Spring 2018

Charney Hall Redesign Using Cross-Laminated Timber

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I hereby recommend that the
SENIOR DESIGN PROJECT REPORT
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CHARNEY HALL REDESIGN USING CROSS-LAMINATED TIMBER

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CHARNEY HALL REDESIGN USING CROSS-LAMINATED TIMBER

by

Andrew Callens,
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SENIOR DESIGN PROJECT REPORT

submitted to
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of

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ABSTRACT

Santa Clara University's new law building, Charney Hall, was constructed in 2018 using steel and concrete, but was redesigned by this team using Cross-Laminated Timber (CLT) and Glue-Laminated Timber (glulams). Charney Hall is a non-symmetric, incongruent structure with large open rooms up to 6,000 square feet. Glulams are made of several parallel planks of wood glued together with structural epoxy to obtain higher strength in the longitudinal direction. CLT panels are similar to glulams, but the longitudinal grains of wood planks are oriented in perpendicular layers in order to increase strength along the weak and strong axes of the member. These engineered wood products capture the strength and longevity of steel and concrete while lowering the environmental impact during the manufacturing and construction process, so the purpose of this design was to show the applicability of these materials in the United States. The completion of this design required an understanding of product information and material properties provided by manufacturers such as Structurlam along with an understanding of the fire, seismic, and safety research that a few organizations, such as Portland and Oregon State Universities, have conducted. This structural redesign included the design of the gravity system by way of the glulam beams and columns and the CLT floor diaphragms. It also included the design of CLT shear walls for the lateral system and a few poignant connection designs.

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CHAPTER 1

INTRODUCTION

1.1 Sustainability in Construction

Incredible advances have occurred in the last decade regarding the ability to build stronger, more stable, longer-lasting infrastructure. The construction industry, however, is currently one of the largest producers of carbon dioxide, as evidenced by Figure 1, which shows that the manufacturing and construction industries were responsible for emitting 6,066 million metric tons of CO_2 in 2015.

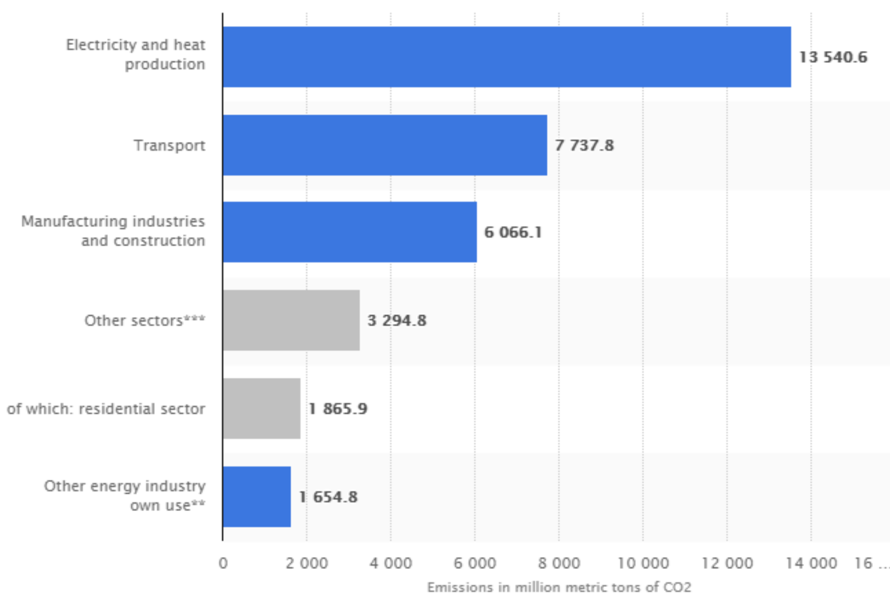

Figure 1: Comparison of carbon dioxide emissions by industry.
Source: International Energy Agency, 2017.

The two most common building materials in developed countries for anything other than residential structures are steel and concrete, which are harmful to the environment to manufacture because their production takes immense amounts of energy and water and emits copious amounts of pollution, such as carbon dioxide, into the atmosphere. Although it is widely accepted that these two materials are deleterious, their use has not diminished because they are able to provide the strength, stiffness, stability, and serviceability required of large buildings, bridges, and the like.

As climate change and environmental impacts become more concerning, many countries around the world are beginning to develop, research, and utilize building materials and construction practices that aim to mitigate the negative effects construction can have on surrounding ecological systems. Along with this push to include environmental consideration in the construction industry, economic construction will always be a high priority. The quest for increased sustainability in the implementation of new or repairing of old infrastructure must, therefore, also respect economic concerns in materials and constructability.

1.2 The Need for Sustainable Materials

There are a few different strategies available for reducing the environmental harm that a project inflicts, such as being recycling and reusing waste materials, sourcing locally to reduce pollution from shipping and transportation efforts, and reducing or eliminating the use of equipment that produces various forms of pollution. All of these strategies are beneficial, especially when used collectively, but this project team wanted to focus on improving the production of construction materials as a way to mitigate environmental impact. The materials used in a design can also significantly change the sustainability of a project, as shown in Figure 2, which compares steel, concrete, and wood in terms of total energy use in production, greenhouse gas index, air pollution index, solid waste, and ecological resource impact use.

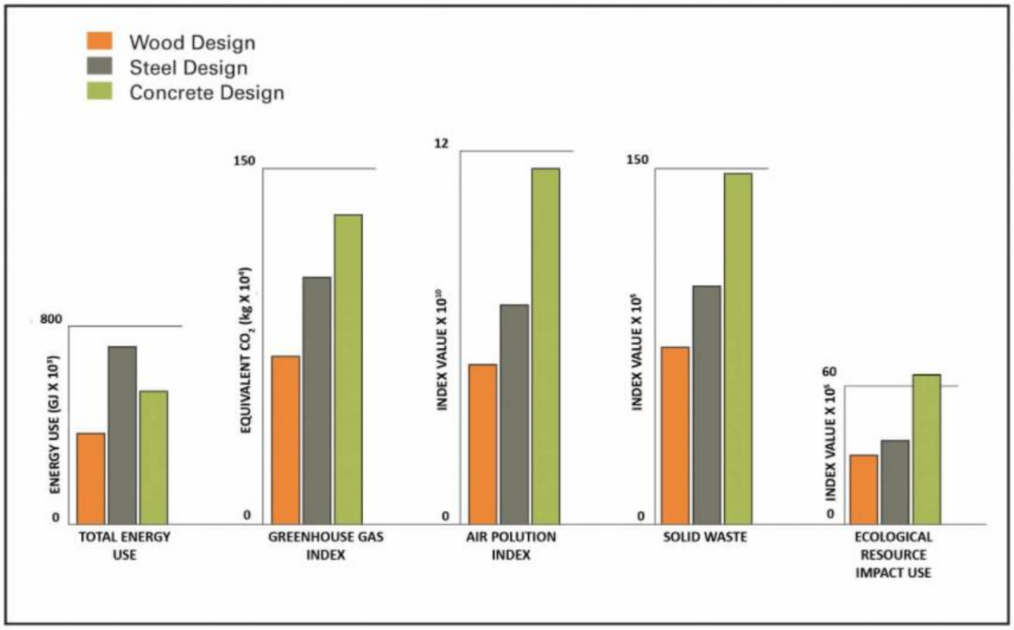

Figure 2: Comparison of the sustainability of wood, steel, and concrete.
Source: naturally:wood, 2018.

1.3 Site Details and Description

The building used for this project is Santa Clara University's (SCU's) Charney Hall Law Building, located at 500 El Camino Real in Santa Clara, California. This building, as shown in Figure 3, is 96,000 square feet, three stories tall, and has a mechanical deck and patio on the roof, contributing to its 59.5-foot height. It was constructed from October of 2016 to March of 2018 with a pile foundation, steel framing, concrete decking, and a lateral system composed of steel braced frames.

Figure 3: Rendering of Santa Clara University's Charney Hall Law Building.
Source: SCB, 2017.

This building was specifically chosen for this design project for several reasons. The first is that it represents current architecture and design criteria, so use in a design involving emerging materials would clearly illustrate their applicability to Silicon Valley aesthetics. Second, the architect behind this design had a specific vision and layout planned which would require the design team to create structural integrity for the anticipated uses of the building, while maintaining the intended features and functionality. Third, the layout involves high ceilings, walls of windows, and large, open rooms as sizable as 6,000 square feet for the anticipated classrooms and mock-courtroom. The architectural plans for Charney Hall are included in this report in Appendix A. There is a significant lack of symmetry and congruity within and between each story, which makes the building incredibly unique and required the design team to be creative and innovative in their approach to bringing the architect's vision to life. Finally, this building is located in an active seismic zone, which provided an extra design challenge but also exemplified the efficacy of emerging materials in seismic occurrences. Charney Hall of Law proved to be an excellent choice that encouraged the design team to put basic engineering principles to use in new ways and showing the usability of sustainable materials in the United States.

1.4 Project Scope

The scope of this project included the design of the gravity system, lateral system, and specific connections. For the gravity system, the location of beams and columns, as well as their individual sizes, were determined along with the sizes of the floor and roof diaphragms. The lateral system design involved sizing the shear walls that would resist the lateral load and ensuring the floor and roof diaphragms from the gravity design could transfer the expected lateral forces to the resisting system of shear walls. The connection design included schematics of primary connections throughout the structure and a detailed design for connecting the CLT panels to the glulam beams.

The scope of this project did not include design of the foundation or any non-structural elements. The pile foundation used for the existing Charney Hall structure was assumed to be sufficient for the purposes of the redesign, but the team did recognize that it would likely need reevaluation should the building actually be constructed with engineered wood. The main reason for this reevaluation is that the existing structure uses braced frames as the lateral resisting system so earthquake loads are transferred to the foundation as point loads located where columns terminate. Conversely, in a shear wall system, the lateral loads from an earthquake would be transferred to the foundation along the entire length of the shear wall, which would likely require a thicker slab in those locations. As for the non-structural elements of this building, the team assumed that cold-formed steel would be used to support the wall elements where shear walls are not located, but no design was done as they are not considered to be a part of the structural system.

CHAPTER 2

ANALYSIS OF ALTERNATIVES

2.1 Alternative Design Materials for Structural Redesign

The project team discussed the sustainability and applicability of various materials gaining visibility in the construction industry as alternatives for the project. One of the materials discussed was cob, since much research in the United States, especially at SCU, is being dedicated to assess its ability to withstand seismic activity. This alternative and others similar to it, such as earthbags and straw-bale, were quickly ruled out because of the necessary height of the structure as well as its long floor spans and plethora of windows. Light-frame construction involving either cold-formed steel or lumber was also considered, but these options were quickly disregarded, as it was obvious they would not have the necessary strength and stability that this building demands. More feasible options included materials like concrete with admixtures that reduce the necessary amount of cement and different types of engineered woods that have a higher strength than traditional stick framing.

The team decided the best solution was to complete the redesign of the structure using Cross-Laminated Timber (CLT), an engineered wood product, because it is a carbon-sequestering material designed to uphold larger loading. After researching CLT, it became clear that most manufacturers only provide Cross-Laminated Timber in the form of panels which can be used as floor and roof diaphragms or shear walls. The columns and beams framing the structure would need to be a different material, such as steel or Glue-Laminated Timber (glulams). The team decided to use glulams over steel because it is also an engineered wood so it maintains the same benefits as CLT. In total, this structural redesign of

Charney Hall Law Building was completed using two kinds of engineered wood, Cross-Laminated Timber and Glue-Laminated Timber.

2.2 Cross-Laminated Timber

Timber has been used as a building material for thousands of years due to its availability, constructability, and affordability. One downside is that it only has usable strength in the direction of its longitudinal grains, which limits its use to primarily residential structures because they tend to have smaller floor spans, fewer stories, and shorter ceiling heights. Timber is much more sustainable to produce and construct with than steel and concrete, so many researchers in countries like Japan, New Zealand, and Canada have begun investigating how to increase strength and fire resistance of timber to widen its applicability. One particular development is Cross-Laminated Timber, which is constructed by gluing perpendicular layers of wood planks together with structural epoxy, as demonstrated in Figure 4. This increases the strength of the member along both the x and y axes by orienting planks of wood such that each direction runs parallel to the grains of some of the pieces.

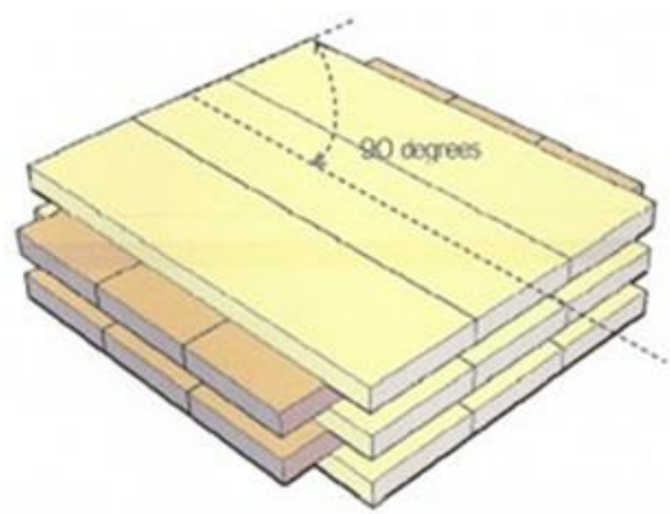

Figure 4: Schematic diagram of a CLT panel construction.
Source: Smartlam, 2018

Several countries have readily adopted this material, but the first commercial Cross-Laminated Timber building in the United States was not completed until 2011, and the first CLT high-rise recently received its building permit in 2017. It is not yet included in the building code except as an alternative material because of its newness, so designing a structure compatible with U.S. standards required much research into the expectations and requirements for heavy timber as well as use of methods developed by specific manufacturers. Without industry standards, there are large variances between manufacturers, so the team chose a specific manufacturer, called Structurlam, to obtain material properties and to be able to specify existing products. Structurlam is based in British Columbia, one of the locations where engineered wood is quite popular, and was able to provide the material properties shown in Table 1, as well as design guides for CLT diaphragms.

Table 1: Structurlam table of CLT panel grades, strengths, and physical properties.
Source: Structurlam, 2016.

