysis will be specifically on the existing and new additions themselves, but will also take into account any significant changes to general conditions costs due to scheduling differences resulting from the shift from steel to wood for the primary structural material.

4.2 Schedule Analysis

The schedule analysis includes the determination of the schedule for both the existing and redesigned addition and a comparison between the two. Only the addition gravity system structure, partitions, and lateral system were included in the schedule analysis. The schedule analysis is based on determining the total time required for the items within the scope of the analysis as well as an approximate schedule using RS Means Building Construction Cost Data 2014 (Waier, 2014).

4.2.1 Existing Addition

The existing addition schedule used crew sizes and labor time values from RS Means to determine the time required to construct the addition. The total time for construction of the entire addition is known to be 19 months, from February of 2013 to August of 2014. The resulting schedule information for the existing steel addition is presented in figure 4.1, an approximate schedule which accounts for weekends and potential overlap of work. It was assumed that the next set of columns could not be installed until the concrete deck topping has cured for a week. It was also assumed that the partitions would not be installed on a level until the concrete fully cured to 28 days. Since the CMU walls are only in the original portion of the building, they can be constructed concurrent with the addition.

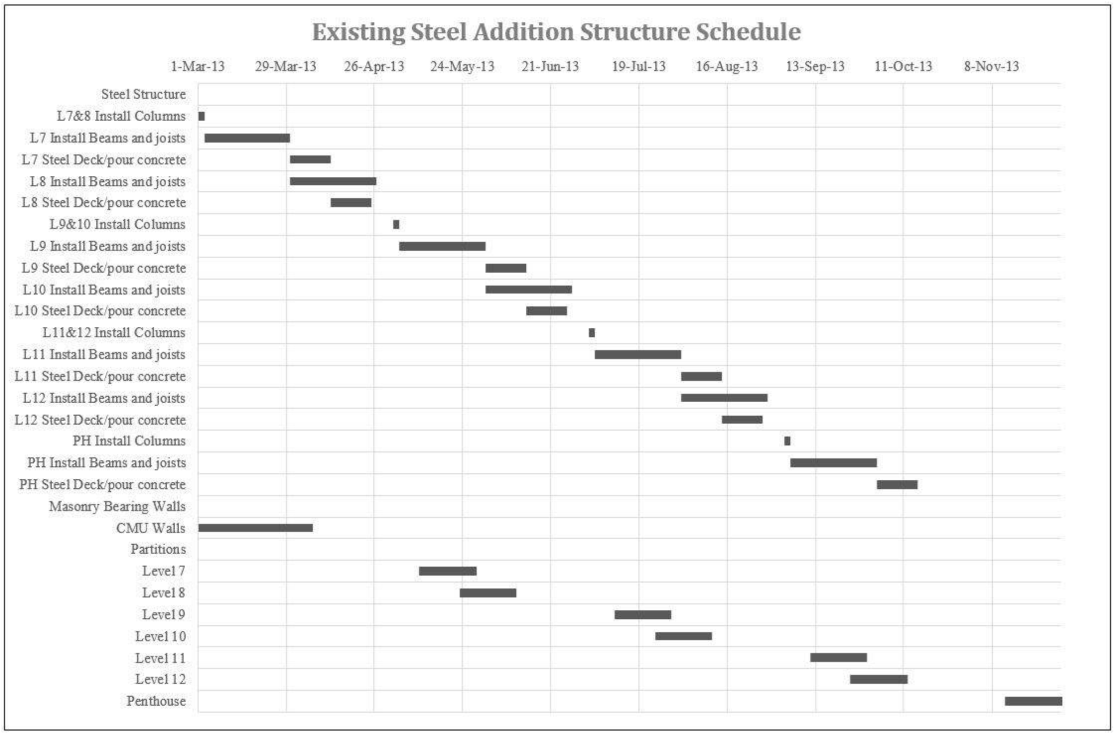


Figure 4.1: Total scheduling time required for construction of existing addition

4.2.2 Redesigned Wood Addition

The redesigned wood addition uses CLT for the gravity system, which does not appear in RS Means because it is relatively new to the US. In many resources, the construction process of CLT panels is said to be similar to a pre-cast concrete panel system, so the CLT floor panel and partition installation used RS Means data based on similar concrete panel sizes and types. The resulting labor time and schedule is presented in figure 4.2. Since gravity system elements are prefabricated, the elements can be installed one after another. The concrete shear walls in the concrete portion were not included in the schedule because their construction would most likely occur near the beginning of the project during renovation work.

4.2.3 Schedule Comparison

The schedule time for both the redesign and the existing building were compared to each other and to the total addition and renovation time line of 19 months. Based on the approximate schedules, the steel addition would take 9.5 months to construct, and the redesigned wood addition takes about 4 months to construct. Therefore, the redesigned addition's structure requires less than half the schedule of the steel addition. The difference in scheduling is reasonable because the pre-fabricated panelized system can be installed much more quickly than the typical deck on beam and joist system. A 5 month reduction in the construction time of the structural system is significant when considering the original timeline of 19 months. A shorter schedule can have

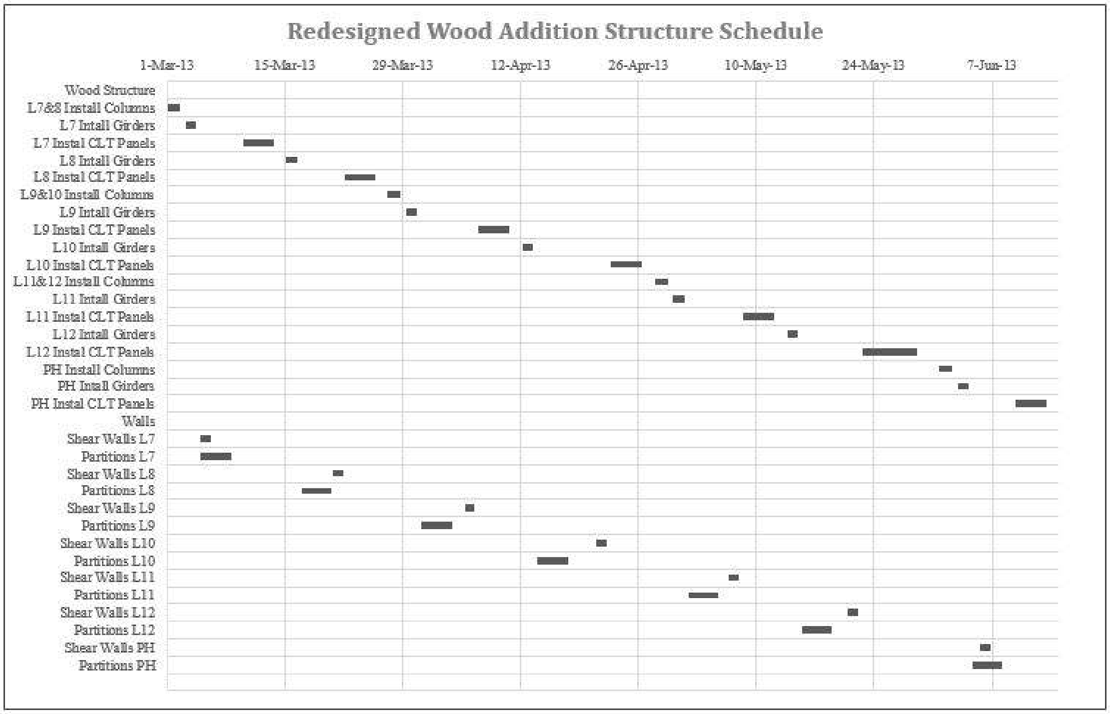


Figure 4.2: Total scheduling time required for construction of existing addition

desirable benefits because the building can be open to use and can begin bringing in profit for the owner sooner.

4.3 Cost Analysis

The cost analysis includes a unit cost estimate for both the existing and redesigned addition and a comparison between the two. The addition gravity system structure, partitions, and lateral system were included in the cost analysis. There may be some differences in cost due to the removal of drop ceilings, changes to the fire protection systems, an alternate mechanical system due to lack of concealed spaces, and more, but the scope of the breadth will focus directly on the structural costs. The cost analysis will also account for differences in general conditions costs due to schedule differences.

4.3.1 Existing Addition

A unit cost method was used to estimate the cost in which each item was listed, quantified, and priced for material and labor costs. The existing steel addition cost estimate included the items and quantities shown in table 4.1.

Cost data for the existing addition was all directly used from RS Means. The subtotal was found for all components within the scope of the construction management breadth. An estimated 5 percent waste was added to materials that require in-field cutting or experience damage during shipping and handling, including steel deck, welded wire fabric, and masonry units. General conditions was estimated to be an additional 10 percent of the subtotal based on RS Means suggestions. Sales tax was taken as 5.75 percent based on the Maryland state tax, and other costs were added as per the recommendations in RS Means. The cost table for the existing steel addition is shown in table 4.1. The total cost of the renovation and addition to 11141 Georgia Ave is known to be approximately 44 million dollars.

Project Name: 11141 Georgia Ave Existing Addition							
Location:	Wheaton Ave, Maryland						
Line Number	Description	Qty	Unit	Material	Labor	Estimate Total	
05 12 23.75 0900	W 10x49	3058	L.F.	$ 218,922.22	$ 15,595.80		
05 12 23.75 1300	W 12x22	2016	L.F.	$ 64,512.00	$ 6,431.04		
05 12 23.75 0740	W10x33	4788	L.F.	$ 229,824.00	$ 24,418.80		
05 12 23.75 1520	W12x35	625	L.F.	$ 31,875.00	$ 2,168.75		
05 12 23.75 2700	W16x26	310	L.F.	$ 11,780.00	$ 871.10		
05 12 23.75 1520	W14x22	630	L.F.	$ 23,940.00	$ 1,789.20		
05 21 19 10 0160	Open Web Joist 12K3	16200	L.F.	$ 76,464.00	$ 63,342.00		
05 21 19 10 0200	Open Web Joist 16K3	1100	L.F.	$ 5,720.00	$ 2,475.00		
05 31 13.50 5140	Floor Decking, Composite decking, 1.5" deep, 20 ga.	77040	S.F.	$ 180,504.72	$ 36,208.80		
05 31 13.50 2100	Roof Decking, under 50 squares, 1.5" deep, 22 ga.	4300	S.F.	$ 9,318.10	$ 1,720.00		
05 05 21.90 2010	Weld, 4 passes, 1/2" thick plus avg 150% (half over head)	522	L.F.	$ 872.78	$ 15,111.90		
05 05 21.90 2010	Weld, 4 passes, 1/2" thick + 20% for vertical	1380	L.F.	$ 2,307.36	$ 31,960.80		
05 05 23.10 2200	3/4" diameter bolts 2" long	8330	Ea	$ 13,119.75	$ 28,405.30		
05 12 23.78 0320	Angles, 3"x3"	1960	L.F.	$ 3,743.60	$ 3,214.40		
03 22 11.10 0200	Welded Wire Fabric 6x6 W2.1xW2.1	813.4	C.S.F	$ 15,389.53	$ 21,148.40		
03 30 53.40 3250	Elevated Slab, regular 4000 psi conc., 2-1/2" floor fill	81340	S.F.	$ 81,421.34	$ 69,139.00		
05 41 13.30 5190	Framing, stud walls, 10' high, 6" wide, studs 12" O.C.	4630	L.F.	$ 74,080.00	$ 66,672.00		
Division 05	Subtotal			$ 1,043,794.40	$ 390,672.29	$ 1,434,466.69	Division 05
04 22 10.34 5600	8" CMU solid grouted reinforced altenate cources	8860	S.F.	$ 33,579.40	$ 42,085.00		
03 21 11.60 0700	Reinforcing in place, walls, #3 to #7	2.78	Ton	$ 2,780.00	$ 1,501.20		
Division 03/04	Subtotal			$ 36,359.40	$ 43,586.20	$ 79,945.60	Division 03/04
		Subtotal		$ 1,080,153.80	$ 434,258.49	$ 1,514,412.29	Subtotal
Division 01	General Requirements @10%			$ 108,015.38	$ 43,425.85		Gen. Requirements
		Estimate Subtotal		$ 1,188,169.18	$ 477,684.34	$ 1,665,853.52	Estimate Subtotal
		Sales Tax @ 5.75%		$ 68,319.73			Sales Tax
		Subtotal A		$ 1,256,488.91	$ 477,684.34		Subtotal
		GC O & P		$ 125,648.89	$ 262,248.70		GC O&P
		Subtotal B		$ 1,382,137.80	$ 739,933.04	$ 2,122,070.84	Subtotal
		Contingency @5%				$ 106,103.54	Contingency
		Sutotal C				$ 2,228,174.38	Subtotal
		Location Adjustment Factor		97.2		-$ 62,388.88	Location Adjustment
		Grand Total				$ 2,165,785.50	Grand Total

Table 4.1: Cost analysis of existing steel addition

4.3.2 Redesigned Wood Addition

The unit cost method was also used in the wood addition to be consistent. The items and their quantities used in the cost estimate are shown in table 4.2.

Most elements could be found in RS Means, however, a slightly more involved process was used to determine the cost values for the CLT components of the building. Cost data from Structurlam, a CLT manufacturer in the Canada and the US, was provided in Canadian dollars in a presentation discussing how to develop a CLT project (Green, 2012). The current exchange rate of 0.79 US cents for every Canadian dollar was used to convert that amount to US dollars. Since all elements are pre-fabricated, no factors were added to the material values for waste. Costs were also included for soundproofing the walls and floors since this will add a significant cost over the steel addition, which doesn't require the same level of sound insulation. General conditions was taken to be 5 percent based on RS Means recommendations and the reduced schedule time for the wood addition compared to the steel addition. All other cost additions are the same as mentioned before in the existing addition cost analysis. The cost table for the redesigned wood addition is shown in table 4.2.

Project Name: 11141 Georgia Ave Wood Addition							
Location:	Wheaton Ave, Maryland						
Line Number	Description	Qty	Unit	Material	Labor	Estimate Total	
From Structurlam	5-ply CLT Panels (including visual grading)	59400	S.F.	$ 571,558.68	$ 11,731.50		
Products Budget	7-ply CLT Panels (including visual grading)	18580	S.F.	$ 246,153.41	$ 3,669.55		
Pricing Provided in a	9-ply CLT Panels (including visual grading)	1560	S.F.	$ 25,301.17	$ 542.26		
CLT Presentation	3-ply Partitions	42390	S.F.	$ 261,207.18	$ 8,372.03		
07 21 16.20 1320	Blanket Insulation, mineral wool batts 3.5" thick	38405	S.F.	$ 23,043.20	$ 8,833.23		
06 11 10.40 6125	Studs 2" x 3", pneumatic nailed	56	MBF	$ 42,367.50	$ 57,902.25		
09 81 16.10 4200	Sound Attenuation for Floor	79540	S.F.	$ 132,036.40	$ 192,486.80		
06 18 13.20 8138	Straight Glulam Beam, 20' span, 6.75" x 15" (Typ.)	180	Ea	$ 86,400.00	$ 11,610.00		
06 18 13.20 8142	Straight Glulam Beam, 20' span, 6.75" x 18" (Perim.)	132	Ea	$ 75,900.00	$ 8,844.00		
06 18 13.20 4400	Alternate Pricing, columns including hardware	26.07	MBF	$ 79,513.50	$ 24,375.45		
Division 06	Subtotal			$ 1,543,481.05	$ 328,367.06	$ 1,871,848.11	Division 06
03 30 53.40 4200	Wall, free-standing, 8" thick	714	C.Y.	$ 108,528.00	$ 138,516.00		
03 21 11.60 0700	Reinforcing in place, walls, #3 to #7	7.335	Ton	$ 7,335.00	$ 3,960.90		
Division 03/04	Subtotal			$ 115,863.00	$ 142,476.90	$ 258,339.90	Division 03/04
		Subtotal		$ 1,659,344.05	$ 470,843.96	$ 2,130,188.01	Subtotal
Division 01	General Requirements @5%			$ 82,967.20	$ 23,542.20		Gen. Requirements
		Estimate Subtotal		$ 1,742,311.25	$ 494,386.16	$ 2,236,697.41	Estimate Subtotal
		Sales Tax @ 5.75%		$ 100,182.90			Sales Tax
		Subtotal A		$ 1,842,494.15	$ 494,386.16		Subtotal
		GC O & P		$ 92,124.71	$ 271,418.00		GC O&P
		Subtotal B		$ 1,934,618.86	$ 765,804.16	$ 2,700,423.01	Subtotal
		Contingency @5%				$ 135,021.15	Contingency
		Sutotal C				$ 2,835,444.17	Subtotal
		Location Adjustment Factor		97.2		-$ 79,392.44	Location Adjustment
		Grand Total				$ 2,756,051.73	Grand Total

Table 4.2: Cost analysis of redesigned wood addition

4.3.3 Cost Comparison

The cost estimates of both the redesign and the existing building were compared to each other and to the total addition and renovation cost of 40 million dollars. The estimated cost of the existing steel structure was $2.17 million, and the estimated cost of the redesigned wood structure was $2.76 million, which is about a 30 percent increase. This is fairly significant in the context of the structural system, although the increase results in approximately a 1.5 percent increase in the total addition cost, from $44 million to $44.59 million. The average costs per square foot of the steel system and wood redesign are $19.04 and $25.06 respectively when considering the subtotal cost of the project. Many published design case studies have found that tall CLT buildings can be cost competitive within 5 percent of the total project costs with equivalent designs using other materials. This is consistent with the Georgia Ave redesign, and several factors explain the difference in cost in the 11141 Georgia Ave case study.

