Mass Timber High-Rise Design Research:
Museum Tower in Los Angeles Reimagined in Mass Timber

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Abstract

This study demonstrates a design of a code-compliant, high-rise mass timber apartment tower in Los Angeles. Using the existing reinforced concrete Museum Tower Apartment building in downtown Los Angeles as a basis, the study demonstrates architectural, structural and fire performance improvements and tradeoffs of the mass timber design compared to the reinforced concrete design.

The existing, 20 story building is a reinforced concrete perimeter moment frame with a beamless interior utilizing post-tensioned slabs. Cladding is painted structural concrete and window wall glazing. No additional fireproofing is added to the concrete structure. The theoretical mass timber building is designed to match the existing building architectural massing, but uses wood-steel buckling-restrained brace frames, glulam columns and beamless composite concrete-cross laminated timber floor slabs. Cladding is weather-coated mass timber and window wall glazing. Fire protection is provided by oversized structural members with a sacrificial char layer as well as intumescent paint on exposed steel connections. The study demonstrates that mass timber provides a viable alternative to reinforced concrete construction in Los Angeles.

Introduction

Mass timber structures offer sustainable advantages over steel and concrete because wood is a naturally renewable resource with relatively low embodied energy. Advancements in structural wood materials, such as cross laminated timber (CLT), have made larger and taller structures possible and opened up the possibility for wood to replace steel and concrete in many applications.

Various research projects, such as the Tall Timber Research Project by SOM (2013), have demonstrated design approaches to tall wood buildings in the United States, though to date no modern building in the United States has exceeded the prescriptive building code height maximums to enter the realm of 'tall' wood buildings. However, prescriptive-code-exceeding (or 'tall') wood projects are on the cusp of a breakthrough in the United States, supported largely by the USDA tall wood buildings competition, launched in 2014 with an offering of up to $2M to offset the cost of tall wood design projects.

Perkins+Will exceeded the Canadian building code's prescriptive maximum height limitation in 2012 with the completion of a tall wood building in Vancouver, BC – the University of British Columbia's Earth Sciences Building. Following this project, a continued interest in tall wood buildings has produced the following research:

Summary Report: Survey of International Tall Wood Buildings, (FII, BSLC, 2014). This report gathered data from ten modern tall wood buildings worldwide and documented stakeholder experiences and lessons learned from the projects. The goal of the report was to share data from completed projects with the global design community and the public.

In another study, Perkins+Will surveyed 518 randomly selected respondents in the United States to understand the general public's perceived barriers to tall wood building design and construction. The survey, by Shawna Hammon, AIA, presented the following twelve potential barriers and asked respondents to rank them in order of importance: Fire, strength, deforestation, durability, moisture, termites, insurance costs, construction cost, construction time, acoustic performance, livability and aesthetic appeal.

Fifty-two percent of respondents indicated they would be willing to live in a tall wood building. Those respondents who claimed to be not at all familiar with tall wood buildings pinpointed fire, deforestation, and strength as the greatest barriers while those who said they were familiar with tall wood construction identified moisture, insurance costs, construction cost and durability as the greatest barriers.

Clearly there is a wide range of education required in order to reconcile codes and public perception with material science, performance based design and construction methods.

Purpose of Study

The purpose of this research is to generate new knowledge surrounding tall wood buildings for use by designers, the public, and authorities having jurisdiction. To understand the position of the Los Angeles Department of Building and Safety on tall mass timber projects, the authors met with several members of the structural department. Discussions revealed that what authorities having jurisdiction are looking for is a clear, code-approvable approach to tall wood design. By showcasing the combination of performance based fire design and a materials science-based structural design combined with an acknowledgment of required ICC-approved testing, this study seeks to demonstrate a clear, code-approvable path to high-rise mass timber construction in Los Angeles.

Goals

1. To meet or exceed existing concrete tower performance in the following areas: vibration, deflection, lateral force resistance, thermal performance, acoustic performance and compliance with modern code requirements.

2. To demonstrate a mass timber design that is competitive with the low floor-to-floor heights (8'-8") of the existing post-tensioned slab tower.

3. To demonstrate fire performance equivalency to Type 1A construction.

1. Architectural Design

The residential design of Museum Tower has been re-imagined to meet current market demand for sustainable living, in-unit services and functional planning. Floor to ceiling window walls and in-unit washer-dryers as well as wall-less open plans in living spaces contemporize the design.

The exterior expression of the building seeks to expose various wood elements including exterior wood columns, wood-steel buckling-restrained braced frames, and cross laminated timber soffits.

1.1 Floor Plan Layout

For the purposes of this study, the concrete tower is idealized to have 20 identical floors (see Figures 1.1a and b). In reality, the first three floors of the existing Museum Tower Apartment building are podium levels with a larger floor plate (to accommodate diverse programs on these levels). The penthouse and mezzanine have also been disregarded. The prototype mass timber tower also includes 20 identical floors which share a layout that was developed in this study (see Figures 1.2a and b). This idealization of the concrete and timber models is done to facilitate a more direct comparison.

The architectural floor plan for the wood tower has a different layout than the concrete tower based on different column spacing. Columns are pulled in from cantilevered slab edges and are more closely spaced than in the reinforced concrete tower. The arrangement of all units, stairs and elevators in the wood tower results in an efficient double loaded corridor and eliminates the blind corners of the concrete tower.

The concrete tower typical floor plan has twelve units, some of which are repeating, which results in eight unique unit types. The wood tower floor plan also has twelve units, with three unique unit types. Each type is repeated four times on the typical floor plan with minor variations to accommodate local conditions such as interior braced frames and stair cores. The layout of the typical floor meets the following design criteria. Improvements relative to the existing concrete tower are noted with an asterisk (*).