CLT Grade	CLT Series	Weight lbs/ft^2	Major Strength Direction					Minor Strength Direction				
			$F_bS_{eff,0}$ (lbs-ft/ft)	$EI_{eff,0}$ (10^6lbs-in^2/ft)	$GA_{eff,0}$ (10^6lbs/ft)	$M_{allow,0}$ (lbs-ft/ft)	$V_{allowq,0}$ (lbs/ft)	$F_bS_{eff,0}$ (lbs-ft/ft)	$EI_{eff,90}$ (10^6lbs-in^2/ft)	$GA_{eff,90}$ (10^6lbs/ft)	$M_{allow,90}$ (lbs-ft/ft)	$V_{allowq,90}$ (lbs/ft)
V2.1	87V	7.5	1,444	56	0.5	1,444	1,220	37	0.4	0.3	32	240
	139V	11.9	3,329	206	1.0	3,329	1,770	537	21	0.6	457	850
	191V	16.3	5,917	503	1.4	5,917	2,290	1,216	83	0.91	1,034	1080
	243V	20.8	9,212	995	1.9	9,219	2,800	2,133	209	1.2	1,814	1320
V2M1.1	105V	9	2,042	96	0.5	2,042	1,440	277	3.7	0.53	235	495
	175V	15	4,701	366	1.1	4,701	1,980	2,403	96	1.1	2,042	1440
	245V	21	8,315	906	1.6	8,315	2,500	5,531	366	1.6	4,701	1970
	315V	27	12,896	1,806	2.1	12,896	3,025	9,782	906	2.1	8,315	2470
E1M4	87E	8.2	3,465	72	0.5	3,465	1,220	37	0.4	0.38	32	240
	139E	13	7,983	264	1.0	7,983	1,770	537	21	0.77	457	945
	191E	17.8	14,183	645	1.5	14,183	2,280	1,216	83	1.1	1,034	1200
	243E	22.7	22,075	1,278	2.0	22,075	2,800	2,133	209	1.5	1,814	1460
E1M5	105E	9.7	4,900	123	0.5	4,901	1,430	277	3.7	0.66	235	495
	175E	16.1	11,261	469	1.1	11,261	1,980	2,403	96	1.3	2,042	1590
	245E	22.5	19,897	1,161	1.6	19,897	2,500	5,531	366	2.0	4,701	2180
	315E	28.8	30,837	2,314	2.1	30,838	3,000	9,782	906	2.6	8,315	2750

Structurlam produces panels with both visual and non-visual surface qualities so that, if desired, one or both faces of the panel may be exposed. The V series, included in the chart above, is exclusively made from visual grade lumber while the E series can be either surface quality. Structurlam's panels can be used as diaphragms and shear walls and range from 3.43 to 12.42 inches thick. In terms of span, the CLT panels can be produced at 7-foot-10.5-inch (7'-10.5") or 9-foot-10.5-inch (9'-10.5") widths and can span up to 40 feet long.

2.3 Glue-Laminated Timber

Glue-Laminated Timber has existed longer than and inspired the creation of Cross-Laminated Timber since CLT, so the concept is very similar. The primary difference is that glulams are constructed in parallel layups rather than perpendicular ones, as shown in Figure 5. In this way, glulams are more applicable for beams and columns and CLT is more applicable for diaphragms and shear walls.

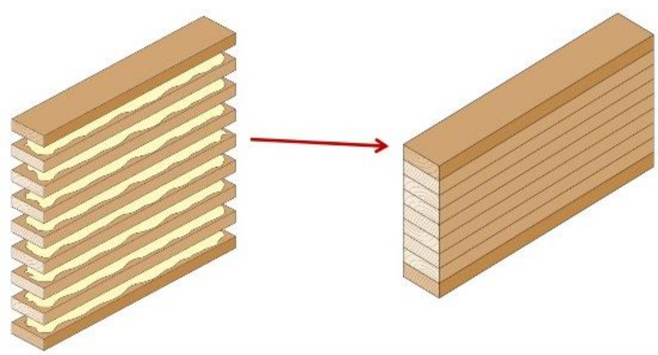

Figure 5: Schematic diagram of a glulam beam construction.
Source: Conestoga, 2017.

This orientation increases glulam strength in the longitudinal direction of the wood planks but does not significantly improve the member's strength perpendicular to the grain. There is a higher prevalence of codes, guides, and product information regarding glulam beams, one of which is the Glulam Product Guide published by The Engineered Wood

Association (APA) that includes the capacities and sizes of standard beam sizes, as well as the intrinsic properties of standard types of wood used, as shown in Table 2.

Table 2: Reference design values for structural glue-laminated softwood timber.
Source: APA, 2016.

REFERENCE DESIGN VALUES FOR STRUCTURAL GLUED LAMINATED SOFTWOOD TIMBER
(Members stressed primarily in bending)
(Tabulated design values are for normal load duration and dry service conditions.)

	Bending About X-X Axis Loaded Perpendicular to Wide Faces of Laminations						
	Extreme Fiber in Bending				Modulus of Elasticity		
	Bottom of Beam Stressed in Tension (Positive Bending)	Top of Beam Stressed in Tension (Negative Bending)	Compression Perpendicular to Grain	Shear Parallel to Grain	For Deflection Calculations		For Stability Calculations
Stress Class	F_{bx}^{+} (psi)	$F_{bx}^{-\,a}$ (psi)	$F_{c\perp x}$ (psi)	$F_{vx}^{\,b}$ (psi)	$E_{x\,true}$ (10^6 psi)	$E_{x\,app}$ (10^6 psi)	$E_{x\,min}$ (10^6 psi)
16F-1.3E	1600	925	315	195	1.4	1.3	0.69
20F-1.5E	2000	1100	425	195[d]	1.6	1.5	0.79
24F-1.7E	2400	1450	500	210[d]	1.8	1.7	0.90
24F-1.8E	2400	1450[e]	650	265[f]	1.9	1.8	0.95
26F-1.9E[g]	2600	1950	650	265[f]	2.0	1.9	1.00
28F-2.1E SP[g]	2800	2300	805	300	2.2[i]	2.1[i]	1.09
30F-2.1E SP[g,h]	3000	2400	805	300	2.2[i]	2.1[i]	1.09

The first number of the stress class denotes the allowable bending stress from tension in hundreds of pounds per square inch (psi), while the second number signifies the modulus of elasticity in millions of psi. The 2800 psi class was chosen because it is the strongest variety that can be made into beams large enough for this project's purposes.

2.4 Fire Resistance of Engineered Wood

An immense concern regarding the use of any kind of wood in construction is that it is highly flammable and excellent fuel for a fire. Extensive research has been executed to prove the ability of heavy timber to provide an acceptable fire rating so that it could resurface as a usable construction material. For example, the American Wood Council (AWC) conducted five full-scale tests in 2017 on a two-story, heavily furnished test building, shown in Figure 6, specifically geared toward determining the fire safety of heavy timber.

Figure 6: AWC test building for CLT fire resistance research.
Source: American Wood Council, 2017.

The first test utilized gypsum wall board to protect the heavy timber, and it lasted through three hours of fire with no significant charring of the wood. The second test also used gypsum protection but with approximately thirty percent of the CLT ceiling left exposed, and the panels lasted through a four-hour fire by self-extinguishing due to char. The third test left half of the CLT walls completely exposed, but a layer of char formed, protecting most of the structural integrity of the material. The fourth and fifth tests left all of the heavy timber exposed but utilized the installed sprinkler systems to control the fire. In the fourth test, the sprinklers activated rapidly, as they likely would in a real structure, while the fifth tested let the fire burn for almost half an hour before activating the sprinklers. The sprinkler system quickly extinguished the fire in both cases. Another concern is that, although the structural epoxy used for these products is not flammable, it will lose capacity if the temperature exceeds its melting point. The gravity analysis, therefore, should ensure the members have a least enough remaining strength to carry fire responders and their equipment.

CHAPTER 3

DESIGN CRITERIA AND STANDARDS

3.1 Design Criteria for Charney Hall

The deflection limits for floor members are L/360 for live load and L/240 for combined dead and live load according to Table 1604.3 of the 2016 California Building Code. There is a prevalence of vibration in long spans of engineered wood members, so the design team decided to increase the requirement to L/480 for the live load deflection limit. Charney Hall was determined to be a Risk Category III structure because it corresponds to "buildings and other structures whose primary occupancy is public assembly with an occupant load greater than three hundred" (2016 CBC). The Soil Site Class was determined to be D using the United States Geological Survey (USGS) Soil Type and Shaking Hazard Map for the San Francisco Bay Area. This map, shown in Figure 7, denotes Santa Clara University's location with the red pin and proves that SCU resides in the NEHRP D zone.

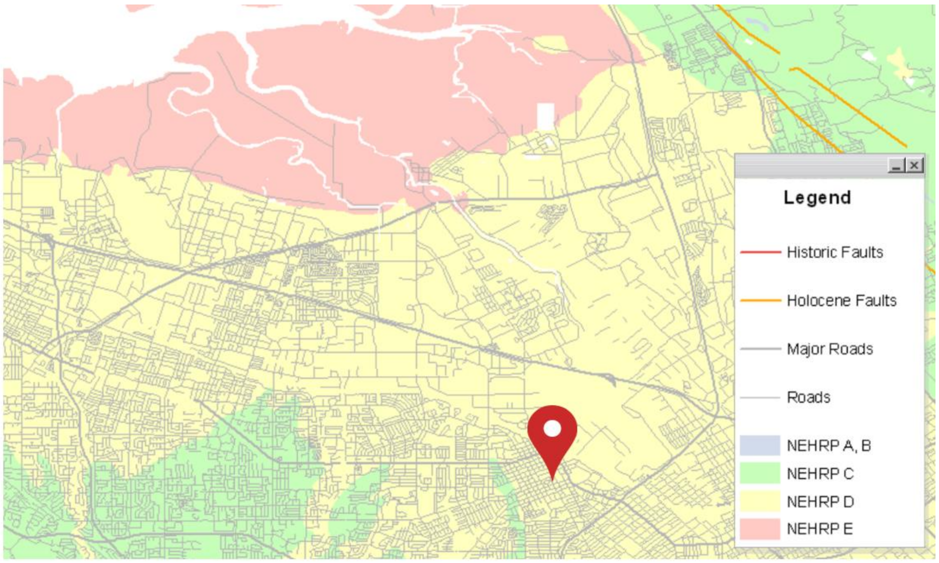

Figure 7: USGS soil type and shaking hazard map in the San Francisco bay area.
Source: USGS, 2018 Soil Type and Shaking Hazard maps.

From the established risk category and ASCE 7-10 Table 1.5-2, it was determined that the importance factor for this structure is 1.25. The location, soil site class, and risk category were entered into the USGS Seismic Design Maps in order to obtain the design response spectrum values, as shown in Figure 8, that were used in the Equivalent Lateral Force Procedure for assessing the expected lateral design loads.

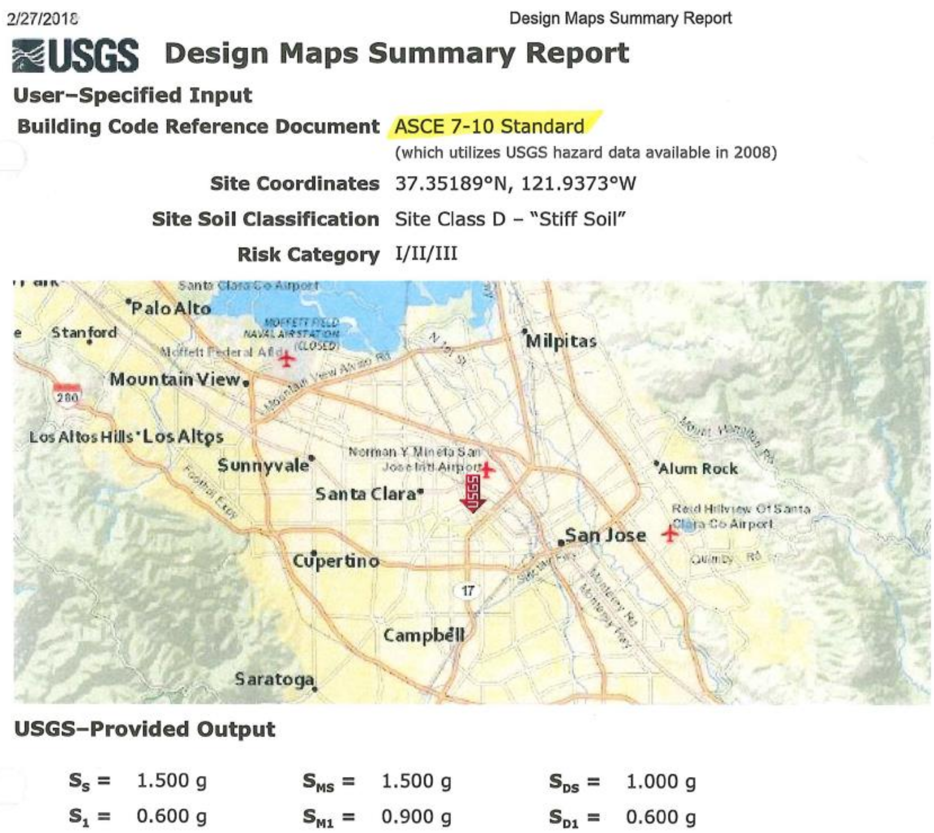

Figure 8: USGS seismic design maps summary report for SCU.
Source: USGS, 2018 U.S. Seismic Design Maps.

Beyond the technical design criteria established above, the primary focus for this project team was to respect the architect's vision for and the intended uses of Santa Clara University's Charney Hall Law Building. This vision included a 6,000 square-foot courtroom, several 150-person classrooms, library stacks for law references, and many smaller classrooms, offices, and study spaces scattered throughout each level. This team also sought to avoid infringing on the intended room sizes as often as possible and to retain the open spaces in floors or ceilings that give the structure its connected feel. Finally, it was very important that the depths of the glulam beams for the gravity system did not exceed 36 inches to allow for a CLT panel to be laid on top without infringing on ceiling height by exceeding the probable depth of a comparable I-beam and concrete deck configuration.

3.2 Codes and Guides Used for Design

A comprehensive list of the various codes and guides used for the structural redesign of Charney Hall is as follows:

- American Society of Civil Engineers (ASCE) 7-10 2010
- California Building Code (CBC) 2016
- American National Standards Institute (ANSI) 117-2015
- American Wood Council (AWC) National Design Specification (NDS) 2015
- AWC Special Design Provisions for Wind and Seismic (SDPWS) 2015
- American Concrete Institute (ACI) 318-14
- CLT Handbook (U.S. Edition) 2013
- The Engineered Wood Association (APA) Glued-Laminated Beam Design Tables 2016
- Structurlam CrossLam CLT Technical Design Guide 2016

CHAPTER 4

STRUCTURAL REDESIGN OF CHARNEY HALL

4.1 Design Approach

The gravity design approach involved first establishing a framing method so that a continuous load path would be ensured and could be used in the design of each member. A preliminary framing plan, based on the existing architectural plans, was then selected to estimate where beams and columns could be placed in the structure. The 2016 CBC was used to determine the expected live loads for different parts of the building based on their intended uses, and live load maps were developed that delineated these different locations. CLT floor diaphragms, glulam beams, and glulam columns were then sized according to the expected loads, and the framing layout was adjusted as necessary.

