Use of Mass Timber for Multi-Story Laboratory and Office Building

A Major Qualifying Project

Submitted on:

March 16, 2023

Submitted to:

Project Advisor Professor Leonard D. Albano, Civil and Environmental Engineering, WPI

Submitted by: Jane T. Richardson

Paul E. Williamson

Desmond C. Woodson

Sponsored by: Michael J. Richard, Ph.D. P.E., Consulting Engineer with Simpson Gumpertz & Heger

Thomas Neal, AIA, Consulting Architect with OTJ

Abstract

The goal of this project was to design a five-story mixed laboratory and Class A office building using mass timber, to accommodate for the constraints of fire resistance and to investigate the vibrational performance of CLT. A cost analysis was provided as a comparative study between mass timber and typical construction materials.

This report provides the design process for a mass timber building incorporated with fire resistance design from the NDS. Following the initial design process and consultation with the project sponsors, a vibrational analysis was performed to ensure the design met the required human acceptance criteria which also reflects the serviceability of the space for laboratory instruments and activities. Finally, two estimates for the final building design were procured from leading mass timber fabricators and analyzed to establish a baseline cost for construction in the Boston, Massachusetts area.

Acknowledgments

We would like to acknowledge and thank Dr. Michael Richard of Simpson Gumpertz & Heger for sponsoring this project, providing guidance on the design of mass timber structures, and participating in weekly meetings between our team and Professor Albano. His support throughout this process was shown while helping the team interpret the language of building codes and technical guides and when providing his expertise on mass timber design.

We would also like to acknowledge and thank Thomas Neal of OTJ for sponsoring the project. providing floor plans for the design of the building, and participating in our design update meetings. He gave valuable insight from an architect's point of view and helped facilitate the real-world interactions between an engineer and architect on a project.

We would also like to acknowledge and thank Stephanie Bishop for allowing us to use her vibrational analysis spreadsheet tool and for providing further insight on how to address issues of increased vibration in our design.

We would also like to acknowledge and thank Professor Leonard Albano for his support and guidance throughout this project. His understanding of structural design, the behavior of wood, and construction practices have been beneficial to the team and an invaluable resource.

We would also like to acknowledge and thank our contacts at two different mass timber fabrication companies. These two contacts provided us with valuable cost data for our project to help provide an understanding of the overall cost of our design. These individuals and their companies will remain anonymous to protect their company's cost data.

Finally, the team would like to acknowledge and thank Ronald Mandella for boosting our morale throughout the project.

Authorship

This is the final project report to complete the Major Qualifying Project (MQP) and was authored by Jane Richardson, Paul Williamson, and Desmond Woodson. The project involved performing structural design calculations to develop a mass timber building design and spreadsheets were utilized to accelerate these calculations in the design process. This project was collaborative and below is a breakdown of the detailed areas the team members were responsible for.

Jane Richardson: Introduction, Background, Methodology, Cost Analysis, Conclusion, and Appendices

Paul Willamson: Background and Design for a Mass Timber Laboratory and Office Building

Desmond Woodson: Background, Design Layouts, and Vibrational Analysis

Capstone Design Statement

WPI's Civil, Environmental, and Architectural Engineering Department requires the ABET (Accreditation Board for Engineering and Technology) standards to be met in all capstone design projects. The Major Qualifying Project (MQP) is a professionally disciplined project which involves significant levels of independent research and design to address a problem found in industry. The main objective of this MQP was to design a five-story mixed-use laboratory and office building using cross-laminated timber (CLT) and glue laminated timber (glulam or GLT) as the primary structural members. The design addresses vibrational and fire analysis of mass timber members and the associated decisions made to accommodate each constraint. A cost-benefit analysis for the final building design was performed with the assistance of two mass timber fabricators. To complete this work, the following realistic constraints were taken into consideration: economic, social, health and safety, sustainability, environment, and ethics.

Economic

To address economic constraints in our design, the team compared cost estimates from two mass timber fabricators. The design used in the estimate met all current and local building codes and standards. These estimates included unit pricing on mass timber members, costs of manufacturing and transportation, and associated taxes and fees based on the project location. The variances in market pricing for materials and transportation were taken into consideration.

Social

To address social constraints in our design, perceptions about the use of timber in fire resistance and vibrational design were addressed in comparison with other common building materials.

Health and Safety

To address health and safety constraints in our design, current building codes and standards were utilized on the local, state, and international level for fire resistance and vibrational design. Building materials and dimensions were designed in accordance with Type IV-HT building classification as defined by the 2021 International Building Code (IBC) and Massachusetts State Building Code. The use of the IBC along with the *2019 Canadian CLT Handbook* and *2018*

National Design Specification (NDS) for Wood Construction were used to define the constraints for fire resistance and vibrational design.

Sustainability

To address sustainability constraints in our design, mass timber was utilized in the design of the building. Compared to alternative building materials such as structural steel and reinforced concrete, mass timber construction results in fewer greenhouse emissions and waste as well as faster construction schedules in many cases.

Environment

To address environmental constraints in our design, trucking has been employed to deliver the prefabricated mass timber panels and members. Combined with an accelerated construction schedule, this process for erection will limit traffic and noise disturbances in the immediate and surrounding areas around the project site.

Ethics

To maintain ethical practices in our design, the team conducted themselves with professionalism, integrity, and an advanced interest in the health, safety, and welfare of the public. Issues arising during the design process were communicated with the project sponsors and collaborating professionals to find effective solutions and provide a professionally finished end product.

Professional Licensure Statement

Professional licensure is required for those aspiring to become professional engineers in order to better protect the public. To receive a professional license, an engineer must meet the education, examination, and professional work experience qualifications set by governing state boards. The National Council of Examiners for Engineering and Surveying (NCEES) is an organization dedicated to providing a pathway for engineers and surveyors to obtain professional licensure. Once professional licensure is obtained, an engineer or surveyor can sign and stamp project drawings to signify the set meets all required and applicable safety and design standards.

The process of becoming a licensed professional engineer is as follows:

1. Acquire a bachelor's degree from an ABET-accredited university
2. Take and pass the fundamentals of engineering (FE) exam to become an engineer in training (EIT)
3. Complete a minimum of four years of work under the direction of another professional engineer in desired discipline
4. Take and pass the principles and practice of engineering (PE) exam

In this MQP, the role of a professional engineer would include designing, reviewing, and stamping structural plans for the five-story building. Coordination with other technical consultants would be necessary as well for developing consistent plans for the integration of mechanical, electrical, plumbing, and fire protection systems.

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1. Introduction

Mass timber, otherwise known as engineered wood, is a building material consisting of either wooden lumber, panels, veneers, or fibers bound together in layers. Besides reliability, timber has proven itself to be a valuable material within construction due to its high-strength-to-weight ratio as well as design flexibility and renewability. The building designed in this project was classified as Type IV-HT for its use of mass timber and Class A for occupancy. Cross-laminated timber (CLT) and glue-laminated timber (glulam) were used in this project for the main structural framing, flooring, roofing, and lateral bracing.

CLT has continually risen in popularity since the early 20th century. The low weight-to-strength ratio of CLT has attracted the attention of engineers and contractors around the world, with CLT being recognized as a code-compliant construction material in the 2015 International Building Code. Cross-laminated timber is a semi-rigid composite engineered timber that is constructed with three, five, or seven layers of timber boards glued together, with each layer oriented perpendicular to the adjacent. Due to the diagonally crossing configuration of the timber members, CLT performs well against shear in-plane and tension perpendicular to the plane, making CLT a popular choice for structural floors and walls.

The goal of this project was to design a five-story mixed laboratory and Class A office building using mass timber, to accommodate for the constraints of fire resistance and to investigate the vibrational effects on CLT. A cost analysis was provided as a comparative study between mass timber and typical construction materials. Based on the goals, the following objectives were defined for the project:

Objective 1: Design a Mixed Laboratory and Class A Office Mass Timber Building

Objective 2: Evaluate the Design for Vibration Performance

Objective 3: Perform a Cost Analysis for the Design Options

2. Background

2.1 Mass Timber

Mass timber, otherwise known as engineered wood, is a building material consisting of either wooden lumber, panels, veneers, or fibers bound together in layers (Arquivo.pt, n.d.). Alongside reliability, timber has proven itself to be a valuable material within construction due to its high-strength-to-weight ratio as well as design flexibility and renewability. Mass timber provides increased structural stability and uniformity when compared to traditional wood products (Udele et al., 2022). The off-site fabrication of mass timber members provides additional advantages including the precision of members and increased speed of construction (Smith et al., 2017). For example, the Ascent, a 25-story tall mass timber hybrid building in Milwaukee, “required 90% less construction traffic, 75% fewer workers on-site, and is 25% faster than traditional construction” says the developers (Mass Timber, n.d.). The quicker construction phase is more cost-effective for the owner and contractor, allowing the building to be occupied and produce revenue sooner. (Castle, 2021)

For any project to be categorized as a mass timber project, the primary load-bearing structure must be constructed of either solid or engineered wood; this does not include non-structural timber accents (Mass Timber 101: Understanding the Emerging Building Type, 2017). Optimizing the use of mass timber in construction creates an end product that will be more homogeneous from a structural perspective. These types of projects present both a cost and schedule benefit when compared to those of traditional site-built construction due to the off-site fabrication process previously mentioned. Mass timber construction (MTC) has not had a dominant presence in North America as a result of the lack of both qualitative and quantitative research on its performance. Concerns with this type of construction include the issues of acoustics and vibration, wind, and component flexibility. Research efforts from Smith et al. demonstrate that the benefits of MTC and engineered wood products such as glue-laminated timber and cross-laminated timber can provide alternative construction resources and applications that reduce environmental impacts and construction costs if accepted (Smith et al., 2017).

2.2 Glue-Laminated Timber (Glulam)

Glue-laminated timber otherwise known as glulam is another type of structural engineered wood that consists of wood laminations joined together in series by weather-resistant adhesive. Glulam members can be used in various applications of a load-bearing structure such as a beam, girder, column, and more. This type of wood is readily produced in various sizes, curved shapes, and species. The high strength and lightweight attributes allow for distances of up to 100 feet to be spanned without the assistance of intermediate supports, in turn requiring fewer joints (Migliani, 2019). Another beneficial characteristic of glulam is its high degree of fire endurance due to the charring effect that takes place. Based on findings from a study conducted in 1961 by the Southwest Research Institute, glulam was found to perform better than steel in the face of a fire (Douglas, 2000). In addition to this, a 2002 environmental impact case study found that it takes about two to three times more energy and about six to twelve times more fossil fuel for the manufacturing of steel beams in comparison to glulam beams (Petersen & Solberg, 2005). There are four different appearance grades for glulam as defined by the American National Standards Institute (ANSI); framing, industrial, architectural, and premium. Each of these four options is dependent upon where the glulam is being used within the structure, as well as the aesthetic appeal associated with it. Altogether glue-laminated timber can be seen as a cost-effective and resource-efficient material providing a number of advantages over other construction materials like concrete and steel.

2.3 Cross-Laminated Timber (CLT)

Cross-laminated timber (CLT) has continually risen in popularity since the early 20th century. The low weight-to-strength ratio of CLT has attracted the attention of engineers and contractors around the world, with CLT being recognized as a code-compliant construction material in the 2015 International Building Code. Cross-laminated timber is a semi-rigid composite engineered timber that is constructed with three, five, or seven layers of timber boards glued together, with each layer oriented perpendicular to the adjacent. The board thickness averages between 5/8 to 2 inches and between 2.4 and 9.5 inches in width; panels are typically 2 to 10 feet wide and span 60 or more feet in length (Think Wood, 2022 & Cross-Laminated Timber (CLT) - APA – the Engineered Wood Association, n.d.). Due to the diagonally crossing configuration of the timber members, CLT performs well against shear in-plane and tension perpendicular to the plane, making CLT a popular choice for structural floors and walls (Brandner et al., 2016).

Unlike steel and carbon, timber construction materials take more carbon dioxide out of the environment than what is produced during the manufacturing and installation process. Mass timber buildings are now being referred to as “carbon sinks”, as one ton of timber can store up to 520 lbs of carbon (Churkina et al., 2020). Additionally, as the use of structural timber increases, the demand for concrete and steel will decrease, once again helping reduce global CO_2 emissions. Construction is responsible for approximately 40% of all global carbon emissions, but hybrid and mass timber construction can provide a 15 to 26% reduction in global warming potential (Moore, 2022).

Another reason why cross-laminated timber use is on the rise is due to the quick and easy on-site installation. CLT panels are prefabricated off-site to any desired specification and size, even with pre-cut windows, doorways, stairs, and ducts. The prefabrication process generates almost no waste on-site and keeps construction workers on the ground while fabricating.

2.4 Vibrations

Vibration within timber floor systems is a common phenomenon typically governed by the mass, stiffness, and damping of the timber member. Other influencing factors include the system boundary conditions and excitation factors induced by humans or equipment placed directly on the floor (Huang et al., 2020). The *2019 Canadian CLT Handbook* defines two types of vibrations that can occur in CLT flooring: transient vibration and resonance. These vibrations are quantified through the fundamental natural frequency recorded for a given floor system and defined as a function of the floor mass and stiffness. In relation to floor stiffness, static deflection is the main dependent factor whereas the velocity and acceleration responses of the floor system are more likely dependent on the mass and excitation of the system in addition to stiffness (Karacabeyli & Gagnon, 2020).

In a CLT floor system, the vibration performance is affected by the spacing of supports and the size of screws used on connections because these two factors have a direct influence on stiffness. Additionally, designing a CLT floor with two-way supports rather than one-way was found to provide a stiffer floor as all beams will be in bending when the first natural frequency occurs. This frequency will only increase marginally as the magnitude of the bending stiffness will be larger in comparison to the magnitude of the torsional stiffness in the beam (Huang et al., 2020). In general, a natural frequency rating of above 8 Hz is recommended for a good and comfortable performance of CLT floor systems (Karacabeyli & Gagnon, 2020).

Studies, such as Huang et al. (2021), investigated the difference between the vibrational performance of solid CLT flooring panels in comparison to hollow-core cross-laminated timber (HC-CLT). A heel drop test was performed to find that a 3-ply CLT floor designed for the experiment had a base natural frequency of approximately 5 Hz. The static bending stiffness was deeply considered in this study, and the researchers defined the following equation for calculating the static bending stiffness of a CLT panel

$$(EI)_{CLTpanel} = \sum_{t=1}^{n} E_i I_i + \sum_{t=1}^{n} \gamma_i E_i A_i z_i^2 \quad \text{[Eqn. 1]}$$

where E_i is defined as the modulus of elasticity for layer *i*, I_i and A_i refer to the moment of inertia and area of layer *i*, and z_i refers to the distance from the centroid of layer *i*. γ_i represents the efficiency factor for connections; this value is non-zero for layers in the longitudinal direction except for the middle layer which is equal to unity (Huang et al., 2021). This study confirmed the concept that the bending stiffness of a CLT floor system was directly related to the thickness and spacing of the supporting timber members. However, a previous study performed by Huang et al. (2020) found that after a certain point, incremental increases to the beam size would not improve floor serviceability any further. The opposite was found in terms of reducing the beam size which could increase the resonance due to excitation and vibration (Huang et al., 2020).

The *2019 Canadian CLT Handbook* provides a method for the design of vibration-controlled CLT floors and exemplifies the use of fundamental natural frequency and static deflection in calculation. The span length for a vibration-controlled CLT floor was defined as

$$L \leq 0.11 \frac{\left(\frac{(EI)_{eff}}{10^6}\right)^{0.29}}{m^{0.12}} \quad \text{[Eqn. 2]}$$

where L is a function of the effective bending stiffness and mass of the panel. This panel was to meet the simple requirement of being on a load-bearing wall or supported by rigid beams. When comparing the span length and performance of calculated and actual CLT panels, it was found that the spans calculated with Eqn. 2 could be increased by up to 20% in order to account for inherent stiffness features for spans measuring less than 8 meters and floors without concrete topping.

The bending stiffness for a 1-meter-wide CLT panel was calculated in the outlined method using

$$(EI)_{app} = 0.9(EI)_{eff} \text{ [Eqn. 3].}$$

This bending stiffness, $(EI)_{app}$, was used as an approximation of the effective bending stiffness, $(EI)_{eff}$, taken in the major strength direction.

The equation for calculating the fundamental natural frequency of a CLT panel was defined as

$$f = \frac{3.142}{2L^2}\sqrt{\frac{(EI)_{app}}{\rho A}} \text{ [Eqn. 4]}$$

utilizing the vibration-controlled span calculated using Eqn. 2, applied bending stiffness, density, and cross-sectional area of a 1-meter-wide CLT panel.

Static deflection for the 1-meter-wide CLT panel was calculated through the following equation

$$d = \frac{1000pL^3}{48(EI)_{app}} \text{ [Eqn. 5]}$$

which again utilized the vibration-controlled span length, bending stiffness approximation, and load p. Load p was defined as a 1000 N or 1 kN load inducing the static deflection along the mid-span of the panel.

The static deflection and fundamental natural frequency were related to each other in the criterion for human acceptability of vibration.

$$\frac{f}{y^{x1}} \geq C \text{ [Eqn. 6]}$$

Above represents the human acceptability criterion of a CLT panel, C, as a function of the fundamental natural frequency, f, divided by the static deflection, y^{x1}. From this relation, the borderline of human acceptability was defined in Eqn. 7 below where the natural frequency divided by the static deflection is equal to the coefficient of human acceptability; this represents the minimum value or ratio of the natural frequency to the static deflection.

$$\frac{f}{y^{x1}} = C \text{ [Eqn. 7]}$$

When the equations above were used to check CLT floors already existing in the field, it was found that the majority of field floors had been designed more conservatively than the vibration-controlled design spans.

In addition to checking the CLT floor panels for vibration, it is important to check that the supporting beams meet the required stiffness criteria. If a supporting beam does not have adequate stiffness, then the flexibility of the beam can cause higher vibrations to occur in the floor panels. To check the supporting beam stiffness, the following equation can be used

$$(EI)_{beam} \geq F_{span} 132.17 l_{beam}^{6.55} \quad \text{[Eqn. 8]}$$

where EI_{beam} is the supporting beam bending stiffness, l_{beam} is the clear span of the supporting beam, and F_{span} is a constant which is either 1.0 for simple span beams or 0.7 for multi-span beams.

2.5 Building Codes

Building codes are set rules and regulations that a built structure must conform to in order to assure the health and safety of the public. These codes can vary from state to state but are ultimately considered to be part of jurisdictional law based on the enactment by the government. Unique to Massachusetts is building code 780 CMR 16.00, which details the structural design requirements based on a variety of factors from loading and building types to where the structure will be built specifically. Section 1605.00 outlines specific load combinations that a building must be able to safely resist and equations for different loading scenarios. Table 1607.01 provides the minimum uniformly distributed live loads and minimum concentrated live loads based on the building type. In regard to laboratory spaces specifically, the structure must be able to withstand at least 100 psf as well as a 2,000 lbs concentrated load. The dead loads are calculated based on the summation of material and construction weights identified in Table C-1 of ASCE 7. However, if definite information on these loads cannot be provided or obtained, then the applicable values will be subject to building official approval. Other necessary loads such as snow loads, seismic loads, and wind speeds were obtained from the 9th edition Massachusetts IBC structural amendment. This amendment provides the most current predetermined design values on the basis of the town or city location where construction will take place.

3. Methodology

The goal of this project was to design a five-story mixed laboratory and Class A office building using mass timber, to accommodate for the constraints of fire resistance and to investigate the vibrational effects on CLT. A cost analysis was provided as a comparative study between mass timber and typical construction materials. Based on the goals, the following objectives were defined for the project:

Objective 1: Design a Mixed Laboratory and Class A Office Mass Timber Building

Objective 2: Conduct a Vibrational Analysis of the Design

Objective 3: Perform a Cost Analysis for the Design Options

3.1 Design a Mixed Laboratory and Class A Office Mass Timber Building

Design for the five-story laboratory and office building started with reviewing the Massachusetts state building codes specific to Boston as well as the International Building Codes (IBC). A conceptual design was developed through the provided loading conditions, the geographic impact of the project location, and the desired end use for the building. A typical bay was designed for the building in order to size members. As shown in Figure 3.1 below, the bay would consist of four glulam columns supporting four glulam girders with a CLT floor panel on top and an additional glulam joist running down the center of the span in the same direction as the CLT panel configuration.

Figure 3.1. Sample Bay

A spreadsheet was created to perform all calculations for member sizing, including the design for fire resistance, and automated the process as different members were tested and selected. These members were selected using the Nordic X-Lam and Lam+ Technical Guides and the *2018 National Design Specification (NDS) for Wood Construction*. The Nordic X-Lam Technical Guide specifies the design criteria and conditions for the CLT panels available for use, and the Nordic Lam+ Technical Guide lists the specifications for glulam sizing. The NDS was used primarily to determine factored loading conditions for the bay and to provide a base for fire design.

3.2 Evaluate the Design for Vibration Performance

The vibrational analysis of the building began with an introductory tutorial from a graduate student at WPI who recently authored a vibrational analysis tool for mass timber assemblies. After becoming comfortable with the tool, the building assembly was entered into the spreadsheet to evaluate how the detailed design performed under the desired criteria. The non-fire design was first checked followed by the fire design. If neither design met the criteria, the building was redesigned with either different thicknesses for the concrete topping or girder and joist depths until a design was developed that met the criteria.

References for the vibrational analysis included the *2019 Canadian CLT Handbook*, *U.S. Mass Timber Floor Vibration Design Guide*, and *AISC Design Guide 11 Floor Vibrations Due to Human Activity*.

3.3 Perform a Cost Analysis for the Design Options

After completing the final design for the building, a quantity takeoff was performed from the 2D floor plans and structural designs. The main members focused on in the takeoff were the CLT panels and glulam column, joists, and girders. The dimensions of all the required CLT panels were specified to include the length and width of members, and the same was done with each glulam member along with their estimated run lengths and cross-sectional dimensions.

Once the takeoff sheet was complete, it was added to a project package that also included the architectural plans, Revit model, IFC file exported from the Revit model, and multiple renderings of the building. This package was sent to two mass timber fabricators to put in a request for quote (RFQ) for the project. The costs of materials and fabrication as well as the associated costs of transportation and delivery to Boston, Massachusetts were requested with each of the RFQs. Each company was met with to go over the scope of the project's work and what was desired from the estimates. Two weeks after the requests were made, each company was followed up with and final estimates were provided soon after. A breakdown of each RFQ is provided in the Cost Analysis section. The resulting cost estimates were compared to each other and to the cost of using alternative construction materials such as steel or concrete.

4. Design for a Mass Timber Laboratory and Office Building

4.1 Floor Layouts

The layout of the structure was provided by a professional architect, Tom Neal, based on an existing structure in Cambridge, MA (see Appendix B). The first floor consists of a cafe and seating area located in the main entrance's atrium, whose front street-facing walls are made of glass, allowing for natural light to illuminate the floor. The café and main atrium can be used by employees for lunch and will be large enough to host events for the building. On the remaining half of the first floor, there is a level-2 biosafety laboratory and a conference and boardroom. The rear half of the building will be rented out for individual shops and stores. The upper four floors will be mixed-use spaces and include multiple level-2 biosafety laboratories and class-A offices. The office space for each floor includes private and shared offices, meeting and conference rooms, private phone rooms, and both men's and women's restrooms. A large central staircase connects the third and fourth floors for the purpose of having one tenant occupy both floors. Figure 4.1 presents a typical floor layout for the building and the full layout for each floor can be found in Appendix B. The third floor is about one-half lab and one-half office space, while the fourth floor is only office space. The second and fifth floors are intended for one tenant each and will have a mix of lab and office space. An enclosed penthouse will be located on the roof to protect the structure's mechanical equipment.

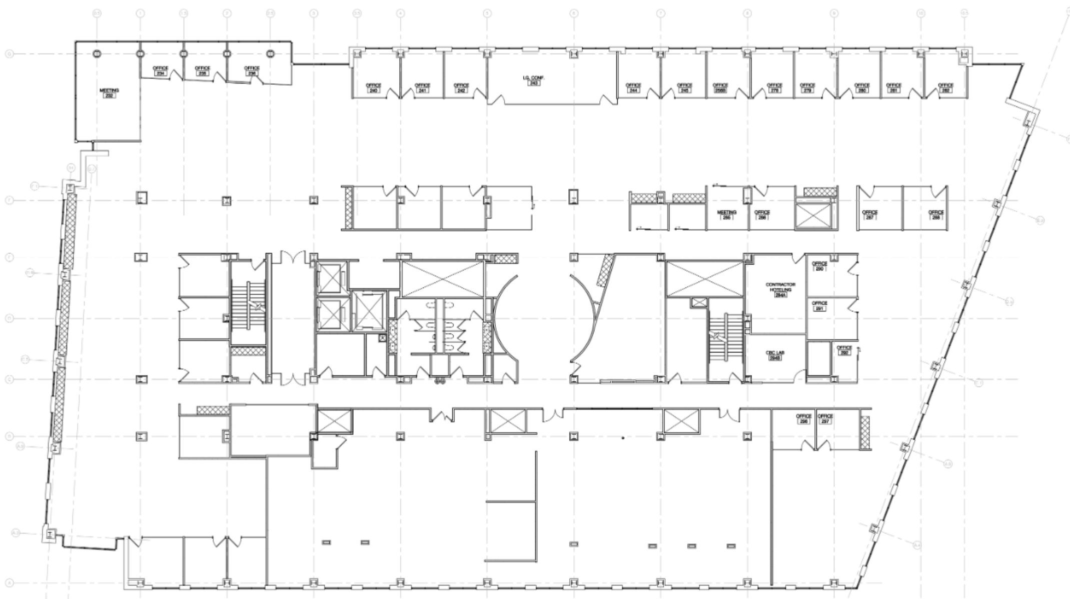

Figure 4.1. Typical Office Floor Layout

4.2 Gravity-Load Resisting System Design

With this project being located in Boston, Massachusetts, the design needs to meet the building code required by the state and local authority having jurisdiction. The first steps taken were to determine the gravitational loading that will be carried by the structure. Looking at the floor loads, Chapter 16 of the 2021 International Building Code (IBC) requires a live load for office spaces of 100 psf. (ICC, 2020) This live load was applied to each floor level in addition to the respective dead load. Next, the roof loading conditions were calculated. Since the roof is supporting a mechanical penthouse, the live load requirements are higher at 150 psf. The roof is flat and designed to have sufficient drainage to prevent unwanted ponding on the roof, allowing the rain load to be ignored. Snow is an additional gravity load that is a concern for structures built in cold climates. For Boston, Chapter 16 of the Ninth Edition of the MA State Building Code 780 states that the minimum flat roof snow load is 30 psf (BBRS, 2018). ASCE 7-10 Chapter 7.8 requires any roof structure with projections over 15 feet in length to have snow drift calculations as well (ASCE, 2010). Since our penthouse measures 40 feet by 30 feet, the snow drift calculations were applied and the snow drift loading was determined to be 83.34 psf.

After determining all gravity loads, the best order to design the structural members was to design them in the order of the load path. By this precept, the floor and roof decking systems were first designed, followed by the girders and joists, then the columns, shear walls, and finally the cross bracing.

4.2.1 CLT Floor and Roof Paneling

For the flooring systems of the structure, CLT panels were utilized so that the design would consist of panels all the same thickness for ease of constructability. CLT panels were designed separately for the roofing system based on its respective loading conditions, but all panels in the building, including the floors, would be eight feet wide. The orientation of the decking runs west to east in the floor plans, with the longest clear span distance of the CLT panels being 21 feet and 4 and a quarter inches. Since the loading is the same on all floors, this span was designed as the worst-case scenario, and the same panel thickness was applied throughout the floors. Another assumption made in the design was to continuously span the panels because this will allow the member to perform better in deflection and bending moment compared to that of a single-span member while also helping improve its vibrational properties. This meant that the panels were spanning two bays and across three supports, two at the ends and one in the center.

The floors were loaded with the 100 psf live load plus a 75.72 psf dead load. The dead loading includes carpeted/vinyl flooring, a five-inch thick concrete layer to aid in vibration performance (see Section 6 for further detail), MEPs, acoustical fiberboard, and a suspended steel channel system. The CLT panels were selected from the Nordic X-Lam Technical Guide, and their respective self-weight was added to the total dead load (Nordic Structures, 2022). Using the ASD approach, eight loading combinations (LC) were compared and the worst-case loading condition was selected. In this case, LC2 (DL+LL) governed the design of the CLT flooring, and LC3 (DL + (Lr or S or R)) governed the roofing. The determined worst-case load was then applied to the selected panel and evaluated for its resistance in shear, bending moment, and deflection. Excel spreadsheets, provided in Appendix C, were created to streamline the calculations and test different member sizes. If the selected CLT panel did not pass all of the criteria, a thicker panel from Nordic was selected, and the calculations were redone.

