Cross-Laminated Timber

A Major Qualifying Project

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Submitted to:

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Abstract

Mass timber is a framing category that uses large wood panels, including CLT. The goal was to explore the effectiveness of CLT through designing a renovation of an office building utilizing mass timber and comparing it to a steel alternative. *ASCE 7-10, IBC-2015, AWC-NDS, AISC-15* references were used to ensure structurally sound designs. While the current cost of CLT is high due to a lack of manufacturers, the sustainability, manufacturability, and constructability benefits make CLT a competitive building material.

Acknowledgments

The team would like to acknowledge and thank Michael Richard of Simpson Gumpertz and Heger (SGH) for sponsoring this project and taking time to meet with the team and provide valuable feedback throughout its duration. Michael's knowledge of mass timber, CLT, and structural steel design and construction helped the team work through questions that arose throughout the project duration. He also guided them to important information and sources used throughout the project. Being able to meet with Michael allowed the team to share progress and examples of their work, allowing the team to work through specific questions or issues with ease.

The team would also like to acknowledge and thank Professor Leonard Albano for his continued support of this project. He provided expertise in the structural design of the case study building and the construction industry, making it possible for the team to complete all of the objectives of the project. Professor Albano also made himself available to answer any questions that arose throughout the project and provided guidance to allow the project to progress throughout its duration.

Authorship

Both members of the group contributed to the writing of this report as well as the project proposal and creation of tables and figures. Both members also contributed to the design calculations for both the mass timber and structural steel frame designs. In addition, editing of the report was done by both members of the group. The following details some of the leadership roles each member took on:

Isaiah Aridou: Dealt with the sustainability and environmental aspects of each design.
Olivia Hauber: Took on the cost analysis, manufacturability, and constructability of the designs.
Dealt with the acoustic and vibration evaluation.

Capstone Design Statement

To complete the Capstone Design aspect of this project, the team designed a gut renovation of a five-story office building in Boston, MA. Two designs were completed: one using cross-laminated timber (CLT) with mass timber elements, and the other using a structural steel frame with a cast-in-place concrete slab on a metal deck. The designs were analyzed and compared to help determine the effectiveness of CLT. Several real-world constraints were addressed while completing this project.

Sustainability

To address the sustainability constraint of our capstone design, the team created two designs for the interior structural system: one using CLT floor and wall panels with Glued-Laminated (glulam) beams and columns, and the other using structural steel with a cast-in-place concrete slab on a metal deck. The team focused on CLT, which is a more sustainable alternative to other building materials, such as steel or concrete.

Economics

To address the economic constraint of our capstone design, the team compared the economical differences between the two designs. The team used different cost parameters, such as the cost of the materials, manufacturing, transportation, labor, and estimated time of construction. Since there are far fewer CLT manufacturers in the United States than steel manufacturers, including the cost of the manufacturing and transportation of the materials was necessary to create a more complete comparison of the economic impact of each design alternatives.

Health and Safety

To address the health and safety constraints of the capstone design, the team addressed the safety concerns that come with the design of a multi-story office building made of mass timber or steel. To create safe and realistic designs, the team followed the guidelines for CLT and mass timber found in the *CLT Handbook*, the American National Standards Institute and APA - The Engineered Wood Association's *Standard for Performance-Rated Cross-Laminated Timber*, the

American Wood Council's (AWC) *Manual for Engineered Wood Construction*, and the AWC's *National Design Specification for Wood Construction*. The steel design followed the guidelines from the American Institute for Steel Construction's 15th edition of the *Steel Construction Manual*. Both designs also followed the requirements from the American Society of Civil Engineers' *Minimum Design Loads for Buildings and Other Structures 7-10,* the *International Building Code of 2015,* and the *International Existing Building Code of 2015* with the *Massachusetts State Building Code 780 Amendments 9th Edition*.

Ethics

The team addressed ethical concerns throughout the project. The team worked ethically throughout the project and followed the ethical guidelines put in place by the American Society of Civil Engineers. These guidelines include creating safe and sustainable structures, acting professionally and avoiding conflicts of interest, and treating everyone involved in the project fairly (American Society of Civil Engineers [ASCE], 2017).

Manufacturability and Constructability

To address the manufacturability and constructability constraints of the capstone design, the team addressed the limited knowledge and experience in the use of CLT in North America. The team used standard and readily available sections for both the mass timber and steel frame designs. The team took into account the limited number of CLT manufacturers in the United States. The team also considered the limited experience a construction team may have when working with CLT. In addition, the team made design decisions that used repetition and promoted ease of construction. To address the regulations, design factors, and structural analysis, the team referenced the *CLT Handbook*, the *International Building Code*, the *International Existing Building Code*, and the American Institute of Steel Construction's 15th edition of the *Steel Construction Manual*.

Professional Licensure Statement

Professional licensure is important and required in the Civil Engineering industry to maximize the impact one can have on their community. Only a licensed Professional Engineer (P.E.) has the ability to seal and sign off on designs, confirming that the design meets the required safety standards and will be effective for societal use.

To achieve a professional license, an aspiring Civil Engineer must first graduate from an ABET-accredited college or university. The aspiring Civil Engineer must then pass the Fundamentals of Engineering (F.E.) exam, which will allow them to become an Engineer in Training (E.I.T.). An E.I.T. must then work under the direct supervision of a P.E. for at least four years, with some states requiring longer. In some states, earning a Master's degree can shorten this working period by up to a year. After gaining the proper experience of working under a P.E., as prescribed by their state's licensing board, the E.I.T. can apply to take the Principles and Practice of Engineering (P.E.) exam. After passing the P.E. exam, the E.I.T. must also submit a portfolio to their state's licensing board in order to earn their license and seal.

In order to maintain their license, a P.E. must pay annual dues to renew it. They must work ethically and responsibly as their work will have a direct impact on their community. Achieving professional licensure will also allow a Civil Engineer to further advance their career. Many companies even require their engineers to earn their professional licensure in order to get promotions. This is because P.E.s are recognized as individuals who are trustworthy and knowledgeable about their industry. P.E.s can be easily recognized as ethical workers by potential clients and are respected by their peers in the industry.

In a gut renovation project, much like the one completed, the P.E. would oversee and ensure correct calculations throughout the project in order to ensure the safety and effectiveness of the structure. They would also ensure that all designed elements follow the guidelines and regulations put forth in all applicable building and design codes. As the Engineer of Record (EOR), the P.E. would make the final decision on the member sizes used throughout the design before sealing and signing off on the design.

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1.0 Introduction

Mass timber is a building framing category that uses large wood panels for construction (ReThink Wood, n.d., pp. 2-4). Mass timber encompasses several building materials, including nail-laminated timber (NLT), dowel-laminated timber (DLT), structural composite lumber (SCL), glued-laminated timber (GLT or glulam), and cross-laminated timber (CLT). This project's main focus was on CLT. CLT is a relatively new building material that is gaining popularity across the globe. CLT was first introduced in Europe in the 1990s and spread to North America in the early 2000s. The spread of CLT was helped by the global interest in more sustainable construction, which is one of CLT's greatest advantages, along with its construction speed. The spread and use of CLT, however, has been much slower in North America than in Europe, although popularity in the United States is increasing. This slower spread has led to fewer manufacturers in North America and less research being conducted locally to help improve the application and more widespread acceptance of this relatively new construction material. An aspect of CLT that still requires research is the acoustic and vibration performance as these areas still have many unknowns.

The goal of this project was to explore the effectiveness of CLT through a case study of a gut renovation of a five-story building in Boston, MA using CLT and mass timber elements. The building was originally constructed in 1907 to be used by the New England Confectionery Company. The existing building consisted of heavy timber with multi-wythe mass masonry exterior walls. In this study, the building was designed to be completely renovated into an office building. This case study was based in part on a project completed by Simpson Gumpertz & Heger (SGH). The four objectives that were identified to complete this case study were:

Objective 1: Establishing Alternative Solutions in CLT
Objective 2: Establishing Alternative Solutions in Steel
Objective 3: Evaluating and Comparing the Design Solutions in CLT and Structural Steel
Objective 4: Reviewing Acoustic and Vibration Design Alternatives

Two designs were completed in this case study: one using CLT with mass timber elements, and the other using structural steel with a cast-in-place concrete slab on a metal deck. This allowed for a comparison of the effectiveness of the two building materials. In addition, current research being done on acoustic and vibration performance of CLT was reviewed and analyzed to estimate how those areas would impact the mass timber design. The results of this case study allowed the effectiveness of CLT to be explored, from the design to the cost to the manufacturability of the material.

2.0 Background

Figure 2.1: CLT panel example from Structurlam Products, Ltd.

CLT is a prefabricated engineered wood panel that consists of multiple layers of laminates that are stacked in alternating directions (APA - The Engineered Wood Association [APA], n.d.a). The individual layers, also known as plies, of CLT can be bonded together with a structural adhesive or metal fasteners. An odd number of layers is typical, with 3, 5, and 7 layers being the most common, although even layered panels do exist. Using an odd number of layers, or plies, creates a direction of greater strength for specific applications, i.e. floors, roofs, or walls (Evans, n.d.). CLT is a relatively new construction material, with its first introduction being in Austria, Germany, and Switzerland in the 1990s and it spread across Europe by the early 2000s (Greenspec, n.d.; North Carolina State University [NC State], n.d.). Although CLT was also introduced in North America in the early 2000s, its spread and use in North America has been much slower than in Europe (Pei et al., 2016). By 2016, there were 13 CLT manufacturers across Europe with a projection of 17 manufacturers by the end of 2020 (Ebner, 2017). In contrast, there were only eight manufacturers in North America by 2019 (six of which have locations in the United States) with the hope of an additional Canadian manufacturer by 2020 (Golenda, 2019; Sorensen, 2019).

The slower spread of CLT in North America could be attributed to this lack of local manufacturers, which has led to higher cost premiums. Another factor that may have slowed the spread of CLT across North America, and specifically in the United States, was that CLT was not recognized by the *International Building Code* (*IBC*) or *National Design Specification* (*NDS*) until 2015 (Koch & Kam-Biron, 2020; Laguarda Mallo & Espinoza, 2014). This very late addition can be attributed to building code limitations and the challenge of meeting structural capabilities for large wood buildings. Now, with codes being changed to accommodate new technology, wood structures can be permitted to reach greater heights than before (Coats &

Richardson, 2013) . Since CLT is a newer building material, there are still many unknowns, leading to questions about its effectiveness in comparison to other building materials, such as steel.

2.1 Mass Timber

Mass timber is a building framing category that uses large wood panels and members for floor, roof, and wall construction (ReThink Wood, n.d., pp. 2-4). Mass timber encompasses several building materials, including nail-laminated timber (NLT), dowel-laminated timber (DLT), structural composite lumber (SCL), glued-laminated timber (GLT or glulam), and cross-laminated timber (CLT). Each of these mass timber options include several layered wood panels, but they differ in the ways the panels are orientated and held together. NLT, for example, uses nails and screws to bind individual timber members together while DLT is held together with dowels. Both CLT and glulam can be glued together with durable and moisture resistant adhesive; CLT, however, is unique in having the panels orientated in alternating, perpendicular directions, which allows for two-way spanning.

2.2 The Advantages and Disadvantages of CLT

One of the biggest disadvantages of CLT in North America has been its late introduction to the continent. With less time for CLT to establish itself in North America, there is a lack of tenured CLT manufacturers, raising the issues of time and cost when working on CLT buildings within the United States. Another looming disadvantage is the lack of data within commercial construction supporting the life cycle of CLT and the claim that along with mass timber elements they can both be a major climate change solution (Robbins, 2019). Beverly Law, a professor of global change biology and terrestrial systems science at Oregon State University, recognizes the lack of analysis of carbon emitted by mass timber production since it is a huge and complex task to assess the factors of CO_2 produced in forest ecosystems as well as in production (Robbins, 2019).

A great advantage for CLT is its application in building construction ranging from public to institutional use to even multifamily buildings (ReThink Wood, n.d.). In the case of school buildings, CLT is especially helpful due to its prefabricated state when fitting a project into a time frame as short as the summer when students are away from school and still being able to finish within the timeframe. This shows how valuable CLT can be for projects of all sizes as efficiency in erection time can help reduce the overall project duration. As of 2018, there has been a looming boom for CLT manufacturing in the U.S. with four factories in production (two of which are making architectural CLT), five factories coming online, and three more announced across eight states (Jenkins, 2018).

A great example of mass timber construction in North America can be seen at the University of British Columbia with the Brock Commons building. This is an 18-story tall wood hybrid building, with 17 of those stories comprised of mass timber. The wood structure was completed in less than 70 days after the prefabricated components had arrived on site, which was four months faster than a project of a similar size (Think Wood, 2020). In terms of environmental impact estimated by the Wood Carbon Calculator for Buildings, based on research by Sathre, R. and J. O'Connor, the avoided and sequestered greenhouse gases from the wood used in the building is equal to removing 511 cars off the road for a year, and the total amount of carbon dioxide avoided by using wood products over other materials in the building is equivalent to 2,432 metric tons (Think Wood, 2020).

CLT has developed a criteria, or "sweet spot," for projects where if three of the five conditions are met, then CLT should be strongly considered (Morrow, 2018). These five conditions are: labor costs, labor scarcity, Anti-Terrorism Force Protection (ATFP) Standoff, high foundation costs, and schedule constraints. CLT construction can be found to be cost competitive for building projects between six and 14 stories and at its most optimal for construction between eight to 12 stories (Schmitt, 2020).

2.3 Sustainability and Forestry

In recent years the need for green building materials has become a growing concern due to the rapid changing of Earth's climate. A good example of CLT's growing popularity and application to sustainability can be seen from the U.S Department of Defense's use of CLT for its on-base housing due to its general resilience and resistance to explosive forces (Jenkins, 2018). The

Mjøstårnet, located in Brumunddal, Norway, is an example proving modern tall buildings can be built with green sustainable materials (Moelven, 2019). This Norwegian constructed building stands at 280 feet (85.4 meters) tall with 37,073 square feet (11,300 square meters) of space and boasts a hotel, apartments, offices, a restaurant, common areas, and even a swimming hall (Moelven, 2019). This high-rise structure showcases how capable and versatile CLT can be in place of typical materials such as steel and concrete. However CLT is currently a more optimal option when used in the six to 12 story range (Morrow, 2018).

From an environmental standpoint, CLT has been viewed very positively because it can be seen as a solution to reducing carbon emissions (Sierra Club, 2019). Concrete, for example, is one of the most highly used substances on the planet, second only to water, and is responsible for eight percent of global CO_2 emissions (Sierra Club, 2019). CLT can be seen as the rationale substitution to a building material such as concrete to help reduce a building's embodied carbon. The Life Cycle Assessment (LCA) is a tool used to assess environmental impacts and resources associated with a product’s life cycle, from raw material acquisition, via production and use phases, to waste management (Finnveden et al. 2009). Embodied carbon measures emissions from extraction, manufacturing, transporting, and the use of a building material. Combined, these emissions account for 11 percent of total carbon emissions globally using a life cycle assessment.

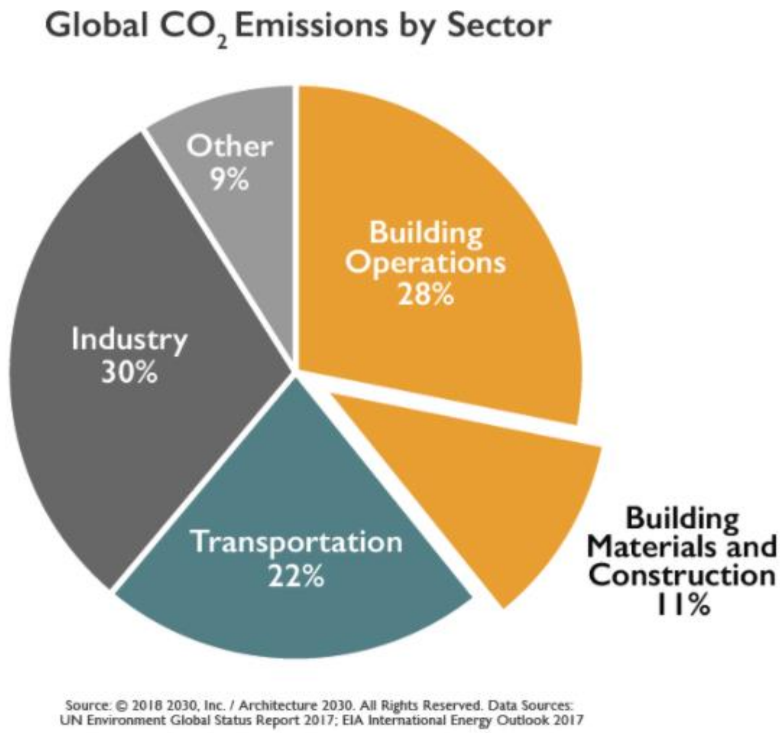


Figure 2.2: Global CO2 Emissions by Sector provided by Architecture 2030

While reducing carbon emissions by using CLT may be the hope, the need for timber will only rise with CLT's popularity and, if not managed properly, could lead to the deforestation of forests that store large amounts of carbon. As promoted by the Sierra Club to effectively counter this issue, proper forest stewardship and protection must be used. This is why the Sierra Club is in support of the protection of public lands to ensure the safety of primary forest while also allowing younger forest degraded by past logging to recover.

2.4 The Need for Research into the Acoustic and Vibration Performance of CLT

Due to CLT being a relatively new construction material not only in North America but also globally, there are quite a few areas that still require research to improve its performance. One such area is the acoustic and vibration performance. At present, the acoustic performance of CLT alone is not adequate. For acoustic performance in buildings, the mass of the building elements plays a key role in reducing sound transmission between the rooms and floors. Unfortunately, CLT's advantage of being a lighter material becomes a disadvantage when it comes to acoustics

(Preager, 2019). Due to CLT's higher strength-to-weight ratio and lower density in comparison to typical concrete slabs or masonry walls, the acoustic separation between rooms and floors in CLT buildings is worse than buildings that use these traditional materials. The acoustic separation of CLT structures also does not currently meet the *IBC* requirements on their own, with CLT having a sound transmission class (STC) of approximately 40 when the *IBC* requires an STC of at least 50 (Metropolitan Acoustics, 2019; Preager, 2019; The International Code Council [ICC], 2015). In order to comply with the *IBC*, additional barriers, such as a gypcrete topping or decouplers, are typically used to enhance the acoustic properties of CLT (McLain, 2019).

In hopes of improving the acoustic properties of CLT, research has been, and continues to be, conducted worldwide. In 2016, Antonio Di Bella, Nicola Granzotto, and Luca Barbaresi conducted an experiment to identify a spectrum of the normal impact sound pressure level of a CLT floor in order to create a tool that allows estimations of the noise insulation of a CLT floor (Di Bella et al., 2016). In 2013, Mariana Perez and Marta Fuente conducted research on a two-story experimental facility to create a predictive model of the acoustic behavior of CLT structures (Perez & Fuente, 2013). These studies, along with other research being conducted, look to better understand acoustic performance in relation to CLT and how the design of CLT can be adjusted to improve its acoustic properties.

Research is also being conducted into the vibration and seismic resistance of CLT structures. Traditional, lightweight joisted wood flooring systems are typically smaller and lighter than CLT floors, while typical concrete slabs are heavier and larger. This indicates that the fundamental frequency of CLT should be between the fundamental frequency of lightweight floors of greater than 15 Hz and the fundamental frequency of concrete slabs of less than nine Hz, which was confirmed through tests run by FPInnovations (Hu & Gagnon, 2012; Pirvu, 2015). Based on CLT's fundamental frequency being between the fundamental frequency of lightweight floors and concrete slabs, it has been determined that the current standards for the vibration design of lightweight and heavy floors are not adequate for CLT floors. This has led many to conduct research on how to design CLT floors for vibration performance.

Research is also being conducted into the seismic resistance of CLT. CLT has been increasingly used for floor diaphragms and shear walls to provide better seismic resistance for buildings. Due to this, research is being conducted globally to determine how CLT can be used to strengthen new and existing structures against seismic activity. In 2012, Lin Hu and Sylvain Gagnon conducted research to better predict the vibration performance of CLT floors as the existing design methods for lightweight and heavy floors are not applicable to CLT floors (Hu & Gagnon, 2012). Through this study, a new design method for floor vibrations was created for CLT floors, which can be used to provide better vibration and seismic performance within CLT structures. Other research, however, has found that there are currently too many unknowns with CLT since it is a relatively new building material, indicating that more research is needed into CLT as a material and its relation to seismic resistance.

2.5 Design Standards and Specifications

The introduction of CLT in North America has led to its inclusion in several engineering publications and building codes that were used throughout this project. These include the *CLT Handbook*, the American National Standards Institute (ANSI) and APA - The Engineered Wood Association's (APA) *Standard for Performance-Rated Cross-Laminated Timber* (*ANSI-APA PRG*), the American Wood Council's (AWC) *Manual for Engineered Wood Construction* (*AWC-2018*), and the AWC's *NDS for Wood Construction* (*AWC-NDS*). The design requirements of steel were referenced from the American Institute for Steel Construction's (AISC) 15th edition of the *Steel Construction Manual* (*AISC-15*). Both designs also referred to the American Society of Civil Engineers' *Minimum Design Loads for Buildings and Other Structures* (*ASCE 7-10*), the *IBC* of 2015 (*IBC-2015*), and the *International Existing Building Code* (IEBC) of 2015 (*IEBC-2015*) for the design requirements of CLT and the building codes as well as the *Massachusetts State Building Code 780 Amendments 9th Edition Chapter 16 Structural Design Amendments* (*780 CMR 16*) for applicable local requirements.

2.5.1 Seismic Design

With the increase of interest in CLT construction over the years, multiple countries have begun adopting provisions for CLT into their design standards. However, due to legal differences and differences in economics between regions some fundamental issues are addressed differently, and one of these particular issues is seismic design (Tannert et al 2018). The applicable seismic response modification factors for the United States range from R = 3 to 3.5 depending on the results of FEMA P695 analysis (Tannert et al 2018). For the seismic design of steel R = 3 is used when the structural steel system is not specifically detailed for seismic resistance, which considers the fact that Massachusetts is not prone to frequent earthquakes and composite steel is being used for the building (Hamburger, 2009).

2.5.2 CLT Manufacturers

As previously mentioned, there were nine CLT manufacturers operating in North America as of 2019. A summary of these manufacturers can be found in Table 2.5.2. In contrast, there were 60 steel manufacturers in operation in 2018 in the United States alone ("Steel companies of the United States", 2018).

Table 2.5.2: CLT Manufacturers

Manufacturer	**Location(s)**	**Website**
Dr Johnson Wood Innovations	Riddle, Oregon, United States	https://drjlumber.com/
Element5 Co.	Toronto, Ontario, Canada Montréal, Québec, Canada Ripon, Québec, Canada	https://elementfive.co/
Freres Lumber Co., Inc.	Lyons, Oregon, United States Mill City, Oregon, United States	https://frereslumber.com/
Nordic Structures	Montréal, Québec, Canada	https://www.nordic.ca/en/home
Sterling Solutions	Phoenix, Illinois, United States Lufkin, Texas, United States	https://www.sterlingsolutions.com/
StructureCraft	Abbotsford, British Columbia, Canada	https://structurecraft.com/
StructurLam	Penticton, British Columbia, Canada Vancouver, British Columbia,	https://www.structurlam.com/

	Canada Portland, Oregon, United States Granite Bay, California, United States Austin, Texas, United States	
Western Structures, Inc.	Veneta, Oregon, United States	https://westernstructures.com/

While the number of CLT manufacturers in North America is growing, there seems to be three major areas where these manufacturers are: British Columbia, Québec, and Oregon. The different manufacturers each have information on their websites regarding the products they offer and the projects in which they have been involved. Some of the manufacturers' websites, such as Nordic Structures (Nordic) and Structurlam, also include product catalogs detailing the typical member sizes that can be produced. None of these websites, however, include pricing information for their products. Instead, contact and quotes pages are used to allow owners, designers, or contractors to begin working with the manufacturer on their project.

3.0 Methodology

The goal of this project was to explore the effectiveness of CLT through a case study of a gut renovation of a five-story building in Boston, MA using CLT and mass timber elements and comparing the design to a structural steel frame with a cast-in-place concrete slab on a metal deck. The team designed for the building to be completely renovated into an office building. This case study was based on a project completed by SGH. The four objectives that were identified to complete this case study are:

Objective 1: Establish Alternative Solutions in CLT
Objective 2: Establish Alternative Solutions in Steel
Objective 3: Evaluate and Compare Design Solutions in CLT and Structural Steel
Objective 4: Review Acoustic and Vibration Design Alternatives

3.1 Objective 1: Establishing Alternative Solutions in CLT

Objective 1 was used to establish a CLT renovation with mass timber elements of the case study building based on the floor plans provided by SGH. A breakdown of each level in the floor plan was conducted to address the design of each floor. Two heavy-timber elements were chosen for the design process: CLT was used in the design of the floor and walls while glulam was used in the design of the beams and columns. Design calculations for the mass timber building included the gravitational loads of the building, including the self-weight; lateral load resistance; and a load takedown for the existing masonry exterior of the building. All floors but the roof were designed using five-ply CLT panels, while the roof system used three-ply panels. Some CLT member lengths were rounded up to the nearest ⅛ of an inch due to potential discrepancies found from manual measurements within the floor plan.

References, such as the *AWC NDS* of 2018, the *CLT Handbook*, and *ASCE 7-10*, as well as *IBC* and *IEBC* of 2015, were used during the design process to ensure design factors and code requirements were being followed. Google Sheets for each floor were created in order to assist in the repetitive design calculations.

3.2 Objective 2: Establishing Alternative Solutions in Steel

Objective 2 establishes an alternative design of a steel frame renovation with a cast-in-place concrete slab on a metal deck for the case study building in conjunction with the floor plans provided by SGH. The design used a series of composite structural steel beams supported by wide-flange columns for each individual floor. Design calculations for the steel frame included gravitational loads of the building, including self-weight, and lateral load resistance.

References including *AISC-15*, *ASCE 7-10* along with the *IBC* and *IEBC* of 2015 were used during the design process to ensure design factors and code requirements were being followed. Google Sheets for each floor were created in order to assist in the design calculations.

3.3 Objective 3: Evaluate and Compare the Design Solutions in CLT and Structural Steel

Once the designs of both the mass timber and steel frame renovations were completed, the team moved on to Objective 3 and reviewed the two designs to evaluate and compare a cost analysis of each design as well as the manufacturability and constructability of each approach to determine the more effective design of the two. The unit cost of the CLT members was calculated from information provided by Nordic through the sponsor of the project. The unit cost of the steel and glulam members were found through the RSMeans publications *Assemblies Costs* and *Building Construction Costs*. To evaluate both the CLT and steel design's manufacturability and constructability the team established a set of criteria, looking at whether similar members of the material could be obtained, if the members were readily available, and which of the fabrication processes would be more efficient.

3.4 Objective 4: Assess Acoustic and Vibration Design Alternatives

For Objective 4 the team looked into case studies and current research of CLT structures in relation to acoustic and vibration performance and their potential impact on the mass timber design. The team investigated the design for acoustics and vibrations for CLT based on design examples and reference calculations within these studies.

4.0 The Case Study Building

Figure 4.1: Exterior of the Case Study Building

In order to explore the effectiveness of using CLT for a building structure, a case study of a gut renovation of a five-story building in the seaport district of Boston, MA was used. The building was originally constructed out of heavy timber with multi-wythe mass masonry exterior walls in 1907 to be used as a factory by the New England Confectionery Company. The building was constructed to be almost symmetrical in an "H" shape. A typical floor plan can be seen in Figure 4.3.

Figure 4.2: Interior of the Case Study Building

This case study involved designing the building for a complete renovation into an office space. The new office space included the existing masonry exterior, five masonry staircases, and three central elevator shafts. A new lobby was attached to the existing structure, and each occupied floor would now contain office spaces. The current floor plans for the existing building can be found in Appendix B.

The design of this renovation was completed twice. The first design used mass timber, utilizing CLT walls and floors and glulam beams and columns. The second design was of a steel frame with a cast-in-place concrete slab on a metal deck. This provided a comparison between CLT and a common construction material that is widely used in construction in Boston, which allowed the relative effectiveness of CLT to be analyzed. In both designs, the existing structure in the central portion of the building remained, meaning no renovations were made in that area. In order to determine the seismic loadings on the building in both the mass timber and steel frame designs, a seismic weight per floor was needed. This seismic weight per floor included the weight of the existing exterior mass masonry walls, which would remain constant between both designs. The weight of the mass masonry walls was 34 kips per floor.

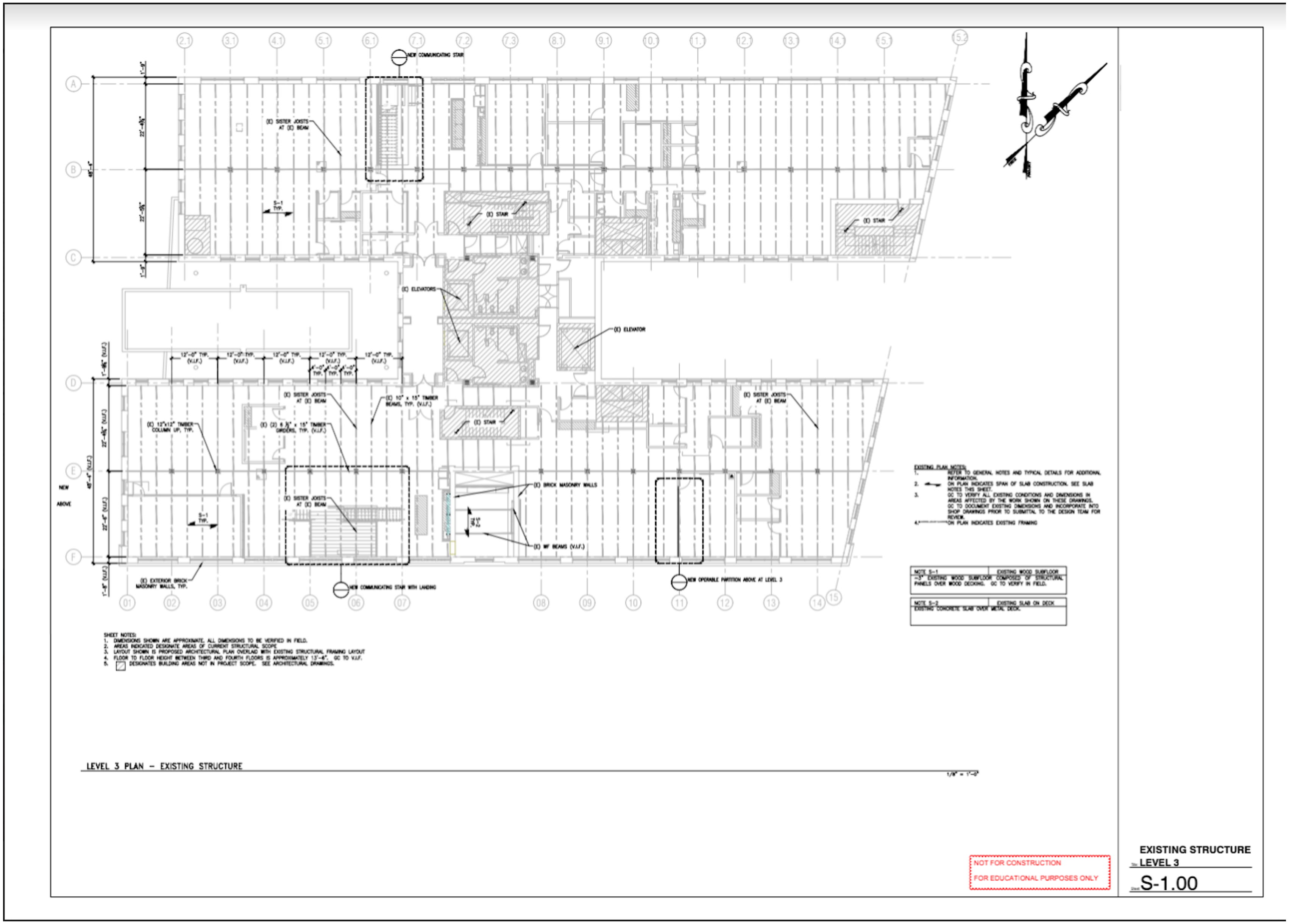


Figure 4.3: Typical Floor Plan

5.0 Mass Timber Design

The CLT and glulam design were completed using the Allowable Stress Design (ASD) practice. The use of both heavy timber elements results in a hybrid structural system that assists in addressing the limitation of space flexibility by using CLT for the floors and walls while the glulam is used for the columns and beams (Liu, 2016). Nordic in Québec, Canada was the chosen manufacturer for the mass timber design as they were the closest manufacturer to Boston that produces both CLT and glulam, and this firm has a history of supplying these materials for construction projects in the New England area. Since the building is fairly symmetrical, the design process was simplified. The design consisted of 90 beams along the column lines and 28 columns per occupied floor, and 150 beams (90 beams along the column lines and 60 infill beams) and 28 columns for the roof level. Each floor had an area of approximately 18,600 square feet. The attached lobby consisted of four beams with an approximate area of 1,800 square feet. Each floor was 13 ½ feet in height for a total building height of 81 feet. The framing layout for each floor was determined using the floor plans provided by SGH, which can be seen in Appendix B.

The mass timber framing system provides resistance to the gravity loads (dead loads, live loads, and snow loads) by allowing the loads to transfer from member to member, with each member providing adequate support. The gravity loads begin at the roof, with the roof beams, walls, and floors transferring the loads through the columns to the 4th floor, where the loads are once again transferred to the beams, walls, and floors. This process continues until all of the load is transferred to the foundation of the building. In order to provide adequate resistance to the lateral loads (seismic loads and wind loads), shear walls were incorporated. These walls prevent individual members, and therefore the building, from deflecting, or swaying, in a horizontal direction. The shear walls also allow these lateral loads to be transferred down the building to the foundation.

5.1 Loadings Considered in the Mass Timber Design

There were five load types considered in the design: dead loads, live loads, snow loads, seismic loads, and wind loads, and these are listed in Table 5.1.1. The weight of the CLT floors along with the weight of mechanical, electrical and plumbing (MEP) systems and hung ceilings and finishes were considered when calculating the dead loads. After a beam size was chosen, the self-weight of the beam was also added to the existing dead load. The live loads were determined using Table 4-1 in *ASCE 7-10* (American Society of Civil Engineers [ASCE] & Structural Engineering Institute, 2010). The 80 pounds per square foot for corridors was used only for members that were completely within a corridor space. If a member supported both an office space and a corridor, the 100 pounds per square foot load for a Class A office space was used in order to design for the highest possible load. Class A office spaces are newer spaces that are designed to have a high quality infrastructure (Golden, 2016). The snow loads were determined using the provisions of *780 CMR 16* (Office of Public Safety and Inspections, 2017).

The seismic loads were determined according to the provisions of *780 CMR 16* and *ASCE 7-10* (Office of Public Safety and Inspections, 2017; ASCE & Structural Engineering Institute, 2010). A seismic analysis spreadsheet was used to simplify the seismic loading calculations (ICC, 2012). This spreadsheet used the seismic risk category of the building, the soil classification of the site, local seismic data, and the weight of the building elements to determine the seismic loadings. The seismic forces were converted from the story forces in kips obtained from the spreadsheet to pounds per foot of building width. The building was determined to be in seismic risk category II, and the unknown soil was classified as site class D. The weight of the designed structural members as well as the existing exterior masonry wall were used when determining the seismic loadings on the building. Since the height of each floor is considered when determining the seismic loadings, each floor had a slightly different seismic load with the higher floors having slightly higher loads. This is due to the distribution of story forces that roughly conforms to the first mode shape for the building, making the building's horizontal deflection act similarly as it would to a cantilever beam up from the building's foundation (Murty et al., n.d.). The difference in loading, however, was minimal. Since the roof level had smaller, lighter structural members, especially with a three-ply CLT floor versus a five-ply CLT floor on the other floors in the building, the seismic loading for the roof level was smaller than the floors below. The seismic loading calculations from the seismic analysis spreadsheet can be found in Figures 5.1.1 and 5.1.2.

The wind loads were determined by using the requirements of *780 CMR 16* and *ASCE 7*-10 (Office of Public Safety and Inspections, 2017; ASCE & Structural Engineering Institute, 2010). A wind loadings spreadsheet was also used to simplify the calculations of the wind loadings (FLSmidth, n.d.). This spreadsheet used the risk category of the building, local wind speed data, and wind uplift forces to determine the wind loadings on the building. The wind loadings were represented as pounds per square foot of exposed wall area. Similar to the seismic loads, the wind loads are different on each floor with higher floors having larger loads since wind speed increases with height. The wind loading calculations from the wind loads spreadsheet can be found in Figure 5.1.3.