- A double loaded corridor serves all the units *
- Two egress stairs serve each floor
- Two passenger elevators and one freight per floor.
- Each floor has a telecom closet
- Each floor has a trash room with waste chute for recycling and trash
- Space for a washer/dryer is provided in each unit *
- All units have access to private balconies *
- Outward-facing exterior walls have floor-to-ceiling

window wall glazing, except at braced frames. *
(upturned perimeter moment frame beams of the
concrete tower raise the window sill 26" above the floor
level)
- Each floor of the wood tower contains four of each
  of the following unit types:
    -Two Bedroom: Gross leasable area (GLA): 995sf
    -One Bedroom: GLA: 667 - 799sf
   -Studio Units: GLA: 592sf

1.2 Beamless Floor Design and Building Height

To minimize the floor-to-floor heights of the wood tower, a
beamless floor design is used. Floor slabs are composed of
two slab types. Thicker slabs (2.5" composite topping over 7
plies of 1.375" laminations = 12.125" overall thickness) span
between columns and act like very wide, shallow beams.
Thinner slabs (2.5" composite topping over 5 plies of 1.375"

laminations = 9.375") span in the transverse direction and are
supported by the thick slabs.   All slabs are oversized to
maintain strength after charring in a fire event (see Sections 2
and 3 below).   This allows wood to be exposed on the
underside of the structural decks, revealing the warmth of the
natural finish to the living area. The increased height at the
thinner slabs provides space to route utilities and conduit,
which are concealed by gypsum board soffits.

The beamless floor design minimizes the required floor to
floor height for the wood tower. With an 8'-0" interior clear
height in living spaces, the typical floor to floor is 9'-0 1/8".
Typical floor to floor height for the concrete tower with 8"
post-tensioned slabs is 8'-8".

The existing concrete tower has atypical floor heights on the
first four floors and top two floors as follows: Ground: 14'-3",
$2^{nd} – 4^{th}$: 10'-6", 19th: 11'-0", 20th: 12'-0". In these instances,
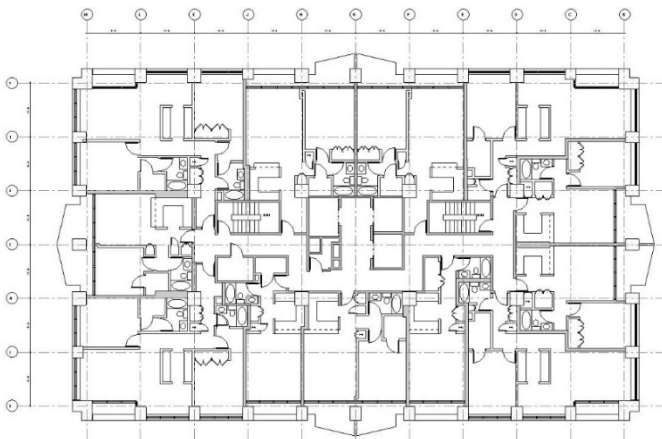
**Figures 1.1a and b. Floor Plan and Perspective View
of Existing Museum Tower Apartment Tower**
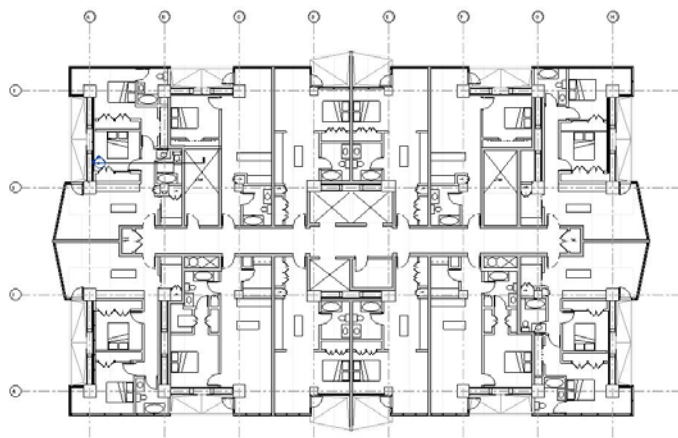
**Figures 1.2a and b. Floor Plan and Perspective View
of Prototype Mass Timber Apartment Tower**

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the wood tower floor to floor height matches the concrete tower floor to floor height.

The resulting overall height of the wood tower is 194'-10 ¾" while the existing concrete tower is 190'-1".

1.3 Wall and Floor Acoustics

Many contemporary mass timber design approaches use CLT as a load-bearing and lateral force resisting element within party walls. This study takes a different approach that is more typical of high-rise steel and concrete construction – an open floor plan with demising walls that accommodate future plan revisions without modification to the structure.

For party walls, USG's RAL-TL-87-140 is selected as a high acoustic performing (STC 60) space-efficient, 8"-wide, UL listed assembly with one layer of 0.5" sheetrock attached to 6" 20 gauge studs with 3" insulation, a channel fastened to the studs and supporting two more layers of 0.5" sheetrock. Stud tracks are mounted directly to the 2.5" concrete composite floor deck at the bottom and to the under-side of the CLT slab at the top. Joints between wall elements are assumed taped, with caulking is assumed to be applied at wall to concrete floor joint and wall-to-underside -of-CLT slabs to reduce flanking.