4.2 Framing Layout

Typical engineered wood structures in countries, such as Canada, Japan, and New Zealand, use a framing layout where beams connect to columns and the decking lays across the beams before another column is placed above. The glulam beams then get connected to the columns at the ends, and the columns get connected to the CLT panels above and below them. This platform framing provides a clear and continuous load path, as shown in Figure 9, and can also involve a rigid foam and concrete layer to coat the CLT floor diaphragm. These countries have utilized engineered woods much more in construction than other places, so their framing methods were trusted and adopted for this project.

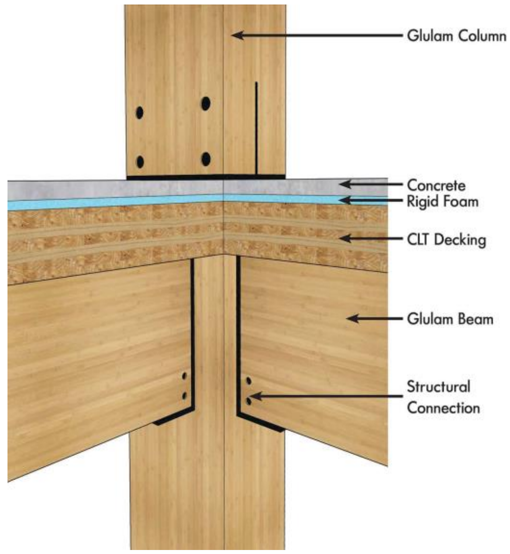

Figure 9: Schematic of CLT-panel and glulam-beam-and-column framing system.
Source: OBD, 2015.

4.3 Load Determination

The dead load distributed across each floor was estimated by totaling the expected glulam and CLT member weights, per their respective product information guides, and averaging that weight across the area of each floor. This estimation amounted to a conservative dead load of 45 pounds per square foot (psf) on the first three floors, but the dead load for the mechanical deck was raised to 50 psf because of the permanent equipment to be stored there. The anticipated live loads for the structure were determined based on the intended use of each portion of the building shown on the architectural plans. Assessment of the structure proved that four live loading categories were necessary: corridors and classrooms, offices, library stacks, and roof. The library stacks would expect a large amount of live load given the presence of many books, so the building code dictated it be designed for 150 psf. Corridors and classrooms are places where large assembly is expected so they require a live load of 100 psf, but offices only require 80 psf of live load for design. Finally, the roof live load mandated by the building code is 20 psf which was also used as the live loading for the mechanical deck. The live and dead loads experienced on the ground floor are assumed to go directly into the pile foundation mentioned earlier, so determination of those loads was unnecessary for this project. The live load maps for floors two and three are included in Figure 10 and Figure 11, respectively, where purple indicates offices, blue indicates library stacks, red indicates roof, and the uncolored portions signify classrooms and corridors.

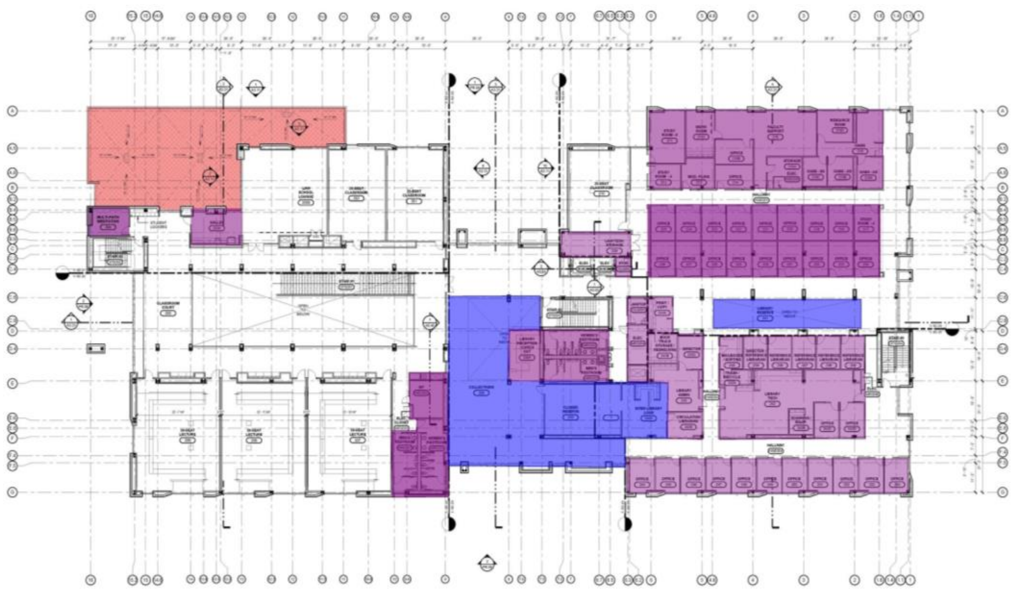

Figure 10: Second story live load map.
Source: Charney Hall architectural plans [edited].

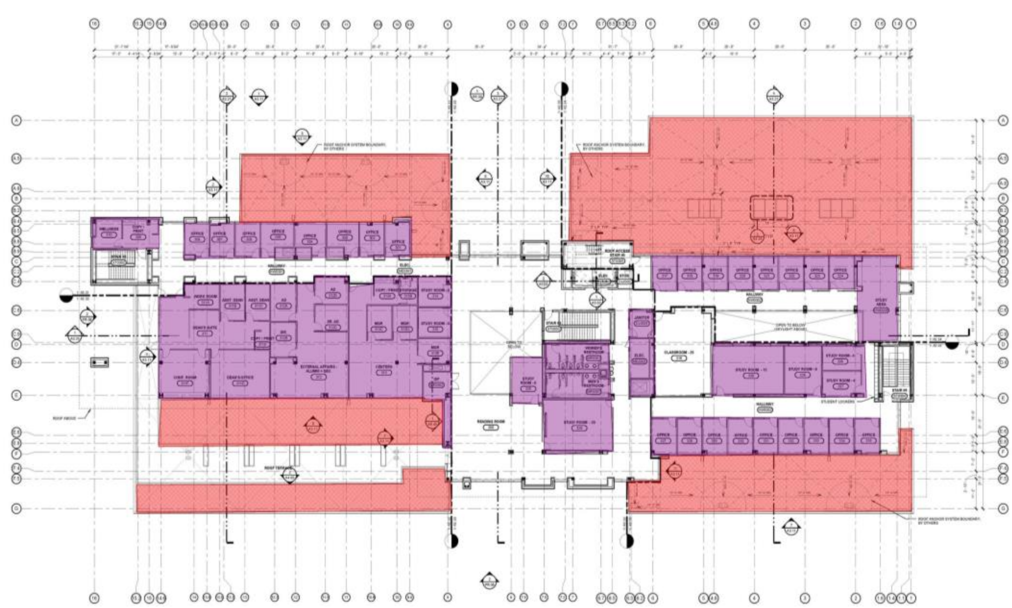

Figure 11: Third story live load map.

Source: Charney Hall architectural plans [edited].

4.4 Preliminary Beam and Column Placement

The initial placement of beams and columns in this redesign centered around the existing structural plans for Charney Hall, and slab directionality was chosen based on the shortest span between neighboring beams. More beams were added to the layout when loading demands largely exceeded the capacity of reasonable beam sizes and it was desirable to reduce the tributary area. This desired reduction mainly occurred when framing the large classrooms on the first floor. Also, Structurlam's design guides note that because of vibration and deflection concerns, a CLT slab span should be limited to 20-25 feet. This limitation led to the inclusion of more beams to mitigate these concerns and satisfy the suggested lengths.

4.5 Beam Sizing and Schedule

The majority of the beam design was straight forward since plentiful information has been published by APA and other organizations to aid in design with glulams. For a complete record of the assorted beam sizes and locations, see the beam schedule in Appendix C. The added beams, as mentioned above, were not able to frame directly into columns, so they were framed into perpendicular beams and accounted for when sizing the girders they frame into.

Despite the ease of most of the building, certain areas like the aforementioned 62’ by 99’ courtroom and 34’ by 45’ classrooms, located on the first floor, posed issues. These challenges arose because honoring the intended use of these spaces required maintaining the large open room without inserting columns that may obstruct the audience’s view from various angles. From the APA Glue-Laminated Beam Design Tables document, it was determined that the load demand and deflection limits would require a 45-inch deep section that would have to be custom-made to achieve a width of 14 inches. As mentioned before, one criteria of the gravity system design was a maximum beam depth of 36 inches, so as not to inflict on the intended ceiling height of the floor below. This issue required creativity and innovation to overcome. The project team discussed using two beams directly next two each other, but this raised concerns about using excess material and not being the most economic solution. The team also investigated the possibility of tying the beams upward such that the load was transferred back up to the second floor and over to a congruent portion of the structure. The ultimate solution for this predicament involved using the strength from the CLT panel that overlays the beam for increased capacity by forming a T-beam, as shown in Figure 12.

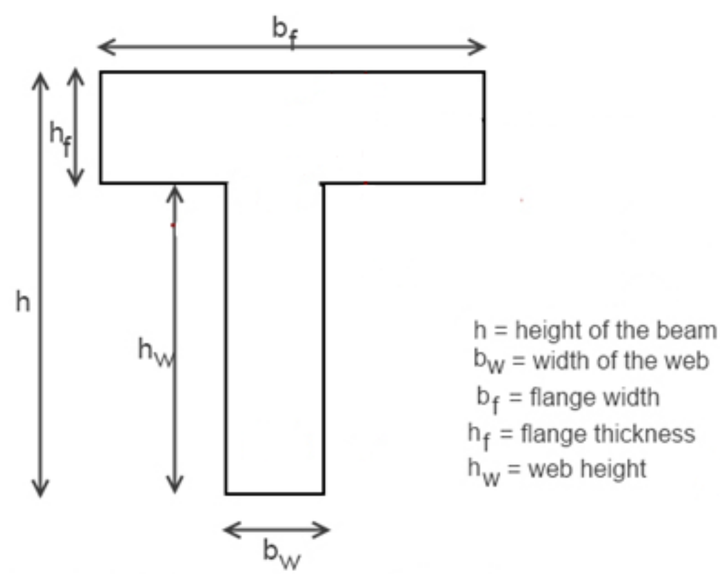

Figure 12: Schematic of a T-beam showing effective flange width.
Source: Highways for Life, 2016 [edited].

The concrete code ACI 318-14 was the inspiration for this solution as it discusses how to utilize concrete slabs in the assessment of concrete beam capacity. Engineered woods behave very differently than concrete and are less understood from a capacity perspective, so the capacity of the T-beam was calculated with incremental effective flange widths (dimension b_f in Figure 12). For the complete calculation, see Appendix . The maximum applied moment was calculated by setting the equation for bending stress equal to the allowable bending stress for a particular beam size. Since the beams will be simply supported and, therefore, always in positive bending, the allowable stress was chosen from the tension section of the Glulam Design Values for Softwood Timber table shown previously in Table 2 and discussed in Section 2.3 of this report. A similar process was followed for calculating the T-beam shear capacity using the allowable shear stresses also listed in Table 2. For the complete calculation, see Appendix C, and for the beam schedule, see Appendix D.

The moment and shear capacities from this analysis were then compared with the factored moment demand on the beams in question. If the T-beam was insufficient for the loading or required an effective flange width that exceeded the maximum set by the concrete code, a larger beam section was used for the analysis until the load could be sustained using reasonable beam and flange sizes. Although the CLT panels are laid across the entire floor, the areas where they counted toward the beam capacity in a T-beam fashion were above the courtroom and large classrooms on the first floor and below the library stacks on the second floor. The effective flange widths used in each case ended up being at least forty percent (40%) lower than the ACI minimum.

In order for this T-beam behavior to succeed, the CLT panel must be connected to the glulam beam in such a way that the two act together. The first step in designing this connection was to calculate the shear flow at the face where the two members meet. The maximum CLT panel thickness was used in this calculation since it is more conservative, and the maximum required flow was 96 kips per foot. The initial connection idea was lag screws, but each screw has a shear capacity below two kips, so lag screws alone would be insufficient. Another idea was to run a steel angle bracket along the span to screw into the side of the beam and the underside of the panel, but this would involve more steel than was desired. The use of structural epoxy similar to what connects the lams of wood together in engineered wood products was suggested, and a suitable epoxy from Loctite was chosen. The specified Premium Construction Epoxy has a shear capacity, when used with dry lumber, of 593 pounds per square inch, so if it is used over the maximum beam width of 14.25 inches, it would have a 101,460 pounds-per-foot shear flow capacity. Although the lag screws now seem unnecessary since the capacity of the epoxy already exceeds the shear flow demand, lag screws were included in pairs at 12 inches on center to account for loads from emergency responders if a fire should melt the epoxy while firefighters remain in the structure. The final design of the T-beam connection is shown in Figure 13.

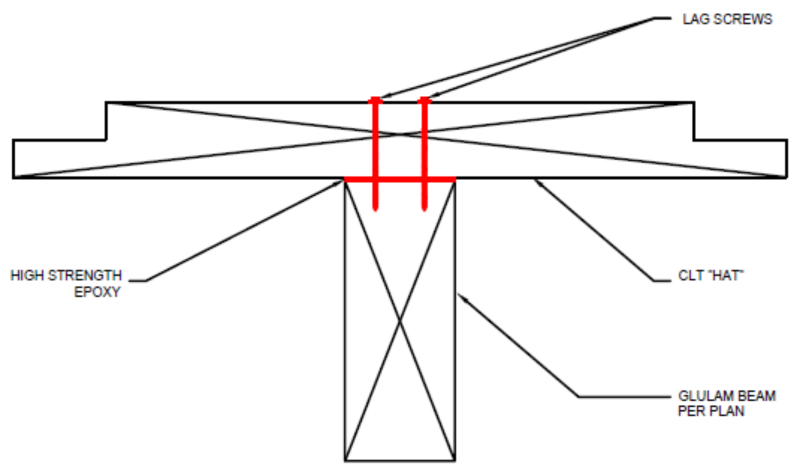

Figure 13: Schematic of the CLT panel and glulam beam connection for T-beam locations.
Source: Project team using AutoCAD.