Table 5.1.1 Loads Considered in the Mass Timber Design

Load Type	Load	Elements Considered
Dead Load	25.6 psf for lobby and ground floor through 4th floor	MEP, Self-weight of CLT floors
	17.4 psf for the roof	
Live Loads	100 psf for lobby and ground floor	Lobbies and first-floor corridors
	80 psf for corridors on 1st, 2nd, 3rd, and 4th floors	Corridors above the first floor
	100 psf for office spaces	Offices
	20 psf for the roof	Roof
Snow Load	40 psf for ground	Snow load from Massachusetts Structural Design Amendments
	30 psf for the roof	
Seismic Loads	14.2 plf for lobby and ground floor	Seismic parameters from Massachusetts Structural

<table>
<tr><td rowspan="5"></td><td>14.2 plf for 1st floor</td><td rowspan="5">Design Amendments, risk category II, soil site class D</td></tr>
<tr><td>14.2 plf for 2nd floor</td></tr>
<tr><td>14.2 plf for 3rd floor</td></tr>
<tr><td>14.2 plf for 4th floor</td></tr>
<tr><td>4.88 plf for the roof</td></tr>
<tr><td rowspan="6">Wind Load</td><td>36.5 psf for ground floor</td><td rowspan="6">Wind speeds from Massachusetts Structural Design Amendments</td></tr>
<tr><td>39.2 psf for 1st floor</td></tr>
<tr><td>41.3 psf for 2nd floor</td></tr>
<tr><td>42.9 psf for 3rd floor</td></tr>
<tr><td>44.2 psf for 4th floor</td></tr>
<tr><td>45.3 psf for the roof</td></tr>
</table>

SEISMIC BASE SHEAR AND VERTICAL SHEAR DISTRIBUTION

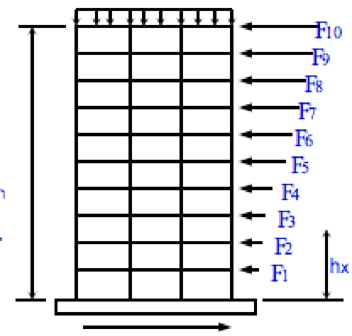
Per IBC 2012 and ASCE 7-10 Specifications

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 =	2210	
Spectral Accel., S_S =	0.217	ASCE 7-10 Figures 22-1 to 22-11
Spectral Accel., S_1 =	0.068	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 =	81.000	ft.
Actual Calc. Period, Tc =	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)



$V = Cs*W = \Sigma(Fi) = 36.48$ 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	81.000	42.20
5	67.500	81.52
4	54.000	81.52
3	40.500	81.52
2	27.000	81.52
1	13.500	84.39

Total Weight, $W = \Sigma W_x$ = 452.67 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} =	0.347	$S_{MS} = F_a*S_S$, ASCE Eqn. 11.4-1, page 65
S_{M1} =	0.163	$S_{M1} = F_v*S_1$, ASCE Eqn. 11.4-2, page 65

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

S_{DS} =	0.231	$S_{DS} = 2*S_{MS}/3$, ASCE 7-10 Eqn. 11.4-3, page 65
S_{D1} =	0.109	$S_{D1} = 2*S_{M1}/3$, ASCE Eqn. 11.4-4, page 65

(continued:)

Figure 5.1.1: Seismic Loading Data for Mass Timber Design

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 =	0.540	sec., $T_a = C_T*h_n$^(x), ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.682	ASCE 7-10 Table 12.8-1, page 90
Period max., $T_{(max)}$ =	0.908	sec., $T_{(max)} = C_u*T_a$, ASCE 7-10 Section 12.8.2, page 90
Fundamental Period, T =	0.540	sec., T = Ta <= Cu*Ta, 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 =	0.093	$C_S = S_{DS}/(R/I)$, ASCE 7-10 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.081	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 =	0.081	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V = 36.48 kips, $V = C_S*W$, ASCE 7-10 Section 12.8.1, Eqn. 12.8-1

Seismic Shear Vertical Distribution:

Distribution Exponent, k = 1.02 — 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/(ΣW_i*h_i^k), ASCE 7-10 Eqn. 12.8-12, page 91

Seismic Level x	Weight, Wx (kips)	h_x^k (ft.)	W_x*h^k (ft-kips)	C_{vx} (%)	Shear, F_x (kips)	Σ Story Shears
6	42.20	88.441	3732.2	0.173	6.31	6.31
5	81.52	73.433	5986.2	0.277	10.11	16.42
4	81.52	58.485	4767.7	0.221	8.06	24.48
3	81.52	43.612	3555.2	0.165	6.01	30.48
2	81.52	28.840	2351.0	0.109	3.97	34.45
1	84.39	14.221	1200.1	0.056	2.03	36.48
Σ =	452.67		21592.5	1.000	36.48	

Comments:

Figure 5.1.2: Seismic Loading Analysis for Mass Timber Design

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	125 ft	
Side Wall Width, L	165 ft	
L/B Ratio	1.33	
Windward Wall Coefficient, C_p	0.80	Figure 27.4-1
Leeward Wall Coefficient, C_p	-0.43	Figure 27.4-1
Side Wall Coefficient, C_p	-0.70	Figure 27.4-1

Roof Pressure Coefficients

Roof Slope, θ	9.5°	
Median Roof Height, h	81 ft	
Velocity Pressure Exposure Coef., K_h	1.21	Table 27.3-1
Velocity Pressure, q_h	43.2 psf	Equation 27.3-1
h/L Ratio	0.49	
Windward Roof Area	0 ft²	
Roof Area Within 41 ft of WW Edge	0 ft²	

<table>
<tr><th rowspan="2">Location</th><th rowspan="2">Min/Max</th><th colspan="4">Horiz Distance From Windward Edge</th><th rowspan="2"></th></tr>
<tr><th>0 ft</th><th>41 ft</th><th>81 ft</th><th>162 ft</th></tr>
<tr><td rowspan="2">Windward Roof Coefficient Normal to Ridge, C_p</td><td>Min</td><td>-0.90</td><td>-0.90</td><td>-0.50</td><td>-0.30</td><td rowspan="6">Figure 27.4-1</td></tr>
<tr><td>Max</td><td>-0.18</td><td>-0.18</td><td>-0.18</td><td>-0.18</td></tr>
<tr><td rowspan="2">Leeward Roof Coefficient Normal to Ridge, C_p</td><td>Min</td><td>-0.90</td><td>-0.90</td><td>-0.50</td><td>-0.30</td></tr>
<tr><td>Max</td><td>-0.18</td><td>-0.18</td><td>-0.18</td><td>-0.18</td></tr>
<tr><td rowspan="2">Roof Coefficient Parallel to Ridge, C_p</td><td>Min</td><td>-0.90</td><td>-0.90</td><td>-0.50</td><td>-0.30</td></tr>
<tr><td>Max</td><td>-0.18</td><td>-0.18</td><td>-0.18</td><td>-0.18</td></tr>
</table>

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

<table>
<tr><th rowspan="3">Height, z</th><th rowspan="3">K_z</th><th rowspan="3">q_z</th><th colspan="4" rowspan="2">Walls</th><th colspan="3">Roof</th><th colspan="2" rowspan="2">Internal</th></tr>
<tr><th colspan="2">Normal to Ridge</th><th rowspan="2">Parallel to Ridge</th></tr>
<tr><th>WW</th><th>LW</th><th>WW + LW</th><th>Side</th><th>WW</th><th>LW</th><th>Positive</th><th>Negative</th></tr>
<tr><td>0 ft</td><td>0.85</td><td>30.3 psf</td><td>20.6 psf</td><td rowspan="11">-16.0 psf</td><td>36.5 psf</td><td rowspan="11">-25.7 psf</td><td rowspan="5">Min: -33.0 psf</td><td rowspan="5">Min: -33.0 psf</td><td rowspan="5">Min: -33.0 psf</td><td>7.8 psf</td><td rowspan="11">-7.8 psf</td></tr>
<tr><td>14 ft</td><td>0.85</td><td>30.3 psf</td><td>20.6 psf</td><td>36.5 psf</td><td>7.8 psf</td></tr>
<tr><td>27 ft</td><td>0.96</td><td>34.3 psf</td><td>23.3 psf</td><td>39.2 psf</td><td>7.8 psf</td></tr>
<tr><td>41 ft</td><td>1.05</td><td>37.3 psf</td><td>25.4 psf</td><td>41.3 psf</td><td>7.8 psf</td></tr>
<tr><td>54 ft</td><td>1.11</td><td>39.6 psf</td><td>26.9 psf</td><td>42.9 psf</td><td>7.8 psf</td></tr>
<tr><td>68 ft</td><td>1.17</td><td>41.5 psf</td><td>28.2 psf</td><td>44.2 psf</td><td rowspan="6">Max: -6.6 psf</td><td rowspan="6">Max: -6.6 psf</td><td rowspan="6">Max: -6.6 psf</td><td>7.8 psf</td></tr>
<tr><td>81 ft</td><td>1.21</td><td>43.2 psf</td><td>29.4 psf</td><td>45.3 psf</td><td>7.8 psf</td></tr>
<tr><td>0 ft</td><td>0.85</td><td>30.3 psf</td><td>20.6 psf</td><td>36.5 psf</td><td>7.8 psf</td></tr>
<tr><td>0 ft</td><td>0.85</td><td>30.3 psf</td><td>20.6 psf</td><td>36.5 psf</td><td>7.8 psf</td></tr>
<tr><td>0 ft</td><td>0.85</td><td>30.3 psf</td><td>20.6 psf</td><td>36.5 psf</td><td>7.8 psf</td></tr>
<tr><td>0 ft</td><td>0.85</td><td>30.3 psf</td><td>20.6 psf</td><td>36.5 psf</td><td>7.8 psf</td></tr>
</table>

Figure 5.1.3: Wind Loading Analysis

5.2 Glued Laminated Timber Beam Design

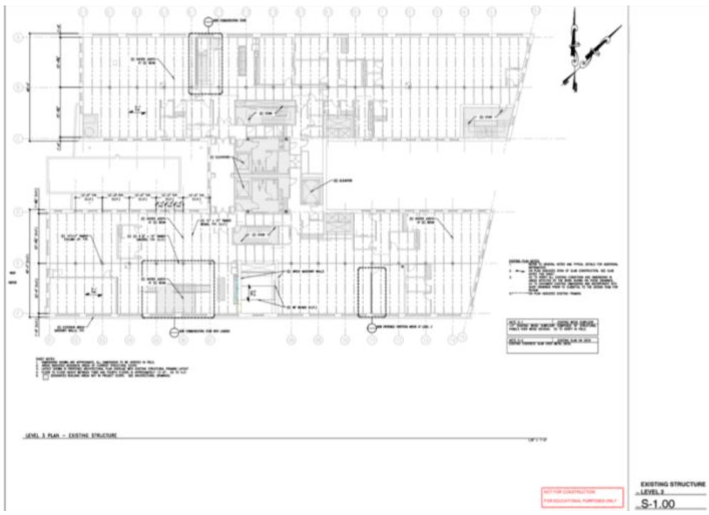


Figure 5.2.1: Typical Floor Plan

The glulam beams were designed to comply with the bending capacity, shear capacity, and deflection limits as prescribed in *ASCE 7-10*, *AWC-NDS*, *AWC-2018*, *780 CMR 16*, and all other applicable building codes and design guides (ASCE & Structural Engineering Institute, 2010; American Wood Council [AWC], 2018b; AWC, 2018a; Office of Public Safety and Inspections, 2017). An iterative process was used to determine the beam sizes to be used throughout the building. First, a beam size from Nordic's Structural Details catalog was selected that met the bending, shear, and deflection criterion (Nordic, 2020). The stress grade of the beams was 24F-ES/NPG as that is the stress grade of glulam beams that Nordic provides. The deflection criterion put forth by the *780 CMR 16* was the main determining factor for the beam sizes as the deflection could not be more than L/360 from the live load or L/240 from the live and dead load combined (Office of Public Safety and Inspections, 2017).

Once an initial beam size was chosen, the next smallest beam size was then examined as smaller and lighter structural members can provide a more economical design, with savings on fabrication, transportation, and erection. The smallest beam size that met the bending, shear, and deflection criterion was chosen, and this beam size was examined for similar beam types (ie. beams in the north-south direction, girders in the east-west direction, etc.). If the already selected beam size resulted in a highly over-designed beam or an under-designed beam, a new beam size was chosen using the same iterative process, mainly occurring with beams that were in contact with the masonry staircases. Intermittent, or infill, beams were added to the roof level to provide adequate support to the roof when combined with a three-ply CLT floor panel. The beam and girders were stacked framed as this would reduce the need for hanger connectors, reducing the cost and labor needed when compared to a flush framed system. While this design process caused some members to be slightly over-designed, having as many beams of the same size as possible would simplify the manufacturing of the beams and the renovation of the building.

The final beam design was composed of 52 17 ⅝” x 15 ⅛” beams (beam sizes are denoted as width x depth) in the north-south direction, two 8 ½” x 8” beams in contact with the staircase in the south-west corner of the building, four 7 ¼” x 6” beams in contact with the staircase in the north of the building, two 7 ¼” x 7 ⅛” beams in contact with the staircase in the north-east corner of the building, and 30 13 ⅝” x 10 ¾” girders in the east-west direction for each floor from the ground floor to the fourth floor, as seen in Figure 5.2.2. The lobby was composed of four 23 ¾” x 21 ¾” beams, as seen in Figure 5.2.3.

The roof was composed of 104 11 ½” x 9 ¾” beams in the north-south direction, four 5 ⅜” x 5” beams in contact with the staircase in the south-west corner of the building, eight 5 ⅜” x 3 ¾” beams in contact with the staircase in the north of the building, four 5 ⅜” x 4 ¼” beams in contact with the staircase in the north-east corner of the building, and 30 9 ½” x 8 ½” girders in the east-west direction, as seen in Figure 5.2.4. Google Sheets were used throughout the iterative design process to ensure correct calculations. These calculations can be found in Appendix C.

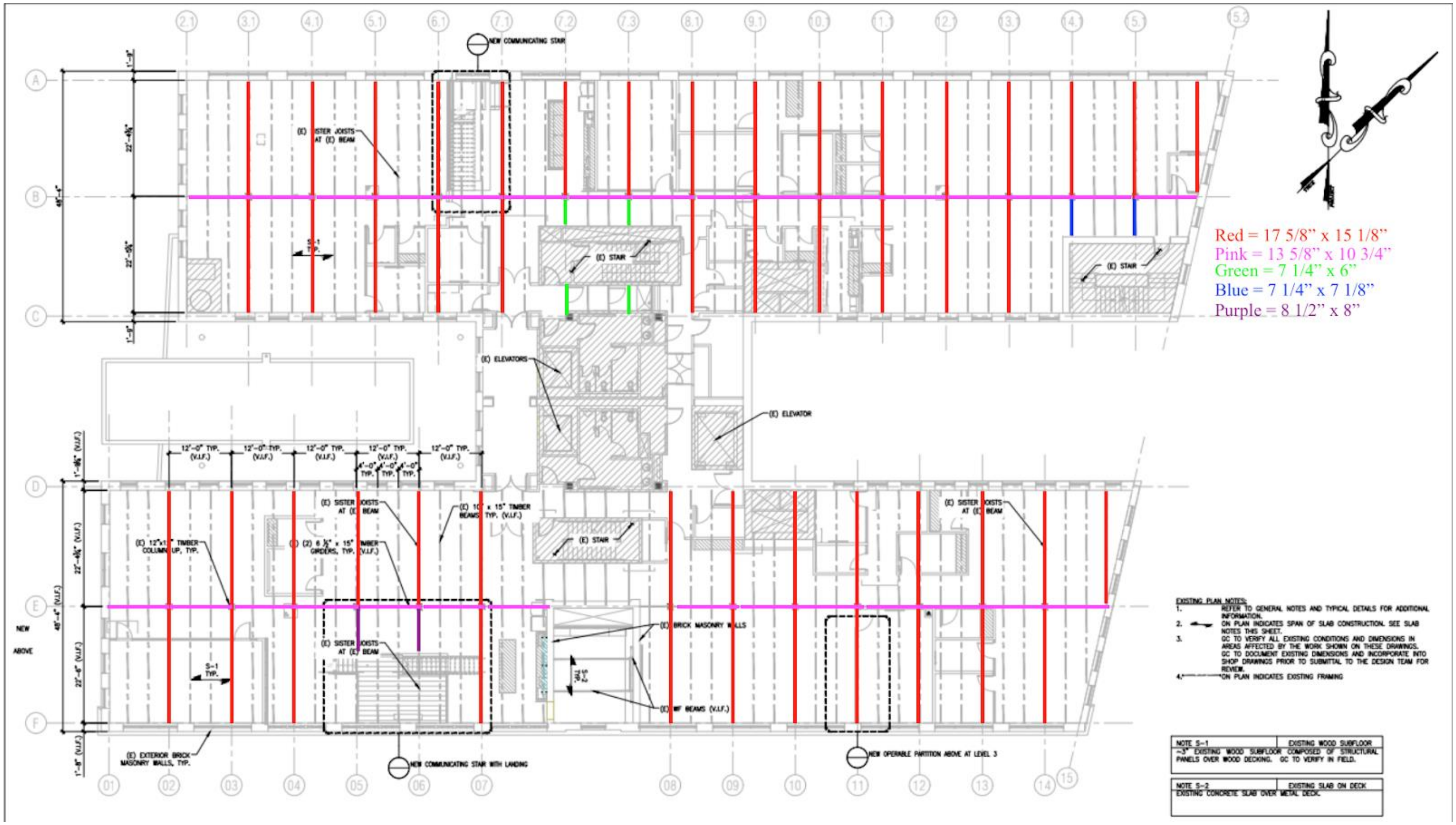


Figure 5.2.2: Ground Floor Through 4th Floor Glulam Beam Sizes

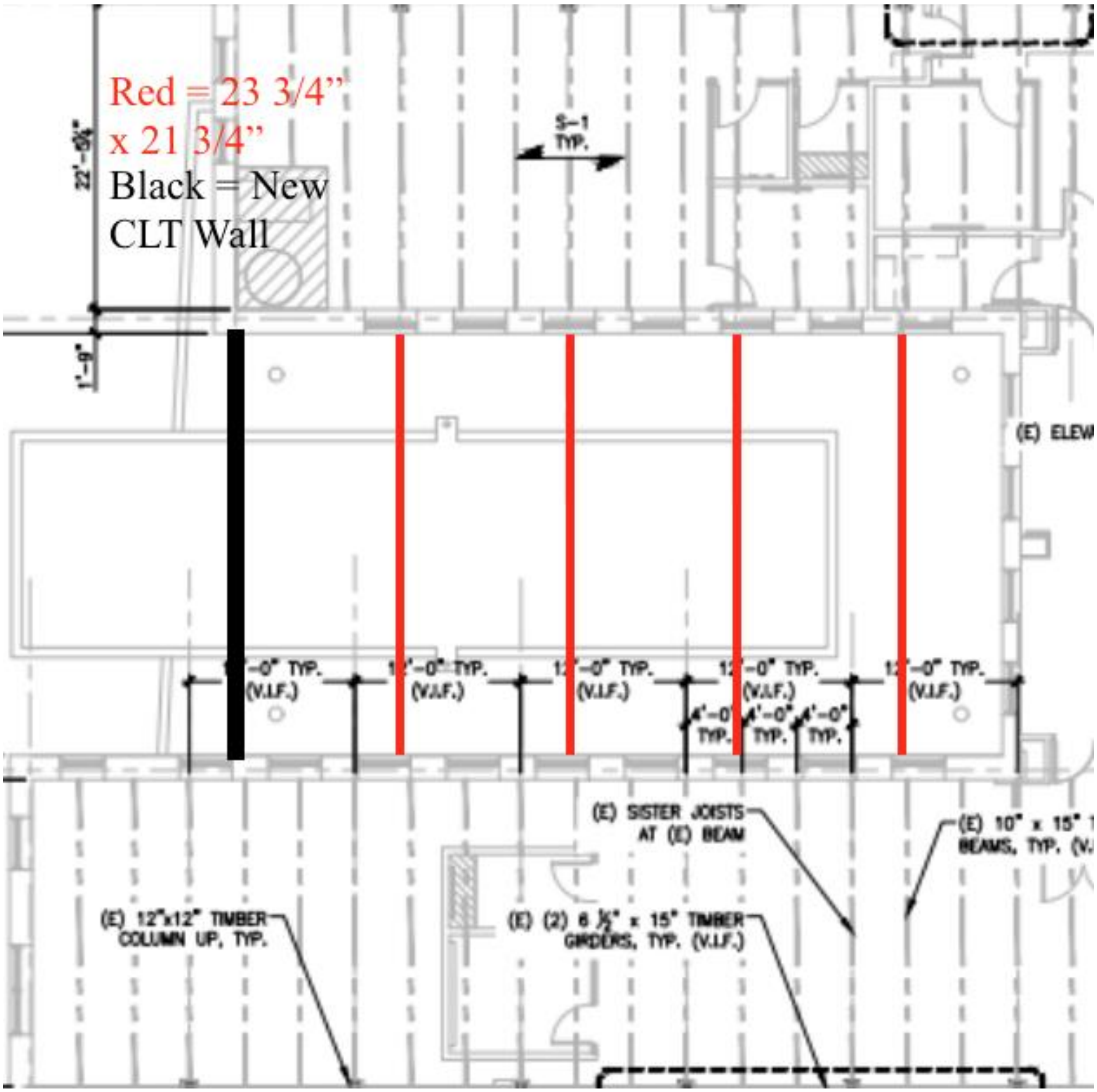


Figure 5.2.3: Attached Lobby Glulam Beam Sizes

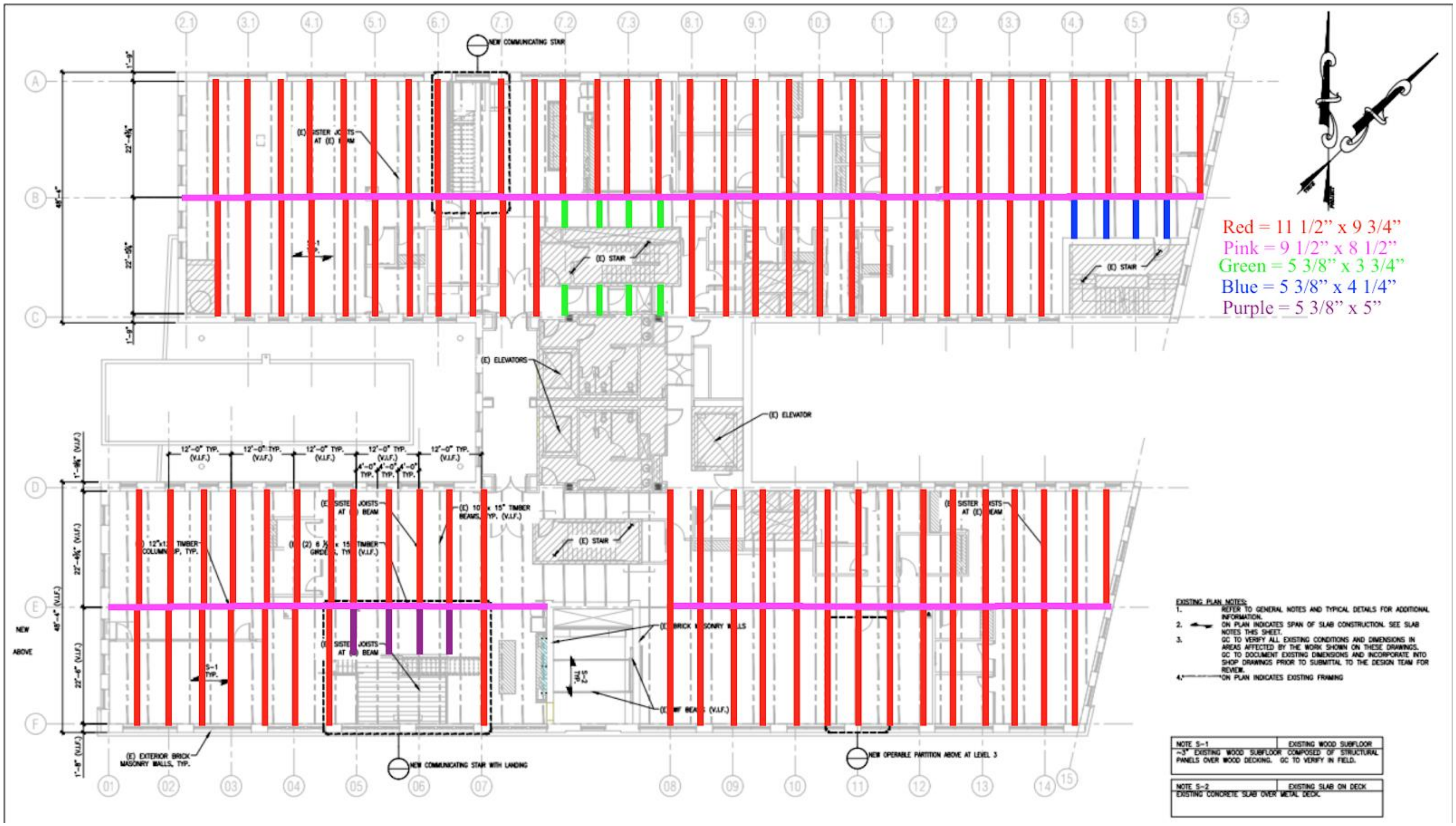


Figure 5.2.4: Roof Glulam Beam Sizes

5.3 Glued Laminated Timber Column Design

The glulam columns were designed to comply with the axial loading capacity, buckling capacity, and shear capacity as prescribed in *ASCE 7-10*, *AWC-NDS*, *AWC-2018, 780 CMR 16*, and all other applicable building codes and design guides (ASCE & Structural Engineering Institute, 2010; AWC, 2018b; AWC, 2018a; Office of Public Safety and Inspections, 2017). When determining the column sizes to be used throughout the case study building, an iterative process was used. First, a column size from Nordic's Structural Details catalog was determined that met the axial loading, buckling, and shear criterion (Nordic, 2020). The stress grade of the columns was 24F-ES/NPG as that is the stress grade of glulam columns that Nordic provides. The columns were kept square for the ease of manufacturing and constructing. Ensuring the column size could adequately support the axial loading was the main determining factor for the column sizes.

Once an initial column size was chosen, the next smallest column size was analyzed as smaller and lighter structural members can provide a more economical design. The smallest column size that met the axial loading, buckling, and shear criterion was chosen, and this column size was examined for the other columns throughout the building. If the already selected column size resulted in a highly over-designed or under-designed column, a new column size was determined using the same iterative process. This mainly occurred for the columns supporting the roof of the building. While this design process caused some columns to be slightly over-designed, having as many columns of the same size as possible would ease the manufacturing of the columns and the renovation of the building.

The final column design was composed of 28 9" x 9" columns for each floor from the ground floor to the fourth floor, as seen in Figure 5.3.1At the fifth floor level, the roof was supported by 28 8" x 8" columns, as seen in Figure 5.3.2. Google Sheets were used throughout the iterative design process to ensure correct calculations. These calculations can be found in Appendix C.

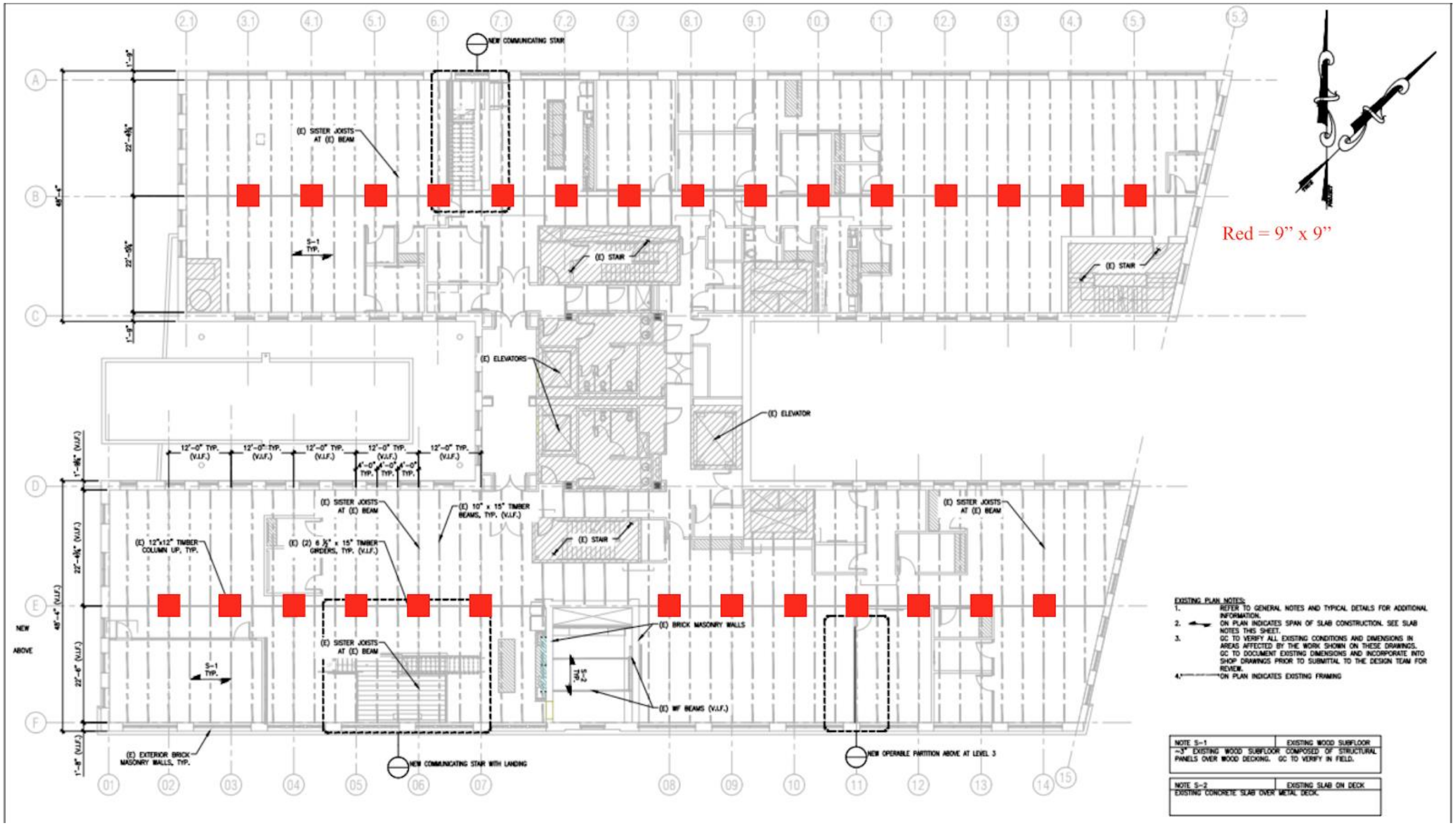


Figure 5.3.1: Ground Floor Through 4th Floor Glulam Column Sizes

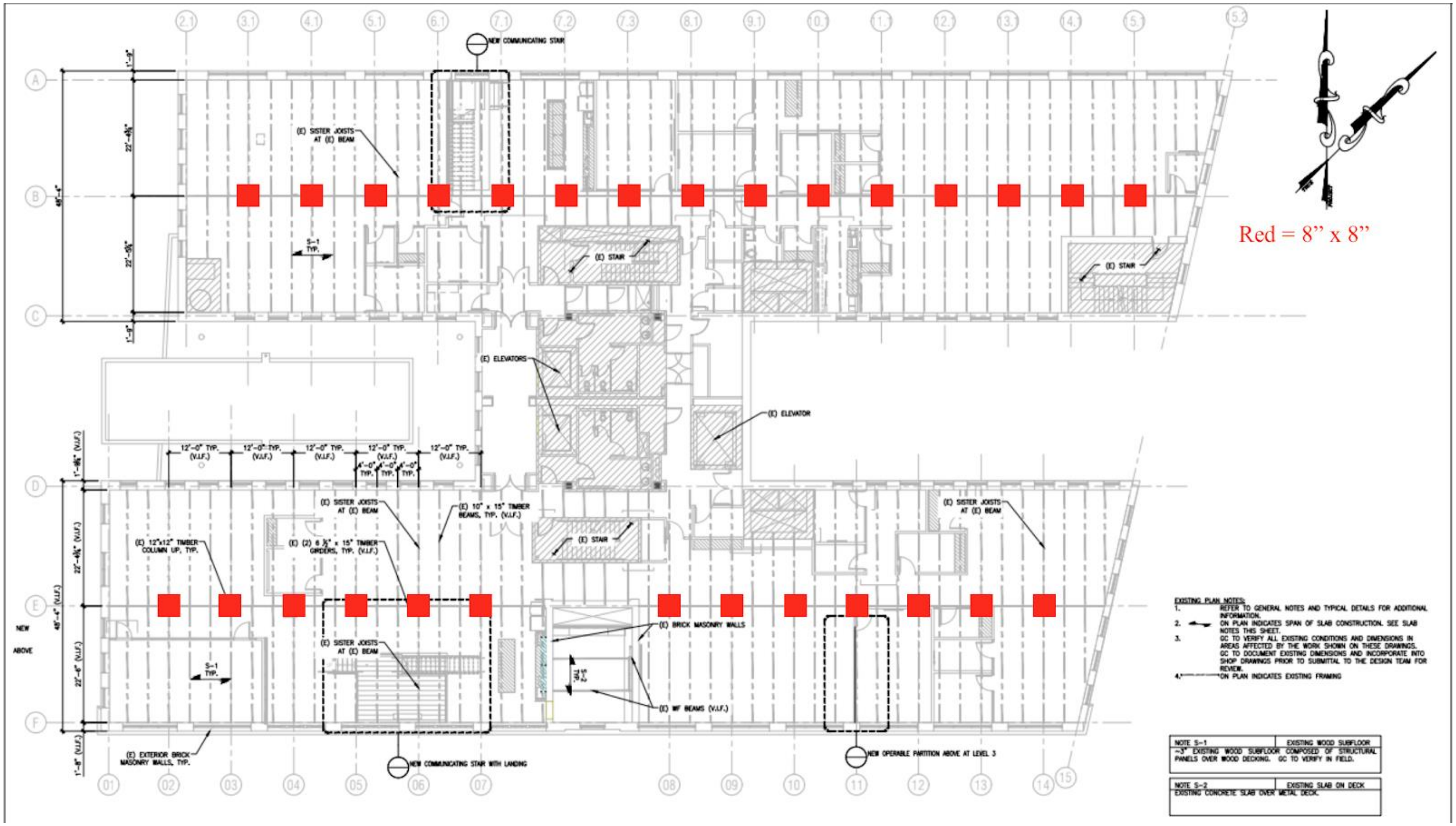


Figure 5.3.2: Roof Glulam Column Sizes

5.4 Cross-Laminated Timber Floor and Wall Design

5.4.1 Cross-Laminated Timber Floor Design

The CLT floors were designed to comply with the flexural strength capacity, shear strength capacity, and deflection limits as prescribed in *ASCE 7-10, AWC-NDS, AWC-2018, ANSI-APA PRG, 780 CMR 16,* and all other applicable building codes and design guides (ASCE & Structural Engineering Institute, 2010; AWC, 2018b; AWC, 2018a; APA & American National Standards Institute [ANSI], 2018; Office of Public Safety and Inspections, 2017). When determining the CLT floor panel size, an iterative process was used. First, a five-ply panel was analyzed to determine whether it met the flexural, shear, and deflection criterion. A five-ply panel was examined because it is the typical size used for occupied floors in office buildings. The stress grade of the panel was E1 because that is the stress grade of CLT floor panels that Nordic provides (Nordic, 2020). The panels were designed to span in the east-west direction between the perpendicular beams spanning in the north-south direction. Once it was determined that a five-ply floor panel could be used, a three-ply panel was also examined as smaller and lighter structural members can produce a more economical design. It was determined that a three-ply panel with intermittent beams added to the roof level would provide adequate support to the roof of the building.

The final design consisted of 240 12' x 8' five-ply CLT panels, 30 12' x 8' 7 ¼" five-ply CLT panels for the top half of the building, and 30 12' x 8' 7 ⅜" five-ply CLT panels for the bottom half of the building for each floor from the ground floor to the fourth floor, as seen in Figure 5.4.1.1. The lobby consisted of 16 12' x 8' five-ply CLT panels, as seen in Figure 5.4.1.2. The roof was composed of 480 6' x 8' three-ply CLT panels, 60 6' x 8' 7 ¼" three-ply CLT panels for the top half of the building, and 60 6' x 8' 7 ⅜" three-ply panels for the bottom half of the building, as seen in Figure 5.4.1.3. While continuous spanning panels were considered, the longer length of each panel would have resulted in more plies being needed. In order to keep the five-ply CLT panels for occupied floors and three-ply CLT panels for the roof that are typically used in office buildings and are typically less expensive than panels with more plies, simply span

CLT panels were chosen. Google Sheets were used throughout the iterative design process to ensure correct calculations. These calculations can be found in Appendix C.

The self-weight of the CLT floors were compared to the self-weight of the floors of the existing structure to ensure the new dead load applied to the existing exterior mass masonry walls would be allowable. The case study building has an existing three-inch wood subfloor with an existing one-inch thick layer of gypcrete. The combined self-weight of these existing elements would conservatively be approximately 18 pounds per square foot (APA, n.d.b; Rubio, 2020). The new dead load produced by the self-weight of the five-ply CLT floors was approximately 21 pounds per square foot, and the new dead load produced by the self-weight of the three-ply CLT floors was approximately 12 pounds per square foot.

This means that while the new three-ply CLT floors would produce a smaller dead load on the existing exterior mass masonry walls than the existing floors, the five-ply CLT floors would produce a larger dead load on the mass masonry walls. Typically, if the weight of a new floor is within five percent of the existing floor weight, no changes would be needed for the existing exterior walls. However, the maximum weight the new five-ply CLT floors could have without needing upgrades or retrofits to the existing mass masonry walls would be 19 pounds per square foot. Since the weight of the new five-ply CLT floors was also larger than this, some upgrades or retrofits would need to be made to the existing mass masonry walls, although the upgrade or retrofit would be minimal as the mass masonry walls would only need to additionally support three pounds per square foot. When completing the upgrade or retrofit, new interior column footings will also be placed in order to support any additional loadings from the new mass timber design, which will ensure the existing foundation will be able to support this new design.

The combined dead and live loads of the new CLT floors were also compared to the dead and live loads of the existing wood subfloor with the layer of gypcrete to ensure the new loadings applied to the existing exterior mass masonry walls would be allowable. The existing building has, conservatively, a floor dead load of 23 pounds per square foot, including the self-weight of the floor, MEP, hung ceilings, and finishes. The existing building also has, conservatively, a floor live load of 125 pounds per square foot (ASCE & Structural Engineering Institute, 2010). Combined, the dead and live load of the floors of the existing building is approximately 148 pounds per square foot. The new floor dead load of the five-ply CLT floors was approximately 29 pounds per square foot, including the self-weight of the floor, MEP, hung ceilings, and finishes, while the new floor dead load of the three-ply CLT floors was approximately 20 pounds per square foot. The new floor live load of the five-ply CLT floors was 100 pounds per square foot, while the new floor live load of the three-ply CLT floors was 20 pounds per square foot. Combined, the dead and live load of the new floors was approximately 129 pounds per square foot for the five-ply CLT floors and 40 pounds per square foot for the three-ply CLT floors. Since both the five-ply and three-ply CLT floors have a smaller combined dead and live load than the existing floors, no additional upgrades or retrofits would be needed for the existing exterior mass masonry walls.