CBC 2015 Section 1207 requires floor/ceiling assemblies to achieve a sound transmission class (STC) rating of 50 or higher and an impact insulation class (IIC) of 50 or higher. There are two floor/ceiling assemblies in the wood tower. The thin slabs are composed of 2.5" concrete topping, 5-ply CLT, insulation and suspended gypsum soffiting. The CLT Handbook (2013) identifies similar assemblies (without concrete topping) that are able to obtain sufficiently high STC and IIC ratings to satisfy code requirements. However, testing of the specific assembly is required to confirm code-compliance. The thicker slabs are composed of 2.5" concrete topping over 7-ply CLT with exposed wood ceiling. Testing is required to determine the ability of this assembly to obtain the code-required acoustic performance.

1.4 Moisture and UV Protection

All mass timber elements of the wood tower require special care to ensure protection from moisture during manufacture, shipping, construction and service. All end grains are to receive volatile organic compound (VOC) compliant oil-alkyd primer in the manufacturing facility to prevent water absorption prior to building completion. If wood surface becomes wet during shipping or construction, wood is to be dried before application of finish.

Unfinished mass timber surfaces are subject to ultraviolet light (UV) during shipping and construction, which has a

weathering effect on the wood. Stains perform very well on weathered surfaces and provide a rich visual texture and aesthetic expression of the wood. Additionally, oil-based stains are not subject to the flaking common with paint products and they help to minimize checking and shaking of the wood surface. Pigmentation in the stain helps protect the wood from UV degradation. All exposed wood surfaces are to receive two coats of VOC-compliant, oil-based semi-transparent stain, such as Sansin SDF (32g/L VOC). Prior to application of stain on exterior, surfaces are to be roughened to saw-textured roughness to ensure optimal penetration of the stain into the wood. The design of the wood tower recesses exterior faces of columns, beams and braces within balcony overhangs, which helps to reduce the exposure to weather.

CLT slab edges at the building perimeter are protected from the elements with metal panel cladding to ensure weather-susceptible end-grains are encapsulated.

1.5 Thermal Performance

Similar to the existing concrete tower, the thermal envelope of the wood tower is floor-to-ceiling window wall glazing (Figure 1.3) and exposed structural framing.

The existing concrete tower and the tall wood tower both have significant exterior floor area in the form of balconies. Additionally, both buildings have exterior-exposed structural framing. Thermal transfer through the exterior-exposed elements results in significant heat loss and heat gain in a desert climate such as Los Angeles.
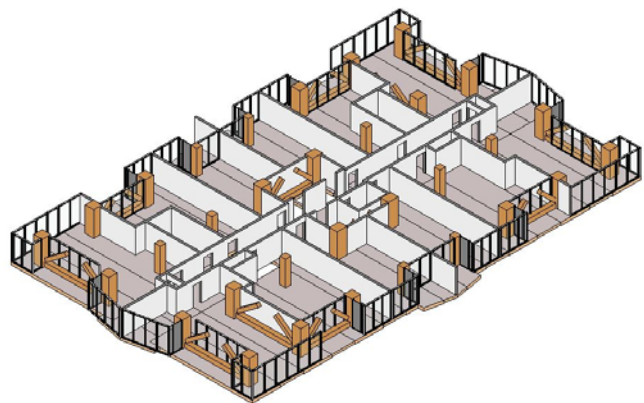

Figure 1.3 Typical Floor with Perimeter Window wall Glazing and Balconies

Concrete conductivity for normal weight concrete is 1.6 W/(mK) (Wang et. al., 2012). CLT conductivity ranges from $\lambda = 0.11$ to 0.14 W/(mK) depending on wood type and manufacturer. The WWPA Product Use Manual provides species-specific guidance. Though there are currently no CLT

manufacturers in California, Douglas Fir-Larch, plentiful in California, has conductivity of $\lambda = 0.14$ W/(mK), and could be used in the future as a sustainably-managed, locally-sourced, mass timber material with good thermal performance.

Because the thermal conductivity of CLT is roughly 10 times less than the conductivity of concrete, this study assumes the thermal performance of the exposed CLT-concrete composite balcony and framing elements slabs of the timber tower will represent a significant improvement over the exposed slabs of the existing concrete tower. Thermal transfer through composite concrete topping at wood tower balconies is mitigated by a continuous, 2.5" deep by 3" wide insulating foam block-out under the window wall sill.

A thermodynamic analysis of the building shell is recommended to study the performance of exposed wood buildings with window walls relative to exposed concrete buildings with window walls.

1.6 Slab Penetrations

Chase walls are provided for routing of potable and waste water (see Figure 1.4). To achieve 2-hr separation between units at chase walls, chases are surrounded by 2-hr rated wall assemblies. Hilti CP 606 flexible firestop sealant fills the joint between gypsum board and slab. A 2-hour rated application of spray-applied intumescent sealant at the wall header extends a minimum of 3" perpendicular from the wall along the ceiling. A bonding agent, such as Isolatek's Bond-Seal, secures the intumescent paint to the CLT slab to ensure proper functioning in a fire. This design provides thermal protection to the wood ceiling to avoid fire entering the wall cavity prematurely.
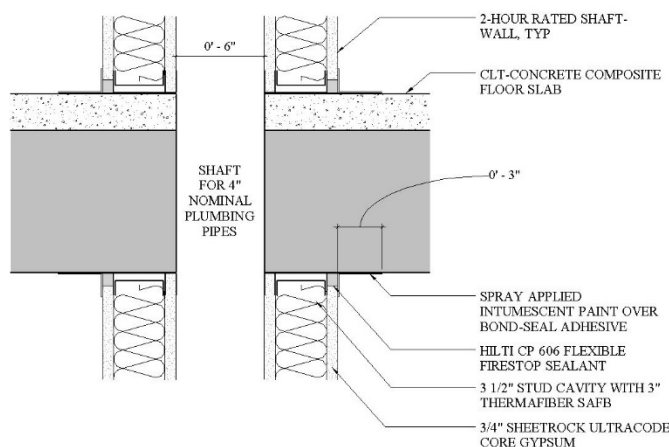

Figure 1.4 Fire-Rated Penetration through CLT Slab.