Since the building was designed from mass timber, fire code regulations were a large factor in the design of the structure. According to the IBC Chapter 6 Table 601 (Figure 4.2.1), the structure was classified as Type IV and heavy timber (HT), and required a fire rating of 1 hour for interior structural members. The char depth of wooden members is dependent on the fire rating of the building which greatly impacts its strength. Since a concrete topping was included in the design, the top surface of the CLT was protected and will not undergo charring. However, the underside was still susceptible to the effects of charring. Once the fire-designed CLT panel allowable loading surpassed the actual loading conditions, the panel was then assessed for its vibration performance which is discussed in further detail in Section 6.

BUILDING ELEMENT	TYPE I		TYPE II		TYPE III		TYPE IV				TYPE V	
	A	B	A	B	A	B	A	B	C	HT	A	B
Primary structural frame[f] (see Section 202)	3[a, b]	2[a, b, c]	1[b, c]	0[c]	1[b, c]	0	3[a]	2[a]	2[a]	HT	1[b, c]	0
Bearing walls												
Exterior[e, f]	3	2	1	0	2	2	3	2	2	2	1	0
Interior	3[a]	2[a]	1	0	1	0	3	2	2	1/HT[g]	1	0
Nonbearing walls and partitions Exterior	See Table 705.5											
Nonbearing walls and partitions Interior[d]	0	0	0	0	0	0	0	0	0	See Section 2304.11.2	0	0
Floor construction and associated secondary structural members (see Section 202)	2	2	1	0	1	0	2	2	2	HT	1	0
Roof construction and associated secondary structural members (see Section 202)	$1^1/_2$[b]	1[b,c]	1[b,c]	0[c]	1[b,c]	0	$1^1/_2$	1	1	HT	1[b,c]	0

Figure 4.2.1. IBC Table 601 Fire-Resistance Rating Requirements for Building Elements (Hours)

The CLT panels for the main roofing system were designed following the same process as the flooring. The roof was subject to a loading condition of 150 psf live load and a 38.4 psf dead load which only consisted of the panels' self-weight and the weight of the MEP. The roof was analyzed for the one-hour fire resistance rating, and once the required criteria were met, the panel was analyzed for vibrations. A fire-resistance rating was not required for the CLT panels used for the penthouse roof. This is because IBC Code 1511.2.4 Exception C states that for building types III – V, a penthouse with fire separation distances greater than twenty feet and permitted to be heavy timber construction shall not be required to have a fire-resistance rating (ICC, 2020). No vibrational analysis was conducted for the penthouse roof either as it was not designed to support any sensitive equipment.

4.2.2 Glulam Girders and Joists

The second set of structural members designed were the girders and joists. These members carry the CLT panels above and transfer this loading into the columns. As the layout of the structure is not completely uniform, various span lengths were required, ranging from around eight to twenty-one feet. Similar to the CLT flooring panels, continuously spanned beams were used to limit the deflections and bending moments in the structure. Three-span continuous girders were designed to measure roughly sixty-five feet in length. The girders in the structure run in the north-south direction whereas the joists run in the east-west direction. The CLT panels span in the same direction as the joists, thus each joist was designed to only support the width of one CLT panel (see Figure 4.2.2.1). The girders, on the other hand, support the loading of multiple CLT panels. Due to the locations of critical loading on the continuously spanned CLT panels, the load supported by the girder was not evenly distributed. For a continuously spanned beam with a uniformly distributed load, the reaction in the center is $\frac{10wL}{8}$, while the reaction at the ends is $\frac{3wL}{8}$. This meant that the most critical loading to design for was supporting the middle of the continuously spanned panel.

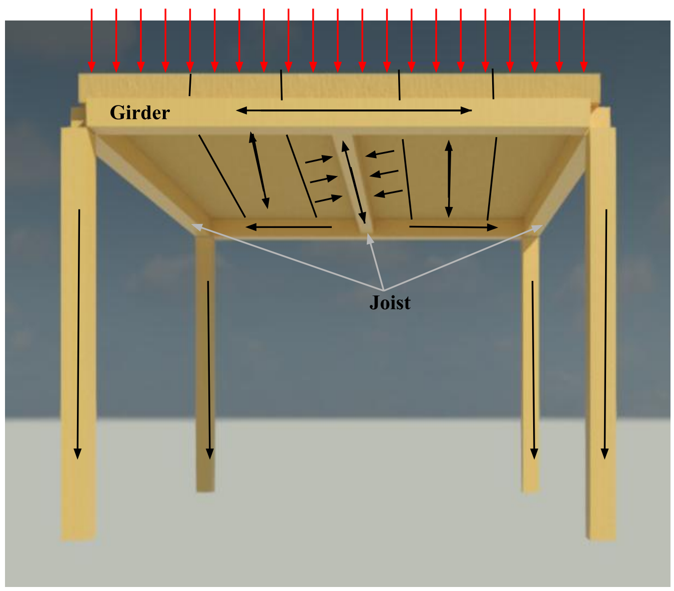

Figure 4.2.2.1. Load Path in a Bay

Once the loading was calculated, the Excel spreadsheets were created for the girders and joists and are presented in Appendix C. Glulam beam dimensions were selected from the Nordic Lam+ Technical Guide and the associated section properties were entered into the spreadsheet (Nordic Structures, 2022). Using ASD design, calculations were run to check for the allowable moment, shear, and deflection. After a beam was sized to meet the criteria for normal conditions, one-hour fire conditions were specified by NDS Chapter 16 Table 16.2.1B (Figure 4.2.2.1) (NDS, 2018). These values were applied to the allowable loading calculations and compared with the actual loading conditions. If the actual loading conditions surpassed the allowable, the width and or depth of the member were increased. The same design process was applied to the roof girders and joists as well. The design of the beam system in the penthouse was simpler as the penthouse is not required to meet any fire conditions. See Figures 4.2.2.3 through 4.2.2.5 for the structural framing plans.

Required Fire Resistance (hr.)	Char Depth, a_{char} (in.)	Effective Char Depth, a_{eff} (in.)
1-Hour	1.5	1.8
1½-Hour	2.1	2.5
2-Hour	2.6	3.2

Figure 4.2.2.2. NDS Table 16.2.1B Effective Char Depths

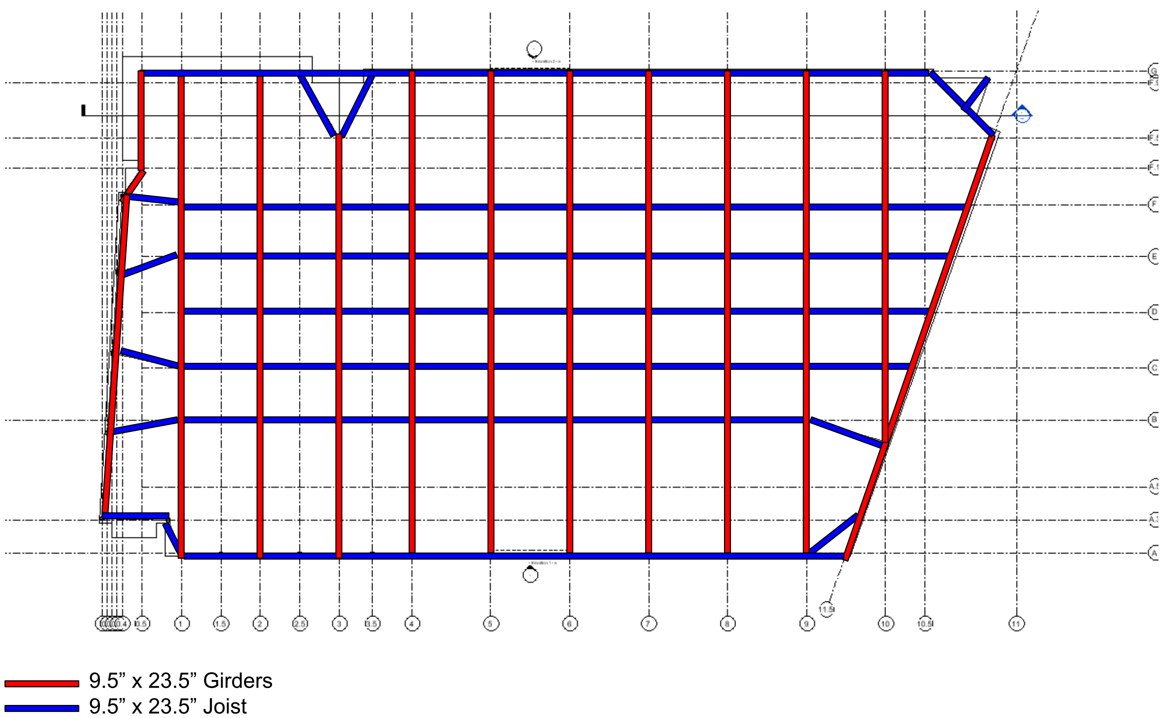

Figure 4.2.2.3. Structural Framing Plan for Floors 1-4

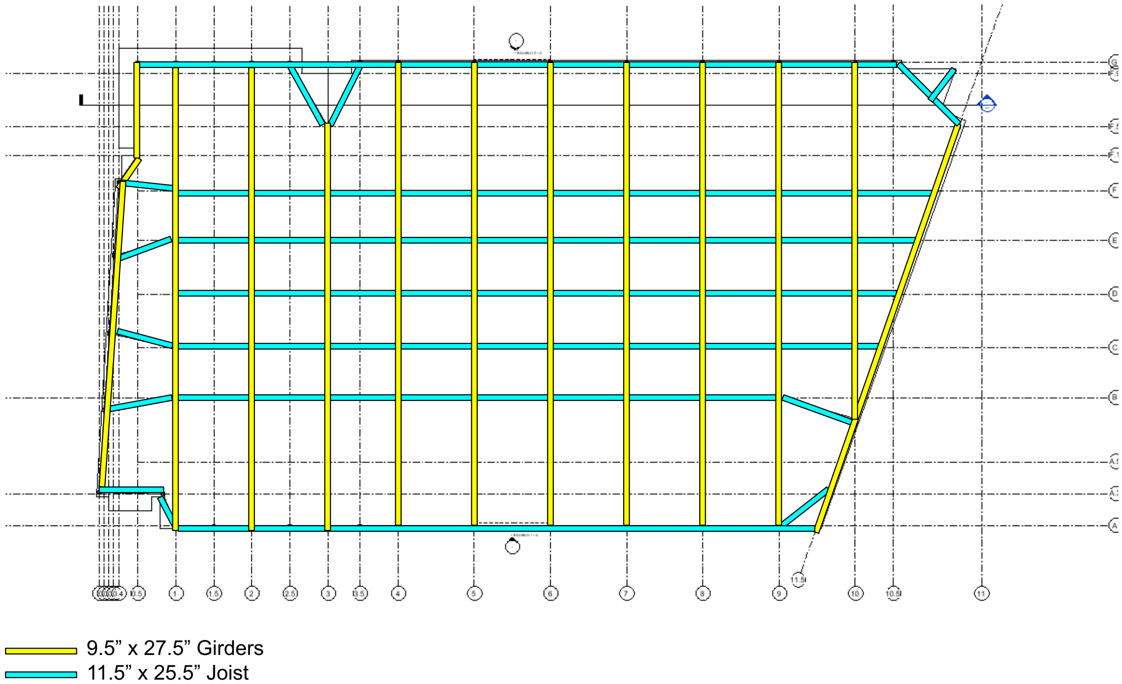

Figure 4.2.2.4. Structural Framing Plan for Roof

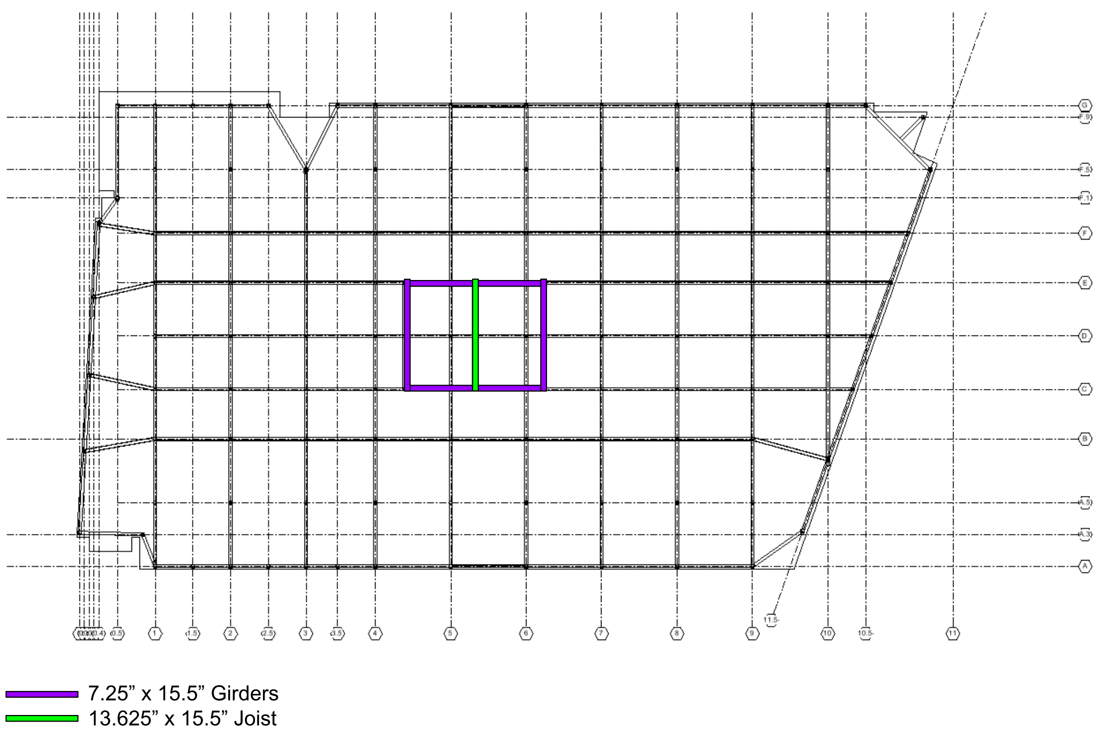

Figure 4.2.2.5. Structural Framing Plan for Penthouse Roof

4.2.3 Glulam Columns

The columns were the last gravity load-resisting members to be designed for as they carry the loading of the supported CLT panels and glulam girders and joists. Again, the most critical loading conditions for a column were selected for the design and used throughout the entire floor to improve the design from a construction standpoint. The columns supporting the structure's roof and penthouse roof were controlled by ASD load combination 3, DL+ (Lr or S or R), while the floor columns were controlled by ASD load combination 1, DL+LL. The interior and exterior columns have different loading conditions. The interior columns support tributary areas in all directions while an exterior column supports loading on only one side. Loading on one side creates eccentricity (an offset between the centroid of the load from the centroid of mass of the supporting member) that increases the bending moment in the column. Through analysis of the interior and exterior columns, it was determined that despite the effects of eccentricity on the exterior columns, the interior columns experienced a higher load and required a larger member size. In the interior, there are two different loading conditions on the girders. In continuously spanned members, the reaction forces in the interior are larger than the sum of the two end reactions. This meant that the most critical columns to design for were those that were placed on the interior of the structure in the middle of a continuously spanned girder.

To design the column, the loading on the supported girders and joists was converted into reaction point loads. The loads on the columns included the gravity loads and the self-weight of the girders, joists, CLT panels, and columns from the overlaying floors and roof. Once the axial loading was calculated, an Excel spreadsheet was created to determine the required column dimensions using ASD methods (Appendix C). Member dimensions and sectional properties were selected from the Nordic Lam+ Technical Guide (Nordic, 2022). Adjustment factors and effective char depths were taken from the NDS to account for a one-hour fire resistance rating. These values were applied to the allowable loading calculations and compared with the actual loading conditions. If the actual loading conditions surpassed the allowable, the width and depth of the column were increased. Once a member met the loading conditions, the columns' slenderness ratio, effective length to depth, was checked to be less than 50 for both the major and minor axes; these requirements were set by Chapter 3.7 of the NDS (NDS, 2018). The structural framing plan for columns is found below in Figure 4.2.3.1 and Figure 4.2.3.2.

Separate calculations were done for all five floors and the penthouse columns. The height of the columns on the main floors is fourteen feet while the columns of the penthouse are eighteen feet tall. The fourteen-foot height of the columns allowed for ample headroom and a satisfactory amount of space for the installation of MEP systems all while maintaining a ceiling height of 8 feet above the finished floor. This also keeps the building height under the 85 foot limit set by Chapter 5 of the IBC for type IV-HT-B structures (ICC, 2021).

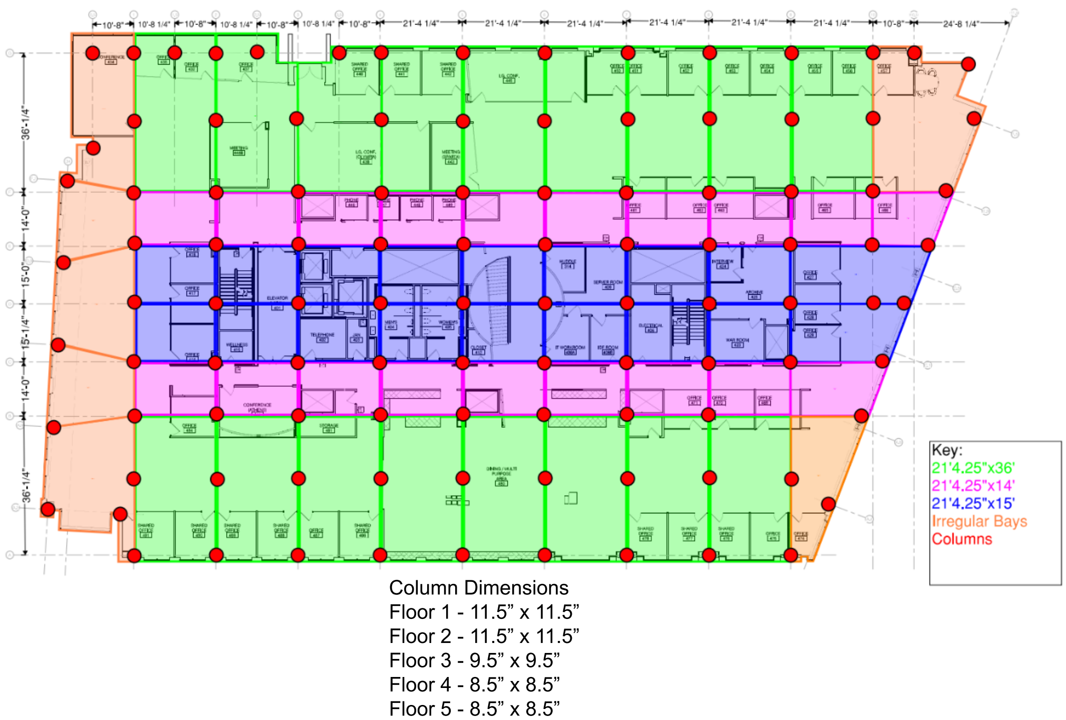

Figure 4.2.3.1. Column Structural Framing Plan for Floors 1 - 5

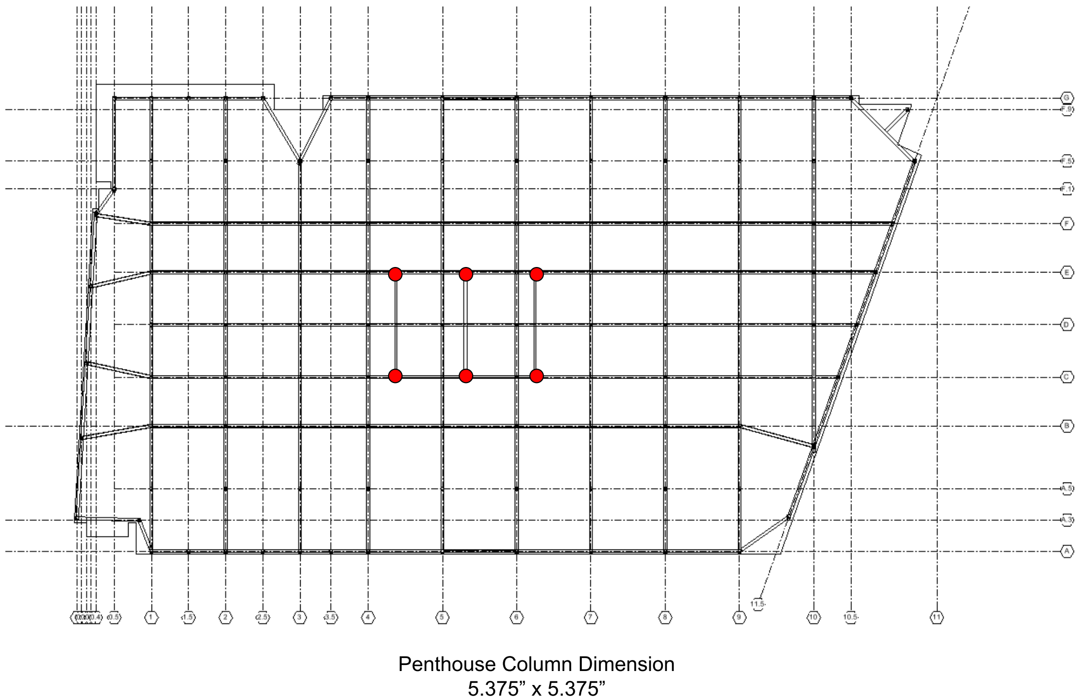

Figure 4.2.3.2. Column Structural Framing Plan for Penthouse

Table 4.2.3 Critical Member Dimensions Fire vs Non-Fire Conditions

Member	Fire Conditions	Non-Fire Conditions
CLT Flooring 1-4	143-5S	175-5S
CLT Roof	143-5S	175-5S
Glulam Girders Floors 1-4	9.5” x 25.5”	9.5” x 23.5”
Glulam Girder Roofing	9.5” x 27.5”	9.5” x 23.5”
Glulam Columns	Floor 1 - 11.5” x 11.5” Floor 2 - 11.5” x 11.5” Floor 3 - 9.5” x 9.5” Floor 4 - 8.5” x 8.5” Floor 5 -8.5” x 8.5”	Floor 1 - 11.5” x 11.5” Floor 2 - 11.5” x 11.5” Floor 3 - 9.5” x 9.5” Floor 4 - 7.25” x 7.25” Floor 5 - 7.25 “ x 7.25”

4.3 Lateral Load-Resisting System Design

The lateral load-resisting system is an important element that prevents collapse by lateral forces. For this structure, two lateral-resisting systems were analyzed: shear walls and cross bracing. The lateral force-resisting systems perform best when symmetrically placed in the structure away from the center point of the floor layout which allows for a longer moment arm. With this in mind, the most optimal location to place the resisting systems was on the outer bays of the structure. However, a design using CLT panels as shear walls on the exterior of the building would create an architectural conflict with the curtain walls. This led to the design of an inverted V-bracing for the lateral system in the west-east direction that is structurally efficient and allows for window installations in the bays (See Figure 4.3.2). As shown in Figure 4.3.1, the cross bracing was located on each floor near the center of the north and south exterior walls of the building. Shear walls in the north-south direction were designed as interior walls for the stairway shafts of the building.

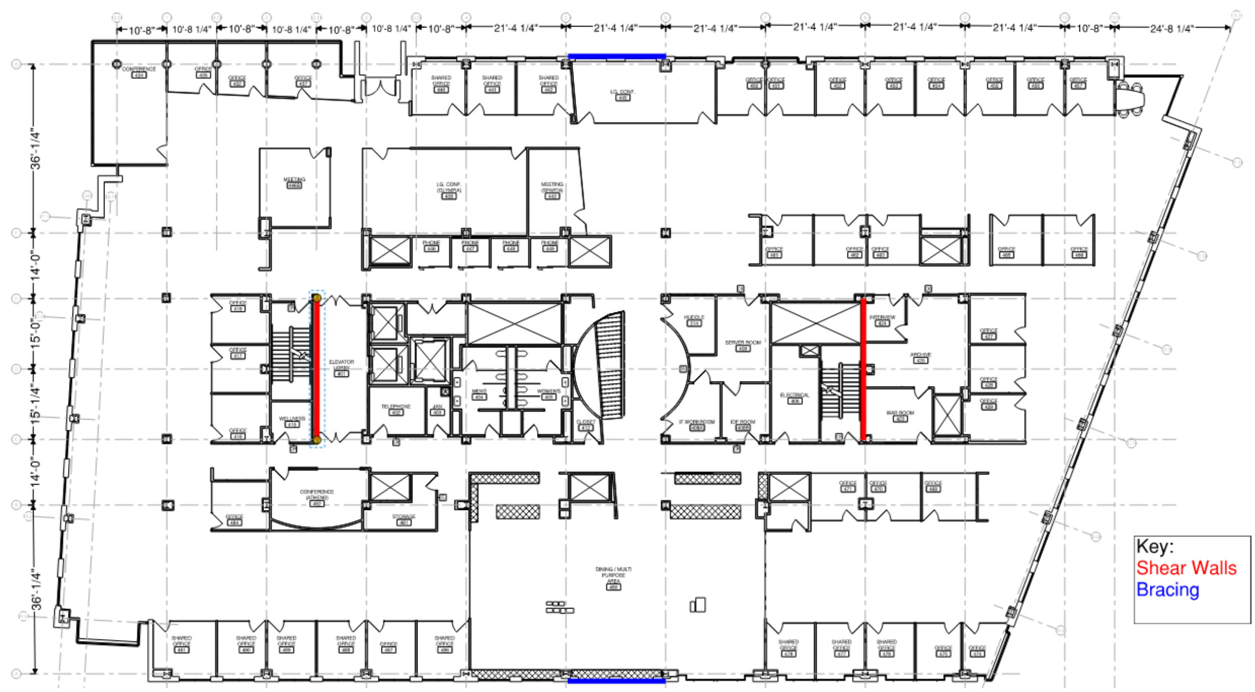

Figure 4.3.1. Lateral Support System

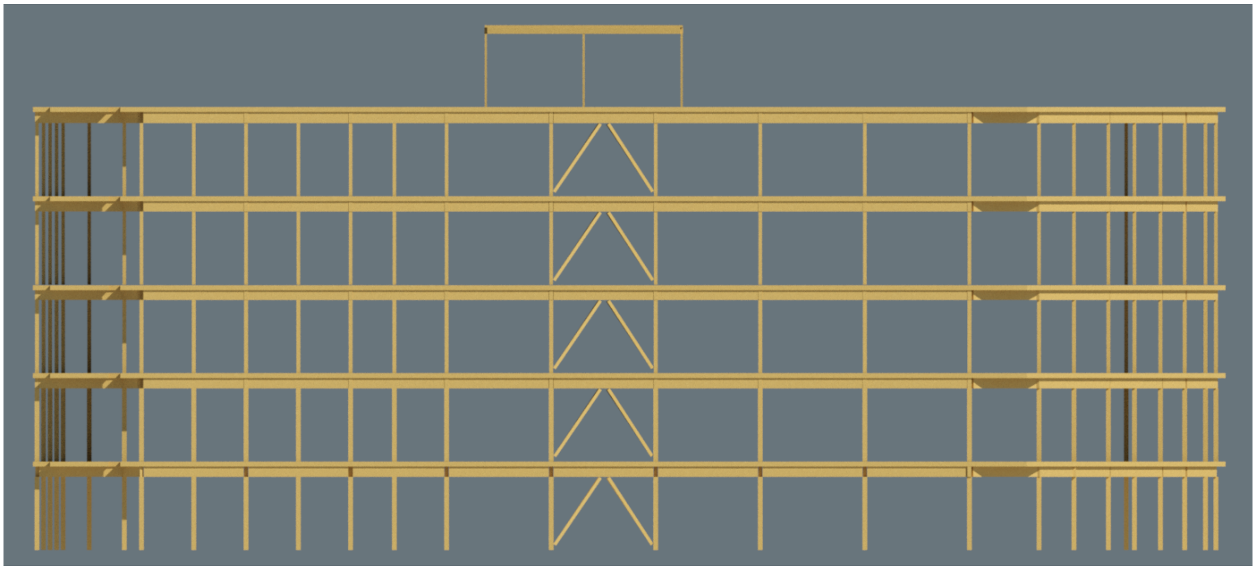

Figure 4.3.2. Elevation View with Inverted V-Bracing

4.3.1 Seismic Loading

The seismic loading on the structure was calculated using the *ASCE 7-10* Seismic Analysis Program Excel spreadsheet. This program combined IBC and ASCE 7-10 design guidelines and values based upon construction location. This building is assumed to be constructed in Boston Massachusetts, so the spectral accelerations, S_s and S_1, were determined based on the city's location. As the exact site in the city is unknown, the soil classification was reasonably assumed to be Class D, for stiff soils. Risk Category II was also assumed for this building since it would pose less of a hazard than those defined for Risk Category III in the IBC. The height above grade and weight of each story was entered into the spreadsheet and the shear forces were calculated. The *ASCE 7-10* Seismic Analysis Program can be found in Appendix D with the key values summarized in Table 4.3.1 below.