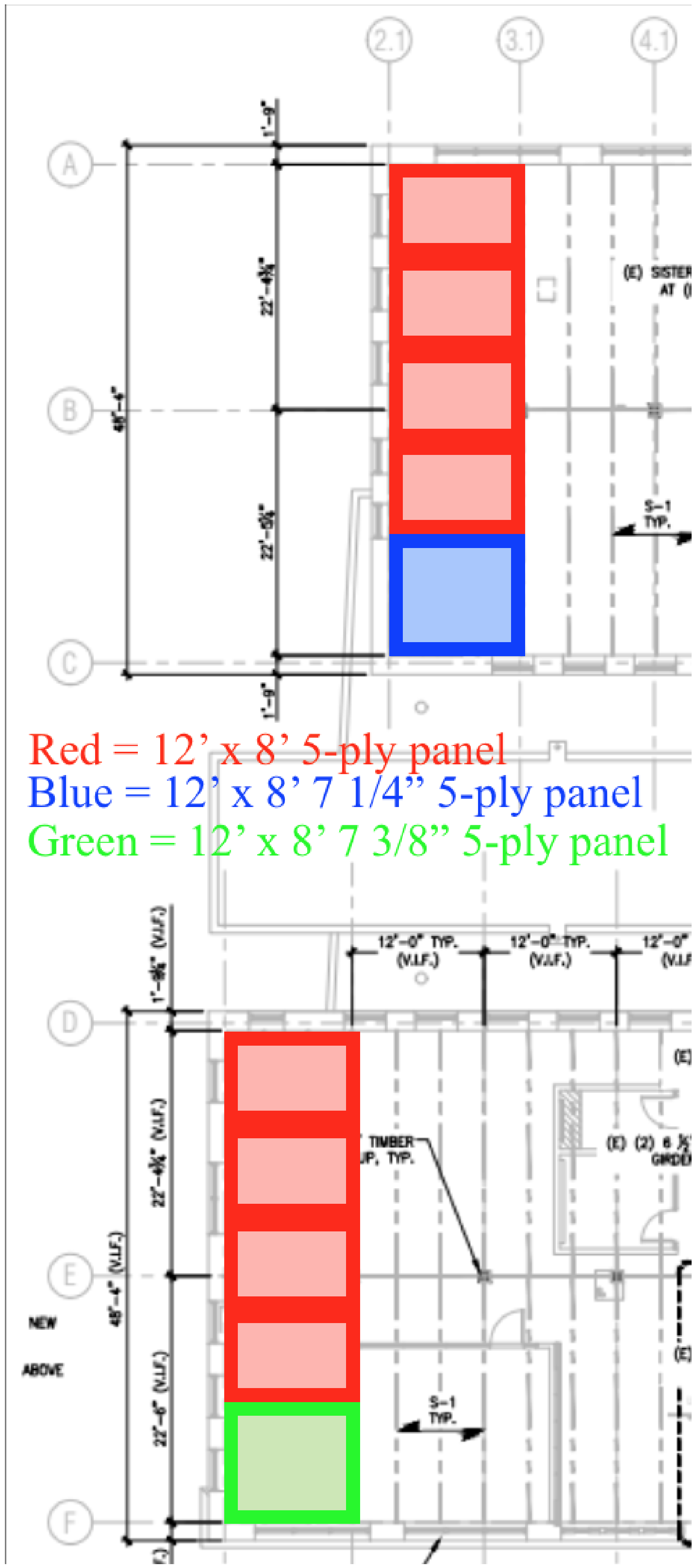


Figure 5.4.1.1: Ground Floor Through 4th Floor CLT Floor Panel Sizes

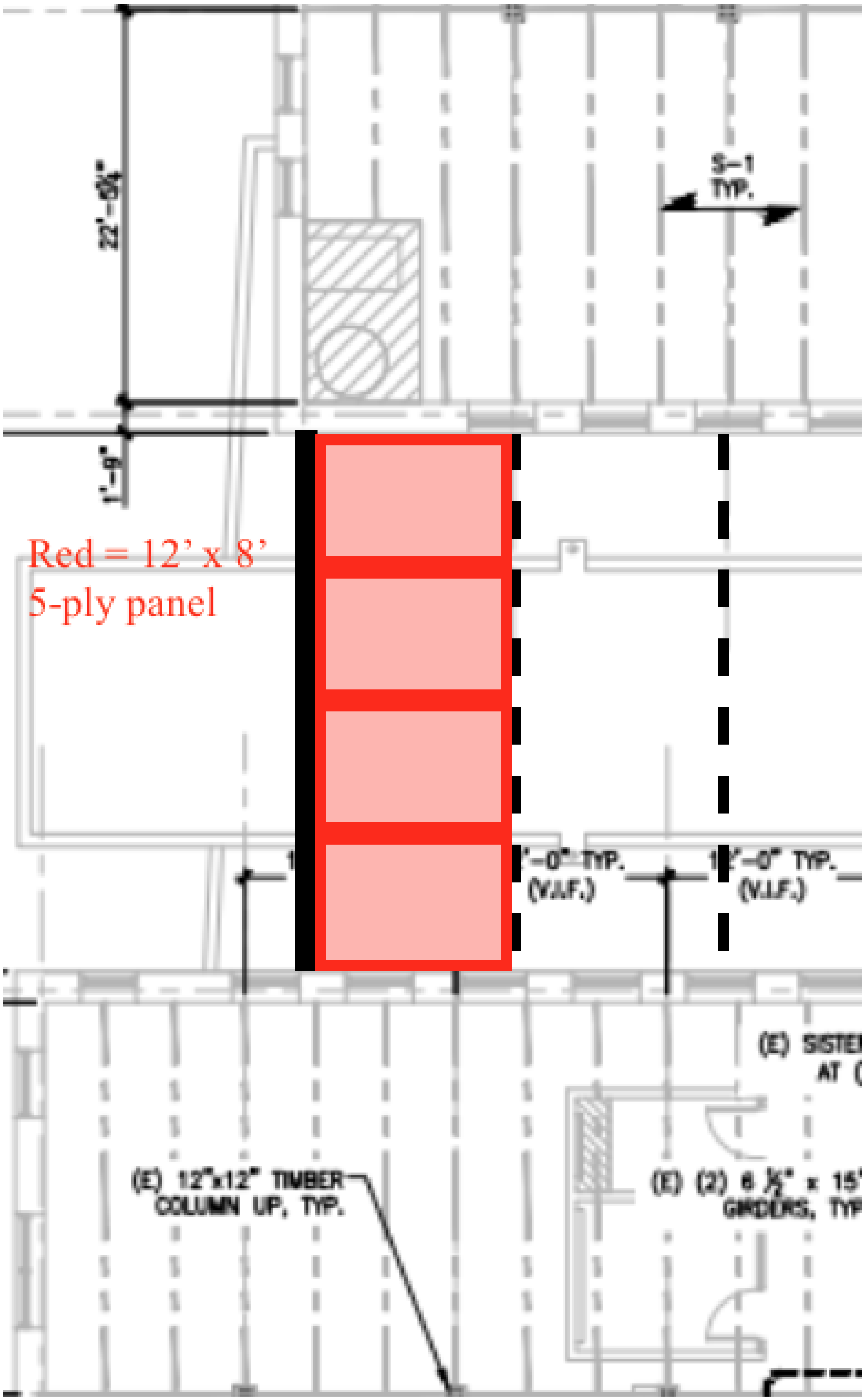


Figure 5.4.1.2: Attached Lobby CLT Floor Panel Sizes

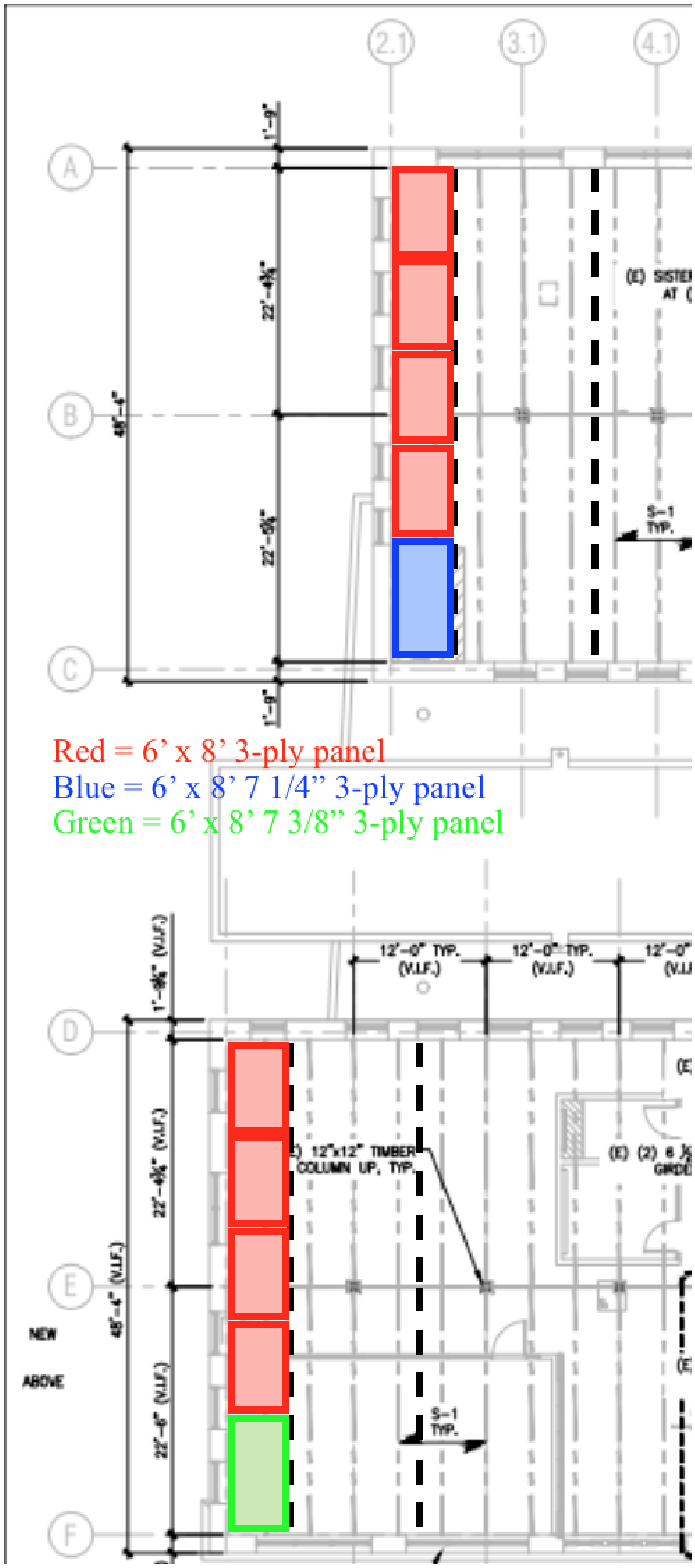


Figure 5.4.1.3: Roof CLT Floor Panel Sizes

5.4.2 Cross-Laminated Timber Wall Design

The CLT walls were designed to comply with the axial loading capacity as prescribed in *ASCE 7-10, AWC-NDS, AWC-2018, ANSI-APA PRG, 780 CMR 16*, and all other applicable building codes and design guides (ASCE & Structural Engineering Institute, 2010; AWC, 2018b; AWC, 2018a; APA & ANSI, 2018; Office of Public Safety and Inspections, 2017). Wall panels were needed to enclose the stairways, elevators, and some masonry elements throughout the building. When determining the CLT wall panel size, an iterative process was used. First, a five-ply panel was analyzed to determine whether it met the axial loading criterion. The stress grade of the panel was E1 because that is the stress grade of CLT wall panels that Nordic provides (Nordic, 2020). Once it was determined that a five-ply wall panel could be used, a three-ply panel was also examined as smaller and lighter structural members can produce a more economical design. Each wall was also analyzed as a shear wall to provide resistance to the lateral loadings placed on the building by the seismic and wind loads. An example of the load path for the lateral loadings through these shear walls can be seen in Figure 5.4.2.1.

The final design consisted of one 5 ½’ x 13 ½’ five-ply CLT panel, one 15’ 10 ⅝” x 13 ½’ five-ply CLT panel, two 21’ 2 ⅛” x 13 ½’ five-ply CLT panels, one 8’ 5 ¾” x 13 ½ five-ply CLT panel, two 27’ 2 ⅛” x 13 ½’ five-ply CLT panels, two 11’ 3 ⅝” x 13 ½’ five-ply CLT panels, two 37’ 5” x 13 ½’ five-ply CLT panels, two 9’ 10 ⅝” x 13 ½’ five-ply CLT panels, one 12’ 4 ¼” x 13 ½’ five-ply CLT panel, one 24’ x 13 ½’ five-ply CLT panel, one 14’ 10” x 13 ½’ five-ply CLT panel, one 17’ 7 ⅞” x 13 ½ five-ply CLT panel, two 13’ 11 ⅜” x 13 ½’ five-ply CLT panels, two 20’ 10” x 13 ½’ five-ply CLT panels, two 8’ 10” x 13 ½’ five-ply CLT panels, and two 27’ 10 ⅝” x 13 ½’ five-ply CLT panels for each floor from the ground floor to the roof, as seen in Figure 5.4.2.2. The exterior lobby wall consists of one 32’ x 27’ five-ply CLT panel, as seen in Figure 5.4.2.3. Due to the larger size of the exterior lobby wall, the fabricator may need to fabricate smaller wall pieces for shipping. Google Sheets were used throughout the iterative design process to ensure correct calculations. These calculations can be found in Appendix C.

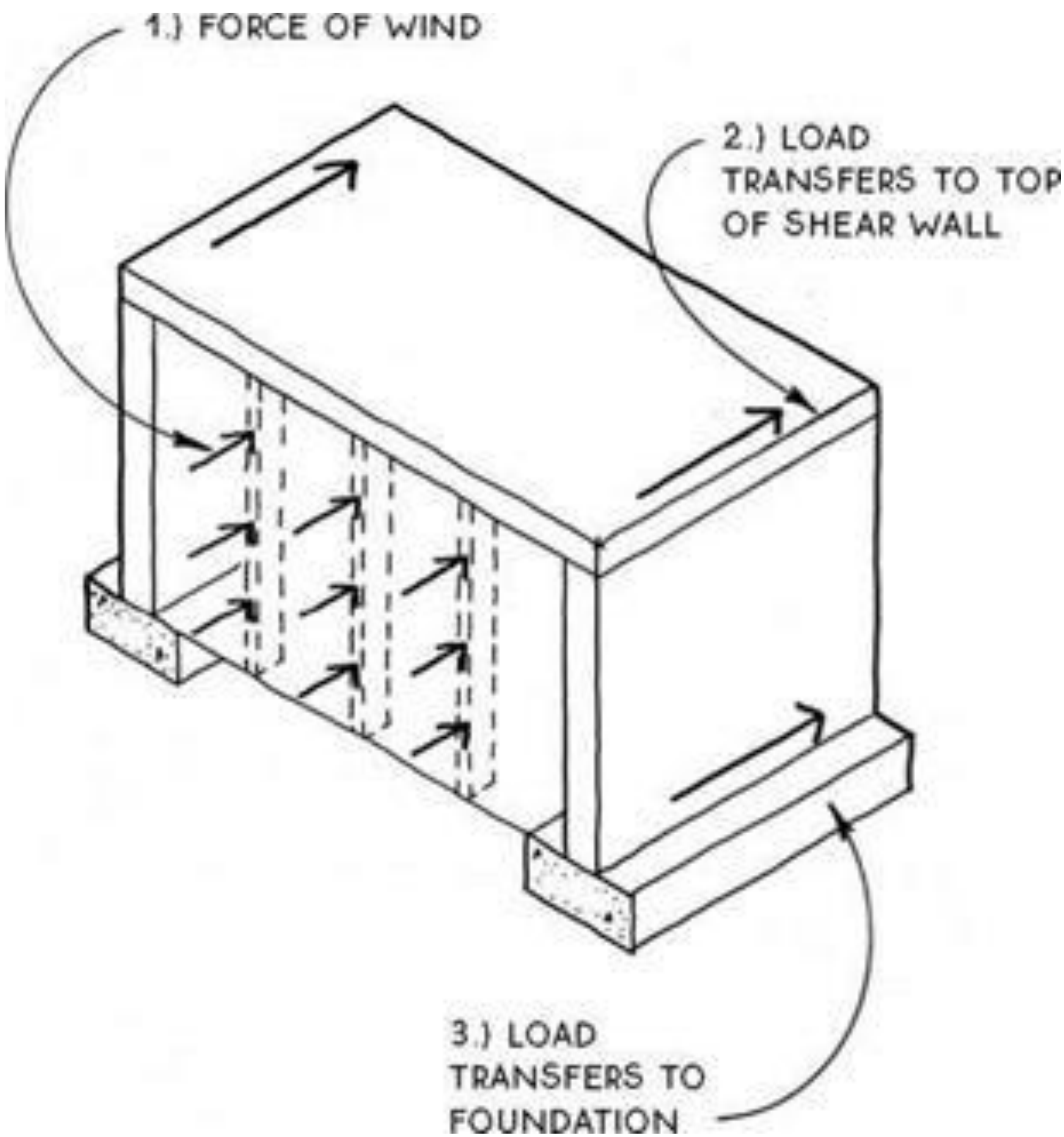


Figure 5.4.2.1: Load Path for Lateral Loads Through a Shear Wall from TeamCivil

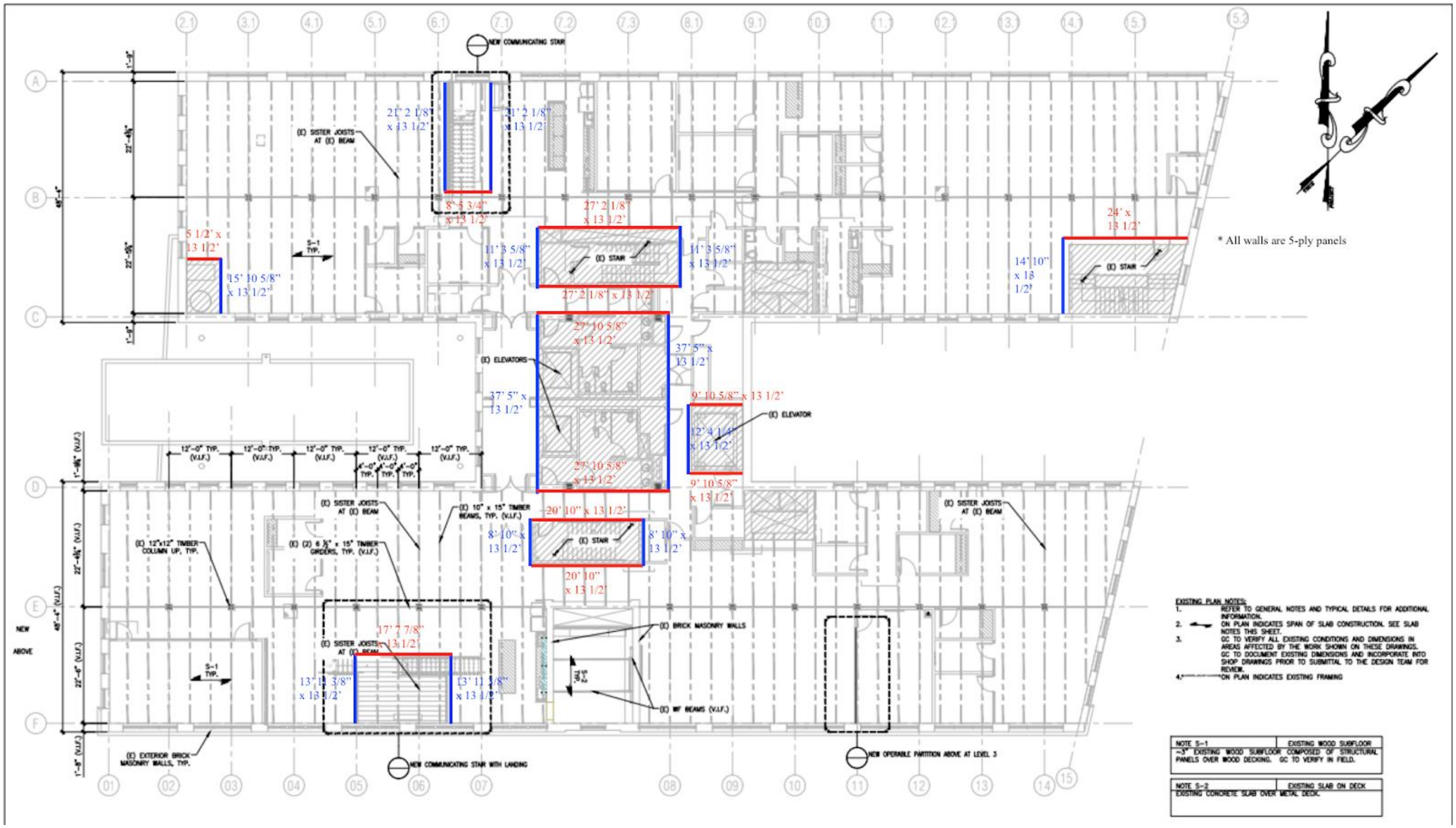


Figure 5.4.2.2: Ground Floor Through 4th Floor CLT Wall Panel Sizes

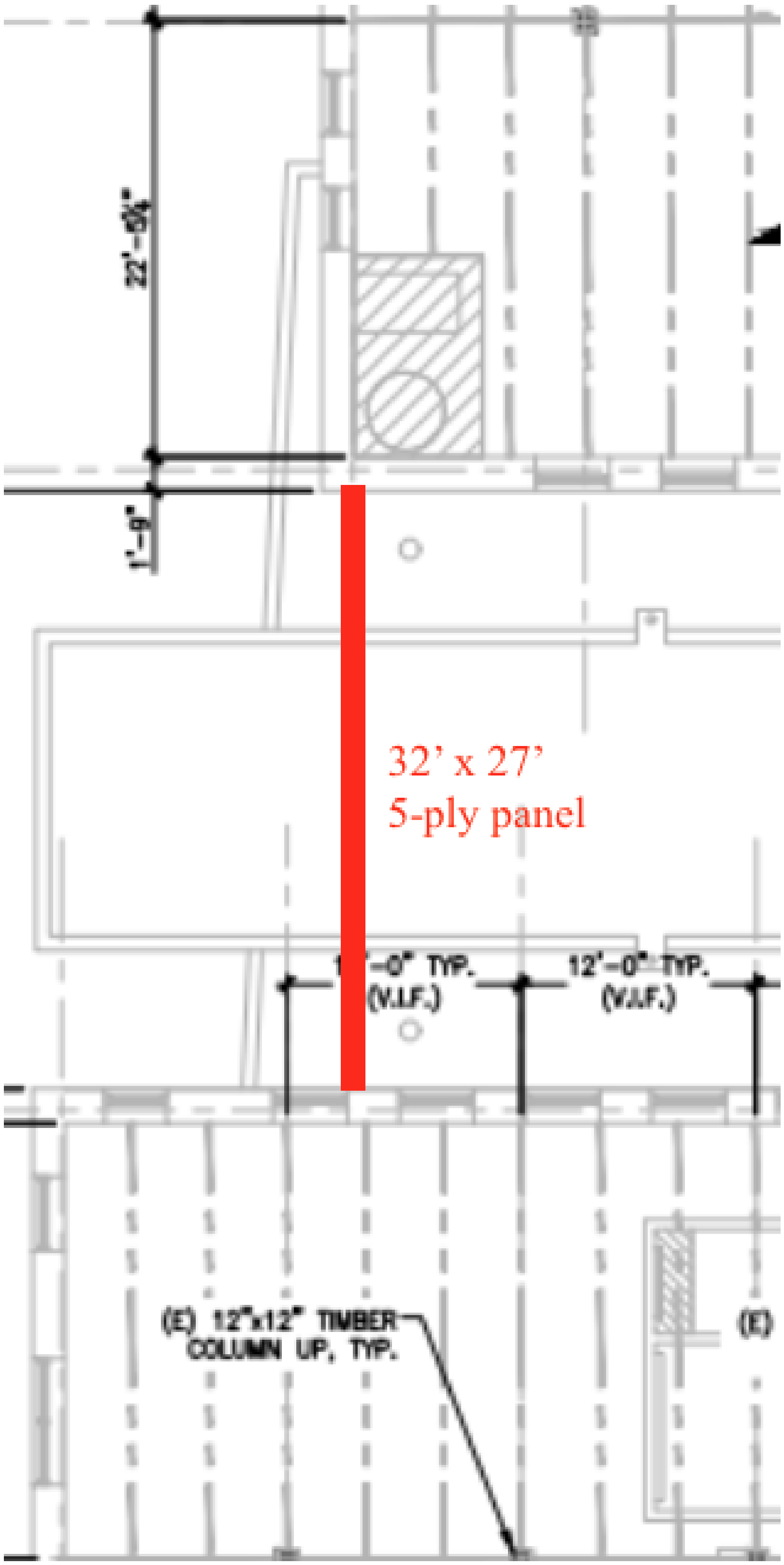


Figure 5.4.2.3: Attached Lobby CLT Wall Panel Size

6.0 Steel Frame Design

The steel frame design was completed in a similar manner as the mass timber design. The steel design, however, was completed using the Load and Resistance Factor Design (LRFD) practice. The framing of the building was kept the same as in the mass timber design, so the design process was simplified. The design consisted of 90 beams and 28 columns per floor, with each floor having an area of approximately 18,600 square feet. The attached lobby consisted of four beams with an approximate area of 1,800 square feet. The attached lobby for the steel design would also include a new masonry exterior wall. Each floor was 13 ½ feet in height for a total building height of 81 feet. The framing and floor plan for each floor was determined using the floor plans provided by SGH, which can be seen in Appendix B.

The steel framing system resists gravity loads in the same way as the mass timber framing system: by transferring the loads from member to member beginning at the roof of the building and continuing until all of the gravity loads are transferred to the foundation. In order to provide adequate resistance to the lateral loads, braced frames were used within the framing system. These bracings perform similarly to a shear wall in that they prevent individual members, and the building as a whole, from deflecting horizontally and allow lateral loads to transfer down through the building to the foundation. Bracings create a truss-like system to provide more stability and limit horizontal drift.

6.1 Loadings Considered in the Steel Frame Design

The same five load types as for the CLT alternative were considered in the design of the structural steel system: dead loads, live loads, snow loads, seismic loads, and wind loads. The loadings considered for the design can be found in Table 6.1.1. The weight of a four-inch thick concrete slab on a metal deck as well as the weight of mechanical, electrical and plumbing (MEP) systems and hung ceilings and finishes were considered when calculating the dead loads. Once a beam size was chosen, the dead load was updated to include the member's self-weight. The live loads remained the same from the mass timber design, with the 80 pounds per square foot for corridors used only for members that were just supporting a corridor space and the 100 pounds per square foot load for a Class A office space used for members that supported both an office space and a corridor in order to design for the highest possible load. The snow loads and the wind loads remained the same from the mass timber design.

The seismic, or earthquake, loads were determined using the provisions of *780 CMR 16* and *ASCE 7-10* (Office of Public Safety and Inspections, 2017, ASCE & Structural Engineering Institute, 2010). A seismic analysis spreadsheet was used to ease the seismic loading calculations (ICC, 2012). This spreadsheet used the seismic risk category of the building, soil classification of the site, local seismic data, and the weight of the building elements to determine the seismic loadings. The seismic forces were converted from story forces in kips to pounds per foot of building width. The building was determined to be in risk category II, and the unknown soil was classified as site class D. The weight of the designed structural members as well as the existing exterior masonry wall were used when determining the seismic loadings on the building. Since the height of each floor is considered when determining the seismic loadings, each floor had a slightly different seismic load with the higher floors having slightly higher loads. This is due to the distribution of story forces that roughly conform to the first mode shape for the building, making the building's horizontal deflection act similarly as it would to a cantilever beam up from the building's foundation (Murty et al., n.d.). This change in loading, however, was minimal. The seismic loading calculations from the seismic analysis spreadsheet can be found in Figures 6.1.1 and 6.1.2.

Table 6.1.1 Loads Considered in the Steel Frame Design

Load Type	Load	Elements Considered
Dead Load	55 psf	MEP, Self-weight of concrete slab
Live Loads	100 psf for lobby and ground floor	Lobbies and first floor corridors
	80 psf for 1st, 2nd, 3rd, and 4th floors	Corridors above the first floor
	100 psf for office spaces	Offices
	20 psf for the roof	Roof

Snow Loads	40 psf for ground	Snow load from Massachusetts Structural Design Amendments
	30 psf for the roof	
Seismic Loads	30.6 plf for lobby and ground floor	Seismic parameters from Massachusetts Structural Design Amendments, risk category II, soil site class D
	30.6 plf for 1st floor	
	30.6 plf for 2nd floor	
	30.6 plf for 3rd floor	
	30.6 plf for 4th floor	
	30.6 plf for the roof	
Wind Load	36.5 psf for lobby floor	Wind speeds from Massachusetts Structural Design Amendments
	39.2 psf for 1st floor	
	41.3 psf for 2nd floor	
	42.9 psf for 3rd floor	
	44.2 psf for 4th floor	
	45.3 psf for the roof	

SEISMIC BASE SHEAR AND VERTICAL SHEAR DISTRIBUTION

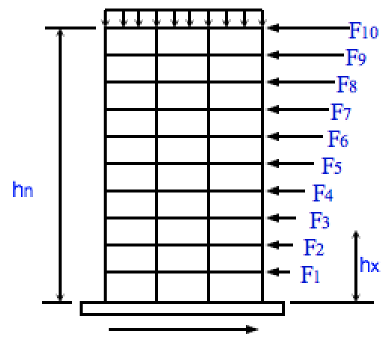
Per IBC 2012 and ASCE 7-10 Specifications

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 =	2210	
Spectral Accel., S_s =	0.217	ASCE 7-10 Figures 22-1 to 22-11
Spectral Accel., S_1 =	0.068	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 =	81.000	ft.
Actual Calc. Period, T_c =	0.000	sec. from independent analysis
Seismic Resist. System =	B3	Steel ordinary concentrically braced frames (ASCE 7-10 Table 12.2-1)



V = Cs*W = Σ(Fi) = 38.75 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	81.000	58.69
5	67.500	112.25
4	54.000	112.25
3	40.500	112.25
2	27.000	112.25
1	13.500	117.37

Total Weight, W = ΣWx = 625.06 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} =	0.347	$S_{MS} = F_a*S_s$, ASCE Eqn. 11.4-1, page 65
S_{M1} =	0.163	$S_{M1} = F_v*S_1$, ASCE Eqn. 11.4-2, page 65

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

S_{DS} =	0.231	$S_{DS} = 2*S_{MS}/3$, ASCE 7-10 Eqn. 11.4-3, page 65
S_{D1} =	0.109	$S_{D1} = 2*S_{M1}/3$, ASCE Eqn. 11.4-4, page 65

(continued:)

Figure 6.1.1: Seismic Loading Data for Steel Frame Design

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 =	0.540	sec., $T_a = C_T*h_n$^(x), ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.682	ASCE 7-10 Table 12.8-1, page 90
Period max., $T_{(max)}$ =	0.908	sec., $T_{(max)} = C_u*T_a$, ASCE 7-10 Section 12.8.2, page 90
Fundamental Period, T =	0.540	sec., T = Ta <= Cu*Ta, ASCE 7-10 Section 12.8.2, page 90

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	3.25	ASCE 7-10 Table 12.2-1, pages 73-75
Overstrength Factor, Ω_o =	2	ASCE 7-10 Table 12.2-1, pages 73-75
Defl. Amplif. Factor, C_d =	3.25	ASCE 7-10 Table 12.2-1, pages 73-75
Cs =	0.071	$C_s = S_{DS}/(R/I)$, ASCE 7-10 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.062	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: Cs =	0.062	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V = 38.75 kips, V = Cs*W, ASCE 7-10 Section 12.8.1, Eqn. 12.8-1

Seismic Shear Vertical Distribution:

Distribution Exponent, k = 1.02 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/(ΣW_i*h_i^k), ASCE 7-10 Eqn. 12.8-12, page 91

Seismic Level x	Weight, Wx (kips)	hx^k (ft.)	Wx*h^k (ft-kips)	Cvx (%)	Shear, Fx (kips)	Σ Story Shears
6	58.69	88.441	5190.6	0.174	6.75	6.75
5	112.25	73.433	8242.8	0.277	10.72	17.47
4	112.25	58.485	6564.9	0.220	8.54	26.00
3	112.25	43.612	4895.4	0.164	6.37	32.37
2	112.25	28.840	3237.3	0.109	4.21	36.58
1	117.37	14.221	1669.2	0.056	2.17	38.75
Σ =	625.06		29800.2	1.000	38.75	

Comments:

Figure 6.1.2: Seismic Loading Analysis for Steel Frame Design

The self-weight of the concrete slab on a metal deck was compared to the self-weight of the floors of the existing structure to ensure the new dead load applied to the existing exterior mass masonry walls would be allowable. The case study building has an existing three-inch wood subfloor with an existing one-inch thick layer of gypcrete. The combined self-weight of these existing elements would conservatively be approximately 18 pounds per square foot (APA, n.d.b; Rubio, 2020). The new dead load produced by the self-weight of the concrete slab on a metal deck was approximately 50 pounds per square foot.

This means that the new concrete slab on a metal deck would produce a larger dead load on the mass masonry walls than the existing floors. Typically, if the weight of a new floor is within five percent of the existing floor weight, no changes would be needed for the existing exterior walls. However, the maximum weight the new concrete slab on a metal deck could have without needing upgrades or retrofits to the existing mass masonry walls would be 19 pounds per square foot. Since the weight of the new concrete slab on a metal deck was also larger than this, upgrades or retrofits would need to be made to the existing mass masonry walls in order to provide support for an additional 32 pounds per square foot. When completing the upgrade or retrofit, new interior column footings will also be placed in order to support any additional loadings from the new steel frame design, which will ensure the existing foundation will be able to support this new design.

The combined dead and live loads of the new concrete slab on a metal deck was also compared to the dead and live loads of the existing wood subfloor with the layer of gypcrete to ensure the new loadings applied to the existing exterior mass masonry walls would be allowable. The existing building has, conservatively, a floor dead load of 23 pounds per square foot, including the self-weight of the floor, MEP, hung ceilings, and finishes. The existing building also has, conservatively, a floor live load of 125 pounds per square foot (ASCE & Structural Engineering Institute, 2010). Combined, the dead and live load of the floors of the existing building is approximately 148 pounds per square foot. The new floor dead load of the concrete slab on a metal deck was approximately 55 pounds per square foot, including the self-weight of the floor, MEP, hung ceilings, and finishes. The new floor live load of the concrete slab on a metal deck was 100 pounds per square foot for the ground floor through the 4th floor and 20 pounds per square foot for the roof. Combined, the dead and live load of the new concrete slab on a metal deck was approximately 155 pounds per square foot for the ground floor through the 4th floor and 75 pounds per square foot for the roof.

This means that the roof level would not require any additional upgrades or retrofits to the existing exterior mass masonry walls due to the roof level having a smaller combined dead and live load than the existing building. The ground floor through fourth floor, however, would produce a larger combined dead and live load than the existing building. But, if the combined dead and live loads of a new floor is within five percent of the existing floor's combined dead and live loads, no changes would be needed for the existing exterior walls. In this case, the maximum combined dead and live loads the new concrete slab on a metal deck could have without needed additional upgrades or retrofits would be 155 pounds per square foot. Since this is the loading the concrete slab on a metal deck would produce, no additional upgrades or retrofits would be needed.

6.2 Steel Beam Design

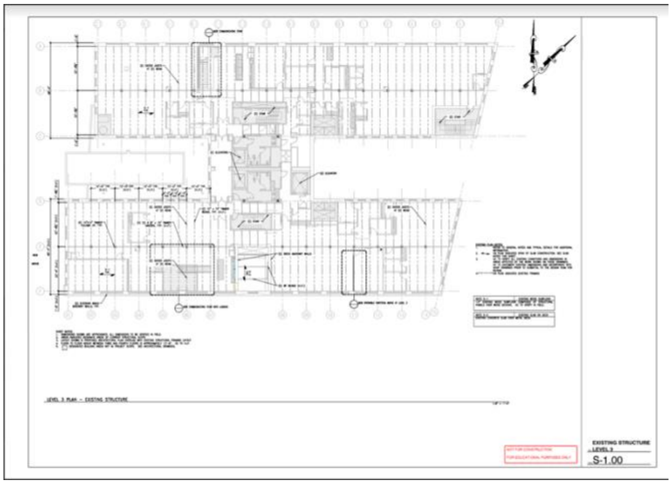


Figure 6.2.1: Typical Floor Plan

The steel beam design was completed in a similar fashion as the glulam beam design. The steel beams were designed to comply with the bending capacity, shear capacity, and deflection limits as established in *ASCE 7-10*, *AISC-15*, *780 CMR 16*, and other applicable building codes and design guides (ASCE & Structural Engineering Institute, 2010; American Institute of Steel Construction [AISC], 2017; Office of Public Safety and Inspections, 2017). When deciding on the beam size to be used throughout the building, an iterative process was used. First, a beam size from *AISC-15* was identified that met the bending, shear, and deflection criterion. The bending capacity criterion was the main determining factor for the beam sizes as the member needed to be capable of handling the bending moment caused by the loadings. Once an appropriate beam size was selected, the next smallest beam size was examined as smaller and lighter structural members provide a more economical design. The smallest beam size that met the bending, shear, and deflection criterion was chosen, and this beam size was examined for applicability to the other beams throughout the building. If the already selected beam size resulted in a highly over-designed beam or an under-designed beam, a new beam size was chosen using the same iterative process. This mostly happened with beams that were in contact with a staircase. Since the cast-in-place concrete slab on a metal deck could remain the same on the roof level, no intermittent roof beams were needed in the steel design. While this design process caused some members to be slightly over-designed, having as many beams of the same size as possible would ease the manufacturing of the beams and the renovation of the building.

The final beam design was composed of 52 W16x31 beams spanning in the north-south direction, eight W12x14 beams in contact with the staircases throughout the floor, and 30 W12x22 girders spanning east-west for each floor from the ground floor to the fourth floor, as seen in Figure 6.2.2. The lobby was composed of four W24x62 beams, as seen in Figure 6.2.3.

The roof was composed of 52 W14x22 beams spanning in the north-south direction, eight W12x14 beams in contact with the staircases throughout the floor, and 30 W12x14 girders spanning east-west, as seen in Figure 6.2.4. Google Sheets were used throughout the iterative design process to ensure correct calculations. These calculations can be found in Appendix D.