2. Structural Approach

The structural design approach for the prototype mass timber tower uses existing codes, standards, materials science, and current research to explore the possibility of a building type for which there are incomplete codes or standards. Some of the design elements used in the study, such as glulam columns, bolted connections, and cross laminated timber (CLT) floor panels, are codified in NDS 2015. However, CLT and concrete composite floor systems and wood cased / steel core buckling-restrained braces are not specifically included in any standards and the design of those elements has been interpolated from parallel standards such as AISC 360 and AISC 341.

Consideration of fire is a critical element in the design of mass timber buildings. The design approach to fire resistance utilizes a sacrificial layer (or char layer) on exposed mass timber elements, such as CLT floor framing and glulam columns. The approach considers two basic load cases: $1.2D + 1.6L$ under normal operating conditions, and $1.2D + 0.5L$ during a fire event in which the CLT panel or glulam column section has been reduced because fire has consumed the exterior fibers of the structural member. These load cases are developed in accordance with commentary section C2.5 of ASCE 7-10, Load Combinations for Extraordinary Events. Further fire design data are provided in Section 3.

2.1 Floor Design

2.1.1 Introduction to CLT

Cross laminated timber (CLT) is a relatively new building material developed in Austria and Germany in the early 1990s. It is made up of sawn lumber planks laid out in orthogonal directions and glued together with structural adhesives (Figure 2.1). This creates a structural element that behaves like a two-way slab where the longitudinal direction is the strong axis and the transverse direction is the weak axis. CLT panels are manufactured in sections as wide as 10 feet and as long as 60 feet.
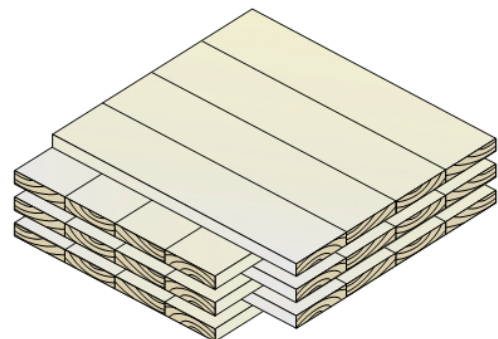

Figure 2.1 - Isometric View of CLT Panel Showing Alternating Directions of Planks (*CLT Handbook* 2013)

In 2011, "PRG 320 - Standard for Performance-Rated Cross laminated Timber" was published and afterwards adopted by the 2015 International Building Code. PRG 320 details test requirements and quality assurance standards for manufactured CLT panels, as well as provides design properties including: bending stress, compression stress, tension stress, shear stress, and modulus of elasticity for both the major and minor axes for several grades of CLT panels. For this study, type E2 is used because is made of Douglas Fir-Larch, which is selected because it is readily available in the southern California region, and in the future could provide a sustainably-managed wood source for local CLT production. The 2015 National Design Specification (NDS) for Wood Construction gives adjustment factors and design considerations to be used with the corresponding stress values given in PRG 320 for both Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD).

2.1.2 Floor Panel Layout and Design

The columns of the prototype mass timber tower are laid out in a rectangular grid pattern as shown in Figure 2.2. To reduce deflections between bays and around the perimeter, ten-foot-wide by 36-foot-long panels span between columns and cantilever from the columns in both the major and minor axes. The columns are each one story tall and stack on top of each other so that floor plates can be dropped into place after columns are erected. The floor acts as a composite system, using a 2.5" normal weight concrete topping securely connected to the CLT panels with fully-threaded screws in a uniform pattern in each direction. The total dead loads applied to the floor panels are 64 psf and 56 psf for seven-layer and five-layer CLT panels respectively. This includes the weight of the concrete and CLT panels as well as a 10psf superimposed dead load for flooring, ceiling, and MEP. The residential live load is 40 psf in addition to a 15 psf allowance for partitions.