The question that follows a sensitive connection design like this is how it will be constructed to ensure the necessary strength is provided. This project team determined that the connection would have to be prefabricated so that the epoxy could be carefully applied in a controlled environment. Unfortunately, it would not be possible to transport a beam with entire panels attached, so the idea of a hat section was proposed. This hat section involves notching out pieces on each end of a small section of CLT panel that is glued and screwed to a glulam beam, as shown in Figure 13 above. In this way, the large portions of the CLT panel could be notched in the opposite manner and attached in the field with bolts. This notch connection would require a bolt specification and spacing design to ensure that the load experienced by the panel is adequately transferred through the connection, but this design was not included in the scope of this project. Also, the notched section would have to occur outside the bounds of the effective flange width contributing to the beam strength, but the farther out the notches are placed, the more moment they experience, so careful design and analysis would be needed to determine where it is safe to connect the panels.

4.6 Column Sizing

The beams loads were used to ascertain the necessary size of each column. The columns on each floor support the loads from beams framing in as well as the loads transferred from the stories above them. The live loads were not reduced for this process until reaching the first floor. The design team ended up using 12” by 12” columns in every location, which is smaller than the existing I-section steel columns. This size was chosen based on the required size for the columns under the library stack, which represented a worst case scenario, as demonstrated in Appendix D . Distributing the same column size throughout the building is a conservative design, but it would save money and construction time if all columns are the same standard size.

4.7 Diaphragm Sizing and Schedule

Deciding the thickness of the floor diaphragms for each floor included assessment of deflection limits and moment and shear demands in the same way that beams were sized. The concern of vibration governs the allowable panel span, as shown in Table 3, which is based on deflection limits, applied live load, and vibration concerns. The 315 E Grade Cross-Laminated Timber panels from Structurlam were used so that the allowable span and the amount of flexural and shear capacity would be maximized. As mentioned in Section 4.4, the span limit required the addition of beams to mitigate vibration and deflection concerns.

Table 3: Table of CLT floor panel maximum spans per live loading.
Source: Structurlam, 2016.

CrossLam® CLT Series		FLOOR LIVE LOAD (psf)									
		40 RESIDENTIAL		50 OFFICE/ CLASSROOM		75 MECHANICAL ROOM		100 ASSEMBLY/ STORAGE		150 LIBRARY	
		Vibration	Deflection L/240	Vibration	Deflection L/240	Vibration	Deflection L/240	Vibration	Deflection L/240	Vibration	Deflection L/240
SINGLE SPAN	87 V	10.58	12.33	10.58	11.95	10.58	10.56	10.58	9.71[a]	10.58	8.18[a]
	87 E	11.37	13.33	11.37	12.67	11.37	11.43	11.37	10.55	11.37	9.33
	105 V	12.04	14.58	12.04	13.86	12.04	12.51	12.04	11.48[a]	12.04	9.69[a]
	105 E	12.93	15.77	12.93	15.00	12.93	13.54	12.93	12.51	12.93	11.07
	139 V	14.65	18.68	14.65	17.81	14.65	16.15	14.65	14.49[a]	14.65	12.27[a]
	139 E	15.75	20.17	15.75	19.24	15.75	17.47	15.75	16.19	15.75	14.41
	175 V	16.78	22.24	16.78	21.23	16.78	18.93[a]	16.78	17.01[a]	16.78	14.45[a]
	175 E	18.01	24.01	18.01	22.93	18.01	20.88	18.01	19.38	18.01	17.28
	191 V	18.30	24.65	18.30	23.56	18.30	21.10[a]	18.30	18.99[a]	18.30	16.16[a]
	191 E	19.65	26.58	19.65	25.43	19.65	23.21	19.65	21.58	19.65	19.29
	245 V	20.98	29.30	20.98	27.81[a]	20.98	24.48[a]	20.98	22.12[a]	20.98	18.91[a]
	245 E	22.50	31.57	22.50	30.27	22.50	27.74	22.50	25.85	22.50	23.16
	243 V	21.68	30.34	21.68	29.08	21.68	25.79	21.68	23.30	21.68	19.92
	243 E	22.91	32.67	22.91	31.33	22.91	28.73	22.91	26.80	22.91	24.04
	315 V	24.86	35.47[a]	24.86	33.49[a]	24.86	29.69[a]	24.86	26.95[a]	24.86	23.18[a]
	315 E	26.66	38.72	26.66	37.23	26.66	34.29	26.66	32.07	26.66	28.86

Despite shorter spans, initial analysis of heavily loaded areas like the large classrooms showed that the anticipated demand on the floor panels would be higher than any product Structurlam could provide. The ASCE 7-10, Table 4-2 was referenced regarding whether this situation would merit live load reductions. Table 4, below, states that the live load element factor for one-way slabs is one (1), so the tributary area of the slab simply has to exceed 400 square feet for it to qualify for live load reduction per Section 4.7.2.

Table 4: ASCE 7-10, Table 4-2: live load element factor, K_{LL}.
Source: American Society of Civil Engineers, 2010.

Element	K_{LL}[a]
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified, including:	1
Edge beams with cantilever slabs	
Cantilever beams	
One-way slabs	
Two-way slabs	
Members without provisions for continuous shear transfer normal to their span	

[a]In lieu of the preceding values, K_{LL} is permitted to be calculated.

In the courtroom, the beams are typically spaced at 20 feet on center and they are 60.25 feet long, so the tributary area is considered to be 1205 square feet. Even though several panels are used to cover this space, they are considered to act as one since they are connected together to collectively transfer load to the beam. Similarly, the large classrooms have beams at 17 feet on center the run for 42 feet, so the tributary area is 714 square feet. Both of these areas qualified to be designed with reduced live loads. Although the library stack load also posed issues for the diaphragm design, Section 4.7.3 of ASCE 7-10 notes that live loads above 100 psf cannot be reduced, so more beams were included under the stacks to reduce the tributary area of each panel.

The reduced live load was calculated from Equation 4.7-1 in Section 4.7.2 of the ASCE 7-10 code. Appendix E shows a sample diaphragm calculation, in which the dead and live loads imposed on the CLT panels for the sake of choosing the necessary thickness were not factored because the panel strength values listed in Table 1 are presented with Allowable Stress Design method rather than Load and Resistance Factor Design method. The moment and shear demands obtained from the adjusted loads were then used to determine what panel thickness and stress class. For the complete diaphragm calculation, see Appendix F.

4.8 Lateral Design

The first step toward completing a lateral design was to calculate the expected story shear and overturning moment per the seismic design criteria established in Section 3.1 of this report. These calculations required estimating the structure weight and utilizing the Equivalent Lateral Force Procedure outlined in Section 12.8 of ASCE 7-10, as shown in Appendix G. This procedure led to a maximum story shear of 871 kips and allowed the design team to estimate the necessary amount of feet of shear wall per floor. The highlighted walls shown in Figure 14, Figure 15, Figure 16 and Figure 17, illustrate these proposed locations of shear walls for the first, second, and third stories as well as the mechanical deck. Locations were chosen based on where braced frames were placed in the existing structure and where thicker walls would not impede building use.

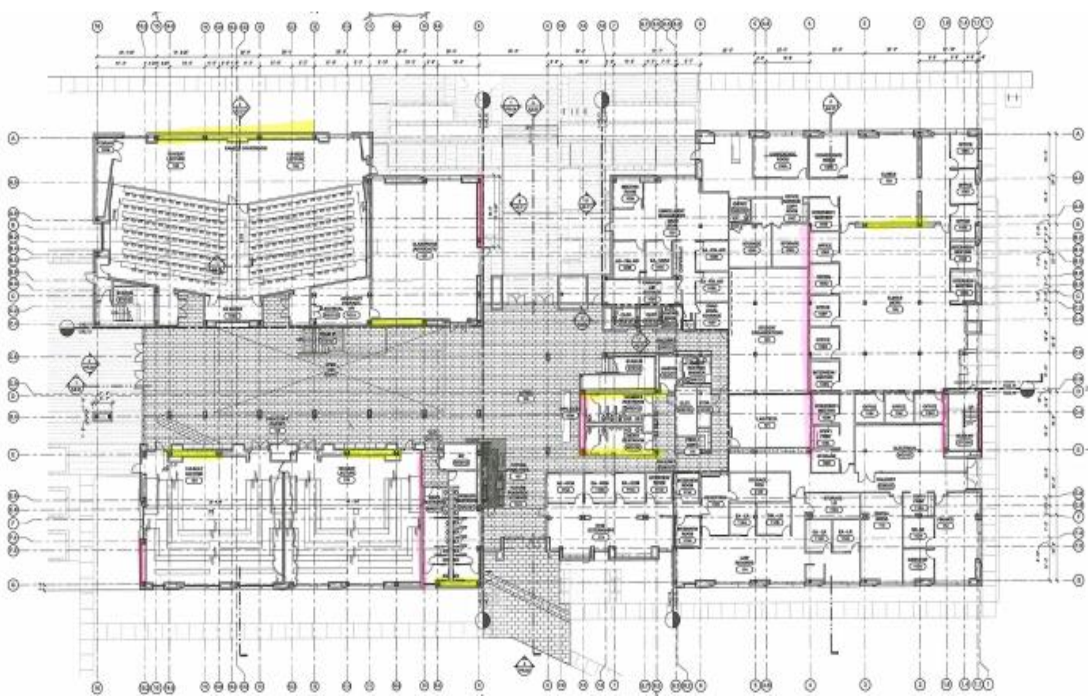

Figure 14: First story proposed shear walls.
Source: Charney Hall architectural plans [edited].

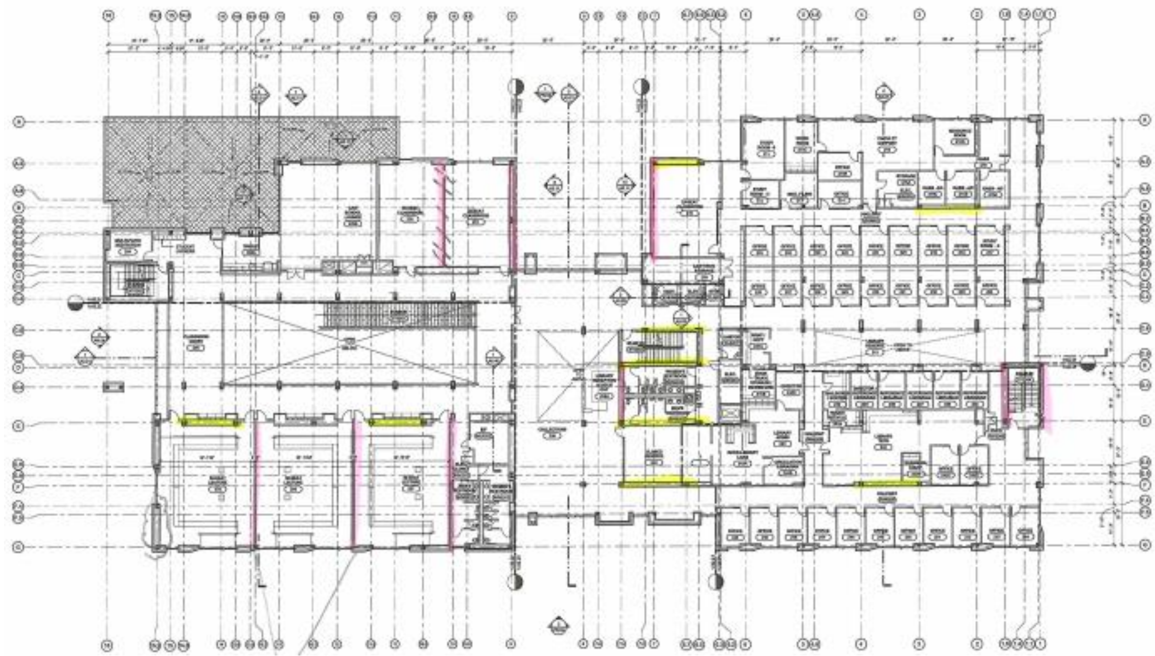

Figure 15: Second story proposed shear walls.
Source: Charney Hall architectural plans [edited].

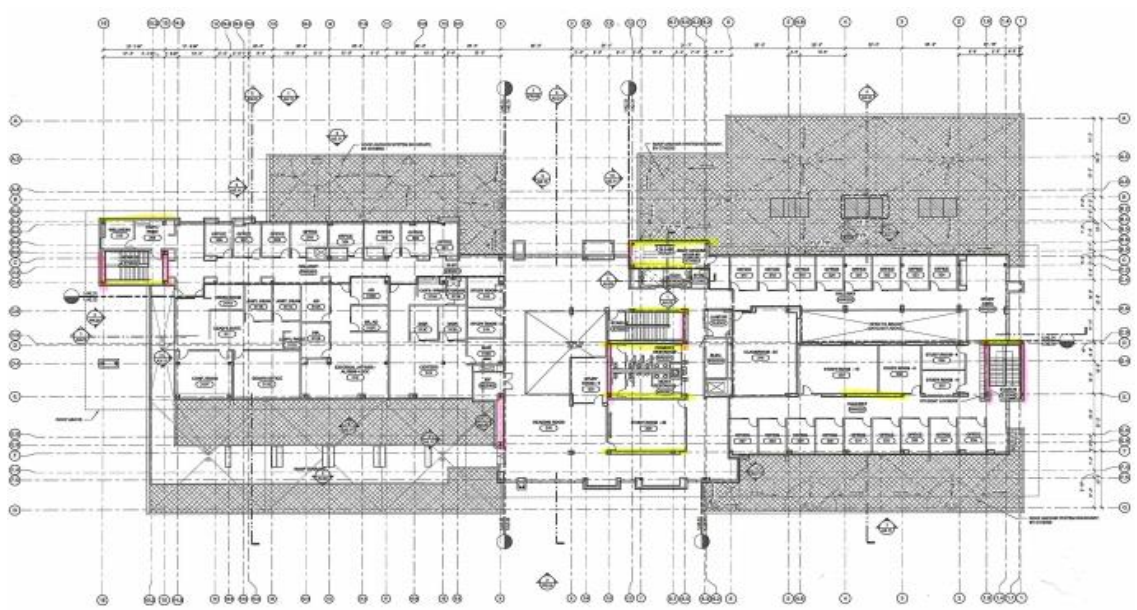

Figure 16: Third story proposed shear walls.
Source: Charney Hall architectural plans [edited].