Table 4.3.1. Summary of Key Seismic Values

Factors	
SDS	0.235
SD1	0.104
Fundamental Period (Seconds)	0.651
Response Mod. Coef. R	2.5
CS	0.94
Base Shear (kips)	281.42
Story Forces (kips)	
Penthouse	14.69
5	91.64
4	70.91
3	52.71
2	34.86
1	16.61

4.3.2 Wind Loading

The wind loading on the structure was calculated using FLSmidth's MWFRS Wind Load Excel sheet based on ASCE 7-10 (Appendix D). The spreadsheet takes into account the structure's geographical location, risk category, windward and leeward face geometry, and roof geometry. Based on Table 1.5-1 and Figure 26.5-1A of ASCE 7-10, the building falls into risk category II and is exposed to basic wind speeds of up to 128 mph. As the structure does not have an actual location in Boston, the site's topography, vegetation, and constructed facilities are unknown. Due to this missing information, exposure category C was conservatively selected. This category is for structures in "open terrain with scattered obstructions having heights generally less than 30 ft."(ASCE, 2010). Lastly, the building's geometry was entered into the FLSmidth spreadsheet in order to produce the necessary calculations. A summary of the key values can be found in Table 4.3.2 below. It was then observed that the wind pressure on the walls of the structure was highest at the roof level and decreased as you traveled down. (See Figures 4.3.2.1 and 4.3.2.2).

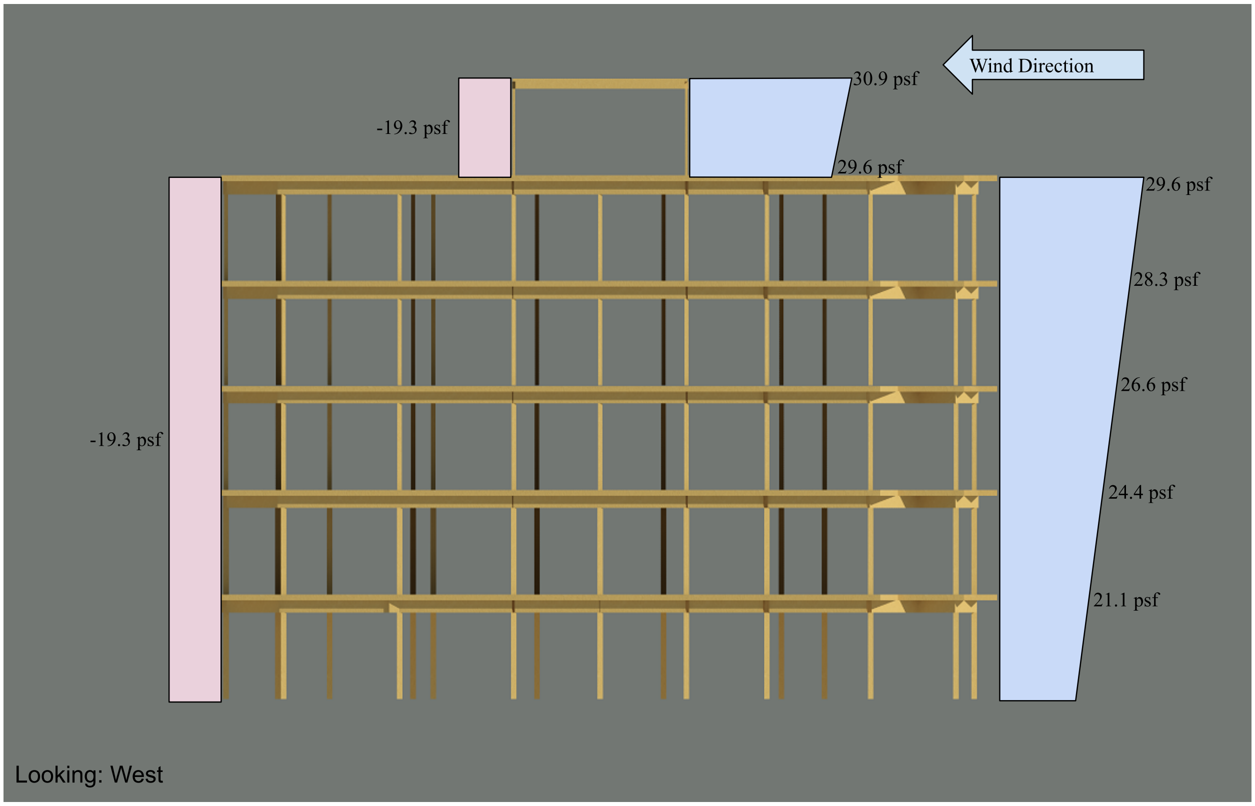

Figure 4.3.2.1. Wind Load Distribution on North-South Direction

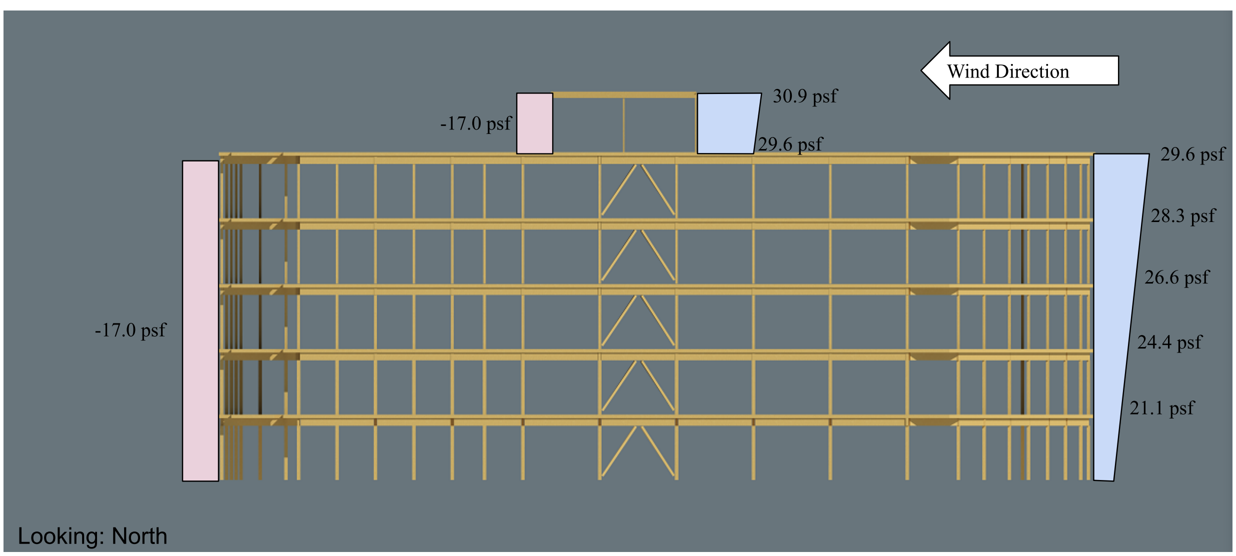

Figure 4.3.2.2. Wind Load Distribution on West-East Direction

Table 4.3.2. Summary of Key Wind Values

Factors	
Wind Direction (K_d)	0.85
Topographic (K_{zt})	1.00
Gust Effect (G)	0.85
North-South Windward	
WW + LW (psf)	40.4 to 50.3
LW (psf)	-19.3
Side (psf)	-27.1
West-East Windward	
WW + LW (psf)	38.1 to 48.0
LW (psf)	-17.0
Side (psf)	-27.1

4.3.3 Shear Walls and Cross Bracing

Through analysis, it was determined that the lateral wind loading was more critical than that of seismic loading. To resist the lateral loading caused by the wind forces, two different forms of shear resistance were analyzed. First, CLT panels were placed on all five floors, and using an Excel spreadsheet (Appendix C), the required shear wall lengths were calculated. Although the wind load on each floor increased with each story, the greatest length of shear walls was required at the bottom of the building. This is due to the fact that the first floor needs to resist the total lateral force acting on the overlying floors. CLT panel thicknesses were selected from the Nordic X-Lam Technical Guide and the required shear wall length per floor was calculated. If the wall length was determined to be too long, the panel thickness was increased until the shear wall length was satisfactory. Once the shear wall length was satisfactory, fire conditions were then placed on the panel and the required shear wall length per floor was re-calculated (Appendix C).

Table 4.3.3. Shear Wall with Fire Conditions Summary

	North-South Side (Short Side)	East-West Side (Long Side)
Shear Wall Panel	197-7S	197-7S
Wall Length Floor 1 (ft)	20	42
Wall Length Floor 2 (ft)	18	34
Wall Length Floor 3 (ft)	14	26
Wall Length Floor 4 (ft)	10	18
Wall Length Floor 5 (ft)	6	10
Wall Length Penthouse (ft)	2	4
Total Shear Wall Length (ft)	70	134

5. Design Layouts

The initial design process began with the assumption that a typical bay size would be a standard eleven by twenty-two-foot section. Following this scale, the provided architectural floor plans were measured using BlueBeam and divided into bays based on the respective column placement.

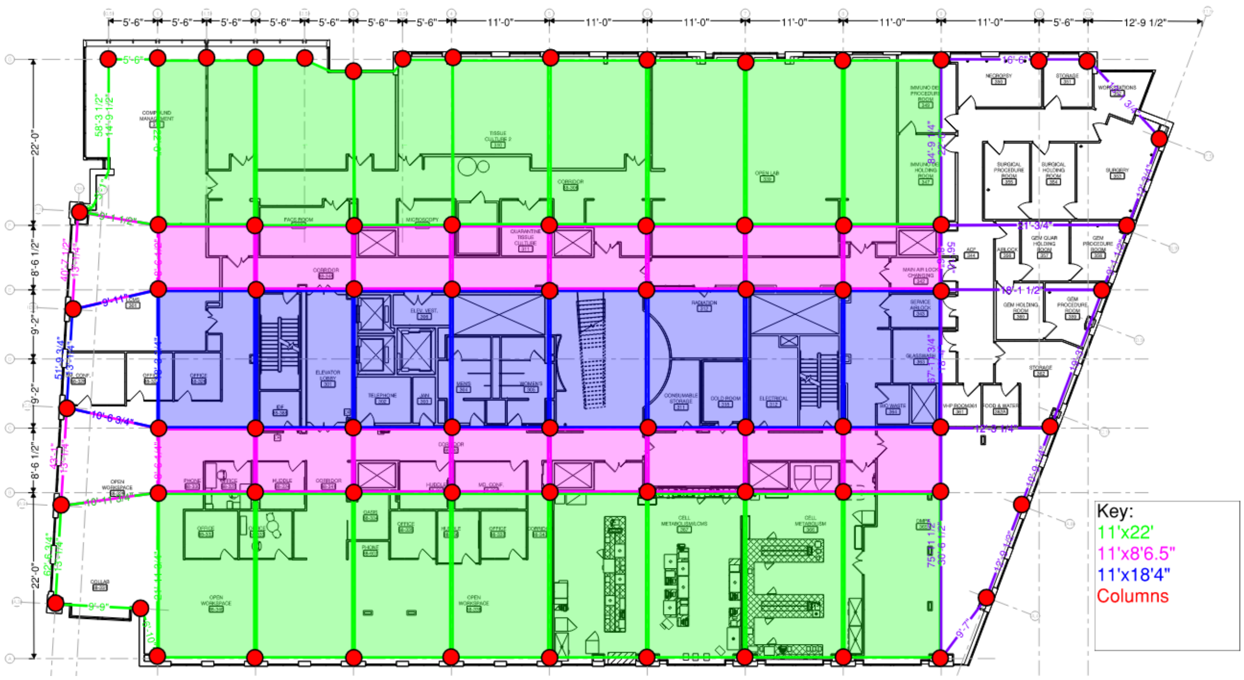

Figure 5.1. Original Floor Plan Layout

The layout shown in Figure 5.1 governed the design process for the structural design and analysis spreadsheet tool that had been created for calculations. During this process, the structural design had taken into account a conservative two-hour fire rating which required a 3.2-inch effective char-depth for any exposed side to be deducted from the overall member dimensions. Under these circumstances, the sizing of members was forced to significantly increase in order to make up for the loss of cross-sectional area due to char. In turn, this extended the height of each story and produced a total building height of over 106 feet which placed the design into high-rise jurisdiction and outside the intended project scope.

In addition to the need for decreasing the overall building height, new bulletin changes had been received from the architect which provided true-to-scale measurements of the floor plans (Figure 5.2). These new measurements were doubled in scale compared to the previous assumption with the initial design.

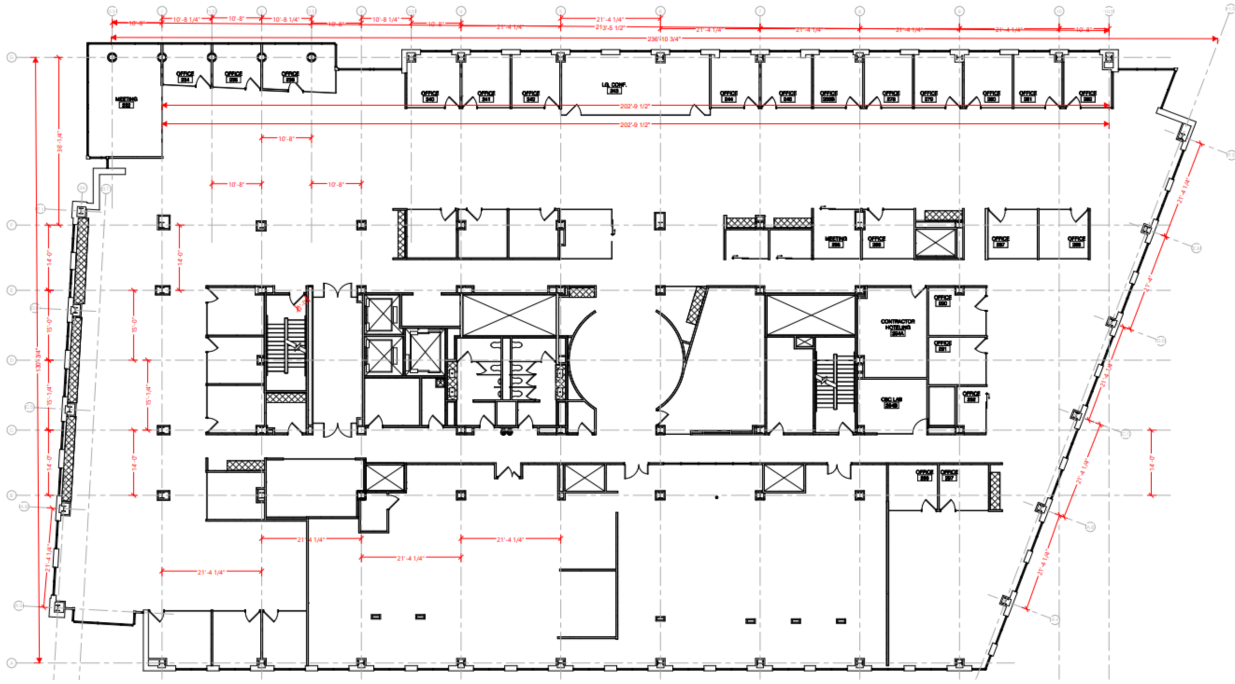

Figure 5.2. Bulletin Changes Provided by the Architect

Now the question became: how can the building height be decreased while maintaining the same structural bay layout, but larger in scale? The resolution came with alterations within the floor plan spacing as well as adjustments in how fire protection would be achieved within the regulation of the code. As depicted in Figure 5.3, a number of columns were added to create more manageable span lengths which would not require oversized structural members. However, the tradeoff would require room layouts to be slightly reconfigured, and some of the larger open areas would be aesthetically diminished due to the columns in the noticeable view. The approach to fire protection transitioned from designing for a two-hour fire rating using effective char-depth calculations to utilizing an intumescent paint that acts as a fire retardant. This allowed for the design of structural members to take on a much more efficient size. Additionally, the column height had been dropped from fifteen feet in the original design to fourteen feet in order to reduce the overall building height all while maintaining sufficient spacing above the ceiling for MEPs. Following this complete redesign, the height of the building still exceeded the seventy-foot high rise limitation set by the Massachusetts building code (MA State Board of

Building Regulations and Standards, 2009) as well as the allowable building height above the grade plane set by the IBC (ICC, 2021). However, this was due to the added height of the penthouse rather than the occupiable floor levels and ultimately solidified the conclusion that this project would require special provisions which would need to be acquired through permits from local and state authorities (ICC, 2021).

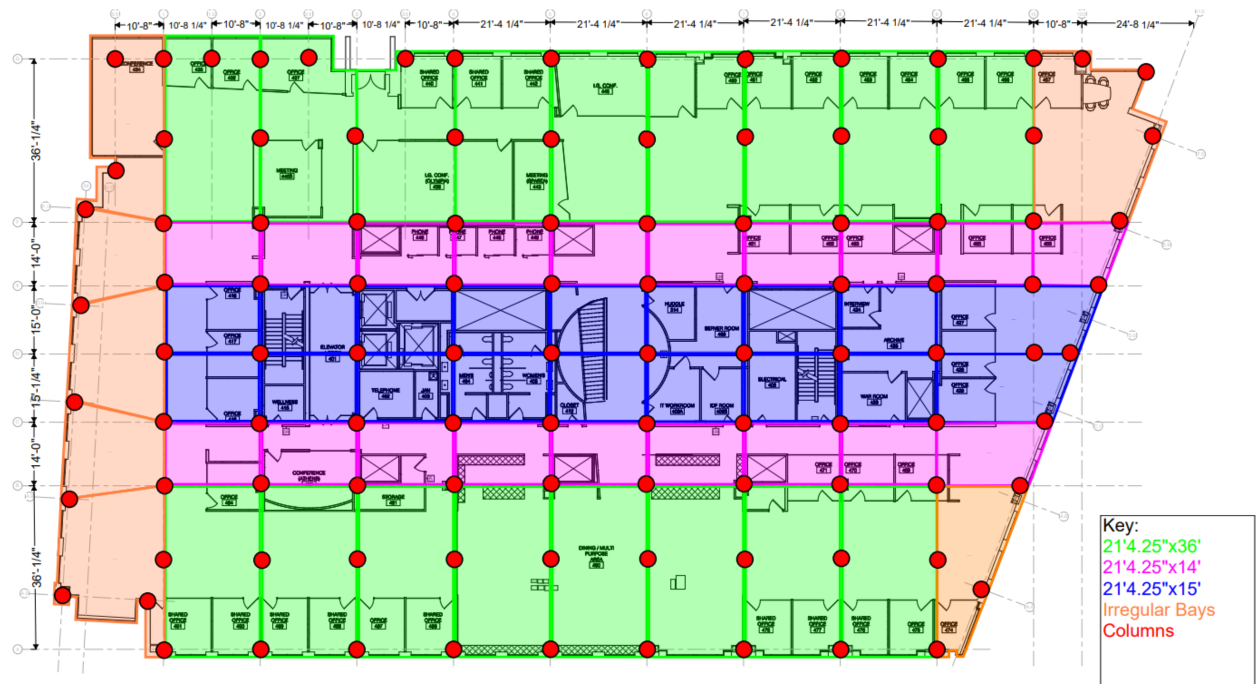

Figure 5.3. Final Floor Plan Layout

The last design alternative explored the use of metal decking as opposed to CLT panels. During this phase of the design process, a Vulcraft Steel Deck catalog was used to expedite the calculations because it provided the allowable superimposed uniform load values in correlation to the deck's clear span length. Upon review of the prescribed loading criteria for the structure, it was determined that sixteen gauge 2LVI composite decking would be needed to safely carry all applied loading and maintain an acceptable level of vibration throughout the building (Vulcraft, 2020).

6. Vibrational Analysis

Once the final design had been achieved, design properties were transferred into a vibration design analysis spreadsheet provided by graduate student Stephanie Bishop who developed the tool as part of a research project. This tool requires a series of inputs as specified in Table 6.1. in order to produce vibrational analysis results and even recommendations on how to better enhance performance, see Appendix E for more information.

Table 6.1. Vibration Design Analysis Spreadsheet Inputs

Design Inputs
1. Performance Target
2. Walking Parameters
3. Floor Layout
4. Floor Material Properties
5. Damping
6. Loading

Prior to incorporating a composite floor system into the final design, the use of solely mass timber had been proven to be insufficient for achieving an acceptable level of vibration for lab space. The root mean square (RMS) velocity fell just under 500,000 mips with over 2% of gravity acceleration. To provide some context, depending on the level of the laboratories being designed for, an acceptable RMS velocity will typically range from 125 mips to 8,000 mips with 0.3% gravity acceleration. The more sensitive the research and instrumentation are then the lower the RMS velocity will need to be to not interfere with equipment being used within the space. Even in comparison to the original design which was about half the scale of the final design with larger oversized members, it was still insufficient in meeting vibration requirements for a laboratory. This led to the conclusion that a composite floor system using an added concrete layer would be required to fulfill such needs, especially with the longer span lengths of the final design.

The vibration analysis tool also provided graphs that display the design results in comparison to human perception based on the noticeability of RMS velocity and peak acceleration.

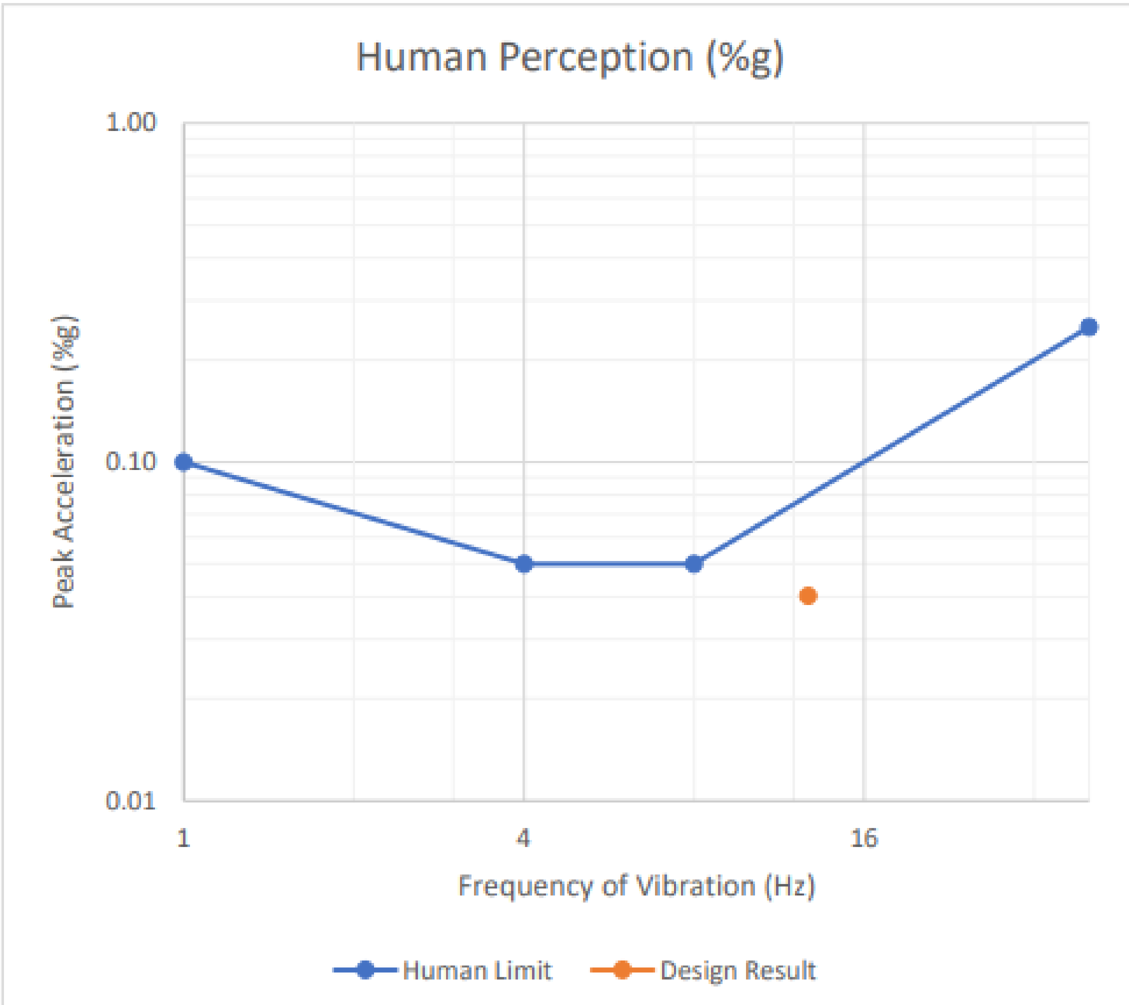

Figure 6.1. Human Perception of Vibration-Based On Peak Acceleration

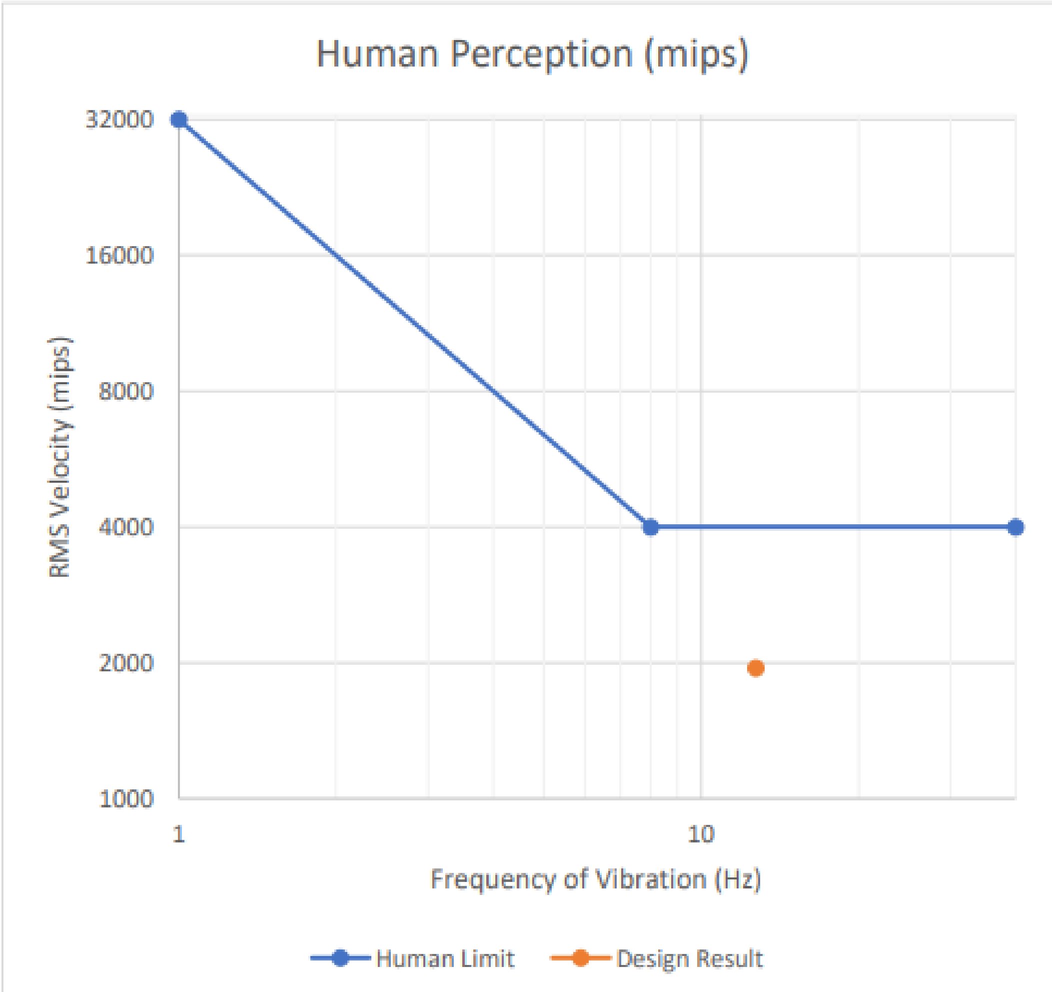

Figure 6.2. Human Perception of Vibration-Based On RMS Velocity

These graphs were used to determine how thick of a concrete layer would be needed to increase damping enough to not only meet standard lab requirements but also fall below the threshold of human perception, see Appendix E for more information.