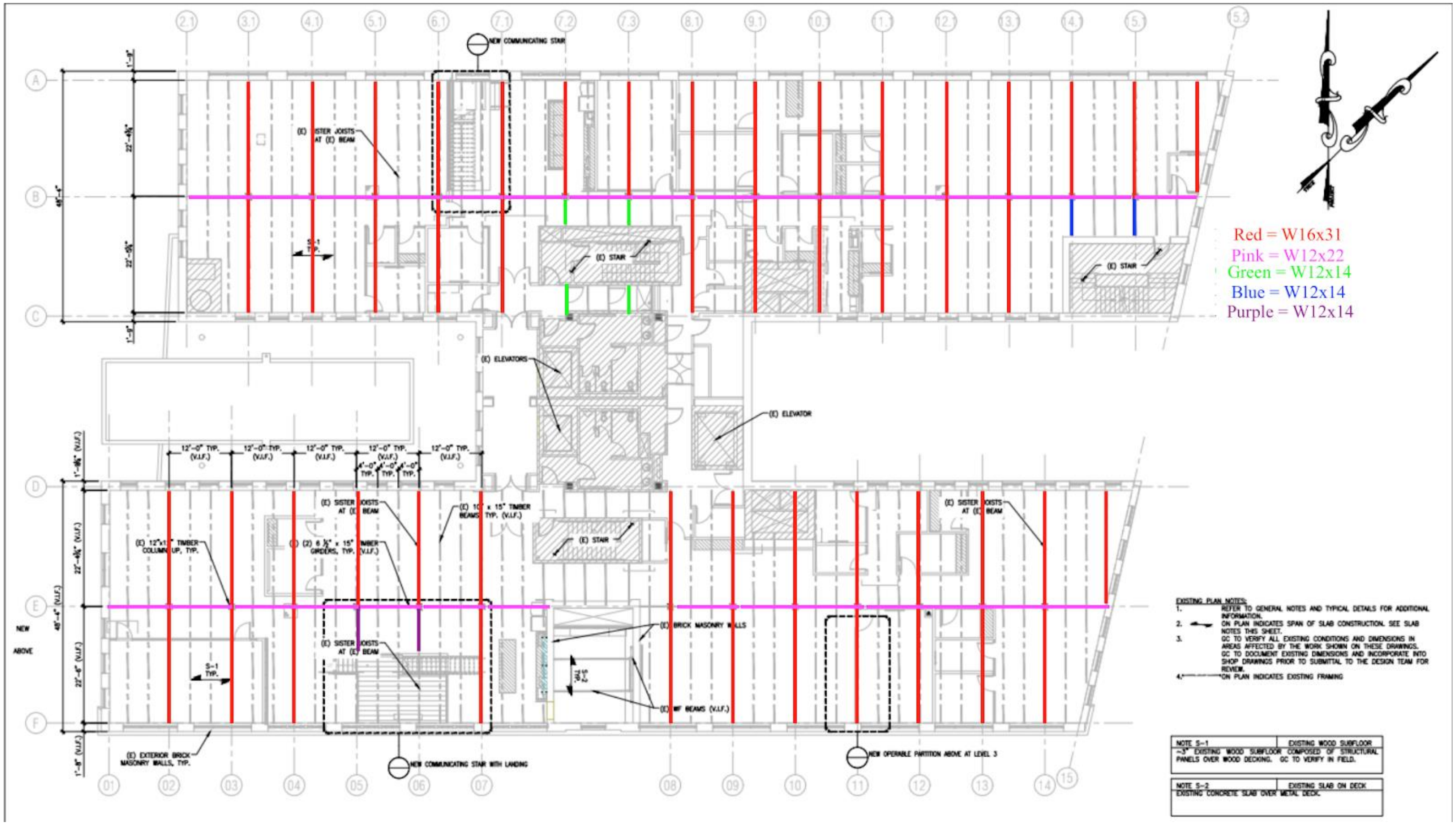


Figure 6.2.2: Ground Floor Through 4th Floor Steel Beam Sizes

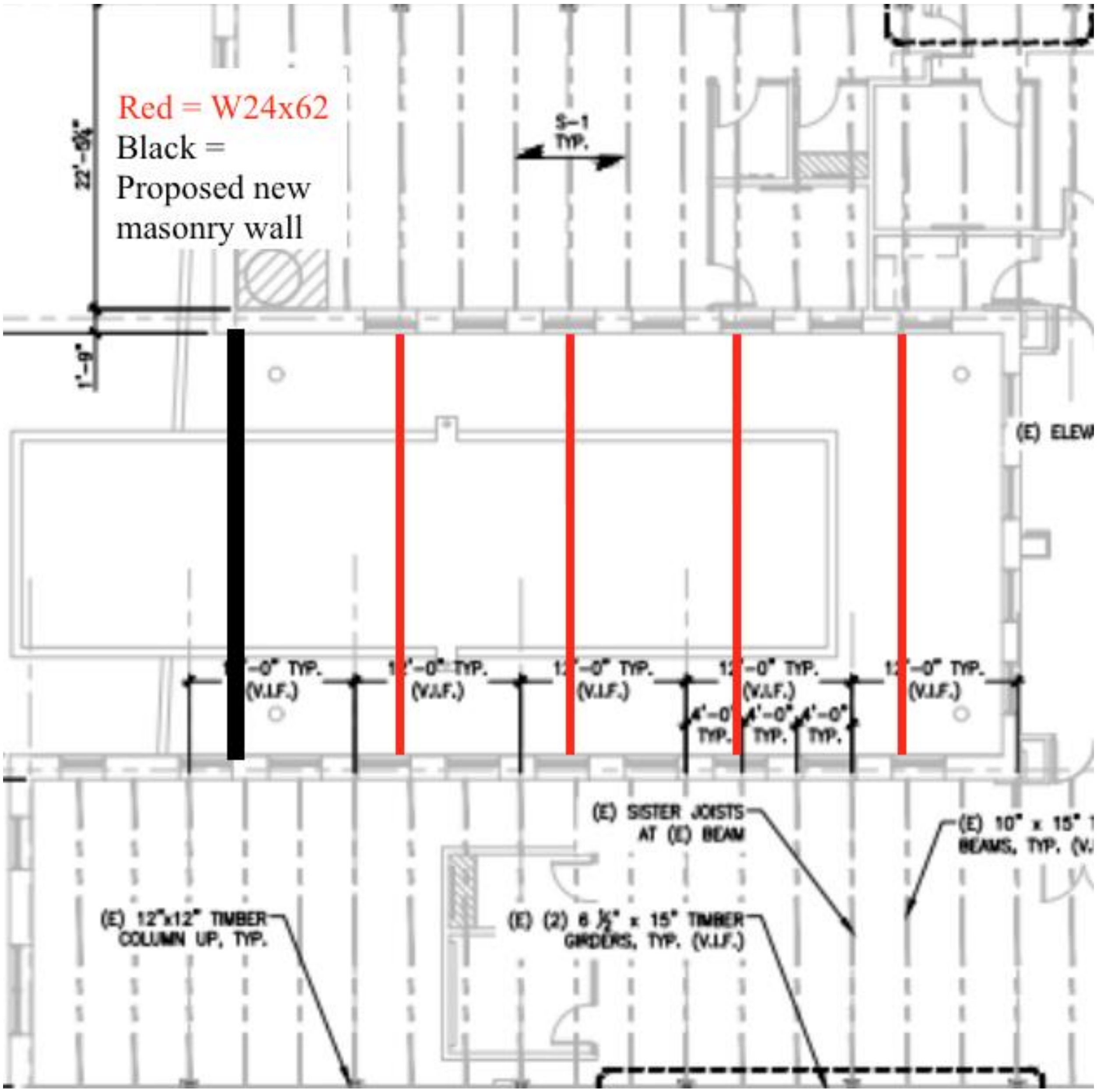


Figure 6.2.3: Attached Lobby Steel Beam Sizes

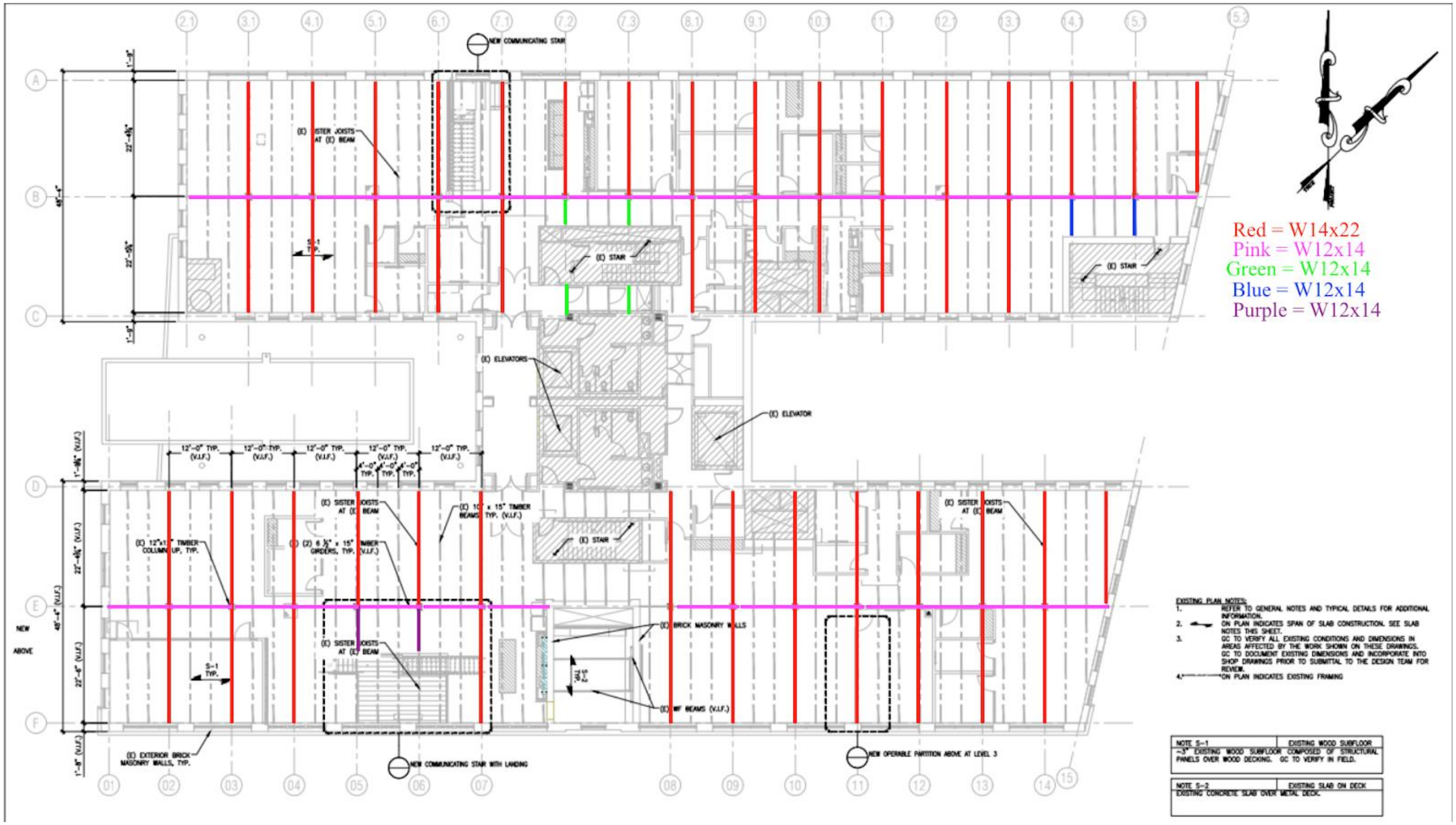


Figure 6.2.4: Roof Steel Beam Sizes

6.3 Steel Column Design

The steel column design was also completed in a similar fashion as the glulam column design. The steel columns were designed to comply with the axial loading capacity, buckling capacity, and shear capacity as prescribed in *ASCE 7-10*, *AISC-15, 780 CMR 16*, and all other applicable building codes and design guides (ASCE & Structural Engineering Institute, 2010; AISC, 2017; Office of Public Safety and Inspections, 2017). When determining the column sizes to be used throughout the building, an iterative process was used. First, a column size from *AISC-15* was determined that met the axial loading, buckling, and shear criterion. Ensuring the column size could adequately support the axial loading was the main determining factor for the column sizes. Once an initial column size was chosen, the next smallest column size was analyzed as smaller and lighter structural members can provide a more economical design. The smallest column size that met the axial loading, buckling, and shear criterion was chosen and that column size was examined for the other columns throughout the building. While this design process caused some columns to be slightly over-designed, having as many columns of the same size as possible would ease the manufacturing of the columns and the renovation of the building. The final column design consisted of 28 W8x31 columns for each floor from the ground floor to the roof, as seen in Figure 6.3.1. Google Sheets were used throughout the iterative design process to ensure correct calculations. These calculations can be found in Appendix D.

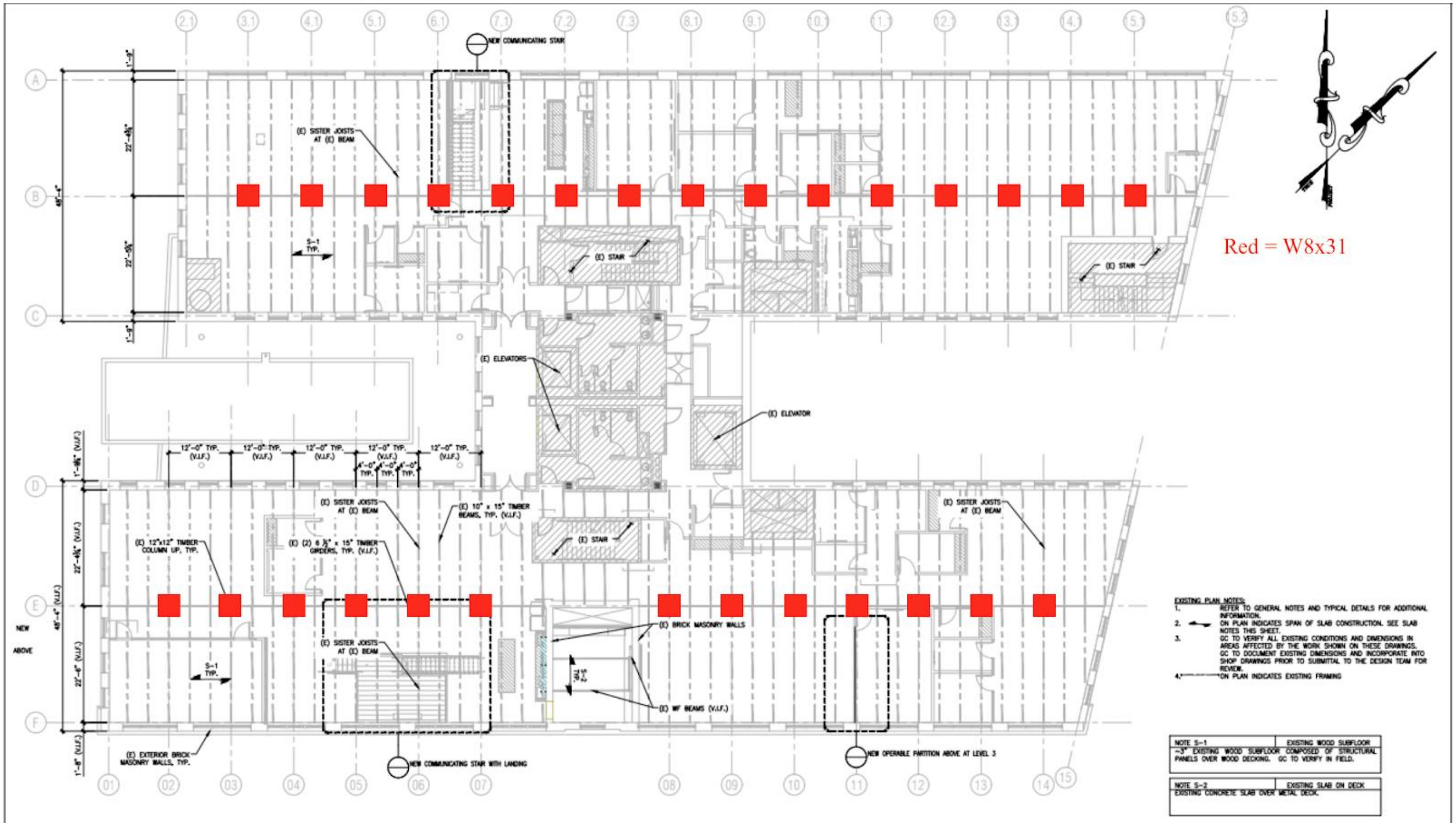


Figure 6.3.1: Ground Floor Through Roof Steel Column Sizes

6.4 Steel Bracing Design

In order to resist the lateral loading from the seismic and wind loads, bracing throughout the building was needed. The load path for the lateral loadings through the bracing supports can be seen in Figure 6.4.1. It was determined that lateral bracings were needed every third bay in the east-west direction to support the north-south direction beams for each half of the building. This determination was made to resist the horizontal deflection, or sway, of the building. The introduction of braces to the design also reduced the impact of lateral loads on the beam and column members, as they were designed to resist vertical loads (Bwail, 2019). To provide more stability, the bracings were placed on the opposite bay in the north-south direction on each floor. For example, a bracing supporting beam A5.1-B5.1 on the ground floor would be placed on beam B5.1-C5.1 on the first floor. The maximum spacing for the brace on the W16x31 beams was determined to be 9' 10", meaning that two inverted V-bracings were needed for each beam. Inverted V-bracings, also known as chevron bracings, were chosen as they can provide the most resistance to sway (Alshamrani et al., 2009). This type of bracing also allows the tenants flexibility with movement about the floor by allowing doorways and corridors to be placed along the bracing lines (AISC, n.d.). The steel bracings and the associated welded gusset plate connection were designed to comply with the bending moment capacity, net and shear rupture capacity, buckling capacity, tension capacity, shear capacity, and deflection limits as prescribed in *ASCE 7-10*, *AISC-15*, *780 CMR 16*, and all other applicable building codes and design guides (ASCE & Structural Engineering Institute, 2010; AISC, 2017; Office of Public Safety and Inspections, 2017).

When determining the bracing size to be used throughout the building, an iterative process was used. First, a bracing size from *AISC-15* was determined that met the required criterion. Once an initial bracing size was chosen, the next smallest size was analyzed as smaller and lighter structural members can provide a more economical design. The smallest bracing size that met the required criterion was chosen and that size was examined for application to the other bracings throughout the building. The final bracing design consisted of four HSS5x5x⅜ braces with ¾" thick gusset plates placed every three bays in the east-west direction to support the north-south direction beams on each half of the floor, as seen in Figure 6.4.2. Google Sheets were used throughout the iterative design process to ensure correct calculations. These calculations can be found in Appendix D.

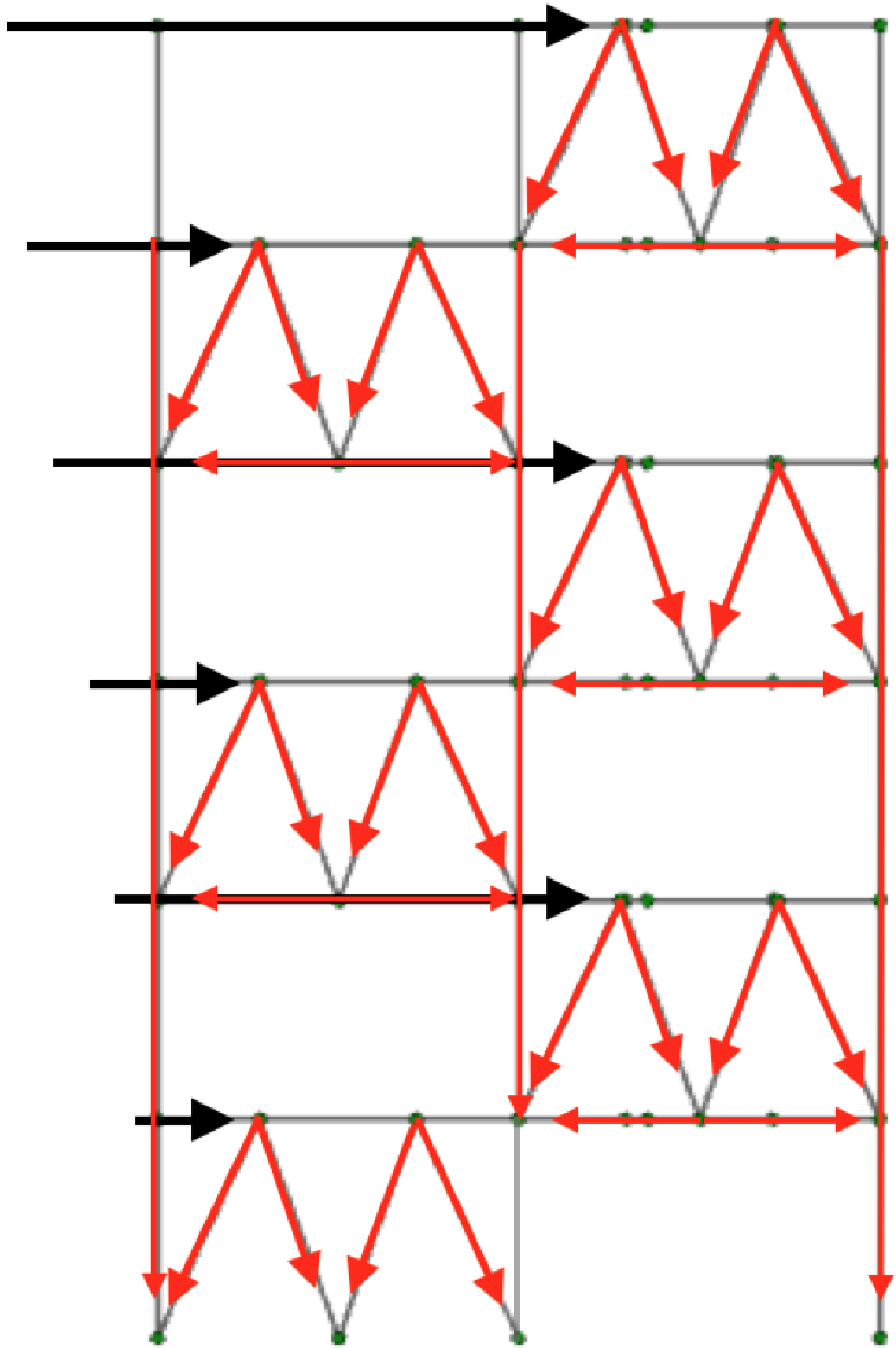


Figure 6.4.1: Load Path of Lateral Loads Through the Braced Frame

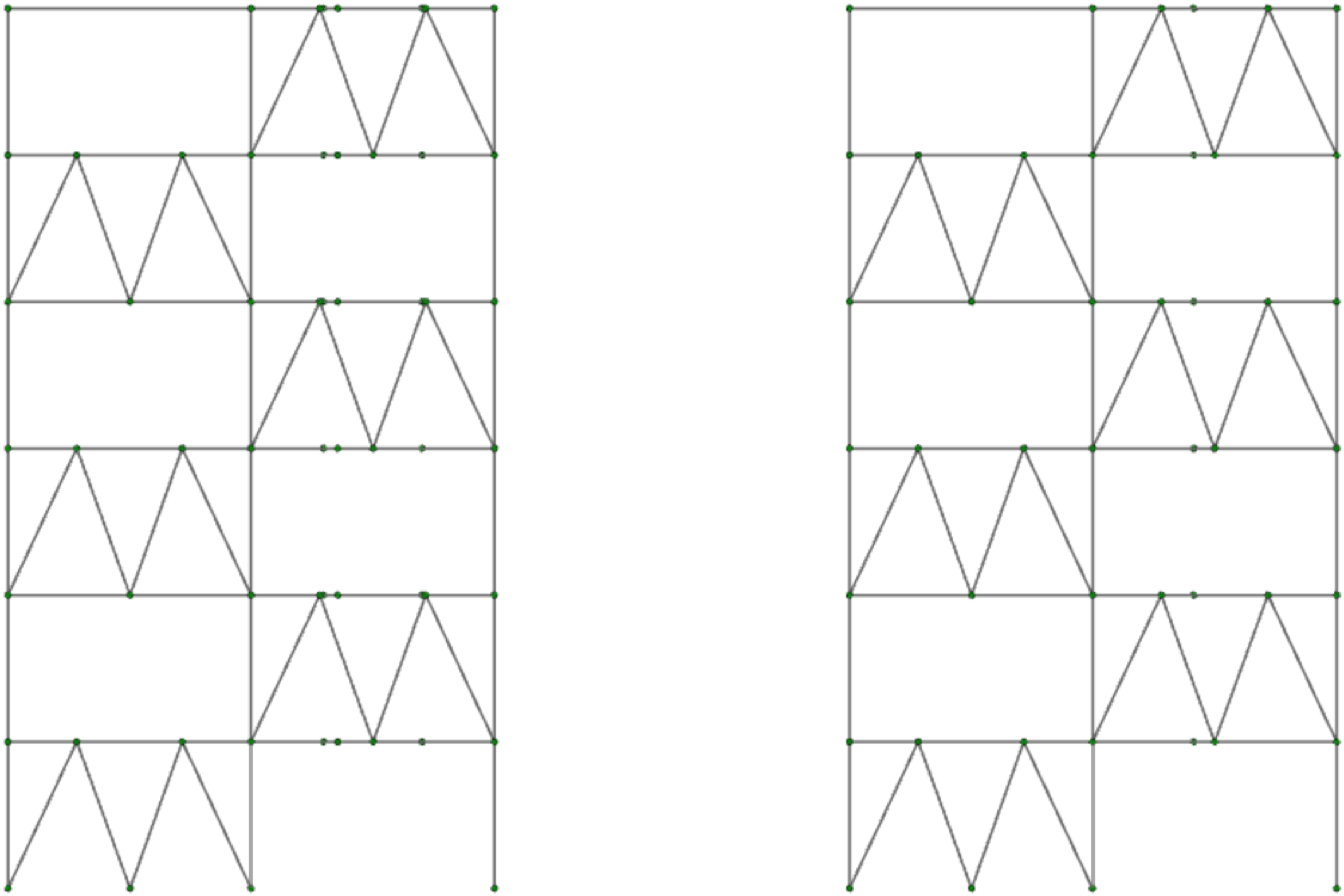


Figure 6.4.2: Braced Frame Design

7.0 Evaluation

The biggest factors that can affect the material or design an owner selects for a project are the total project cost and duration. Therefore, a cost analysis was performed for the structural members for both the mass timber and steel frame designs, estimating the cost of the in-place costs (cost of the materials, labor, and equipment) for the gut renovation project. However, the duration of the manufacturing and construction process for the project impacts not only the total project duration, but also the overall cost of the project. The age old aphorism “time is money” is especially true in the construction industry as labor costs are determined by the project duration. Because of this, the manufacturability and constructability of the mass timber and steel frame design were also evaluated.

7.1 Cost Analysis of the Mass Timber and Steel Frame Designs

The costs of the structural elements for the mass timber design can be seen in Table 7.1.1. The glulam beam costs for the mass timber design were calculated using *Building Construction Costs with RSMeans Data* (R.S. Means Company, 2019). While this data did not include the cost of the glulam beam sizes determined for the mass timber design, an average of the cost per cubic inch of the glulam beams listed in *Building Construction Costs with RSMeans Data* was calculated and used as a form of unit cost. The costs for the glulam columns costs for the mass timber design were calculated using *Assemblies Costs with RSMeans Data* (R.S. Means Company, 2016). The costs for the CLT floors and wall panels were calculated using information from Nordic (M. Richard, personal communication, March 12, 2021). In the case of missing information, conservative extrapolations were made. The total structural in-place cost of the mass timber design was estimated to be approximately $4,900,000, or $43 per square foot.

Table 7.1.1: Cost Analysis for Mass Timber Design		
Structural Element	**Unit Cost**	**Total Cost**
Glulam Beams		
17 ⅝” x 15 ⅛”	$0.03/cubic in	$554,000
13 ⅝” x 10 ¾”	$0.03/cubic in	$94,000
8 ½” x 8”	$0.03/cubic in	$2,300
7 ¼” x 6”	$0.03/cubic in	$2,100
7 ¼” x 7 ⅛”	$0.03/cubic in	$1,500
11 ½” x 9 ¾”	$0.03/cubic in	$93,000
9 ½” x 8 ½”	$0.03/cubic in	$10,000
5 ⅜” x 5”	$0.03/cubic in	$360
5 ⅜” x 3 ¾”	$0.03/cubic in	$380
5 ⅜” x 4 ¼”	$0.03/cubic in	$260
23 ¾” x 21 ¾”	$0.03/cubic in	$24,000
	Total Glulam Beam Cost	$780,000
Glulam Columns		
9’ x 9’	$3,600/mbf	$46,000
8’ x 8’	$3,400/mbf	$6,900
	Total Glulam Columns Cost	$53,000
CLT Floors		
8’ x 12’	$20/sq ft	$2,300,000
8’ 7 ¼” x 12’	$20/sq ft	$310,000
8’ 7 ⅜” x 12’	$20/sq ft	$310,000
8’ x 6’	$12/sq ft	$280,000
8’ 7 ¼” x 6’	$12/sq ft	$37,000

8’ 7 ¾” x 6’	$12/sq ft	$37,000
	Total CLT Floor Cost	$3,300,000
CLT Walls		
5 ½’ x 13 ½’	$20/sq ft	$8,900
15’ 10 10/17” x 13 ½’	$20/sq ft	$26,000
21’ 2 2/17” x 13 ½’	$20/sq ft	$69,000
8’ 5 11/17” x 13 ½’	$20/sq ft	$14,000
27’ 2 2/17” x 13 ½’	$20/sq ft	$88,000
11’ 3 9/17” x 13 ½’	$20/sq ft	$37,000
37’ 4 16/17” x 13 ½’	$20/sq ft	$120,000
9’ 10 10/17” x 13 ½’	$20/sq ft	$32,000
12’ 4 4/17” x 13 ½’	$20/sq ft	$20,010
24’ x 13 ½’	$20/sq ft	$39,000
14’ 9 15/17” x 13 ½’	$20/sq ft	$24,000
17’ 7 13/17” x 13 ½’	$20/sq ft	$29,000
13’ 11 5/17” x 13 ½’	$20/sq ft	$43,000
20’ 9 15/17” x 13 ½’	$20/sq ft	$67,000
8’ 9 15/17” x 13 ½’	$20/sq ft	$29,000
27’ 10 10/17” x 13 ½’	$20/sq ft	$90,000
32’ x 27’	$20/sq ft	$17,000
	Total CLT Wall Cost	$750,000
	Total Mass Timber Design Cost	$4,900,000
		$43/sq ft

The costs of the structural elements for the steel frame design can be seen in Table 7.1.2. The structural in-place costs for the steel frame design were calculated using *Building Construction*

Costs with RSMeans Data (R.S. Means Company, 2019). The in-place cost for the gusset plates were calculated by taking the average price of two metals manufacturer's gusset plate prices (Metals Depot, n.d.; Midwest Steel and Aluminum, n.d.). In the case of missing information, conservative extrapolations were made. The total structural in-place cost of the steel frame design was estimated to be approximately $980,000, or $8.70 per square foot.

Table 7.1.2: Cost Analysis for Steel Frame Design

Structural Element	**Unit Cost**	**Total Cost**
Steel Beams		
W16x31	$57/linear ft	$330,000
W12x22	$42.50/linear ft	$77,000
W12x14	$33/linear ft	$24,000
W24x62	$107/linear ft	$14,000
	Total Steel Beam Cost	$450,000
Steel Columns		
W8x31	$58.96/linear ft	$130,000
	Total Steel Column Cost	$130,000
Concrete Slab		
4" Thick Concrete Slab	$270/cubic yd	$380,000
	Total Concrete Slab Cost	$380,000
Bracings		
HSS5x5x⅜	$103/brace	$22,000
¾" Gusset Plate	$41.35/plate	$8,900
	Total Bracings Cost	$31,000
	Total Steel Frame Design Cost	$989,000
		$8.70/sq ft

In addition, both designs would require some upgrades or retrofits to the existing exterior mass masonry walls due to the new weights produced from the designed floors as well as new interior column footings to ensure the building's foundation could adequately support the new designs. Both of these would increase the overall cost of both designs. Even with this, it is clear that the in-place costs for the steel frame design were much less than the mass timber design from the cost analysis. While glulam members can be less expensive than steel members, fabricated glulam does tend to be more expensive than steel (Buckland Timber, n.d.). This was seen with the beam cost for each design, with the glulam beams being comparable, but ultimately more expensive, than the steel beams. The glulam beams were also expected to be more expensive than the steel beams due to the mass timber design including infill beams on the roof level to allow for a three-ply CLT floor. The glulam columns, however, were less expensive than the steel columns. The biggest difference in the costs between the two designs was the CLT floor and the concrete slab on a metal deck costs. However, this does fall in line with previous comparisons of CLT and concrete structures and structural members (Came, 2018).

The steel cost per square foot seemed to be lower than the $15-$25 per square foot that is expected of a steel frame (Cost Hack, 2020). This could be due to the lack of inclusion of finishes and fire protection material for the steel members or indicative that the steel frame design was lighter than a typical steel frame. The mass timber cost per square foot, however, does fall in line with the expected cost per square foot of $48-56 for CLT structures since glulam elements, which are less expensive than CLT, were used (Concrete Reinforcing Steel Institute, 2018). Since CLT is still a relatively newer material and there are a limited number of manufacturers, the in-place costs would be higher than materials that are more readily available, like steel.

In addition to the in-place costs, the transportation of the materials from the manufacturer to the project site should also be considered. Nordic, the selected manufacturer for the mass timber design, is approximately 250 miles away from the case study building, while the closest steel manufacturer to the case study building, Boston Welding & Design, Inc., is approximately 10 miles away. While CLT is comparable to traditional construction materials in terms of transportation cost, the large difference in the locations of these manufacturers would cause the transportation of the mass timber materials to be more expensive than the steel frame design

(Lewis et al., 2016). Once again, the limit of CLT manufacturers due to the slower spread of CLT in North America would cause the mass timber design to be a more expensive option than the steel frame design. However, what CLT, and therefore the mass timber design, lacks in material and transportation costs can be improved by the manufacturability and constructability of the design.

7.2 Manufacturability and Constructability of the Mass Timber and Steel Frame Designs

Easing the manufacturing and construction of a building can reduce the overall duration of a building renovation, which would ultimately reduce the overall cost of the project. One way the design of the case study building aimed to ease the manufacturing of the building materials was to select readily available member sizes and using a typical member size as much as possible throughout the building. Readily available members are member sizes that manufacturers regularly make, so choosing these members would reduce the overall fabrication time. The mass timber structural members were selected from the Nordic Structural Details catalog, making these member sizes readily available through Nordic (Nordic, 2020). The steel members were selected from *AISC-15*. Within the design tables provided in *AISC-15*, some member sizes are bolded. These members are more efficient and widely used (Pham, 2016).

As mentioned in Chapters 5 and 6, once a member size was selected, it was analyzed for its applicability to all the other members throughout the building. This was done to also ease the manufacturing and construction of these members. Ordering multiple members of the same size would allow the manufacturer to produce the materials more efficiently as manufacturers tend to produce in batches of the same size. Using this method can reduce the time to set up the fabrication and decrease waste (Gemma, 2019). In addition, having one member size throughout the building would allow for a faster and smoother construction by reducing confusion and the risk of members being placed in the wrong location.

In general, mass timber construction tends to be completed faster than steel construction. CLT especially can be erected quickly due to the prefabrication of the panels. This off-site prefabrication allows construction crews to simply place the panels, reducing the overall labor cost and construction duration as well as improving the safety of the construction site (Di Bella & Mitrovic, 2020). In fact, CLT construction has been found to have up to a 20% shorter duration than concrete construction, and concrete construction can be up to twice as fast as steel construction (Di Bella & Mitrovic, 2020; Whirlwind Team, 2016). In addition, prefabrication can allow elements such as doors and windows to also be installed off-site, which contributes to reducing the overall construction duration.

Since CLT is a newer material, it is likely that construction crews who have limited experience with CLT may require additional construction time due to the learning curve of working with a new material. While this should not prolong the duration of the project to the point where it is a longer duration than steel construction, it should be planned for since it is likely that a construction crew completing the renovation would have limited experience using CLT. Despite having a higher in-place cost, the ability to have a shorter manufacturing duration because of the use of repetitive readily available sizes throughout the building as well as a shorter construction duration due to prefabrication, reduces the overall cost of the project and makes the mass timber design a competitive option when compared to the steel frame design.

8.0 Acoustic and Vibration Performance of CLT

In order to determine the implications of the acoustic and vibration performance of CLT and how design standards and requirements would affect the mass timber design of the case study building, several studies were identified, read, and analyzed for key findings and understanding. A summary of these studies is presented in Table 8.1.

Table 8.1: Studies on the Acoustic and Vibration Performance of CLT

Name of the Study	**Authors**	**Where the Study Was Completed**	**Year of Study**	**Types of Tests**
"The use of cross laminated timber for long span flooring in commercial buildings"	Kirsten Lewis, Bella Basaglia, Rijun Shrestha, and Keith Crews	University of Technology Sydney, Sydney, Australia	2016	● Discussion of timber floor design methods ● Finite element analysis ● Experimental modal analysis
"Acoustic characteristics of cross-laminated timber systems"	Antonino Di Bella and Milica Mitrovic	University of Padova, Padova, Italy	2020	● Review of the evolution of acoustic research on CLT
"Seismic design of a six-storey CLT building in Italy"	D. Vassallo, M. Follesa, and M. Fragiacomo	Florence, Italy	2018	● Description of the design and construction of a six-story building, with an emphasis on seismic and vibration design
"Controlling cross-laminated timber (CLT) floor vibrations: Fundamentals and methods"	Lin Hu and Sylvain Gagnon	FPInnovations, Quebec, Canada	2012	● Creation of a new design method to predict the vibration performance of CLT floors
"Vibrations in residential timber floors: A	Whokko Schirén and Trixie Swahn	Linnæus University, Småland,	2019	● Evaluated current floor structures in Sweden to determine

comparison between the current and revised Eurocode 5”		Sweden		if they would be able to pass a new vibration design method criterion under review for Eurocode 5

Since these studies were conducted outside of the United States, the codes and requirements discussed are based on international and local codes. The main code referenced throughout these studies was Eurocode 5, which addresses the design of timber structures. These studies highlighted three main factors that affect the acoustic and vibration of CLT panels, especially CLT floors: the fundamental natural frequency of the panels, the stiffness of the panels, and the velocity and acceleration of the floor.

Humans are sensitive to vibrations between 4 and 8 Hz, so floors are typically designed to either exceed that range or implement measures that will limit the susceptibility of that range (Schirén & Swahn, 2019). In addition, vibrations caused by normal walking tend to have a momentary duration for floors with a fundamental natural frequency above 8 Hz (Hu & Gagnon, 2012). In general, low-frequency floors, usually made of concrete, have a fundamental natural frequency of less than 8 Hz, while high-frequency floors, typically made of timber, steel, or lightweight concrete, tend to have a fundamental natural frequency above 8 Hz (Schirén & Swahn, 2019). Since CLT floor panels can be heavier than typical timber floors, however, there is a concern that the fundamental natural frequency of CLT could be below 8 Hz, requiring special design. Many codes internationally, however, only include guidance on designing timber floors above 8 Hz (Lewis et al., 2016; Schirén & Swahn, 2019; Vassallo et al., 2018).

In order to find the fundamental natural frequency of the floors used for the case study building, the equation $f_1 = \frac{\pi}{2l^2}\sqrt{\frac{(EI)_l}{m}}$ was used, where f_1 is the fundamental natural frequency, l is the length of the CLT floor panel, $(EI)_l$ is the longitudinal elastic modulus, and m is the mass of the CLT panel (Lewis et al., 2016; Schirén & Swahn, 2019). From this, it was found that the typical floor used from the ground floor through the fourth floor would have a fundamental natural frequency of 13 Hz and the floor used for the roof level would have a fundamental natural frequency of 35 Hz. Since both of these were above 8 Hz, no special design would be required. If the floors had a fundamental natural frequency below 8 Hz, however, this could be improved by selecting specific coatings for the floor panels or increasing the mass of the floor panels (Di Bella & Mitrovic, 2020; Schirén & Swahn, 2019).

The stiffness of the CLT floor panels also plays a role in the acoustic and vibration performance of mass timber buildings because the stiffness controls the deflection of the CLT floor panels. In the design method proposed by Schirén and Swahn (2019), a stiffness criteria was created to predict the floor performance level in terms of acoustic and vibration performance. This floor performance level goes from Level I to Level VII, where "Level I is excellent and Level VII is unacceptable" (Schirén and Swahn, 2019, p. 27). The stiffness criteria to predict the floor performance level uses the equation $w_{225\ lbs} = \frac{Fl^3}{48(EI)_l b_{ef}}$, where $w_{225\ lbs}$ is the stiffness of the floor when a concentrated static force of 225 lbs is applied and $b_{ef} = \frac{l}{1.1}\sqrt[4]{\frac{(EI)_t}{(EI)_l}}$, where $(EI)_l$ is the longitudinal modulus of elasticity and $(EI)_t$ is the transverse modulus of elasticity (Schirén and Swahn, 2019). From this, the stiffness of the floor used from the ground floor through the fourth floor would be 0.0146 in (0.371 mm), which translated to a floor performance level of Level III, which is good. The stiffness of the floor used for the roof level would be 0.0226 in (0.573 mm), which translated to a floor performance level of Level IV, which is fair.

In addition to the vibration and acoustic performance of the CLT floor panels, the stiffness of the floor panels can also indicate the vibrations the floor will undergo due to the seismic performance of a building. The high in-plane and out-of-plane stiffness and strength in both the longitudinal and transverse directions of CLT panels are what makes CLT suitable for seismic resistant construction (Di Bella & Mitrovic, 2020). In order to properly provide seismic resistance, the CLT panels must limit the floor deflection due to a concentrated static force of 225 lbs (1 kN) to 0.0787 in (2 mm) (Hu & Gagnon, 2012; Lewis et al., 2016; Schirén & Swahn, 2019; Vassallo et al., 2018). Using the equation $\Delta = \frac{Fl^3}{48(EI)_l}$, where F is the concentrated static force, l is the length of the CLT floor panel, and $(EI)_l$ is the longitudinal elastic modulus, it was found that the typical floor used from the ground floor through the fourth floor would have a deflection of 0.0173 in (0.438 mm) and the floor used for the roof level would have a deflection of 0.0564 in (1.43 mm) (Lewis et al., 2016; Schirén & Swahn, 2019). Since both of these are less than the limit of 0.0787 in, the floors in the case study building are capable of adequately resisting seismic forces in terms of deflection. The stiffness of these CLT floor panels could be improved by using a stress grade other than E1 that has a higher longitudinal elastic modulus, or through the use of hold-down anchors or similar connections (Breneman, 2017; Vassallo et al., 2018).

The final major factor affecting the acoustic and vibration performance of CLT floor panels is the velocity and acceleration of the floor. Limiting the velocity and acceleration for CLT floor panels can also help with the seismic resistance of mass timber buildings (Arnold, 2004). The velocity and acceleration of the floors are affected by the damping ratio, the stiffness, and the excitation of the floor (Hu & Gagnon, 2012). The unit impulse velocity response was limited to $v \leq \beta^{(f_1\zeta-1)}$, where β is the point load deflection limit, f_1 is the fundamental natural frequency, ζ is the modal damping ratio, and $v = \frac{4(0.4+0.6n_{40})}{mbl+200}$, where m is the mass of the CLT floor panel, b is the width of the CLT floor panel, l is the length of the floor panel, and $n_{40} = \left(\left(\left(\frac{40}{f_1}\right)^2 - 1\right)\left(\frac{b}{l}\right)^4\left(\frac{(EI)_l}{(EI)_t}\right)\right)^{0.25}$, where $(EI)_l$ is the longitudinal modulus of elasticity and $(EI)_t$ is the transverse modulus of elasticity (Lewis et al., 2016; Schirén and Swahn, 2019). From this, it was found that the unit impulse velocity response for the typical floor used from the ground floor through the fourth floor would be 1.72 ft/s (0.523 m/s) limited by 6 ft/s (1.83 m/s) and the floor used for the roof level would be 5.28 ft/s (1.61 m/s) limited by 16.2 ft/s (4.95 m/s). Since both floors would have a unit impulse velocity response less than their limit, the floor would be able to properly resist seismic forces. While there was also acceleration criteria discussed, it only needs to be checked for floors with a fundamental natural frequency between 4 and 8 Hz (Schirén and Swahn, 2019).

A set of velocity criteria equations can be used to determine the seismic response modification factor, R, for the CLT floor panels. The R value can then be used to predict the floor performance level, in a similar way as the stiffness criteria (Schirén and Swahn, 2019). The R value for the typical floor used from the ground floor through the fourth floor would be 7.96, which translated to a floor performance level of Level II, which is great. The floor used for the roof level would have an R value of 14.2, which translated to a floor performance level of Level

IV, which is fair. The velocity and acceleration of the CLT floor panels could be improved by increasing the overall stiffness of the panels and improving connections between the panels and/or adding coatings to the floor panels to improve the damping ratio (Di Bella & Mitrovic, 2020; Hu & Gagnon, 2012). Based on the reviewed studies, the CLT floors selected for the mass timber design seem like they would perform fairly in terms of acoustic and vibration performance as well as seismic resistance. The stiffness, velocity, and acceleration of the floor panels throughout the building could be improved, but they do seem to follow the guidelines and requirements of the design methods presented in the various studies, as well as the codes discussed within the studies.

9.0 Conclusions

The four objectives of the project were completed. The first objective was completed through the design of a gut renovation of a five-story building in Boston, MA using CLT with mass timber elements. The second objective was completed by establishing a similar design utilizing a steel frame with a cast-in-place concrete slab on a metal deck. The third objective was completed by comparing the in-place cost, transportation, manufacturability, and constructability of the two design options. Finally, the fourth objective was completed through analyzing recent studies on the acoustic and vibration performance of CLT and evaluating how this could impact the mass timber design.

CLT did outmatch the steel design in terms of sustainability, manufacturability, and constructability, but due to the scarcity of mass timber suppliers and manufacturers in North America, the cost alone for the five-ply CLT, equating to $20 per square foot, exceeds the expected total cost of the steel design, which is approximately $15 per square foot. Because of this, it is currently unlikely for a CLT building with mass timber elements, like the case study building, to be selected over a steel frame design. However, once CLT has a wider spread throughout the United States, it is likely that more CLT manufacturers outside of Oregon will begin operation in the coming years due to an increase in demand. Once this happens, the material and transportation costs for CLT should decrease. Combining lower material and transportation costs with the already established sustainability, manufacturability, and constructability benefits will make CLT a very competitive option when compared to traditional building materials, such as steel.