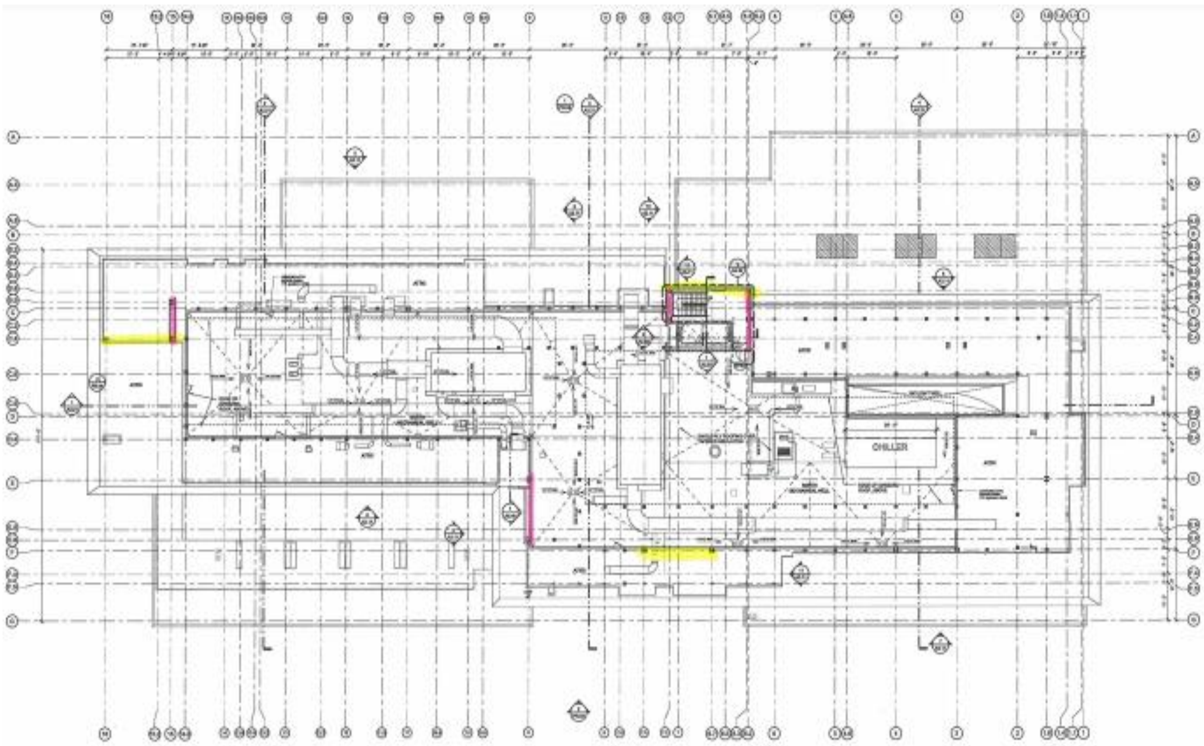

Figure 17: Mechanical deck proposed shear walls.
Source: Charney Hall architectural plans [edited].

The CLT panels used for the floor are considered rigid diaphragms, so the lateral force transferred to each shear wall was based on relative stiffness. The project team created a SAP2000 model of the structure to mimic the behavior of the building and assess whether the number and locations of anticipated shear walls was adequate for satisfying inter-story drift limits, total drift limits, and building torsion limits. One particular area of concern in the lateral design was on the first floor where the large classrooms are located because the outer wall is made up of windows that stretch over all three stories and therefore, can not be a shear wall location. The wall of windows and the 40-foot span of the classroom could contribute to the corner of the building acting as a cantilever in an earthquake and causing an unwanted amount of drift or high stress to the diaphragm transferring the load to the closest shear walls. The project team was expecting to use steel braced frames along the windowed walls as the engineers for the current structure did to account for the cantilevered portion.

Although stress did end up being concentrated in the center of the structure as shown in Figure 18, the CLT floor diaphragms were able to transfer lateral load to the shear walls, so the walls of windows did not deflect more than the allowable limits. The NDS SDPWS 2015 restricts the inter-story drift limit to two percent (2%) of the story height, which this building also satisfied even though each story had a different stiffness due to the discontinuous shear wall layout. From these results, the team discerned that the expected use of braced frames was not actually necessary to uphold the design criteria of this structure, so 12”-thick, 315 E grade shear walls were placed at the proposed locations shown in Figures 14-17, above.

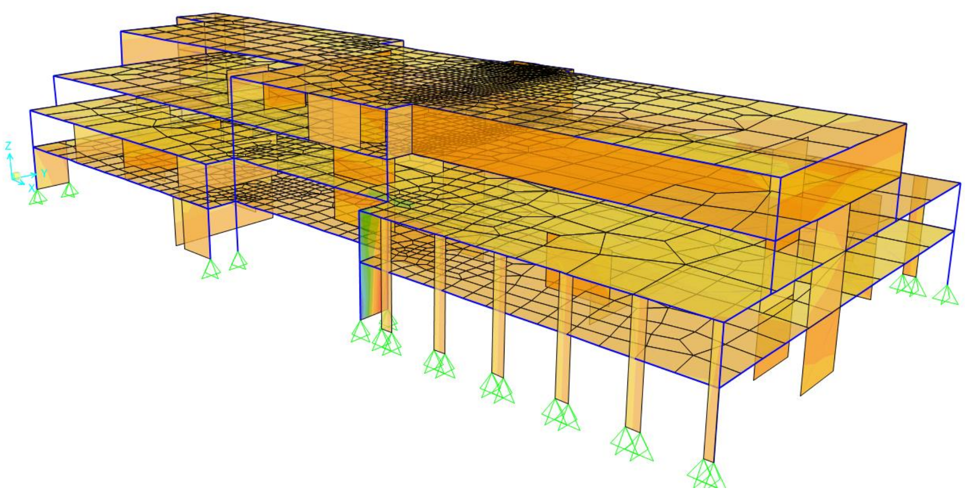

Figure 18: Stress results for the Charney Hall shear wall model.
Source: Project team using SAP2000.

4.9 Connections

With the exception of the special CLT panel connection to the glulam beam for T-beam capacity purposes, most connections were not actually designed, but schematics were chosen from common practices in countries who use engineered wood products more frequently than the United States. For example, the expected column connection, whether to foundation or a CLT floor panel, will likely mimic one of the options presented in Figure 19 that include steel plates embedded into the column that get bolted into the surface below.

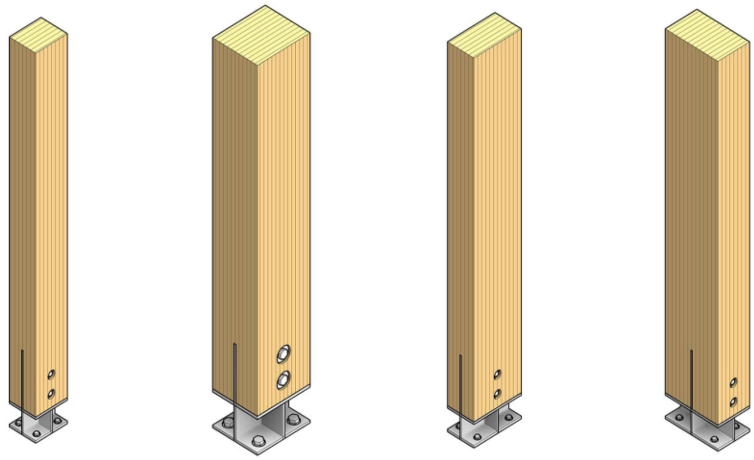

Figure 19: Schematic of glulam column connection to surface below.
Source: OBD, 2017.

Another expected connection is the glulam beams to the glulam columns in a way that simulates pinned behavior. There are a few options regarding how this may be accomplished. The first option is shown in Figure 20, where steel brackets shaped like upside-down T's are embedded in the column and then into the beam, after which the beam end is bolted to securely attach the glulam to its now interior steel plate.

Figure 20: Glulam beam to column connection using embedded steel.
Source: Iversen, 2012.

Another option for the column-beam connection is as shown in Figure 21, below, where a steel plate with a small shelf on it wraps around the top of the column and attaches to a steel cap around the end of the beam that has an attached lip intended to rest on the steel shelf attached to the column.

Figure 21: Glulam beam to column connection using steel encasing.
Source: Evans, 2013.

After investigating what the glulam connections would look like, research was done regarding how CLT panels are typically attached to each other as well as to the foundation. Most of the beams in Charney Hall have tributary areas larger than a producible and transportable CLT panel, so in many places, separate panels would need to be attached together to act as one. The best way to accomplish this connection is by using plywood splices that insert into notches in the panel before screws are run through, as shown in Figure 22. This procedure works with panels that are oriented in parallel and perpendicular directions because the notches are manufactured when the product is produced, so the lam intended for the splice will simply be manufactured shorter than the other layers.

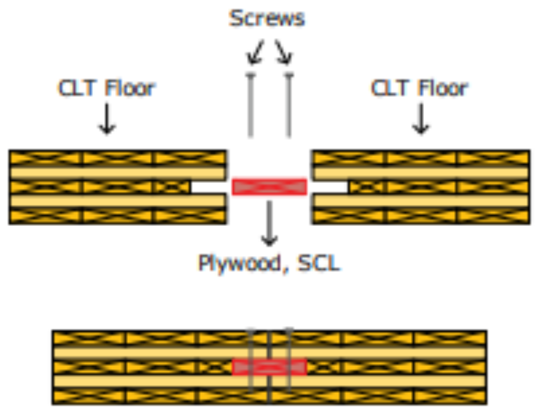

Figure 22: Schematic of CLT panel-to-panel connection.
Source: Sustainable Construction Service, 2016 CLT panel-to-panel connection.

The next consideration was how the CLT panel shear walls on the first floor would be connected to the foundation. Similar to the beam-column connections for glulam beams, upside-down T's made of steel can be embedded in the CLT and then bolted into the foundation, or embedded in both the foundation and the CLT, as shown in Figure 23. Additional anchors may be necessary at the ends of walls for the force couple resisting the moment from the lateral force, but they were not included in the scope of connection design provided by this project.

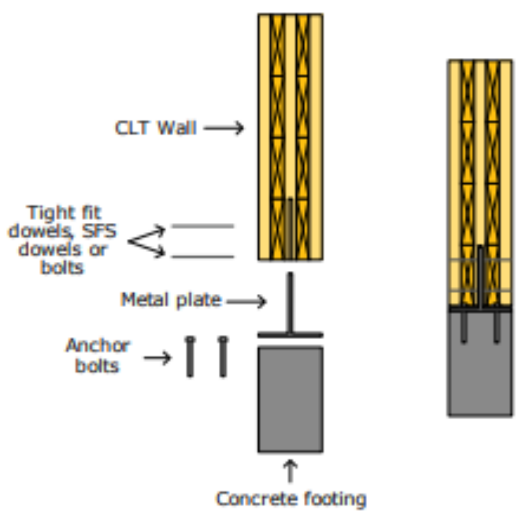

Figure 23: Schematic of CLT panel connection to foundation.
Source: Sustainable Construction Services, 2016 CLT wall-to-concrete connection.

Similarly, the same mechanisms would be used to attach a shear walls to a CLT panel if, as shown in Figure 24, a shear wall above first floor must rest on the floor diaphragm.

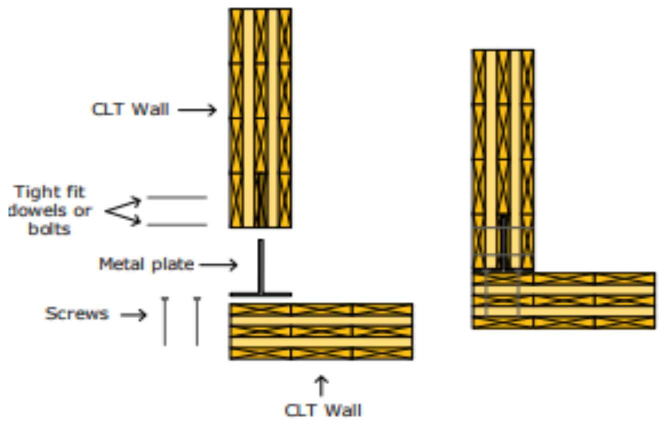

Figure 24: Schematic of CLT shear wall connection to CLT diaphragm.
Source: Sustainable Construction Services, 2016 CLT wall-to-roof/floor connection.

Finally, if shear walls are located in the same place on two consecutive floors, as shown in Figure 25, embedded steel T-sections maybe be used above and below the diaphragm.

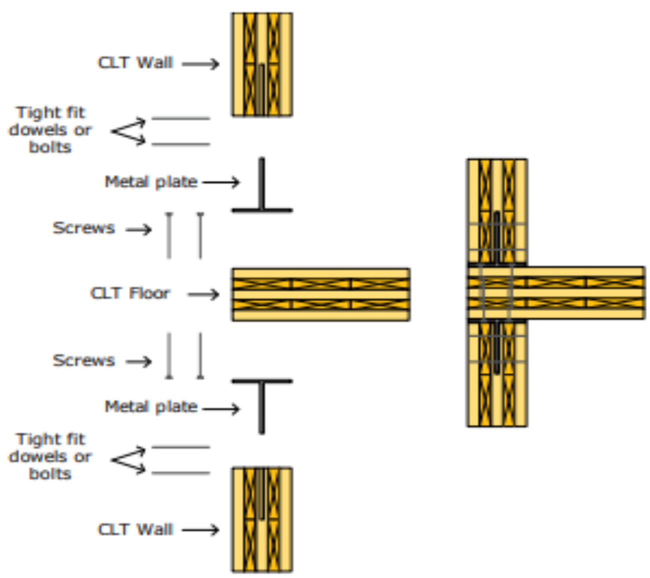

Figure 25: Schematic of CLT two-story shear wall connection to CLT diaphragm.
Source: Sustainable Construction Services, 2016 CLT wall-to-roof/floor connection.

CHAPTER 5

COST ESTIMATE

5.1 Cost Comparability Research

A specific cost estimate and comparison for this redesign was not part of the project scope because the cost for the structural portion of the existing Santa Clara University's Charney Hall Law Building could not be disclosed. Research has been done to understand how engineered wood structures typically compare with traditional steel and concrete construction like from the World Conference on Timber Engineering. They published a case study done by Maria Fernanda Laguarda Mallo and Omar Espinoza regarding a 40,000 square-foot performing arts building in Napa, California. This case seemed relevant to the Charney Hall project is because the performing arts center also needed to accommodate long spans to provide unobstructed views much like the mock courtroom and large classrooms present in the law building. The researchers for this case study evaluated the cost of the building as if it were traditional steel and concrete and then provided cost estimates for four different scenarios of varying use of Cross-Laminated Timber whose results are shown in Table 5. Basic CLT Options 1 and 2 replace the walls and roofs with CLT panels from two different manufacturers, and Green Options 1 and 2 replace the walls and roofs with CLT and the columns and beams with glulams from two different manufactures. This table signifies that the cost decreases when more engineered wood is used, which is promising because, as mentioned in Section 1.1, even construction companies and project owners who are working to be more sustainable want to have an economically reasonable project. The study also shows that using s=engineered wood reduces construction time which could save even more.

Table 5: Cost comparison between traditional construction and engineered wood construction.Source: Mallo, 2016.