The composite CLT floor is designed to meet stress and deflection criteria dictated by CBC 2015 and LABC 2014. Vibration analysis was also considered using AISC Design Guide 11 as a guideline. In order to check deflection and vibration criteria, the stiffness of the composite CLT panel is calculated. The out-of-plane stiffness of CLT panels is dependent not only on the bending stiffness (EI) of the section but also on the shear stiffness (GA) of the section. This is because CLT panels are anisotropic, and the cross laminations of the CLT panel tend to have significant rolling shear deformations (Figure 2.3). The amount of shear deformation contribution depends on the loading pattern, fixity at the ends of the panel, span-to-depth ratio, and span length
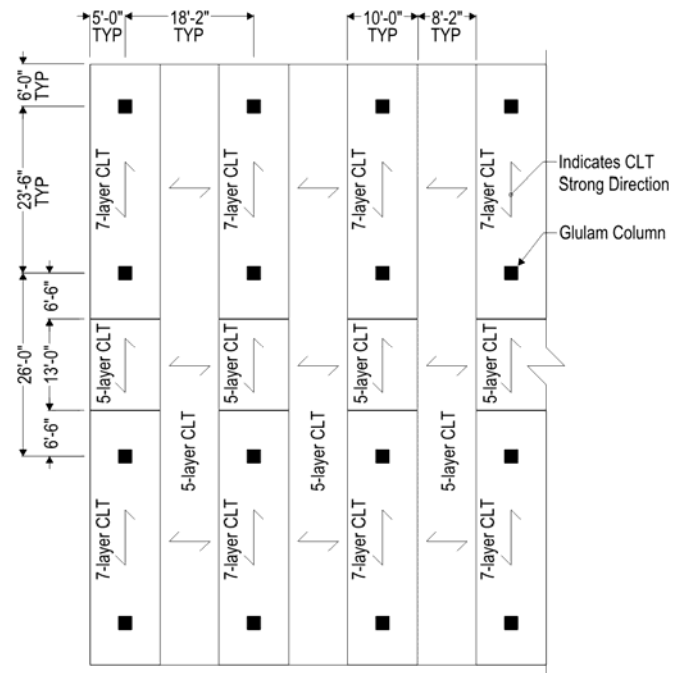
Figure 2.2 Partial Plan View of CLT Floor Panel Layout
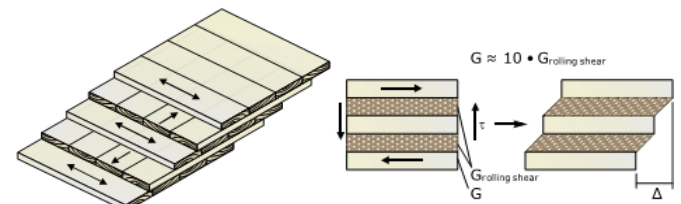
Figure 2.3 Rolling Shear Deformation of a five-layer CLT Panel (*CLT Handbook* 2013)

2.1.3 Effective Stiffness Properties of Composite CLT

In order to check serviceability and vibration requirements, PRG 320 gives stiffness properties for three-, five-, and seven-layer CLT panels of all grades. For CLT configurations outside of those given in PRG 320, stiffness properties may be calculated using the *CLT Handbook*. This study uses the "Shear Analogy" method (Kreuzinger, 1999) because it is well documented in the *CLT Handbook* with examples, is the most precise design method according to current literature, and is adopted by PRG 320.

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The *CLT Handbook* derives the following formulae for calculating the effective bending stiffness (EI_{eff}) and shear stiffness (GA_{eff}):

$$EI_{eff} = \sum_{i=1}^{n} E_i b_i \frac{E_i^3}{12} + \sum_{i=1}^{n} E_i A_i z_i^2$$

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i b}\right) + \left(\frac{h_1}{2G_1 b}\right)\right]}$$

These formulae are used to calculate EI_{eff} and GA_{eff} for a five-layer grade 'E2' CLT panel. The values calculated match the values listed in PRG 320 (see Figure 2.4 & 2.5). This same approach is used to calculate EI_{eff} and GA_{eff} for CLT panels with a 2.5" normal weight concrete for both the major and minor axis directions (Figure 2.6). The effective bending stiffness (EI_{eff}) of the composite five-layer CLT panel is over four times greater than the bare five-layer CLT panel, while the effective shear stiffness of the composite panel is about twice as stiff as the bare panel.
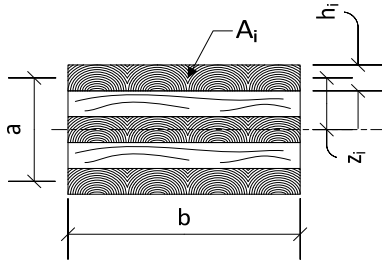

Figure 2.4 Cross Section of a five-layer CLT Panel

2.1.4 Computer Modeling of CLT Floor

SAFE 2014 is used to model the CLT floor. CLT is an anisotropic material and has different stiffness properties in each direction. Therefore, this study uses SAFE because the program allows for the creation of orthotropic slabs and the definition of a specific effective slab thickness in each orthogonal direction. The composite CLT effective bending stiffness (EI_{eff}) is captured by back-calculating the effective thickness of the slabs in each orthogonal direction based on the Modulus of Elasticity (E) defined in SAFE. In order to capture the deformations due to shear, shear property modifiers are calculated for each slab thickness in each orthogonal direction based on the calculated effective shear stiffness (GA_{eff}) and the effective slab thickness (t_{eff}), Shear Modulus (G), and Modulus of Elasticity (E) defined in the SAFE model.

2.1.5 Vibration of Composite CLT Floor

Each CLT floor panel is modeled as shown in Figure 2.2 as a separate element in SAFE with the edges of each element released in bending. The seven-layer panels are meshed at the columns so that the cantilevered ends can be assigned a different slab property (Figure 2.7). This is necessary because the cantilevered ends are subject to negative bending and could potentially crack the topping slab rendering it ineffective for stiffness contributions. Therefore, the cantilevered portions of the seven-layer CLT panels over the columns are assigned non-composite effective thicknesses and shear property modifiers in each direction. The edges between the non-

Layer	h (in)	Orientation to Span	Material	E (psi)	z (in)	$Ebh^3/12$ (lb-in²)	EAz^2 (lb-in²)	$Ebh^3/12+EAz^2$ (lb-in²)	G (psi)	h/(Gb) (in²/lb)
1	1.375	Parallel	Wood	1,500,000	2.75	3,899,414	187,171,875	191,071,289	93,750	0.00000122
2	1.375	Perpendicular	Wood	46,667	1.375	121,315	1,455,781	1,577,096	8,750	0.00001310
3	1.375	Parallel	Wood	1,500,000	0	3,899,414	0	3,899,414	93,750	0.00000122
4	1.375	Perpendicular	Wood	46,667	1.375	121,315	1,455,781	1,577,096	8,750	0.00001310
5	1.375	Parallel	Wood	1,500,000	2.75	3,899,414	187,171,875	191,071,289	93,750	0.00000122
t_{total} =	6.875					Effective Bending Stiffness EI_{eff} =		389,196,185 lb-in²		
						Effective Shear Stiffness GA_{eff} =		1,056,402 lb		