As a renovation and addition project, there is limited design freedom, and as a result, there are limitations which dictate the structural system design. The existing building shape caused high wind loads on the addition, thus requiring a concrete shear wall lateral system, dramatically increasing the cost of the lateral system. Furthermore, since the existing building's foundations were adequate for the steel addition, they also worked for the wood design. This meant that although normally a wood building would likely benefit significantly from the lightweight structure and have smaller foundation sizes, the redesigned addition was not able to take advantage of its low weight. Floor to ceiling heights were greater in the wood design than in the steel design. Although this was not explored within this thesis, it presents the option to reduce the floor to floor heights, thus reducing the total enclosures cost, wind loading, and column size requirements.

Even with several factors, the wood design was not that much more expensive than the steel addition, due to simpler connections and a quicker construction schedule.

The case study of the 11141 Georgia Ave addition suggests that a wood addition structural system is cost competitive with a steel addition in the context of the overal project, but not when looking at the structural system itself. However, more design studies should be completed before making generalizations. Other loading requirements and building configurations will significantly effect the cost of a wood structure alternative. Furthermore, if CLT buildings in the US are successful enough to cause an increase in CLT construction, the cost of material and labor will go down as the amount of suppliers increases and the construction industry becomes more familiar with the construction process. Therefore, although currently a wood addition is more expensive in the case of the Georgia Ave apartment building, it is possible that prices will eventually decrease.

4.4 Conclusions

The schedule analysis determined that the redesigned wood structure can be built in half the time of the steel structure, while the cost analysis found that the wood system is about 30 percent greater than the cost of the steel structure. Therefore, the wood design is feasible with regards to cost and schedule when looking at the design holistically. Although the wood system is more expensive, it only adds approximately 1.5 percent to the total project cost, and depending on the owner's needs this could potentially be worth it. The wood addition provides a reduced schedule time, the opportunity to market the apartment building as more sustainable, and higher ceilings resulting in apartments which feel larger and slightly more pleasing to occupants. If the exposed wood approach had been used, it may have been even more desirable to invest the extra cost into the project since it would add a unique look to the apartments.

5 Mechanical Breadth

5.1 Introduction

Since no concealed spaces are allowed in heavy timber construction, the ductwork, wiring, and other mechanical systems which are normally hidden above a drop ceiling will be exposed. This is an important difference between the proposed wood redesign and the existing steel structure with drop ceilings. Therefore, it is important for the mechanical equipment to be arranged aesthetically such that the apartments are just as appealing as in typical competing apartment buildings. The mechanical breadth will determine the changes that need to be made for aesthetic purposes and will look in detail at one instance of an equipment location change and how that would affect cost and the overall system.

5.2 Exposed Mechanical Systems

Mechanical systems are typically concealed in walls and floor spaces. The benefits behind this practice include; the ability to focus on the quality of the system rather than its looks, fewer interior surfaces to clean, acoustical insulation from the sound of moving water and air, and more control over the interior space design and aesthetics. Leaving the mechanical equipment exposed challenges those benefits, however some benefits of exposed equipment includes a potential visual interest and the ability to easily detect and repair leaks. (Walter T. Grondzik, 2010) Since the aesthetics of the concealed mechanical equipment is of interest in the redesign and aesthetics can be somewhat subjective, design guidelines should be chosen to help make design decisions.

5.2.1 Case Studies

In order to determine the guidelines which will be used to make design decisions, a few case studies of apartments with exposed mechanical equipment follow. The apartment in figure 5.1 shows exposed mechanical, electrical, and plumbing equipment. The mechanical duct work is kept simple, following a straight line. It it tucked to the edge of the furthest room seen, and although it passes through the middle of the room closest to the viewer, it is placed along the imaginary border between the living room and kitchen space. The electrical equipment is also fairly simple and takes only ninety degree turns when required. All equipment is painted white to blend into the white ceiling.

Figure 5.1: Example of exposed mechanical equipment. Source: Houzz.com

Figure 5.2 shows an apartment with exposed mechanical and electrical equipment. As with the previous example, the mechanical duct work is simple and follows a straight path. The electrical conduit is also fairly simple, using straight lines and right angles where necessary, and using the beam as a location for the equipment. Contrary to the previous example, the equipment has kept its original finishes and has not been painted. Here, it blends in with the gray ceiling, such that it works aesthetically. The decision to leave the equipment bare and unpainted can be a design choice which makes a space look more industrial, so the choice will generally be a matter of taste and preference.

Figure 5.2: Example of exposed mechanical equipment. Source: Houzz.com

5.2.2 Design Guidelines

From the previous case study examples, certain design guidelines have been chosen to organize the mechanical equipment aesthetically. It seems that no matter that aesthetic intent, it is important for the equipment to be kept as simple and straight as possible, and to make turns using ninety degree turns. This guideline keeps the equipment from looking cluttered or disorganized.

Additionally, the appearance of the duct work should be considered. Figure 5.3 shows a sample apartment at 11141 Georgia Ave. The apartment pictured shows clearly that the design intends for the apartments to look and feel relatively upscale and open. Buildings constructed using CLT have the option to leave the wood exposed due to the inherent fire resistance of the CLT floor panels. However, the structural redesign of this thesis chose to encapsulate the wood in drywall to provide better performance in a fire. Therefore, the mechanical systems in the addition will be painted white to blend in with the ceiling and keep the apartment space feeling clean and open.

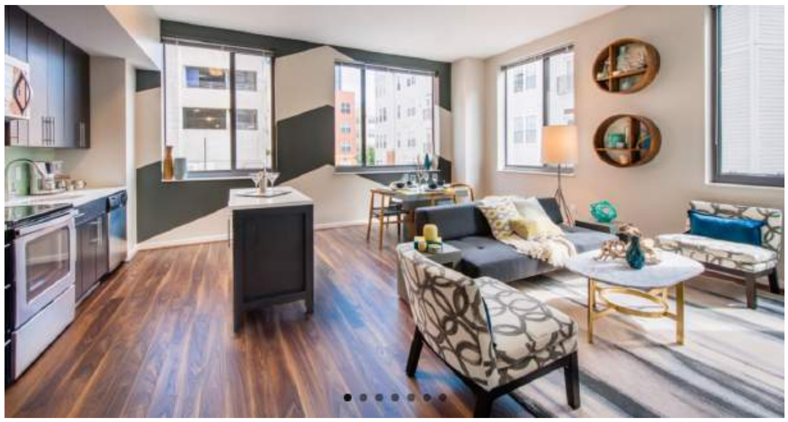


Figure 5.3: Sample apartment at 11141 Georgia Ave. Source: The George Apartments online gallery.

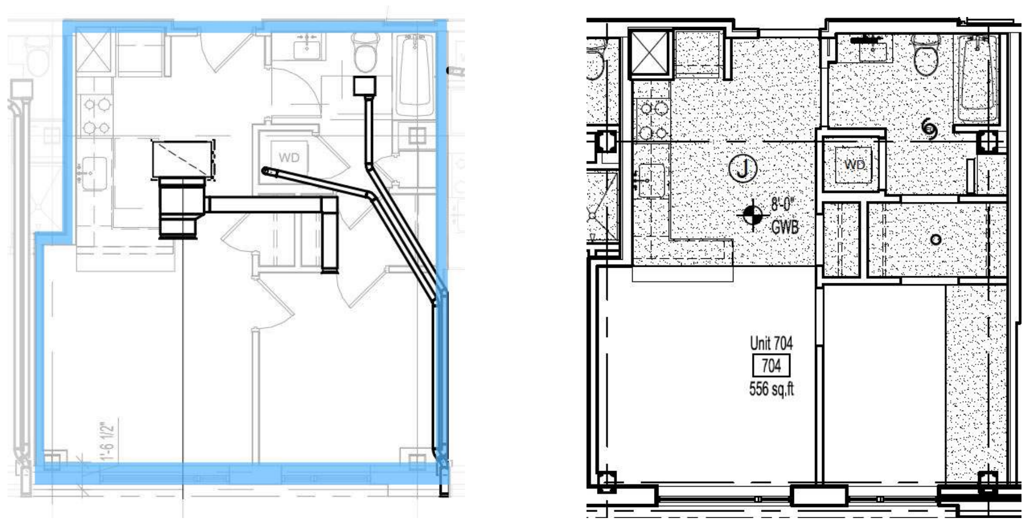
It should be noted that had the structural design relied only on the char method and left the wood CLT ceiling exposed, the decision would be to not paint the equipment to add visual interest complementary to the wood ceiling. The difference in decisions regarding the appearance of the mechanical equipment is primarily due to the drastic difference in the feel of a wood ceiling versus a white drywall ceiling. Where a wood ceiling makes a space feel small and warm, a white ceiling makes it feel large and clean. Therefore, the aesthetic decisions in this thesis are driven by the structural decision to encapsulate the wood in drywall.

5.2.3 Mechanical Layout Redesign

For the mechanical breadth, the layout redesign will consider the layout of the mechanical equipment only, rather than including the electrical and plumping equipment as well. The previously noted design examples and the guidelines derived from them will be used in creating a new layout. A mechanical layout will be completed specifically for a single typical room.

Existing Mechanical Layout

The existing mechanical layout includes a few key features. Cooling for the building comes from condensing units located on the roof, and heating comes from electrical heaters and heat pumps. Water is piped from the cooling and heating units to each apartment, which has an air handling unit with duct work, as well as exhaust ducts leading from the laundry room and bathroom to the exterior. A typical apartment is shown in figure 5.4 (a). The gypsum ceiling in the existing building is typically installed up against the steel in most living room and bedroom areas. The ceiling is lowered to allow room for mechanical equipment in the bathroom and kitchen areas, as shown in a typical apartment plan in figure 5.4 (b).



(a) *Single apartment mechanical plan outlined in blue. Source: Mechanical Drawings*

(b) *Shaded portions show where ceiling is lowered to leave room for equipment. Source: Architectural Drawings*

Figure 5.4: Typical Apartment Layouts showing mechanical equipment and lowered ceiling

New Mechanical Layout

The new gravity structural system will have higher ceilings overall, but exposed girders. Therefore, in figure 5.4 (a), there will be a girder along the column line at the right side of the room, passing through the edge of the bedroom. The exhaust ductwork passes diagonally through some of the rooms and across the girder, so changes will be made to those ducts in the new layout. The new layout is shown in figure 5.5.

In the new typical layout, the exhaust ducts now each follow a path perpendicular over to the girder. There, they will follow the girder line, tucked between the girder and the wall until they reach the exterior. The existing air handling unit is fairly large and bulky, and would likely be noisier without a drop ceiling. Therefore, a new mechanical system was chosen in order to meet the design guidelines determined previously.

A Variable Refrigerant Volume (VRV) system was chosen due to the small sizes of its piping and equipment. Rather than using water or air to circulate cooled or heated material, this system uses refrigerant, and is a more efficient use of space. It is also a system which can be controlled on an individual apartment basis. The VRV system would have a similar layout to the existing system as condensing units are placed on the roof, and the chilled refrigerant is piped similar to the water system to individual VRV unts in the apartments. Shown in figure 5.6 is a figure of a

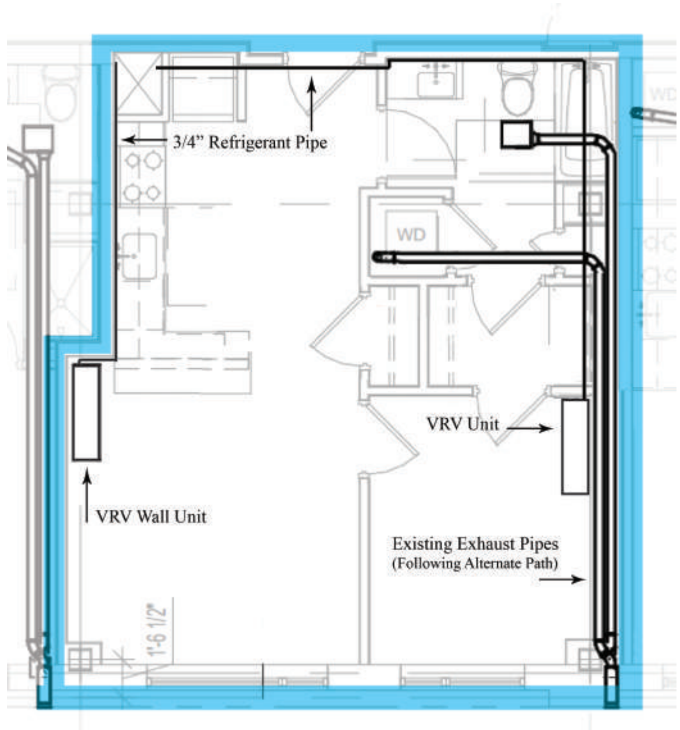


Figure 5.5: New Mechanical Layout; Typical Apartment

typical VRV system layout, and the various available VRV units. The pipe size required for the Georgia Ave apartments is 3/4".

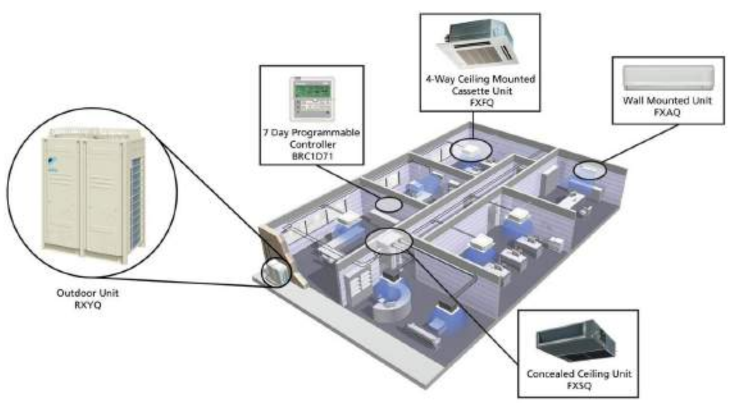
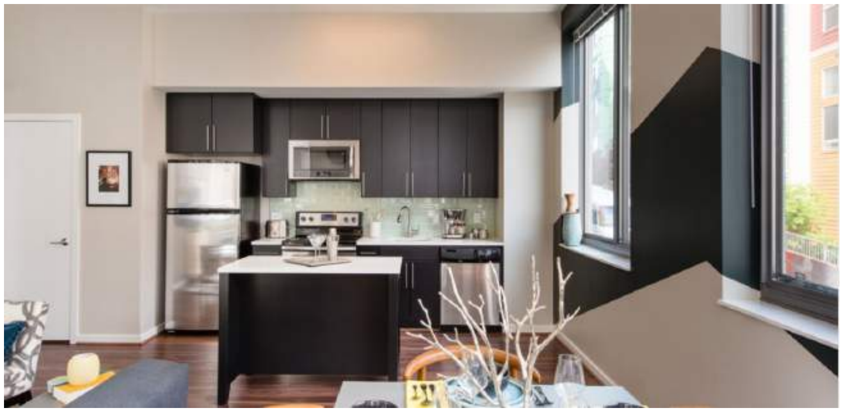


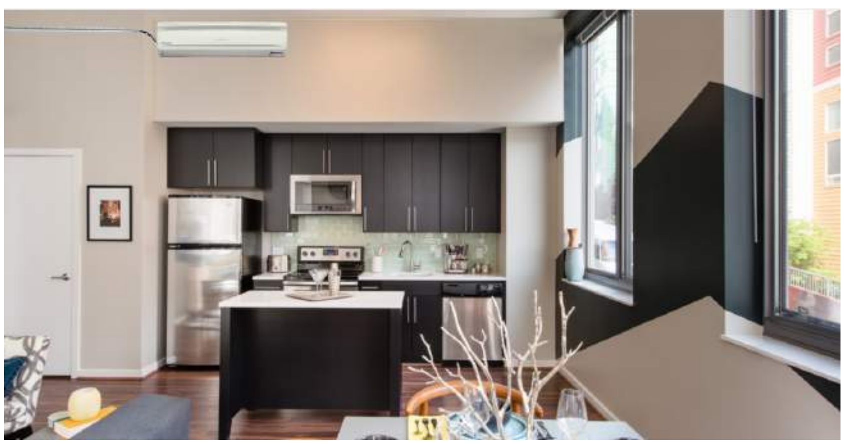
Figure 5.6: Figure of typical VRV system layout possibilities. Source:

The existing system did not supply outside air to the apartments, and thus not additional outdoor supply air was provided in the redesign. The design used operable windows in the apartments to achieve outdoor air requirements.