Element	Concrete/ Steel option	CLT options			
		Basic CLT option 1	Basic CLT option 2	Green option 1	Green option 2
	(Concrete walls/roof, steel beams, light-steel frame)	(CLT walls/roof, steel beams, light-steel frame)		(CLT walls/roof, glulam beams, wood-frame)	
Structural Walls	$1,071,680	$624,417	$414,901	$624,417	$414,901
Concrete Slab	$256,416	$256,416	$256,416	$256,416	$256,416
Roof System	$600,975	$427,809	$289,339	$427,809	$289,339
Interior Walls*	$155,304	$155,304	$155,304	$297,666	$297,666
Steel Beams	$506,575	$506,575	$506,575	-	-
Glulam Beams	-	-	-	$29,022	$29,022
Extra CLT Walls	-	-	-	$115,407	$84,977
Extras for CLT**	-	$595,241	$595,241	$654,768	$654,768
TOTAL $	2,590,950	2,565,763	2,217,777	2,405,506	2,027,091
SQFT	40,065	40,065	40,065	40,065	40,065
Cost per sqft	$64	$64	$55	$60	$50

* Interior walls for concrete and basic CLT options are in light-steel frame construction. Interior walls for CLT Green options are in wood-frame construction.

** Extras for CLT includes labor cost and connectors for CLT

There are some concerns with this research that merit further investigation and study into the cost differences between these methods. The first concern is that it is unclear whether or not transportation costs are considered as a part of the material costs. The issue with this is that lumber and manufacturing plants are not located nearby, so the glulams and CLT panels would likely have to be transported from the Pacific Northwest or Canada, which could raise prices. Also, recent policies indicate that tariffs may be placed on Canadian lumber which could increase prices even more if the intended supplier was stationed there. Despite the concerns that merit further study, these results show promise for the future of engineered wood as both a sustainable and economic option.

CHAPTER 6

CONCLUSION

6.1 Areas for Growth

There are yet a few areas where engineered wood can improve before implementation in the United States is as effortless as other materials. Despite the incredible capacity that Glue-Laminated Timber and Cross-Laminated Timber have for allowing unique, innovative projects while reducing environmental impact, the U.S. has no specific codes for these specific engineered wood products, so it is much more challenging to get a building permit, and it requires a great deal more work from the engineer to research common procedures in other countries and stitch together multiple codes that each have some relevance. Also, heavy timber is not a widely available resource, and there are few CLT and glulam manufacturers nearby since the demand is still low, so transportation costs will be higher, especially if the tariffs on lumber from Canada do get imposed. The material itself also needs more research regarding seismic capabilities including response modification and overstrength factors, as well as other failure modes such as warping, punch through, and creep which there is currently limited information on.

6.2 Applicability of Engineered Wood Products

In spite of the remaining challenges for engineered wood, this project demonstrated its potential and how beneficial it could be to put in the work to overcome the obstacles. The completion of this design, which included the gravity and lateral systems for a structurally unique building in Santa Clara, California, proves that is is feasible to use engineered wood products in place of steel and concrete for larger construction projects in the Bay Area. The goals of this project to maintain the functionality and aesthetics of the structure while replacing as much of the steel and concrete as possible with engineered wood were highly successful. This project shows how effective engineers and architects can be regarding stewardship of the environment when making decisions about building materials and construction methods. Best of all, this project efficaciously justifies that time, energy, and money should be invested in continuing the advancement of engineered wood products in the United States.

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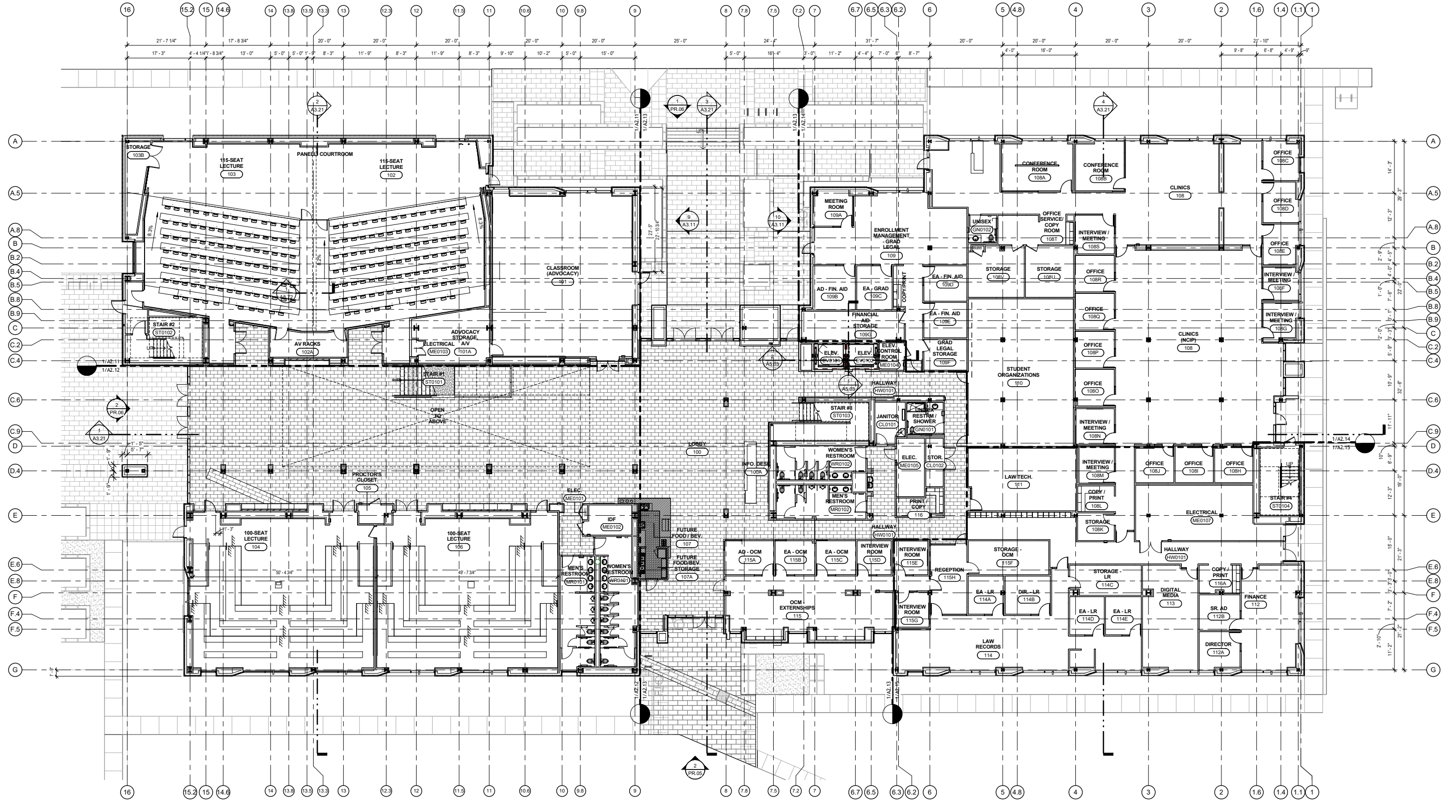
Appendix A

Architectural Plans



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GENERAL NOTES:

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2. REFER TO SHEET A4.01 FOR EXTERIOR PARTITION TYPES AND SHEET A8.01 FOR INTERIOR PARTITION TYPES. REFER TO SHEET A8.20 FOR EXTERIOR DOOR SCHEDULE INFORMATION AND SHEETS A8.21 AND A8.22 FOR INTERIOR DOOR SCHEDULE INFORMATION, INCLUDING DOOR WIDTHS AND FIRE RATED ASSEMBLIES.
3. REFER TO STRUCTURAL DRAWINGS FOR DESIGN OF GUARDRAIL ATTACHMENT TO THE BUILDING STRUCTURE.
4. REFER TO LANDSCAPE DRAWINGS FOR SITE RELATED DETAILS

NO.	DATE	DESCRIPTION	NO.	DATE	DESCRIPTION
	2016.06.17	ISSUED FOR BACKCHECK #1			
	2016.05.06	PERMIT PACKAGE			
	2016.03.11	100% DESIGN DEVELOPMENT			
	2016.01.15	50% DESIGN DEVELOPMENT			
	2015.12.01	25% DESIGN DEVELOPMENT			
	2015.10.16	100% SCHEMATIC DEVELOPMENT			

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OVERALL FIRST FLOOR PLAN

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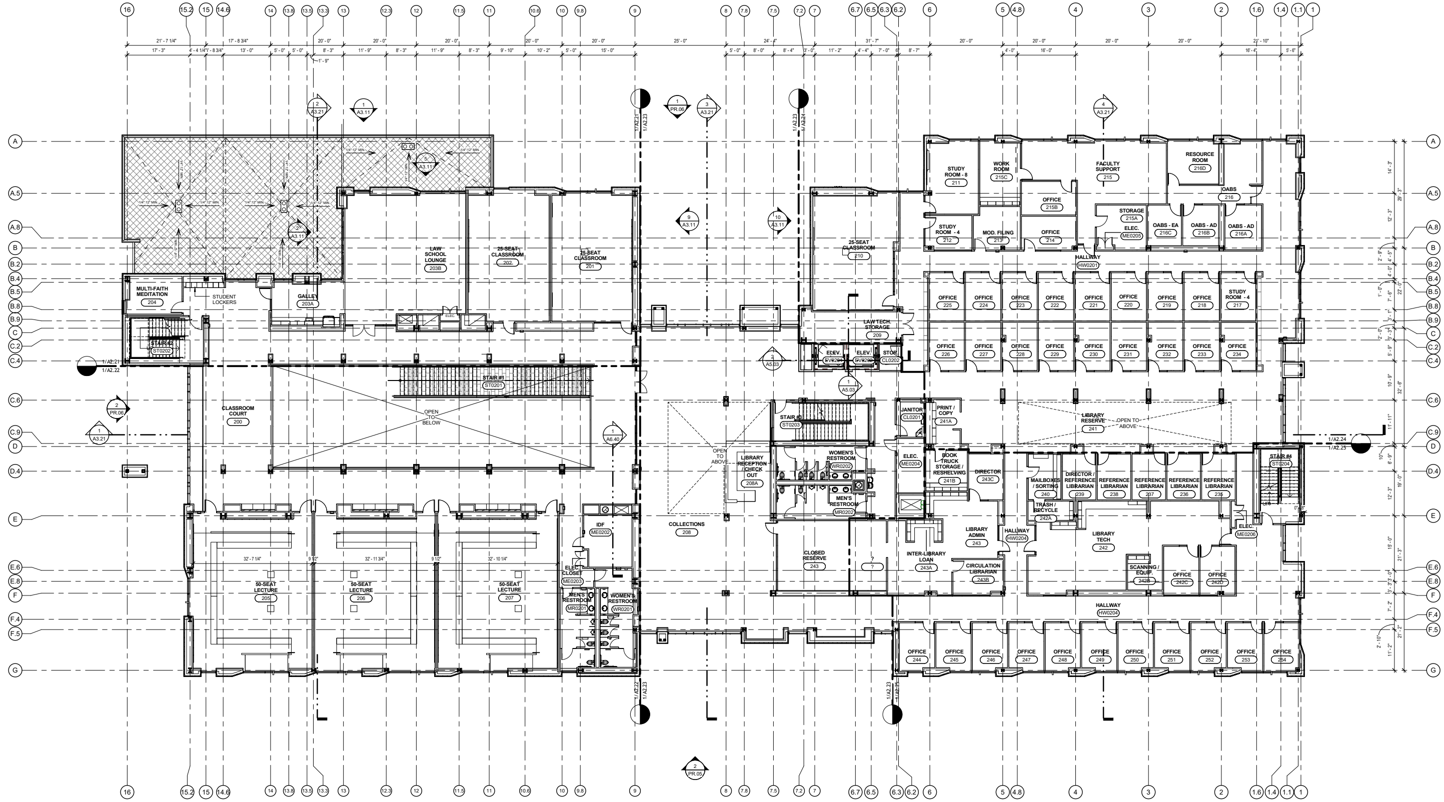
Project Number: 2015029
Sheet Scale: 3/32" = 1'-0"
Sheet Number: A2.01



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	2016.01.15	50% DESIGN DEVELOPMENT			
	2015.12.01	25% DESIGN DEVELOPMENT			
	2015.10.16	100% SCHEMATIC DEVELOPMENT			

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OVERALL SECOND FLOOR PLAN

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Project Number: 2015029

Sheet Scale: 3/32" = 1'-0"
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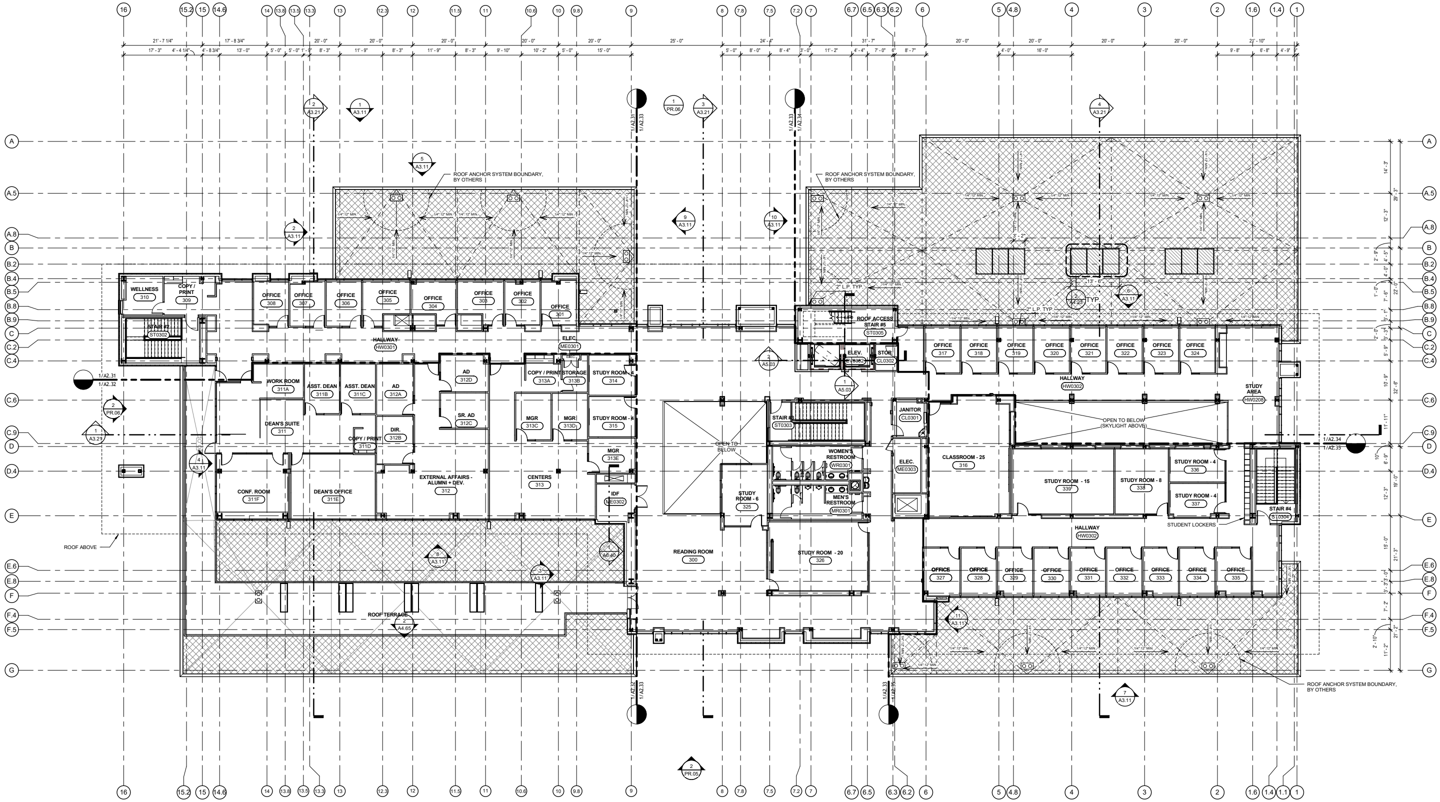