Figure 2.5 Calculated Effective Stiffness Properties for the Major Axis of a five-layer CLT Panel

Layer	h (in)	Orientation to Span	Material	E (psi)	z (in)	$Ebh^3/12$ (lb-in²)	EAz^2 (lb-in²)	Sum of Layer	G (psi)	h/(Gb) (in²/lb)
1	2.500	Parallel	Concrete	3,155,924	3.4375	49,311,316	1,118,750,491	1,168,061,808	1,314,968	0.00000016
2	1.375	Parallel	Wood	1,500,000	1.5	3,899,414	55,687,500	59,586,914	93,750	0.00000122
3	1.375	Perpendicular	Wood	46,667	0.125	121,315	12,031	133,346	8,750	0.00001310
4	1.375	Parallel	Wood	1,500,000	1.25	3,899,414	38,671,875	42,571,289	93,750	0.00000122
5	1.375	Perpendicular	Wood	46,667	2.625	121,315	5,305,781	5,427,096	8,750	0.00001310
6	1.375	Parallel	Wood	1,500,000	4	3,899,414	396,000,000	399,899,414	93,750	0.00000122
t_{total} =	9.375					Effective Bending Stiffness EI_{eff} =		1,675,679,867 lb-in²		
						Effective Shear Stiffness GA_{eff} =		1,886,307 lb		

Figure 2.6 Calculated Effective Stiffness Properties for Major Axis of five-layer CLT Panel with 2.5" NW Concrete Topping
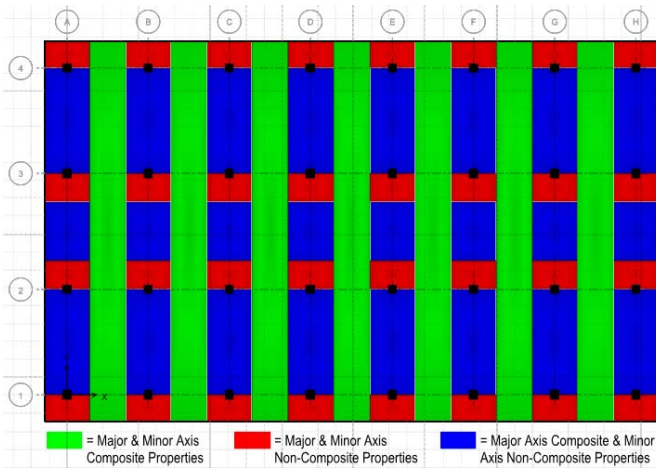
Figure 2.7 Safe Floor Mesh of Non-Composite and Composite CLT Panels

Figure 2.9 Peak Acceleration Ratio vs Floor Frequencies for the First Six Modes

composite and composite seven-layer panels are not released for bending. Modal analysis in SAFE is used to determine the natural frequency of the floor. Mode shapes for the first mode is shown in Figure 2.8. For the modal mass the self-weight of the CLT panel & topping slab, a realistic superimposed dead load of 5psf, and 20% of the design live load are used as recommend by AISC Design Guide #11. Since full height partition walls separate the units in this building, a damping ratio of 5% is used. A constant walking force of 80lbs (equivalent to a 150-pound person walking at a frequency of 2Hz) is assumed. Based on all of these parameters a peak acceleration ratios of less than 0.15% of g for all slab frequencies is obtained. As shown in Figure 2.9, these peak acceleration ratios are all less than the perceivable threshold for humans of 0.5%g in a residential building based on recommendations from the Applied Technology Council (ATC Design Guide 1).
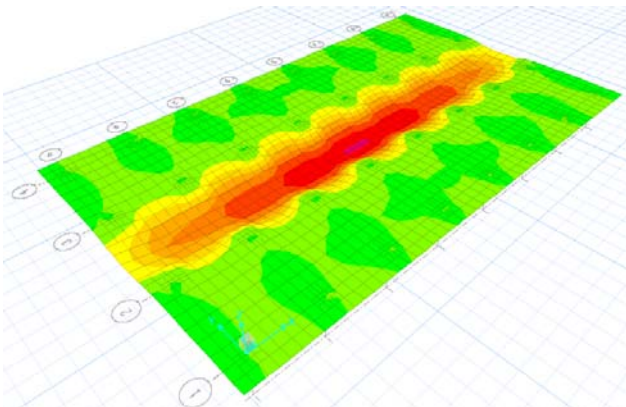
2.1.6 Deflection of Composite CLT Floor and Creep

To investigate the pre-composite and long-term deflection of the composite CLT floor this study used two separate SAFE models. One model has CLT panels modeled with non-composite effective thicknesses and shear property modifiers and the second model includes composite properties of the CLT panels experiencing positive bending, similar to the vibration model. This study takes the deflection reported in the first model due to the CLT and concrete topping self-weight and adds it to the deflection of the second model under long term service loads. The worst case initial dead load deflection from the non-composite model is 0.38" (Figure 2.10). The largest combined dead and live load deflection from the post-composite model is 0.18" for both cases. The total long term dead and live load deflection with a creep factor of 2 is 0.38" + (2 x 0.18") + 0.18" = 0.92". The allowable dead and live load deflection for this 26-foot span is 1.3'' (NDS 2015).