In the analysis of sixteen gauge metal decking, using the same design parameters and loading conditions, it was found that seven and three-quarters inches of concrete would be required to meet the vibrational criteria for the project. Even with a corrugated height of two inches, the combination of steel decking and concrete would be slightly over two inches shallower than CLT and concrete.

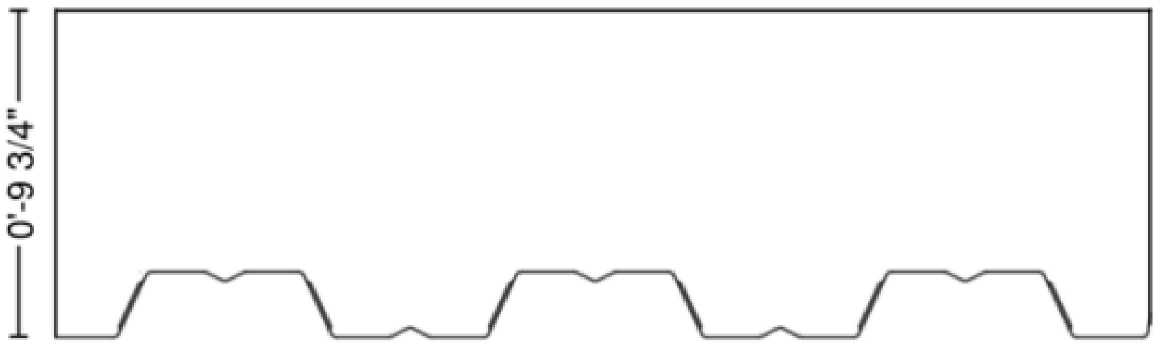

Figure 6.3. Concrete On Metal Decking

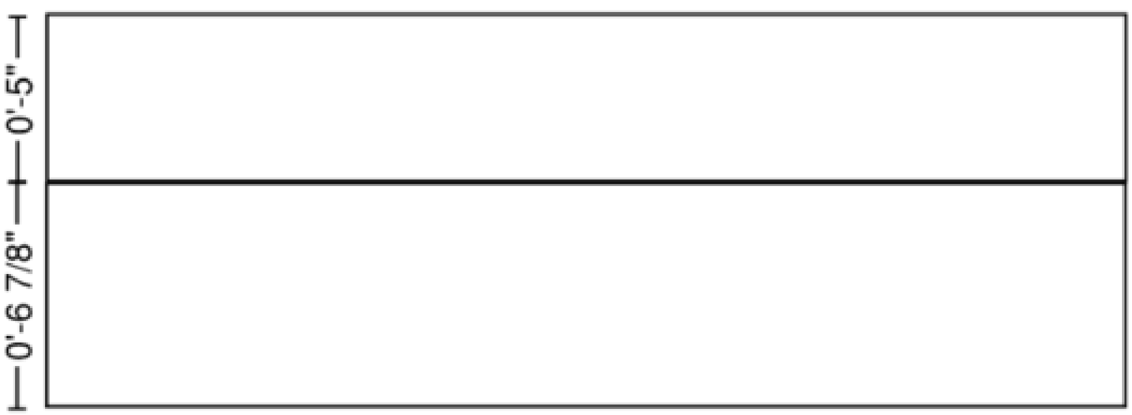

Figure 6.4. Concrete On CLT Paneling

7. Cost Analysis

The costs of manufacturing, transportation, and installation were obtained from two mass timber fabricators. Table 7.1 summarizes the total material quantities required for the construction of our final design which was presented in our RFQ; a more in-depth summary is provided in Appendix E. Mass timber fabricator #1 was able to provide the costs for trucking and associated taxes for delivery and installation in Boston, Massachusetts; however, fabricator #2 did not disclose these costs. To create a more comparative cost analysis, the same costs were applied to fabricator #2's material estimate.

Table 7.1. Material Quantities

	Total Volumes		Total Areas	
	m^3	CF	m^2	SF
CLT	2,442	86,238	13,930	150,000
Glulam	1,110	39,195	-	-

Table 7.2. Estimate Breakdown from Fabricator #1

Material/Item	Unit Price	Total Cost
Glulam (Includes Equipment)	$52.82 per CF	$2,070,224
CLT (Includes Equipment)	$19.65 per SF	$2,947,816
Trucking (105 Trucks)	$5,340.96 per truck	$560,801
Tax	-	$350,000
FSC	-	$21,159
	Estimate Total	$5,950,000
	Building Cost per SF	$39.67

Table 7.3. Estimate Breakdown from Fabricator #2

Material/Item	Unit Price	Total Cost
Glulam	$90 per CF	$3,527,550
CLT	$35 per 5 Ply SF (estimated based on the $25 per 3 Ply SF)	$5,250,000
Trucking (105 Trucks)	$5,340.96 per truck	$560,801
Tax	-	$350,000
FSC	-	$21,159
	Estimate Total	$9,709,510
	Building Cost per SF	$64.73

From the two estimates, an average cost for construction of the building was found to be $7,829,755 with a cost per square foot ranging from $39.67 to $64.73. The cost estimates for this building are subject to change as a result of fluctuations based on supply-demand trends. For the final building design, a five-inch concrete topping is specified on top of the CLT panels in order to reduce the effects of vibration. If this cost is included, the average cost would increase by approximately $289,352 to $8,119,107.

The alternative sixteen gauge metal decking design option would have a cost of about $2,684,063 as shown below in Table 7.4. Compared to the cost for the concrete topping and CLT panels alone, the estimate from the fabricators was significantly higher than the cost of using metal decking instead of CLT.

Table 7.4. Alternative Cost with Metal Decking

Material/Item	Unit Price	Total Cost
16 Gage Metal Decking	$15.00 per SF	$2,250,000
7.5 Inch Concrete Topping	$4.63 per CF	$434,063
	Total	$2,684,063

8. Conclusions and Recommendations

The goal of this project was to design a mixed laboratory and Class A office building using mass timber and to accommodate for the constraints of fire resistance and vibrational design. A cost analysis was provided as a comparative study between mass timber and typical construction materials such as structural steel and reinforced concrete. Based on the goals, the following objectives were defined for the project:

Objective 1: Design a Mixed Laboratory and Class A Office Mass Timber Building

Objective 2: Evaluate the Design for Vibration Performance

Objective 3: Perform a Cost Analysis for the Design Options

Each of these three objectives were successfully completed for the project. The completion of objective one was marked by the development of a detailed building design which accounted for a one-hour fire resistance rating and applicable loading conditions including lateral loading and snow drift. The second objective was completed with a redesigned building layout to accommodate performance requirements for floor vibrations. This objective began by analyzing the vibrational performance of the design using a spreadsheet tool developed by Stephanie Bishop. Once the design's performance was established in relation to the target performance, a concrete floor topping was incorporated into the design to reduce the peak acceleration and RMS velocity. The floor panels, girders, joists, and columns were redesigned accordingly to accommodate the increased dead load due to the weight of the concrete. The final objective was completed after receiving estimates from two leading mass timber fabricators and comparing the costs to typical construction materials. It was found that the estimated building cost per square foot was between $39.67 to $64.73 for the mass timber design and when the cost of the concrete floor topping is factored in, the cost per square foot will increase by $1.93.

Key takeaways from this project were that a five-story mixed-use laboratory and office building would be difficult to design with mass timber and meet height requirements. Without the penthouse on top of the building, the overall height would be in compliance with IBC standards, however, with the penthouse, a variance or special permit would be required because the overall height will then be categorized as a high-rise building. The height of this building largely lends itself to the influence of fire resistance and vibrational analysis. However, the use of intumescent paint can be evaluated as a design option to mitigate the impact of structural member upsizing due to the required fire rating for the building. Another key takeaway was that the cost of mass timber construction was higher compared to construction using steel and reinforced concrete. Although there was no difference in height for this building between the two options, it was found that the use of metal gage decking would be a lower cost alternative to CLT panels. The fabrication and treatment for mass timber products to be equal in strength and performance to that of structural steel would drive up the cost.

Recommendations

During this project, challenges and questions for future study with CLT emerged. First, there is little data on the costs to construct a building completely made of mass timber structural elements due to its relatively new stance in the industry. With the limited research available into the vibration analysis of CLT panels and mass timber structures, there is plenty of room for future study and analysis. Second, it would be of interest to dive deeper into the study of vibrations with mass timber to get a better understanding of the behaviors that occur. Further tests and research can be conducted to refine the equations used for vibrational analysis to better encapsulate the behaviors of wood as opposed to steel. Altogether, mass timber construction could promote more sustainable engineering and construction practices with additional research and testing.

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Appendix A: Project Proposal

Use of Mass Timber for Multi-Story Laboratory Building

A Major Qualifying Project Proposal

Submitted on:

October 10, 2022

Submitted to:

Project Advisor: Professor Leonard D. Albano, Civil and Environmental Engineering, WPI

Submitted by: Jane T. Richardson

Paul E. Williamson

Desmond C. Woodson

Sponsored by: Michael J. Richard, Ph.D. P.E., Consulting Engineer with Simpson Gumpertz & Heger

Thomas Neal, AIA, Consulting Architect with OTJ

Introduction

Mass timber is a common building material consisting of either wooden lumber, panels, veneers, or fibers bound together in layers. In addition to reliability, timber has proven itself to be a valuable material within construction due to its high-strength-to-weight ratio as well as design flexibility and renewability. Cross-laminated timber (CLT) is an example of mass timber with a semi-rigid composite engineered timber structure that is constructed with three, five, or seven layers of timber boards glued together; each layer is oriented perpendicular to the adjacent. Due to this configuration of the timber members, CLT performs well against shear in-plane and tension perpendicular to the plane, making CLT a popular choice for structural floors and walls. CLT is widely produced and used throughout the European region as it had been developed in Austria back in 1996. CLT manufacturers are much more limited within the United States as the material's implementation into current building codes still continues to grow. Building codes have previously been restrictive on the use of CLT in buildings that are six or more stories. This however is starting to change as more research and studies have been conducted to reassure the structural properties of CLT (Hanes, 2019).

The goal of this project is to design a mixed space 5-story building consisting of biosafety level 2 laboratories and Class-A office spaces using mass timber. This project will investigate the vibrational effects on CLT and evaluate the cost estimate differential in using mass timber over other construction materials.

To achieve our goal, we have identified 3 objectives:

Objective 1: Design a Mixed Laboratory and Class-A Office Mass Timber Building
Objective 2: Conduct a Vibrational Analysis of the Design
Objective 3: Perform a Cost Analysis for the Design Options

The first phase of this project is to design a mass timber mixed space office building based off of floor plans provided by a professional architect. The base structural design will meet all structural requirements specified in the Massachusetts and International Building Codes. Our base structural design will be analyzed for vibration resistance, and compared to the industry standards for allowable vibrations in office and laboratory spaces. If the original structural design does not meet the requirements, revisions will be made until the vibration resistance is satisfactory. Finally, we will run a cost analysis, comparing financial benefits of different construction materials against that of mass timber, and recommending a mass timber supplier.

Background

<u>Mass Timber</u>

Mass timber, otherwise known as engineered wood, is a building material consisting of either wooden lumber, panels, veneers, or fibers bound together in layers (Arquivo.pt, n.d.). Besides reliability, timber has proven itself to be a valuable material within construction due to its high-strength-to-weight ratio as well as design flexibility and renewability. Mass timber provides increased structural stability and uniformity when compared to traditional wood products used in construction (Udele et al., 2022). The off-site fabrication of mass timber members provides additional advantages including the precision of members and increased speed of construction (Smith et al., 2017). The Ascent, a 25-story tall mass timber hybrid building in Milwaukee, "required 90% less construction traffic, 75% fewer workers on-site, and is 25% faster than traditional construction" says the developers (Mass Timber, n.d.). The quicker construction phase is more cost-effective for the owner and contractor, allowing the building to be occupied and produce revenue sooner. (Castle, 2021)

For any project to be categorized as a mass timber project, the primary load-bearing structure must be constructed of either solid or engineered wood; this does not include non-structural timber accents (Mass Timber 101: Understanding the Emerging Building Type, 2017). Optimizing the use of mass timber in construction creates an end product which will be more homogeneous from a structural perspective. These types of projects present both a cost and schedule benefit when compared to those of traditional site-built construction due to the off-site fabrication process previously mentioned. Mass timber construction (MTC) has not had a dominant presence in North America as the result of the lack of both qualitative and quantitative research on its performance. Concerns with this type of construction include the issues of acoustics and vibration, wind, and component flexibility. Research efforts from Smith et al. demonstrate that the benefits of MTC and engineered wood products such as glue-laminated timber and cross-laminated timber can provide an alternative construction resource and application that reduce environmental impacts and construction costs if accepted (Smith et al., 2017).

<u>Glue-Laminated Timber (Glulam)</u>

Glue-laminated timber otherwise known as glulam is another type of structural engineered wood that consists of wood laminations joined together in series by weather-resistant adhesive. Glulam members can be used in various applications of a load-bearing structure such as a beam, girder, column, and more. This type of wood is readily produced in various sizes, curved shapes, and species. The high strength and lightweight attributes allows for distances of up to 100 feet to be spanned without the assistance of intermediate supports, in turn requiring fewer joints (Migliani, 2019). Another beneficial characteristic of glulam is its high degree of fire endurance due to the charring effect that takes place. Based on findings from a study conducted in 1961 by the Southwest Research Institute, glulam was found to perform better than steel in the face of a fire (Douglas, 2000). In addition to this, a 2002 environmental impact case study found that it takes about 2 to 3 times more energy and about 6 to 12 times more fossil fuel for the manufacturing of steel beams in comparison to glulam beams (Petersen & Solberg, 2005). There are four different appearance grades for glulam as defined by the American National Standards Institute (ANSI); framing, industrial, architectural, and premium. Each of these four options are dependent upon where the glulam is being used within the structure, as well as the aesthetic appeal associated with it. Altogether glue-laminated timber can be seen as a cost-effective and resource-efficient material providing a number of advantages over other construction materials like concrete and steel.

Cross-Laminated Timber (CLT)

Cross-laminated timber (CLT) has continually risen in popularity since the early 20th century. The low weight-to-strength ratio of CLT has attracted the attention of engineers and contractors around the world, with CLT being recognized as a code-compliant construction material in the 2015 International Building Code. Cross-laminated timber is a semi-rigid composite engineered timber that is constructed with three, five, or seven layers of timber boards glued together, with each layer oriented perpendicular to the adjacent. The board thickness averages between 5/8 to 2 inches and between 2.4 and 9.5 inches in width; panels are typically 2 to 10 feet wide and span 60 or more feet in length (Think Wood, 2022 & Cross-Laminated Timber (CLT) - APA – the Engineered Wood Association, n.d.). Due to the diagonally crossing configuration of the timber members, CLT performs well against shear in-plane and tension perpendicular to plane, making CLT a popular choice for structural floors and walls (Brandner et al., 2016).

Unlike steel and carbon, timber construction materials take more carbon dioxide out of the environment than what is produced during the manufacturing and installation process. Mass timber buildings are now being referred to as “carbon sinks”, as one ton of timber can store up to 520 lbs of carbon (Churkina et al., 2020). Additionally, as the use of structural timber increases, the demand for concrete and steel will decrease, once again helping reduce global CO_2 emissions. Construction is responsible for approximately 40% of all global carbon emissions, but hybrid and mass timber construction can provide a 15-26% reduction in global warming potential (Moore, 2022).

Another reason why cross-laminated timber use is on the rise is due to the quick and easy on-site installation. CLT panels are prefabricated off-site to any desired specification and size, even with pre-cut windows, doorways, stairs, and ducts. The prefabrication process generates almost no waste on-site, and keeps construction workers on the ground while fabricating.

Vibrations

Vibration within timber floor systems is a common phenomenon typically governed by the mass, stiffness and damping of the timber member. Other influencing factors include the system boundary conditions and excitation factors induced by humans or equipment placed directly on the floor (Huang et al., 2020). The Canadian CLT Handbook defines two types of vibrations that can occur in CLT flooring: transient vibration and resonance. These vibrations are quantified through the fundamental natural frequency recorded for a given floor system and defined as a function of the floor mass and stiffness. In relation to floor stiffness, static deflection is the main dependent factor whereas the velocity and acceleration responses of the floor system are more likely dependent on the mass and excitation of the system in addition to stiffness (Karacabeyli & Gagnon, 2020).

In a CLT floor system, the vibration performance is affected by the spacing of supports and size of screws used on connections as these two factors have a direct influence on the stiffness. Additionally, designing a CLT floor with two-way supports rather than one-way was found to provide a stiffer floor as all beams will be in bending when the first natural frequency occurs. This frequency will only increase marginally as the magnitude of the bending stiffness will be larger in comparison to the magnitude of the torsional stiffness in the beam (Huang et al., 2020). In general, a frequency rating of above 8 Hz is recommended for a good and comfortable performance of CLT (Karacabeyli & Gagnon, 2020).

Studies, such as Huang et al. (2021), investigated the difference between the vibrational performance of solid CLT flooring panels in comparison to hollow-core cross-laminated timber (HC-CLT). A heel drop test was performed to find that a 3-ply CLT floor designed for the experiment had a base natural frequency of approximately 5 Hz. The static bending stiffness was deeply considered in this study and the researchers defined the following equation for calculating the static bending stiffness of a CLT panel

$$(EI)_{CLTpanel} = \sum_{t=1}^{n} E_i I_i + \sum_{t=1}^{n} \gamma_i E_i A_i z_i^2 \quad \text{[Eqn. 1]}$$

where E_i is defined as the modulus of elasticity for layer i, I_i and A_i refer to the moment of inertia and area of layer i, and z_i refers to the distance from the centroid of layer i. γ_i represents the efficiency factor for connections; this value is non-zero for layers in the longitudinal direction except for the middle layer which is equal to unity (Huang et al., 2021). This study confirmed the concept that bending stiffness of a CLT floor system was directly related to the thickness and spacing of the supporting timber members. However, a previous study performed by Huang et al. (2020) found that after a certain point incremental increases to the beam size would not improve floor serviceability any further. The opposite was found in terms of reducing the beam size where this could increase the resonance due to excitation and vibration (Huang et al., 2020).

The Canadian CLT handbook provides a method for the design of vibration-controlled CLT floors and exemplifies the use of fundamental natural frequency and static deflection in calculation. The span length for a vibration-controlled CLT floor was defined as

$$L \leq 0.11 \frac{\left(\frac{(EI)_{eff}}{10^6}\right)^{0.29}}{m^{0.12}} \quad \text{[Eqn. 2]}$$

where L is a function of the effective bending stiffness and mass of the panel. This panel was to meet the simple requirement of being on a load-bearing wall or supported by rigid beams. When comparing the span length and performance of calculated and actual CLT panels, it was found that the spans calculated with Eqn. 2 could be increased by up to 20% in order to account for inherent stiffness features for spans measuring less than 8 meters and floors without topping.

The bending stiffness for a 1-meter-wide CLT panel was calculated in the outlined method using

$$(EI)_{app} = 0.9(EI)_{eff} \text{ [Eqn. 3].}$$

This bending stiffness, $(EI)_{app}$, was used as an approximation of the effective bending stiffness, $(EI)_{eff}$, taken in the major strength direction.

The equation for calculating the fundamental natural frequency of a CLT panel was defined as

$$f = \frac{3.142}{2L^2}\sqrt{\frac{(EI)_{app}}{\rho A}} \text{ [Eqn. 4]}$$

utilizing the vibration-controlled span calculated using Eqn. 2, applied bending stiffness, density, and cross-sectional area of a 1-meter-wide CLT panel.

Static deflection for the 1-meter-wide CLT panel was calculated through the following equation

$$d = \frac{1000pL^3}{48(EI)_{app}} \text{ [Eqn. 5]}$$

which again utilized the vibration-controlled span length, bending stiffness approximation and load p. Load p was defined as a 1000 N or 1 kN load inducing the static deflection along the mid-span of the panel.

The static deflection and fundamental natural frequency were related to each other in the criterion for human acceptability of vibration.

$$\frac{f}{y^{x1}} \geq C \text{ [Eqn. 6]}$$

Above represents the human acceptability criterion of a CLT panel, C, as a function of the fundamental natural frequency, f, divided by the static deflection, y^{x1}. From this relation, the borderline of human acceptability was defined in Eqn. 7 below where the natural frequency divided by the static deflection is equal to the coefficient of human acceptability; this represents the minimum value or ratio of the natural frequency to the static deflection.

$$\frac{f}{y^{x1}} = C \text{ [Eqn. 7]}$$

When the equations above were used to check CLT floors already existing in the field, it was found that the majority of field floors had been designed more conservatively than the vibration-controlled design spans.

In addition to checking the CLT floor panels for vibration, it is important to check that the supporting beams meet the required stiffness criteria. If a supporting beam does not have adequate stiffness, then the flexibility of the beam can cause higher vibrations to occur in the floor panels. To check the supporting beam stiffness, the following equation can be used

$$(EI)_{beam} \geq F_{span} 132.17 l_{beam}^{6.55} \text{ [Eqn. 8]}$$

where EI_{beam} is the supporting beam bending stiffness, l_{beam} is the clean span of the supporting beam, and F_{span} is a constant which is either 1.0 for simple span beams or 0.7 for multi-span beams.

Building Codes

Building codes are set rules and regulations that a built structure must conform to in order to assure the health and safety of the public. These codes can vary from state to state but are ultimately considered to be part of jurisdictional law based on enactment by the government. Unique to Massachusetts is building code 780 CMR 16.00, which details the structural design requirements based on a plethora of factors from loading and building types to where the structure will be built specifically. Section 1605.00 outlines specific load combinations that a building must be able to safely resist and equations for different loading scenarios. Table 1607.01 provides the minimum uniformly distributed live loads and minimum concentrated live loads based on the building type. In regard to laboratory spaces specifically, the structure must be able to withstand at least 100 psf as well as a 2,000 lbs concentrated load. The dead loads are calculated based upon the summation of material and construction weights identified in Table C-1 of ASCE 7. However, if definite information on these loads can not be provided or obtained, then the applicable values will be subject to building official approval. Other necessary loads such as snow loads, seismic loads, and wind speeds were obtained from the 9th edition Massachusetts IBC structural amendment. This amendment provides the most current predetermined design values on the basis of town or city location where construction will take place.

Methodology

Objective 1: Design the Lab		
Tasks	Resources	Actions

Create Conceptual Design	• Mass Building Codes
	• International Building Code (IBC)
		• Determine loading conditions
		  ○ Account for: occupancy, superimposed dead (MEP, flooring, etc.), snow and wind loads
		  ○ Building layout to be based upon architectural plans provided

Create Detailed Design	• CLT Handbook • 2018 NDS • Mass Timber Design Manual	• Inquire with different engineered timber manufacturers to find available member dimensions. • Size structural members based upon our determined loading conditions ○ Including the design of girders, columns, CLT paneling, and connections.
Review Detailed Design	• Bluebeam	• Present our design to sponsors and advisors.
Finalize Design	• CLT Handbook • 2018 NDS • Mass Timber Design Manual	• Consider feedback provided during the "Review Detailed Design" task. ○ Adjust structural design if applicable.
Objective 2: Analyze Vibrations		
Tasks	Resources	Actions

Run Vibration Analysis	• Canadian CLT Vibration Handbook
	• U.S. Mass Timber Floor Vibrations Design Guide
	• AISC Design Guide 11
		• Define areas and load paths for analysis
		  ○ Includes lab and office space and areas with heavy equipment
		  ○ Span across 1 bay
		• Define loading parameters
		  ○ Includes walking rates, number of people at a time, assumed force exerted by each person

Compare Results with Student Spreadsheet	• Student Spreadsheet	• Input same parameters used in our analysis into the student spreadsheet

		• Compare the results for both • Note differences and either adjust our analysis or confer with the master's student
Adjust Design	• Notes from comparison of our analysis to the student spreadsheet	• Make any changes to the design to better account for expected vibration and/or bring into tolerance level
Objective 3: Run Cost Analysis		
Tasks	Resources	Actions
Develop an Accurate Takeoff of all Structural Members	• Student spreadsheet of finalized design	• Account for all CLT flooring and glulam columns and girders
Solicit Pricing from Suppliers	• Nordic • Smartlam • Structurlam	• Create a detailed RFQ • Contact Nordic, Smartlam, & Structurlam
Create a Cost Analysis Report & Provide Justification for Supplier Selection	• Excel cost spreadsheet • Received quotes	• Define cost benefits and any other differences between suppliers • Calculate a unit price of Mass Timber and compare it to that of steel

Schedule

Deadline	Task	Resources/Steps Involved
October 10, 2022	Complete Proposal Submittal	• Research
December 16, 2022	End of B Term Submittal	• Student Excel spreadsheet
March 3, 2022	End of C Term Submittal	• Final MQP report
Objective 1: Design the Lab		
September 23, 2022	Create Conceptual Design	• Determine load-carrying requirements • Review building codes
October 24, 2022	Create Detailed Design	• Calculations based on design manuals
November 2, 2022	Review Detailed Design	• Present design to sponsors and advisors
November 9, 2022	Finalize Design	• Considering feedback provided during the "Review Detailed Design" task
Objective 2: Analyze Vibrations		
November 22, 2022	Run Vibration Analysis	• Canadian CLT Vibration Handbook • U.S. Mass Timber Floor Vibrations Design Guide • AISC Design Guide 11 • Define load paths and parameters
December 5, 2022	Compare Results with Student Spreadsheet	• Utilize spreadsheet from master's student • Note differences in results
December 12, 2022	Adjust Design	• Modify design to bring vibration levels into compliance
Objective 3: Run Cost Analysis		
December 16,	Determine the overall cost of	• Analyze the finalized design in

2022 (Subject to change based on RFQ response time)	mass timber required by the structural design	terms of cost per square foot