The new mechanical system with therefore have a VRV wall unit. Figure 5.7 shows an existing apartment photo in (a) and a modified image proposing a sample apartment in (b). The new apartment figure shows that some mechanical items will be seen, however they will be relatively small, sleek, and similar in color to their surroundings. The redesigned apartment in figure 5.7 (b) also shows the slightly raised ceilings due to the redesigned structure. This helps place the mechanical equipment even higher up so that it is not quite as noticeable.



(a) *Sample Apartment before ceiling height and mechanical changes*



(b) *Sample Apartment after ceiling height and mechanical changes*

Figure 5.7: Comparison of effect of higher ceilings and exposed mechanical systems

5.3 Conclusions

The mechanical breadth studied the implications of having no concealed spaces and leaving the mechanical equipment exposed. The existing mechanical layout was altered such that being exposed, it is relatively sleek and aesthetically pleasing and comparable to a typical equivalent apartment. A VRV system was used for its efficiently and slimness, and sidewall units were provided in the apartments in the living room and bedroom so they can by controlled individually. Hand calculations for pipe sizings have been provided in Appendix D.

6 Conclusions

6.1 Summary

The redesign of the addition to 11141 Georgia Ave included the design of a wood gravity system and a concrete shear wall lateral system. The gravity system design included the design of glulam columns and girders as well as cross laminated timber floor panels which spanned approximately 20' between girders. The gravity system elements were checked for strength, deflections, and fire performance based on the encapsulation method, with deflections controlling most sizing requirements. Concrete shear walls were designed for the lateral system, resulting in a typical 8" concrete wall which was adequate with 2 curtains typical of number 4 bars and number 6's at the ends for flexure. The lateral layout was controlled by the semi-rigid behavior of the wood floor diaphragm and serviceability.

The schedule analysis determined that the redesigned wood structure can be built in half the time of the steel structure, while the cost analysis found that the wood system is about 30 percent greater than the cost of the steel structure adding 1.5 percent to the total project cost. Therefore, the wood design is feasible with regards to cost and schedule when looking at the design holistically, and depending on the owner's needs the redesigned addition could be a feasible alternate. The mechanical breadth studied the implications of having no concealed spaces and leaving the mechanical equipment exposed. The existing mechanical layout was altered such that being exposed, it is relatively sleek and aesthetically pleasing and comparable to a typical equivalent apartment. A VRV system was used for its efficiently and slimness, and sidewall units were provided.

6.2 Heavy Timber in the Redesign

The use of wood in the redesign has several significant effects on the design work, some, but not all, of which were fully explored in the work of this thesis. There are both challenges and benefits to a taller wood building. Challenges in the redesign work included conceptual connection design for the encapsulation method and lateral design of a system with a semi-rigid diaphragm. The challenges of using wood to build include design for fire safety, water protection of the wood elements during construction, sound and vibration performance, and more. Benefits in the work of the thesis included achieving a more lightweight structure, a faster schedule, and higher ceilings. The benefits of using wood in construction include the sustainable nature of wood, architectural interest, and a competitive schedule. Although the wood alternate addition may be more or less beneficial for the owner depending on goals relating to cost schedule, and marketing, the addition is ultimately structurally feasible and could potentially be built to meet fire safety requirements.

A Gravity System Calculations

A.1 Introduction

Included in this Appendix are all the calculations completed for both the existing gravity system and the wood redesign gravity system. These calculations are provided to show more specifically what was done to reach the design choices and conclusions.

A.2 Existing Gravity System

Calculations determining loads in the existing gravity system follow. The methods and process used for determining the gravity loads is described in chapter 1.

Tech Report 2 | Roof Loads | Samantha deVries

Roof Dead Load

Penthouse Roof: Load (psf)

Joist/Beam Allowance	10
Roof Decking	10
Roofing System	7
	$\boxed{27\text{ psf}}$

12th Floor Terrace:

Concrete/Deck	37
Joist/Beam Allowance	10
4" rigid insulation	3
Drop Ceiling	5
MEP	15
Sprinklers	3
Pavers or Tiles	25
	$\boxed{98\text{ psf}}$

Roof Live Load

Penthouse Roof:

Code minimum is 20 psf
(Table 4-1: Ordinary flat roofs)

Use $\boxed{30\text{ psf}}$ (value used in design)

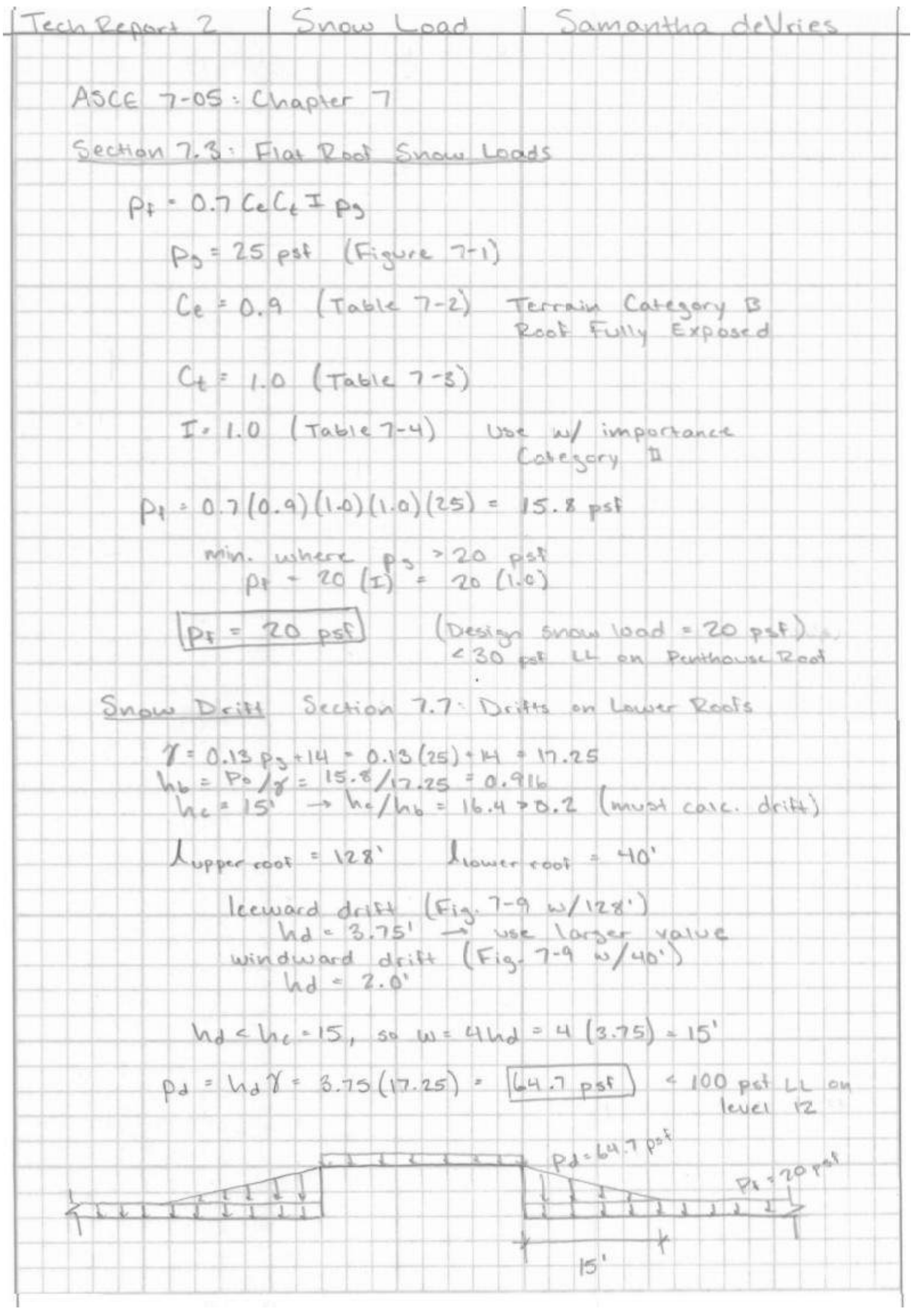
12th Floor Terrace:

Table 4-1: Roofs used for assemby purposes

Use $\boxed{100\text{ psf}}$ (same as design value)

* Note: drawings indicate that snow load must be used instead as the live load where it is the larger value

Figure A.1: Roof Load Calculations



Tech Report 2 | Snow Load | Samantha deVries

ASCE 7-05 : Chapter 7

Section 7.3 : Flat Roof Snow Loads

$p_f = 0.7\, C_e C_t I\, p_g$

$p_g = 25$ psf (Figure 7-1)

$C_e = 0.9$ (Table 7-2) Terrain Category B, Roof Fully Exposed

$C_t = 1.0$ (Table 7-3)

$I = 1.0$ (Table 7-4) Use w/ importance Category II

$p_f = 0.7(0.9)(1.0)(1.0)(25) = 15.8$ psf

min. where $p_g > 20$ psf
$p_f = 20\,(I) = 20\,(1.0)$

$\boxed{p_f = 20 \text{ psf}}$ (Design snow load = 20 psf) < 30 psf LL on Penthouse Roof

Snow Drift Section 7.7 : Drifts on Lower Roofs

$\gamma = 0.13\, p_g + 14 = 0.13(25) + 14 = 17.25$
$h_b = p_f / \gamma = 15.8 / 17.25 = 0.916$
$h_c = 15' \rightarrow h_c / h_b = 16.4 > 0.2$ (must calc. drift)

$l_{upper\ roof} = 128'$ $l_{lower\ roof} = 40'$

leeward drift (Fig. 7-9 w/ 128')
$h_d = 3.75' \rightarrow$ use larger value
windward drift (Fig. 7-9 w/ 40')
$h_d = 2.0'$

$h_d < h_c = 15$, so $w = 4h_d = 4(3.75) = 15'$

$p_d = h_d \gamma = 3.75(17.25) = \boxed{64.7 \text{ psf}}$ < 100 psf LL on level 12

Figure A.2: Snow Load and Drift Calculations

Tech Report 2 | Floor Loads | Samantha deVries

Floor Dead Loads

Concrete Floor	Load (psf)
Drop Ceiling	5
MEP	15
Sprinklers	3
Concrete 6 1/2"	81.25
or 8" × 150 pcf	100

6 1/2" slab : $\boxed{105 \text{ psf}}$
8" slab : $\boxed{123 \text{ psf}}$

Steel Framed Floors	
Ceiling	5
MEP	15
Sprinklers	3
Beam / Joist Allowance	15
Concrete / Deck	37

$\boxed{75 \text{ psf}}$

Floor Live Loads

Area	Code Min. (psf)	Design Value
Residential	40	40
Lobbies / Stairs / Exits	100	100
Penthouse Floor	100	100
Lobby Floor	100	100
Corridors above 1st Floor	40	40
12th Floor Corridors	40	100
Parking	40	40

Note: Residential Areas also receive a 20 psf partition Allowance

Figure A.3: Floor Load Calculations

Tech Report 2 | Exterior Wall Loads | Samantha deVries

Typical Existing Building Wall Dead Load:

Applied as a line load at the edge of the slab

8" Brick Layer (assume hard brick)

130 pcf × 8/12 = 87 psf × 11' typ. = 957 plf

3/4" layer gypsum board

50 pcf × 0.75/12 × 11' = 34.4 plf

Total = $\boxed{992 \text{ plf}}$

Typical Addition Wall Dead Load:

Composite Metal Panel

5 psf × 11' = 55 plf

CMU Infill (or Brick facade w/out metal panel)

29 psf (CMU) × 11' = 319 plf or 38 psf (brick medium weight) × 11' = 418 plf

Water Membrane

2 psf × 11' = 22 plf

3/4" gypsum board = 34.4 plf

Fibrous glass insulation

1.1 psf × 11 = 12.1 plf

Total: at metal panels = $\boxed{443 \text{ plf}}$

at brick faces = $\boxed{487 \text{ plf}}$

Figure A.4: Exterior Wall Load Calculations

| Tech Report 2 | Gravity Loads | Samantha deVries |

Non-Typical Dead Loads

Floors & Roofs:

At 3/4" drop panels (7' × 7') existing building

3/4" × 150 pcf = $\boxed{9 \text{ psf}}$

Existing Building Perimeter Beams

12" × 150 pcf × 12" width (avg.) = $\boxed{150 \text{ plf}}$

16" depth = $\boxed{200 \text{ plf}}$

18" = $\boxed{225 \text{ plf}}$

24" = $\boxed{300 \text{ plf}}$

30" = $\boxed{375 \text{ plf}}$

(Note: there is a large variety of perimeter beam sizes, so this is a sample to provide a range of additional Load)

Figure A.5: Non-Typical Load Calculations

A.3 Wood Redesign

A.3.1 CLT Panel Calculations

Included below are the excel tables used to determine final CLT panel sizes. These calculations follow the methods and process described in chapter 3.

Typical CLT Floor Panel Design							
Strength Checks							
Level	**Span**	**Panel**	**FbSeff***	**D+L***	**Cd**	**M**	**Ok?**
Typical Level	20.8	5-ply	10400	76	1	4090.3	good
12th Level	20.8	7-ply	18375	140	1	7534.8	good
Penthouse Roof	20.8	5-ply	10400	66	1	3552.1	good
*9-ply would have higher FbSeff, however value was not tabulated and 7-ply value worked,							

Deflection Checks											
Level	**Span**	**Panel**	**EI**	**D**	**L**	**Defl L**	**Defl D+L**	**L limit**	**D+L limt**	**L OK?**	**D OK?**
Typical Level	20.8	5-ply	4.40E+08	36	40	0.38	1.03	0.69	1.04	good	good
12th Level	20.8	7-ply	1.09E+09	40	100	0.38	0.69	0.69	1.04	good	good
Penthouse Roof	20.8	5-ply	4.40E+08	36	30	0.28	0.97	0.69	1.04	good	good

Fire Design Check									
Level	**Span**	**Panel**	**Orig. h**	**Resid. H**	**Approx**	**FbSeff**	**D+L***	**M**	**OK?**
Typical Level	26	5-ply	9.625	7.125	5-ply	10400	43	3634	good
12th Level	26	7-ply	12.375	9.875	7-ply	18375	70	5915	good
Penthouse Roof	26	5-ply	9.625	7.125	5-ply	10400	39	3296	good
*D+L is reduced using the same assumptions as before									