ROOF MATERIAL LEGEND:

- [07 5400] SINGLE PLY MEMBRANE ROOFING WITH BALLAST
- [07 5556] ROOF PAVERS OVER HOT FLUID-APPLIED PROTECTED MEMBRANE ROOFING
- [07 5400] SINGLE PLY MEMBRANE ROOFING
- CLAY TILE ROOF

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	2015.12.01	25% DESIGN DEVELOPMENT			
	2015.10.16	100% SCHEMATIC DEVELOPMENT			

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OVERALL THIRD FLOOR PLAN

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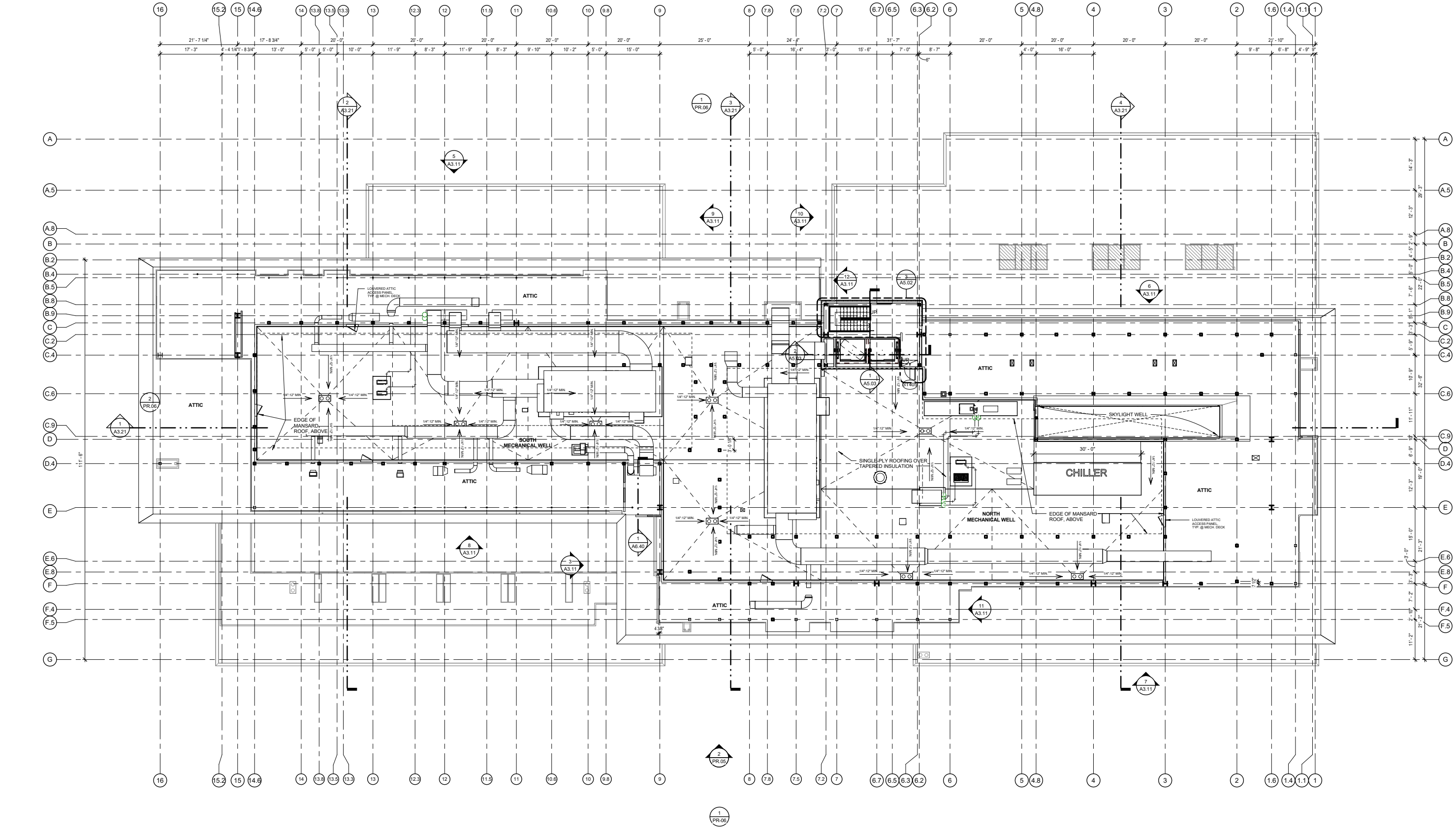
Project Number: 2015029

Sheet Scale: 3/32" = 1'-0"

Sheet Number: A2.03



ROOF MATERIAL LEGEND:

- [07 5400] SINGLE PLY MEMBRANE ROOFING WITH BALLAST
- [07 5556] ROOF PAVERS OVER HOT FLUID-APPLIED PROTECTED MEMBRANE ROOFING
- [07 5400] SINGLE PLY MEMBRANE ROOFING
- CLAY TILE ROOF

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	2015.10.16	100% SCHEMATIC DEVELOPMENT			

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OVERALL MECHANICAL DECK PLAN

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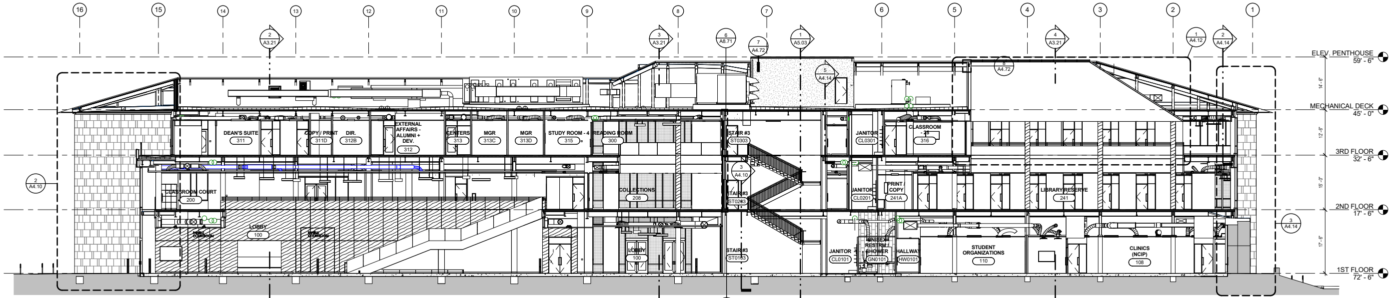
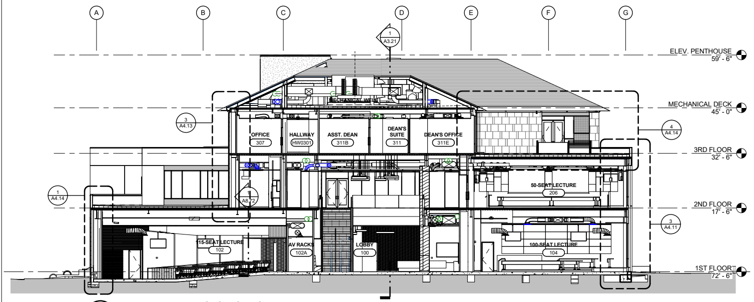
Project Number: 2015029
Sheet Scale: 3/32" = 1'-0"
Sheet Number: A2.04



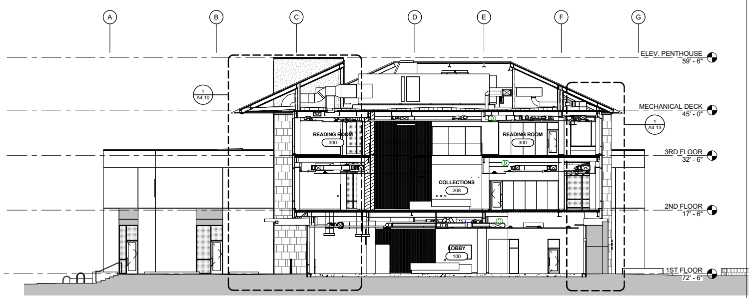
1 N-S BUILDING SECTION

SCALE: 3/32" = 1'-0"



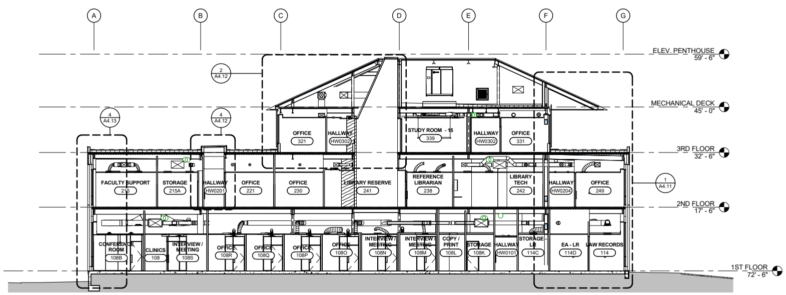
2 E-W BUILDING SECTION A

SCALE: 3/32" = 1'-0"



3 E-W BUILDING SECTION B

SCALE: 3/32" = 1'-0"



4 E-W BUILDING SECTION C

SCALE: 3/32" = 1'-0"

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BUILDING SECTIONS

Drawn By: CB / PW

Checked By: OG / JP

Project Number: 2015029

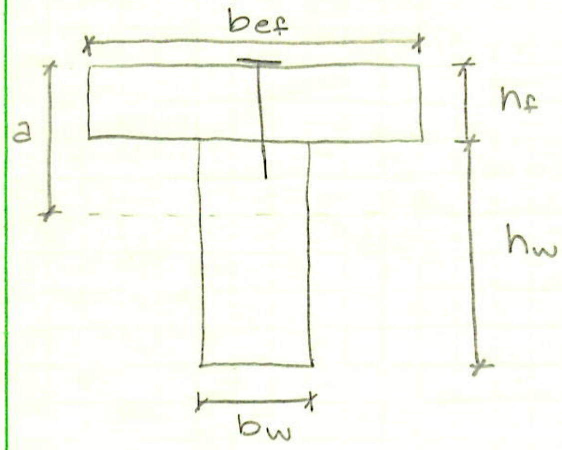
Sheet Scale: 3/32" = 1'-0"

Sheet Number: A3.21

Appendix B

Sample Beam Calculations

T-Beam Capacity



flange = CLT floor Panel

web = Glulam Beam

↳ Moment Capacity =

[Tension Force × (dist. from comp. reactant to tens. reactant)]

↳ Neutral Axis =

since $F_{CLT} \approx F_{glulam}$, neutral axis occurs where area above equals area below

↳ Compression Area $= (b_{ef} \times h_f) + ((a - h_f) \times b_w)$

↳ Tension Area $= (h_w - (a - h_f)) \times b_w$

↳ $(b_{ef} \times h_f) + ((a - h_f) \times b_w) = (h_w - (a - h_f)) \times b_w$

$2((a - h_f) \times b_w) = b_w h_w - (b_{ef} h_f)$

$a = ([b_w h_w - (b_{ef} h_f)] / 2 b_w) + h_f$

↳ Centroid of compression $= \Sigma A_c y / A_c$

↳ Centroid of tension $= \Sigma A_t y / A_t$

↳ Tension Resultant $= 0.85 f_{CLT} \times$ (tension area)

↳ Moment of Inertia $= \Sigma (I_c + Ad^2)$

↳ bef in concrete:

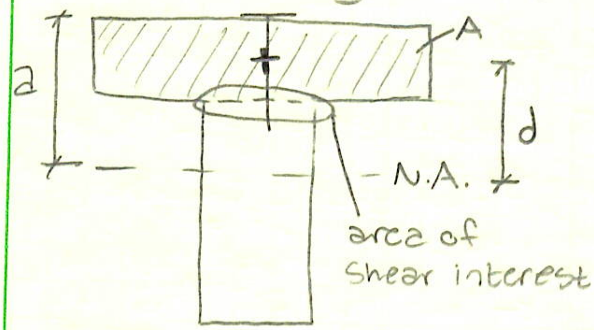
min of: $2(8)h = 2(8)(40 \text{ in}) = 640 \text{ in}$

$2 s_w/2 = (20' \times 12) - 14.25 = 225.75 \text{ in}$

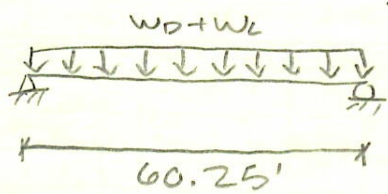
$2 \ell_n/8 = (60.25 \times 12)/4 = 180.75 \text{ in}$ ← governs

14.25" × 36" beam w/ 4" CLT panel needs $b_{ef} = 110.25$ in to resist deflection of L/480 from live load

T-Beam Lag Bolt Calcs



q = shear flow = $\frac{VQ}{I} = \frac{(lb)(ft^3)}{(ft^4)}$ = lb/ft



$w_D = 45$ psf

$w_L = 100$ psf

trib. width = 20 ft

$w_U = 1.2(45 \times 20) + 1.6(100 \times 20) = 4280$ lb/ft

$V_{u\,max} = \frac{w_u L}{2} = \frac{(4280\text{ lb/ft})(60.25')}{2} = 128{,}935$ lb

$Q = Ad$

I = moment of inertia of whole cross-section

For 14.25" x 36" beam w/ 4" CLT:

$q = 16{,}440$ lb/ft ($b_{ef} = 110"$)