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Appendix B: Floor Plans

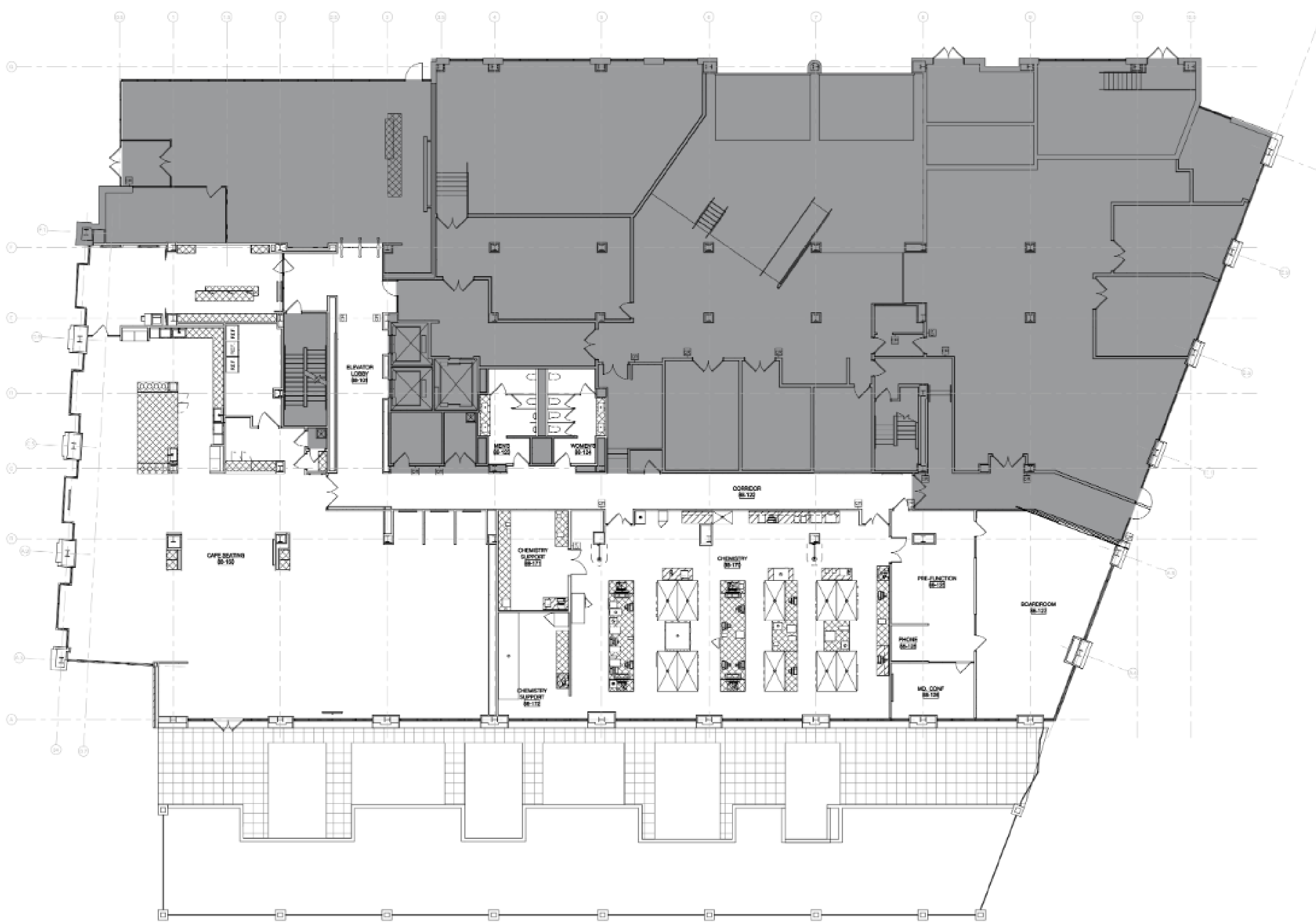


Figure B1. First Floor Plan

Figure B2. Second Floor Plan

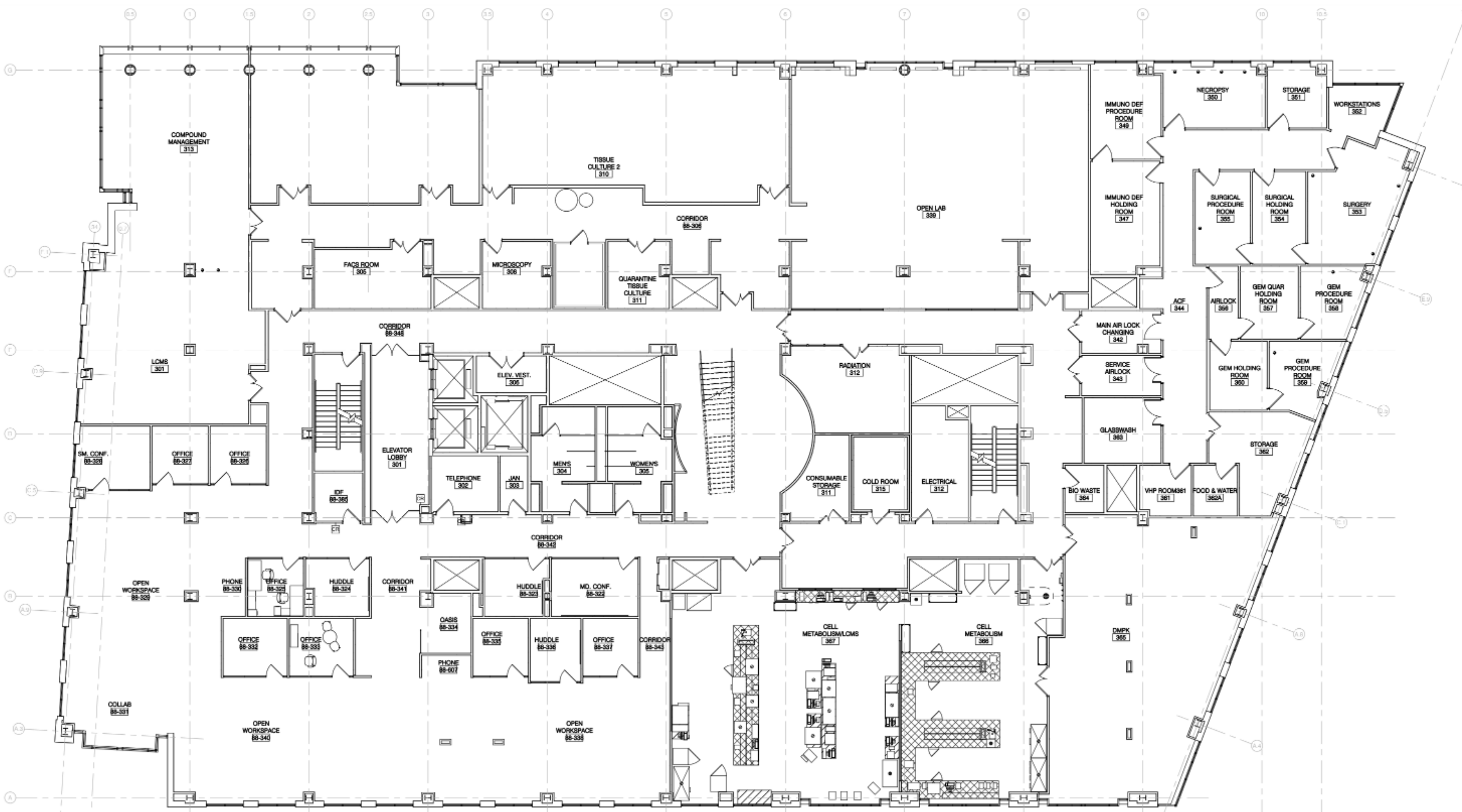


Figure B3. Third Floor Plan

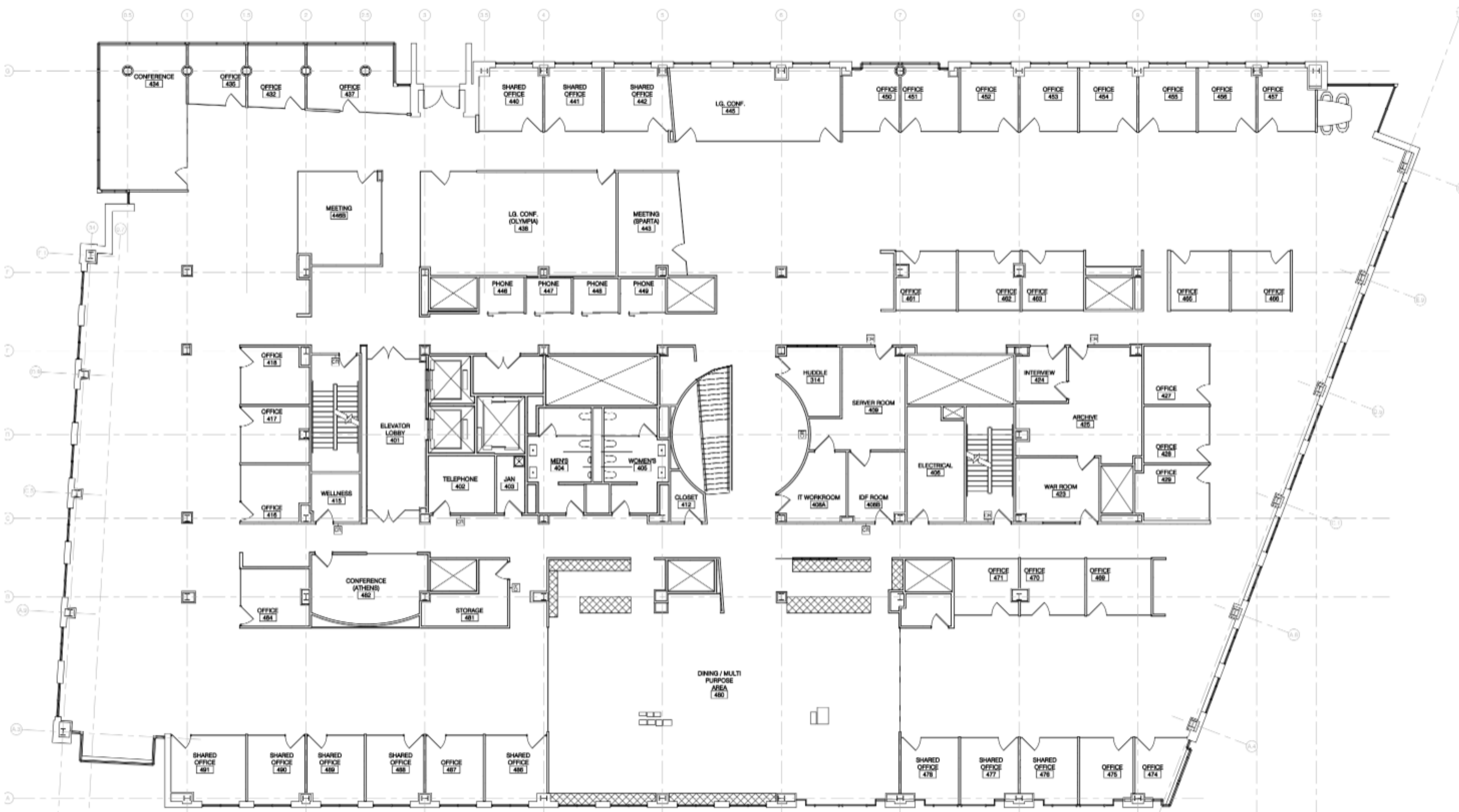


Figure B4. Fourth Floor Plan

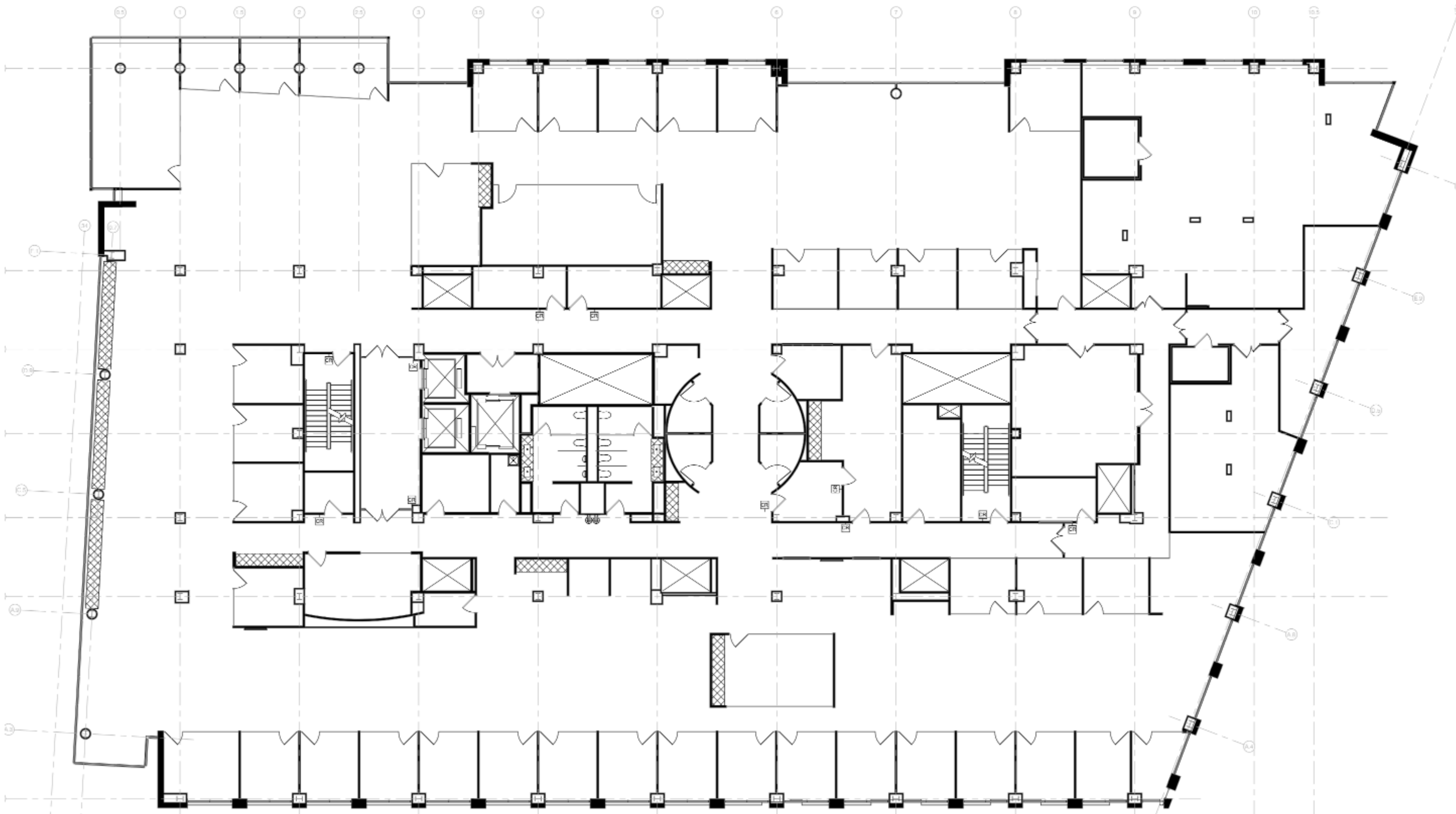


Figure B5. Fifth Floor Plan

Appendix C: Mass Timber Design Calculations

Non-Fire CLT Floor

Loading Calculations without CLT Weight			
	Length, L (ft)	Width, w (ft)	
Panel Size	21.35	8	
Loading			
Variable	Equation	Value	Units
DL		75.72	psf
LL		100	psf
DL + LL	DL + LL	175.72	psf
Effective Width (we)		1	ft
Load (P)	(DL + LL)*We	175.72	plf
Maximum Bending Moment with CLT Self-Weight			
Variable	Equation	Calculated Value	Units
Mmax,initial	(P*(L^2))/8	11060.53	lb-ft
Nordic Panel Section			
Select a	175-5S		
Variable	Value	Units	
CLT Self-Weight (SW)	18.4	psf	
Meff,f,0	10400	lbf-ft/ft	
Vs,0	2480	lbf/ft	
Loading Calculations with CLT Self-Weight			
Variable	Equation	Calculated Value	Units
DL + LL + SW	DL + LL + SW	194.12	psf
New Load (Pn)	(DL + LL + SW)*We	194.12	plf
Shear Calculation Across an 8 ft Section			
Variable	Equation	Calculated Value	Units
V	Vs,0*w	19840	lb-ft

Location: Floor		Reference: Nordic	CLT Type: 175-5S	
Nordic Reference Material				
For CLT Layup E1 (psi)				
Variable	Description	Major Strength Direction	Minor Strength Direction	Units
Fb	Bending at Extreme Fiber	1950	500	psi
E	Modulus of Elasticity	1700000	1200000	psi
Ft	Tension Parallel to Grain	1375	250	psi
Fc	Compression Parallel to Grain	1800	650	psi
Fv	Shear Parallel to Grain Strength	135	135	psi
Fs	Rolling Shear Strength	45	45	psi
Fcp	Compression Perpendicular to Grain	425	425	psi
G	Shear Modulus	106250	75000	psi
Gs	Rolling Shear Modulus	10625	7500	psi
For CLT (175-5S)				
Variable	Description	Major Strength Direction	Minor Strength Direction	Units
FbS(eff)	Bending Moment Capacity	15361.33	3938.80	lbf-ft/ft
V0	Shear Capacity	2480	1490	lbf/ft
EI(eff,0)	Bending Stiffness	440000000	81000000	lbf-in^2/ft
EI(efft,0)	Total Bending Stiffness of Section	9394000000	648000000	lbf-in^2
GA(eff,0)	Shear Rigidity	920000	1200000	lbf/ft
GA(efft,0)	Total Shear Rigidity	19642000	9600000	lbf
Section-Specific Loading and Dimensions				
Floor		All		
Section				
Length (L)		21.35		ft
Width (w)		8		ft
Thickness (t), Longitudinal & Transverse Direction		1.375	1.375	in
Number of Layers (*l*)		5		layers
Total Thickness (tv)		6.875		in
Roof Live Load (Lr)		0		psf
Ground Snow Load (S)		0		psf
Total Dead Load (DL)		94.12		psf
	Carpet/Vinyl Flooring	2.3		psf
	Concrete (5" Thick)	60.42		psf
	MEP	10		psf
	Acoustical Fiber Board	1		psf
	Suspended Steel Channel System	2		psf
	CLT Self-Weight	18.4		psf
Live Load (LL)		100		psf
Wind (W)		0		mph
Earthquake (E)		0		psf
	Ss	0.217		
	S1	0.069		

Actual Loading			
Variable	Description	Calculation	Units
LC1	DL	94.12	psf
LC2	DL + LL	194.12	psf
LC3	DL + (Lr or S or R)	94.12	psf
LC4	DL + 0.75LL + 0.75(Lr or S or R)	169.12	psf
LC5	DL + (0.6W or 0.7E)	94.12	psf
LC6a	DL+ 0.75LL + 0.75(0.6W) + 0.75(Lr or S or R)	169.12	psf
LC6b	DL + 0.75L L+ 0.75(0.7E) + 0.75S	169.12	psf
LC7	0.6DL + 0.6W	56.47	psf
LC8	0.6DL + 0.7E	56.47	psf
NDS Factors			
Variable	Description	Factor	
CD	Load Duration Factor	0.9	
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
Allowable Values			
Variable	Equation	Calculated Value	Units
Fb(Seff)'	Fb(Seff)*CD*CM*Ct*CL	13825.20	lb-ft
Fs(Ib/Q)eff	Fs(Ib/Q)eff*CM*Ct, For Longitudinal Direction	15468.75	lb
Fs(Ib/Q)eff	Fs(Ib/Q)eff*CM*Ct, For Transverse Direction	15468.75	lb
(EI)app'	(EI)app*CM*Ct	405981823.6	lb-ft^2
Allowable Deflection			
Deflection	Equation	Calculated Value	Units
Delta LL	L/360	0.71	in
Delta DL+LL	L/240	1.0675	in

(EI)eff				
I		(b*tv^3)/12	324.95	in^4
(EI)eff			552416992.2	lb-in^2
(Ib/Q)eff				
Layer	E	z (in)	Ehz	Units
1	1700000	3.4375	8035156.25	
2	1200000	2.0625	3403125	
3	1700000	0.6875	1607031.25	
		Sum of Ehz	13045312.5	lb
	Longitudinal	(Ib/Q)eff	343.75	in^2
	Transverse	(Ib/Q)eff	343.75	in^2
EIapp				
Ks	Uniformly Distributed w/ Pinned Connections		11.5	
EIapp	(EIeff)/(1+((Ks*EIeff)/(Gaeff(L^2))))		405981823.6	lb-in^2/ft
(S)eff				
Layer	Equation		Calculated Val	Units
	(b*tv^2)/6		94.53	in^3

Actual Moment and Shear				
Variable	Equation	Calculated Value	Units	Demand Capacity Ratio
Mmax	(Pn*(L^2))/8	11060.53	lb-ft	80.00%
Vmax	5*(Pn*L)/8	2590.29	lb	16.75%

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity Ratio
Delta LL	(LL*(L/2)^4)/(185*EI)	0.031	in	4.37%
Delta DL+LL	((LL+DL+SW)*((L/2)^4))/(185*EI)	0.060	in	5.65%

Fire CLT Floor

Loading Calculations without CLT Weight			
	Length, L (ft)	Width, w (ft)	
Panel Size	21.35	8	
Loading			
Variable	Equation	Value	Units
DL		75.72	psf
LL		100	psf
DL + LL	DL + LL	175.72	psf
Effective Width (we)		1	ft
Load (P)	(DL + LL)*We	175.72	plf
Maximum Bending Moment without CLT Self-Weight			
Variable	Equation	Calculated Value	Units
Mmax,initial	(P*(L^2))/8	10012.14	lb-ft
Nordic Panel Section			
Select a	143-5S		
Variable	Value	Units	
CLT Self-Weight (SW)	15.1	psf	
Meff,f,0	7725	lbf-ft/ft	
Vs,0	2030	lbf/ft	
Loading Calculations with CLT Self-Weight			
Variable	Equation	Calculated Value	Units
DL + LL + SW	DL + LL + SW	183.17	psf
New Load (Pn)	(DL + LL + SW)*We	183.17	plf
Shear Calculation Across an 8 ft Section			
Variable	Equation	Calculated Value	Units
V	Vs,0*w	16240	lb-ft

Location: Floor	Reference: Nordic	CLT Type: 143-5S

Nordic Reference Material				
For CLT Layup E1 (psi)				
Variable	Description	Major Strength Direction	Minor Strength Direction	Units
Fb	Bending at Extreme Fiber	1950	500	psi
E	Modulus of Elasticity	1700000	1200000	psi
Ft	Tension Parallel to Grain	1375	250	psi
Fc	Compression Parallel to Grain	1800	650	psi
Fv	Shear Parallel to Grain Strength	135	135	psi
Fs	Rolling Shear Strength	45	45	psi
Fcp	Compression Perpendicular to Grain	425	425	psi
G	Shear Modulus	106250	75000	psi
Gs	Rolling Shear Modulus	10625	7500	psi
For CLT (143-5S)				
Variable	Description	Major Strength Direction	Minor Strength Direction	Units
FbS(eff)	Bending Moment Capacity	4754.95	1219.22	lbf-ft/ft
V0	Shear Capacity	2030	1040	lbf/ft
EI(eff,0)	Bending Stiffness	95135639.06	67154568.75	lbf-in^2/ft
EI(efft,0)	Total Bending Stiffness of Section	2031145894	537236550	lbf-in^2
GA(eff,0)	Shear Rigidity	406406.25	341250	lbf/ft
GA(efft,0)	Total Shear Rigidity	8676773.438	2730000	lbf

Section-Specific Loading and Dimensions				
Floor		All		
Section				
Length (L)		21.35		ft
Width (w)		8		ft
Thickness (t), Longitudinal & Transverse Direction		1.375	0.75	in
Number of Layers (l)		5		layers
Total Thickness (tv)		3.825		in
Roof Live Load (Lr)		0		psf
Ground Snow Load (S)		0		psf
Total Dead Load (DL)		83.17		psf
	Carpet/Vinyl Flooring	2.3		psf
	Concrete (5" Thick)	60.42		psf
	MEP	10		psf
	Acoustical Fiber Board	1		psf
	Suspended Steel Channel System	2		psf
	CLT Self-Weight	7.45		psf
Live Load (LL)		100		psf
Wind (W)		0		mph
Earthquake (E)		0		psf
	Ss	0.217		
	S1	0.069		

Actual Loading			
Variable	Description	Calculation	Units
LC1	DL	83.17	psf
LC2	DL + LL	183.17	psf
LC3	DL + (Lr or S or R)	83.17	psf
LC4	DL + 0.75LL + 0.75(Lr or S or R)	158.17	psf
LC5	DL + (0.6W or 0.7E)	83.17	psf
LC6a	DL+ 0.75LL + 0.75(0.6W) + 0.75(Lr or S or R)	158.17	psf
LC6b	DL + 0.75L L+ 0.75(0.7E) + 0.75S	158.17	psf
LC7	0.6DL + 0.6W	49.90	psf
LC8	0.6DL + 0.7E	49.90	psf
NDS Factors			
Variable	Description	Factor	
CD	Load Duration Factor	0.9	
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
Allowable Values			
Variable	Equation	Calculated Value	Units
Fb(Seff)'	Fb(Seff)*2.85*CF*CV*Cfu	13551.62	lb-ft
Fs(Ib/Q)eff	Fs(Ib/Q)eff*CM*Ct, For Longitudinal Direction	2663.98	lb
Fs(Ib/Q)eff	Fs(Ib/Q)eff*CM*Ct, For Transverse Direction	8953.9425	lb
(EI)app'	(EI)app*CM*Ct	91387553.45	lb-ft^2
Allowable Deflection			
Deflection	Equation	Calculated Value	Units
Delta LL	L/360	0.71	in
Delta DL+LL	L/240	1.0675	in

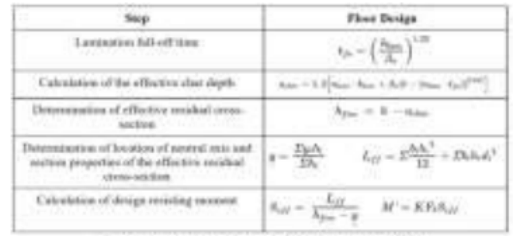


Figure 5a: Char Equations for Floor Design (NDS, 2018)

(EI)eff				
I		(b*tv^3)/12	55.96	in^4
(EI)eff			9.5E+07	lb-in^2
(Ib/Q)eff				
Layer	E	z (in)	Ehz	Units
1	1700000	1.9125	4470469	
2	1200000	0.5375	886875	
		Sum of Ehz	5357344	lb
	Longitudinal	(Ib/Q)eff	59.20	in^2
	Transverse	(Ib/Q)eff	198.98	in^2
EIapp				
Ks	Uniformly Distributed w/ Pinned Connections		11.5	
EIapp	(EIeff)/(1+((Ks*EIeff)/(Gaeff(L^2))))		9.1E+07	lb-ft^2
(S)eff				
Layer	Equation		Calculated	Units
	(b*tv^2)/6		29.26	in^3

Actual Moment and Shear				
Variable	Equation	Calculated Value	Units	Demand Capacity Ratio
Mmax	(Pn*(L^2))/8	10436.77	lb-ft	77.01%
Vmax	(Pn*L)/2	1955.37	lb	73.40%

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity Ratio
Delta LL	(LL*(L/2)^3)/(185*EI)	0.144	in	20.20%
Delta DL+LL	((LL+DL+SW)*((L/2)^3))/(185*EI)	0.263	in	24.67%

Non-Fire CLT Roof

Loading Calculations without CLT Weight			
	Length, L (ft)	Width, w (ft)	
Panel Size	21.35	8	
Loading			
Variable	Equation	Value	Units
DL		20	psf
Lr		150	psf
DL+Lr	DL+Lr	170.00	psf
Effective Width (we)		1	ft
Load (P)	(DL + Lr)*We	170	plf
Maximum Bending Moment with CLT Self-Weight			
Variable	Equation	Calculated Value	Units
Mmax,initial	(P*(L^2))/8	10734.62	lb-ft
Nordic Panel Section			
Select a	175-5S		
Variable	Value	Units	
CLT Self-Weight (SW)	18.4	psf	
Meff,f,0	10400	lbf-ft/ft	
Vs,0	2480	lbf/ft	
Loading Calculations with CLT Self-Weight			
Variable	Equation	Calculated Value	Units
DL + Lr + SW	DL + LL + SW	188.40	psf
New Load (Pn)	(DL + LL + SW)*We	188.4	plf
Shear Calculation Across an 8 ft Section			
Variable	Equation	Calculated Value	Units
V	Vs,0*w	19840	lb-ft

Location: Roof		Reference: Nordic		CLT Type: 175-5S

Nordic Reference Material				
For CLT Layup E1 (psi)				
Variable	Description	Major Strength Direction	Minor Strength Direction	Units
Fb	Bending at Extreme Fiber	1950	500	psi
E	Modulus of Elasticity	1700000	1200000	psi
Ft	Tension Parallel to Grain	1375	250	psi
Fc	Compression Parallel to Grain	1800	650	psi
Fv	Shear Parallel to Grain Strength	135	135	psi
Fs	Rolling Shear Strength	45	45	psi
Fcp	Compression Perpendicular to Grain	425	425	psi
G	Shear Modulus	106250	75000	psi
Gs	Rolling Shear Modulus	10625	7500	psi
For CLT (175-5S)				
Variable	Description	Major Strength Direction	Minor Strength Direction	Units
FbS(eff)	Bending Moment Capacity	15361.33	3938.80	lbf-ft/ft
V0	Shear Capacity	2480	1490	lbf/ft
EI(eff,0)	Bending Stiffness	440000000	81000000	lbf-in^2/ft
EI(efft,0)	Total Bending Stiffness of Section	9394000000	648000000	lbf-in^2
GA(eff,0)	Shear Rigidity	920000	1200000	lbf/ft
GA(efft,0)	Total Shear Rigidity	19642000	9600000	lbf
Section-Specific Loading and Dimensions				
Floor		All		
Section				
Length (L)		21.35		ft
Width (w)		8		ft
Thickness (t), Longitudinal & Transverse Direction		1.375	1.375	in
Number of Layers (l)		5		layers
Total Thickness (tv)		6.875		in
Roof Live Load (Lr)		150		psf
Ground Snow Load (S)		40		psf
Snow Drift (SD)		83.34		psf
Total Dead Load (DL)		38.4		psf
	MEP	20		psf
	CLT Self-Weight	18.4		psf
Live Load (LL)		0		psf
Wind (W)		0		mph
Earthquake (E)		0		psf
	Ss	0.217		
	S1	0.069		