Figure 2.8 Mode 1 7.71Hz from SAFE Modal Analysis

Figure 2.10 Initial Dead load Deflection of Non-Composite CLT floor (in)

2.1.7 Flexure Strength of CLT Floor

The floor panels are designed for several loading conditions. The first condition checks that the bare CLT panels can support the wet we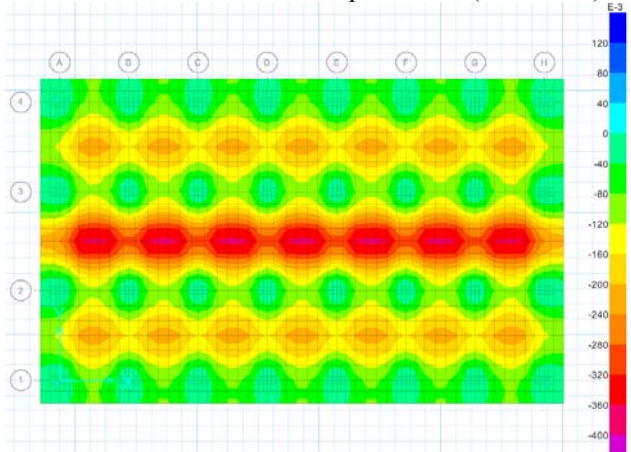ight of the 2.5" concrete topping as well as construction live loads. The worst-case ultimate bending moment in the seven-layer CLT panel is 73kip-ft/10ft panel width = 7.3kip-ft/ft. This is much less than the ultimate bending capacity of 33.7kip-ft/ft using the allowable capacities from PRG 320 table A2 and multiplying by the adjustment factors in NDS 2015. Similarly, results show that the bare CLT panels can resist the final design load case 1.2D + 1.6L. The controlling flexural design condition is the fire event, where CLT floor panels resist a load of 1.2D + 0.5L assuming two bottom laminations of the panels are consumed by fire and unusable. For example, all 7-ply CLT panels are reduced to the section properties of a 5-ply CLT panel. The ultimate major axis moments for this case are shown in Figure 2.11.
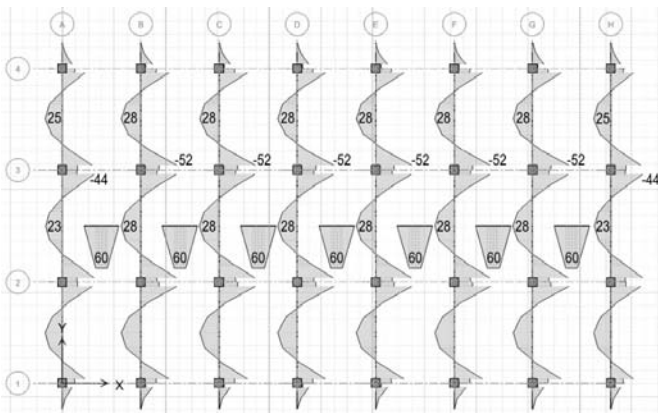
Figure 2.11 Ultimate Bending Moment Diagram for Fire Event (1.2D+0.5L) (kip-ft)

2.1.8 Punching Shear of CLT Floor

Similar to the flexural design of the CLT panels, punching shear design has two load cases (Figure 2.12). The final design load case is 1.2D + 1.6L. All the layers of the CLT panel can be used to resist punching shear, and the CLT panels have a full 6" of bearing on the glulam columns all the way around. The fire event load case is 1.2D + 0.5L. Only five layers of CLT can be used to resist punching shear because two have been burnt in the fire. There is only 1" of bearing all the way around the column because the first 4" of the column have been burnt through on all sides of the column.

2.1.9 Unbalanced Loading of CLT Panels at Column Connection

When the 7-layer CLT panels are installed on top of the glulam columns, there is a temporary unbalanced load on the 7-layer panel if a 5-layer panel is connected on one side only. In order to prevent the 7-layer panel from tipping over, diagonal full thread screws must be installed through the glulam column into the CLT panel as shown in Figure 2.13. Once the column above the CLT panel is in place, the unbalanced load is resisted by the column above. An unbalanced load conditions exists permanently at the exterior columns, and there is always the possibility of a live load unbalance at interior columns. The positive attachment of the CLT panel to the glulam column, as well as having the column above the floor installed, serves to resist this unbalanced load.

Figure 2.12 Final Design Condition (left) and Burn Condition (right)

7-Layer CLT Panel 9.625"

27" SQ Glu-Lam Column

Diagonal Fastener to Prevent CLT Panel from Tipping Before Upper Column is in Place

Figure 2.13 Temporary Fasteners to Prevent CLT Panel from Tipping Over During Construction

2.1.10 Non-frame Column Design

Non-frame columns at the base of the mass timber high-rise are designed for an ultimate (1.2D + 1.6L) and fire (1.2D + 0.5L) scenario. The ultimate load on a typical bottom story interior column is 1,181 kips which includes self-weight, superimposed dead load, reduced live load, and partition load. A 26" square, 13'-0" tall, 24F-1.8E glulam column has an ultimate capacity of 1,849 kips, resulting in a demand-capacity ratio of 64%. When subjected to fire loads, the axial load on the column is reduced to 873 kips and compared to the capacity of a column whose cross section has been reduced by the thickness of the char layer. As determined by the fire analysis outlined in Section 3, the column dimensions are reduced by 3.75" on each side, resulting in an 18.5" square cross section. The ultimate capacity of an 18.5" square glulam column is 1,148 kips, which yields a demand-capacity ratio of 76%.