Table A.1: CLT Panel Design for Typical bay

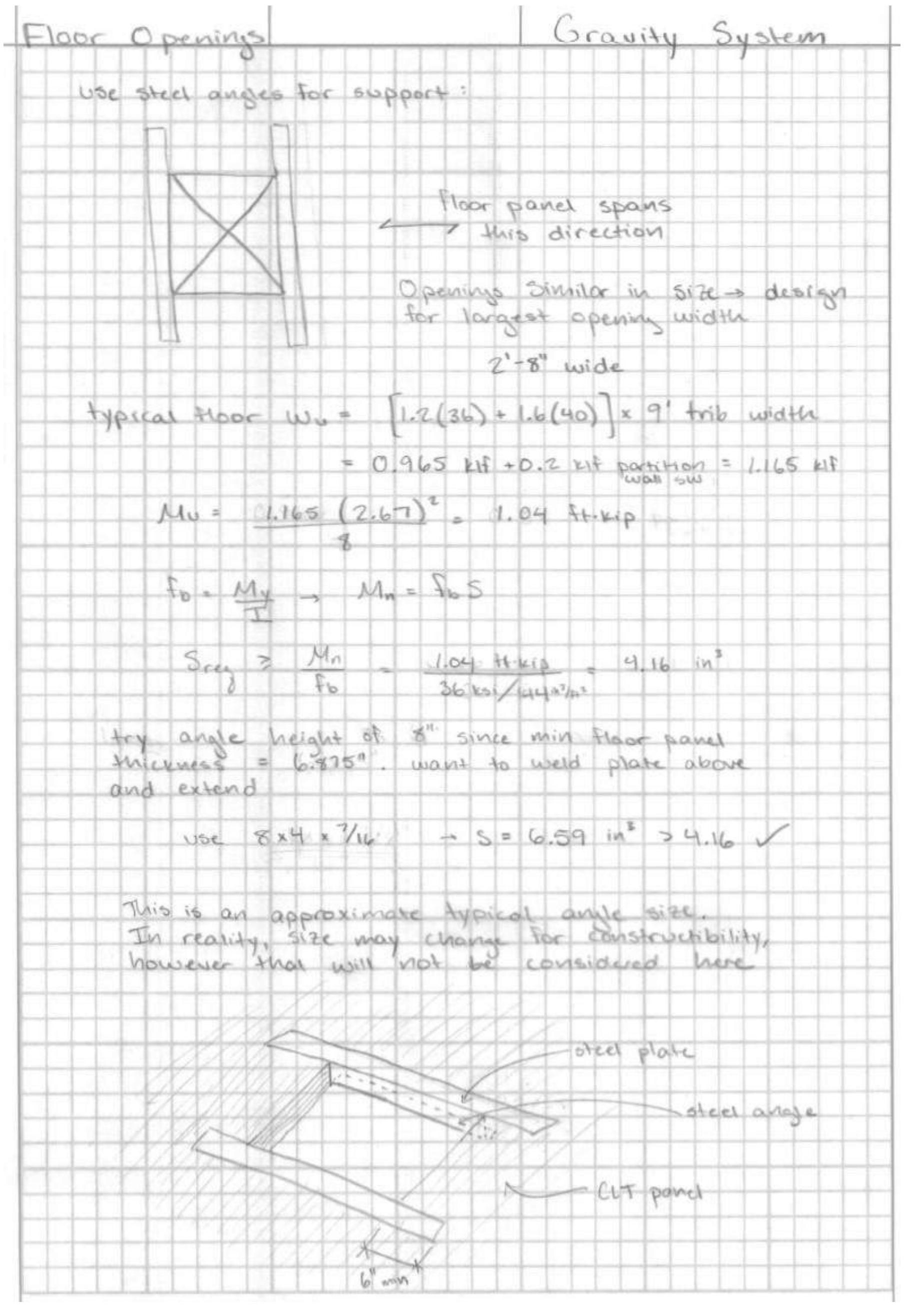
Non Typical CLT Floor Panel Design							
Strength Checks							
Level	**Span**	**Panel**	**FbSeff***	**D+L***	**Cd**	**M**	**Ok?**
Typical Level	26	7-ply	18375	80	1	6760	good
12th Level	26	9-ply	18375	144	1	12168	good
Penthouse Roof	26	7-ply	18375	70	1	5915	good
*D+L controlled over other combinations *9-ply would have higher FbSeff, however value was not tabulated and 7-ply value worked, so new FbSeff was not calculated to save time							

Deflection Checks											
Level	**Span**	**Panel**	**EI**	**D**	**L**	**Defl L**	**Defl D+L**	**L limit**	**D+L limt**	**L OK?**	**D OK?**
Typical Level	26	7-ply	1.09E+09	40	40	0.38	1.13	0.87	1.30	good	good
12th Level	26	9-ply	1.60E+09	44	100	0.64	1.21	0.87	1.30	good	good
Penthouse Roof	26	7-ply	1.09E+09	40	30	0.28	1.04	0.87	1.30	good	good

Fire Design Check									
Level	**Span**	**Panel**	**Orig. h**	**Resid. H**	**Approx**	**FbSeff**	**D+L***	**M**	**OK?**
Typical Level	26	7-ply	9.625	7.125	5-ply	10400	46	3887	good
12th Level	26	9-ply	12.375	9.875	7-ply	18375	73	6169	good
Penthouse Roof	26	7-ply	9.625	7.125	5-ply	10400	42	3549	good
*D+L is reduced using the same assumptions as before									

Table A.2: CLT Panel Design for 26' bay

Typical Opening Calculation



Floor Openings | Gravity System

use steel angles for support:

floor panel spans this direction

Openings similar in size → design for largest opening width

2'-8" wide

typical floor $w_u = \left[1.2(36) + 1.6(40)\right] \times 9'$ trib width

$= 0.965$ klf $+ 0.2$ klf partition wall sw $= 1.165$ klf

$M_u = \dfrac{1.165\,(2.67)^2}{8} = 1.04$ ft-kip

$f_b = \dfrac{My}{I} \rightarrow M_n = f_b S$

$S_{req} \geq \dfrac{M_n}{f_b} = \dfrac{1.04 \text{ ft-kip}}{36\text{ ksi} / 144 \text{ in}^2/\text{ft}^2} = 4.16 \text{ in}^3$

try angle height of 8" since min floor panel thickness = 6.875". want to weld plate above and extend

use $8 \times 4 \times 7/16 \rightarrow S = 6.59 \text{ in}^3 > 4.16$ ✓

This is an approximate typical angle size. In reality, size may change for constructibility, however that will not be considered here

steel plate

steel angle

CLT panel

6" min

Figure A.6: Typical Opening Calculations

A.3.2 Girder Calculations

	bw	bf	dc	dt	dtc	NA	I	St	Sb	EI
Normal Conditions	4	12	6.875	27	20.125	11.44	12758.8	820.1	1115.0	2.30E+10
	4	12	6.875	25.5	18.625	10.71	10549.8	713.3	985.1	1.90E+10
	4	12	6.875	24	17.125	9.98	8623.1	615.0	864.1	1.55E+10
	4	12	6.875	22.5	15.625	9.25	6958.3	525.2	752.1	1.25E+10
	4	12	6.875	21	14.125	8.53	5535.1	443.8	649.0	9.96E+09
	4	12	6.875	19.5	12.625	7.81	4333.2	370.7	554.8	7.80E+09
	4	12	6.875	18	11.125	7.10	3332.2	305.7	469.3	6.00E+09
	4	12	6.875	16.5	9.625	6.40	2511.8	248.7	392.5	4.52E+09
	4	12	6.875	15	8.125	5.71	1851.3	199.3	324.1	3.33E+09
	4	12	6.875	13.5	6.625	5.05	1330.0	157.4	263.5	2.39E+09
	4	12	6.875	12	5.125	4.42	927.0	122.2	209.9	1.67E+09
	4	12	6.875	10.5	3.625	3.85	620.3	93.2	161.3	1.12E+09
Residual Section during Fire	4	7	6.875	24.5	17.625	11.05	6676.6	496.2	604.5	1.20E+10
	4	7	6.875	23	16.125	10.32	5478.5	431.9	531.1	9.86E+09
	4	7	6.875	21.5	14.625	9.59	4438.1	372.6	462.8	7.99E+09
	4	7	6.875	20	13.125	8.87	3543.7	318.3	399.7	6.38E+09
	4	7	6.875	18.5	11.625	8.15	2783.4	268.9	341.6	5.01E+09
	4	7	6.875	17	10.125	7.44	2145.3	224.4	288.4	3.86E+09
	4	7	6.875	15.5	8.625	6.74	1617.4	184.6	240.1	2.91E+09
	4	7	6.875	14	7.125	6.05	1187.9	149.4	196.3	2.14E+09
	4	7	6.875	12.5	5.625	5.38	844.6	118.7	156.9	1.52E+09
	4	7	6.875	11	4.125	4.75	575.3	92.0	121.2	1.04E+09
	4	7	6.875	9.5	2.625	4.16	367.4	68.8	88.3	6.61E+08
	4	7	6.875	8	1.125	3.67	207.1	47.9	56.4	3.73E+08

Table A.3: Calculated Properties for Inverted T-Beam Girders

Typical Girder Redesign for Inverted T-Shape

	Level	Span	Gird. sw	Floor L+D	D+L**	M (in-lbs)	bf	bw	Depth	Cv	Sact	Sreq	OK?
Strength Design	Typical Level	20	26	700	726	435600	12	4	15	0.90	324.1	201.1	good
	12th Level	20	50	1400	1450	870000	12	4	18	0.89	469.3	409.0	good
	Penthouse Roof	20	50	660	710	426000	12	4	15	0.90	324.1	196.7	good

	Level	Span	L (plf)	D+L	EI	Defl. L	Defl. D+L	Lim. L	Lim. D+L	L OK?	D+L OK?
Defl. Design	Typical Level	20	400	726	3.33E+09	0.432	0.96	0.667	1.0	good	good
	12th Level	20	1000	1450	6.00E+09	0.600	1.00	0.667	1.0	good	good
	Penthouse Roof	20	300	710	3.33E+09	0.324	0.989	0.667	1.0	good	good

	Level	Span	D+L	Orig w	Orig h	Resid w	Resid h	Seff	Red. Load	M (in-lb)	Sreq	OK?
Fire/ Char Design	Typical Level	20	726	12	15	7	12.5	182.3	404.5	242700	70	good
	12th Level	20	1450	12	18	7	15.5	280.3	737.5	442500	130	good
	Penthouse Roof	20	710	12	15	7	12.5	182.3	427.5	256500	74	good

*Or along Grid 4 at 12th Level and Penthouse

Table A.4: Typical Girder Design for Inverted T-Shape

Typical Perimeter Girder in the E-W Direction

	Level	Span	Wall sw	Gird. Sw	D (plf)*	M (in-lbs)	bw	Depth	Cv	Sact	Sreq	OK?
Strength Design	Typical Level	21	450	50	500	330750	12	15	0.90	324.1	153.4	good
	12th Level parapet	21	200	50	250	165375	12	13	0.91	209.9	75.6	good
	12th Level penthouse	21	350	50	400	264600	12	15	0.90	324.1	122.7	good
	Penthouse Roof	21	200	50	250	165375	12	13	0.91	209.9	75.6	good

	Level	Span	D (plf)*	EI	Defl.	Defl. Lim.	OK?
Defl. Design	Typical Level	21	500	3.33E+09	0.985	1.05	good
	12th Level parapet	21	250	1.67E+09	0.983	1.05	good
	12th Level penthouse	21	400	3.33E+09	0.788	1.05	good
	Penthouse Roof	21	250	1.67E+09	0.983	1.05	good

	Level	Span	D (plf)*	Orig w	Orig h	Resid w	Resid h	Seff	Red. Load	M (in-lb)	Sreq	OK?
Fire/ Char Design	Typical Level	21	500	12	15	7	12.5	182.3	121.2	80201	23.3	good
	12th Level parapet	21	250	12	13	7	10.5	128.6	56.4	37308	10.7	good
	12th Level penthouse	21	400	12	15	7	12.5	182.3	88.3	58422	16.9	good
	Penthouse Roof	21	250	12	13	7	10.5	128.6	56.4	37308	10.7	good

*Dead Loads here include approx. girder self-weight and exterior wall load. Floor dead and live loads are assumed to be carried to the typical floor girders by the CLT panel and are not included. Therefore there is no live on carried by this girder type.

Table A.5: Non-typical Girder Design

Perimeter Girder Along Grid 2* (West side)

	Level	Span	Wall sw	Gird. sw	Floor L+D	D+L**	M (in-lbs)	bf	Depth	Cv	Sact	Sreq	OK?
Strength Design	Typical Level	20	450	50	1092	1592	955200	12	19.5	0.88	554.8	452.7	good
	12th Level parapet	20	200	50	1872	2122	1273200	12	21	0.87	649.0	607.9	good
	12th Level penthouse	20	350	50	2880	3280	1968000	12	25.5	0.86	985.1	958.0	good
	Penthouse Roof	20	200	50	962	1212	727200	12	18	0.89	469.3	341.9	good

	Level	Span	L (plf)	D+L	EI	Defl. L	Defl. D+L	Lim. L	Lim. D+L	L OK?	D+L OK?
Defl. Design	Typical Level	20	520	1592	7.80E+09	0.240	0.982	0.667	1	good	good
	12th Level parapet	20	1300	2122	9.96E+09	0.470	0.915	0.667	1	good	good
	12th Level penthouse	20	2000	3280	1.90E+10	0.379	0.743	0.667	1	good	good
	Penthouse Roof	20	390	1212	6.00E+09	0.234	0.974	0.667	1	good	good

	Level	Span	D+L	Orig w	Orig h	Resid bf	Resid h	Resid bw	Seff	Red. Load	M (in-lb)	Sreq	OK?
Fire/ Char Design	Typical Level	20	1592	12	19.5	7	17	4	240.1	1012.0	607200	179.9	good
	12th Level parapet	20	2122	12	21	7	18.5	4	341.6	1136.5	681900	203.5	good
	12th Level penthouse	20	3280	12	25.5	7	23	4	531.1	1280.0	768000	233.7	good
	Penthouse Roof	20	1212	12	18	7	15.5	4	196.3	772.5	463500	136.2	good

*Or along Grid 4 at 12th Level and Penthouse

**D+L was the controlling case for other girders, and will therefore be the only case considered in non typical giders

Table A.6: Non-typical Girder Design

Perimeter Girder Along Grid 12* (East side)

	Level	Span	Wall sw	Gird. sw	Floor L+D	D+L**	M (in-lbs)	bf	Depth	Cv	Sact	Sreq	OK?
Strength Design	Typical Level	20	450	50	840	1340	804000	12	19.5	0.88	554.8	381.0	good
	12th Level parapet	20	200	50	1512	1762	1057200	12	19.5	0.88	554.8	501.0	good
	12th Level penthouse	20	350	50	2880	3280	1968000	12	25.5	0.86	985.1	958.0	good
	Penthouse Roof	20	200	50	735	985	591000	12	18	0.89	469.3	277.9	good

	Level	Span	L (plf)	D+L	EI	Defl. L	Defl. D+L	Lim. L	Lim. D+L	L OK?	D+L OK?
Defl. Design	Typical Level	20	420	1340	7.80E+09	0.194	0.831	0.667	1	good	good
	12th Level parapet	20	1050	1762	7.80E+09	0.485	0.978	0.667	1	good	good
	12th Level penthouse	20	2000	3280	1.90E+10	0.379	0.743	0.667	1	good	good
	Penthouse Roof	20	315	985	6.00E+09	0.189	0.792	0.667	1	good	good