Actual Loading			
Variable	Description	Calculation	Units
LC1	DL	38.4	psf
LC2	DL + LL	38.4	psf
LC3	DL + (Lr or S or R)	188.40	psf
LC4	DL + 0.75LL + 0.75(Lr or S or R)	150.9	psf
LC5	DL + (0.6W or 0.7E)	38.4	psf
LC6a	DL+ 0.75LL + 0.75(0.6W) + 0.75(Lr or S or R)	150.9	psf
LC6b	DL + 0.75L L+ 0.75(0.7E) + 0.75S	100.91	psf
LC7	0.6DL + 0.6W	23.04	psf
LC8	0.6DL + 0.7E	23.04	psf
NDS Factors			
Variable	Description	Factor	
CD	Load Duration Factor	0.9	
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
Allowable Values			
Variable	Equation	Calculated Value	Units
Fb(Seff)'	Fb(Seff)*CD*CM*Ct*CL	13825.20	lb-ft
Fs(Ib/Q)eff	Fs(Ib/Q)eff*CM*Ct, For Longitudinal Direction	15468.75	lb
Fs(Ib/Q)eff	Fs(Ib/Q)eff*CM*Ct, For Transverse Direction	15468.75	lb
(EI)app'	(EI)app*CM*Ct	405981823.6	lb-ft^2
Allowable Deflection			
Deflection	Equation	Calculated Value	Units
Delta LL	L/360	0.71	in
Delta DL+LL	L/240	1.0675	in

Snow Drift			
Variable	Description	Value	Units
Gamma (y)	Snow Density	20	pcf
Ps	Design Snow Load per ASCE 7.7-1	30	psf
S	Ground Snow Load	40	psf
Ll	Length of Lower Roof	235	ft
Lu	Length of Upper Roof	40	ft
hr	Roof Height	18	ft
Snow Drift Heights			
Variable	Equation	Calculated Value	Units
hb	y/Ps	1.5	ft
hc	hr - hb	16.5	ft
hc/hb	hc/hb	11	ft
Windward Direction			
Variable	Equation	Calculated Value	Units
hd	(3/4)*(0.43*(Ll^(1/3))*((Pg+10)^(1/4))-1.5)	4.17	ft
Leeward Direction			
Variable	Equation	Calculated Value	Units
hd	0.43(Lu^(1/3))*((Pg+10)^(1/4))-1.5	2.41	ft
Maximum Drift			
hd		4.17	ft
Width of Snow Drift Surcharge			
Variable	Equation	Calculated Value	Units
w	4hd	16.67	ft
Snow Drift Surcharge Load			
Variable	Equation	Calculated Value	Units
Sd	hd*y	83.34	psf

if < 0.2, then no drift calculation necessary

(EI)eff				
I		(b*tv^3)/12	324.95	in^4
(EI)eff			552416992.2	lb-in^2
(Ib/Q)eff				
Layer	E	z (in)	Ehz	Units
1	1375	3.4375	6499.023438	
2	250	2.0625	708.984375	
3	1375	0.6875	1299.804688	
		Sum of Ehz	8507.8125	lb
	Longitudinal	(Ib/Q)eff	343.75	in^2
	Transverse	(Ib/Q)eff	343.75	in^2
EIapp				
Ks	Uniformly Distributed w/ Pinned Connections		11.5	
EIapp	(EIeff)/(1+((Ks*EIeff)/(Gaeff(L^2))))		405981823.6	lb-ft^2
(S)eff				
Layer	Equation		Calculated Value	Units
	(b*tv^2)/6		94.53	in^3

Actual Moment and Shear				
Variable	Equation	Calculated Value	Units	Demand Capacity Ratio
Mmax	(Pn*(L^2))/8	10734.62	lb-ft	77.65%
Vmax	(Pn*L)/2	2011.17	lb	13.00%

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity Ratio
Delta LL	(LL*(L/2)^3)/(185*EI)	0.047	in	6.55%
Delta DL+LL	((LL+DL+SW)*((L/2)^3))/(185*EI)	0.084	in	7.91%

Fire CLT Roof

Loading Calculations without CLT Weight			
	Length, L (ft)	Width, w (ft)	
Panel Size	21.35	8	
Loading			
Variable	Equation	Value	Units
DL		20	psf
Lr		150	psf
DL+Lr	DL+Lr	170.00	psf
Effective Width (we)		1	ft
Load (P)	(DL + Lr)*We	170	plf
Maximum Bending Moment with CLT Self-Weight			
Variable	Equation	Calculated Value	Units
Mmax,initial	(P*(L^2))/8	10546.59	lb-ft
Nordic Panel Section			
Select a	143-5S		
Variable	Value	Units	
CLT Self-Weight (SW)	15.1	psf	
Meff,f,0	7725	lbf-ft/ft	
Vs,0	2030	lbf/ft	
Loading Calculations with CLT Self-Weight			
Variable	Equation	Calculated Value	Units
DL + Lr + SW	DL + LL + SW	185.10	psf
New Load (Pn)	(DL + LL + SW)*We	185.1	plf
Shear Calculation Across an 8 ft Section			
Variable	Equation	Calculated Value	Units
V	Vs,0*w	16240	lb-ft

Location: Roof	Reference: Nordic	CLT Type:	143-5S

Nordic Reference Material				
For CLT Layup E1 (psi)				
Variable	Description	Major Strength Direction	Minor Strength Direction	Units
Fb	Bending at Extreme Fiber	1950	500	psi
E	Modulus of Elasticity	1700000	1200000	psi
Ft	Tension Parallel to Grain	1375	250	psi
Fc	Compression Parallel to Grain	1800	650	psi
Fv	Shear Parallel to Grain Strength	135	135	psi
Fs	Rolling Shear Strength	45	45	psi
Fcp	Compression Perpendicular to Grain	425	425	psi
G	Shear Modulus	106250	75000	psi
Gs	Rolling Shear Modulus	10625	7500	psi
For CLT (143-5S)				
Variable	Description	Major Strength Direction	Minor Strength Direction	Units
FbS(eff)	Bending Moment Capacity	4754.95	1219.22	lbf-ft/ft
V0	Shear Capacity	2030	1040	lbf/ft
EI(eff,0)	Bending Stiffness	267000000	26000000	lbf-in^2/ft
EI(efft,0)	Total Bending Stiffness of Section	5700450000	208000000	lbf-in^2
GA(eff,0)	Shear Rigidity	406406.25	341250	lbf/ft
GA(efft,0)	Total Shear Rigidity	8676773.438	2730000	lbf
Section-Specific Loading and Dimensions				
Floor		All		
Section				
Length (L)		21.35		ft
Width (w)		8		ft
Thickness (t), Longitudinal & Transverse Direction		1.375	0.75	in
Number of Layers (*l*)		5		layers
Total Thickness (tv)		3.825		in
Roof Live Load (Lr)		150		psf
Ground Snow Load (S)		40		psf
Snow Drift (SD)		83.34		psf
Total Dead Load (DL)		35.1		psf
	MEP	20		psf
	CLT Self-Weight	15.1		psf
Live Load (LL)		0		psf
Wind (W)		0		mph
Earthquake (E)		0		psf
	Ss	0.217		
	S1	0.069		

Actual Loading			
Variable	Description	Calculation	Units
LC1	DL	35.1	psf
LC2	DL + LL	35.1	psf
LC3	DL + (Lr or S or R)	185.10	psf
LC4	DL + 0.75LL + 0.75(Lr or S or R)	147.6	psf
LC5	DL + (0.6W or 0.7E)	35.1	psf
LC6a	DL+ 0.75LL + 0.75(0.6W) + 0.75(Lr or S or R)	147.6	psf
LC6b	DL + 0.75L L+ 0.75(0.7E) + 0.75S	97.61	psf
LC7	0.6DL + 0.6W	21.06	psf
LC8	0.6DL + 0.7E	21.06	psf
NDS Factors			
Variable	Description	Factor	
CD	Load Duration Factor	0.9	
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
Allowable Values			
Variable	Equation	Calculated Value	Units
Fb(Seff)'	Fb(Seff)*2.85*CF*CV*Cfu	13551.62	lb-ft
Fs(Ib/Q)eff	Fs(Ib/Q)eff*CM*Ct, For Longitudinal Direction	2663.98	lb
Fs(Ib/Q)eff	Fs(Ib/Q)eff*CM*Ct, For Transverse Direction	8953.9425	lb
(EI)app'	(EI)app*CM*Ct	239439546.8	lb-ft^2
Allowable Deflection			
Deflection	Equation	Calculated Value	Units
Delta LL	L/360	0.71	in
Delta DL+LL	L/240	1.0675	in

Snow Drift			
Variable	Description	Value	Units
Gamma (y)	Snow Density	20	pcf
Ps	Design Snow Load per ASCE 7.7-1	30	psf
S	Ground Snow Load	40	psf
Ll	Length of Lower Roof	235	ft
Lu	Length of Upper Roof	40	ft
hr	Roof Height	18	ft
Snow Drift Heights			
Variable	Equation	Calculated Value	Units
hb	y/Ps	1.5	ft
hc	hr - hb	16.5	ft
hc/hb	hc/hb	11	ft
Windward Direction			
Variable	Equation	Calculated Value	Units
hd	(3/4)*(0.43*(*Ll* ^(1/3))*((Pg+10)^(1/4))-1.5)	4.17	ft
Leeward Direction			
Variable	Equation	Calculated Value	Units
hd	0.43(*Lu* ^(1/3))*((Pg+10)^(1/4))-1.5	2.41	ft
Maximum Drift			
hd		4.17	ft
Width of Snow Drift Surcharge			
Variable	Equation	Calculated Value	Units
w	4hd	16.67	ft
Snow Drift Surcharge Load			
Variable	Equation	Calculated Value	Units
Sd	hd*y	83.34	psf

if ≤ 0.2, then no drift calculation necessary

(EI)eff				
I		(b*tv^3)/12	55.96	in^4
(EI)eff			95135639	lb-in^2
(Ib/Q)eff				
Layer	E	z (in)	Ehz	Units
1	1700000	1.9125	4470469	
2	1200000	0.5375	483750	
		Sum of Ehz	4954219	lb
	Longitudinal	(Ib/Q)eff	59.20	in^2
	Transverse	(Ib/Q)eff	198.98	in^2
EIapp				
Ks	Uniformly Distributed w/ Pinned Connections		11.5	
EIapp	(Eleff)/(1+((Ks*Eleff)/(Gaeff(L^2))))		2.39E+08	lb-ft^2
(S)eff				
Layer	Equation		Calculated	Units
	(b*tv^2)/6		29.26	in^3

Actual Moment and Shear				
Variable	Equation	Calculated Value	Units	Demand Capacity Ratio
Mmax	(Pn*(L^2))/8	10546.59	lb-ft	77.83%
Vmax	(Pn*L)/2	1975.94	lb	74.17%

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity Ratio
Delta LL	(LL*(L/2)^3)/(185*EI)	0.077	in	10.80%
Delta DL+LL	((LL+DL+SW)*((L/2)^3))/(185*EI)	0.137	in	12.88%

CLT Penthouse

Loading Calculations without CLT Weight			
	Length, L (ft)	Width, w (ft)	
Panel Size	30	8	
Loading			
Variable	Equation	Value	Units
DL		20	psf
S		30	psf
DL+S	DL+S	50.00	psf
Effective Width (we)		1	ft
Load (P)	(DL + S)*We	50	plf
Maximum Bending Moment with CLT Self-Weight			
Variable	Equation	Calculated Value	Units
Mmax,initial	(P*(L^2))/8	7323.75	lb-ft
Nordic Panel Section			
Select a	143-5S		
Variable	Value	Units	
CLT Self-Weight (SW)	15.1	psf	
Meff,f,0	7725	lbf-ft/ft	
Vs,0	2030	lbf/ft	
Loading Calculations with CLT Self-Weight			
Variable	Equation	Calculated Value	Units
DL + LL + SW	DL + LL + SW	65.10	psf
New Load (Pn)	(DL + LL + SW)*We	65.10	plf
Shear Calculation Across an 8 ft Section			
Variable	Equation	Calculated Value	Units
V	Vs,0*w	16240	lb-ft

Location:	Penthouse	Reference:	Nordic	CLT Type:	143-5S

Nordic Reference Material				
For CLT Layup E1 (psi)				
Variable	Description	Major Strength Direction	Minor Strength Direction	Units
Fb	Bending at Extreme Fiber	1950	500	psi
E	Modulus of Elasticity	1700000	1200000	psi
Ft	Tension Parallel to Grain	1375	250	psi
Fc	Compression Parallel to Grain	1800	650	psi
Fv	Shear Parallel to Grain Strength	135	135	psi
Fs	Rolling Shear Strength	45	45	psi
Fcp	Compression Perpendicular to Grain	425	425	psi
G	Shear Modulus	106250	75000	psi
Gs	Rolling Shear Modulus	10625	7500	psi
For CLT (143-5S)				
Variable	Description	Major Strength Direction	Minor Strength Direction	Units
FbS(eff)	Bending Moment Capacity	10283.20313	2636.71875	lbf-ft/ft
V0	Shear Capacity	2030	1040	lbf/ft
EI(eff,0)	Bending Stiffness	267000000	26000000	lbf-in^2/ft
EI(efft,0)	Total Bending Stiffness of Section	8010000000	208000000	lbf-in^2
GA(eff,0)	Shear Rigidity	960000	780000	lbf/ft
GA(efft,0)	Total Shear Rigidity	28800000	6240000	lbf
Section-Specific Loading and Dimensions				
Floor		All		
Section				
Length (L)		30		ft
Width (w)		8		ft
Thickness (t), Longitudinal & Transverse Direction		1.375	0.75	in
Number of Layers (*l*)		5		layers
Total Thickness (tv)		5.625		in
Roof Live Load (Lr)		20		psf
Snow Load (S)		30		psf
Total Dead Load (DL)		35.1		psf
	MEP	20		psf
	CLT Self-Weight	15.1		psf
Live Load (LL)		0		psf
Wind (W)		0		mph
Earthquake (E)		0		psf
	Ss	0.217		
	S1	0.069		

Actual Loading			
Variable	Description	Calculation	Units
LC1	DL	35.1	psf
LC2	DL + LL	35.1	psf
LC3	DL + (Lr or S or R)	65.1	psf
LC4	DL + 0.75LL + 0.75(Lr or S or R)	57.6	psf
LC5	DL + (0.6W or 0.7E)	35.1	psf
LC6a	DL+ 0.75LL + 0.75(0.6W) + 0.75(Lr or S or R)	57.6	psf
LC6b	DL + 0.75L L+ 0.75(0.7E) + 0.75S	57.6	psf
LC7	0.6DL + 0.6W	21.06	psf
LC8	0.6DL + 0.7E	21.06	psf
NDS Factors			
Variable	Description	Factor	
CD	Load Duration Factor	0.9	
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
Allowable Values			
Variable	Equation	Calculated Value	Units
Fb(Seff)'	Fb(Seff)*CD*CM*Ct*CL	9254.882813	lb-ft
Fs(Ib/Q)eff	Fs(Ib/Q)eff*CM*Ct, For Longitudinal Direction	8472.365702	lb
Fs(Ib/Q)eff	Fs(Ib/Q)eff*CM*Ct, For Transverse Direction	28476.5625	lb
(EI)app'	(EI)app*CM*Ct	260569330.9	lb-ft^2
Allowable Deflection			
Deflection	Equation	Calculated Value	Units
Delta LL	L/360	1	in
Delta DL+LL	L/240	1.5	in

(EI)eff				
I		(b*tv^3)/12	177.98	in^4
(EI)eff			302563476.6	lb-in^2
(Ib/Q)eff				
Layer	E	z (in)	Ehz	Units
1	1700000	2.8125	6574218.75	
2	1200000	1.4375	1293750	
3	1700000	0.6875	876562.5	
		Sum of Ehz	8744531.25	lb
	Longitudinal	(Ib/Q)eff	188.2747934	in^2
	Transverse	(Ib/Q)eff	632.8125	in^2
Elapp				
Ks	Uniformly Distributed w/ Pinned Connections		11.5	
Elapp	(Eleff)/(1+((Ks*Eleff)/(Gaeff(L^2))))		260569330.9	lb-ft^2
(S)eff				
Layer	Equation		Calculated Val	Units
	(b*tv^2)/6		63.28	in^3

Actual Moment and Shear				
Variable	Equation	Calculated Value	Units	Demand Capacity Ratio
Mmax	(Pn*(L^2))/8	7323.75	lb-ft	79.13%
Vmax	(Pn*L)/2	976.50	lb	11.53%

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity Ratio
Delta LL	(LL*(L/2)^3)/(185*EI)	0.010	in	1.02%
Delta DL+LL	((LL+DL+SW)*((L/2)^3))/(185*EI)	0.033	in	2.22%

Non-Fire Glulam Floor

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	9.5	in
Depth, d	Depth	23.5	in
Triburaty Width, *w*	Tributary Width	13.35	ft
Ix	Moment of Inertia	10274.15	in^4
Sx	Section Modulus	874.40	in^3
A	Area	223	in^2
rx	Radius of Gyration	6.78	in
L	Length	18.01	ft

Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fbx'	Bending Moment	2400	psi
Fclx	Compression Perpendicular to Grain	600	psi
Fvx	Shear Parallel to Grain	300	psi
Ex	Apparent Modulus of Elasticity	1800000	psi

Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))	0.893	
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	

Loading		
Variable	Value	Units
LL	100	psf
DL	94.12	psf
SW	52.71	plf

Loading Conditions			
Variable	Equation	Calculated Value	Units
Factored Load	1.2DL + 1.6LL	272.94	psf
Factored Load*Tributary Area	(1.2DL + 1.6LL)**w*	3643.80	plf
Total Load, w	((1.2DL+1.6LL)**w*)+SW	3696.51	plf

Allowable Loading			
Variable	Equation	Calculated Value	Units
F'b	Fb*CM*Ct*CL*CV*Cfu*Cc*CI*KF*Phi*Lambda	3700.26	psi
F'v	Fv*CM*Ct*Cvr	300.00	psi
F'clx	Fclx*CM*Ct*Cb	600.00	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Moment, M	(wL^2)/8	149875.21	lb*ft	
Max Bending Moment	M/Sx	2056.85	psi	55.59%
Shear, Vmax	(wL/2)	32812.44	lbs	
Fv	3V/2bd	220.46	psi	73.49%
Fc(perpendicular)	V/bd	146.98	psi	24.50%

Allowable Deflection			
Variable	Equation	Calculated Value	Units
Delta LL	L/360	0.60	in

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity
Delta	(5wL^4)/(384EI)	0.473	in	79%

Fire Glulam Floor

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	5.9	in
Depth, d	Depth	23.7	in
Triburaty Width, *w*	Tributary Width	13.35	ft
Ix	Moment of Inertia	6545.09	in^4
Sx	Section Modulus	552.33	in^3
A	Area	140	in^2
rx	Radius of Gyration	6.84	in
L	Length	18.01	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fbx'	Bending Moment	2400	psi
Fclx	Compression Perpendicular to Grain	600	psi
Fvx	Shear Parallel to Grain	300	psi
Ex	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CD	Load Duration Factor	1	
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))	0.935	
CF	Size Factor	1	
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	

Loading			
Variable		Value	Units
LL		100	psf
DL		94.12	psf
SW		33.02	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Factored Load	DL + LL	194.12	psf
Factored Load*Tributary Area	(DL + LL)**w*	2591.50	plf
Total Load, w	((DL+LL)**w*)+SW	2624.52	plf

Allowable Loading			
Variable	Equation	Calculated Value	Units
F'b	Fb*2.85*CF*CV*Cfu*CL	6398.15	psi
F'v	Fv*CD*CM*Ct*Cvr	300.00	psi
F'clx	Fclx*CM*Ct*Cb	600.00	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Moment, M	(wL^2)/8	106411.09	lb*ft	
Max Bending Moment	M/Sx	2311.91	psi	36.13%
Shear, Vmax	(5wL/8)	23336.48	lbs	
Fv	3V/2bd	250.34	psi	83.45%
Fc(perpendicular)	V/bd	166.89	psi	27.82%

Allowable Deflection			
Variable	Equation	Calculated Value	Units
Delta LL	L/360	0.60	in

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity
Delta	(5wL^4)/(384EI)	0.527	in	87.84%

Non-Fire Glulam Roof

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	9.5	in
Depth, d	Depth	23.5	in
Triburaty Width, *w*	Tributary Width	13.35	ft
Ix	Moment of Inertia	10274.15	in^4
Sx	Section Modulus	874.40	in^3
A	Area	223	in^2
rx	Radius of Gyration	6.78	in
L	Length	18.01	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fbx'	Bending Moment	2400	psi
Fclx	Compression Perpendicular to Grain	600	psi
Fvx	Shear Parallel to Grain	300	psi
Ex	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))	0.893	
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	

Loading			
Variable		Value	Units
LL		150	psf
DL		38.4	psf
SD		83.34	psf
SW		52.71	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Factored Load	1.2DL + 1.6LL + 0.5SD	327.75	psf
Factored Load*Tributary Area	(1.2DL + 1.6LL + 0.5SD)**w*	4375.48	plf
Total Load, w	((1.2DL+1.6LL)**w*)+SW	4428.19	plf

Allowable Loading			
Variable	Equation	Calculated Value	Units
F'b	Fb*CM*Ct*CL*CV*Cfu*Cc*CI*KF*Phi*Lambda	3700.26	psi
F'v	Fv*CM*Ct*Cvr	300.00	psi
F'clx	Fclx*CM*Ct*Cb	600.00	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Moment, M	(wL^2)/8	179540.93	lb*ft	
Max Bending Moment	M/Sx	2463.98	psi	66.59%
Shear, Vmax	(5wL/8)	39401.16	lbs	
Fv	3V/2bd	264.73	psi	88.24%
Fc(perpendicular)	V/bd	125.59	psi	20.93%

Allowable Deflection			
Variable	Equation	Calculated Value	Units
Delta LL	L/360	0.60	in

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity
Delta	(wL^4)/(185EI)	0.567	in	94%

Fire Glulam Roof

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	5.9	in
Depth, d	Depth	25.7	in
Triburaty Width, *w*	Tributary Width	13.35	ft
Ix	Moment of Inertia	8345.84	in^4
Sx	Section Modulus	649.48	in^3
A	Area	152	in^2
rx	Radius of Gyration	7.42	in
L	Length	18.01	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fbx'	Bending Moment	2400	psi
Fc⊥x	Compression Perpendicular to Grain	600	psi
Fvx	Shear Parallel to Grain	300	psi
Ex	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CD	Load Duration Factor	1	
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))	0.928	
CF	Size Factor	1.000	
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	

Loading			
Variable		Value	Units
LL		100	psf
DL		38.4	psf
SD		83.34	psf
SW		35.80	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Factored Load	DL + LL	191.31	psf
Factored Load*Tributary Area	(DL + LL)**w*	2553.94	plf
Total Load, w	((DL+LL)**w*)+SW	2589.74	plf

Allowable Loading			
Variable	Equation	Calculated Value	Units
F'b	Fb*2.85*CF*CV*Cfu*CL	3846.21	psi
F'v	Fv*CD*CM*Ct*Cvr	300.00	psi
F'c⊥x	Fc⊥x*CM*Ct*Cb	600.00	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Moment, M	(wL^2)/8	105001.19	lb*ft	
Max Bending Moment	M/Sx	1940.03	psi	50.44%
Shear, Vmax	(5wL/8)	29150.80	lbs	
Fv	3V/2bd	288.37	psi	96.12%
Fc(perpendicular)	V/bd	192.25	psi	32.04%

Allowable Deflection			
Variable	Equation	Calculated Value	Units
Delta LL	L/360	0.60	in

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity
Delta	(wL^4)/(185EI)	0.169	in	28.22%

Glulam Penthouse

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	7.25	in
Depth, d	Depth	15.5	in
Triburaty Width, *w*	Tributary Width	15	ft
Ix	Moment of Inertia	2249.84	in^4
Sx	Section Modulus	290.30	in^3
A	Area	112	in^2
rx	Radius of Gyration	4.47	in
L	Length	20	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fbx'	Bending Moment	2400	psi
Fc⊥x	Compression Perpendicular to Grain	600	psi
Fvx	Shear Parallel to Grain	300	psi
Ex	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))	0.946	
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	

Loading			
Variable		Value	Units
DL		35.1	psf
S		30	psf
SW Girder		26.53	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Factored Load	1.2DL + 1.6S	90.12	psf
Factored Load*Tributary Area	(1.2DL + 1.6S)**w*	1351.80	plf
Total Load, w	((1.2DL+1.6S)**w*)+SW	1378.33	plf

Allowable Loading			
Variable	Equation	Calculated Value	Units
F'b	Fb*CM*Ct*CL*CV*Cfu*Cc*CI*KF*Phi*Lambda	3921.87	psi
F'v	Fv*CM*Ct*Cvr	300.00	psi
F'c⊥x	Fc⊥x*CM*Ct*Cb	600.00	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Moment, M	(wL^2)/8	68916.65	lb*ft	
Max Bending Moment	M/Sx	2848.76	psi	72.64%
Shear, Vmax	(5wL/8)	16897.50	lbs	
Fv	3V/2bd	225.55	psi	75.18%
Fc(perpendicular)	V/bd	72.68	psi	12.11%

Allowable Deflection			
Variable	Equation	Calculated Value	Units
Delta LL	L/360	0.67	in

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity
Delta	(wL^4)/(185EI)	0.509	in	76%

Non-Fire Joist Floor

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	9.5	in
Depth, d	Depth	23.5	in
Triburaty Width, *w*	Tributary Width	8	ft
Ix	Moment of Inertia	10274.15	in^4
Sx	Section Modulus	874.40	in^3
A	Area	223	in^2
rx	Radius of Gyration	6.78	in
L	Length	21.35	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fbx'	Bending Moment	2400	psi
Fclx	Compression Perpendicular to Grain	600	psi
Fvx	Shear Parallel to Grain	300	psi
Ex	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))	1	
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	

Loading			
Variable		Value	Units
LL		100	psf
DL		94.12	psf
SW		52.71	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Factored Load	1.2DL + 1.6LL	272.94	psf
Factored Load*Tributary Area	(1.2DL + 1.6LL)**w*	2183.55	plf
Total Load, w	((1.2DL+1.6LL)**w*)+SW	2236.26	plf

Allowable Loading			
Variable	Equation	Calculated Value	Units
F'b	Fb*CM*Ct*CL*CV*Cfu*Cc*CI*KF*Phi*Lambda	3637.85	psi
F'v	Fv*CM*Ct*Cvr	300.00	psi
F'clx	Fclx*CM*Ct*Cb	600.00	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Moment, M	(wL^2)/8	127417.42	lb*ft	
Max Bending Moment	M/Sx	1748.65	psi	48.07%
Shear, Vmax	(5wL/8)	29136.77	lbs	
Fv	3V/2bd	195.77	psi	65.26%
Fc(perpendicular)	V/bd	130.51	psi	21.75%

Allowable Deflection			
Variable	Equation	Calculated Value	Units
Delta LL	L/360	0.71	in

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity
Delta	(5wL^4)/(384EI)	0.565	in	79%

Fire Joist Floor

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	5.9	in
Depth, d	Depth	23.7	in
Triburaty Width, *w*	Tributary Width	8	ft
Ix	Moment of Inertia	6545.09	in^4
Sx	Section Modulus	552.33	in^3
A	Area	140	in^2
rx	Radius of Gyration	6.84	in
L	Length	21.35	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fbx'	Bending Moment	2400	psi
Fclx	Compression Perpendicular to Grain	600	psi
Fvx	Shear Parallel to Grain	300	psi
Ex	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CD	Load Duration Factor	1	
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))	1	
CF	Size Factor	1	
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	

Loading			
Variable		Value	Units
LL		100	psf
DL		94.12	psf
SW		33.02	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Factored Load	DL + LL	194.12	psf
Factored Load*Tributary Area	(DL + LL)**w*	1552.96	plf
Total Load, w	((DL+LL)**w*)+SW	1585.98	plf

Allowable Loading			
Variable	Equation	Calculated Value	Units
F'b	Fb*2.85*CF*CV*Cfu*CL	6290.22	psi
F'v	Fv*CD*CM*Ct*Cvr	300.00	psi
F'clx	Fclx*CM*Ct*Cb	600.00	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Moment, M	(wL^2)/8	90365.41	lb*ft	
Max Bending Moment	M/Sx	1963.30	psi	31.21%
Shear, Vmax	(5wL/8)	20722.31	lbs	
Fv	3V/2bd	222.29	psi	74.10%
Fc(perpendicular)	V/bd	148.20	psi	24.70%

Allowable Deflection			
Variable	Equation	Calculated Value	Units
Delta LL	L/360	0.71	in

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity
Delta	(5wL^4)/(384EI)	0.629	in	88%