After completing this project, some ideas and suggestions for future projects include surveying owners, developers, manufacturers, and contractors on their awareness and willingness to use CLT, completing a full acoustics and vibration design for the case study building, exploring the fire protection capabilities of CLT structures, and exploring the option of a hybrid mass timber and steel building. A survey for manufacturers and contractors could be conducted to help push awareness for CLT and can also assess how willing they are to begin using it. By using an already designed building, a full acoustic and vibration design of a CLT structure could be performed to find what elements of the building would be most affected and to evaluate how much the design changes based on acoustic and vibration performance would differ from the original design. Through the background research, the team found some information on the fire protection capabilities of CLT and how it can be a better option than traditional timber buildings in this regard. It would be interesting to evaluate the fire protection capabilities of CLT structures in comparison to other building materials, especially as the construction of extensive wood structures has been steered away in the past due to its flammability. Finally, a hybrid building utilizing both mass timber and steel elements could be designed to establish how the hybrid of the materials compare to a design using just one of the materials. This would reflect the established use of hybrid CLT high-rise buildings in Europe.

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Appendix A: Project Proposal

Cross-Laminated Timber

A Major Qualifying Project Proposal

Submitted on

October 5th, 2020

Submitted to:

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

Submitted by: Isaiah Aridou, Civil Engineering

Olivia Hauber, Civil Engineering

Sponsored by: Simpson Gumpertz & Heger

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Capstone Design Statement

To complete the Capstone Design aspect of this project, we will be designing a gut renovation of a five-story office building in Boston, MA. Two designs will be completed: one using cross-laminated timber (CLT) and one using steel. The designs will be analyzed and compared to help determine the effectiveness of CLT. We plan on addressing several real-world constraints while designing for this project.

Sustainability

To address the sustainability constraint of our capstone design, we will be creating two designs: one using CLT and one using steel. We will be focusing on CLT, which is a more sustainable alternative to other building materials, such as steel or concrete. Both designs will be analyzed for their sustainability using a number of factors such as CO_2 emissions and energy savings.

Economics

To address the economic constraint of our capstone design, we will be comparing the economical differences between the two designs. We will be using different cost parameters, such as the cost of the materials, manufacturing, transportation, labor, and estimated time of construction. Since there are far fewer CLT manufacturers in the United States than steel manufacturers, taking the cost of manufacturing and transportation of the materials into account is necessary to create a more complete comparison of the economic impact of our designs.

Health and Safety

To address the health and safety constraints of our capstone design, we will be addressing the safety concerns that come with the design of a multi-story office building made of CLT or steel. To create safe and realistic designs, we will be following the guidelines for CLT found in the *CLT Handbook*, the American National Standards Institute and APA - The Engineered Wood Association's *Standard for Performance-Rated Cross-Laminated Timber*, the American Wood Council's (AWC) *Manual for Engineered Wood Construction*, and the AWC's *National Design Specification for Wood Construction*. The steel design will follow the guidelines from the American Institute for Steel Construction's 15th edition of the *Steel Construction Manual*. Both designs will also follow the guidelines from the American Society of Civil Engineers' *Minimum Design Loads for Buildings and Other Structures 7-10*, the *International Building Code of 2015*, the *International Existing Building Code of 2015*, and all local building codes.

Ethics

To address the ethical constraint of our capstone design, we will be addressing ethical concerns throughout the project. We will be working ethically throughout this project and will follow the ethical guidelines put in place by the American Society of Civil Engineers. These guidelines include creating safe and sustainable structures, acting professionally and avoiding conflicts of interest, and treating everyone involved in the project fairly (American Society of Civil Engineers [ASCE], 2017).

Manufacturability and Constructability

To address the manufacturability and constructability constraints of our capstone design, we will be addressing the lack of knowledge and experience of CLT in North America. We will be using standard and readily available sections for both the CLT and steel designs. We will be taking into account the shortage of CLT manufacturers in the United States. We will also consider the lack of knowledge a construction team may have for working with CLT. In addition, we may make design decisions that use repetition and promote ease of construction. To address the regulations, design factors, and structural analysis, we will be referencing the *CLT Handbook*, the *International Building Code*, the *International Existing Building Code*, and the American Institute of Steel Construction's 15th edition of the *Steel Construction Manual*.

1.0 Introduction

Cross-laminated timber (CLT) is a relatively new building material that is gaining popularity across the globe. CLT was first introduced in Europe in the 1990s and spread to North America in the early 2000s. The spread of CLT was helped by the global interest in more sustainable construction, which is one of CLT's greatest advantages, along with its construction speed. The spread and use of CLT, however, has been much slower in North America than Europe. This has led to fewer manufacturers in North America and less research being conducted locally to help improve this new construction material. An aspect of CLT that still requires research is the acoustic and vibration performance as both areas still have many unknowns.

The goal of this project is to explore the effectiveness of CLT in New England. This will be done through a case study of a gut renovation of a five-story building in Boston, MA using CLT. The building was originally constructed in 1907 to be used by the New England Confectionery Company. We will be designing for the building to be completely renovated into an office building. This case study is based on a project being completed by Simpson Gumpertz & Heger (SGH). The four objectives that have been identified to complete this case study are:

Objective 1: Evaluate the Design Implications of CLT
Objective 2: Evaluate the Design Implications of Steel
Objective 3: Assess Acoustic and Vibration Design Alternatives
Objective 4: Compare the Design Solutions of CLT and Steel

Two designs will be completed in this case study: one using CLT and one using steel. This will allow for a comparison of the effectiveness of the two building materials. We will also design for the acoustic and vibration performance of CLT based on the current research that is being done in those areas. The results of this case study will allow the effectiveness of CLT to be explored, from the design to the cost to the manufacturability of the material.

2.0 Background

CLT is a prefabricated engineered wood panel that consists of multiple layers of panels that are stacked in alternating directions (APA - The Engineered Wood Association [APA], n.d.). The individual layers of CLT can be bonded together with a structural adhesive or metal fasteners. CLT is a relatively new construction material with its first introduction being in Austria, Germany, and Switzerland in the 1990s and was spread across Europe by the early 2000s (Greenspec, n.d.; North Carolina State University [NC State], n.d.). Although CLT was also introduced in North America in the early 2000s, it's spread and use in North America has been much slower than in Europe (Pei et al., 2016). Since CLT is a newer building material, there are still many unknowns, leading to questions about its effectiveness in comparison to other building materials, such as steel.

2.1 The Advantages and Disadvantages of CLT

One of the biggest disadvantages for CLT in North America has been its late introduction to the continent. With less time for CLT to establish itself in North America, there is a lack of tenured CLT manufacturers raising the issues of time and cost when working on CLT buildings within the United States. Another looming disadvantage is the lack of data supporting CLT (Robbins, 2019). Beverly Law, a professor of global change biology and terrestrial systems science at Oregon State University, recognizes the lack of analysis of carbon emitted by mass timber production since it is a huge and complex task to assess the factors of CO_2 produced in forest ecosystems as well as in production (Robbins, 2019).

A great advantage for CLT is its application in construction ranging from public to institutional use to even schools and multifamily buildings (reThink Wood). In the case of schools, CLT is especially helpful due to its prefabricated state when fitting a project into a time frame as short as the summer when students are away from school and still being able to finish within the timeframe. This shows how valuable CLT can be for projects of all sizes in reducing their duration significantly. As of 2018, there has been a looming boom for CLT manufacturing in the U.S. with: four factories in production, two of which are making architectural CLT five factories coming online, and three more announced across eight states (Jenkins, 2018).

2.2 Sustainability and Forestry

In recent years the need for green building materials has become a growing concern due to the rapid changing of Earth's climate. A good example of CLT's growing popularity and application to sustainability can be seen from the U.S Department of Defense's use of CLT for its on-base housing due to its general resilience and resistance to explosive forces (Jenkins, 2018). The Mjøstårnet is an example proving modern tall buildings can be built with green sustainable materials (Moelven, 2019). This Norwegian constructed building stands at 280 feet (85.4 meters) tall with 37,073 square feet (11,300 square meters) of space and boasts a hotel, apartments, offices, a restaurant, common areas, and even a swimming hall. This high-rise structure showcases how capable and versatile CLT can be in place of typical materials such as steel and concrete.

From an environmental standpoint, CLT has been viewed very positively as it can be seen as a solution to reducing carbon emissions (Sierra Club, 2019). This may be the hope but the need for timber will only rise with CLT's popularity and, if not managed properly, could lead to the deforestation of forests that store large amounts of carbon. As promoted by the Sierra Club to effectively counter this issue, proper forest stewardship and protection must be used. Concrete, for example, is one of the most highly used substances on the planet, second only to water, and is responsible for eight percent of global CO_2 emissions (Sierra Club, 2019). CLT can be seen as the rationale substitution to a building material such as concrete to help reduce a building's embodied carbon. Embodied carbon measures emissions from extraction, manufacturing, transporting, and the use of a building material which accounts for 10 percent of emissions globally using the life cycle assessment (LCA).

2.3 The Need for Research into the Acoustic and Vibration Performance of CLT

Due to CLT being a relatively new construction material not only in North America but also globally, there are quite a few areas that still require research to improve the performance of CLT. One such area is the acoustic and vibration performance of CLT. At present, the acoustic performance of CLT alone is not adequate. Since CLT is not as large or thick as a typical concrete slab or masonry wall, the acoustic separation between rooms and floors in CLT buildings is worse than buildings that use these traditional materials. The acoustic separation of CLT structures also does not currently meet the *International Building Code* (*IBC*) requirements on their own, with CLT having a sound transmission class (STC) of approximately 40 when the *IBC* requires an STC of at least 50 (Metropolitan Acoustics, 2019; The International Code Council [ICC], 2015). In order to comply with the *IBC*, additional barriers are typically used to enhance the acoustic properties of CLT. In hopes of improving the acoustic properties of CLT, research has been and continues to be conducted worldwide. In 2016, Antonio Di Bella, Nicola Granzotto, and Luca Barbaresi conducted an experiment to identify a spectrum of the normal impact sound pressure level of a CLT floor in order to create a tool that allows estimations of the noise insulation of a CLT floor (Di Bella, Granzotto, & Barbaresi, 2016). In 2013, Mariana Perez and Marta Fuente conducted research on a two-story experimental facility to create a predictive model of the acoustic behavior of CLT structures (Perez & Fuente, 2013). These studies, along with other research being conducted, look to better understand acoustics in relation to CLT and how the design of CLT can be adjusted to improve its acoustic properties.

Research is also being conducted into the vibration and seismic resistance of CLT structures. Traditional lightweight joisted wood flooring systems are typically smaller and lighter than CLT floors, while typical concrete slabs are heavier and larger. This indicates that the fundamental frequency of CLT should be between the fundamental frequency of lightweight floors of greater than 15 Hz and the fundamental frequency of concrete slabs of less than nine Hz, which was confirmed through tests run by FPInnovations (Hu & Gagnon, 2012; Pirvu, 2015). Based on CLT's fundamental frequency being between the fundamental frequency of lightweight floors and concrete slabs, it has been determined that the current standards for the vibration design of lightweight and heavy floors are not adequate for CLT floors. This has led many to conduct research on how to design CLT floors for vibrations. Research is also being conducted into the seismic resistance of CLT. CLT has been increasingly used for floor diaphragms and shear walls to provide better seismic resistance for buildings. Due to this, research is being conducted globally to determine how CLT can be used to strengthen new and existing structures against seismic activity. In 2012, Lin Hu and Sylvain Gagnon conducted research to better predict the vibration performance of CLT floors as the existing design methods for lightweight and heavy floors are not applicable to CLT floors. Through this study, a new design method for floor vibrations was created for CLT floors, which can be used to provide better vibration and seismic performance within CLT structures (Hu & Gagnon, 2012). Other research, however, has found that there are currently too many unknowns with CLT since it is a relatively new building material, indicating that more research is needed into CLT as a material and its relation to seismic resistance.

2.4 Design Standards and Specifications

The introduction of CLT in North America has led to its inclusion in several engineering publications and building codes that will be used throughout this report. These include the *CLT Handbook*, the American National Standards Institute and APA - The Engineered Wood Association's *Standard for Performance-Rated Cross-Laminated Timber* (*ANSI-APA PRG*), the American Wood Council's (AWC) *Manual for Engineered Wood Construction* (*AWC-2018*), and the AWC's *National Design Specification for Wood Construction* (*AWC-NDS*). The report will also refer to the American Society of Civil Engineers' *Minimum Design Loads for Buildings and Other Structures* (*ASCE 7-10*), the *IBC* of 2015 (*IBC-2015*), and the *International Existing Building Code* of 2015 (*IEBC-2015*) for the design requirements of CLT and the building codes. The design requirements of steel will be referenced from the American Institute for Steel Construction's 15th edition of the *Steel Construction Manual* (*AISC-15*).

3.0 Methodology

Goal: To address the effectiveness of a CLT design for the renovation of a future office building to be used by SGH.

3.1 Objective 1: Evaluate the Design Implications of Steel

<table>
<tr><th>Steps</th><th>Scope</th><th>References</th></tr>
<tr><td>Design a steel frame</td><td>Design steel frame based on floor plans provided by SGH
Design for similar structural members (girders, columns, etc.)
Design for gravitational and vertical loads of the building, including self-weight
Design for lateral load resistance
Complete a load takedown for the foundation and masonry exterior of the building and adjust the design as needed
Complete design calculations
Will use design software (ie. RISA, AutoCAD, Excel, etc.) to help ensure correct calculations</td><td>Floor plans of the building provided by SGH
United States and local design requirements and building codes
AISC-15
ASCE 7-10
IBC-2015
IEBC-2015
Level 3 Alteration</td></tr>
</table>

3.2 Objective 2: Evaluate the Design Implications of CLT

Steps	Scope	References

Design CLT renovation	• Design all CLT walls, floors, etc. based on floor plans provided by SGH
	    ○ Design for gravitational and vertical loads of the building, including self-weight
	    ○ Design for lateral load resistance
	    ○ Complete a load takedown for the foundation and masonry exterior of the building and adjust the design as needed
	    ○ Use glulam for beam and column design
	• Complete design calculations
	    ○ Will use design software (ie. RISA, AutoCAD, Excel, etc.) to help ensure correct calculations
		• Floor plans of the building provided by SGH
		• United States and local design requirements and building codes
		    ○ *CLT Handbook*
		    ○ *ANSI-APA PRG*
		    ○ *AWC-2018*
		    ○ *AWC-NDS*
		    ○ *ASCE 7-10*
		    ○ *IBC-2015*
		    ○ *IEBC-2015*
		        ▪ Level 3 Alteration
		• Design Example
		    ○ (Brandner, Flatscher, Ringhofer, Schickhofer, & Thiel, 2016)

3.3 Objective 3: Assess Acoustic and Vibration Design Alternatives

Steps	Scope	References

Design for acoustic/vibrations for CLT	• Design for acoustics/vibrations for CLT based on design examples and reference calculations	• *CLT Handbook*
		• Design Examples
		    ○ (Bella & Mitrovic, 2020)
		    ○ (Vassallo, Follesa, & Fragiacomo, 2018)

		○ (Hu & Gagnon, 2012) ○ (Lewis, Basaglia, Shrestha, & Crews, 2016)

3.4 Objective 4: Compare the Design Solutions of CLT and Steel

Steps	Scope	References
Compare the CLT and Steel Designs	• Complete a cost analysis for both designs and compare them • Compare the manufacturability and constructability of the designs	• CLT and steel design results

Proposed Project Schedule

	Term:	A		B									C							
Task Schedule for MQP	Week of:	10/12	Break	10/21	10/26	11/2	11/9	11/16	11/23	11/30	12/7	Break	1/13	1/18	1/25	2/1	2/8	2/15	2/22	3/1
1. Design																				
CLT:																				
Finalize Layout and Loadpath																				
Gravitational Loads																				
Lateral Loads																				
Load Takedown for Foundation and Masonry																				
Steel:																				
Finalize Layout and Loadpath																				
Gravitational Loads																				
Lateral Loads																				
Load Takedown for Foundation and Masonry																				
2. Acoustics & Vibrations																				
Design for Acoustics																				
Design for Vibrations																				
Potential Lit Review																				
3. Design Evaluations																				
Cost Analysis for CLT																				
Cost Analysis for Steel																				
Compare Cost Analyses																				
Compare Manufacturability & Constructability of Both Designs																				
4. Additional Tasks																				
End of B Term Report											12/11									
Finalize Project																				
Final Report																				
Drafts of Final Report																2/5		2/19		
Submit Final Report																				3/5
Final Recommendations to Sponsor																				
Recommendations for Future MQP																				
Final Poster																				
Complete Final Poster																				3/5

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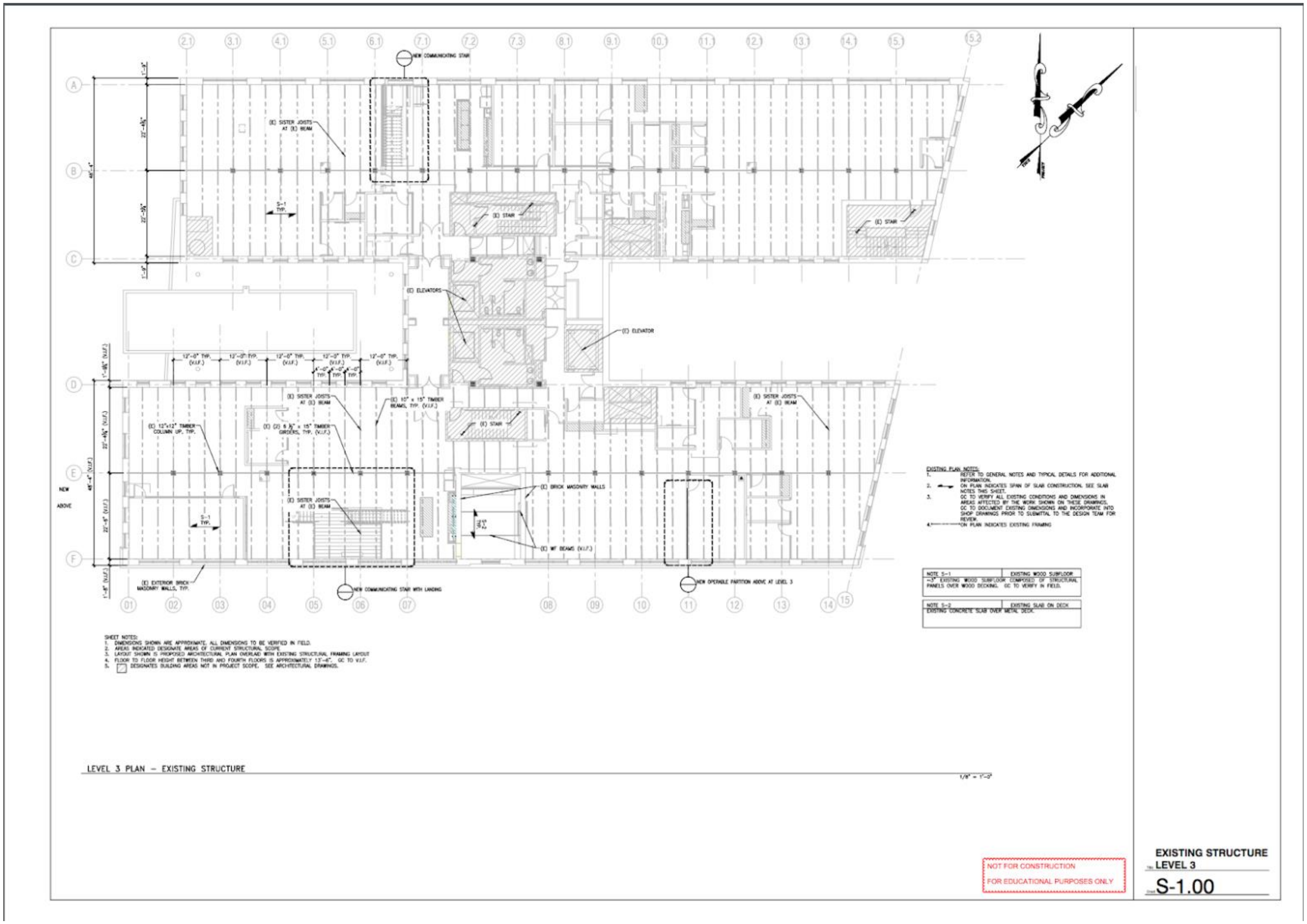
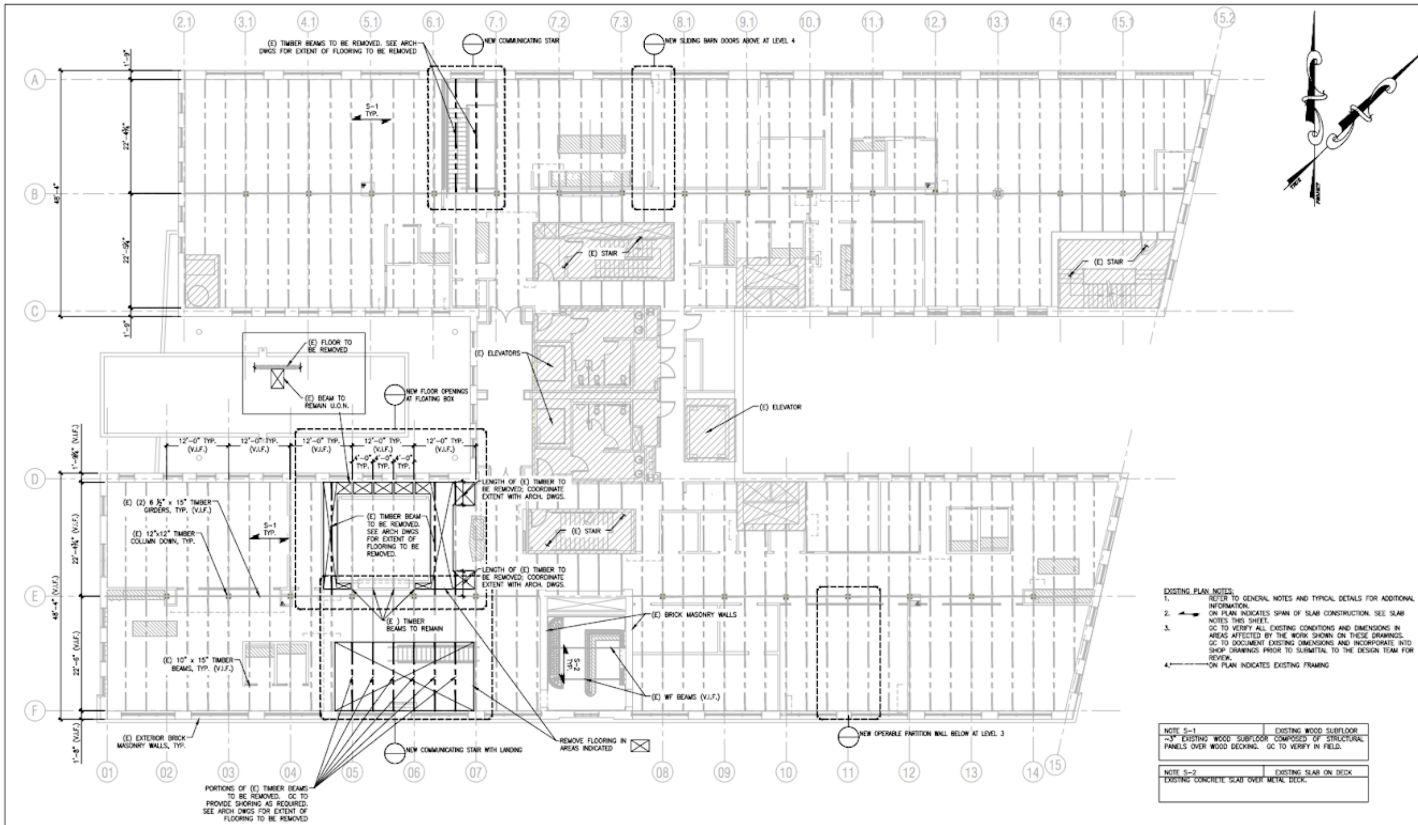
Appendix B: The Floorplans of the Case Study Building





EXISTING PLAN NOTES:

1. REFER TO GENERAL NOTES AND TYPICAL DETAILS FOR ADDITIONAL INFORMATION.
2. ON PLAN INDICATES SPAN OF SLAB CONSTRUCTION. SEE SLAB NOTES THIS SHEET.
3. GC TO VERIFY ALL EXISTING CONDITIONS AND DIMENSIONS IN AREAS AFFECTED BY THE WORK SHOWN ON THESE DRAWINGS. GC TO DOCUMENT EXISTING DIMENSIONS AND INCORPORATE INTO SHOP DRAWINGS PRIOR TO SUBMITTAL TO THE DESIGN TEAM FOR REVIEW.
4. ON PLAN INDICATES EXISTING FRAMING

NOTE S-1	EXISTING WOOD SUBFLOOR
~3" EXISTING WOOD SUBFLOOR COMPOSED OF STRUCTURAL PANELS OVER WOOD DECKING. GC TO VERIFY IN FIELD.	

NOTE S-2	EXISTING SLAB ON DECK
EXISTING CONCRETE SLAB OVER METAL DECK.	

SHEET NOTES:

1. DIMENSIONS SHOWN ARE APPROXIMATE. ALL DIMENSIONS TO BE VERIFIED IN FIELD.
2. AREAS INDICATED DESIGNATE AREAS OF CURRENT STRUCTURAL SCOPE
3. LAYOUT SHOWN IS PROPOSED ARCHITECTURAL PLAN OVERLAID WITH EXISTING STRUCTURAL FRAMING LAYOUT
4. FLOOR TO FLOOR HEIGHT BETWEEN THIRD AND FOURTH FLOORS IS APPROXIMATELY 13'-6". GC TO V.I.F.
5. DESIGNATES BUILDING AREAS NOT IN PROJECT SCOPE. SEE ARCHITECTURAL DRAWINGS.

LEVEL 4 PLAN – EXISTING STRUCTURE

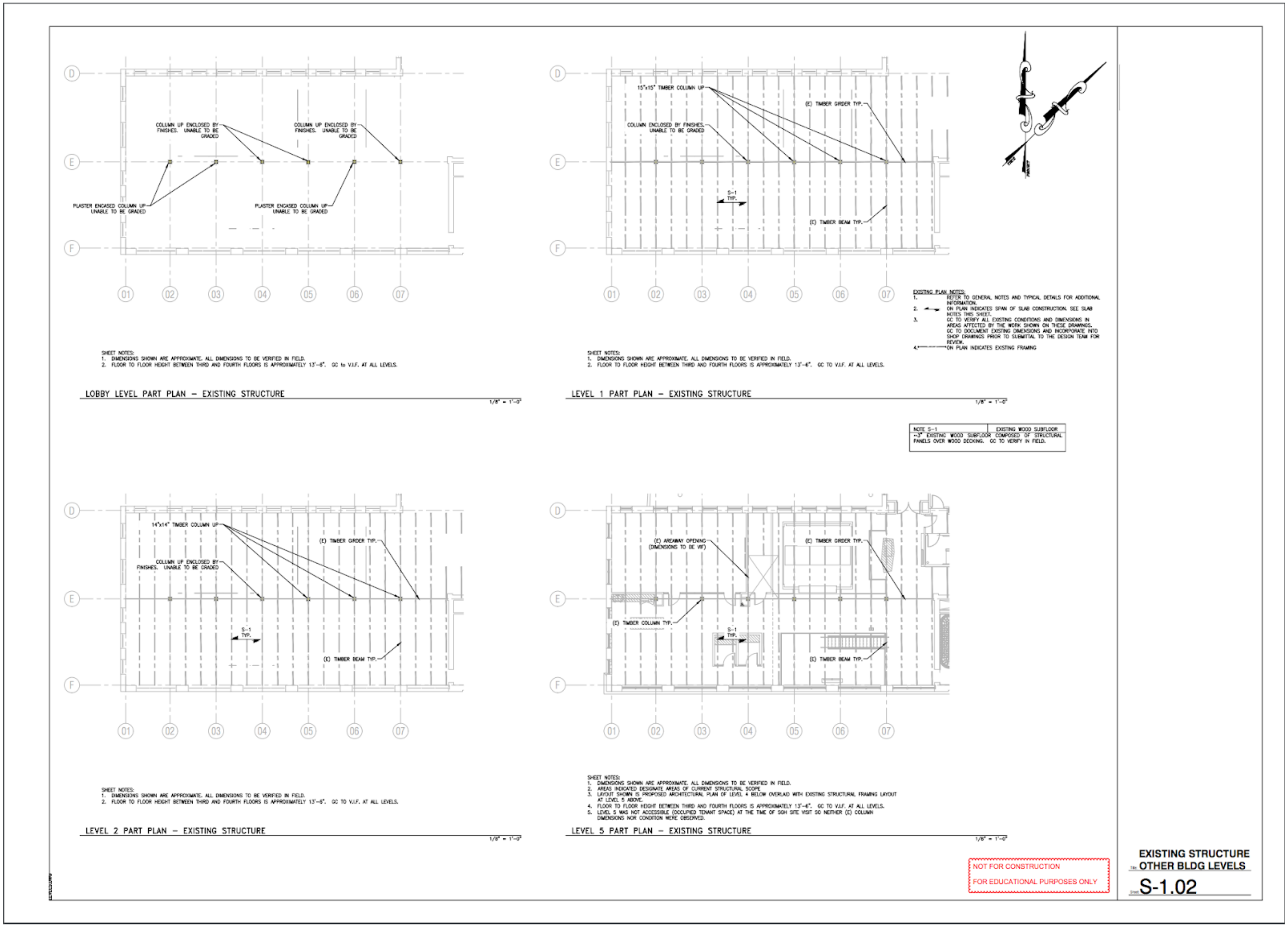
1/8" = 1'-0"

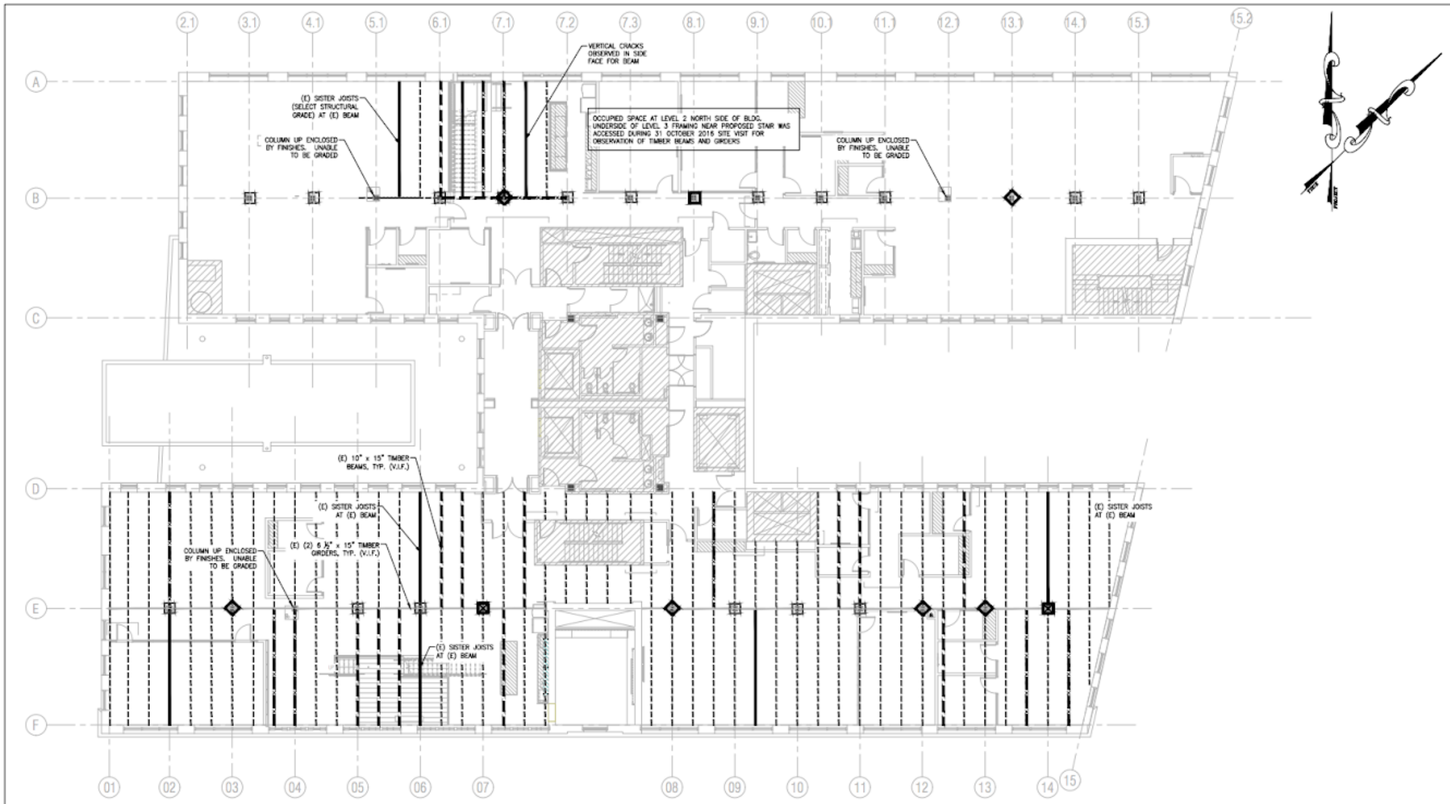
NOT FOR CONSTRUCTION

FOR EDUCATIONAL PURPOSES ONLY

EXISTING STRUCTURE AND DEMO - LEVEL 4

S-1.01


COLUMN UP ENCLOSED BY FINISHES. UNABLE TO BE GRADED
PLASTER ENCASED COLUMN UP UNABLE TO BE GRADED
SHEET NOTES:
1. DIMENSIONS SHOWN ARE APPROXIMATE. ALL DIMENSIONS TO BE VERIFIED IN FIELD.
2. FLOOR TO FLOOR HEIGHT BETWEEN THIRD AND FOURTH FLOORS IS APPROXIMATELY 13'-6". GC to V.I.F. AT ALL LEVELS.
LOBBY LEVEL PART PLAN – EXISTING STRUCTURE
1/8" = 1'-0"
15"x15" TIMBER COLUMN UP
(E) TIMBER GIRDER TYP.
COLUMN ENCLOSED BY FINISHES. UNABLE TO BE GRADED
S-1 TYP.
(E) TIMBER BEAM TYP.
EXISTING PLAN NOTES:
1. REFER TO GENERAL NOTES AND TYPICAL DETAILS FOR ADDITIONAL INFORMATION.
2. ON PLAN INDICATES SPAN OF SLAB CONSTRUCTION. SEE SLAB NOTES THIS SHEET.
3. GC TO VERIFY ALL EXISTING CONDITIONS AND DIMENSIONS IN AREAS AFFECTED BY THE WORK SHOWN ON THESE DRAWINGS. GC TO DOCUMENT EXISTING DIMENSIONS AND INCORPORATE INTO SHOP DRAWINGS PRIOR TO SUBMITTAL TO THE DESIGN TEAM FOR REVIEW.
4. ON PLAN INDICATES EXISTING FRAMING
SHEET NOTES:
1. DIMENSIONS SHOWN ARE APPROXIMATE. ALL DIMENSIONS TO BE VERIFIED IN FIELD.
2. FLOOR TO FLOOR HEIGHT BETWEEN THIRD AND FOURTH FLOORS IS APPROXIMATELY 13'-6". GC TO V.I.F. AT ALL LEVELS.
LEVEL 1 PART PLAN – EXISTING STRUCTURE
1/8" = 1'-0"
NOTE S-1 | EXISTING WOOD SUBFLOOR
~3" EXISTING WOOD SUBFLOOR COMPOSED OF STRUCTURAL PANELS OVER WOOD DECKING. GC TO VERIFY IN FIELD.
14"x14" TIMBER COLUMN UP
(E) TIMBER GIRDER TYP.
COLUMN UP ENCLOSED BY FINISHES. UNABLE TO BE GRADED
S-1 TYP.
(E) TIMBER BEAM TYP.
SHEET NOTES:
1. DIMENSIONS SHOWN ARE APPROXIMATE. ALL DIMENSIONS TO BE VERIFIED IN FIELD.
2. FLOOR TO FLOOR HEIGHT BETWEEN THIRD AND FOURTH FLOORS IS APPROXIMATELY 13'-6". GC TO V.I.F. AT ALL LEVELS.
LEVEL 2 PART PLAN – EXISTING STRUCTURE
1/8" = 1'-0"
(E) AREAWAY OPENING (DIMENSIONS TO BE VIF)
(E) TIMBER GIRDER TYP.
(E) TIMBER COLUMN TYP.
S-1 TYP.
(E) TIMBER BEAM TYP.
SHEET NOTES:
1. DIMENSIONS SHOWN ARE APPROXIMATE. ALL DIMENSIONS TO BE VERIFIED IN FIELD.
2. AREAS INDICATED DESIGNATE AREAS OF CURRENT STRUCTURAL SCOPE
3. LAYOUT SHOWN IS PROPOSED ARCHITECTURAL PLAN OF LEVEL 4 BELOW OVERLAID WITH EXISTING STRUCTURAL FRAMING LAYOUT AT LEVEL 5 ABOVE.
4. FLOOR TO FLOOR HEIGHT BETWEEN THIRD AND FOURTH FLOORS IS APPROXIMATELY 13'-6". GC TO V.I.F. AT ALL LEVELS.
5. LEVEL 5 WAS NOT ACCESSIBLE (OCCUPIED TENANT SPACE) AT THE TIME OF SGH SITE VISIT SO NEITHER (E) COLUMN DIMENSIONS NOR CONDITION WERE OBSERVED.
LEVEL 5 PART PLAN – EXISTING STRUCTURE
1/8" = 1'-0"
NOT FOR CONSTRUCTION
FOR EDUCATIONAL PURPOSES ONLY
EXISTING STRUCTURE
OTHER BLDG LEVELS
S-1.02



LEVEL 3 PLAN – TIMBER GRADING

1/8" = 1'-0"

SHEET NOTES:

1. OBSERVATIONS SHOWN WERE MADE DURING SGH SITE VISITS ON DATES 14 AND 30 SEPTEMBER 2016 AND 31 OCTOBER 2016
2. THE VISUAL GRADING OF EXISTING TIMBERS WAS BASED ON OBSERVATIONS OF ALL ACCESSIBLE TIMBER SURFACES AT THE TIME OF SGH SITE VISITS. OBSTRUCTED TIMBERS AND INACCESSIBLE SURFACES COULD NOT BE GRADED OR GRADED FULLY. EXISTING TIMBERS COULD ONLY BE REVIEWED BASED ON EXPOSED TIMBER FACES. PORTIONS OF TIMBERS ENCLOSED OR COVERED BY FINISHES OR OTHER MATERIAL ARE NOTED TO BEST EXTENT POSSIBLE.
3. ALL TIMBERS ASSUMED TO BE SOUTHERN PINE SPECIES BASED ON PREVIOUS TESTING ON LIMITED MATERIAL SAMPLE.
4. TIMBER VISUAL GRADING BASED ON THE 2014 STANDARD GRADING RULES FOR SOUTHERN PINE LUMBER BY THE SOUTHERN PINE INSPECTION BUREAU.
5. LAYOUT SHOWN IS PROPOSED ARCHITECTURAL PLAN OVERLAID WITH EXISTING STRUCTURAL FRAMING LAYOUT
6. BEAMS SHOWN ARE VISIBLE WHEN STANDING ON LEVEL 2 AND LOOKING UP. COLUMNS SHOWN ARE VISIBLE WHEN STANDING ON LEVEL 3.
7. DESIGNATES BUILDING AREAS NOT IN PROJECT SCOPE. SEE ARCHITECTURAL DRAWINGS.