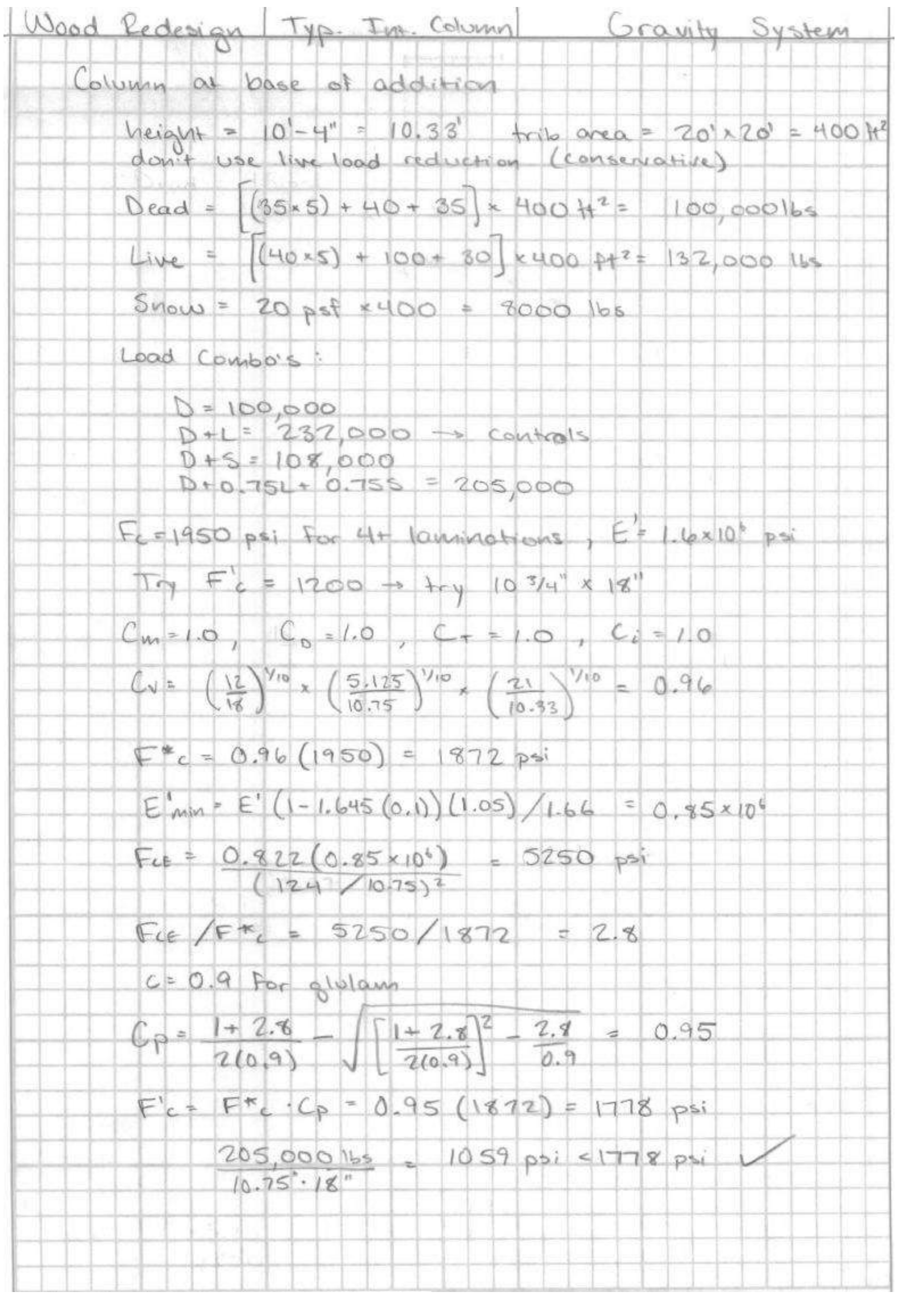
	Level	Span	D+L	Orig w	Orig h	Resid bf	Resid h	Resid bw	Seff	Red. Load	M (in-lb)	Sreq	OK?
Fire/ Char Design	Typical Level	20	1340	12	19.5	7	17	4	240.1	858.0	514800	152	good
	12th Level parapet	20	1762	12	19.5	7	17	4	288.4	954.0	572400	170	good
	12th Level penthouse	20	3280	12	25.5	7	23	4	531.1	1280.0	768000	234	good
	Penthouse Roof	20	985	12	18	7	15.5	4	196.3	628.5	377100	111	good

*Or along Grid 4 at 12th Level and Penthouse

**D+L was the controlling case for other girders, and will therefore be the only case considered in non typical giders

Table A.7: Non-typical Girder Design

A.3.3 Column Calculations



Wood Redesign | Typ. Int. Column | Gravity System

Column at base of addition

height = 10'-4" = 10.33'   trib area = 20' × 20' = 400 ft²

don't use live load reduction (conservative)

Dead = $[(35 \times 5) + 40 + 35] \times 400\ \text{ft}^2 = 100{,}000\ \text{lbs}$

Live = $[(40 \times 5) + 100 + 30] \times 400\ \text{ft}^2 = 132{,}000\ \text{lbs}$

Snow = 20 psf × 400 = 8000 lbs

Load Combo's:

D = 100,000
D+L = 232,000 → controls
D+S = 108,000
D+0.75L+0.75S = 205,000

$F_c = 1950$ psi for 4+ laminations, $E' = 1.6 \times 10^6$ psi

Try $F'_c = 1200$ → try 10 3/4" × 18"

$C_M = 1.0$, $C_D = 1.0$, $C_t = 1.0$, $C_i = 1.0$

$$C_V = \left(\frac{12}{18}\right)^{1/10} \times \left(\frac{5.125}{10.75}\right)^{1/10} \times \left(\frac{21}{10.33}\right)^{1/10} = 0.96$$

$F^*_c = 0.96(1950) = 1872$ psi

$E'_{min} = E'(1 - 1.645(0.1))(1.05)/1.66 = 0.85 \times 10^6$

$$F_{cE} = \frac{0.822(0.85 \times 10^6)}{(124/10.75)^2} = 5250\ \text{psi}$$

$F_{cE}/F^*_c = 5250/1872 = 2.8$

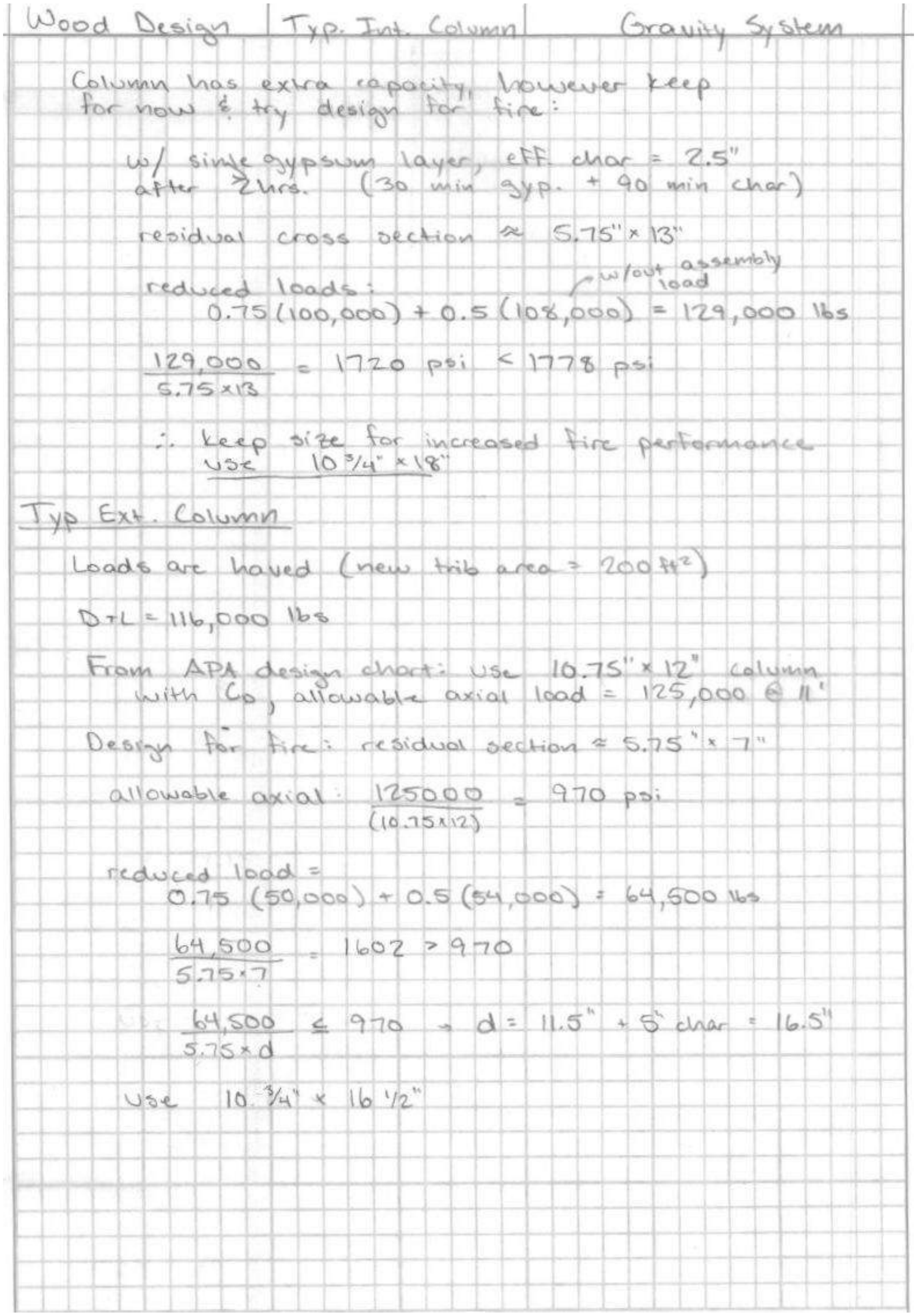
$c = 0.9$ for glulam

$$C_P = \frac{1+2.8}{2(0.9)} - \sqrt{\left[\frac{1+2.8}{2(0.9)}\right]^2 - \frac{2.8}{0.9}} = 0.95$$

$F'_c = F^*_c \cdot C_P = 0.95(1872) = 1778$ psi

$$\frac{205{,}000\ \text{lbs}}{10.75" \cdot 18"} = 1059\ \text{psi} < 1778\ \text{psi}\ ✓$$

Figure A.7: Typical Column Calculations at Base of Addition



Wood Design | Typ. Int. Column | Gravity System

Column has extra capacity, however keep for now & try design for fire:

w/ single gypsum layer, eff. char = 2.5" after 2 hrs. (30 min gyp. + 90 min char)

residual cross section ≈ 5.75" × 13"

reduced loads: (w/out assembly load)

$0.75(100{,}000) + 0.5(108{,}000) = 129{,}000$ lbs

$\frac{129{,}000}{5.75 \times 13} = 1720$ psi < 1778 psi

∴ keep size for increased fire performance
use 10 3/4" × 18"

Typ Ext. Column

Loads are halved (new trib area = 200 ft²)

D+L = 116,000 lbs

From APA design chart: use 10.75" × 12" column with C_D, allowable axial load = 125,000 @ 11'

Design for fire: residual section ≈ 5.75" × 7"

allowable axial $\frac{125000}{(10.75 \times 12)} = 970$ psi

reduced load =
$0.75(50{,}000) + 0.5(54{,}000) = 64{,}500$ lbs

$\frac{64{,}500}{5.75 \times 7} = 1602 > 970$

$\frac{64{,}500}{5.75 \times d} \leq 970 \rightarrow d = 11.5" + 5"$ char $= 16.5"$

Use 10 3/4" × 16 1/2"

Figure A.8: Typical Column Calculations at Base of Addition

Gravity Loads (psf, lbs for SW)			
Level	**Dead**	**Live**	**C. SW (per floor)**
Typical Level	36	40	415
12th Level	40	100	470
Roof	36	30	670

Floor Heights (ft)	
Typical Level	10.33
12th Level	11.67
Roof	16.75

Wood Properties	
Fc (psi)	1950
E' (psi)	1.60E+06
Cm	1
Cd	1
Ci	1
Ct	1
E'min	8.50E+05

(a) *General Column Design Information*

Column and Ext. Wall Load Information				
Col. Type*	**Trib Area**	**Wall Load (lbs)**		
		Typ. Level	**12th Level**	**Roof**
Typ. Int.	415	0	0	4150
Typ. Ext.	208	9338	10686	-
A	285	9338	10686	4150
B	130	10350	11845	-
C	335	0	0	0
D	300	0	0	0
E	475	0	0	-
F	260	10350	11845	-

(b) *Exterior Wall Load Information*

Figure A.9: General Column Design Information

Column Design: Various Levels, Strength, Fire Performance (See Design Summary for Splicing and Final Sizing Choices)																
Lev	Type*	D+L (lbs)	width	depth	Cv	F*c	Fce	Fce/F*c	Cp	F'c	fc	str ok?	red. D+L	resid. A	fc (fire)	fire ok?
Level 7	Typ. Int.	250555	12	12.375	0.98	1917	6548	3.42	0.96	1844	1687	0.915	131061	51.6	2539	0.860
	Typ. Ext.	168079	12	12.375	0.98	1917	6548	3.42	0.96	1844	1132	0.614	113571	51.6	2200	0.745
	A	231749	12	12.375	0.98	1917	6548	3.42	0.96	1844	1561	0.846	149435	51.6	2895	0.981
	B	133740	10.5	12	1.00	1948	5013	2.57	0.95	1842	1061	0.576	99434	38.5	2583	0.877
	C	199525	10.5	12	1.00	1948	5013	2.57	0.95	1842	1584	0.860	102911	38.5	2673	0.907
	D	179015	10.5	12	1.00	1948	5013	2.57	0.95	1842	1421	0.771	92411	38.5	2400	0.815
	E	249545	12	12.375	0.98	1917	6548	3.42	0.96	1844	1680	0.911	125884	51.6	2438	0.826
	F	201340	12	12.375	0.98	1917	6548	3.42	0.96	1844	1356	0.735	133364	51.6	2583	0.875
Level 8	Typ. Int.	218600	10.5	12	1.00	1948	5013	2.57	0.95	1842	1735	0.942	112905	38.5	2933	0.995
	Typ. Ext.	142518	10.5	12	1.00	1948	5013	2.57	0.95	1842	1131	0.614	94978	38.5	2467	0.837
	A	200336	12	12.375	0.98	1917	6548	3.42	0.96	1844	1349	0.731	127531	51.6	2470	0.837
	B	113095	10.5	12	1.00	1948	5013	2.57	0.95	1842	898	0.487	83183	38.5	2161	0.733
	C	173650	10.5	12	1.00	1948	5013	2.57	0.95	1842	1378	0.748	88195	38.5	2291	0.777
	D	155800	10.5	12	1.00	1948	5013	2.57	0.95	1842	1237	0.671	79200	38.5	2057	0.698
	E	213030	10.5	12	1.00	1948	5013	2.57	0.95	1842	1691	0.918	105148	38.5	2731	0.927
	F	170815	10.5	12	1.00	1948	5013	2.57	0.95	1842	1356	0.736	111523	38.5	2897	0.983
Level 9	Typ. Int.	186645	10.5	12	1.00	1948	5013	2.57	0.95	1842	1481	0.804	94749	38.5	2461	0.835
	Typ. Ext.	116958	10.5	12	1.00	1948	5013	2.57	0.95	1842	928	0.504	76385	38.5	1984	0.673
	A	168924	10.5	12	1.00	1948	5013	2.57	0.95	1842	1341	0.728	105628	38.5	2744	0.931
	B	92450	8.5	12	1.02	1990	3285	1.65	0.89	1780	906	0.509	66931	24.5	2732	0.959
	C	147775	10.5	12	1.00	1948	5013	2.57	0.95	1842	1173	0.637	73479	38.5	1909	0.648
	D	132585	8.5	12	1.02	1990	3285	1.65	0.89	1780	1300	0.730	65989	24.5	2693	0.946
	E	176515	10.5	12	1.00	1948	5013	2.57	0.95	1842	1401	0.761	84411	38.5	2193	0.744
	F	140290	10.5	12	1.00	1948	5013	2.57	0.95	1842	1113	0.605	89681	38.5	2329	0.791
Level 10	Typ. Int.	154690	10.5	12	1.00	1948	5013	2.57	0.95	1842	1228	0.667	76593	38.5	1989	0.675
	Typ. Ext.	91397	8.5	12	1.02	1990	3285	1.65	0.89	1780	896	0.503	57792	24.5	2359	0.828
	A	137511	10.5	12	1.00	1948	5013	2.57	0.95	1842	1091	0.593	83724	38.5	2175	0.738
	B	71805	8.5	12	1.02	1990	3285	1.65	0.89	1780	704	0.396	50680	24.5	2069	0.726
	C	121900	8.5	12	1.02	1990	3285	1.65	0.89	1780	1195	0.672	58763	24.5	2398	0.842
	D	109370	8.5	12	1.02	1990	3285	1.65	0.89	1780	1072	0.602	52778	24.5	2154	0.757
	E	140000	8.5	12	1.02	1990	3285	1.65	0.89	1780	1373	0.771	63675	24.5	2599	0.913
	F	109765	8.5	12	1.02	1990	3285	1.65	0.89	1780	1076	0.605	67840	24.5	2769	0.972
Level 11	Typ. Int.	122735	8.5	12	1.02	1990	3285	1.65	0.89	1780	1203	0.676	58436	24.5	2385	0.838
	Typ. Ext.	65837	8.5	12	1.02	1990	3285	1.65	0.89	1780	645	0.363	39200	24.5	1600	0.562
	A	106099	8.5	12	1.02	1990	3285	1.65	0.89	1780	1040	0.584	61820	24.5	2523	0.886
	B	51160	8.5	12	1.02	1990	3285	1.65	0.89	1780	502	0.282	34429	24.5	1405	0.493
	C	96025	8.5	12	1.02	1990	3285	1.65	0.89	1780	941	0.529	44046	24.5	1798	0.631
	D	86155	8.5	12	1.02	1990	3285	1.65	0.89	1780	845	0.475	39566	24.5	1615	0.567
	E	103485	8.5	12	1.02	1990	3285	1.65	0.89	1780	1015	0.570	42939	24.5	1753	0.615
	F	79240	8.5	12	1.02	1990	3285	1.65	0.89	1780	777	0.437	45999	24.5	1878	0.659
Level 12	Typ. Int.	90780	8.5	10.5	1.02	1992	2574	1.29	0.84	1678	1017	0.606	40280	19.3	2092	0.779
	Typ. Ext.	40276	6.75	10.5	1.05	2039	1623	0.80	0.66	1355	568	0.419	20607	9.6	2141	0.988
	A	74686	8.5	10.5	1.02	1992	2574	1.29	0.84	1678	837	0.499	39916	19.3	2074	0.772
	B	30515	6.75	10.5	1.05	2039	1623	0.80	0.66	1355	431	0.318	18178	9.6	1889	0.871
	C	70150	8.5	10.5	1.02	1992	2574	1.29	0.84	1678	786	0.468	29330	19.3	1524	0.567
	D	62940	6.75	12	1.03	2012	1623	0.81	0.67	1349	777	0.576	26355	12.3	2151	0.997
	E	66970	6.75	12	1.03	2012	1623	0.81	0.67	1349	827	0.613	22203	12.3	1812	0.840
	F	48715	6.75	12	1.03	2012	1623	0.81	0.67	1349	601	0.446	24158	12.3	1972	0.914
Penthouse	Typ. Int.	32210	8.5	10.5	0.99	1922	1250	0.65	0.57	1102	361	0.328	20838	19.3	1082	0.614
	A	23630	8.5	10.5	0.99	1922	1250	0.65	0.57	1102	265	0.240	15768	19.3	819	0.465
	C	22780	6.75	12	1.00	1940	788	0.41	0.38	742	281	0.379	13568	12.3	1108	0.933
	D	20470	6.75	12	1.00	1940	788	0.41	0.38	742	253	0.341	12203	12.3	996	0.839

*Column Types are labeled on the following floor plan

Note: As long as "OK?" column values are less than 1.0, the size has passed design checks. (Value is ratio of fp/F'c)

Table A.8: Column Excel Calculations

B Lateral System Redesign Calculations

B.1 Introduction

Included in this Appendix are all the calculations completed for both the existing lateral system and the wood redesign lateral system. These calculations are provided to show more specifically what was done to reach the design choices and conclusions.