Non-Fire Joist Roof

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	9.5	in
Depth, d	Depth	23.5	in
Triburaty Width, *w*	Tributary Width	8	ft
Ix	Moment of Inertia	10274.15	in^4
Sx	Section Modulus	874.40	in^3
A	Area	223	in^2
rx	Radius of Gyration	6.78	in
L	Length	21.35	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fbx'	Bending Moment	2400	psi
Fclx	Compression Perpendicular to Grain	600	psi
Fvx	Shear Parallel to Grain	300	psi
Ex	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))	1	
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	

Loading			
Variable		Value	Units
LL		150	psf
DL		38.4	psf
SD		83.34	psf
SW		52.71	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Factored Load	1.2DL + 1.6LL + 0.5SD	327.75	psf
Factored Load*Tributary Area	(1.2DL + 1.6LL + 0.5SD)**w*	2622.01	plf
Total Load, w	((1.2DL+1.6LL)**w*)+SW	2674.72	plf

Allowable Loading			
Variable	Equation	Calculated Value	Units
F'b	Fb*CM*Ct*CL*CV*Cfu*Cc*CI*KF*Phi*Lambda	3637.85	psi
F'v	Fv*CM*Ct*Cvr	300.00	psi
F'clx	Fclx*CM*Ct*Cb	600.00	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Moment, M	(wL^2)/8	152399.69	lb*ft	
Max Bending Moment	M/Sx	2091.50	psi	57.49%
Shear, Vmax	(5wL/8)	34987.42	lbs	
Fv	3V/2bd	235.08	psi	78.36%
Fc(perpendicular)	V/bd	186.10	psi	31.02%

Allowable Deflection			
Variable	Equation	Calculated Value	Units
Delta LL	L/360	0.71	in

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity
Delta	(5wL^4)/(384EI)	0.676	in	95.0%

Fire Joist Roof

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	7.9	in
Depth, d	Depth	23.7	in
Triburaty Width, *w*	Tributary Width	8	ft
Ix	Moment of Inertia	8763.77	in^4
Sx	Section Modulus	739.56	in^3
A	Area	187	in^2
rx	Radius of Gyration	6.84	in
L	Length	21.35	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fbx'	Bending Moment	2400	psi
Fc┴x	Compression Perpendicular to Grain	600	psi
Fvx	Shear Parallel to Grain	300	psi
Ex	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CD	Load Duration Factor	1	
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))	1	
CF	Size Factor	1	
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	

Loading			
Variable		Value	Units
LL		150	psf
DL		38.4	psf
SD		83.34	psf
SW		44.21	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Factored Load	DL + LL	241.31	psf
Factored Load*Tributary Area	(DL + LL)**w*	1930.45	plf
Total Load, w	((DL+LL)**w*)+SW	1974.66	plf

Allowable Loading			
Variable	Equation	Calculated Value	Units
F'b	Fb*2.85*CF*CV*Cfu*CL	3702.42	psi
F'v	Fv*CD*CM*Ct*Cvr	300.00	psi
F'c┴x	Fc┴x*CM*Ct*Cb	600.00	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Moment, M	(wL^2)/8	112511.77	lb*ft	
Max Bending Moment	M/Sx	1825.60	psi	49.31%
Shear, Vmax	(wL/2)	20607.58	lbs	
Fv	3V/2bd	165.10	psi	55.03%
Fc(perpendicular)	V/bd	110.07	psi	18.34%

Allowable Deflection			
Variable	Equation	Calculated Value	Units
Delta LL	L/360	0.71	in

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity
Delta	(5wL^4)/(384EI)	0.585	in	82%

Joist Penthouse

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	13.625	in
Depth, d	Depth	15.5	in
Triburaty Width, *w*	Tributary Width	8	ft
Ix	Moment of Inertia	4228.15	in^4
Sx	Section Modulus	545.57	in^3
A	Area	211	in^2
rx	Radius of Gyration	4.47	in
L	Length	30	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fbx'	Bending Moment	2400	psi
Fc⊥x	Compression Perpendicular to Grain	600	psi
Fvx	Shear Parallel to Grain	300	psi
Ex	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))	0.853	
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	

Loading		
Variable	Value	Units
DL	35.1	psf
S	30	psf
SW	49.86	plf

Loading Conditions			
Variable	Equation	Calculated Value	Units
Factored Load	1.2DL + 1.6S	90.12	psf
Factored Load*Tributary Area	(1.2DL + 1.6S)**w*	360.48	plf
Total Load, w	((1.2DL+1.6S)**w*)+SW	410.34	plf

Allowable Loading			
Variable	Equation	Calculated Value	Units
F'b	Fb*CM*Ct*CL*CV*Cfu*Cc*CI*KF*Phi*Lambda	3535.77	psi
F'v	Fv*CM*Ct*Cvr	300.00	psi
F'c⊥x	Fc⊥x*CM*Ct*Cb	600.00	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Moment, M	(wL^2)/8	46163.67	lb*ft	
Max Bending Moment	M/Sx	1015.39	psi	28.72%
Shear, Vmax	(5wL/8)	6759.00	lbs	
Fv	3V/2bd	48.01	psi	16.00%
Fc(perpendicular)	V/bd	54.51	psi	9.08%

Allowable Deflection			
Variable	Equation	Calculated Value	Units
Delta LL	L/360	1.00	in

Actual Deflection				
Variable	Equation	Calculated Value	Units	Demand Capacity
Delta	(5wL^4)/(384EI)	0.983	in	98.3%

Non-Fire Column Floor

4th Floor Column Calcs

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	7.25	in
Depth, d	Depth	7.25	in
Triburaty Area, A	Tributary Area	384.375	ft^2
Ix	Moment of Inertia	230.2347	in^4
Sx	Section Modulus	63.51302	in^3
A	Area	53	in^2
rx	Radius of Gyration	2.09	in
L	Length	14	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fc	Compression Parallel to Grain	2300	psi
Fb	Bending Moment	2400	psi
Fc⊥	Compression Perpendicular to Grain	600	psi
Fv	Shear Parallel to Grain	300	psi
Eapp	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))		
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CF	Size Factor, For Fc	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	
c	Coefficient for Glulam, For Cp	0.9	
Ke	Buckling Length Coefficient for Compression Members	0.65	

Loading			
Variable		Value	Units
LL		100	psf
DL		94.12	psf
SW		12.77560764	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Reaction Force R1	10wL/8 (Girder)	81831.58	lbs
Reaction Force R2	Sw*L (Joist)	579.83	lbs
Reaction Force Total Rt	R1 + R2 + Rfloors above	183405.25	lbs

Allowable Loading			
Variable	Equation	Calculated Value	Units
Fc*	Fc*CM*Ct*CF*2.40*0.9*0.8	3974.4	psi
Eapp		1800000	psi
le	Ke*L	109.2	in
FcE	(0.822*Eapp)/((le/d)^2)	6521.913869	psi
Cp	((1+(FcE/Fc*))/(2c))-SQRT((((1+(FcE/Fc*))/(2c))^2)-((FcE/Fc*)/c))	0.893279717	
F'c	Fc*CM*Ct*CF*Ci*Cp*2.40*0.9*0.8	3550.250907	psi
Allowable Load	F'c*b*d	186610.0633	lb
Allowable Buckling Stress	Allowable Load/A	3550.250907	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Actual Load	Factored Load**w*	183405.25	lb	98.3%
Actual Buckling Stress	Actual Load/A	3489.28	psi	98.3%

Slenderness Check			
Direction	Equation	Calculated Value	
X-X	le/d	15.06	<50
Y-Y	le/b	15.06	<50

3rd Floor Column Calcs

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	9.5	in
Depth, d	Depth	9.5	in
Triburaty Area, A	Tributary Area	218.4	ft^2
Ix	Moment of Inertia	678.7552	in^4
Sx	Section Modulus	142.8958	in^3
A	Area	90	in^2
rx	Radius of Gyration	2.74	in
L	Length	14	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fc	Compression Parallel to Grain	2300	psi
Fb	Bending Moment	2400	psi
Fcl	Compression Perpendicular to Grain	600	psi
Fv	Shear Parallel to Grain	300	psi
Eapp	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))		
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CF	Size Factor, For Fc	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	
c	Coefficient for Glulam, For Cp	0.9	
Ke	Buckling Length Coefficient for Compression Members	0.65	

Loading			
Variable		Value	Units
LL		100	psf
DL		94.12	psf
SW		5.835763889	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Reaction Force R1	10wL/8 (Girder)	81831.58	lbs
Reaction Force R2	Sw*L (Joist)	579.83	lbs
Reaction Force Total Rt	R1 + R2 + Rfloors above	265995.52	lbs

Allowable Loading			
Variable	Equation	Calculated Value	Units
Fc*	Fc*CM*Ct*CF*2.40*0.9*0.8	3974.4	psi
Eapp		1800000	psi
le	Ke*L	109.2	in
FcE	(0.822*Eapp)/((le/d)^2)	6521.913869	psi
Cp	((1+(FcE/Fc*))/(2c))-SQRT((((1+(FcE/Fc*))/(2c))^2)-((FcE/Fc*)/c))	0.893279717	
F'c	Fc*CM*Ct*CF*Ci*Cp*2.40*0.9*0.8	3550.250907	psi
Allowable Load	F'c*b*d	320410.1444	lb
Allowable Buckling Stress	Allowable Load/A	3550.250907	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Actual Load	Factored Load**w*	265995.52	lb	83.0%
Actual Buckling Stress	Actual Load/A	2947.32	psi	83.0%

Slenderness Check			
Direction	Equation	Calculated Value	
X-X	le/d	11.49	<50
Y-Y	le/b	11.49	<50

2nd Floor Column Calcs

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description		Unit
Width, b	Width	11.5	in
Depth, d	Depth	11.5	in
Triburaty Area, A	Tributary Area	218.4	ft^2
Ix	Moment of Inertia	1457.505	in^4
Sx	Section Modulus	253.4792	in^3
A	Area	132	in^2
rx	Radius of Gyration	3.32	in
L	Length	14	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fc	Compression Parallel to Grain	2300	psi
Fb	Bending Moment	2400	psi
Fcl	Compression Perpendicular to Grain	600	psi
Fv	Shear Parallel to Grain	300	psi
Eapp	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))		
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CF	Size Factor, For Fc	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	
c	Coefficient for Glulam, For Cp	0.9	
Ke	Buckling Length Coefficient for Compression Members	0.65	

Loading			
Variable		Value	Units
LL		100	psf
DL		94.12	psf
SW		5.835763889	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Reaction Force R1	10wL/8 (Girder)	81831.58	lbs
Reaction Force R2	Sw*L (Joist)	579.83	lbs
Reaction Force Total Rt	R1 + R2 + Rfloors above	348488.63	lbs

Allowable Loading			
Variable	Equation	Calculated Value	Units
Fc*	Fc*CM*Ct*CF*2.40*0.9*0.8	3974.4	psi
Eapp		1800000	psi
le	Ke*L	109.2	in
FcE	(0.822*Eapp)/((le/d)^2)	6521.913869	psi
Cp	((1+(FcE/Fc*))/(2c))-SQRT((((1+(FcE/Fc*))/(2c))^2)-((FcE/Fc*)/c))	0.893279717	
F'c	Fc*CM*Ct*CF*Ci*Cp*2.40*0.9*0.8	3550.250907	psi
Allowable Load	F'c*b*d	469520.6824	lb
Allowable Buckling Stress	Allowable Load/A	3550.250907	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Actual Load	Factored Load**w*	348488.63	lb	74.2%
Actual Buckling Stress	Actual Load/A	2635.07	psi	74.2%

Slenderness Check			
Direction	Equation	Calculated Value	
X-X	le/d	9.50	<50
Y-Y	le/b	9.50	<50

1st Floor Column Calcs

Member Length & Properties, NDS Supplement Table 1C

Variable	Description		Unit
Width, b	Width	11.5	in
Depth, d	Depth	11.5	in
Triburaty Area, A	Tributary Area	218.4	ft^2
Ix	Moment of Inertia	1457.505	in^4
Sx	Section Modulus	253.4792	in^3
A	Area	132	in^2
rx	Radius of Gyration	3.32	in
L	Length	14	ft

Nordic Lam+ 24F-ES/NPG

Variable	Description	Value	Units
Fc	Compression Parallel to Grain	2300	psi
Fb	Bending Moment	2400	psi
Fc⊥	Compression Perpendicular to Grain	600	psi
Fv	Shear Parallel to Grain	300	psi
Eapp	Apparent Modulus of Elasticity	1800000	psi

Adjustment Factors

Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))		
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CF	Size Factor, For Fc	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	
c	Coefficient for Glulam, For Cp	0.9	
Ke	Buckling Length Coefficient for Compression Members	0.65	

Loading

Variable	Value	Units
LL	100	psf
DL	94.12	psf
SW	5.835763889	plf

Loading Conditions

Variable	Equation	Calculated Value	Units
Reaction Force R1	10wL/8 (Girder)	81831.58	lbs
Reaction Force R2	Sw*L (Joist)	579.83	lbs
Reaction Force Total Rt	R1 + R2 + Rfloors above	430981.75	lbs

Allowable Loading

Variable	Equation	Calculated Value	Units
Fc*	Fc*CM*Ct*CF*2.40*0.9*0.8	3974.4	psi
Eapp		1800000	psi
le	Ke*L	109.2	in
FcE	(0.822*Eapp)/((le/d)^2)	6521.913869	psi
Cp	((1+(FcE/Fc*))/(2c))-SQRT((((1+(FcE/Fc*))/(2c))^2)-((FcE/Fc*)/c))	0.893279717	
F'c	Fc*CM*Ct*CF*Ci*Cp*2.40*0.9*0.8	3550.250907	psi
Allowable Load	F'c*b*d	469520.6824	lb
Allowable Buckling Stress	Allowable Load/A	3550.250907	psi

Actual Loading

Variable	Equation	Calculated Value	Units	Demand Capacity
Actual Load	Factored Load*w	430981.75	lb	91.8%
Actual Buckling Stress	Actual Load/A	3258.84	psi	91.8%

Slenderness Check

Direction	Equation	Calculated Value	
X-X	le/d	9.50	<50
Y-Y	le/b	9.50	<50

Fire Column Floor

4th Floor Column Calcs			
Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	4.9	in
Depth, d	Depth	4.9	in
Triburaty Area, A	Tributary Area	384.375	ft^2
Ix	Moment of Inertia	48.04001	in^4
Sx	Section Modulus	19.60817	in^3
A	Area	24	in^2
rx	Radius of Gyration	1.41	in
L	Length	14	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fc	Compression Parallel to Grain	2300	psi
Fb	Bending Moment	2400	psi
Fcl	Compression Perpendicular to Grain	600	psi
Fv	Shear Parallel to Grain	300	psi
Eapp	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))		
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CF	Size Factor, For Fc	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	
c	Coefficient for Glulam, For Cp	0.9	
Ke	Buckling Length Coefficient for Compression Members	0.65	

Loading			
Variable		Value	Units
LL		100	psf
DL		94.12	psf
SW		5.835763889	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Reaction Force R1	10wL/8 (Girder)	57330.45	lbs
Reaction Force R2	Sw*L (Joist)	486.28	lbs
Reaction Force Total Rt	R1 + R2 + Rfloors above	58384.71	lbs

Allowable Loading			
Variable	Equation	Calculated Value	Units
Fc*	Fc*2.58*CF	5934	psi
Eapp		1800000	psi
le	Ke*L	109.2	in
FcE	(0.822*Eapp)/((le/d)^2)	6047.658284	psi
Cp	((1+(FcE/Fc*))/(2c))-SQRT((((1+(FcE/Fc*))/(2c))^2)-((FcE/Fc*)/c))	0.766879312	
F'c	Fc*2.58*CF*Cp	4550.661837	psi
Allowable Load	F'c*b*d	109261.3907	lb
Allowable Buckling Stress	Allowable Load/A	4550.661837	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Actual Load	Factored Load**w*	58384.71	lb	53.4%
Actual Buckling Stress	Actual Load/A	2431.68	psi	53.4%

Slenderness Check			
Direction	Equation	Calculated Value	
X-X	le/d	22.29	<50
Y-Y	le/b	22.29	<50

3rd Floor Column Calcs

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	5.9	in
Depth, d	Depth	5.9	in
Triburaty Area, A	Tributary Area	384.375	ft^2
Ix	Moment of Inertia	100.978	in^4
Sx	Section Modulus	34.22983	in^3
A	Area	35	in^2
rx	Radius of Gyration	1.70	in
L	Length	14	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fc	Compression Parallel to Grain	2300	psi
Fb	Bending Moment	2400	psi
Fcl	Compression Perpendicular to Grain	600	psi
Fv	Shear Parallel to Grain	300	psi
Eapp	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))		
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CF	Size Factor, For Fc	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	
c	Coefficient for Glulam, For Cp	0.9	
Ke	Buckling Length Coefficient for Compression Members	0.65	

Loading			
Variable		Value	Units
LL		100	psf
DL		94.12	psf
SW		8.460763889	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Reaction Force R1	10wL/8 (Girder)	57330.45	lbs
Reaction Force R2	Sw*L (Joist)	486.28	lbs
Reaction Force Total Rt	R1 + R2 + Rfloors above	116283.13	lbs

Allowable Loading			
Variable	Equation	Calculated Value	Units
Fc*	Fc*2.58*CF	5934	psi
Eapp		1800000	psi
le	Ke*L	109.2	in
FcE	(0.822*Eapp)/((le/d)^2)	6047.658284	psi
Cp	((1+(FcE/Fc*))/(2c))-SQRT((((1+(FcE/Fc*))/(2c))^2)-((FcE/Fc*)/c))	0.766879312	
F'c	Fc*2.58*CF*Cp	4550.661837	psi
Allowable Load	F'c*b*d	158408.5385	lb
Allowable Buckling Stress	Allowable Load/A	4550.661837	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Actual Load	Factored Load*w	116283.13	lb	73.4%
Actual Buckling Stress	Actual Load/A	3340.51	psi	73.4%

Slenderness Check			
Direction	Equation	Calculated Value	
X-X	le/d	18.51	<50
Y-Y	le/b	18.51	<50

2nd Floor Column Calcs

Member Length & Properties, NDS Supplement Table 1C

Variable	Description		Unit
Width, b	Width	7.9	in
Depth, d	Depth	7.9	in
Triburaty Area, A	Tributary Area	384.375	ft^2
Ix	Moment of Inertia	324.584	in^4
Sx	Section Modulus	82.17317	in^3
A	Area	62	in^2
rx	Radius of Gyration	2.28	in
L	Length	14	ft

Nordic Lam+ 24F-ES/NPG

Variable	Description	Value	Units
Fc	Compression Parallel to Grain	2300	psi
Fb	Bending Moment	2400	psi
Fcl	Compression Perpendicular to Grain	600	psi
Fv	Shear Parallel to Grain	300	psi
Eapp	Apparent Modulus of Elasticity	1800000	psi

Adjustment Factors

Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))		
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CF	Size Factor, For Fc	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	
c	Coefficient for Glulam, For Cp	0.9	
Ke	Buckling Length Coefficient for Compression Members	0.65	

Loading

Variable	Value	Units
LL	100	psf
DL	94.12	psf
SW	15.16909722	plf

Loading Conditions

Variable	Equation	Calculated Value	Units
Reaction Force R1	10wL/8 (Girder)	57330.45	lbs
Reaction Force R2	Sw*L (Joist)	486.28	lbs
Reaction Force Total Rt	R1 + R2 + Rfloors above	174218.31	lbs

Allowable Loading

Variable	Equation	Calculated Value	Units
Fc*	Fc*2.58*CF	5934	psi
Eapp		1800000	psi
le	Ke*L	109.2	in
FcE	(0.822*Eapp)/((le/d)^2)	6047.658284	psi
Cp	((1+(FcE/Fc*))/(2c))-SQRT((((1+(FcE/Fc*))/(2c))^2)-((FcE/Fc*)/c))	0.766879312	
F'c	Fc*2.58*CF*Cp	4550.661837	psi
Allowable Load	F'c*b*d	284006.8052	lb
Allowable Buckling Stress	Allowable Load/A	4550.661837	psi

Actual Loading

Variable	Equation	Calculated Value	Units	Demand Capacity
Actual Load	Factored Load*w	174218.31	lb	61.3%
Actual Buckling Stress	Actual Load/A	2791.51	psi	61.3%

Slenderness Check

Direction	Equation	Calculated Value	
X-X	le/d	13.82	<50
Y-Y	le/b	13.82	<50

1st Floor Column Calcs

Member Length & Properties, NDS Supplement Table 1C

Variable	Description		Unit
Width, b	Width	7.9	in
Depth, d	Depth	7.9	in
Triburaty Area, A	Tributary Area	384.375	ft^2
Ix	Moment of Inertia	324.584	in^4
Sx	Section Modulus	82.17317	in^3
A	Area	62	in^2
rx	Radius of Gyration	2.28	in
L	Length	14	ft

Nordic Lam+ 24F-ES/NPG

Variable	Description	Value	Units
Fc	Compression Parallel to Grain	2300	psi
Fb	Bending Moment	2400	psi
Fc⊥	Compression Perpendicular to Grain	600	psi
Fv	Shear Parallel to Grain	300	psi
Eapp	Apparent Modulus of Elasticity	1800000	psi

Adjustment Factors

Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
x	For all species except Southern Pine	10	
CV	((21/L)^(1/x))*((12/d)^(1/x))*((5.125/b)*(1/x))		
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CF	Size Factor, For Fc	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	
c	Coefficient for Glulam, For Cp	0.9	
Ke	Buckling Length Coefficient for Compression Members	0.65	

Loading

Variable	Value	Units
LL	100	psf
DL	94.12	psf
SW	15.16909722	plf

Loading Conditions

Variable	Equation	Calculated Value	Units
Reaction Force R1	10wL/8 (Girder)	57330.45	lbs
Reaction Force R2	Sw*L (Joist)	486.28	lbs
Reaction Force Total Rt	R1 + R2 + Rfloors above	232247.41	lbs

Allowable Loading

Variable	Equation	Calculated Value	Units
Fc*	Fc*2.58*CF	5934	psi
Eapp		1800000	psi
le	Ke*L	109.2	in
FcE	(0.822*Eapp)/((le/d)^2)	6047.658284	psi
Cp	((1+(FcE/Fc*))/(2c))-SQRT((((1+(FcE/Fc*))/(2c))^2)-((FcE/Fc*)/c))	0.766879312	
F'c	Fc*2.58*CF*Cp	4550.661837	psi
Allowable Load	F'c*b*d	284006.8052	lb
Allowable Buckling Stress	Allowable Load/A	4550.661837	psi

Actual Loading

Variable	Equation	Calculated Value	Units	Demand Capacity
Actual Load	Factored Load**w*	232247.41	lb	81.8%
Actual Buckling Stress	Actual Load/A	3721.32	psi	81.8%

Slenderness Check

Direction	Equation	Calculated Value	
X-X	le/d	13.82	<50
Y-Y	le/b	13.82	<50

Non-Fire Column Roof

Top Floor Column Calcs (Roof Loading)

Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	7.25	in
Depth, d	Depth	7.25	in
Triburaty Area, A	Tributary Area	384.375	ft^2
Ix	Moment of Inertia	230.2347	in^4
Sx	Section Modulus	63.51302	in^3
A	Area	53	in^2
rx	Radius of Gyration	2.09	in
L	Length	14	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fc	Compression Parallel to Grain	2300	psi
Fb	Bending Moment	2400	psi
Fc⊥	Compression Perpendicular to Grain	600	psi
Fv	Shear Parallel to Grain	300	psi
Eapp	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
Cfu	Flat Use Factor	1	
CF	Size Factor, For Fc	1	
CI	Stress Interaction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.4	
Phi	Resistance Factor	0.9	
Lambda	Time Effect Factor	0.8	
c	Coefficient for Glulam, For Cp	0.9	
Ke	Buckling Length Coefficient for Compression Men	0.65	

Loading			
Variable		Value	Units
LL		150	psf
DL		38.4	psf
SD		83.34	psf
SW		12.77560764	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Reaction Force R1	10wL/8 (Girder)	99689.58	lbs
Reaction Force R2	Sw*L (Joist)	1125.40	lbs
Reaction Force Total Rt		100814.98	lbs

Allowable Loading			
Variable	Equation	Calculated Value	Units
Fc*	Fc*CM*Ct*CF*2.40*0.9*0.8	3974.4	psi
Eapp		1800000	psi
le	Ke*L	109.2	in
FcE	(0.822*Eapp)/((le/d)^2)	6521.913869	psi
Cp	((1+(FcE/Fc*))/(2c))-SQRT((((1+(FcE/Fc*))/(2c))^2)-((FcE/Fc*)/c))	0.893279717	
F'c	Fc*CM*Ct*CF*Ci*Cp*2.40*0.9*0.8	3550.250907	psi
Allowable Load	F'c*b*d	186610.0633	lb
Allowable Buckling Stress	Allowable Load/A	3550.250907	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Actual Load	Factored Load*w	100814.98	lb	54.0%
Actual Buckling Stress	Actual Load/A	1918.00	psi	54.0%

Slenderness Check			
Direction	Equation	Calculated Value	
X-X	le/d	15.06	<50
Y-Y	le/b	15.06	<50

Fire Column Roof

Top Floor Column Calcs (Roof Loading) with Fire			
Member Length & Properties, NDS Supplement Table 1C			
Variable	Description	Value	Unit
Width, b	Width	4.9	in
Depth, d	Depth	4.9	in
Triburaty Area, A	Tributary Area	384.375	ft^2
Ix	Moment of Inertia	48.04	in^4
Sx	Section Modulus	19.61	in^3
A	Area	24	in^2
rx	Radius of Gyration	1.41	in
L	Length	14	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fc	Compression Parallel to Grain	2300	psi
Fb	Bending Moment	2400	psi
Fc⊥	Compression Perpendicular to Grain	600	psi
Fv	Shear Parallel to Grain	300	psi
Eapp	Apparent Modulus of Elasticity	1800000	psi
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
Cfu	Flat Use Factor	1	
Cc	Curvature Factor	1	
CF	Size Factor, For Fc	1	
CI	Stress Interaction Factor	1	
Cvr	Shear Reduction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.54	
Phi	Resistance Factor	0.85	
Lambda	Time Effect Factor	0.8	
c	Coefficient for Glulam, For Cp	0.9	
Ke	Buckling Length Coefficient for Compression Members	0.65	

Loading			
Variable		Value	Units
LL		150	psf
DL		38.4	psf
SD		83.34	psf
SW		5.835763889	plf
Loading Conditions			
Variable	Equation	Calculated Value	Units
Reaction Force R1	10wL/8 (Girder)	58301.60	lbs
Reaction Force R2	Sw*L (Joist)	943.82	lbs
Reaction Force Total Rt		59245.42	lbs

Allowable Loading			
Variable	Equation	Calculated Value	Units
Fc*	Fc*2.58*CF	5934	psi
Eapp		1800000	psi
le	Ke*L	109.2	in
FcE	(0.822*Eapp)/((le/d)^2)	6047.658284	psi
Cp	((1+(FcE/Fc*))/(2c))-SQRT((((1+(FcE/Fc*))/(2c))^2)-((FcE/Fc*)/c))	0.766879312	
F'c	Fc*2.58*CF*Cp	4550.661837	psi
Allowable Load	F'c*b*d	109261.3907	lb
Allowable Buckling Stress	Allowable Load/A	4550.661837	psi

Actual Loading				
Variable	Equation	Calculated Value	Units	Demand Capacity
Actual Load	Factored Load**w*	59245.42	lb	54.2%
Actual Buckling Stress	Actual Load/A	2467.53	psi	54.2%

Slenderness Check			
Direction	Equation	Calculated Value	
X-X	le/d	22.29	<50
Y-Y	le/b	22.29	<50

Column Penthouse

Penthouse Column Calcs (Penthouse Roof Loading)

Member Length & Properties, NDS Supplement Table 1C

Variable	Description	Value	Unit
Width, b	Width	5.375	in
Depth, d	Depth	5.375	in
Triburaty Area, A	Tributary Area	384.375	ft^2
Ix	Moment of Inertia	69.56	in^4
Sx	Section Modulus	25.88	in^3
A	Area	29	in^2
rx	Radius of Gyration	1.55	in
L	Length	18	ft
Nordic Lam+ 24F-ES/NPG			
Variable	Description	Value	Units
Fc	Compression Parallel to Grain	2300	psi
Fb	Bending Moment	2400	psi
Fcl	Compression Perpendicular to Grain	600	psi
Fv	Shear Parallel to Grain	300	psi
Eapp	Apparent Modulus of Elasticity	1800000	psi
Adjustment Factors			
Variable	Description	Value	Units
CM	Wet Service Factor	1	
Ct	Temperature Factor	1	
CL	Beam Stability Factor	1	
Cfu	Flat Use Factor	1	
CF	Size Factor, For Fc	1	
CI	Stress Interaction Factor	1	
Cb	Bearing Area Factor	1	
KF	Format Conversion Factor	2.4	
Phi	Resistance Factor	0.9	
Lambda	Time Effect Factor	0.8	
c	Coefficient for Glulam, For Cp	0.9	
Ke	Buckling Length Coefficient for Compression Members	0.65	