REPAIR SCOPE:

SISTER ALL BEAMS OF THIS GRADE ON PLAN PER DETAIL 1/S4.00

SEE S2.00 FOR OTHER BEAMS TO BE STRENGTHENED

TOTAL NUMBER OF BEAMS TO BE STRENGTHENED ON THIS LEVEL IS 28 UNLESS BEAM HAS ALREADY BEEN SISTERED PREVIOUSLY

SEE S2.00 FOR GIRDERS TO BE STRENGTHENED

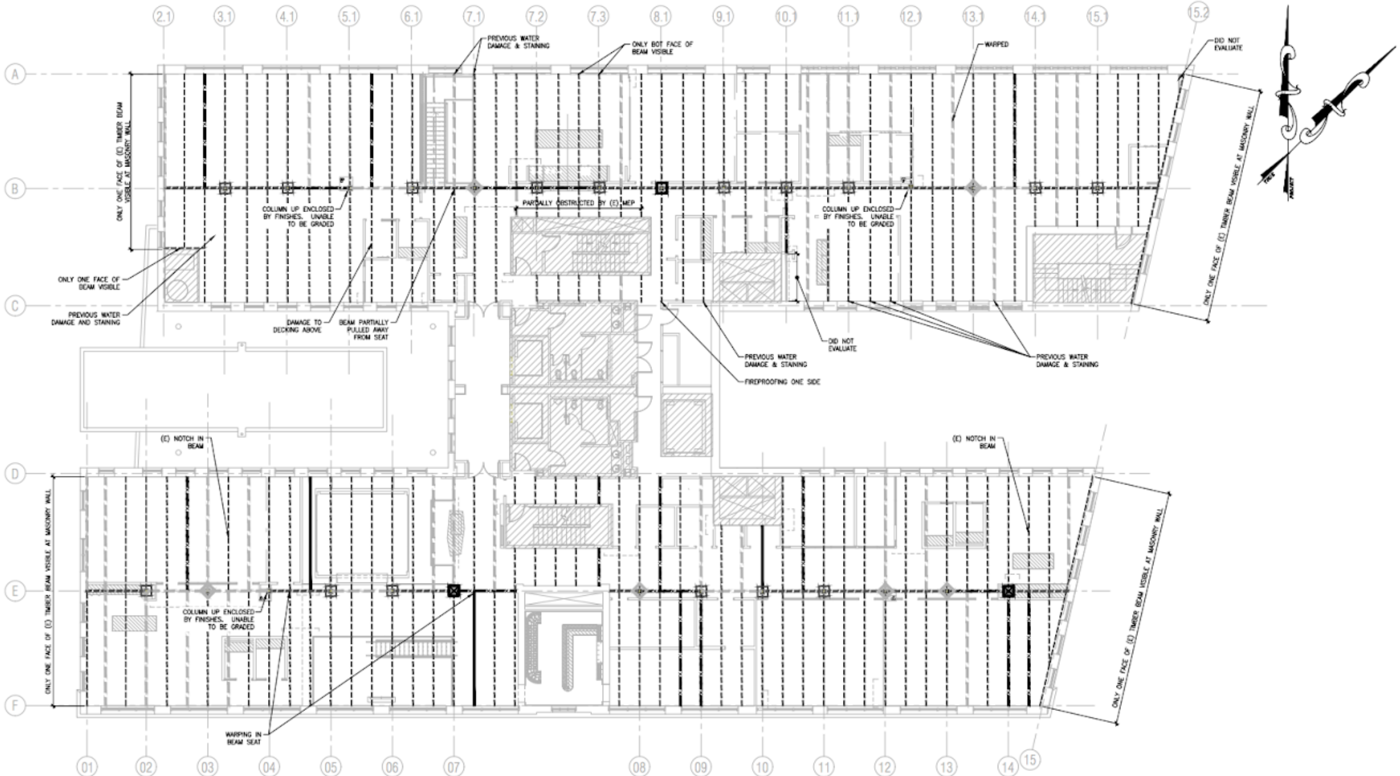
SEE S2.00 AND S2.02 FOR COLUMNS TO BE STRENGTHENED

GRADING LEGEND:	SUMMARY: BEAMS		COLUMNS		GIRDERS	
SELECT STRUCTURAL TIMBER GRADE	78	73%	16	57%	1	6%
NO. 1 TIMBER GRADE	12	11%	6	21%	1	6%
NO. 2 TIMBER GRADE	8	8%	2	7%	1	6%
NO. 3 TIMBER GRADE	8	8%	1	4%	0	0%
UNGRADED	0	0%	3	11%	14	82%
TOTAL	106	100%	28	100%	17	100%

NOT FOR CONSTRUCTION
FOR EDUCATIONAL PURPOSES ONLY

TIMBER GRADING
LEVEL 3

S-1.10



SHEET NOTES:
1. OBSERVATIONS SHOWN WERE MADE DURING SGH SITE VISITS ON DATES 14 AND 30 SEPTEMBER 2016
2. THE VISUAL GRADING OF EXISTING TIMBERS WAS BASED ON OBSERVATIONS OF ALL ACCESSIBLE TIMBER SURFACES AT THE TIME OF SGH SITE VISITS. OBSTRUCTED TIMBERS AND INACCESSIBLE SURFACES COULD NOT BE GRADED OR GRADED FULLY. EXISTING TIMBERS COULD ONLY BE REVIEWED BASED ON EXPOSED TIMBER FACES. PORTIONS OF TIMBERS ENCLOSED OR COVERED BY FINISHES OR OTHER MATERIAL ARE NOTED TO BEST EXTENT POSSIBLE.
3. EXISTING TIMBERS COULD ONLY BE REVIEWED BASED ON EXPOSED TIMBER FACES. PORTIONS OF TIMBERS ENCLOSED OR COVERED BY FINISHES OR OTHER MATERIAL ARE NOTED.
4. ALL TIMBERS ASSUMED TO BE SOUTHERN PINE SPECIES BASED ON PREVIOUS TESTING ON LIMITED MATERIAL SAMPLE.
5. TIMBER VISUAL GRADING BASED ON THE 2014 STANDARD GRADING RULES FOR SOUTHERN PINE LUMBER BY THE SOUTHERN PINE INSPECTION BUREAU.
6. LAYOUT SHOWN IS PROPOSED ARCHITECTURAL PLAN OVERLAID WITH EXISTING STRUCTURAL FRAMING LAYOUT
7. BEAMS SHOWN ARE VISIBLE WHEN STANDING ON LEVEL 2 AND LOOKING UP. COLUMNS SHOWN ARE VISIBLE WHEN STANDING ON LEVEL 3.
8. DESIGNATES BUILDING AREAS NOT IN PROJECT SCOPE. SEE ARCHITECTURAL DRAWINGS.

REPAIR SCOPE:

SISTER ALL BEAMS OF THIS GRADE ON PLAN PER DETAIL 1/S4.00

SEE S2.01 FOR OTHER BEAMS TO BE STRENGTHENED

TOTAL NUMBER OF BEAMS TO BE STRENGTHENED ON THIS LEVEL IS 16

SEE S2.01 FOR GIRDERS TO BE STRENGTHENED

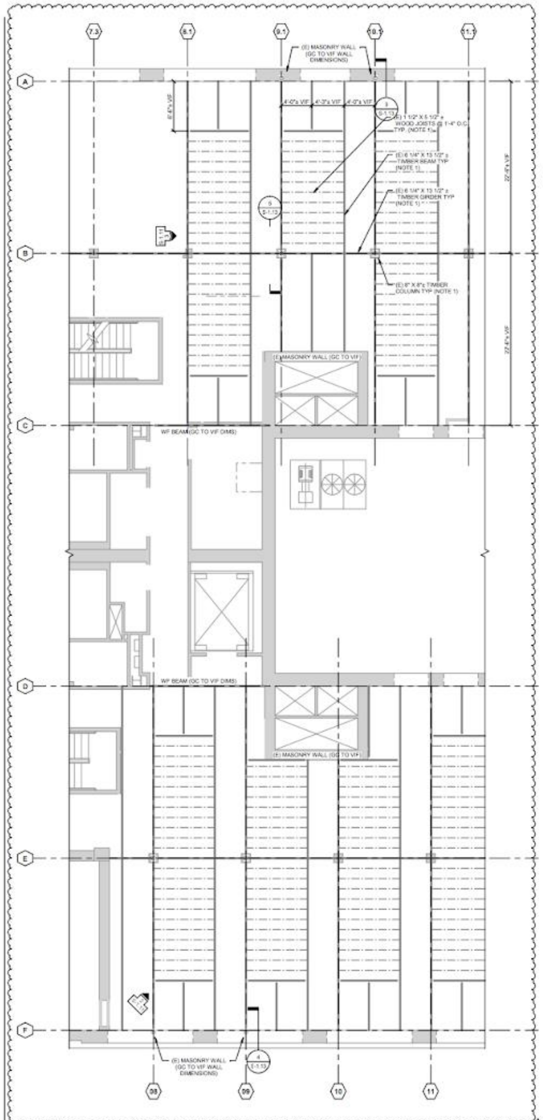
GRADING LEGEND:	SUMMARY					
	BEAMS		COLUMNS		GIRDERS	
SELECT STRUCTURAL TIMBER GRADE	150	73%	16	57%	19	61%
NO. 1 TIMBER GRADE	36	17%	6	21%	6	19.5%
NO. 2 TIMBER GRADE	13	6%	2	7%	6	19.5%
NO. 3 TIMBER GRADE	3	2%	1	4%	0	0%
UNGRADED	4	2%	3	11%	0	0%
TOTAL	206	100%	28	100%	31	100%

LEVEL 4 PLAN – TIMBER GRADING

1/8" = 1'-0"

NOT FOR CONSTRUCTION
FOR EDUCATIONAL PURPOSES ONLY

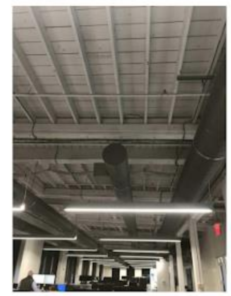
TIMBER GRADING
LEVEL 4
S-1.11



1 ROOF PART PLAN
3/16" = 1'-0"

PLAN NOTES:
1. ALL EXISTING FRAMING SHOWN IS BASED ON LIMITED SITE OBSERVATIONS MADE BY SGH ON 09 DECEMBER 2019. MANY AREAS OF ROOF FRAMING WERE OBSTRUCTED FROM VIEW BY EXISTING TENANT CEILING FINISHES AND PARTITIONS AT LEVEL 5 BELOW. PRIOR TO FABRICATION AND CONSTRUCTION, THE GC IS TO V/F ALL EXISTING CONDITIONS SHOWN IN THE AREA OF THE NEW RTU SUPPORTS. GC IS TO NOTIFY THE EOR IF EXPOSED EXISTING FRAMING VARIES FROM CONDITIONS SHOWN ON PLAN.
2. IN THE ABSENCE OF VISUAL STRESS GRADING INFORMATION, THE EXISTING ROOF TIMBERS ARE ASSUMED TO BE SOUTHERN PINE (SPIB) NO. 3 GRADE.
3. SEE ARCHITECTURAL AND MECHANICAL DRAWINGS FOR INFORMATION ON RTU DIMENSIONS AND LOCATIONS.
4. "⊗" INDICATES LOADING POINT FOR RTU DUNNAGE ONTO EXISTING BUILDING. GC IS TO ALIGN RTU LOAD POINTS OVER EXISTING TIMBER COLUMNS AND MASONRY WALLS BELOW.

2 EXISTING CONDITION - ROOF (EAST WING)
1/8" = 1'-0"



3 EXISTING CONDITION - ROOF (WEST WING)
1/8" = 1'-0"

NOT FOR CONSTRUCTION
FOR EDUCATIONAL PURPOSES ONLY

Appendix C: Mass Timber Design Calculations

Dead Load Breakdown

The uniform dead load throughout the building included an estimation for mechanical, electrical, and plumbing systems (MEP), hung ceilings and finishes, and the self-weight of the CLT floors. An assumption of five pounds per square foot was made for the MEP and hung ceilings and finishes (“Structural Loads”, n.d.). The CLT floor panels had an additional assumed dead load of three pounds per square foot for a hardwood finish on the floor. The dead load of the CLT floor panels was calculated to be approximately 21 pounds per square foot for the five-ply panels and approximately 12 pounds per square foot for the three-ply panels.

Beam Design

Attached Lobby

		Units	Equation	Notes		
C3.1-D3.1						
24F-ES/NPG						
Loadings	Tributary width	12	ft			
DL	25.6	psf	307	plf	5 psf for MEP + 20.6 psf for CLT Floor	
LL	100	psf	1200	plf		
S	40	psf	480	plf	Ground	
R			snow governs			
W	36.5	psf	493	plf	Boston=128 mph for Risk Category II	
E			14.3	plf	Eh+Ev	Risk Category II, Soil Site Class D, Design Category B
Eh	0.03851	lbs	rho*Qe	Rho= 1.0		
Ev	14	plf	.2*SDs*DL			
Qe	38.51	kips	Fx+V			
V	36.48	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	84.39	kips				
Cs	0.093					
Cs max	0.081			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	2.5					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.231					
SD1	0.109					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2.5					
Cd	2.5					
k	1.02					
hx^k	14.2	ft				
Wx*h^k	1200	kip-ft				
Cvx	0.056					
Fx	2.03	kips				
Load Combinations						
DL	307	plf				
DL+LL	1507	plf				
DL+S	787	plf				
DL+.75LL+.75S	1567	plf				
DL+.6W	603	plf				

DL+.75LL+.75(.6W)+.75S	1789	plf		Controlling Load Combination		
DL+.75LL+.75(.7E)+.75S	1575	plf				
.6DL+.6W	480	plf				
.6DL+.7E	194	plf				
Iteration 1						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	0.99		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV		
CV	0.91		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL		
CP	0.10		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	32.0	ft		32'		
b	17.625	in				
d	15.1	in				
Ag	266.58	in^2	b*d			
Sxx	672.00	in^3	(b*d^2)/6			
Ixx	5081.99	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	36831	psi	(1.2Emin')/((RB)^2)			
RB	5.72		sqrt((le*d)/((b)^2))			
le	671.30	in	1.63lu+3d	b/c lu/d>7		
lu	384	in				
lu/d	25.39					
le/d	44.4		<= 50 so good			
To Find CP						

Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	419	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3646	psi	Fb*CD*CM*Ct*CV*Cfu*Cc*Cl			
Bending Capacity						
w(beam weight)	67	plf				
wu	1789	plf		Using controlling load combination		
M	2850174	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	781.6	in^3	M/Fb'	S<S(req'd) so need new size		
Iteration 2						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV		
CV	0.90		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL		
CP	0.11		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			

Length (l)	32.0	ft		32'		
b	19.75	in				
d	15.9	in				
Ag	313.53	in^2	b*d			
Sxx	829.55	in^3	(b*d^2)/6			
Ixx	6584.56	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*CI			
FbE	43916	psi	(1.2Emin')/((RB)^2)			
RB	5.24		sqrt((le*d)/((b)^2))			
le	673.55	in	1.63lu+3d	b/c lu/d>7		
lu	384	in				
lu/d	24.19					
le/d	42.4		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	458	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3617	psi	Fb*CD*CM*Ct*CV*Cfu*Cc*CI			
Bending Capacity						
w(beam weight)	78	plf				
wu	1789	plf		Using controlling load combination		
M	2868204	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	793.0	in^3	M/Fb'	S>S(req'd) so good		
M*	3693709.651	lb-in	Fb*Sx			
M'	3333849	lb-in	M* *CV	M'>=M max		
Deflection						
wLL	1200	plf				

Delta LL	2.263	in	(5wLL*l^4)/(384E'*I)	Delta LL>1" so need new size		
L/360	1.067	in				
Req'd Ix	13970	in^4				
Iteration 3						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV		
CV	0.88		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL		
CP	0.19		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	32.0	ft		32'		
b	23.75	in				

d	21.8	in		
Ag	516.56	in^2	b*d	
Sxx	1872.54	in^3	(b*d^2)/6	
Ixx	20363.86	in^4	(b*d^3)/12	
To Find CL				
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl	
FbE	45170	psi	(1.2Emin')/((RB)^2)	
RB	5.16		sqrt((le*d)/((b)^2))	
le	691.17	in	1.63lu+3d	b/c lu/d>7
lu	384	in		
lu/d	17.66			
le/d	31.8		<= 50 so good	
To Find CP				
Fc*	4308	psi	Fc*CD*CM*Ct	
FcE	817	psi	(.822Emin')/((le/d)^2)	
c	0.9			
Fb'	3528	psi	Fb*CD*CM*Ct*CV*Cfu*Cc*Cl	
Bending Capacity				
w(beam weight)	129	plf		
wu	1789	plf		Using controlling load combination
M	2946168	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8	
S(req'd)	835.1	in^3	M/Fb'	S>S(req'd) so good
M*	8337777.909	lb-in	Fb*Sx	
M'	7339921	lb-in	M* *CV	M'>=M max
Deflection				
wLL	1200	plf		

Delta LL	0.732	in	(5wLL*l^4)/(384E'*I)	Delta LL<L/360 so good		
L/360	1.067	in				
wDL	307	plf				
Delta DL	0.187	in	(5wDL*l^4)/(384E'*I)			
Delta DL+LL	0.919	in		Delta DL+LL<L/240 so good		
L/240	1.60	in				
Shear Capacity						
P	66745.51176	lb	4*(M allowable/l)			
M allowable	6407569.129	lb-in	M max-M beam wgt			
M max	6605929.129	lb-in	Fb'*Sx			
M beam wgt	198360	lb-in	(w beam wgt*l^2)/8			
V	33372.75588	lb	P/2			
fv	97	psi	(3V)/(2b*d)			
Fv'	235	psi	Fv*CD*CM*Ct*Cvr	fv<Fv' so good		

Ground Floor Through 4th Floor

Typical North-South Direction Beams

		Units	Equation	Notes		
D2-E2						
24F-ES/NPG						
Loadings	Tributary width	12	ft			
DL	25.6	psf	308	plf	5 psf for MEP + 20.6 psf for CLT Floor	
LL	100	psf	1200	plf		
S	40	psf	480	plf	Ground	
R			snow governs			
W	36.5	psf	493	plf	Boston=128 mph for Risk Category II	
E			14.3	plf	Eh+Ev	Risk Category II, Soil Site Class D, Design Category B
Eh	0.03851	lbs	rho*Qe	Rho= 1.0		
Ev	14	plf	.2*SDs*DL			
Qe	38.51	kips	Fx+V			
V	36.48	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	84.39	kips				
Cs	0.093					
Cs max	0.081			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	2.5					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.231					
SD1	0.109					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2.5					
Cd	2.5					
k	1.02					
hx^k	14.2	ft				
Wx*h^k	1200	kip-ft				
Cvx	0.056					
Fx	2.03	kips				
Load Combinations						
DL	308	plf				
DL+LL	1508	plf				
DL+S	788	plf				
DL+.75LL+.75S	1568	plf				
DL+.6W	603	plf				

DL+.75LL+.75(.6W)+.75S	1789	plf		Controlling Load Combination		
DL+.75LL+.75(.7E)+.75S	1575	plf				
.6DL+.6W	480	plf				
.6DL+.7E	194	plf				
Iteration 1						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	0.15		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Applies b/c less than CV		
CV	1.07		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Doesn't apply b/c greater than CL		
CP	0.08		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	22.40	ft		22' 4 3/4"		
b	1.5	in			guess	
d	9.38	in			guess	
Ag	14.06	in^2	b*d			
Sxx	21.97	in^3	(b*d^2)/6			
Ixx	103.00	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	620	psi	(1.2Emin')/((RB)^2)			
RB	44.07		sqrt((le*d)/((b)^2))			
le	466.19	in	1.63lu+3d	b/c lu/d>7		
lu	268.75	in				
lu/d	28.67					
le/d	49.7		<= 50 so good			

To Find CP							
Fc*	4308	psi	Fc*CD*CM*Ct				
FcE	333	psi	(.822Emin')/((le /d)^2)				
c	0.9						
Fb'	614	psi	Fb*CD*CM*Ct* CL*Cfu*Cc*Cl				
Bending Capacity							
w(beam weight)	3.52	plf					
wu	1789	plf		Using controlling load combination			
M	1348796	lb-in	(wu*l^2)/8+(w(b eam weight)*l^2)/8				
S(req'd)	2196.0	in^3	M/Fb'	S<S(req'd) so need new beam size			
Iteration 2							
Fb	4452.7	psi		30.7 MPa			
Fv	362.6	psi		2.5 MPa			
Fc (perp.)	1087.8	psi		7.5 MPa			
E	1899994.4	psi		13100 MPa			
Emin	1003197.0	psi	.528E				
Fc	4786.3	psi		33 MPa			
Ft	2958.8	psi		20.4 MPa			
CD	0.9						
CM	1.0			Moisture content in service <16%			
Ct	1.0			Temp. <100 F in Boston			
Cfu	1.0						
Cc	1.0						
Cb	1.0						
Cl	1.0						
Cvr	0.72			Cyclic Loading			
CL	1.00		((1+(FbE/Fb*))/ 1.9)-sqrt((((1+(FbE/Fb*))/1.9)^ 2)-((FbE/Fb*)/.9 5))	Doesn't apply b/c greater than CV			
CV	0.89		(21/l)^(1/20)*(1 2/d)^(1/20)*(5.1 25/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL			
CP	0.40		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^ 2)-((FcE/Fc*)/c)))				
E'	1899994	psi	E*CM*Ct				
Emin'	1003197	psi	Emin*CM*Ct				

Length (l)	22.40	ft		22' 4 3/4"		
b	23.75	in			guess	
d	24.13	in			guess	
Ag	572.97	in^2	b*d			
Sxx	2303.81	in^3	(b*d^2)/6			
Ixx	27789.73	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	55142	psi	(1.2Emin')/((RB)^2)			
RB	4.67		sqrt((le*d)/((b)^2))			
le	510.44	in	1.63lu+3d	b/c lu/d>7		
lu	268.75	in				
lu/d	11.14					
le/d	21.2		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	1842	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3573	psi	Fb*CD*CM*Ct*CV*Cfu*Cc*Cl			
Bending Capacity						
w(beam weight)	143	plf				
wu	1789	plf		Using controlling load combination		
M	1453920	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	406.9	in^3	M/Fb'	Very overdesigned so need new size		
Iteration 3						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		

E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV		
CV	0.94		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL		
CP	0.12		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	22.40	ft		22' 4 3/4"		
b	15.5	in			guess	
d	12.13	in			guess	
Ag	187.94	in^2	b*d			
Sxx	379.79	in^3	(b*d^2)/6			
Ixx	2302.48	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	50277	psi	(1.2Emin')/((RB)^2)			

RB	4.89		sqrt((le*d)/((b)^2))			
le	474.44	in	1.63lu+3d	b/c lu/d>7		
lu	268.75	in				
lu/d	22.16					
le/d	39.1		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	539	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3778	psi	Fb*CD*CM*Ct*CV*Cfu*Cc*CI			
Bending Capacity						
w(beam weight)	47	plf				
wu	1789	plf		Using controlling load combination		
M	1381500	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	365.7	in^3	M/Fb'	S>S(req'd) so good		
M*	1691076.985	lb-in	Fb*Sx			
M'	1594076	lb-in	M* *CV	M'>=M max		
Deflection						
wLL	1200	plf				
Delta LL	1.6	in	(5wLL*l^4)/(384E'*I)	Delta LL>1" so need new size		
L/360	0.747	in				
Required Ix	4789	in^4				
Iteration 4						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		

E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV		
CV	0.93		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL		
CP	0.18		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	22.40	ft		22' 4 3/4"		
b	17.6	in				
d	15.1	in				
Ag	266.58	in^2	b*d			
Sxx	672.00	in^3	(b*d^2)/6			
Ixx	5081.99	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	51143	psi	(1.2Emin')/((RB)^2)			

RB	4.85		sqrt((le*d)/((b)^2))			
le	483.44	in	1.63lu+3d	b/c lu/d>7		
lu	268.75	in				
lu/d	17.77					
le/d	32.0		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	807	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3712	psi	Fb*CD*CM*Ct*CV*Cfu*Cc*Cl			
Bending Capacity						
w(beam weight)	67	plf				
wu	1789	plf		Using controlling load combination		
M	1396291	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	376.1	in^3	M/Fb'	S>S(req'd) so good		
M*	2992182.5	lb-in	Fb*Sx			
M'	2771681	lb-in	M* *CV	M'>=M max		
Deflection						
wLL	1200	plf				
Delta LL	0.703	in	(5wLL*l^4)/(384E'*I)	Delta LL<L/360 so good		
L/360	0.747	in				
wDL	308	plf				
Delta DL	0.180	in	(5wDL*l^4)/(384E'*I)			
Delta DL+LL	0.884	in		Delta DL+LL<L/240 so good		
L/240	1.12	in				
Shear Capacity						
P	36381.35118	lb	4*(M allowable/l)			
M allowable	2444372.032	lb-in	M max-M beam wgt			
M max	2494512.714	lb-in	Fb'*Sx			

M beam wgt	50140.68127	lb-in	(w beam wgt*l^2)/8			
V	18190.67559	lb	P/2			
fv	102	psi	(3V)/(2b*d)			
Fv'	235	psi	Fv*CD*CM*Ct* Cvr	fv<Fv' so good		

Beams in Contact with the Staircase in the South-West Corner of the Building

		Units	Equation	Notes		
E5-F5						
24F-ES/NPG						
Loadings	Tributary width	12	ft			
DL	25.6	psf	307	plf	5 psf for MEP + 20.6 psf for CLT Floor	
LL	100	psf	1200	plf		
S	40	psf	480	plf	Ground	
R			snow governs			
W	36.5	psf	493	plf	Boston=128 mph for Risk Category II	
E			14.3	plf	Eh+Ev	Risk Category II, Soil Site Class D, Design Category B
Eh	0.03851	lbs	rho*Qe	Rho= 1.0		
Ev	14	plf	.2*SDs*DL			
Qe	38.51	kips	Fx+V			
V	36.48	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	84.39	kips				
Cs	0.093					
Cs max	0.081			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	2.5					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.231					

SD1	0.109					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2.5					
Cd	2.5					
k	1.02					
hx^k	14.2	ft				
Wx*h^k	1200	kip-ft				
Cvx	0.056					
Fx	2.03	kips				
Load Combinations						
DL	307	plf				
DL+LL	1507	plf				
DL+S	787	plf				
DL+.75LL+.75S	1567	plf				
DL+.6W	603	plf				
DL+.75LL+.75(.6W)+.75S	1789	plf		Controlling Load Combination		
DL+.75LL+.75(.7E)+.75S	1575	plf				
.6DL+.6W	480	plf				
.6DL+.7E	194	plf				
Iteration 1						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		

Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV		
CV	0.97		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL		
CP	0.69		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	9.33	ft		9 1/3'		
b	17.625	in				
d	15.1	in				
Ag	266.58	in^2	b*d			
Sxx	672.00	in^3	(b*d^2)/6			
Ixx	5081.99	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	108472	psi	(1.2Emin')/((RB)^2)			
RB	3.33		sqrt((le*d)/((b)^2))			

le	227.94	in	1.63lu+3d	b/c lu/d>7		
lu	112	in				
lu/d	7.40					
le/d	15.1		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	3631	psi	(.822Emin')/((le /d)^2)			
c	0.9					
Fb'	3878	psi	Fb*CD*CM*Ct* CV*Cfu*Cc*Cl			
Bending Capacity						
w(beam weight)	67	plf				
wu	1789	plf		Using controlling load combination		
M	242463	lb-in	(wu*l^2)/8+(w(b eam weight)*l^2)/8			
S(req'd)	62.5	in^3	M/Fb'	Very overdesigned so need new size		
Iteration 2						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					

Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Applies b/c less than CV		
CV	1.04		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Doesn't apply b/c greater than CL		
CP	0.28		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	9.33	ft		9 1/3'		
b	8.5	in				
d	8.0	in				
Ag	68.00	in^2	b*d			
Sxx	90.67	in^3	(b*d^2)/6			
Ixx	362.67	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*CI			
FbE	52634	psi	(1.2Emin')/((RB)^2)			
RB	4.78		sqrt((le*d)/((b)^2))			
le	206.56	in	1.63lu+3d	b/c lu/d>7		
lu	112	in				
lu/d	14.00					
le/d	25.8		<= 50 so good			
To Find CP						

Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	1237	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3991	psi	Fb*CD*CM*Ct*CL*Cfu*Cc*CI			
Bending Capacity						
w(beam weight)	17	plf				
wu	1789	plf		Using controlling load combination		
M	235976	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	59.1	in^3	M/Fb'	Sx>S(req'd) so good		
M*	403707.7493	lb-in	Fb*Sx			
M'	402058	lb-in	M* *CL	M'>=M max		
Deflection						
wLL	1200	plf				
Delta LL	0.297	in	(5wLL*l^4)/(384E'*I)	Delta LL<L/360 so good		
L/360	0.311	in				
wDL	307	plf				
Delta DL	0.076	in	(5wDL*l^4)/(384E'*I)			
Delta DL+LL	0.373	in		Delta DL+LL<L/240 so good		
L/240	0.47	in				
Shear Capacity						
P	12843.97149	lb	4*(M allowable/l)			
M allowable	359631.2018	lb-in	M max-M beam wgt			
M max	361852.5351	lb-in	Fb'*Sx			
M beam wgt	2221.333333	lb-in	(w beam wgt*l^2)/8			
V	6421.985746	lb	P/2			
fv	142	psi	(3V)/(2b*d)			
Fv'	235	psi	Fv*CD*CM*Ct*Cvr	fv<Fv' so good		

Beams in Contact with the Staircase in the North of the Building

		Units	Equation	Notes		
B7.2-C7.2 Top 1/2						
24F-ES/NPG						
Loadings	Tributary width	12	ft			
DL	25.6	psf	307	plf	5 psf for MEP + 20.6 psf for CLT Floor	
LL	100	psf	1200	plf		
S	40	psf	480	plf	Ground	
R			snow governs			
W	36.5	psf	493	plf	Boston=128 mph for Risk Category II	
E			14.3	plf	Eh+Ev	Risk Category II, Soil Site Class D, Design Category B
Eh	0.03851	lbs	rho*Qe	Rho= 1.0		
Ev	14	plf	.2*SDs*DL			
Qe	38.51	kips	Fx+V			
V	36.48	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	84.39	kips				
Cs	0.093					
Cs max	0.081			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	2.5					
Ss	0.217					
S1	0.068					

Fa	1.6					
Fv	2.4					
SDs	0.231					
SD1	0.109					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2.5					
Cd	2.5					
k	1.02					
hx^k	14.2	ft				
Wx*h^k	1200	kip-ft				
Cvx	0.056					
Fx	2.03	kips				
Load Combinations						
DL	307	plf				
DL+LL	1507	plf				
DL+S	787	plf				
DL+.75LL+.75S	1567	plf				
DL+.6W	603	plf				
DL+.75LL+.75(.6W)+.75S	1789	plf		Controlling Load Combination		
DL+.75LL+.75(.7E)+.75S	1575	plf				
.6DL+.6W	480	plf				
.6DL+.7E	194	plf				
Iteration 1						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			

Fc	4786.3	psi		33 MPa
Ft	2958.8	psi		20.4 MPa
CD	0.9			
CM	1.0			Moisture content in service <16%
Ct	1.0			Temp. <100 F in Boston
Cfu	1.0			
Cc	1.0			
Cb	1.0			
Cl	1.0			
Cvr	0.72			Cyclic Loading
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV
CV	0.98		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL
CP	0.86		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))	
E'	1899994	psi	E*CM*Ct	
Emin'	1003197	psi	Emin*CM*Ct	
Length (l)	6.7	ft		6 2/3'
b	17.625	in		
d	15.1	in		
Ag	266.58	in^2	b*d	
Sxx	672.00	in^3	(b*d^2)/6	
Ixx	5081.99	in^4	(b*d^3)/12	
To Find CL				
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl	
FbE	140661	psi	(1.2Emin')/((RB)^2)	
RB	2.93		sqrt((le*d)/((b)^2))	
le	175.78	in	1.63lu+3d	b/c lu/d>7
lu	80	in		
lu/d	5.29			

le/d	11.6		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	6106	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3944	psi	Fb*CD*CM*Ct*CV*Cfu*Cc*Cl			
Bending Capacity						
w(beam weight)	67	plf				
wu	1789	plf		Using controlling load combination		
M	123705	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	31.4	in^3	M/Fb'	Very overdesigned so need new size		
Iteration 2						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		

CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Applies b/c less than CV		
CV	1.08		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Doesn't apply b/c greater than CL		
CP	0.30		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	6.7	ft		6 2/3'		
b	7.25	in				
d	6.0	in				
Ag	43.50	in^2	b*d			
Sxx	43.50	in^3	(b*d^2)/6			
Ixx	130.50	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*CI			
FbE	71065	psi	(1.2Emin')/((RB)^2)			
RB	4.12		sqrt((le*d)/((b)^2))			
le	148.40	in	1.63lu+3d	b/c lu/d>7		
lu	80	in				
lu/d	13.33					
le/d	24.7		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	1348	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3995	psi	Fb*CD*CM*Ct*CL*Cfu*Cc*CI			
Bending Capacity						

w(beam weight)	11	plf				
wu	1789	plf		Using controlling load combination		
M	119988	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	30.0	in^3	M/Fb'	Very overdesigned so need new size		
M*	193690.6665	lb-in	Fb*Sx			
M'	193115	lb-in	M* *CL	M'>=M max		
Deflection						
wLL	1200	plf				
Delta LL	0.215	in	(5wLL*l^4)/(384E'*I)	Delta LL<L/360 so good		
L/360	0.222	in				
wDL	307	plf				
Delta DL	0.055	in	(5wDL*l^4)/(384E'*I)			
Delta DL+LL	0.270	in		Delta DL+LL<L/240 so good		
L/240	0.33	in				
Shear Capacity						
P	8653.945363	lb	4*(M allowable/l)			
M allowable	173078.9073	lb-in	M max-M beam wgt			
M max	173803.9073	lb-in	Fb'*Sx			
M beam wgt	725	lb-in	(w beam wgt*l^2)/8			
V	4326.972681	lb	P/2			
fv	149	psi	(3V)/(2b*d)			
Fv'	235	psi	Fv*CD*CM*Ct*Cvr	fv<Fv' so good		

Beams in Contact with the Staircase in the North-East Corner of the Building

		Units	Equation	Notes		
B14.1-C14.1						
24F-ES/NPG						
Loadings	Tributary width	12	ft			
DL	25.6	psf	307	plf	5 psf for MEP + 20.6 psf for CLT Floor	
LL	100	psf	1200	plf		
S	40	psf	480	plf	Ground	
R			snow governs			
W	36.5	psf	493	plf	Boston=128 mph for Risk Category II	
E			14.3	plf	Eh+Ev	Risk Category II, Soil Site Class D, Design Category B
Eh	0.03851	lbs	rho*Qe	Rho= 1.0		
Ev	14	plf	.2*SDs*DL			
Qe	38.51	kips	Fx+V			
V	36.48	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	84.39	kips				
Cs	0.093					
Cs max	0.081			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	2.5					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.231					

SD1	0.109					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2.5					
Cd	2.5					
k	1.02					
hx^k	14.2	ft				
Wx*h^k	1200	kip-ft				
Cvx	0.056					
Fx	2.03	kips				
Load Combinations						
DL	307	plf				
DL+LL	1507	plf				
DL+S	787	plf				
DL+.75LL+.75S	1567	plf				
DL+.6W	603	plf				
DL+.75LL+.75(.6W)+.75S	1789	plf		Controlling Load Combination		
DL+.75LL+.75(.7E)+.75S	1575	plf				
.6DL+.6W	480	plf				
.6DL+.7E	194	plf				
Iteration 1						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					

Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV		
CV	0.98		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL		
CP	0.79		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	8.0	ft		8'		
b	17.625	in				
d	15.1	in				
Ag	266.58	in^2	b*d			
Sxx	672.00	in^3	(b*d^2)/6			
Ixx	5081.99	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	122487	psi	(1.2Emin')/((RB)^2)			
RB	3.14		sqrt((le*d)/((b)^2))			
le	201.86	in	1.63lu+3d	b/c lu/d>7		
lu	96	in				
lu/d	6.35					
le/d	13.3		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	4630	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3908	psi	Fb*CD*CM*Ct*CV*Cfu*Cc*Cl			
Bending Capacity						
w(beam weight)	67	plf				

wu	1789	plf		Using controlling load combination		
M	178136	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	45.6	in^3	M/Fb'	Very overdesigned so need new size		
Iteration 2						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
CI	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Applies b/c less than CV		
CV	1.06		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Doesn't apply b/c greater than CL		
CP	0.29		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	8.0	ft		8'		
b	7.25	in				
d	7.1	in				
Ag	51.66	in^2	b*d			
Sxx	61.34	in^3	(b*d^2)/6			

Ixx	218.53	in^4	(b*d^3)/12	
To Find CL				
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*CI	
FbE	49934	psi	(1.2Emin')/((RB)^2)	
RB	4.91		sqrt((le*d)/((b)^2))	
le	177.86	in	1.63lu+3d	b/c lu/d>7
lu	96	in		
lu/d	13.47			
le/d	25.0		<= 50 so good	
To Find CP				
Fc*	4308	psi	Fc*CD*CM*Ct	
FcE	1323	psi	(.822Emin')/((le/d)^2)	
c	0.9			
Fb'	3990	psi	Fb*CD*CM*Ct*CL*Cfu*Cc*CI	
Bending Capacity				
w(beam weight)	13	plf		
wu	1789	plf		Using controlling load combination
M	172978	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8	
S(req'd)	43.4	in^3	M/Fb'	Sx>S(req'd) so good
M*	273134.1039	lb-in	Fb*Sx	
M'	271953	lb-in	M* *CL	M'>=M max
Deflection				
wLL	1200	plf		
Delta LL	0.266	in	(5wLL*l^4)/(384E'*I)	Delta LL<L/360 so good
L/360	0.267	in		
wDL	307	plf		
Delta DL	0.068	in	(5wDL*l^4)/(384E'*I)	
Delta DL+LL	0.335	in		Delta DL+LL<L/240 so good
L/240	0.400	in		
Shear Capacity				
P	10146.58817	lb	4*(M allowable/l)	
M allowable	243518.116	lb-in	M max-M beam wgt	
M max	244757.866	lb-in	Fb'*Sx	
M beam wgt	1239.75	lb-in	(w beam wgt*l^2)/8	
V	5073.294084	lb	P/2	
fv	147	psi	(3V)/(2b*d)	
Fv'	235	psi	Fv*CD*CM*Ct*Cvr	fv<Fv' so good

Typical East-West Direction Girders

		Units	Equation	Notes		
B2.1-B3.1						
24F-ES/NPG						
Loadings	Tributary width	22.4	ft			
DL	25.6	psf	574	plf	5 psf for MEP + 20.6 psf for CLT Floor	
LL	100	psf	2241.666667	plf		
S	40	psf	896.6666667	plf	Ground	
R			snow governs			
W	36.5	psf	493	plf	Boston=128 mph for Risk Category II	
E			26.6	plf	Eh+Ev	Risk Category II, Soil Site Class D, Design Category B
Eh	0.03851	lbs	rho*Qe	Rho= 1.0		
Ev	27	plf	.2*SDs*DL			
Qe	38.51	kips	Fx+V			
V	36.48	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	84.39	kips				
Cs	0.093					
Cs max	0.081			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	2.5					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.231					
SD1	0.109					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2.5					
Cd	2.5					
k	1.02					
hx^k	14.2	ft				
Wx*h^k	1200	kip-ft				
Cvx	0.056					
Fx	2.03	kips				
Load Combinations						
DL	574	plf				
DL+LL	2816	plf				
DL+S	1471	plf				
DL+.75LL+.75S	2928	plf				
DL+.6W	870	plf				

DL+.75LL+.75(.6W)+.75S	3149	plf		Controlling Load Combination		
DL+.75LL+.75(.7E)+.75S	2942	plf				
.6DL+.6W	640	plf				
.6DL+.7E	363	plf				
Iteration 1						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV		
CV	0.96		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL		
CP	0.51		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	12.0	ft		12'		
b	17.625	in				
d	15.1	in				
Ag	266.58	in^2	b*d			
Sxx	672.00	in^3	(b*d^2)/6			
Ixx	5081.99	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	88272	psi	(1.2Emin')/((RB)^2)			
RB	3.69		sqrt((le*d)/((b)^2))			
le	280.10	in	1.63lu+3d	b/c lu/d>7		
lu	144	in				
lu/d	9.52					
le/d	18.5		<= 50 so good			
To Find CP						

Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	2405	psi	(.822Emin')/((le /d)^2)			
c	0.9					
Fb'	3830	psi	Fb*CD*CM*Ct* CV*Cfu*Cc*Cl			
Bending Capacity						
w(beam weight)	67	plf				
wu	3149	plf		Using controlling load combination		
M	694656	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	181.4	in^3	M/Fb'	S>S(req'd) so good		
M*	2992182.5	lb-in	Fb*Sx			
M'	2859516	lb-in	M* *CV	M'>=M max		
Deflection						
wLL	2242	plf				
Delta LL	0.108	in	(5wLL*l^4)/(384 E'*I)	Very overdesigned so need smaller size		
L/360	0.400	in				
wDL	574	plf				
Iteration 2						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					

Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV		
CV	0.98		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL		
CP	0.30		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	12	ft		12'		
b	13.625	in				
d	10.8	in				
Ag	146.47	in^2	b*d			
Sxx	262.42	in^3	(b*d^2)/6			
Ixx	1410.52	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	77870	psi	(1.2Emin')/((RB)^2)			
RB	3.93		sqrt((le*d)/((b)^2))			
le	266.97	in	1.63lu+3d	b/c lu/d>7		
lu	144	in				
lu/d	13.40					
le/d	24.8		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	1337	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3946	psi	Fb*CD*CM*Ct*CV*Cfu*Cc*Cl			
Bending Capacity						
w(beam weight)	37	plf				

wu	3149	plf		Using controlling load combination		
M	688170	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	174.4	in^3	M/Fb'	S>S(req'd) so good		
M*	1168480.921	lb-in	Fb*Sx			
M'	1150615	lb-in	M* *CV	M'>=M max		
Deflection						
wLL	2242	plf				
Delta LL	0.390	in	(5wLL*l^4)/(384E'*I)	Delta LL<L/360 so good		
L/360	0.400	in				
wDL	574	plf				
Delta DL	0.100	in	(5wDL*l^4)/(384E'*I)			
Delta DL+LL	0.490	in		Delta DL+LL<L/240 so good		
L/240	0.60	in				
Shear Capacity						
P	28545.67212	lb	4*(M allowable/l)			
M allowable	1027644.196	lb-in	M max-M beam wgt			
M max	1035553.509	lb-in	Fb'*Sx			
M beam wgt	7909.3125	lb-in	(w beam wgt*l^2)/8			
V	14272.83606	lb	P/2			
fv	146	psi	(3V)/(2b*d)			
Fv'	235	psi	Fv*CD*CM*Ct*Cvr	fv<Fv' so good		

Roof

Typical North-South Direction Beams

		Units	Equation	Notes		
D2-E2						
24F-ES/NPG						
Loadings	Tributary width	6	ft			
DL	17.4	psf	104	plf	5 psf for MEP + 12.4 psf for CLT Floor	
LL	100	psf	600	plf		
S	40	psf	240	plf	Ground	
R			snow governs			
W	36.5	psf	493	plf	Boston=128 mph for Risk Category II	
E			4.9	plf	Eh+Ev	Risk Category II, Soil Site Class D, Design Category B
Eh	0.03851	lbs	rho*Qe	Rho= 1.0		
Ev	5	plf	.2*SDs*DL			
Qe	38.51	kips	Fx+V			
V	36.48	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	84.39	kips				
Cs	0.093					
Cs max	0.081			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	2.5					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.231					
SD1	0.109					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2.5					
Cd	2.5					
k	1.02					
hx^k	14.2	ft				
Wx*h^k	1200	kip-ft				
Cvx	0.056					
Fx	2.03	kips				
Load Combinations						
DL	104	plf				
DL+LL	704	plf				
DL+S	344	plf				
DL+.75LL+.75S	734	plf				
DL+.6W	400	plf				

DL+.75LL+.75(.6W)+.75S	956	plf		Controlling Load Combination		
DL+.75LL+.75(.7E)+.75S	737	plf				
.6DL+.6W	358	plf				
.6DL+.7E	66	plf				
Iteration 1						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV		
CV	0.93		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL		
CP	0.18		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	22.4	ft		22' 4 3/4"		
b	17.625	in				
d	15.1	in				
Ag	266.58	in^2	b*d			
Sxx	672.00	in^3	(b*d^2)/6			
Ixx	5081.99	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	51143	psi	(1.2Emin')/((RB)^2)			
RB	4.85		sqrt((le*d)/((b)^2))			
le	483.44	in	1.63lu+3d	b/c lu/d>7		
lu	268.75	in				
lu/d	17.77					
le/d	32.0		<= 50 so good			
To Find CP						

Fc*	4308	psi	Fc*CD*CM*Ct				
FcE	807	psi	(.822Emin')/((le /d)^2)				
c	0.9						
Fb'	3712	psi	Fb*CD*CM*Ct* CV*Cfu*Cc*Cl				
Bending Capacity							
w(beam weight)	67	plf					
wu	956	plf		Using controlling load combination			
M	769500	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8				
S(req'd)	207.3	in^3	M/Fb'	Very overdesigned so need new size			
Iteration 2							
Fb	4452.7	psi		30.7 MPa			
Fv	362.6	psi		2.5 MPa			
Fc (perp.)	1087.8	psi		7.5 MPa			
E	1899994.4	psi		13100 MPa			
Emin	1003197.0	psi	.528E				
Fc	4786.3	psi		33 MPa			
Ft	2958.8	psi		20.4 MPa			
CD	0.9						
CM	1.0			Moisture content in service <16%			
Ct	1.0			Temp. <100 F in Boston			
Cfu	1.0						
Cc	1.0						
Cb	1.0						
Cl	1.0						
Cvr	0.72			Cyclic Loading			
CL	0.99		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Doesn't apply b/c greater than CV			
CV	0.97		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Applies b/c less than CL			
CP	0.08		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))				
E'	1899994	psi	E*CM*Ct				
Emin'	1003197	psi	Emin*CM*Ct				
Length (l)	22.4	ft		22' 4 3/4"			

b	11.5	in				
d	9.8	in				
Ag	112.13	in^2	b*d			
Sxx	182.20	in^3	(b*d^2)/6			
Ixx	888.24	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*CI			
FbE	34942	psi	(1.2Emin')/((RB)^2)			
RB	5.87		sqrt((le*d)/((b)^2))			
le	467.31	in	1.63lu+3d	b/c lu/d>7		
lu	268.75	in				
lu/d	27.56					
le/d	47.9		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	359	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3876	psi	Fb*CD*CM*Ct*CV*Cfu*Cc*CI			
Bending Capacity						
w(beam weight)	28	plf				
wu	956	plf		Using controlling load combination		
M	740449	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	191.0	in^3	M/Fb'	S>S(req'd so good)		
M*	811288.3844	lb-in	Fb*Sx			
M'	784759	lb-in	M* *CV	M'>=M max		
Deflection						
wLL	600	plf				
Delta LL	2.01	in	(5wLL*l^4)/(384E'*I)	Delta LL<L/360 so good		
L/360	0.747	in				
wDL	104	plf				
Delta DL	0.350	in	(5wDL*l^4)/(384E'*I)			
Delta DL+LL	2.363	in		Delta DL+LL<L/240 so good		
L/240	1.12	in				
Shear Capacity						
P	10198.23354	lb	4*(M allowable/l)			

M allowable	685193.8158	lb-in	M max-M beam wgt			
M max	706283.4078	lb-in	Fb'*Sx			
M beam wgt	21089.59198	lb-in	(w beam wgt*l^2)/8			
V	5099.116769	lb	P/2			
fv	68	psi	(3V)/(2b*d)			
Fv'	235	psi	Fv*CD*CM*Ct*Cvr	fv<Fv' so good		

Beams in Contact with the Staircase in the South-West Corner of the Building

		Units	Equation	Notes		
E5-F5						
24F-ES/NPG						
Loadings	Tributary width	6	ft			
DL	17.4	psf	104	plf	5 psf for MEP + 12.4 psf for CLT Floor	
LL	100	psf	600	plf		
S	40	psf	240	plf	Ground	
R			snow governs			
W	36.5	psf	493	plf	Boston=128 mph for Risk Category II	
E			4.9	plf	Eh+Ev	Risk Category II, Soil Site Class D, Design Category B
Eh	0.03851	lbs	rho*Qe	Rho= 1.0		
Ev	5	plf	.2*SDs*DL			
Qe	38.51	kips	Fx+V			
V	36.48	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	84.39	kips				
Cs	0.093					
Cs max	0.081			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	2.5					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.231					
SD1	0.109					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2.5					
Cd	2.5					

k	1.02					
hx^k	14.2	ft				
Wx*h^k	1200	kip-ft				
Cvx	0.056					
Fx	2.03	kips				
Load Combinations						
DL	104	plf				
DL+LL	704	plf				
DL+S	344	plf				
DL+.75LL+.75S	734	plf				
DL+.6W	400	plf				
DL+.75LL+.75(.6W)+.75S	956	plf		Controlling Load Combination		
DL+.75LL+.75(.7E)+.75S	737	plf				
.6DL+.6W	358	plf				
.6DL+.7E	66	plf				
Iteration 1						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	0.99		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Applies b/c less than CV		

CV	1.09		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Doesn't apply b/c greater than CL		
CP	0.12		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	9.3	ft		9 1/3'		
b	5.375	in				
d	5.0	in				
Ag	26.88	in^2	b*d			
Sxx	22.40	in^3	(b*d^2)/6			
Ixx	55.99	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*CI			
FbE	35209	psi	(1.2Emin')/((RB)^2)			
RB	5.85		sqrt((le*d)/((b)^2))			
le	197.56	in	1.63lu+3d	b/c lu/d>7		
lu	112	in				
lu/d	22.40					
le/d	39.5		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	528	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3982	psi	Fb*CD*CM*Ct*CL*Cfu*Cc*CI			
Bending Capacity						
w(beam weight)	7	plf				
wu	956	plf		Using controlling load combination		
M	125813	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	31.6	in^3	M/Fb'	S>S(req'd) so good		
M*	99721.00885	lb-in	Fb*Sx			
M'	99089	lb-in	M* *CL	M'>=M max		
Deflection						
wLL	600	plf				
Delta LL	0.963	in	(5wLL*l^4)/(384E'*I)	Delta LL<L/360 so good		
L/360	0.311	in				
wDL	104	plf				
Delta DL	0.168	in	(5wDL*l^4)/(384E'*I)			

Delta DL+LL	1.131	in		Delta DL+LL<L/240 so good		
L/240	0.47	in				
Shear Capacity						
P	3153.656753	lb	4*(M allowable/l)			
M allowable	88302.38908	lb-in	M max-M beam wgt			
M max	89180.30574	lb-in	Fb'*Sx			
M beam wgt	877.9166667	lb-in	(w beam wgt*l^2)/8			
V	1576.828376	lb	P/2			
fv	88	psi	(3V)/(2b*d)			
Fv'	235	psi	Fv*CD*CM*Ct*Cvr	fv<Fv' so good		

Beams in Contact with the Staircase in the North of the Building

		Units	Equation	Notes		
B7.2-C7.2 Top 1/2						
24F-ES/NPG						
Loadings	Tributary width	6	ft			
DL	17.4	psf	104	plf	5 psf for MEP + 12.4 psf for CLT Floor	
LL	100	psf	600	plf		
S	40	psf	240	plf	Ground	
R			snow governs			
W	36.5	psf	493	plf	Boston=128 mph for Risk Category II	
E			4.9	plf	Eh+Ev	Risk Category II, Soil Site Class D, Design Category B
Eh	0.03851	lbs	rho*Qe	Rho= 1.0		
Ev	5	plf	.2*SDs*DL			
Qe	38.51	kips	Fx+V			
V	36.48	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	84.39	kips				
Cs	0.093					
Cs max	0.081			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	2.5					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.231					
SD1	0.109					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2.5					
Cd	2.5					

k	1.02					
hx^k	14.2	ft				
Wx*h^k	1200	kip-ft				
Cvx	0.056					
Fx	2.03	kips				
Load Combinations						
DL	104	plf				
DL+LL	704	plf				
DL+S	344	plf				
DL+.75LL+.75S	734	plf				
DL+.6W	400	plf				
DL+.75LL+.75(.6W)+.75S	956	plf		Controlling Load Combination		
DL+.75LL+.75(.7E)+.75S	737	plf				
.6DL+.6W	358	plf				
.6DL+.7E	66	plf				
Iteration 1						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Applies b/c less than CV		

CV	1.12		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Doesn't apply b/c greater than CL		
CP	0.13		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	6.7	ft		6 2/3'		
b	5.375	in				
d	3.8	in				
Ag	20.16	in^2	b*d			
Sxx	12.60	in^3	(b*d^2)/6			
Ixx	23.62	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*CI			
FbE	65475	psi	(1.2Emin')/((RB)^2)			
RB	4.29		sqrt((le*d)/((b)^2))			
le	141.65	in	1.63lu+3d	b/c lu/d>7		
lu	80	in				
lu/d	21.33					
le/d	37.8		<= 50 so good			
To Find CP						
Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	578	psi	(.822Emin')/((le/d)^2)			
c	0.9					
Fb'	3994	psi	Fb*CD*CM*Ct*CL*Cfu*Cc*CI			
Bending Capacity						
w(beam weight)	5	plf				
wu	956	plf		Using controlling load combination		
M	64078	lb-in	(wu*l^2)/8+(w(beam weight)*l^2)/8			
S(req'd)	16.0	in^3	M/Fb'	S>S(req'd) so good		
M*	56093.06748	lb-in	Fb*Sx			
M'	55911	lb-in	M* *CL	M'>=M max		
Deflection						
wLL	600	plf				
Delta LL	0.594	in	(5wLL*l^4)/(384E'*I)	Delta LL<L/360 so good		
L/360	0.222	in				
wDL	104	plf				
Delta DL	0.103	in	(5wDL*l^4)/(384E'*I)			

Delta DL+LL	0.698	in		Delta DL+LL<L/240 so good		
L/240	0.333	in				
Shear Capacity						
P	2499.217861	lb	4*(M allowable/l)			
M allowable	49984.35721	lb-in	M max-M beam wgt			
M max	50320.29471	lb-in	Fb'*Sx			
M beam wgt	335.9375	lb-in	(w beam wgt*l^2)/8			
V	1249.60893	lb	P/2			
fv	93	psi	(3V)/(2b*d)			
Fv'	235	psi	Fv*CD*CM*Ct*Cvr	fv<Fv' so good		

Beams in Contact with the Staircase in the North-East Corner of the Building

		Units	Equation	Notes		
B14.1-C14.1						
24F-ES/NPG						
Loadings	Tributary width	6	ft			
DL	17.4	psf	104	plf	5 psf for MEP + 12.4 psf for CLT Floor	
LL	100	psf	600	plf		
S	40	psf	240	plf	Ground	
R			snow governs			
W	36.5	psf	493	plf	Boston=128 mph for Risk Category II	
E			4.9	plf	Eh+Ev	Risk Category II, Soil Site Class D, Design Category B
Eh	0.03851	lbs	rho*Qe	Rho= 1.0		
Ev	5	plf	.2*SDs*DL			
Qe	38.51	kips	Fx+V			
V	36.48	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	84.39	kips				
Cs	0.093					
Cs max	0.081			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	2.5					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.231					
SD1	0.109					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2.5					
Cd	2.5					
k	1.02					
hx^k	14.2	ft				
Wx*h^k	1200	kip-ft				
Cvx	0.056					
Fx	2.03	kips				
Load Combinations						
DL	104	plf				
DL+LL	704	plf				
DL+S	344	plf				
DL+.75LL+.75S	734	plf				
DL+.6W	400	plf				

DL+.75LL+.75(.6W)+.75S	956	plf		Controlling Load Combination		
DL+.75LL+.75(.7E)+.75S	737	plf				
.6DL+.6W	358	plf				
.6DL+.7E	66	plf				
Iteration 1						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Applies b/c less than CV		
CV	1.10		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Doesn't apply b/c greater than CL		
CP	0.12		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	8.0	ft		8'		
b	5.375	in				
d	4.3	in				
Ag	22.84	in^2	b*d			
Sxx	16.18	in^3	(b*d^2)/6			
Ixx	34.38	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	48357	psi	(1.2Emin')/((RB)^2)			
RB	4.99		sqrt((le*d)/((b)^2))			
le	169.23	in	1.63lu+3d	b/c lu/d>7		
lu	96	in				
lu/d	22.59					
le/d	39.8		<= 50 so good			
To Find CP						

Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	520	psi	(.822Emin')/((le /d)^2)			
c	0.9					
Fb'	3989	psi	Fb*CD*CM*Ct* CL*Cfu*Cc*Cl			
Bending Capacity						
w(beam weight)	6	plf				
wu	956	plf		Using controlling load combination		
M	92337	lb-in	(wu*l^2)/8+(w(b eam weight)*l^2)/8			
S(req'd)	23.1	in^3	M/Fb'	S>S(req'd) so good		
M*	72048.4289	lb-in	Fb*Sx			
M'	71726	lb-in	M* *CL	M'>=M max		
Deflection						
wLL	600	plf				
Delta LL	0.846	in	(5wLL*l^4)/(384 E'*I)	Delta LL<L/360 so good		
L/360	0.267	in				
wDL	104	plf				
Delta DL	0.147	in	(5wDL*l^4)/(38 4E'*I)			
Delta DL+LL	0.994	in		Delta DL+LL<L/240 so good		
L/240	0.40	in				
Shear Capacity						
P	2666.879522	lb	4*(M allowable/l)			
M allowable	64005.10852	lb-in	M max-M beam wgt			
M max	64553.35852	lb-in	Fb'*Sx			
M beam wgt	548.25	lb-in	(w beam wgt*l^2)/8			
V	1333.439761	lb	P/2			
fv	88	psi	(3V)/(2b*d)			
Fv'	235	psi	Fv*CD*CM*Ct* Cvr	fv<Fv' so good		

Typical East-West Girders

		Units	Equation	Notes		
B2.1-B3.1						
24F-ES/NPG						
Loadings	Tributary width	22.4	ft			
DL	17.4	psf	390	plf	5 psf for MEP + 12.4 psf for CLT Floor	
LL	100	psf	2241.666667	plf		
S	40	psf	896.6666667	plf	Ground	
R			snow governs			
W	45.3	psf	612	plf	Boston=128 mph for Risk Category II	
E			18.1	plf	Eh+Ev	Risk Category II, Soil Site Class D, Design Category B
Eh	0.03851	lbs	rho*Qe	Rho= 1.0		
Ev	18	plf	.2*SDs*DL			
Qe	38.51	kips	Fx+V			
V	36.48	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	84.39	kips				
Cs	0.093					
Cs max	0.081			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	2.5					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.231					
SD1	0.109					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2.5					
Cd	2.5					
k	1.02					
hx^k	14.2	ft				
Wx*h^k	1200	kip-ft				
Cvx	0.056					
Fx	2.03	kips				
Load Combinations						
DL	390	plf				
DL+LL	2632	plf				
DL+S	1287	plf				
DL+.75LL+.75S	2744	plf				
DL+.6W	757	plf				

DL+.75LL+.75(.6W)+.75S	3019	plf		Controlling Load Combination		
DL+.75LL+.75(.7E)+.75S	2753	plf				
.6DL+.6W	601	plf				
.6DL+.7E	247	plf				
Iteration 1						
Fb	4452.7	psi		30.7 MPa		
Fv	362.6	psi		2.5 MPa		
Fc (perp.)	1087.8	psi		7.5 MPa		
E	1899994.4	psi		13100 MPa		
Emin	1003197.0	psi	.528E			
Fc	4786.3	psi		33 MPa		
Ft	2958.8	psi		20.4 MPa		
CD	0.9					
CM	1.0			Moisture content in service <16%		
Ct	1.0			Temp. <100 F in Boston		
Cfu	1.0					
Cc	1.0					
Cb	1.0					
Cl	1.0					
Cvr	0.72			Cyclic Loading		
CL	1.00		((1+(FbE/Fb*))/1.9)-sqrt((((1+(FbE/Fb*))/1.9)^2)-((FbE/Fb*)/.95))	Applies b/c less than CV		
CV	1.01		(21/l)^(1/20)*(12/d)^(1/20)*(5.125/b)^(1/20) <= 1.0 so use 1.0	Doesn't apply b/c greater than CL		
CP	0.20		((1+(FcE/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((FcE/Fc*)/c)))			
E'	1899994	psi	E*CM*Ct			
Emin'	1003197	psi	Emin*CM*Ct			
Length (l)	12	ft		12'		
b	9.5	in				
d	8.5	in				
Ag	80.75	in^2	b*d			
Sxx	114.40	in^3	(b*d^2)/6			
Ixx	486.18	in^4	(b*d^3)/12			
To Find CL						
Fb*	4007	psi	Fb*CD*CM*Ct*Cc*Cl			
FbE	49120	psi	(1.2Emin')/((RB)^2)			
RB	4.95		sqrt((le*d)/((b)^2))			
le	260.22	in	1.63lu+3d	b/c lu/d>7		
lu	144	in				
lu/d	16.94					
le/d	30.6		<= 50 so good			
To Find CP						

Fc*	4308	psi	Fc*CD*CM*Ct			
FcE	880	psi	(.822Emin')/((le /d)^2)			
c	0.9					
Fb'	3990	psi	Fb*CD*CM*Ct *CL*Cfu*Cc*C I			
Bending Capacity						
w(beam weight)	20	plf				
wu	3019	plf		Using controlling load combination		
M	656464	lb-in	(wu*l^2)/8+(w(b eam weight)*l^2)/8			
S(req'd)	164.5	in^3	M/Fb'	S>S(req'd) so good		
M*	509365.6369	lb-in	Fb*Sx			
M'	507124	lb-in	M* *CL	M'>=M max		
Deflection						
wLL	2242	plf				
Delta LL	1.132	in	(5wLL*l^4)/(384 E'*I)	Delta LL<L/360 so good		
L/360	0.400	in				
wDL	390	plf				
Delta DL	0.197	in	(5wDL*l^4)/(38 4E'*I)			
Delta DL+LL	1.329	in		Delta DL+LL<L/240 so good		
L/240	0.60	in				
Shear Capacity						
P	12556.97484	lb	4*(M allowable/l)			
M allowable	452051.0942	lb-in	M max-M beam wgt			
M max	456411.5942	lb-in	Fb'*Sx			
M beam wgt	4360.5	lb-in	(w beam wgt*l^2)/8			
V	6278.48742	lb	P/2			
fv	117	psi	(3V)/(2b*d)			
Fv'	235	psi	Fv*CD*CM*Ct* Cvr	fv<Fv' so good		

Column Design

Ground Floor Through 4th Floor

		Units	Equation	Notes
B3.1				
FDL	20.6	psf		
RDL	40.0	psf		
CMEP DL	5.0	psf		
FLL	100.0	psf		
RSL	30.0	psf		
1.4D	91.8	psf		
1.2D+1.6F LL+.5RLL	238.7	psf		
1.2D+1.6F LL+.5RSL	253.7	psf		Controlling load combination equation
1.2D+1.6R LL+.5FLL	128.7	psf		
1.2D+1.6R SL+.5FLL	176.7	psf		
1.2D+.5FL L+.5RLL	128.7	psf		
1.2D+.5FL L+.5RSL	143.7	psf		
1.2D+.5FL L+.2RSL	134.7	psf		
Tributary Area	269.0	ft^2		
Pu	68250.7	lbs		
Beam SWDL	206.52	plf		
	14215.70	lbs		
Pu	85309.52	lbs		
Area	17.82	in^2	Pu/Fc	
Fc	4786.3	psi		33 MPa
b	8	in		Square column, nominal size
d	7.25	in		Actual size
Total Area	52.56	in^2	d^2	Total Area>=Area
(k*L)/d	22.34			(k*L)/d<=50
k	1			From Table G1
L	162	in		13.5'
alpha	0.2260		.3E/((Le/d)^2*Fc)	
E	1800000	psi		
Cp	0.2143		((1+alpha)/1.6)-sqrt((((1+alpha)/1.6)^2)-(alpha/.8))	
Fa	1025.593629	psi	Cp*Fc	
P allowable	53907.8	lbs	Fa*Total Area	P allowable<P so need new size
b	9	in		Square column, nominal size
d	8.5	in		Actual size
Total Area	72.25	in^2	d^2	Total Area>=Area
(k*L)/d	19.05882353			(k*L)/d<=50
alpha	0.3106033982		.3E/((Le/d)^2*Fc)	

Cp	0.2874176192		((1+alpha)/1.6)-sqrt((((1+alpha)/1.6)^2)-(alpha/.8))	
Fa	1375.65258	psi	Cp*Fc	
P allowable	99390.9	lbs	Fa*Total Area	
Fce	2150.730864	psi	(.822E'min)/((Le/d)^2)	
E'min	950400	psi	.528E	
c	0.9			For glulam
Fc*	4307.625	psi	Fc*Cd*Cfu	
Cd	0.9			
Cfu	1			
Fce/Fc*	0.50			
Cp	0.4600799879		((1+(Fce/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((Fce/Fc*)/c))	
F'c	1981.9	psi	Fc*Cd*Cfu*Cp	
fc	1180.8	psi	P/Total Area	F'c>fc

Roof

		Units	Equation	Notes
B3.1				
FDL	12.4	psf		
RDL	40.0	psf		
CMEP DL	5.0	psf		
FLL	20.0	psf		
RSL	30.0	psf		
1.4D	80.4	psf		
1.2D+1.6FLL+.5RLL	100.9	psf		
1.2D+1.6FLL+.5RSL	115.9	psf		Controlling load combination equation
1.2D+1.6RLL+.5FLL	78.9	psf		
1.2D+1.6RSL+.5FLL	126.9	psf		
1.2D+.5FLL+.5RLL	78.9	psf		
1.2D+.5FLL+.5RSL	93.9	psf		
1.2D+.5FLL+.2RSL	84.9	psf		
Tributary Area	134.5	ft^2		
Pu	15585.9	lbs		
Beam SWDL	96.44	plf		
	6638.11	lbs		
Pu	23551.60	lbs		
Area	4.92	in^2	Pu/Fc	
Fc	4786.3	psi		
b	9	in		Square column, nominal size
d	8.5	in		Actual size
Total Area	72.25	in^2	d^2	Total Area>=Area
(k*L)/d	19.06			(k*L)/d<=50
k	1			From Table G1
L	162	in		13.5'
alpha	0.3106		.3E/((Le/d)^2*Fc)	
E	1800000	psi		
Cp	0.2874		((1+alpha)/1.6)-sqrt((((1+alpha)/1.6)^2)-(alpha/.8))	
Fa	1375.653943	psi	Cp*Fc	
P allowable	99391.0	lbs	Fa*Total Area	Very over designed so need new size
b	8	in		Square column, nominal size
d	7.25	in		Actual size
Total Area	52.56	in^2	d^2	Total Area>=Area
(k*L)/d	22.34			(k*L)/d<=50
k	1			From Table G1
L	162	in		13.5'

alpha	0.0109		.3E/((Le/d)^2*Fc)	
E	1800000	psi		
Cp	0.0109		((1+alpha)/1.6)-sqrt((((1+alpha)/1.6)^2)-(alpha/.8))	
Fa	1079.163737	psi	Cp*Fc	
P allowable	56723.5	lbs	Fa*Total Area	P allowable>=P
Fce	1564.675309	psi	(.822E'min)/((Le/d)^2)	
E'min	950400	psi	.528E	
c	0.9			For glulam
Fc*	4307.67	psi	Fc*Cd*Cfu	
Cd	0.9			
Cfu	1			
Fce/Fc*	0.36			
Cp	0.3450514709		((1+(Fce/Fc*))/(2c))-sqrt((((1+(Fce/Fc*))/(2c))^2)-((Fce/Fc*)/c))	
F'c	1486.4	psi	Fc*Cd*Cfu*Cp	
fc	448.1	psi	P/Total Area	F'c>fc

Floor Design

Attached Lobby

Lobby		Units	Equation	Notes
L	12	ft		
DL	8			MEP + finish
LL	100	psf		
Density	36	pcf		
Adjustment Factors				
Cd	0.9			
Cm	1			
Ct	1			
Initial Estimate				
W	108	lb/ft/ft width	DL+LL	
M max 1	1944.0	ft-lbs/ft width	WL^2/8	
V max 1	648.0	lbs/ft width	WL/2	
Assume 5-ply CLT of stress grade E1 with 1 3/8" layer thickness and 6 7/8" total thickness				
(FbS)eff,f,0	10400	ft-lbs/ft width		
Vs,0	1970	lbs/ft width		
Layer thickness	1.38	in		
Total thickness	6.88	in		
Self wgt	20.6	psf		
W	128.6	lb/ft/ft width		
M max 2	2315.3	ft-lbs/ft width	WL^2/8	M max 2<(FbS)eff,f,0 so good
V max 2	771.8	lb/ft width	WL/2	V max 2<Vs,0 so good
Fb(Seff)'	2083.7	ft-lbs/ft width	M max 2*Cd*Cm*Ct	Fb(Seff)'>M max 1 so good
Fs(lb/Q)eff'	771.8	lb/ft width	V max 2*Cm*Ct	Fs(lb/Q)eff'>V max 1 so good
(El)app	347760329.3	psi/ft width	Eleff/(1+(Ks*El eff)/(GAeff*L^2))	
L	144.0	in		
Ks	11.5			
Eleff	440000000	lb-in^2/ft width		
GAeff	920000	lb/ft width		
(El)app'	347760329.3	psi/ft width	(El)app*Cm*Ct	
Delta LL	0.134	in	(5*LL*L^4)/(384*(El)app')	Delta LL<L/360 so good
L/360	0.4	in		
Delta DL+LL	0.173	in	(5*W*L^4)/(384*(El)app')	Delta DL+LL<L/240 so good
L/240	0.6	in		

Ground Floor Through 4th Floor

North 1/2		Units	Equation	Notes
L	12.00	ft		12'
DL	8	psf		MEP + finish
LL	100	psf		
Density	36	pcf		
Adjustment Factors				
Cd	0.9			
Cm	1			
Ct	1			
Initial Estimate				
W	108	lb/ft/ft width	DL+LL	
M max 1	1944.0	ft-lbs/ft width	WL^2/8	
V max 1	648.0	lbs/ft width	WL/2	
Assume 5-ply CLT of stress grade E1 with 1 3/8" layer thickness and 6 7/8" total thickness				
(FbS)eff,f,0	10400	ft-lbs/ft width		
Vs,0	1970	lbs/ft width		
Layer thickness	1.38	in		
Total thickness	6.88	in		
Self wgt	20.6	psf		
W	128.6	lb/ft/ft width		
M max 2	2315.3	ft-lbs/ft width	WL^2/8	M max 2<(FbS)eff,f,0 so good
V max 2	771.8	lb/ft width	WL/2	V max 2<Vs,0 so good
Fb(Seff)'	2083.7	ft-lbs/ft width	M max 2*Cd*Cm*Ct	Fb(Seff)'>M max 1 so good
Fs(Ib/Q)eff'	771.8	lb/ft width	V max 2*Cm*Ct	Fs(Ib/Q)eff'>V max 1 so good
(EI)app	347760329.3	psi/ft width	Eleff/(1+(Ks*EIeff)/(GAeff*L^2))	
L	144	in		
Ks	11.5			
Eleff	440000000	lb-in^2/ft width		
GAeff	920000	lb/ft width		
(EI)app'	347760329.3	psi/ft width	(EI)app*Cm*Ct	
Delta LL	0.13	in	(5*LL*L^4)/(384*(EI)app')	Delta LL<L/360 so good
L/360	0.400	in		
Delta DL+LL	0.173	in	(5*W*L^4)/(384*(EI)app')	Delta DL+LL<L/240 so good
L/240	0.60	in		

South 1/2		Units	Equation	Notes
L	12.00	ft		12'
DL	8			MEP + finish
LL	100	psf		
Density	36	pcf		
Adjustment Factors				
Cd	0.9			
Cm	1			
Ct	1			
Initial Estimate				
W	108	lb/ft/ft width	DL+LL	
M max 1	1944.0	ft-lbs/ft width	WL^2/8	
V max 1	648.0	lbs/ft width	WL/2	
Assume 5-ply CLT of stress grade E1 with 1 3/8" layer thickness and 6 7/8" total thickness				
(FbS)eff,f,0	10400	ft-lbs/ft width		
Vs,0	1970	lbs/ft width		
Layer thickness	1.38	in		
Total thickness	6.88	in		
Self wgt	20.6	psf		
W	128.6	lb/ft/ft width		
M max 2	2315.3	ft-lbs/ft width	WL^2/8	M max 2<(FbS)eff,f,0 so good
V max 2	771.8	lb/ft width	WL/2	V max 2<Vs,0 so good
Fb(Seff)'	2083.7	ft-lbs/ft width	M max 2*Cd*Cm*Ct	Fb(Seff)'>M max 1 so good
Fs(Ib/Q)eff'	771.8	lb/ft width	V max 2*Cm*Ct	Fs(Ib/Q)eff'>V max 1 so good
(EI)app	347760329.3	psi/ft width	Eleff/(1+(Ks*EIeff)/(GAeff*L^2))	
L	144	in		
Ks	11.5			
Eleff	440000000	lb-in^2/ft width		
GAeff	920000	lb/ft width		
(EI)app'	347760329.3	psi/ft width	(EI)app*Cm*Ct	
Delta LL	0.13	in	(5*LL*L^4)/(384*(EI)app')	Delta LL<L/360 so good
L/360	0.400	in		
Delta DL+LL	0.173	in	(5*W*L^4)/(384*(EI)app')	Delta DL+LL<L/240 so good
L/240	0.60	in		

Roof

North 1/2		Units	Equation	Notes	South 1/2		Units	Equation	Notes
L	6.00	ft		6'	L	6.00	ft		6'
DL	8	psf		MEP + finish	DL	8			MEP + finish
LL	20	psf			LL	20	psf		
Density	36	pcf			Density	36	pcf		
Adjustment Factors					**Adjustment Factors**				
Cd	0.9				Cd	0.9			
Cm	1				Cm	1			
Ct	1				Ct	1			
Initial Estimate					**Initial Estimate**				
W	28	lb/ft/ft width	DL+LL		W	28	lb/ft/ft width	DL+LL	
M max 1	126.0	ft-lbs/ft width	WL^2/8		M max 1	126.0	ft-lbs/ft width	WL^2/8	
V max 1	84.0	lbs/ft width	WL/2		V max 1	84.0	lbs/ft width	WL/2	
Assume 3-ply CLT of stress grade E1 with 1 3/8" layer thickness and 4 1/8" total thickness					Assume 3-ply CLT of stress grade E1 with 1 3/8" layer thickness and 4 1/8" total thickness				
(FbS)eff,f,0	4525	ft-lbs/ft width			(FbS)eff,f,0	4525	ft-lbs/ft width		
Vs,0	1430	lbs/ft width			Vs,0	1430	lbs/ft width		
Layer thickness	1.38	in			Layer thickness	1.38	in		
Total thickness	4.13	in			Total thickness	4.13	in		
Self wgt	12.4	psf			Self wgt	12.4	psf		
W	40.4	lb/ft/ft width			W	40.4	lb/ft/ft width		
M max 2	181.7	ft-lbs/ft width	WL^2/8	M max 2<(FbS)eff,f,0 so good	M max 2	181.7	ft-lbs/ft width	WL^2/8	M max 2<(FbS)eff,f,0 so good
V max 2	121.1	lb/ft width	WL/2	V max 2<Vs,0 so good	V max 2	121.1	lb/ft width	WL/2	V max 2<Vs,0 so good
Fb(Seff)'	163.5	ft-lbs/ft width	M max 2*Cd*Cm*Ct	Fb(Seff)'>M max 1 so good	Fb(Seff)'	163.5	ft-lbs/ft width	M max 2*Cd*Cm*Ct	Fb(Seff)'>M max 1 so good
Fs(Ib/Q)eff'	121.1	lb/ft width	V max 2*Cm*Ct	Fs(Ib/Q)eff'>V max 1 so good	Fs(Ib/Q)eff'	121.1	lb/ft width	V max 2*Cm*Ct	Fs(Ib/Q)eff'>V max 1 so good
(EI)app	73974438.52	psi/ft width	EIeff/(1+(Ks*EIeff)/(GAeff*L^2))		(EI)app	73974438.52	psi/ft width	EIeff/(1+(Ks*EIeff)/(GAeff*L^2))	
L	72	in			L	72	in		
Ks	11.5				Ks	11.5			
EIeff	115000000	lb-in^2/ft width			EIeff	115000000	lb-in^2/ft width		
GAeff	460000	lb/ft width			GAeff	460000	lb/ft width		
(EI)app'	73974438.52	psi/ft width	(EI)app*Cm*Ct		(EI)app'	73974438.52	psi/ft width	(EI)app*Cm*Ct	
Delta LL	0.01	in	(5*LL*L^4)/(384*(EI)app')	Delta LL<L/360 so good	Delta LL	0.01	in	(5*LL*L^4)/(384*(EI)app')	Delta LL<L/360 so good
L/360	0.2	in			L/360	0.2	in		
Delta DL+LL	0.016	in	(5*W*L^4)/(384*(EI)app')	Delta DL+LL<L/240 so good	Delta DL+LL	0.016	in	(5*W*L^4)/(384*(EI)app')	Delta DL+LL<L/240 so good
L/240	0.3	in			L/240	0.3	in		

Wall Design

Attached Lobby

Lobby Wall		Units	Equation	Notes
L	27	ft		
Assume 5-ply CLT of stress grade E1 with 1 3/8" layer thickness and 6 7/8" total thickness				
le	27	ft		
Cd	0.9			
Cm	1			
Ct	1			
Fc	4786.25	psi		33 MPa
(EI)eff	440000000	lb-in^2/ft width		
(GA)eff	920000	lb/ft width		
Ks	11.8			
Layer thickness	1.375	in		
Total thickness	6.875	in		
A	49.5	in^2/ft width		
le/d	47.1			le/d<50 so good
Cp	0.094		((1+(Pce/Pc*)/(2c))-sqrt(((1+(Pce/Pc*))/(2c))^2 -((Pce/Pc*)/c))	
c	0.9			For CLT
Pc*	213227.4375	lbs	Fc*A	
Fc*	4307.625	psi	Fc*Cd*Cm*Ct	
Pc*	213.2274375	kips		
Pce	20351.0	lbs	(pi^2(EI)app-min')/((le)^2)	
(EI)app-min'	216459217.4	lb-in^2/ft width	.5184(EI)app	
(EI)app	417552502.7	lb-in^2/ft width	((EI)eff)/(1+((Ks (EI)eff)/((GA)eff L^2)))	
L	324	in		
Pce	20.4	kips		
Pce/Pc*	0.10			
P allowable	20.1	kips/ft width	Pc*Cp	
Width	32.0	ft		
Tributary Width	6.75	ft		
W, psf	135951	lb/ft	P allowable*Tributary Width	
	13595	psf		
D+L allowable per floor	6798	psf		2 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Ground Floor Through Roof

Typical Walls		Units	Equation	Notes
L	13.5	ft		
Assume 5-ply CLT of stress grade E1 with 1 3/8" layer thickness and 6 7/8" total thickness				
le	13.5	ft		
Cd	0.9			
Cm	1			
Ct	1			
Fc	4786.25	psi		33 MPa
(EI)eff	440000000	lb-in^2/ft width		
(GA)eff	920000	lb/ft width		
Ks	11.8			
Layer thickness	1.375	in		
Total thickness	6.875	in		
A	49.5	in^2/ft width		
le/d	23.6			le/d<50 so good
Cp	0.316		((1+(Pce/Pc*)/(2c))-sqrt(((1+(Pce/Pc*))/(2c))^2-((Pce/Pc*)/c))	
c	0.9			For CLT
Pc*	213227.4375	lbs	Fc*A	
Fc*	4307.625	psi	Fc*Cd*Cm*Ct	
Pc*	213.2274375	kips		
Pce	70598.8	lbs	(pi^2(EI)app-min')/((le)^2)	
(EI)app-min'	187727338.5	lb-in^2/ft width	.5184(EI)app	
(EI)app	362128353.5	lb-in^2/ft width	((EI)eff)/(1+((Ks(EI)eff)/((GA)effL^2)))	
L	162	in		
Pce	70.6	kips		
Pce/Pc*	0.33			
P allowable	67.5	kips/ft width	Pc*Cp	

Wall 1				
Width	5.5	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 2				
Width	15.9	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 3				
Width	21.2	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 4				
Width	21.2	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 5				
Width	8.47	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 6				
Width	27.2	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 7				
Width	27.2	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 8				
Width	11.3	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 9				
Width	11.3	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 10				
Width	37.4	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 11				
Width	37.4	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 12				
Width	9.88	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 13				
Width	9.88	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 14				
Width	12.4	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 15				
Width	24	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 16				
Width	14.8	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 17				
Width	17.6	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 18				
Width	13.4	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 19				
Width	13.4	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 20				
Width	20.8	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 21				
Width	20.8	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 22				
Width	8.82	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 23				
Width	8.82	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 24				
Width	27.9	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Wall 25				
Width	27.9	ft		
Tributary Width	6.75	ft		
W, psf	455457	lb/ft	P allowable*Tributary Width	
	45546	psf		
D+L allowable per floor	7591	psf		6 floors
D+L Lobby	125.6	psf		D+L Lobby<D+L allowable per floor so good
Fv	135	psi		
Fv*tv'	835.3125	lb/in	Fv*total thickness*Cd*Cm*Ct	
	10023.75	lbs/ft	fv<Fvtv' so good	
fv	507	lbs/ft		