B.2 Existing Lateral System

Sample excel calculations determining loads in the existing lateral system follow. The methods and process used for determining the lateral loads is described in chapter 1.

B.2.1 Wind Loads

Wind Load Calculations: Wind Perpendicular to Building

ASCE 7-05, Chapter 6.5: Method 2 - Analytical Procedure

Design Procedure from Section 6.5.3

Blue boxes are input boxes, all else are determined by equations

Building Information	B	214		
	L	60		
	h	153		
	z bar	145		
	Variable	**Value**	**Units**	**Comments**
1. Determine Basic Wind Speed and Directonality Factor				
Basic Wind Speed	V	90	mph	(Fig. 6-1)
Directionality Factor	k_d	0.85		(Table 6-4)
2. Determine Importance Factor				
Occupancy Category		II		(Table 1-1)
Importance Factor	I	1		(Table 6-1)
3 & 9. Exposure Category, Velocity Pressure Exposure Coefficient, and Velocity Pressure				
Exposure Category		B		From Structural Drawings
Velocity Pressure Exposure Coefficient				

Note: Use exposure B, case 2 for MWFRS

Values determined by Interpolation

Height (ft)	K_z	q_z or q_h
8	0.570	11.82
19	0.618	12.81
30	0.700	14.52
41	0.765	15.86
51	0.814	16.88
61	0.854	17.71
73	0.902	18.70
83	0.940	19.49
94	0.972	20.16
104	1.000	20.74
114	1.025	21.25
125	1.053	21.84
136	1.080	22.39
140	1.090	22.60
153	1.116	23.14
158	1.126	23.35

4. Determine Topographic Factor			
Topographic Factor	K_z	1	Value used by structural engineering firm

Table B.1: Wind Load Excel Calculations

5. Determine Gust Effect Factor

The following is based on a flexible building (Section 6.5.8.2)

Input Variables

	b bar	0.45	(Table 6-1)
	α bar	0.25	(Table 6-1)
	ϵ bar	0.33	(Table 6-1)
	l	320.00	(Table 6-1)
	c	0.30	(Table 6-1)
	β	1.50	(C6.5.8)

Output Variables

	n_1	0.49		
	N_1	2.987	R_n	0.070
	η_h	4.012	R_h	0.218
	η_B	5.611	R_B	0.162
	η_L	5.267	R_L	0.172
	$I_{z\ bar}$	0.23	g_q	3.40
	$L_{z\ bar}$	524.125	g_r	4.02
	$V\ bar_{z\ bar}$	86.000	g_v	3.40
	Q	0.82		
	R	0.03		
Gust Effect Factor	G_f	0.83		

6. Determine the Enclosure Classification

Building is considered enclosed (Section 6.5.9)

7. Determine the Internal Pressure Coefficient

	Gc_{pi}	0.18	(Figure 6-5)
	or	-0.18	

8. Determine External Pressure Coefficients

Windward Wall	C_p	0.8	(Figure 6-6)	use with q_z
Leeward Wall	C_p	-0.5	(Figure 6-6)	use with q_h
Side Wall	C_p	-0.7	(Figure 6-6)	use with q_h
Roof (0' to 60')	C_p	-0.9	(Figure 6-6)	

Table B.2: Wind Load Excel Calculations

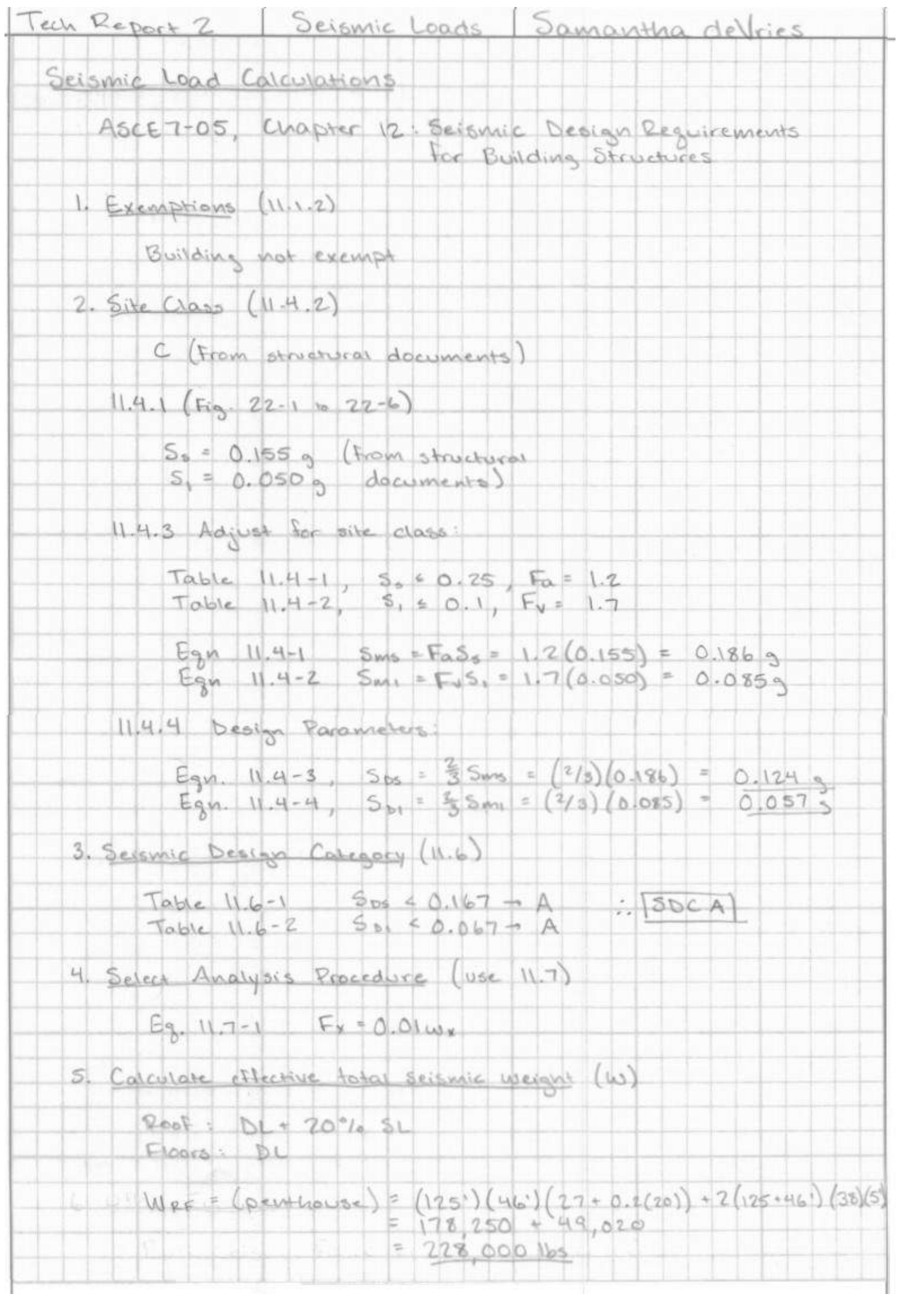
Wind Pressure Chart (Wind Perpendicular to Building)								
Location	z(ft)	q_z or q_h	C_p	G_f	Gc_{pi}	q_iGC_{pi} (psf)	Net Pressure (psf)	
							$q_zG_fC_p$-q_i(+GC_{pi})	$q_zG_fC_p$-q_i(-GC_{pi})
Windward	8	11.82	0.8	0.83	0.18	2.13	5.70	**9.95**
	19	12.81	0.8	0.83	0.18	2.31	6.17	**10.79**
	30	14.52	0.8	0.83	0.18	2.61	6.99	**12.22**
	41	15.86	0.8	0.83	0.18	2.86	7.64	**13.35**
	51	16.88	0.8	0.83	0.18	3.04	8.13	**14.21**
	61	17.71	0.8	0.83	0.18	3.19	8.53	**14.91**
	73	18.70	0.8	0.83	0.18	3.37	9.01	**15.75**
	83	19.49	0.8	0.83	0.18	3.51	9.39	**16.41**
	94	20.16	0.8	0.83	0.18	3.63	9.71	**16.97**
	104	20.74	0.8	0.83	0.18	3.73	9.99	**17.46**
	114	21.25	0.8	0.83	0.18	3.83	10.24	**17.89**
	125	21.84	0.8	0.83	0.18	3.93	10.52	**18.38**
	136	22.39	0.8	0.83	0.18	4.03	10.79	**18.85**
	153	23.14	0.8	0.83	0.18	4.17	11.15	**19.48**
Leeward	All	23.35	-0.5	0.83	0.18	4.20	**-13.86**	-5.46
Side	All	23.35	-0.7	0.83	0.18	4.20	**-17.72**	-9.32
Roof (0' to 60')	153	23.35	-0.9	0.83	0.18	4.20	**-21.59**	-13.18
Low Parapet WW	140	22.60			1.5	33.90		**33.90**
High Parapet WW	158	23.35			1.5	35.02		**35.02**
High Parapet LW	158	23.35			-1.0	-23.35		**-23.35**

Table B.3: Wind Load Excel Calculations

Level	Floor Ht.	Story Ht. * Net Pressure	
		Perpendicular	Parallel
B2	8	79.6	81.4
B1	11	118.7	121.3
L1	11	134.4	137.4
L2	11	146.9	150.2
L3	10	142.1	145.3
L4	10	149.1	152.4
L5	12	189.0	193.1
L6	10	164.1	167.7
L7	11	186.6	190.8
L8	10	174.6	178.4
L9	10	178.9	182.9
L10	11	202.2	206.7
L11	11	207.4	212.0
L12	17	331.2	338.5
Base Shear (kips)		963.9	226.6

Table B.4: Wind Load Excel Calculations

B.2.2 Seismic Loads



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Seismic Load Calculations

ASCE 7-05, Chapter 12: Seismic Design Requirements for Building Structures

1. Exemptions (11.1.2)

Building not exempt

2. Site Class (11.4.2)

C (From structural documents)

11.4.1 (Fig. 22-1 to 22-6)

$S_s = 0.155\,g$ (from structural documents)
$S_1 = 0.050\,g$

11.4.3 Adjust for site class:

Table 11.4-1, $S_s \le 0.25$, $F_a = 1.2$
Table 11.4-2, $S_1 \le 0.1$, $F_v = 1.7$

Eqn 11.4-1 $S_{MS} = F_a S_s = 1.2(0.155) = 0.186\,g$
Eqn 11.4-2 $S_{M1} = F_v S_1 = 1.7(0.050) = 0.085\,g$

11.4.4 Design Parameters:

Eqn. 11.4-3, $S_{DS} = \frac{2}{3} S_{MS} = (2/3)(0.186) = \underline{0.124\,g}$
Eqn. 11.4-4, $S_{D1} = \frac{2}{3} S_{M1} = (2/3)(0.085) = \underline{0.057\,g}$

3. Seismic Design Category (11.6)

Table 11.6-1 $S_{DS} < 0.167 \rightarrow A$ $\therefore \boxed{\text{SDC A}}$
Table 11.6-2 $S_{D1} < 0.067 \rightarrow A$

4. Select Analysis Procedure (use 11.7)

Eq. 11.7-1 $F_x = 0.01 w_x$

5. Calculate effective total seismic weight (w)

Roof: DL + 20% SL
Floors: DL

$W_{RF} = (\text{penthouse}) = (125')(46')(27 + 0.2(20)) + 2(125 + 46')(38)(5)$
$= 178{,}250 + 49{,}020$
$= \underline{228{,}000 \text{ lbs}}$

Figure B.1: Seismic Load Calculations

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$W_{ST\,FL} = (60')(214')(75\text{ psf}) + 2(60+214)(490\text{ plf})$
$= 963{,}000 + 268{,}520$
$= \underline{1{,}232{,}000\text{ lbs}}$

$W_{CONC\,FL} = (60')(214')(105\text{ psf}) + 2(60+214)(992)$
6'½" typ. $= 1{,}348{,}200 + 543{,}616$
$= \underline{1{,}892{,}000\text{ lbs}}$

Total Load =

$W = W_{RF} + 6(W_{ST\,FL}) + 7(W_{CONC\,FL})$
$= 228\text{ k} + 6(1{,}232\text{ k}) + 7(1{,}892\text{ k})$
$\underline{W = 20{,}864\text{ k}}$

6. Other Factors

Basic Seismic Force-Resisting System: Ordinary Concrete Moment Frames and Steel Moment Frames

Response Modification Factor, R = 3 (Table 12.2-1)

7. Calculate Seismic Base Shear (V)

Eqn. 12.8-1 $V = C_s W$

$C_s = S_{DS} / \left(\frac{R}{I}\right)$ Eq. 12.8-2
$= 0.124 / (3/1) = 0.042$

$V = 0.042(20{,}864) = \boxed{876\text{ k}}$

$T_a = 0.1N = 0.1(14) = 1.4\text{ s}$ (Eqn. 12.8-8)

$T_L = 5.5\text{ s}$ (Fig. 22-2)

$\therefore C_s$ need not exceed $\dfrac{S_{D1} T_L}{T^2 (R/I)} = 0.116 > 0.042$ ✓

8. Vertical Distribution of Seismic Forces (F_x)

$F_x = C_{vx} V = \dfrac{w_x h_x^k}{\sum w_i h_i^k} V$

$k = 1.5$ (using linear interpolation)

Figure B.2: Seismic Load Calculations

Level	h_x (ft)	w_x (k)	$w_x h_x^k$	C_{vx}	F_x (k)	V_x (k)	hx * Fx (ft*k)
Penthouse	153	228	526737	0.010	8.8	8.8	1353
12	136	1232	5881051	0.113	98.7	107.6	13426
11	125	1232	5405378	0.104	90.7	198.3	11342
10	114	1232	4929705	0.094	82.8	281.1	9434
9	104	1232	4497275	0.086	75.5	356.6	7851
8	94	1232	4064844	0.078	68.2	424.8	6414
7	83	1232	3589171	0.069	60.3	485.0	5001
6	73	1892	6007649	0.115	100.8	585.9	7362
5	61	1892	5020090	0.096	84.3	670.2	5141
4	51	1892	4197125	0.080	70.5	740.6	3593
3	41	1892	3374159	0.065	56.6	797.3	2322
2	30	1892	2468897	0.047	41.4	838.7	1243
1	19	1892	1563635	0.030	26.2	864.9	499
B1	8	1892	658373	0.013	11.1	876.0	88
Sum		20864	52184088	1.000	876.0		**75070**
							=OTM

Table B.5: Seismic Load Calculations

B.3 Wood Redesign

Included below are the various hand calculation spot checks of the software and any relevant software output for the redesigned lateral system.