Highest capacity of lag screws ≈ 2000 lbs

Example Beam Calc:

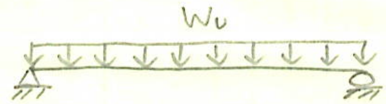
↳ Using cartroom beam under 2nd floor classrooms

Length: 51.25 ft

Trib. Width: 20 ft

Live Load: 100 psf

Dead Load: 45 psf



$E = 2{,}100{,}000$ psi
$\Delta_{max} = L/480$

Beam Demands:

$W_U = 1.2D + 1.6L = 1.2(45 \times 20') + 1.6(100 \times 20') = 4280$ lb/ft

$V_U = wL/2 = (4280 \text{ lb/ft})(51.25\text{ft})/2 = 109{,}675$ lb

$M_U = wL^2/8 = (4280 \text{ lb/ft})(51.25\text{ft})^2/8 = 1{,}405{,}211$ lb-ft

$$I_{req'd} = \frac{5wL^3(480)}{384E} = \frac{5(4280 \text{ lb/ft}/12)(51.25\text{ft}/12)^3(480)}{384(2{,}100{,}000 \text{ lb/in}^2)} = 115{,}381 \text{ in}^4$$

Try a 12"×36" section:

↳ use wood in stress class 28F-2.1E SP

$b_{ef} = 85$ in

comparison to concrete:

↳ $b_{ef} = \min \begin{cases} 2(8)h = 2(8)(48\text{in}) = 768\text{in} \\ 2s_w/2 = (20' \times 12) - 12\text{ in} = 228\text{ in} \\ 2l_n/8 = (51.25 \times 12)/4 = 153.75\text{in} \end{cases}$

↳ 55% of concrete requirements

$\phi V_n = 469{,}129$ lb $> 109{,}675$ lb ✓

$\phi M_n = 1{,}407{,}194$ lb-ft $> 1{,}405{,}211$ lb-ft ✓

$I = 264{,}410 \text{ in}^4 > 115{,}381 \text{ in}^4$ ✓

Appendix C

Beam Schedule

Beam Number:	Width (in):	Depth (in):
1	12	36
2	3	6.875
3	3.5	6.875
4	5	6.875
5	5.5	6.875
6	6.75	6.875
7	8.5	6.875
8	3	11
9	3.5	11
10	5	12.375
11	5.5	12.375
12	6.75	17.875
13	8.5	24.75
14	3	8.25
15	3.5	8.25
16	5	13.75
17	5.5	13.75
18	6.75	20.625
19	8.5	27.5
20	3	13.75
21	3.5	13.75
22	5	13.75
23	5.5	13.75
24	6.75	23.375
25	8.5	30.25
26	3	19.25
27	3.5	19.25
28	5	19.25
29	5.5	19.25
30	6.75	19.25
31	8.5	28.875
32	3	22
33	3.5	22
34	5	22
35	5.5	22
36	6.75	22
37	8.5	33
38	3	24.75
39	3.5	24.75
40	5	24.75
41	5.5	24.75
42	6.75	24.75

43	8.5	34.375
44	3	26.125
45	3.5	26.125
46	5	26.125
47	5.5	26.125
48	6.75	26.125
49	8.5	26.125
50	14	36

Appendix D

Column Calculation

Column Design

First Floor

Total Weight of Structure = 10,258.1 kips

Total # of columns on 1st floor: 102

10,258.1 k / 102 columns = 100.6 k / column

↳ Per Table 17 of APA Design of Structural Glulam Columns a 15-foot column that's 10½" x 13¾" has a capacity of 109 kips. Therefore, a 12" x 12" column will work since it has a similar area.

↳ Use 12" x 12" columns everywhere.

TABLE 17

ALLOWABLE AXIAL LOADS (POUNDS) FOR COMBINATION NO. 47 SOUTHERN PINE GLULAM COLUMNS
Side loads are not permitted. End loads are limited to a maximum eccentricity of either 1/6 column width or depth, whichever is worse.

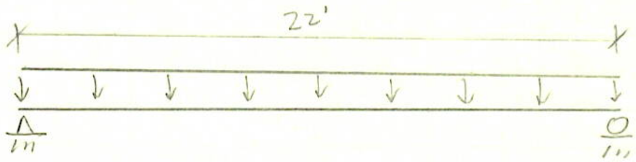
Effective Column Length (ft)	Lamination Net Width = 10-1/2 in.								
	Net Depth = 11 in. (8 lams)			Net Depth = 12-3/8 in. (9 lams)			Net Depth = 13-3/4 in. (10 lams)		
	Load Duration Factor			Load Duration Factor			Load Duration Factor		
	1.00	1.15	1.25	1.00	1.15	1.25	1.00	1.15	1.25
8	105,144	119,084	128,127	120,417	136,731	147,360	135,506	154,145	166,328
9	102,352	115,446	123,870	117,751	133,229	143,242	132,935	150,747	162,311
10	99,308	111,493	119,256	114,805	129,365	138,698	129,841	146,724	157,600
11	96,033	107,256	114,325	111,423	124,987	133,590	126,356	142,170	152,245
12	92,485	102,720	109,080	107,709	120,172	127,971	122,608	137,247	146,448
13	88,649	97,846	103,477	103,780	115,087	122,051	118,580	131,948	140,214
14	84,699	92,874	97,806	99,649	109,774	115,914	114,269	126,001	132,199
15	80,676	87,879	92,177	95,350	104,321	109,691	109,644	118,673	123,982
16	76,637	82,958	86,701	90,943	98,842	103,527	103,700	111,496	116,048
17	72,647	78,196	81,467	86,512	93,457	97,552	97,860	104,603	108,525
18	68,768	73,655	76,527	82,136	88,255	91,337	92,223	98,082	101,485
19	65,047	69,370	71,906	77,886	82,781	85,456	86,855	91,978	94,952
20	61,514	65,355	67,605	73,617	77,673	80,026	81,796	86,304	88,918
21	58,182	61,610	63,616	69,355	72,944	75,024	77,062	81,049	83,360
22	55,054	58,127	59,921	65,384	70,102	70,424	72,649	76,195	78,249
23	52,127	54,891	56,503	61,692	64,544	66,195	68,547	71,716	73,550
24	49,393	51,888	53,341	58,265	60,824	62,304	64,739	67,583	69,227

Appendix E

Diaphragm Calculation

Diaphragm Design

2^{ND} FLOOR - CLASSROOMS (E)



DL = 45 PSF
LL = 100 PSF

LC: D + L

AREA: l = 22'
w = 42.42'

A_T = 933.24 ft^2

REDUCTION: $L^* = (100)\left(0.25 + \frac{15}{\sqrt{933.24 \times 1}}\right) = 74.1$ psf

$W = D + L^*$
$= 45 + 74.1 = 119.1$ psf / ft

$M_u = \frac{(119.1)(22)^2}{8} = \underline{7205.64 \text{ \#-ft/ft}}$

$V_u = \frac{(119.1)(22)}{2} = \underline{1310.1 \text{ \#/ft}}$

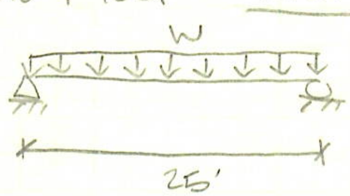
→ USE $\boxed{\text{E1MS - 315 E}}$ M_A = 30,838 #-ft ✓

V_A = 3000 # ✓

L_{MAX} = 26.6 ✓

→ Since max span for cartroom is 21.6 feet, E1MS - 315 E also works there.

2nd Floor - Library Reserves (NW)



DL = 45 psf
LL = 150 psf (non reducible)

w = 45 + 150 = 195 psf

$M_U = \frac{(195 \text{ lb/ft/ft})(25 \text{ ft})^2}{8} = 15{,}234.375$ lb-ft / ft

Diaphragm Design

$$V_u = \frac{(195\ lb/ft/ft)(25\ ft)}{2} = 2437.5\ lb/ft$$

↳ Use EIM5-315E

$M_{allow} = 30838\ lb/ft > M_u$ ✓

$V_{allow} = 3000\ lb/ft > V_u$ ✓

$L_{max} = 26.6' > 25'$ ✓

↳ Use EIM5-315E panels for all floors and roof because if they work in the court-room and library, they will work anywhere

Appendix F

Lateral Demand Calculation

Total Weight:

↳ Level 1: Area $= 40730.07 ft^2$

Dead:

+ panels = 25 psf

+ beams & columns $\approx 35\ lb/ft^3$ (glulam beam design)

volume beams per court room: $(36 \times 24/144)(528.39 ft) = 3170.34 ft^3$

volume columns: $97(12 \times 12/144)(17.5 ft) = 1697.5 ft^3$

Total weight $= 35 lb/ft^3[((1697.5 ft^3)/40730.07) + 3170.3/5935.2]$

+ Total Dead = 25 psf + 20.15 = 45.15 psf

Live:

+ $150 psf \times [(38 ft \times 63 ft) + (33.83 ft \times 10 ft) + (50.3 \times 21.25) + (60 \times 11.917)]$

$= 677,426.25 lb$

+ $20 psf \times [(59.3 \times 37.6) + (14.25 \times 40)] = 55993.6\ lb$

+ $80 psf \times [(14 \times 110.917) + (40.25 \times 60) + (19 \times 39.83) + (20 \times 53) + (8.3 \times 26) + (5.75 \times 7) + (89' \times 29.25) + (89 \times 26.5) + (20 + 34) + (13.58^2 \times 20) + (8.5 \times 16.25)] = 967,375.44$

+ $100 psf \times [(35120.29 - 4516.195 - 2799.68 - 12092.193)] = 1571222 lb$

+ Total Live $= (677425 + \frac{55993}{2} + \frac{967375}{2} + \frac{1571222}{2}) / 35120.29 ft^2$

$= 56.22 lb/ft^2$

↳ Level 2: Area $= 37,925.18 ft^2$

Dead:

+ panels = 25 psf

+ beams & columns $\approx 35\ lb/ft^3$

volume beams per court room: $(36 \times 24/144)(769.\overline{6} ft) = 4617.996 ft^3$

volume columns: $(77)(15) = 1155 ft^3$

Total weight $= 35 lb/ft^3[(1155 ft^3/37925.18) + (4617.996/7851.3)] = 20.70 psf$

+ Total dead = (25psf) + 20.70 psf = 45.70 psf

Live:

+20 psf × [(2039.2 + 6232.2 + 2259.2 + 1411.3 + 2034)] = 279 518 lb

+ 80 psf × (4802.5 + 1317.8 + 278.7 + 1308.8 + 1074.1 + 1114.7 + 1441) = 907008 lb

+ 100 psf × (37925.18 − 13975.9 − 11337.6) = 1261168 lb

+Total live = (279518 + 907008 + 1261168 lb) / 37925.18 ft^2

= 33.25 psf

↳Level 3: Area = 21782.13 ft^2

Dead:

+panels = 25 psf

+beams and columns ≈ 35 lb/ft^3 (glulam beam design)

volume beams: (36×24/144)(533) = 3198 ft^3 / 4068 ft^2

volume columns: (5"×5"/144)(12.5ft)(112) = 243.06 ft^3

Total Weight = 35 lb/ft^3((3198/3983.25) + (243.06/21782.13))

+Total Dead = 25 + 27.9 = 52.9 psf

Live

+20 psf × (21782.13 − (25.5×16)) = 427482.6 lb

+100 psf × (25.5 × 16) = 40800 lb

+Total live = (40800 + 427482.6)/21782.13 = 19.6 psf

↳Mech Deck (Level 4): Area = 27440.2 ft^2

Dead:

+ panels = 25 psf

+ roofing + tiles = 11.7 psf

+Total Dead = 25 psf + 11.7 psf = 36.7 psf

Live:

+ 20 psf × (27440.2 ft^2) = 548803 lb

+ Total live = 20 psf

Total Weight Summary

↳ Level 1 = 101.37 psf (40730.07 ft^2) = 4128.8 kips

↳ Level 2 = (45.70 + 33.25)(37925.18) = 2994.2 kips

↳ Level 3 = (52.9 + 19.6)(21782.13) = 1579.2 kips

↳ Mech Deck (Level 4) = (36.7 + 20)(27440.2) = 1555.9 kips

↳ Total Weight = 10258.1 kips

Equivalent Lateral Force Procedure:

↳ Site Class D - USGIS Soil Type Map

↳ Risk Category III - high occupancy

↳ Design code Reference IBC 2015

$$\left.\begin{aligned} S_s &= 1.5g \\ S_1 &= 0.6g \\ S_{DS} &= 1.0g \\ S_{D1} &= 0.6g \end{aligned}\right\} \text{USGIS Seismic Design Maps}$$

↳ Seismic Importance Factor: 1.25 → ASCE 7-10 Table 1.5-2

↳ Seismic Design Category D

↳ from Table 11.6-1 + Table 11.6-2 in ASCE 7-10 because $S_{DS} \geq 0.50$ + $S_{D1} \geq 0.20$

↳ Approximate Fundamental Period, T_a

$$T_a = C_t h_n^x$$

$$\left.\begin{aligned} C_t &= 0.02 \\ x &= 0.75 \end{aligned}\right\} \text{Table 12.8-2 for "all other systems"}$$

highest Period = 1.18 sec

$h = 59.5\ ft$

lowest period = 0.34 sec

$$T_a = (0.02)(59.5\ ft)^{0.75} = 0.43\ sec$$

↳ Seismic Response Coefficient, C_s

$$C_s = S_{DS} / (R/I_e)$$

$R = 4.3$ → Approx. R-Factor for CLT article

$$C_s = (1.0g)/(4.3/1.25) = \boxed{0.291}$$ ← governs

$$C_{s,max} = S_{D1} / T(R/I_e) = (0.6g)/(0.43)(4.3/1.25) = 0.406$$

$$C_{s,min} = 0.044\ S_{DS} I_e = (0.044)(1.0g)(1.25) = 0.055$$

$C_{s,min} = 0.5S_1/(R/I_e) = 0.5(0.6g)/(4.3/1.25) = 0.087$

↳ Base Shear, V

Total Weight = 10258.1 kips see attached

$V = (0.291)(10258.1k) = 2985.1$ k

↳ Lateral Force per level

Level	Weight	Height	$(W_x \cdot h_x^k)$	C_{vx}	F_x
Mech	1555.9 k	59.5'	92576.05	0.278	829.9
3	1579.2k	45'	71064	0.213	635.8
2	2994.2k	32.5'	97311.5	0.292	871.6
1	4128.8k	17.5'	72254	0.217	647.8
			Σ 333205.6		Σ 2985.1

$T = 0.43 < 0.5$ so $k = 1.0$

$C_{vx} = (W_x h_x^k)/\Sigma W_i h_i^k$