Loading

Variable	Value	Units
DL	20	psf
S	30	psf

Loading Conditions

Variable	Equation	Calculated Value	Units
Reaction Force R1	wL/2 (Girder)	34458.32	lbs
Reaction Force Total Rt		34458.32	lbs

Allowable Loading

Variable	Equation	Calculated Value	Units
Fc*	Fc*CM*Ct*CF*2.40*0.9*0.8	3974.4	psi
Eapp		1800000	psi
le	Ke*L	140.4	in
FcE	(0.822*Eapp)/((le/d)^2)	2168.538037	psi
Cp	((1+(FcE/Fc*))/(2c))-SQRT((((1+(FcE/Fc*))/(2c))^2)-((FcE/Fc*)/c))	0.496628837	
F'c	Fc*CM*Ct*CF*Ci*Cp*2.40*0.9*0.8	1973.80165	psi
Allowable Load	F'c*b*d	57024.36328	lb
Allowable Buckling Stress	Allowable Load/A	1973.80165	psi

Actual Loading

Variable	Equation	Calculated Value	Units	Demand Capacity
Actual Load	Factored Load*w	34458.32	lb	60.4%
Actual Buckling Stress	Actual Load/A	1192.72	psi	60.4%

Slenderness Check

Direction	Equation	Calculated Value	
X-X	le/d	26.12	<50
Y-Y	le/b	26.12	<50

Non-Fire Shear Walls

North-South Side (Short Side)								
Floor	b (Width, ft)	Wind Load (psf)	Height of Story (ft)	Wind Load (wu, plf)	Vu (lbs)	Length of Shear Walls (ft)	Shear (Vt, lbs/ft)	C=T=Vt*Height of Wall (lb)
1	130	34.2	16.9	579.6	37674.5	14	16216.26	227027.62
2	130	37.5	16.9	635.5	41309.8	12	15779.43	220911.96
3	130	39.7	16.9	672.8	43733.3	9	16449.26	230289.63
4	130	41.3	16.9	699.9	45495.8	7	14901.44	208620.11
5	130	42.7	16.5	705.9	45882.3	4	14703.56	205849.77
Penthouse	30	44.0	19.59375	862.1	12931.9	1	12931.88	232773.75

Capacity - Loading Perpendicular to Layers				
Floor Diaphragm	CLT Panel	Rolling Shear (psi)	IB/Q (in^2)	Allowable Shear (lb)
1	197-7S	45	343.75	15468.75
2	197-7S	45	343.75	15468.75
3	197-7S	45	343.75	15468.75
4	197-7S	45	343.75	15468.75
5	197-7S	45	343.75	15468.75
Penthouse	197-7S	45	343.75	15468.75

Capacity - Loading Parallel to Layers					
Floor Walls	CLT Panel	Edgewise Shear (Fv, psi)	Width (in)	Allowable Ve (lbs/ft)	Allowable C=T=Ve*Height of Wall (lb)
1	197-7S	215	7.75	19995	240439.88
2	197-7S	215	7.75	19995	240439.88
3	197-7S	215	7.75	19995	240439.88
4	197-7S	215	7.75	19995	240439.88
5	197-7S	215	7.75	19995	237107.38
Penthouse	197-7S	215	7.75	19995	334083.13

Fire Shear Walls

North-South Side (Short Side)								
Floor	b (Width, ft)	Wind Load (psf)	Height of Story (ft)	Wind Load (wu, plf)	Vu (lbs)	Length of Shear Walls (ft)	Shear (Vt, lbs/ft)	C=T=Vt*Height of Wall (lb)
1	130	34.2	17.0	581.7	37813.2	20	11393.21	159504.95
2	130	37.5	17.0	637.9	41461.9	18	10558.39	147817.43
3	130	39.7	17.0	675.3	43894.3	14	10613.51	148589.11
4	130	41.4	17.0	704.2	45773.9	10	10469.48	146572.72
5	130	42.7	16.5	705.8	45879.0	6	9820.15	137482.08
Penthouse	30	44.0	19.76	869.5	13041.9	2	6520.94	117376.88

Capacity - Loading Perpendicular to Layers				
Floor Diaphragm	CLT Panel	Rolling Shear (psi)	IB/Q (in^2)	Allowable Shear (lb)
1	197-7S	45	99.65	4484.25
2	197-7S	45	99.65	4484.25
3	197-7S	45	99.65	4484.25
4	197-7S	45	99.65	4484.25
5	197-7S	45	99.65	4484.25
Penthouse	197-7S	45	99.65	4484.25

Capacity - Loading Parallel to Layers					
Floor Walls	CLT Panel	Edgewise Shear (Fv, psi)	Width (in)	Allowable Ve (lbs/ft)	Allowable C=T=Ve*Height of Wall (lb)
1	197-7S	215	4.55	11739	164346.00
2	197-7S	215	4.55	11739	164346.00
3	197-7S	215	4.55	11739	164346.00
4	197-7S	215	4.55	11739	164346.00
5	197-7S	215	4.55	11739	164346.00
Penthouse	197-7S	215	4.55	11739	211302.00

Appendix D: Lateral Load Calculations

Seismic Load Calculations

SEISMIC BASE SHEAR AND VERTICAL SHEAR DISTRIBUTION

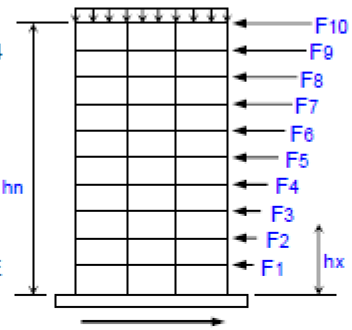
Per IBC 2012 and ASCE 7-10 Specifications

Using Equivalent Lateral Force Procedure for Regular Multi-Level Building/Structural Systems

Job Name:		Subject:			
Job Number:		Originator:		Checker:	

Input Data:

Risk Category =	II	IBC 2012, Table 1604.5, page 336
Importance Factor, I =	1.00	ASCE 7-10 Table 1.5-2, page 5
Soil Site Class =	D	ASCE 7-10 Table 20.3-1, page 204
Location Zip Code =		(not required)
Spectral Accel., S_S =	0.220	ASCE 7-10 Figures 22-1 to 22-11
Spectral Accel., S_1 =	0.065	ASCE 7-10 Figures 22-1 to 22-11
Long. Trans. Period, T_L =	6.000	sec. ASCE 7 Fig's. 22-12 to 22-18
Structure Height, h_n =	104.000	ft.
Actual Calc. Period, T_c =	0.000	sec. from independent analysis
Seismic Resist. System =	B24	Light-framed walls with shear panels of all other materials (ASCE 7-10 Table 12.2-1)



V = Cs*W = Σ(Fi) = 281.42 kips

Seismic Base Shear

(Regular Bldg. Configurations Only)

Structure Weight Distribution:

No. of Seismic Levels = 6

Seismic Level x	Height, hx (ft.)	Weight, Wx (kips)
6	104.000	110.20
5	84.500	859.65
4	68.000	840.31
3	51.000	851.11
2	34.000	870.73
1	17.000	874.22

Total Weight, W = ΣW_x = 4406.22 kips (ASCE 7-10 Section 12.7.2)

Results:

Site Coefficients:

F_a =	1.600	ASCE 7-10 Table 11.4-1, page 66
F_v =	2.400	ASCE Table 11.4-2, page 66

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

S_{MS} =	0.352	$S_{MS} = F_a*S_S$, ASCE Eqn. 11.4-1, page 65
S_{M1} =	0.156	$S_{M1} = F_v*S_1$, ASCE Eqn. 11.4-2, page 65

Design Spectral Response Accelerations for Short and 1-Second Periods :

S_{DS} =	0.235	$S_{DS} = 2*S_{MS}/3$, ASCE 7-10 Eqn. 11.4-3, page 65
S_{D1} =	0.104	$S_{D1} = 2*S_{M1}/3$, ASCE Eqn. 11.4-4, page 65

(continued:)

Seismic Design Category:

Category(for S_{DS}) =	B	ASCE 7-10 Table 11.6-1, page 67
Category(for S_{D1}) =	B	ASCE 7-10 Table 11.6-2, page 67
Use Category =	B	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T =	0.020	ASCE 7-10 Table 12.8-2, page 90
Period Exponent, x =	0.75	ASCE 7-10 Table 12.8-2, page 90
Approx. Period, T_a =	0.651	sec., $T_a = C_T*hn\^(x)$, ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.692	ASCE 7-10 Table 12.8-1, page 90
Period max., $T_{(max)}$ =	1.102	sec., $T_{(max)} = C_u*T_a$, ASCE 7-10 Section 12.8.2, page 90
Fundamental Period, T =	0.651	sec., $T = T_a <= C_u*T_a$, ASCE 7-10 Section 12.8.2, page 90

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	2.5	ASCE 7-10 Table 12.2-1, pages 73-75
Overstrength Factor, Ω_o =	2.5	ASCE 7-10 Table 12.2-1, pages 73-75
Defl. Amplif. Factor, C_d =	2.5	ASCE 7-10 Table 12.2-1, pages 73-75
C_S =	0.094	$C_S = S_{DS}/(R/I)$, ASCE 7-10 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.064	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-10 Eqn. 12.8-3
$C_{S(min)}$ =	0.010	CS(min) = 0.044*SDS*I >= 0.01, ASCE 7-10 Eqn. 12.8-5
Use: C_S =	0.064	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V = 281.42 kips, $V = C_S*W$, ASCE 7-10 Section 12.8.1, Eqn. 12.8-1

Seismic Shear Vertical Distribution:

Distribution Exponent, k = 1.08 k = 1 for T<=0.5 sec., k = 2 for T>=2.5 sec.
k = (2-1)*(T-0.5)/(2.5-0.5)+1, for 0.5 sec. < T < 2.5 sec.

Lateral Force at Any Level: $F_x = C_{vx}*V$, ASCE 7-10 Section 12.8.3, Eqn. 12.8-11, page 91

Vertical Distribution Factor: $C_{vx} = W_x*h_x\^k/(\Sigma W_i*h_i\^k)$, ASCE 7-10 Eqn. 12.8-12, page 91

Seismic Level x	Weight, W_x (kips)	$h_x\^k$ (ft.)	$W_x*h\^k$ (ft-kips)	C_{vx} (%)	Shear, F_x (kips)	Σ Story Shears
6	110.20	147.794	16286.9	0.052	14.69	14.69
5	859.65	118.211	101620.1	0.326	91.64	106.33
4	840.31	93.577	78634.0	0.252	70.91	177.24
3	851.11	68.672	58447.3	0.187	52.71	229.95
2	870.73	44.398	38658.6	0.124	34.86	264.81
1	874.22	21.065	18415.1	0.059	16.61	281.42
Σ =	4406.22		312062.1	1.000	281.42	

Comments:

Wind Loads in the North-South Directon

FLSMIDTH	**MWFRS Wind Loads** *ASCE 7-10* *Enclosed & Partially Enclosed Buildings of All Heights*	Job No: 11054 Designer: DCB Checker:
	Notes: Grinding Building (+/- Z Direction)	Date: 1/16/2023

Basic Parameters

Risk Category	II	Table 1.5-1
Basic Wind Speed, V	128 mph	Figure 26.5-1A
Wind Directionality Factor, K_d	0.85	Table 26.6-1
Exposure Category	C	Section 26.7
Topographic Factor, K_{zt}	1.00	Section 26.8
Gust Effect Factor, G or G_f	0.850	Section 26.9
Enclosure Classification	Enclosed	Section 26.10
Internal Pressure Coefficient, GC_{pi}	+/- 0.18	Table 26.11-1
Terrain Exposure Constant, α	9.5	Table 26.9-1
Terrain Exposure Constant, z_g	900 ft	Table 26.9-1

Wall Pressure Coefficients

Windward Wall Width, B	235 ft	
Side Wall Width, L	130 ft	
L/B Ratio	0.55	
Windward Wall Coefficient, C_p	0.80	Figure 27.4-1
Leeward Wall Coefficient, C_p	-0.50	Figure 27.4-1
Side Wall Coefficient, C_p	-0.70	Figure 27.4-1

Roof Pressure Coefficients

Roof Slope, θ	0.0°	
Median Roof Height, h	104 ft	
Velocity Pressure Exposure Coef., K_h	1.28	Table 27.3-1
Velocity Pressure, q_h	45.5 psf	Equation 27.3-1
h/L Ratio	0.80	
Windward Roof Area	0 ft²	
Roof Area Within 52 ft of WW Edge	0 ft²	

Location	*Min/Max*	*Horiz Distance From Windward Edge*			
		0 ft	*52 ft*	*104 ft*	*208 ft*
Windward Roof Coefficient Normal to Ridge, C_p	Min	0.24	0.24	-0.62	-0.54
	Max	-0.18	-0.18	-0.18	-0.18
Leeward Roof Coefficient Normal to Ridge, C_p	Min	0.24	0.24	-0.62	-0.54
	Max	-0.18	-0.18	-0.18	-0.18
Roof Coefficient Parallel to Ridge, C_p	Min	0.24	0.24	-0.62	-0.54
	Max	-0.18	-0.18	-0.18	-0.18

Figure 27.4-1

Structure Pressure Summary (Add Internal Pressure q_iGC_{pi} or q_hGC_{pi} as Necessary)

Height, z	K_z	q_z	*Walls*				*Roof*			Internal	
							Normal to Ridge		*Parallel to Ridge*		
			WW	*LW*	*WW + LW*	*Side*	*WW*	*LW*		Positive	Negative
17 ft	0.87	31.1 psf	21.1 psf		40.4 psf					8.2 psf	
34 ft	1.01	35.9 psf	24.4 psf		43.8 psf		Min:	Min:	Min:	8.2 psf	
51 ft	1.10	39.1 psf	26.6 psf		45.9 psf		-24.0 psf	-24.0 psf	-24.0 psf	8.2 psf	
68 ft	1.17	41.6 psf	28.3 psf		47.6 psf					8.2 psf	
84 ft	1.22	43.5 psf	29.6 psf		48.9 psf					8.2 psf	
104 ft	1.28	45.5 psf	30.9 psf	-19.3 psf	50.3 psf	-27.1 psf				8.2 psf	-8.2 psf
116 ft	1.31	46.6 psf	31.7 psf		51.0 psf					8.2 psf	
128 ft	1.33	47.5 psf	32.3 psf		51.7 psf		Max:	Max:	Max:	8.2 psf	
140 ft	1.36	48.4 psf	32.9 psf		52.3 psf		-7.0 psf	-7.0 psf	-7.0 psf	8.2 psf	
152 ft	1.38	49.3 psf	33.5 psf		52.9 psf					8.2 psf	
104 ft	1.28	45.5 psf	30.9 psf		50.3 psf					8.2 psf	

Wind Loads in the West-East Direction

FLSMIDTH	**MWFRS Wind Loads** *ASCE 7-10* *Enclosed & Partially Enclosed Buildings of All Heights*	Job No: 11054 Designer: DCB Checker: Date: 3/15/2023
	Notes: Grinding Building (+/- Z Direction)	

Basic Parameters

Risk Category	II	Table 1.5-1
Basic Wind Speed, V	128 mph	Figure 26.5-1A
Wind Directionality Factor, K_d	0.85	Table 26.6-1
Exposure Category	C	Section 26.7
Topographic Factor, K_{zt}	1.00	Section 26.8
Gust Effect Factor, G or G_f	0.850	Section 26.9
Enclosure Classification	Enclosed	Section 26.10
Internal Pressure Coefficient, GC_{pi}	+/- 0.18	Table 26.11-1
Terrain Exposure Constant, α	9.5	Table 26.9-1
Terrain Exposure Constant, z_g	900 ft	Table 26.9-1

Wall Pressure Coefficients

Windward Wall Width, B	130 ft	
Side Wall Width, L	235 ft	
L/B Ratio	1.81	
Windward Wall Coefficient, C_p	0.80	Figure 27.4-1
Leeward Wall Coefficient, C_p	-0.44	Figure 27.4-1
Side Wall Coefficient, C_p	-0.70	Figure 27.4-1

Roof Pressure Coefficients

Roof Slope, θ	0.0°	
Median Roof Height, h	104 ft	
Velocity Pressure Exposure Coef., K_h	1.28	Table 27.3-1
Velocity Pressure, q_h	45.5 psf	Equation 27.3-1
h/L Ratio	0.44	
Windward Roof Area	0 ft²	
Roof Area Within 52 ft of WW Edge	0 ft²	

Location	*Min/Max*	*Horiz Distance From Windward Edge*			
		0 ft	*52 ft*	*104 ft*	*208 ft*
Windward Roof Coefficient Normal to Ridge, C_p	Min	-0.90	-0.90	-0.90	-0.90
	Max	-0.18	-0.18	-0.18	-0.18
Leeward Roof Coefficient Normal to Ridge, C_p	Min	-0.90	-0.90	-0.90	-0.90
	Max	-0.18	-0.18	-0.18	-0.18
Roof Coefficient Parallel to Ridge, C_p	Min	-0.90	-0.90	-0.90	-0.90
	Max	-0.18	-0.18	-0.18	-0.18

Figure 27.4-1

Structure Pressure Summary (Add Internal Pressure q_zGC_{pi} or q_hGC_{pi} as Necessary)

Height, z	*K_z*	*q_z*	*Walls*				*Roof*			Internal	
							Normal to Ridge		*Parallel*		
			WW	*LW*	*WW + LW*	*Side*	*WW*	*LW*	*to Ridge*	Positive	Negative
17 ft	0.87	31.1 psf	21.1 psf		38.1 psf					8.2 psf	
34 ft	1.01	36.0 psf	24.4 psf		41.5 psf		Min:	Min:	Min:	8.2 psf	
51 ft	1.10	39.2 psf	26.6 psf		43.6 psf		-34.8 psf	-34.8 psf	-34.8 psf	8.2 psf	
68 ft	1.17	41.6 psf	28.3 psf		45.3 psf					8.2 psf	
84 ft	1.22	43.5 psf	29.6 psf		46.6 psf					8.2 psf	
104 ft	1.28	45.5 psf	30.9 psf	-17.0 psf	48.0 psf	-27.1 psf				8.2 psf	-8.2 psf
116 ft	1.31	46.6 psf	31.7 psf		48.7 psf					8.2 psf	
128 ft	1.33	47.5 psf	32.3 psf		49.3 psf		Max:	Max:	Max:	8.2 psf	
140 ft	1.36	48.4 psf	32.9 psf		50.0 psf		-7.0 psf	-7.0 psf	-7.0 psf	8.2 psf	
152 ft	1.38	49.3 psf	33.5 psf		50.5 psf					8.2 psf	
104 ft	1.28	45.5 psf	30.9 psf		48.0 psf					8.2 psf	

Appendix E: Vibration Analysis

Design Inputs

DESIGN INPUTS

Performance Target

Occupancy	Premium offices or luxury residences	
Peak Acceleration	0.3	% g
RMS Velocity	8,000 - 16,000	mips

If the facility contains vibration-sensitive equipment or occupancy:

Chosen Acceleration	0.3	% g
Chosen RMS Velocity	8000	mips

Walking Parameters

Occupancy	Laboratories or surgical theaters	
Walker Speed	Very slow (uncommon)	
Steps Per Minute	75	SPM
Walker Frequency	1.25	Hz
Walker Weight	168	lb
Footfall Force	65	lb
Stride Length	2.5	ft

Floor Layout

Supporting System	Beam-supporting	
Span	Two span continuous	
Max. Span Length	21.35	ft
Panel Length	42.7	ft
Panel Width	8	ft
Number of Panels	90	
Define beam system:		
Length of Joists	21.35	ft
Joist Spacing	18.01	ft
Length of Girders	18.01	ft

Floor Material Properties

CLT Panel

Panel Thickness	6.875	in
Number of Layers	5	
Density	32	pcf
Weight	18.4	psf
Layup or Grade	175-5s	
E	1700000	psi
E_{perp}	1200000	psi
Thickness of L layers	1.375	in
Thickness of T layers	1.375	in
$EI_{eff,f,0}$	440	10^6 lbf-in^2/ft
$EI_{eff,f,90}$	81	10^6 lbf-in^2/ft
$GA_{eff,f,0}$	0.92	10^6 lbf-in^2/ft
$GA_{eff,f,90}$	1.2	10^6 lbf-in^2/ft

Glulam Joist

Beam Width	9.5	in
Beam Depth	23.5	in
Area	223.25	in^2
Density	34	pcf
Weight	66.58333333	psf
$E_{x,app}$	1800000	psi
E_sI	17240	10^6 lbf-in^2

Glulam Girder

Beam Width	15.5	in
Beam Depth	19.5	in
Area	302.25	in^2
Density	34	pcf
Weight	55.25	psf
$E_{x,app}$	1800000	psi
E_sI	17240	10^6 lbf-in^2

Concrete Topping

Thickness	5	in
Compressive Strength	4000	psi
Density	145	pcf
Weight	60.41666667	psf
E_c	3604996.533	psi
Composite Case	Topping cast directly on CLT panel with no connection	
Composite Factor	0.05	
Chosen Factor	0	

Damping

Chosen Damping	4	%
Damping Ratio	0.04	β

Loading

Occupancy		
Expected Live Load	Revise	psf
Chosen Live Load	100	psf
MEP system?	No	
Drywall ceiling?	No	
Total Live Load	100	psf

Results and Recommendations

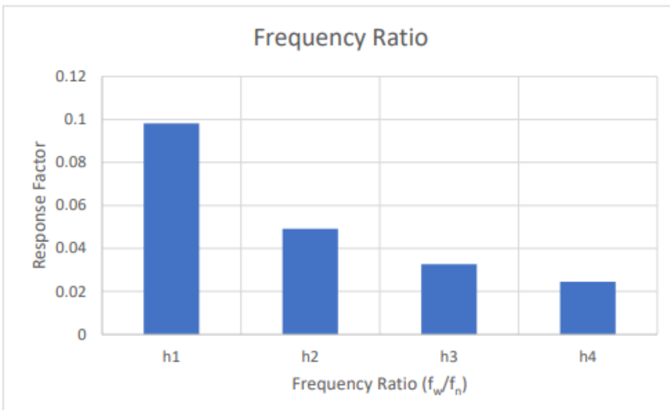
RESULTS & RECOMMENDATIONS

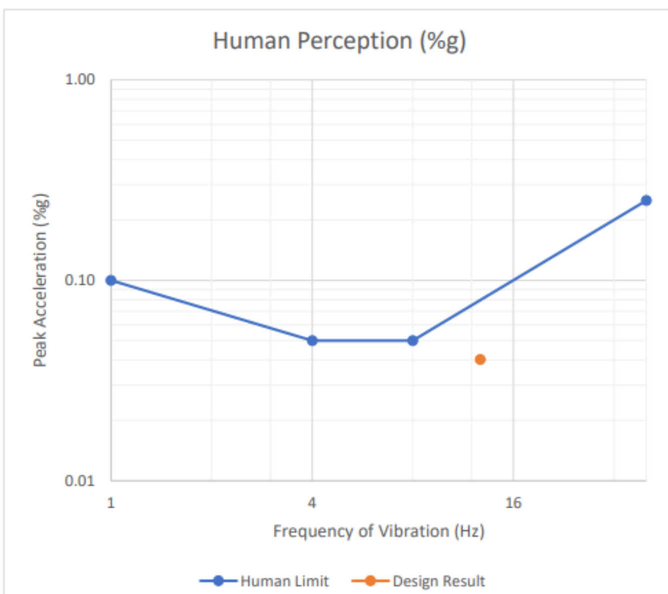
Summary & Goals

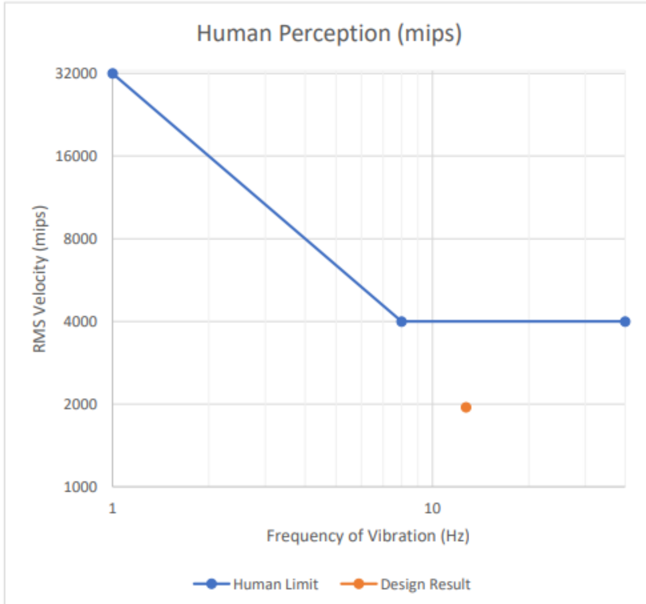
Occupancy	Laboratories or surgical theaters	
Peak Acceleration	0.3	% g
RMS Velocity	8000	mips
Supporting System	Beam-supporting	
Concrete Topping	5	in
Spanning	Two span continuous	

Results

Floor Frequency	12.73	Hz
Peak Acceleration	0.04	% g
RMS Velocity	1,946.20	mips







Recommendations

To lower acceleration,

Increase damping ratio

Add or increase concrete topping

Reduce span lengths

To increase frequency,

Reduce span lengths

Appendix F: RFQ

Quantity Takeoff Sheet

Columns		
Dimension	Length	Quantity
11.5"x11.5"	14'	105
11.5"x11.5"	14'	105
9.5"x9.5"	14'	105
8.5"x8.5"	14'	105
8.5"x8.5"	14'	105
5.375"x5.375"	18'	6

Glulam (Joist)			
Length	9.5"x23.5"	11.5"x25.5"	13.625"x15.5"
	Quantity	Quantity	Quantity
21.35'	172	43	
30'			1

Glulam (Girders)		
Length	9.5"x27.5"	9.5"x23.5"
	Quantity	Quantity
8'	1	4
8'-6"	1	4
11'	1	4
11'-6"	1	4
14'	1	4
15'	2	8
16'-6"	2	8
17'	1	4
18'-6"	1	4
19'	1	4
19'-6"	1	4
20'-6"	1	4
21'-6"	78	312
23'-6"	1	4
24'	1	4
24'-6"	2	8
26'-6"	1	4
65'	19	76
Total	116	464

CLT (175-5S)	
Dimension	Quantity
43'x8'	320
43'x2'	20
22'x8'	80
22'x2'	5
35.5'x8'	30
35.5'x2'	5
32'x8'	15
32'x6'	5
21.5'x8'	30
21.5'x2'	5
Total	515

CLT (143-5S)	
Dimension	Quantity
30'x8'	5

CLT (197-7S)	
Dimension	Quantity
14'x8'	26
14'x6'	1

		Columns	Joist	Girders	CLT - Floor	CLT - Wall
Floor 1	Quantity	105	43	116	103	8
	Member	11.5"x11.5"x14'	9.5"x23.5"x21.35'	9.5"x23.5"	175-5S	193-7S, 62'
Floor 2	Quantity	105	43	116	103	7
	Member	11.5"x11.5"x14'	9.5"x23.5"x21.35'	9.5"x23.5"	175-5S	193-7S, 52'
Floor 3	Quantity	105	43	116	103	5
	Member	9.5"x9.5"x14'	9.5"x23.5"x21.35'	9.5"x23.5"	175-5S	193-7S, 40'
Floor 4	Quantity	105	43	116	103	4
	Member	8.5"x8.5"x14'	9.5"x23.5"x21.35'	9.5"x23.5"	175-5S	193-7S, 28'
Roof	Quantity	105	43	116	103	2
	Member	8.5"x8.5"x14'	11.5"x25.5"x21.35'	9.5"x27.5"	175-5S	193-7S, 16'
Penthouse	Quantity	6	1	4	5	1
	Member	5.375"x5.375"x18'	13.625"x15.5"x30'	7.25"x15.5"	143-5S	193-7S, 6'
Total		531	216	584	520	27