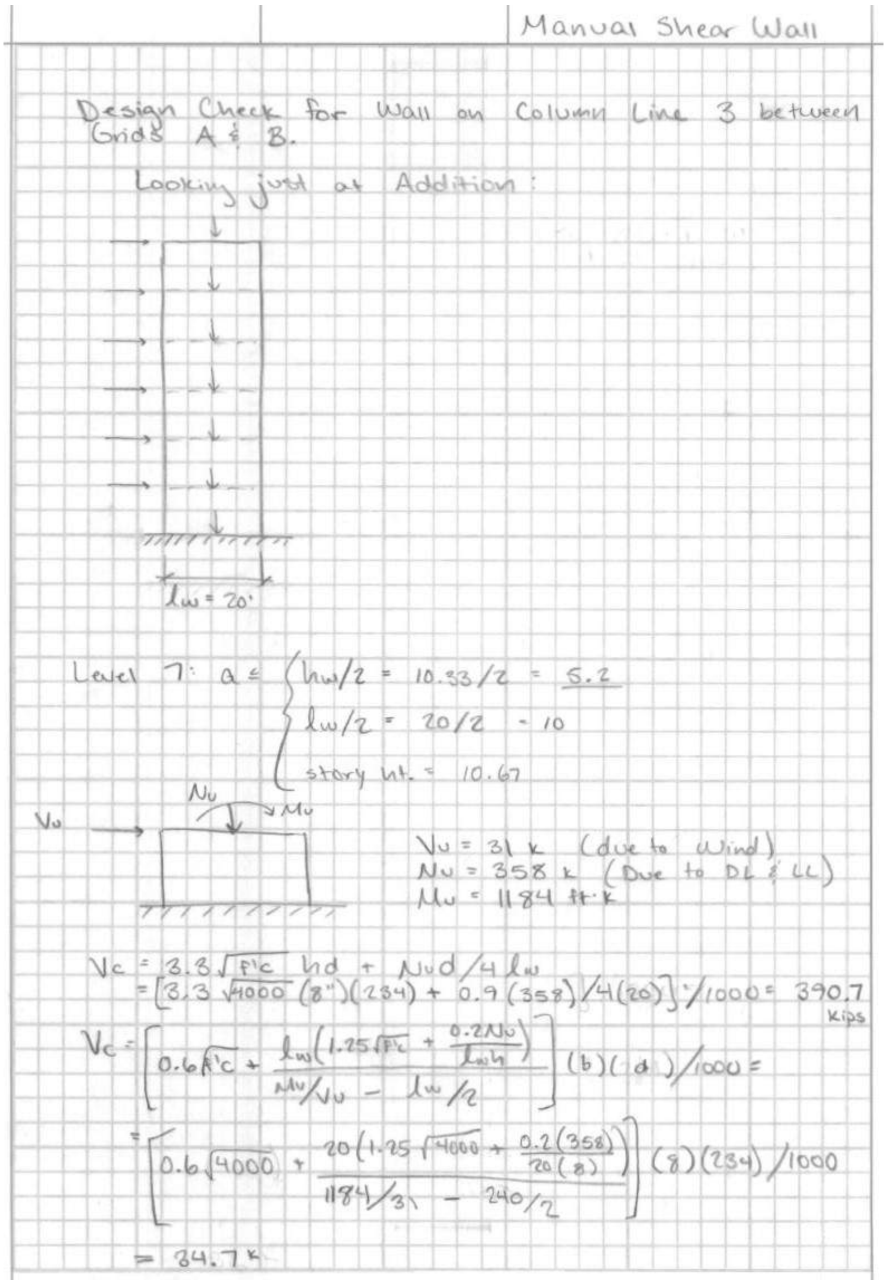


Figure B.3: Shear Wall Spot Check

Manual Shear Wall

or $V_c = 2\sqrt{f'c}\, hd = 2\sqrt{4000}\,(8)(234)/1000 = 237^k$

$V_u > 0.5\phi V_c = 0.5(0.75)(237) = 88.9$

$34.7 < 88.9$, $\therefore$ no shear reinf. req.

Include to meet min reinf. req's

Horizontal : $\rho_t \geq 0.0025 = 8''(240'')(0.0025) = 4.8\,in^2$

$S \leq$ min $\begin{cases} l_w/5 = 20/5 = 4' = 48'' \\ 3h = 3(8) = 24'' \\ 18'' \rightarrow \text{controls} \end{cases}$

#4 bar $\rightarrow$ $A = 0.2\,in^2/bar$ 24 bars

2 curtains, 12 #4's each side @ 10" O.C.

Vertical :

$\rho_l \geq$ min $\begin{cases} 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025) \\ = 0.0025 + 0.5(2.5 - (10.33/20))(0.0025 - 0.0025) \\ = 0.0025 \end{cases}$

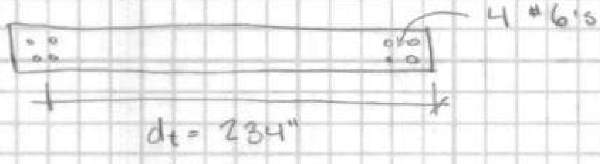
$8''(240'')(0.0025) = 4.8\,in^2 \rightarrow 24$ bars

2 curtains, 12 #4's each side @ 18" O.C.

$S \leq$ min $\begin{cases} l_w/3 = 20'/3 = 6.67' \\ 3h = 24'' \\ 18'' \rightarrow \text{controls} \end{cases}$

Flexural Design (Level 7 : Base Level)

$M_u = 1184$ ft·k check Etabs design



assume case 1:

$M_n = A_s f_y jd = 4(0.44)(60)(0.9)(19.5) = 1853$ ft·k

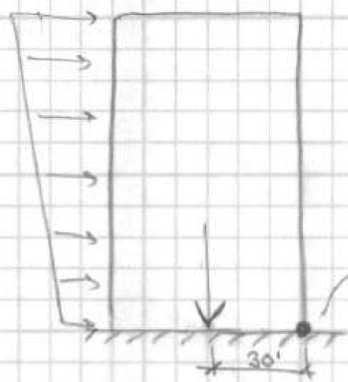
$M_u \geq \phi M_n \rightarrow 1184 \leq 1667$ ft·k ✓ ok

could go to #5's, but should be 2 sizes up from #4's

Figure B.4: Shear Wall Spot Check

Overturning Moment

From Wind Analysis in ETABS, overturning Moment = 34,000 ft·kip



Dead Load of Building =

110 psf (7 floors)(12,800 sf)
+ 36 psf (7 floors)(12,800 sf) = 13,080 Kips

Resisting w/ moment arm of half building width = 30'

13,080 kips (30') = 392,000 ft·k

$M_R > M_{ov}$

392,000 >> 34,000

∴ building can resist overturning moment due to wind

Other direction has much lower overturning moment due to significantly less surface area ∴ will not control.

Figure B.5: Overturning Moment Check

In-Plane Diaphragm Deflections

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \underbrace{\frac{0.25vL}{1000G_a}}_{0} + \underbrace{\frac{\Sigma(x\Delta_c)}{2W}}_{0}$$

G_a = apparent stiffness from nail slip
glued product, ∴ does not apply

Δ_c = diaphragm chord splice slip, DNA

$$\therefore \quad \delta_{dia} = \frac{5vL^3}{8EAW}$$

$v = 4.8$ klf $= 4800$ plf

$L = 26'$

$E = 1.5 \times 10^6$ psi

$A = 3000$ in^2

$W = 30'$ (Bay act individually)

$$\delta_{dia} = \frac{5(4800)(26)^3(1728)}{8(1.5E6)(3000)(30)} = \underline{\underline{0.675 \text{ in}}}$$

From ETABS: $\delta_{dia} = 0.823$

calculated $l/360$ limit $= 0.877$, ∴ acceptable

The hand calc is just a bit lower, most likely b/c the equation couldn't account for shear controlled deflection. ∴ hand calc may be low, ETABS may be conservative.

Figure B.6: In-plane deflection spot check

C Design Tables

C.1 Introduction

Included in this Appendix are the design value tables used in the gravity system redesign. The purpose of this appendix is to provide the specifically referenced tables used in this thesis.

CLT Grade	Laminations in the Major Strength Direction of the CLT						Laminations in the Minor Strength Direction of the CLT					
	$f_{b,0}$ (psi)	E_0 (10^6psi)	$f_{t,0}$ (psi)	$f_{c,0}$ (psi)	$f_{v,0}$ (psi)	$f_{s,0}$ (psi)	$f_{b,90}$ (psi)	E_{90} (10^6psi)	$f_{t,90}$ (psi)	$f_{c,90}$ (psi)	$f_{v,90}$ (psi)	$f_{s,90}$ (psi)
E1	4,095	1.7	2,885	3,420	425	140	1,050	1.2	525	1,235	425	140
E2	3,465	1.5	2,140	3,230	565	190	1,100	1.4	680	1,470	565	190
E3	2,520	1.2	1,260	2,660	345	115	735	0.9	315	900	345	115
E4	4,095	1.7	2,885	3,420	550	180	1,205	1.4	680	1,565	550	180
V1	1,890	1.6	1,205	2,565	565	190	1,100	1.4	680	1,470	565	190
V2	1,835	1.4	945	2,185	425	140	1,050	1.2	525	1,235	425	140
V3	2,045	1.6	1,155	2,755	550	180	1,205	1.4	680	1,565	550	180

For SI: 1 psi = 6.895 kPa

[a] The characteristic values may be obtained from the published allowable design values for lumber in the United States as follows:

$f_{b,0}$ = 2.1 x published allowable bending stress (F_b), $f_{t,0}$ = 2.1 x published allowable tensile stress (F_t),
$f_{c,0}$ = 1.9 x published allowable compressive stress parallel to grain (F_c), $f_{v,0}$ = 3.15 x published allowable shear stress (F_v), and $f_{s,0}$ = 1/3 x calculated $f_{v,0}$.

Table C.1: CLT Material Design Values Table. Source: CLT Handbook.

CLT Grade	CLT Thickness (in.)	Lamination Thickness in CLT Lay-up (in.)							Major Strength Direction			Minor Strength Direction		
		=	⊥	=	⊥	=	⊥	=	$F_bS_{eff,0}$ (lb.-ft./ft.)	$EI_{eff,0}$ (10^6 lb.-in.^{2}/ft.)	$GA_{eff,0}$ (10^6 lb./ft.)	$F_bS_{eff,90}$ (lb.-ft./ft.)	$EI_{eff,90}$ 10^6 lb.-in.^{2}/ft.)	$GA_{eff,90}$ (10^6 lb./ft.)
E1	4 1/8	1 3/8	1 3/8	1 3/8					4,525	115	0.46	160	3.1	0.61
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			10,400	440	0.92	1,370	81	1.2
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	18,375	1,089	1.4	3,125	309	1.8
E2	4 1/8	1 3/8	1 3/8	1 3/8					3,825	102	0.53	165	3.6	0.56
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			8,825	389	1.1	1,430	95	1.1
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	15,600	963	1.6	3,275	360	1.7
E3	4 1/8	1 3/8	1 3/8	1 3/8					2,800	81	0.35	110	2.3	0.44
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			6,400	311	0.69	955	61	0.87
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	11,325	769	1.0	2,180	232	1.3
E4	4 1/8	1 3/8	1 3/8	1 3/8					4,525	115	0.53	180	3.6	0.63
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			10,425	441	1.1	1,570	95	1.3
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	18,400	1,090	1.6	3,575	360	1.9
V1	4 1/8	1 3/8	1 3/8	1 3/8					2,090	108	0.53	165	3.6	0.59
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			4,800	415	1.1	1,430	95	1.2
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	8,500	1,027	1.6	3,275	360	1.8
V2	4 1/8	1 3/8	1 3/8	1 3/8					2,030	95	0.46	160	3.1	0.52
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			4,675	363	0.91	1,370	81	1.0
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	8,275	898	1.4	3,125	309	1.6
V3	4 1/8	1 3/8	1 3/8	1 3/8					2,270	108	0.53	180	3.6	0.59
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			5,200	415	1.1	1,570	95	1.2
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	9,200	1,027	1.6	3,575	360	1.8

For SI: 1 in. = 25.4 mm; 1 ft. = 304.8 mm; 1 lb. = 4.448 N

(a) This table represents one of many possibilities that CLT could be manufactured by varying lamination grades, thicknesses, orientations, and layer arrangements in the lay-up.
(b) Custom CLT grades that are not listed in this table are permitted in accordance with ANSI/APA PRG 320.
(c) The allowable properties can be converted to the characteristic properties by multiplying the tabulated F_bS by 2.1, and EI and GA by 1.0.

Table C.2: CLT Panel Design Table. Source: CLT Handbook.

***DOUGLAS-FIR* GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES**

F_b = 2,400 psi, E = 1.80 x 10^6 psi, F_v = 265 psi

3-1/8-INCH WIDTH

Depth (in.)	**6**	**7-1/2**	**9**	**10-1/2**	**12**	**13-1/2**	**15**	**16-1/2**	**18**	**19-1/2**	**21**	**22-1/2**	**24**	**25-1/2**	**27**
Beam Weight (lb/ft)	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.75	23.44	28.13	32.81	37.50	42.19	46.88	51.56	56.25	60.94	65.63	70.31	75.00	79.69	84.38
S (in.^{3})	18.75	29.30	42.19	57.42	75.00	94.92	117.2	141.8	168.8	198.0	229.7	263.7	300.0	338.7	379.7
I (in.^{4})	56.25	109.9	189.8	301.5	450.0	640.7	878.9	1170	1519	1931	2412	2966	3600	4318	5126
EI (10^6 lb-in.^{2})	101.3	197.8	341.7	542.6	810.0	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)	3750	5859	8438	11480	15000	18980	23440	28360	33750	39610	45940	52730	60000	67730	75940
Shear Capacity (lb)	3313	4141	4969	5797	6625	7453	8281	9109	9938	10770	11590	12420	13250	14080	14910

3-1/2-INCH WIDTH

Depth (in.)	**6**	**7-1/2**	**9**	**10-1/2**	**12**	**13-1/2**	**15**	**16-1/2**	**18**	**19-1/2**	**21**	**22-1/2**	**24**	**25-1/2**	**27**
Beam Weight (lb/ft)	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.00	26.25	31.50	36.75	42.00	47.25	52.50	57.75	63.00	68.25	73.50	78.75	84.00	89.25	94.50
S (in.^{3})	21.00	32.81	47.25	64.31	84.00	106.3	131.3	158.8	189.0	221.8	257.3	295.3	336.0	379.3	425.3
I (in.^{4})	63.00	123.0	212.6	337.6	504.0	717.6	984.4	1310	1701	2163	2701	3322	4032	4836	5741
EI (10^6 lb-in.^{2})	113.4	221.5	382.7	607.8	907.2	1292	1772	2358	3062	3893	4862	5980	7258	8705	10330
Moment Capacity (lb-ft)	4200	6563	9450	12860	16800	21260	26250	31760	37800	44360	51450	59060	67200	75860	85050
Shear Capacity (lb)	3710	4638	5565	6493	7420	8348	9275	10200	11130	12060	12990	13910	14840	15770	16700

5-1/8-INCH WIDTH

Depth (in.)	**12**	**13-1/2**	**15**	**16-1/2**	**18**	**19-1/2**	**21**	**22-1/2**	**24**	**25-1/2**	**27**	**28-1/2**	**30**	**31-1/2**	**33**
Beam Weight (lb/ft)	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.50	69.19	76.88	84.56	92.25	99.94	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123.0	155.7	192.2	232.5	276.8	324.8	376.7	432.4	492.0	555.4	622.7	693.8	768.8	847.5	930.2
I (in.^{4})	738.0	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11530	13350	15350
EI (10^6 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10630	12750	15130	17800	20760	24030	27630
Moment Capacity (lb-ft)	24600	31130	38440	46510	55350	64960	75340	86480	98400	111100	124500	138800	153800	169500	186000
Shear Capacity (lb)	10870	12220	13580	14940	16300	17660	19010	20370	21730	23090	24450	25800	27160	28520	29880

5-1/2-INCH WIDTH

Depth (in.)	**12**	**13-1/2**	**15**	**16-1/2**	**18**	**19-1/2**	**21**	**22-1/2**	**24**	**25-1/2**	**27**	**28-1/2**	**30**	**31-1/2**	**33**
Beam Weight (lb/ft)	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.00	74.25	82.50	90.75	99.00	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132.0	167.1	206.3	249.6	297.0	348.6	404.3	464.1	528.0	596.1	668.3	744.6	825.0	909.6	998.3
I (in.^{4})	792.0	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12380	14330	16470
EI (10^6 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11400	13680	16240	19100	22280	25790	29650
Moment Capacity (lb-ft)	26400	33410	41250	49910	59400	69710	80850	92810	105600	119200	133700	148900	165000	181900	199700
Shear Capacity (lb)	11660	13120	14580	16030	17490	18950	20410	21860	23320	24780	26240	27690	29150	30610	32070

6-3/4-INCH WIDTH

Depth (in.)	**18**	**19-1/2**	**21**	**22-1/2**	**24**	**25-1/2**	**27**	**28-1/2**	**30**	**31-1/2**	**33**	**34-1/2**	**36**	**37-1/2**	**39**
Beam Weight (lb/ft)	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	364.5	427.8	496.1	569.5	648.0	731.5	820.1	913.8	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11070	13020	15190	17580	20210	23100	26240	29660	33370
EI (10^6 lb-in.^{2})	5905	7508	9377	11530	14000	16790	19930	23440	27340	31650	36390	41580	47240	53390	60060
Moment Capacity (lb-ft)	72900	85560	99230	113900	129600	146300	164000	182800	202500	223300	245000	267800	291600	316400	342200
Shear Capacity (lb)	21470	23250	25040	26830	28620	30410	32200	33990	35780	37560	39350	41140	42930	44720	46510

8-3/4-INCH WIDTH

Depth (in.)	**24**	**25-1/2**	**27**	**28-1/2**	**30**	**31-1/2**	**33**	**34-1/2**	**36**	**37-1/2**	**39**	**40-1/2**	**42**	**43-1/2**	**45**
Beam Weight (lb/ft)	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840.0	948.3	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12090	14350	16880	19690	22790	26200	29940	34020	38450	43250	48440	54020	60020	66450
EI (10^6 lb-in.^{2})	18140	21760	25830	30380	35440	41020	47170	53900	61240	69210	77860	87190	97240	108000	119600
Moment Capacity (lb-ft)	168000	189700	212600	236900	262500	289400	317600	347200	378000	410200	443600	478400	514500	551900	590600
Shear Capacity (lb)	37100	39420	41740	44060	46380	48690	51010	53330	55650	57970	60290	62610	64930	67240	69560

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10-year) duration of load and should be adjusted for the design duration of load per the applicable building code.