Cost Analysis

CLT Floors								
$20	/sq ft of 5-ply panels							
96	sq ft	8'x12' panel	23040	sq ft	240 panels	115200	sq ft	5 floors
103.25	sq ft	8' 7 1/4"x12' panel	3097.5	sq ft	30 panels	15487.5	sq ft	5 floors
103.375	sq ft	8' 7 3/8"x12' panel	3101.25	sq ft	30 panels	15506.25	sq ft	5 floors
96	sq ft	8'x12' lobby panel	1536	sq ft	16 panels	1536	sq ft	1 floor
					Total	147729.75	sq ft	
						$2,954,595	for 5-ply panels	
$12	/sq ft of 3-ply panels							
48	sq ft	8'x6' panel	23040	sq ft	480 panels	$276,480		
51.625	sq ft	8' 7 1/4"x6' panel	3097.5	sq ft	60 panels	$37,170		
51.6875	sq ft	8' 7 3/8"x6' panel	3101.25	sq ft	60 panels	$37,215		
		Total	29238.75	sq ft	Total	$3,305,460		
Glulam Beams								
$0.03	/cubic inch	$7.92	/linear inch	$95.09	/linear ft			
71,642.87	cubic inches/beam	17 5/8"x15 1/8" AB	$2,129.58	/beam	$34,073.26	/floor	$170,366.32	5 floors
71,776.16	cubic inches/beam	17 5/8"x15 1/8" BC	$2,133.54	/beam	$23,468.95	/floor	$117,344.76	5 floors
71,642.87	cubic inches/beam	17 5/8"x15 1/8" DE	$2,129.58	/beam	$29,814.11	/floor	$149,070.53	5 floors
71,976.09	cubic inches/beam	17 5/8"x15 1/8" EF	$2,139.48	/beam	$23,534.32	/floor	$117,671.62	5 floors
21,091.50	cubic inches/beam	13 5/8"x10 3/4"	$626.94	/beam	$18,808.30	/floor	$94,041.49	5 floors
7,616.00	cubic inches/beam	8 1/2"x8"	$226.39	/beam	$452.77	/floor	$2,263.85	5 floors
3,480.00	cubic inches/beam	7 1/4"x6"	$103.44	/beam	$413.77	/floor	$2,068.85	5 floors
4,959.00	cubic inches/beam	7 1/4"x7 1/8"	$147.41	/beam	$294.81	/floor	$1,474.06	5 floors
30,133.59	cubic inches/beam	11 1/2"x9 3/4" AB	$895.72	/beam	$28,663.00	/floor	$28,663.00	1 floor
30,189.66	cubic inches/beam	11 1/2"x9 3/4" BC	$897.39	/beam	$19,742.48	/floor	$19,742.48	1 floor
30,133.59	cubic inches/beam	11 1/2"x9 3/4" DE	$895.72	/beam	$25,080.13	/floor	$25,080.13	1 floor
30,273.75	cubic inches/beam	11 1/2"x9 3/4" EF	$899.88	/beam	$19,797.47	/floor	$19,797.47	1 floor
11,628.00	cubic inches/beam	9 1/2"x8 1/2"	$345.64	/beam	$10,369.24	/floor	$10,369.24	1 floor
3,010.00	cubic inches/beam	5 3/8"x5"	$89.47	/beam	$357.89	/floor	$357.89	1 floor
1,612.50	cubic inches/beam	5 3/8"x3 3/4"	$47.93	/beam	$383.45	/floor	$383.45	1 floor
2,193.00	cubic inches/beam	5 3/8"x4 1/4"	$65.19	/beam	$260.75	/floor	$260.75	1 floor
198,360.00	cubic inches/beam	23 3/4"x21 3/4"	$5,896.24	/beam	$23,584.95	/floor	$23,584.95	1 floor
						Total	$782,540.85	

CLT Walls						
$20	/sq ft of 5-ply panels					
Wall 1	74.25	sq ft	445.5	sq ft	6 floors	8910
Wall 2	214.4117647	sq ft	1286.470588	sq ft	6 floors	25729.41176
Wall 3	285.8823529	sq ft	1715.294118	sq ft	6 floors	34305.88235
Wall 4	285.8823529	sq ft	1715.294118	sq ft	6 floors	68611.76471
Wall 5	114.3529412	sq ft	686.1176471	sq ft	6 floors	13722.35294
Wall 6	366.8823529	sq ft	2201.294118	sq ft	6 floors	44025.88235
Wall 7	366.8823529	sq ft	2201.294118	sq ft	6 floors	88051.76471
Wall 8	152.4705882	sq ft	914.8235294	sq ft	6 floors	18296.47059
Wall 9	152.4705882	sq ft	914.8235294	sq ft	6 floors	36592.94118
Wall 10	505.0588235	sq ft	3030.352941	sq ft	6 floors	60607.05882
Wall 11	505.0588235	sq ft	3030.352941	sq ft	6 floors	121214.1176
Wall 12	133.4117647	sq ft	800.4705882	sq ft	6 floors	16009.41176
Wall 13	133.4117647	sq ft	800.4705882	sq ft	6 floors	32018.82353
Wall 14	166.7647059	sq ft	1000.588235	sq ft	6 floors	20011.76471
Wall 15	324	sq ft	1944	sq ft	6 floors	38880
Wall 16	200.1176471	sq ft	1200.705882	sq ft	6 floors	24014.11765
Wall 17	238.2352941	sq ft	1429.411765	sq ft	6 floors	28588.23529
Wall 18	181.0588235	sq ft	1086.352941	sq ft	6 floors	21727.05882
Wall 19	181.0588235	sq ft	1086.352941	sq ft	6 floors	43454.11765
Wall 20	281.1176471	sq ft	1686.705882	sq ft	6 floors	33734.11765
Wall 21	281.1176471	sq ft	1686.705882	sq ft	6 floors	67468.23529
Wall 22	119.1176471	sq ft	714.7058824	sq ft	6 floors	14294.11765
Wall 23	119.1176471	sq ft	714.7058824	sq ft	6 floors	28588.23529
Wall 24	376.4117647	sq ft	2258.470588	sq ft	6 floors	45169.41176
Wall 25	376.4117647	sq ft	2258.470588	sq ft	6 floors	90338.82353
Lobby Wall	864	sq ft	864	sq ft	1 floor	17280
		Total	37673.73529	sq ft		753474.7059
			$753,475			

Glulam Columns						
$3,600	/mbf for 9'x9' columns					
0.091125	mbf/ column	2.5515	mbf/floor	12.7575	mbf	5 floors
			Total	$45,927		
$3,400	/mbf for 8'x8' columns					
0.072	mbf/column	2.016	mbf/floor	2.016	mbf	1 floor
			Total	$6,854		
Design Total Cost	$4,894,257					
Total sq ft of building	113269.5	sq ft				
Cost per sq ft	$43	/sq ft				

Appendix D: Steel Frame Design Calculations

Dead Load Breakdown

The uniform dead load throughout the building included an estimation for mechanical, electrical, and plumbing systems (MEP), hung ceilings and finishes, and the self-weight of the cast-in-place concrete slab on a metal deck. An assumption of five pounds per square foot was made for the MEP and hung ceilings and finishes (“Structural Loads”, n.d.). The dead load produced by the cast-in-place concrete slab on a metal deck was found by assuming a 4” thick slab and multiplying that by the density of concrete (150 pounds per cubic foot) (Vanderwerf, 2007). This resulted in a 50 pounds per square foot dead load for the cast-in-place concrete slab on a metal deck.

Beam Design

Attached Lobby

		Units	Equation	Notes		
C3.1-D3.1						
Loadings	Tributary width	12	ft			
DL	55	psf	660	plf	5 psf for MEP + 50 psf for concrete slab	
LL	100	psf	1200	plf		
S	40	psf	480	plf	Ground	
R			snow governs			
W	36.50	psf		438	plf	Boston=128 mph for Risk Category II
E			30.59	plf	Eh+Ev	Risk Category II, Soil Site Class D
Eh	0.04092	lbs	rho*Qe	Rho= 1.0		
Ev	30.5536	plf	.2*SDs*DL			
Qe	40.92	kips	Fx+V			
V	38.75	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	117.37	kips				
Cs	0.071					
Cs max	0.062			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	3.25					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.2314666667					
SD1	0.1088					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2					
Cd	3.25					
k	1.02					
hx^k	14.221	ft				
Wx*h^k	1669.2	kip-ft				
Cvx	0.056					
Fx	2.17	kips				
Load Combinations						
1.4DL	924	plf				
1.2DL+1.6LL+.5S	2952	plf		Controlling Load Combination		
1.2DL+1.6S+LL	2760	plf				

1.2DL+W+LL+.5S	2232	plf				
1.2DL+E+LL+.2S	2119	plf				
.9DL+W	594	plf				
.9DL+E	625	plf				
Iteration 1						
Span distance	32	ft		32'		
wu	2952	plf		Controlling Load Combination		
Fy	50	ksi				
M	377.856	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	100.7616	in^3		phi=.9		
Zx	110	in^3	Zx>=M/(phi*F)	W21x50		
Update Loadings						
SDL	50	plf				
1.4DL	994	plf				
1.2DL+1.6LL+.5S	3012	plf		Controlling Load Combination		
1.2DL+1.6S+LL	2820	plf				
1.2DL+W+LL+.5S	2292	plf				
1.2DL+E+LL+.2S	2179	plf				
.9DL+W	639	plf				
.9DL+E	670	plf				
wu	3012	plf		Controlling Load Combination		
M	385.536	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	102.8096	in^3		phi=.9		
Zx	110	in^3	Zx>=M/(phi*F) so good	W21x50		
Buckling Calculations						
Flange Local Buckling (FLB)						
bf/2tf	6.1					
const	9.2		.38sqrt(E/Fy)	bf/2tf<=.38sqrt(E/Fy) so good		
Web Local Buckling (WLB)						

h/tw	49.4					
const	90.5		3.76sqrt(E/Fy)	h/tw<=3.76sqrt(E/Fy) so good		
Deflection & Limits						
Ix	984	in^4				
delta LL	0.992	in	(5WL^4)/(384 EI)			
L/360	1.067	in		delta LL<L/360 so good		
delta DL	0.546	in	(5WL^4)/(384 EI)			
delta DL+delta LL	1.538	in		delta DL+delta LL>1" so need new size		
L/240	1.600	in				
Req'd Ix	1024	in^4				
New Ix	1550	in^4		W24x62		
Update Loadings						
SDL	62	plf				
1.4DL	1011	plf				
1.2DL+1.6LL+.5S	3026	plf		Controlling Load Combination		
1.2DL+1.6S+LL	2834	plf				
1.2DL+W+LL+.5S	2744	plf				
1.2DL+E+LL+.2S	2193	plf				
.9DL+W	1088	plf				
.9DL+E	680	plf				
wu	3026	plf		Controlling Load Combination		
M	387.3792	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	103.30112	in^3		phi=.9		
Zx	153	in^3	Zx>=M/(phi*F) so good	W24x62		
Buckling Calculations						
Flange Local Buckling (FLB)						
bf/2tf	5.97					
const	9.2		.38sqrt(E/Fy)	bf/2tf<=.38sqrt(E/Fy) so good		

Web Local Buckling (WLB)						
h/tw	50.1					
const	90.5		3.76sqrt(E/Fy)	h/tw<=3.76sqrt(E/Fy) so good		
Deflection & Limits						
Ix	1550	in^4				
delta LL	0.630	in	(5WL^4)/(384EI)			
L/360	1.067	in		delta LL<L/360 so good		
delta DL	0.346	in	(5WL^4)/(384EI)			
delta DL+delta LL	0.976	in				
L/240	1.600	in		delta DL+delta LL<L/240 so good		

Ground Floor Through 4^{th} Floor

Typical North-South Direction Beams

		Units	Equation	Notes		
D2-E2						
Loadings	Tributary width	12	ft			
DL	55	psf	660	plf	5 psf for MEP + 50 psf for concrete slab	
LL	100	psf	1200	plf		
S	40	psf	480	plf	Ground	
R			snow governs			
W	36.50	psf		438	plf	Boston=128 mph for Risk Category II
E			30.59	plf	Eh+Ev	Risk Category II, Soil Site Class D
Eh	0.04092	lbs	rho*Qe	Rho= 1.0		
Ev	30.5536	plf	.2*SDs*DL			
Qe	40.92	kips	Fx+V			
V	38.75	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	117.37	kips				
Cs	0.071					
Cs max	0.062			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	3.25					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.2314666667					
SD1	0.1088					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2					
Cd	3.25					
k	1.02					
hx^k	14.221	ft				
Wx*h^k	1669.2	kip-ft				
Cvx	0.056					
Fx	2.17	kips				
Load Combinations						
1.4DL	924	plf				
1.2DL+1.6LL+.5S	2952	plf		Controlling Load Combination		
1.2DL+1.6S+LL	2760	plf				

1.2DL+W+LL+.5S	2232	plf				
1.2DL+E+LL+.2S	2119	plf				
.9DL+W	594	plf				
.9DL+E	625	plf				
Iteration 1						
Span distance	22.39583333	ft		22' 4 3/4"		
wu	2952	plf		Controlling Load Combination		
Fy	50	ksi				
M	185.0805664	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	49.35481771	in^3		phi=.9		
Zx	54	in^3	Zx>=M/(phi*F)	W16x31		
Update Loadings						
SDL	31	plf				
1.4DL	967	plf				
1.2DL+1.6LL+.5S	2989	plf		Controlling Load Combination		
1.2DL+1.6S+LL	2797	plf				
1.2DL+W+LL+.5S	2269	plf				
1.2DL+E+LL+.2S	2156	plf				
.9DL+W	622	plf				
.9DL+E	652	plf				
wu	2989	plf		Controlling Load Combination		
M	187.4128825	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	49.97676866	in^3		phi=.9		
Zx	54	in^3	Zx>=M/(phi*F) so good	W16x31		
Buckling Calculations						
Flange Local Buckling (FLB)						
bf/2tf	6.28					
const	9.2		.38sqrt(E/Fy)	bf/2tf<=.38sqrt(E/Fy) so good		
Web Local Buckling (WLB)						

h/tw	51.6					
const	90.5		3.76sqrt(E/Fy)	h/tw<=3.76sqrt(E/Fy) so good		
Deflection & Limits						
Ix	375	in^4				
delta LL	0.625	in	(5WL^4)/(384EI)			
L/360	0.747	in		delta LL<L/360 so good		
delta DL	0.344	in	(5WL^4)/(384EI)			
delta DL+delta LL	0.968	in				
L/240	1.120	in		delta DL+delta LL<L/240 so good		

Beams in Contact with the Staircase in the South-West Corner of the Building

		Units	Equation	Notes		
E5-F5						
Loadings	Tributary width	12	ft			
DL	55	psf	660	plf	5 psf for MEP + 50 psf for concrete slab	
LL	100	psf	1200	plf		
S	40	psf	480	plf	Ground	
R			snow governs			
W	36.50	psf		438	plf	Boston=128 mph for Risk Category II
E			30.59	plf	Eh+Ev	Risk Category II, Soil Site Class D
Eh	0.04092	lbs	rho*Qe	Rho= 1.0		
Ev	30.5536	plf	.2*SDs*DL			
Qe	40.92	kips	Fx+V			
V	38.75	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	117.37	kips				
Cs	0.071					
Cs max	0.062			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	3.25					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.2314666667					
SD1	0.1088					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2					

Cd	3.25					
k	1.02					
hx^k	14.221	ft				
Wx*h^k	1669.2	kip-ft				
Cvx	0.056					
Fx	2.17	kips				
Load Combinations						
1.4DL	924	plf				
1.2DL+1.6LL+.5S	2952	plf		Controlling Load Combination		
1.2DL+1.6S+LL	2760	plf				
1.2DL+W+LL+.5S	2232	plf				
1.2DL+E+LL+.2S	2119	plf				
.9DL+W	594	plf				
.9DL+E	625	plf				
Iteration 1						
Span distance	9.333333333	ft		9 1/3'		
wu	2952	plf		Controlling Load Combination		
Fy	50	ksi				
M	32.144	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	8.571733333	in^3		phi=.9		
Zx	17.4	in^3	Zx>=M/(phi*F)	W12x14		
Update Loadings						
SDL	14	plf				
1.4DL	944	plf				
1.2DL+1.6LL+.5S	2969	plf		Controlling Load Combination		
1.2DL+1.6S+LL	2777	plf				
1.2DL+W+LL+.5S	2249	plf				
1.2DL+E+LL+.2S	2135	plf				
.9DL+W	607	plf				
.9DL+E	637	plf				

wu	2969	plf		Controlling Load Combination		
M	32.32693333	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	8.620515556	in^3		phi=.9		
Zx	17.4	in^3	Zx>=M/(phi*F) so good	W12x14		
Buckling Calculations						
Flange Local Buckling (FLB)						
bf/2tf	8.82					
const	9.2		.38sqrt(E/Fy)	bf/2tf<=.38sqrt(E/Fy) so good		
Web Local Buckling (WLB)						
h/tw	54.3					
const	90.5		3.76sqrt(E/Fy)	h/tw<=3.76sqrt(E/Fy) so good		
Deflection & Limits						
Ix	88.6	in^4				
delta LL	0.080	in	(5WL^4)/(384EI)			
L/360	0.311	in		delta LL<L/360 so good		
delta DL	0.044	in	(5WL^4)/(384EI)			
delta DL+delta LL	0.124	in				
L/240	0.467	in		delta DL+delta LL<L/240 so good		

Beams in Contact with the Staircase in the North of the Building

		Units	Equation	Notes		
B7.2-C7.2 (Top 1/2)						
Loadings	Tributary width	12	ft			
DL	55	psf	660	plf	5 psf for MEP + 50 psf for concrete slab	
LL	100	psf	1200	plf		
S	40	psf	480	plf	Ground	
R			snow governs			
W	36.50	psf		438	plf	Boston=128 mph for Risk Category II
E			30.59	plf	Eh+Ev	Risk Category II, Soil Site Class D
Eh	0.04092	lbs	rho*Qe	Rho= 1.0		
Ev	30.5536	plf	.2*SDs*DL			
Qe	40.92	kips	Fx+V			
V	38.75	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	117.37	kips				
Cs	0.071					
Cs max	0.062			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	3.25					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.2314666667					
SD1	0.1088					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2					
Cd	3.25					
k	1.02					
hx^k	14.221	ft				
Wx*h^k	1669.2	kip-ft				
Cvx	0.056					
Fx	2.17	kips				
Load Combinations						
1.4DL	924	plf				
1.2DL+1.6LL+.5S	2952	plf		Controlling Load Combination		

1.2DL+1.6S+LL	2760	plf		
1.2DL+W+LL+.5S	2232	plf		
1.2DL+E+LL+.2S	2119	plf		
.9DL+W	594	plf		
.9DL+E	625	plf		
Iteration 1				
Span distance	6.666666667	ft		6 2/3'
wu	2952	plf		Controlling Load Combination
Fy	50	ksi		
M	16.4	kip-ft	(wu*L^2)/8	
Bending Capacity				
M/(phi*Fy)	4.373333333	in^3		phi=.9
Zx	17.4	in^3	Zx>=M/(phi*F)	W12x14
Update Loadings				
SDL	14	plf		
1.4DL	944	plf		
1.2DL+1.6LL+.5S	2969	plf		Controlling Load Combination
1.2DL+1.6S+LL	2777	plf		
1.2DL+W+LL+.5S	2249	plf		
1.2DL+E+LL+.2S	2135	plf		
.9DL+W	607	plf		
.9DL+E	637	plf		
wu	2969	plf		Controlling Load Combination
M	16.49333333	kip-ft	(wu*L^2)/8	
Bending Capacity				
M/(phi*Fy)	4.398222222	in^3		phi=.9
Zx	17.4	in^3	Zx>=M/(phi*F) so good	W12x14
Buckling Calculations				
Flange Local Buckling (FLB)				
bf/2tf	8.82			
const	9.2		.38sqrt(E/Fy)	bf/2tf<=.38sqrt(E/Fy) so good
Web Local Buckling (WLB)				
h/tw	54.3			

const	90.5		3.76sqrt(E/Fy)	h/tw<=3.76sqrt(E/Fy) so good		
Deflection & Limits						
Ix	88.6	in^4				
delta LL	0.021	in	(5WL^4)/(384EI)			
L/360	0.222	in		delta LL<L/360 so good		
delta DL	0.011	in	(5WL^4)/(384EI)			
delta DL+delta LL	0.032	in				
L/240	0.333	in		delta DL+delta LL<L/240 so good		

Beams in Contact with the Staircase in the North-East Corner of the Building

		Units	Equation	Notes		
B14.1-C14.1						
Loadings	Tributary width	12	ft			
DL	55	psf	660	plf	5 psf for MEP + 50 psf for concrete slab	
LL	100	psf	1200	plf		
S	40	psf	480	plf	Ground	
R			snow governs			
W	36.50	psf		438	plf	Boston=128 mph for Risk Category II
E			30.59	plf	Eh+Ev	Risk Category II, Soil Site Class D
Eh	0.04092	lbs	rho*Qe	Rho= 1.0		
Ev	30.5536	plf	.2*SDs*DL			
Qe	40.92	kips	Fx+V			
V	38.75	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	117.37	kips				
Cs	0.071					
Cs max	0.062			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	3.25					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.2314666667					
SD1	0.1088					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2					
Cd	3.25					
k	1.02					
hx^k	14.221	ft				
Wx*h^k	1669.2	kip-ft				
Cvx	0.056					
Fx	2.17	kips				
Load Combinations						
1.4DL	924	plf				
1.2DL+1.6LL+.5S	2952	plf		Controlling Load Combination		
1.2DL+1.6S+LL	2760	plf				
1.2DL+W+LL+.5S	2232	plf				

1.2DL+E+LL+.2S	2119	plf				
.9DL+W	594	plf				
.9DL+E	625	plf				
Iteration 1						
Span distance	8	ft		8'		
wu	2952	plf		Controlling Load Combination		
Fy	50	ksi				
M	23.616	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	6.2976	in^3		phi=.9		
Zx	17.4	in^3	Zx>=M/(phi*F)	W12x14		
Update Loadings						
SDL	14	plf				
1.4DL	944	plf				
1.2DL+1.6LL+.5S	2969	plf		Controlling Load Combination		
1.2DL+1.6S+LL	2777	plf				
1.2DL+W+LL+.5S	2249	plf				
1.2DL+E+LL+.2S	2135	plf				
.9DL+W	607	plf				
.9DL+E	637	plf				
wu	2969	plf		Controlling Load Combination		
M	23.7504	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	6.33344	in^3		phi=.9		
Zx	17.4	in^3	Zx>=M/(phi*F) so good	W12x14		
Buckling Calculations						
Flange Local Buckling (FLB)						
bf/2tf	8.82					
const	9.2		.38sqrt(E/Fy)	bf/2tf<=.38sqrt(E/Fy) so good		
Web Local Buckling (WLB)						
h/tw	54.3					
const	90.5		3.76sqrt(E/Fy)	h/tw<=3.76sqrt(E/Fy) so good		
Deflection & Limits						
Ix	88.6	in^4				

delta LL	0.043	in	(5WL^4)/(384EI)			
L/360	0.267	in		delta LL<L/360 so good		
delta DL	0.024	in	(5WL^4)/(384EI)			
delta DL+delta LL	0.067	in				
L/240	0.400	in		delta DL+delta LL<L/240 so good		

Typical East-West Direction Girders

		Units	Equation	Notes		
B2.1-B3.1						
Loadings	Tributary width	22.41	ft			
DL	55	psf	1232.916667	plf	5 psf for MEP + 50 psf for concrete slab	
LL	100	psf	2241.666667	plf		
S	40	psf	896.6666667	plf	Ground	
R			snow governs			
W	36.50	psf		818	plf	Boston=128 mph for Risk Category II
E			57.12	plf	Eh+Ev	Risk Category II, Soil Site Class D
Eh	0.04092	lbs	rho*Qe	Rho= 1.0		
Ev	57.07582222	plf	.2*SDs*DL			
Qe	40.92	kips	Fx+V			
V	38.75	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	117.37	kips				
Cs	0.071					
Cs max	0.062			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	3.25					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.2314666667					
SD1	0.1088					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2					
Cd	3.25					
k	1.02					
hx^k	14.221	ft				
Wx*h^k	1669.2	kip-ft				
Cvx	0.056					
Fx	2.17	kips				
Load Combinations						
1.4DL	1726	plf				
1.2DL+1.6LL+.5S	5515	plf		Controlling Load Combination		
1.2DL+1.6S+LL	5156	plf				

1.2DL+W+LL+.5S	4170	plf				
1.2DL+E+LL+.2S	3958	plf				
.9DL+W	1110	plf				
.9DL+E	1167	plf				
Iteration 1						
Span distance	12	ft		12'		
wu	5515	plf		Controlling Load Combination		
Fy	50	ksi				
M	99.261	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	26.4696	in^3		phi=.9		
Zx	29.3	in^3	Zx>=M/(phi*F)	W12x22		
Update Loadings						
SDL	22	plf				
1.4DL	1757	plf				
1.2DL+1.6LL+.5S	5541	plf		Controlling Load Combination		
1.2DL+1.6S+LL	5182	plf				
1.2DL+W+LL+.5S	4196	plf				
1.2DL+E+LL+.2S	3984	plf				
.9DL+W	1129	plf				
.9DL+E	1187	plf				
wu	5541	plf		Controlling Load Combination		
M	99.7362	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	26.59632	in^3		phi=.9		
Zx	29.3	in^3	Zx>=M/(phi*F) so good	W12x22		
Buckling Calculations						
Flange Local Buckling (FLB)						
bf/2tf	4.74					
const	9.2		.38sqrt(E/Fy)	bf/2tf<=.38sqrt(E/Fy) so good		
Web Local Buckling (WLB)						

h/tw	41.8					
const	90.5		3.76sqrt(E/Fy)	h/tw<=3.76sqrt (E/Fy) so good		
Deflection & Limits						
Ix	156	in^4				
delta LL	0.231	in	(5WL^4)/(384EI)			
L/360	0.400	in		delta LL<L/360 so good		
delta DL	0.127	in	(5WL^4)/(384EI)			
delta DL+delta LL	0.358	in				
L/240	0.600	in		delta DL+delta LL<L/240 so good		

Roof

Typical North-South Direction Beams

		Units	Equation	Notes		
D2-E2						
Loadings	Tributary width	12	ft			
DL	55	psf	660	plf	5 psf for MEP + 50 psf for concrete slab	
LL	20	psf	240	plf		
S	30	psf	360	plf	Ground	
R			snow governs			
W	45.30	psf		544	plf	Boston=128 mph for Risk Category II
E			30.59	plf	Eh+Ev	Risk Category II, Soil Site Class D
Eh	0.04092	lbs	rho*Qe	Rho= 1.0		
Ev	30.5536	plf	.2*SDs*DL			
Qe	40.92	kips	Fx+V			
V	38.75	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	117.37	kips				
Cs	0.071					
Cs max	0.062			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	3.25					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.2314666667					
SD1	0.1088					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2					
Cd	3.25					
k	1.02					
hx^k	14.221	ft				
Wx*h^k	1669.2	kip-ft				
Cvx	0.056					
Fx	2.17	kips				
Load Combinations						
1.4DL	924	plf				
1.2DL+1.6LL+.5S	1356	plf				
1.2DL+1.6S+LL	1608	plf		Controlling Load Combination		

1.2DL+W+LL+.5S	1212	plf				
1.2DL+E+LL+.2S	1135	plf				
.9DL+W	594	plf				
.9DL+E	625	plf				
Iteration 1						
Span distance	22.39583333	ft		22' 4 3/4"		
wu	1608	plf		Controlling Load Combination		
Fy	50	ksi				
M	100.8162435	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	26.8843316	in^3		phi=.9		
Zx	33.2	in^3	Zx>=M/(phi*F)	W14x22		
Update Loadings						
SDL	22	plf				
1.4DL	955	plf				
1.2DL+1.6LL+.5S	1382	plf				
1.2DL+1.6S+LL	1634	plf		Controlling Load Combination		
1.2DL+W+LL+.5S	1238	plf				
1.2DL+E+LL+.2S	1161	plf				
.9DL+W	614	plf				
.9DL+E	644	plf				
wu	1634	plf		Controlling Load Combination		
M	102.4714355	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	27.32571615	in^3		phi=.9		
Zx	33.2	in^3	Zx>=M/(phi*F) so good	W14x22		
Buckling Calculations						
Flange Local Buckling (FLB)						
bf/2tf	7.46					
const	9.2		.38sqrt(E/Fy)	bf/2tf<=.38sqrt(E/Fy) so good		
Web Local Buckling (WLB)						
h/tw	53.3					
const	90.5		3.76sqrt(E/Fy)	h/tw<=3.76sqrt(E/Fy) so good		

Deflection & Limits						
Ix	199	in^4				
delta LL	0.235	in	(5WL^4)/(384EI)			
L/360	0.747	in		delta LL<L/360 so good		
delta DL	0.647	in	(5WL^4)/(384EI)			
delta DL+delta LL	0.883	in		delta DL+delta LL<L/240 so good		
L/240	1.120	in				

Typical East-West Direction Girders

		Units	Equation	Notes		
B2.1-B3.1						
Loadings	Tributary width	22.41	ft			
DL	55	psf	1232.916667	plf	5 psf for MEP + 50 psf for concrete slab	
LL	20	psf	448.3333333	plf		
S	30	psf	672.5	plf	Ground	
R			snow governs			
W	45.30	psf		1015	plf	Boston=128 mph for Risk Category II
E			57.12	plf	Eh+Ev	Risk Category II, Soil Site Class D
Eh	0.04092	lbs	rho*Qe	Rho= 1.0		
Ev	57.07582222	plf	.2*SDs*DL			
Qe	40.92	kips	Fx+V			
V	38.75	kips				
a	1					
Rp	2.5					
z	13.5	ft	height from base			
h	81	ft	total height			
Weight (W)	117.37	kips				
Cs	0.071					
Cs max	0.062			Cs>Cs max so use Cs max		
Cs min	0.01			Cs>=Cs min so good		
R	3.25					
Ss	0.217					
S1	0.068					
Fa	1.6					
Fv	2.4					
SDs	0.2314666667					
SD1	0.1088					
T	0.54					
Importance Factor (Ie * Ip)	1					
TL	6					
Omega	2					
Cd	3.25					
k	1.02					
hx^k	14.221	ft				
Wx*h^k	1669.2	kip-ft				
Cvx	0.056					
Fx	2.17	kips				
Load Combinations						
1.4DL	1726	plf				
1.2DL+1.6LL+.5S	2533	plf				
1.2DL+1.6S+LL	3004	plf		Controlling Load Combination		

1.2DL+W+LL+.5S	2264	plf				
1.2DL+E+LL+.2S	2119	plf				
.9DL+W	1110	plf				
.9DL+E	1167	plf				
Iteration 1						
Span distance	12	ft		12'		
wu	3004	plf		Controlling Load Combination		
Fy	50	ksi				
M	54.069	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	14.4184	in^3		phi=.9		
Zx	17.4	in^3	Zx>=M/(phi*F)	W12x14		
Update Loadings						
SDL	14	plf				
1.4DL	1746	plf				
1.2DL+1.6LL+.5S	2550	plf				
1.2DL+1.6S+LL	3021	plf		Controlling Load Combination		
1.2DL+W+LL+.5S	2281	plf				
1.2DL+E+LL+.2S	2136	plf				
.9DL+W	1122	plf				
.9DL+E	1179	plf				
wu	3021	plf		Controlling Load Combination		
M	54.3714	kip-ft	(wu*L^2)/8			
Bending Capacity						
M/(phi*Fy)	14.49904	in^3		phi=.9		
Zx	17.4	in^3	Zx>=M/(phi*F) so good	W12x14		
Buckling Calculations						
Flange Local Buckling (FLB)						
bf/2tf	8.82					
const	9.2		.38sqrt(E/Fy)	bf/2tf<=.38sqrt(E/Fy) so good		
Web Local Buckling (WLB)						
h/tw	54.3					
const	90.5		3.76sqrt(E/Fy)	h/tw<=3.76sqrt(E/Fy) so good		

Deflection & Limits						
Ix	88.6	in^4				
delta LL	0.081	in	(5WL^4)/(384EI)			
L/360	0.400	in		delta LL<L/360 so good		
delta DL	0.224	in	(5WL^4)/(384EI)			
delta DL+delta LL	0.305	in		delta DL+delta LL<L/240 so good		
L/240	0.600	in				

Column Design

Ground Floor Through Roof

		Units	Equation	Notes	
B3.1					
FDL	100	psf			
RDL	40	psf			
CMEP DL	5	psf			
FLL	100	psf			
RSL	30	psf			
1.4D	203	psf			
1.2D+1.6FLL+.5RLL	334	psf			
1.2D+1.6FLL+.5RSL	349	psf		Controlling load combination equation	
1.2D+1.6RLL+.5FLL	224	psf			
1.2D+1.6RSL+.5FLL	272	psf			
1.2D+.5FLL+.5RLL	224	psf			
1.2D+.5FLL+.5RSL	239	psf			
1.2D+.5FLL+.2RSL	230	psf			
Tributary Area	269	ft^2			
Pu	93881	lbs			
Beam SWDL	106	plf			
	7296.3333	lbs			
Pu	102636.6	lbs			
	102.6	kips			
phi*Pu	92.4	kips		Phi=0.9	
Lc	13.5	ft			
phi*Pu	248	kips		W8x31	From Table 4-1a in *AISC-15*

Bracing Design

Ground Floor Through Roof

Beam	W16x31	Fy	50	ksi	Fu	65	ksi		
Brace	HSS5x5x3/8	Fy	46	ksi	Fu	58	ksi		
Gusset Plate		Fy	46	ksi	Fu	58	ksi		
Beam	d	15.9	in	tw	0.275	in	kdes	0.842	in
Brace	H	5	in	B	5	in	A	6.18	in^2
				t	0.349	in			
Lb	117.2127273	in	(.19ryE)/(RyFy)						
ry	1.17	in							
E	29000	ksi							
Ry	1.1								
	9.767727273	ft							
	9' 10"	Total L = 22' 6", so need 2 inverted V bracings per beam							
Mr	2970	kip-in	(RyFyZ)/alphas						
Zx	54	in^3							
alphas	1			For LRFD					
RyFy	64.4	ksi							
Ry	1.4								
RtFu	75.4	ksi							
Rt	1.3								
Connection Design Forces									
Pt	397.992	kips	RyFyAg						
Ag	6.18	in^2							
Pcre buckling	356.3157227	kips	1.1(1.14)FcreAg						
Fcre	45.97788342	ksi	(.658^(Fy/Fe))*Fy						
Lc/r	2.673796791								
r	1.87	in							
4.71sqrt(E/Fy)	118.2608484	in							
E	29000	ksi		ASTIM A500 Gr. B					
Fe	40035.10277	ksi	(pi^2*E)/((Lc/r)^2)						
Pcre post-buckli	106.8947168	kips	.3*1.1*1.14FcreAg						
Brace length	17.57306177	ft							
Theta 1	50.19442891	degrees							
H1	254.7882696	kips	cos(theta 1)*Pt						
V1	305.7459235	kips	sin(theta 1)*Pt						
Ma	-770.7345158	kip-in							
Lbr1	22.02271555	ft							
L1	197.6850865	in							
KL1/r	118.9281938								
K	1.125								
4.71sqrt(E/Fy)	118.2608484								
Fe	20.23615965	ksi	(pi^2*E)/((Lc/r)^2)						
Fcre	17.76464207	ksi	(.658^(Fy/Fe))*Fy						
Pcre buckling	169.0696515	kips	1.1(1.14)FcreAg						
H1	108.2357534	kips	cos(theta 1)*Pcre buckling						
V1	129.882904	kips	sin(theta 1)*Pcre buckling						
Ma	327.4131541	kip-in							
Pcre post-buckli	50.72089546	kips	.3*Pcre buckling						
H1	32.47072601	kips	cos(theta 1)*Pcre post-buckling						
V1	38.96487121	kips	sin(theta 1)*Pcre post-buckling						
Ma	98.22394622	kip-in							
Brace to Gusset Weld									
phi*Rw	400.896	kips	1.392DLn	phi*Rw>Pt so good					
Pt	397.992	kips							
Shear Rupture of Brace Walls									
phi*Rn	852.59304	kips	.75*.6*RtFu4ltde	phi*Rn>Pt so good					

Pt	397.992	kips			
Gusset Buckling					
Lb	11.625	in			
KL/r	37.58550252				
K	0.7				
phi*Fcr	37.6	ksi		Table 4-14	
W	11.94178003	in	7+2sin(theta s1)lw		
phi*Pc	336.7581967	kips	.75*phi*Fcr*W	phi*Pc>Pcre buckling so good	
Pcre buckling	169.0696515	kips			
Gusset Tension Yield					
phi*Rn	403.0350759	kips		phi*Rn>Pt so good	
Pt	397.992	kips			
Brace Section Net Rupture					
Ae	6.554225	in^2	U(An,br+2Ar)	Ae>Ag,br so good	
U	0.9088888889		1-(xbar/l)		
An,br	5.52375	in^2			
Ar	0.84375	in^2			
Ag,br	6.18	in^2			
Reinforcing Plate Weld					
RyFyAr	46.40625	kips		phi*Rw>RyFyAr so good	
Ry	1.1				
phi*Rw	55.68	kips			
Gusset Shear Yield					
phi*Rn	323.203125	kips	1.0*.6Fyx1tf	phi*Rn>H1 so good	
x1	24.625	in			
tf	0.4375	in			
H1	254.7882696	kips			
Gusset Shear Rupture					
phi*Rn	315.1230469	kips	.75*.6Fux1tf	phi*Rn>H1 so good	
H1	254.7882696	kips			
Gusset Tension Yield					
phi*Rn	484.8046875	kips	.9Fyx1tf	phi*Rn>V1 so good	
V1	305.7459235	kips			
Gusset Bending					
phi*Mn	2430	kip-in	.9FyZx	phi*Mn>Ma so good	
Ma	770.7345158	kip-in			
Gusset to Beam Weld					
Va	254.7882696	kips	H1		
Na	305.7459235	kips	V1		
Ma	770.7345158	kip-in			
Neq	430.9413779	kips			
R	472.5618418	kips	sqrt(Pt^2+Va^2)		
theta	57.37326229	degrees			
mu	1.386450205		1+.5sin(theta)^1.5		
phi*Rw	532.2770895	kips		phi*Rw>R so good	

Cost Analysis

Columns						Beams						
$59	/linear foot	W8x31				$57	/linear foot	W16x31				
$796	/column					$1,277	/beam	D-E	$17,872	/floor	$89,359	5 floors
$22,287	/floor					$1,283	/beam	E-F	$14,108	/floor	$70,538	5 floors
$133,721	6 floors					$1,277	/beam	A-B	$20,425	/floor	$102,125	5 floors
						$1,279	/beam	B-C	$14,068	/floor	$70,342	5 floors
Concrete slab										Total	$332,363	
$270	/cubic yard					$42.50	/linear foot	W12x22				
18575.75	sq ft/floor	1815	sq ft for lobby			$510.00	/beam	E-W beams	$15,300.00	/floor	$76,500.00	5 floors
6191.916667	cubic ft/floor	605	cubic ft for lobby									
229.3302469	cubic yard/floor	22.40740741	cubic yards for lobby			$107	/linear foot	W24x62				
$61,919	/floor	$6,050	lobby			$3,424	/beam	Lobby	$13,696	/floor	$13,696	1 floor
$371,515	6 floors											
						$33	/linear foot	W12x14				
Gusset plate			Bracing			$308	/beam	E5-F5 & E6-F6	$616	/floor	$3,696	6 floors
$41.35	/plate		$103	/bracing		$220	/beam	B7.2-C7.2 - B7.3-C7.3	$880	/floor	$5,280	6 floors
4	plates/beam		4	braces/beam		$264	/beam	B14.1-C14.1 & B15.1-C15.1	$528	/floor	$3,168	6 floors
36	plates/floor		36	braces/floor		$396	/beam	Roof E-W beams	$11,880	/floor	$11,880	1 floor
216	6 floors		216	6 floors						Total	$24,024	
$8,931.60			$22,248									
										Total	$446,583	
$31,179.60												
Total Design Cost	$989,049											
Total sq ft of building	113269.5	sq ft										
Cost per sq ft	$8.73	/sq ft										