Table C.3: Glulam Beam Design Table. Source: APA - The Engineered Wood Association

D Breadth Calculations

D.1 Introduction

Included in this appendix are additional breadth tables and calculations.

D.1.1 Construction Management Breadth

Structurlam Products Ltd Budget Pricing for CrossLam (Cross Laminated Timber Panels) CDN$

			1	2	3			
Panel Type	# of Laminations	Panel Thickness	Blank Panel $/Sq. Ft	Hand Framing (Floor/Roof) $/Sq. Ft	5 Axis Robotic Framing (Walls) $/Sq. Ft	Fasterner, Hardware, Shop Drawings Floor/Roof $/Sq. Ft.	Walls $/Sq. Ft.	Visual Grade $/Sq. Ft
SLT3	3	99mm	5.80	6.05	7.02	2.50	3.00	1.00
SLT5	5	169mm	9.68	9.93	11.21	2.50	3.00	1.00
SLT7	7	239mm	13.77	14.02	15.93	3.00	3.50	1.00
SLT9	9	309mm	17.53	17.97	19.90	3.00	3.50	1.00

Note: it's columns 1 or 2 or 3... not 1 + 2 or 1 +3 or 1 + 2 + 3

Table D.1: Structurlam CLT costs given in Canadian dollars. Source: Michael Green's Presentation on How to Develop a CLT Project

Item Quantities					
Gravity System Items	**Unit**	**Quantity Per Level**			**Total**
		Typ. Level	**12th Level**	**Penthouse**	
Steel Columns	L.F.	455	513	270	3058
Steel Columns	Ton	10.9	12.34	11.43	78.27
W 12x22	L.F.	336	336	0	2016
W10x33	L.F.	798	798	0	4788
W16x26	L.F.	0	0	625	625
W14x22	L.F.	0	0	310	310
W12x30	L.F.	105	105	0	630
Open Web Joist 12K3	L.F.	2700	2700	0	16200
Open Web Joist 16K3	L.F.	0	0	1100	1100
Floor Deck	S.F.	12840	12840	0	77040
Roof Deck	S.F.	0	0	4300	4300
Moment Connection Weld	L.F.	82	82	30	522
Shear Connection Weld	L.F.	207	207	138	1380
Bolts	Ea	1250	1250	830	8330
Connection Angle	L.F.	294	294	196	1960
Welded Wire Fabric	C.S.F.	12840	12840	4300	813.4
Concrete deck topping	CY	12840	12840	4300	81340
Partitions	L.F.	750	750	130	4630
Shear Wall System Items	**Unit**	**Quantity Per Level**			**Total**
		B2	**B1**	**Typ**	
CMU	S.F.	1650	1510	1140	8860
Rebar (#5's @ 24" O.C.)	Ton	0.51	0.47	0.36	2.78

Table D.2: Quantities found for Steel Addition

Item Quantities						
Gravity System Items	Unit	Quantity Per Level			Total
		Typ. Level	12th Level	Penthouse	
5-ply CLT Panels (including visual grading)	S.F.	10780	0	5500	59400
7-ply CLT Panels (including visual grading)	S.F.	1560	10780	0	18580
9-ply CLT Panels (including visual grading)	S.F.	0	1560	0	1560
Double 3-ply Partitions	S.F.	6600	7400	1990	42390
Wall Insulation	S.F.	5980	6704	1803	38405
Studs 2" x 3", pneumatic nailed	MBF	9	10	3	56
Sound Attenuation for Floor	S.F.	12340	12340	5500	79540
Glulam Typ Beams	Ea	27	27	18	180
Glulam Perimeter Beams	Ea	20	20	12	132
Glulam Columns	MBF	3640	4110	3760	26070
Shear Wall System Items	Unit	Quantity Per Level			Total
		Existing Typ.	Addition Typ.	Penthouse	
Cast in Place Concrete	C.Y.	50	50	64	714
Rebar (#4's @ 18" O.C.)	Ton	0.51	0.51	0.705	7.335

Table D.3: Quantities found for Wood Addition

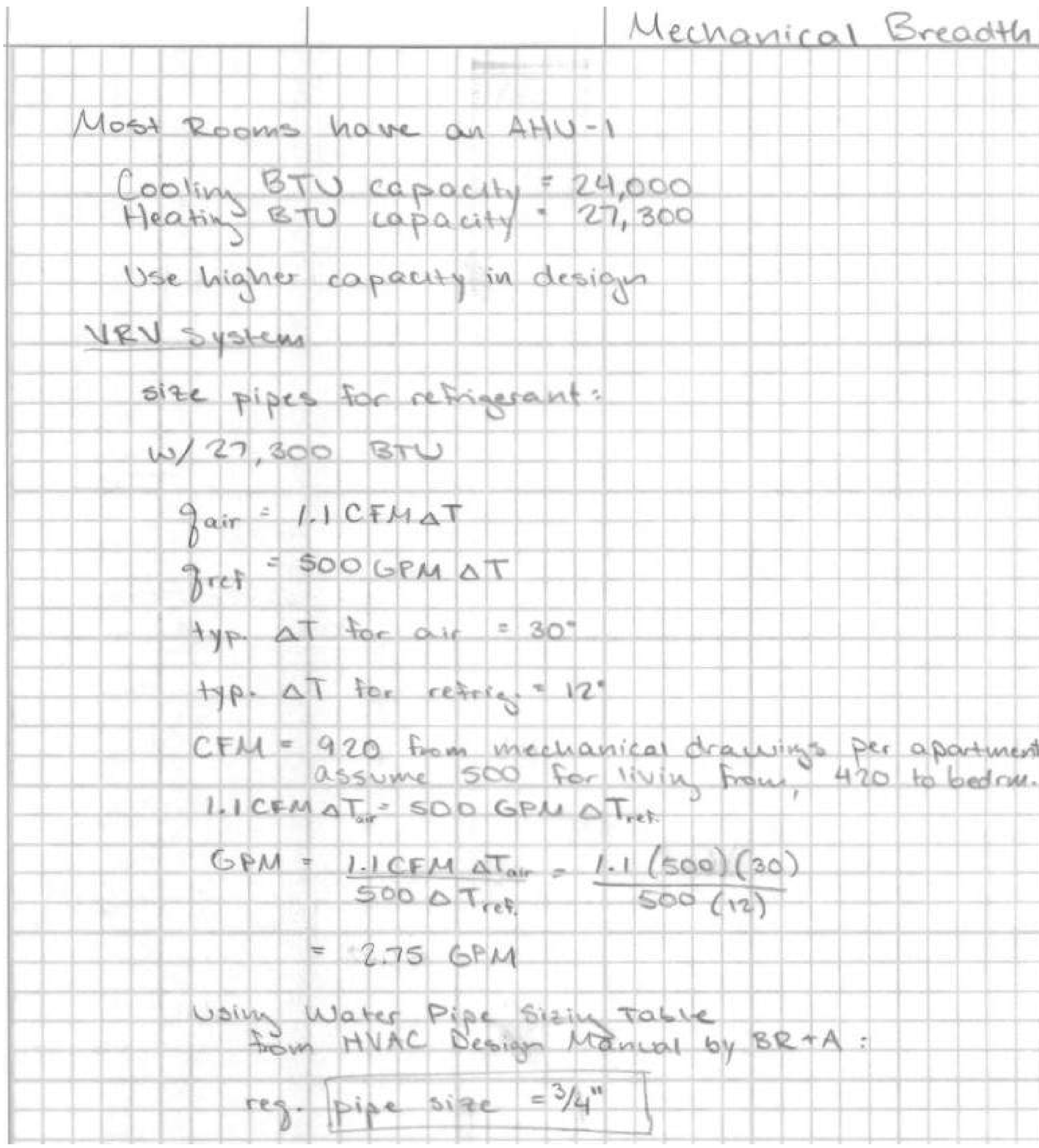
Schedule Analysis: 11141 Georgia Ave Existing Addition						
Item	Qty	Crew Type	# on Crew	Daily Output	Labor Hours	Hrs per item
W10x49	3058	E-2	8	550	0.102	39.0
W12x22	2016	E-2	8	880	0.064	16.1
W10x33	4788	E-2	8	550	0.102	61.0
W12x35	625	E-2	8	810	0.069	5.4
W16x26	310	E-2	8	1000	0.056	2.2
W14x22	630	E-2	8	990	0.057	4.5
Open Web Joist 12K3	16200	E-7	13	1500	0.053	66.0
Open Web Joist 16K3	1100	E-7	13	1800	0.044	3.7
Floor Decking, Composite decking, 1.5" deep, 20 ga.	77040	E-4	8	3800	0.008	77.0
Roof Decking, under 50 squares, 1.5" deep, 22 ga.	4300	E-4	8	4500	0.007	3.8
Weld, 4 passes, 1/2" thick plus avg 150% for half overhead	522	E-14	2	22	0.364	95.0
Weld, 4 passes, 1/2" thick + 20% for vertical	1380	E-14	2	22	0.364	251.2
3/4" diameter bolts 2" long	8330	1 Sswk	1	120	0.067	558.1
Angles, 3"x3"	1960	2 Sswk	2	500	0.032	31.4
Welded Wire Fabric 6x6 W2.1xW2.1	813.4	2 Rodm	2	31	0.516	209.9
Elevated Slab, regular 4000 psi conc., 2-1/2" thick floor fill	81340	C-8	8	2685	0.022	223.7
Framing, stud walls, 10' high, 6" wide, studs 12" O.C.	4630	2 Carp	2	51	0.314	726.9
8" CMU solid grouted reinforced altenate cources	8860	D-8	5	355	0.113	200.2
Reinforcing in place, walls, #3 to #7	2.78	4 Rodm	4	3	10.667	7.4
					Total (days)	322.8
					Weeks (5 d/wk)	64.6
					Months (4 wk/m)	16.1

Table D.4: Scheduling time found for Steel Addition

Schedule Analysis: 11141 Georgia Ave Wood Addition Redesign						
Item	Qty	Crew Type	# on Crew	Daily Output	Labor Hours	Hrs per item
03 41 13.50 Precaset Slab Planks (5-ply CLT)	59400	C-11	10	2400	0.03	178.2
03 41 13.50 Precaset Slab Planks (7-ply CLT)	18580	C-11	10	2800	0.026	48.3
03 41 13.50 Precaset Slab Planks (9-ply CLT)	1560	C-11	10	3200	0.023	3.6
03 47 13.40 Tilt-up walls (Double 3-ply Partitions)	42390	C-14	19	1600	0.09	200.8
Mineral Wool Wall Insulation	38405	1 Carp	1	1600	0.005	192.0
2x3 Studs in wall	56	2 Carp	2	22.222	0.72	20.3
Sound Attenuation for Floor	79540	1 Caro	2	1600	0.0005	19.9
Straight Glulam Beam, 20' span, 6.75" x 15" (Typ Beams)	180	F-3	6	29	1.379	41.4
Straight Glulam Beam, 20' span, 6.75" x 18" (Perim. Beams)	132	F-3	6	28	1.429	31.4
Alternate Pricing, columns including hardware	26.07	F-3	6	2	20	86.9
Wall, free-standing, 8" thick	714	C-14D	27	45.83	4.364	115.4
Reinforcing in place, walls, #3 to #7	7.335	4 Rodm	4	3	10.667	19.6
					Total (days)	119.7
					Weeks (5 d/wk)	23.9
					Months (4 wk/m)	6.0

Table D.5: Scheduling time found for Wood Addition

D.1.2 Mechanical Breadth



Mechanical Breadth

Most Rooms have an AHU-1

Cooling BTU capacity = 24,000
Heating BTU capacity = 27,300

Use higher capacity in design

<u>VRV System</u>

size pipes for refrigerant:

w/ 27,300 BTU

$q_{air} = 1.1\,CFM\,\Delta T$

$q_{ref} = 500\,GPM\,\Delta T$

typ. ΔT for air $= 30°$

typ. ΔT for refrig. $= 12°$

CFM = 920 from mechanical drawings per apartment
assume 500 for living room, 420 to bedrm.

$1.1\,CFM\,\Delta T_{air} = 500\,GPM\,\Delta T_{ref.}$

$$GPM = \frac{1.1\,CFM\,\Delta T_{air}}{500\,\Delta T_{ref.}} = \frac{1.1\,(500)(30)}{500\,(12)}$$

$$= 2.75\ GPM$$

Using Water Pipe Sizing Table
from HVAC Design Manual by BR+A:

req. $\boxed{\text{pipe size} = 3/4''}$

Figure D.1: Mechanical Equipment Sizing Calculations

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Academic Vita

Samantha deVries

Education

Bachelor and Master of Architectural Engineering, Interdisciplinary Science
Pennsylvania State University, State College, PA, expected December 2015
Option: Structural Engineering

Software Skills

Software: ETABS, SAP 2000, Revit, STAAD, RISA 2D, RAM Concept and Structural System, Microsoft Excel

Experience

Rathgeber Goss Associates — Summer 2014
Structural Engineering Intern in Rockville, MD

- Assisted in design of new structures and renovations to existing structures
- Prepared structural documents for submission and recorded the results at a concrete garage survey

Appalachia Service Project — Summers 2011-2013
Center Director in Washington County, VA (Summer 2013)

- Managed the project schedule and $40,000 budget for 28 homes repaired in an ASP county
- Responsible for ensuring the quality work and continual improvement of 3 center staff

Center Staff in Letcher County, KY (Summer 2012) and Mingo County, WV (Summer 2011)

- Supervised 60-80 volunteers and managed building supplies for three projects weekly involving emergency home repair and renovation
- Responsibilities include interviewing families for project selection, managing tool and supply inventory, training volunteers in safety and quality building practices, and promoting positive community relationships

Community Service

Volunteer Climbing Instructor at the State College, PA YMCA
Foster community involvement in the sport of rock climbing by working with other volunteers as a belayer and instructor at the local YMCA

Extra-Curricular Activities

Cellist, Various Penn State Music Ensembles, fall 2010-present
Secretary, Penn State Student Chapter of the Structural Engineering Association, fall 2014-spring 2015
Member, Penn State Outing Club, spring 2011-present
President, Co-Founder, Global Architecture Brigades, fall 2011-spring 2013