Long-term loading is considered for the unusually tall timber elements. Column shortening due to an expected service axial load of D + 0.5L is 0.70" for a 26" square interior column over 20 stories. Long-term loading shortening is calculated as $\Delta_T = K_{cr}* \Delta_{LT} + \Delta_{ST}$ in accordance with NDS 2015 equation 3.5-1, where K_{cr} is 1.5 for glulams in dry service conditions, Δ_{LT} is the long term shortening, and Δ_{ST} is the immediate shortening. The resulting long term shortening is 1.76" over the height of the timber tower. Considerations will need to be made for the timber columns, such as fabrication to lengths longer than required in the final condition and coordination with exterior cladding to accommodate long-term column shortening between floor levels which approaches 3/16" at the bottom story.

2.2 Lateral Design

Two ETABS analysis models were used to determine lateral loads and building response for both the concrete and timber towers. The existing concrete tower model is based on existing structural drawings prepared by John A. Martin and Associates.

2.2.1 Seismic Loads

Seismic loads for the analysis models are determined using the modal response spectrum procedure of ASCE 7-10. The mapped acceleration parameters of the target site are $S_S = 2.254$ and $S_1 = 0.793$. The site coefficients, assuming a Soil Type D which is typical for downtown Los Angeles, are $F_A = 1.0$ and $F_V = 1.5$. The resulting design spectral acceleration parameters are $S_{DS} = 1.503$ and $S_{D1} = 0.793$, and because S_{D1} is greater than 0.750 the buildings are subject to the requirements of Seismic Design Category E.

Accelerations as a percentage of gravity are determined for force and drift per the modal response spectrum procedure. Because the timber tower has such a long period, the force level acceleration of .056 was controlled by the ASCE 7 minimum requirement of 85 percent of the calculated base shear. Similarly, the drift level acceleration of .043 was controlled by ASCE 7 minimum.

2.2.2 Wind Loads

Wind loads for the analysis models are determined using the directional procedure of ASCE 7-10 Chapter 27. The Basic Wind Speed for the site is V = 110 mph. A Gust Factor of 1.01 was determined to account for flexibility of the timber tower. In general, the design wind pressure on the wide face of the timber tower ranges between 45 psf and 66 psf, while the design wind pressures on the narrow face ranges between 40 psf and 60 psf.

2.2.3 Concrete Tower Model Assumptions

The ETABS model representing the existing concrete tower is developed using the existing structural drawings. Concrete moment frames on the four sides of the building are modeled with beam size, column size and concrete grade to match existing drawings. Post-tensioned concrete floor plates are represented by rigid diaphragms.

Concrete columns are cracked to 70% of the gross concrete section while beams are cracked to 50% of the gross concrete section, which is an interpolation of the Los Angeles Tall Buildings Structural Design Council recommendations for a seismic event between service and MCE levels (LATBSDC 2014). The size, shape, and material grade of the lateral force resisting elements of the concrete tower are modeled as they are shown in the existing structural drawings.

The existing building has a larger floor plate podium from the third floor down. As mentioned in Section 1 above, the
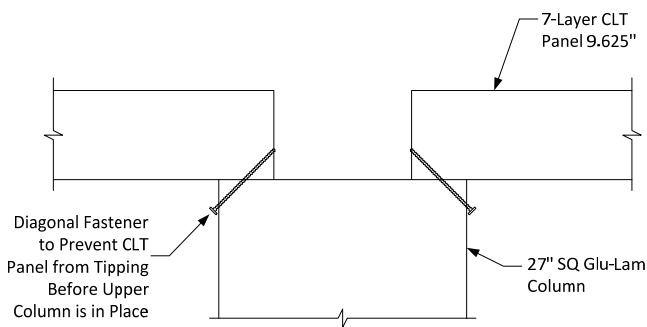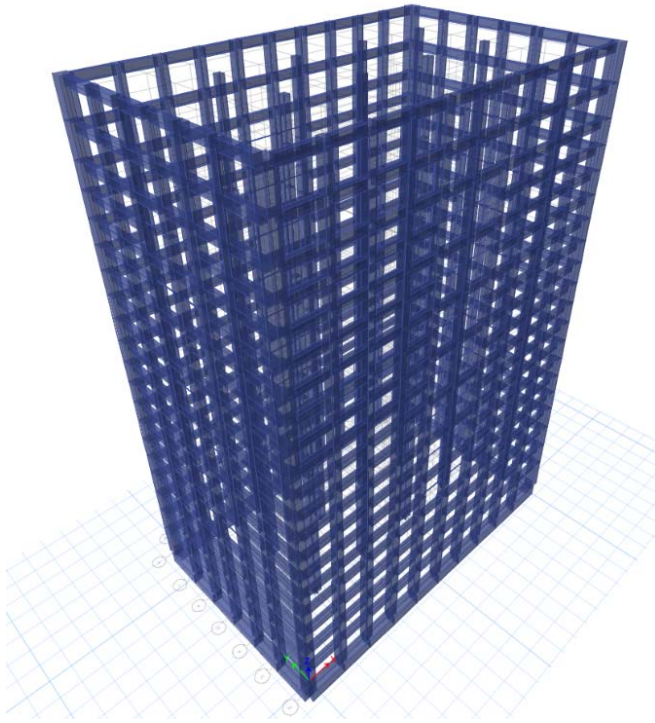
Figure 2.14 ETABS Model: Existing Concrete Tower

Figure 2.15 ETABS Model: Mass Timber Tower

podium is eliminated from the concrete model and the typical tower level is used for all floors. This simplification to the concrete model is made to ensure a more direct comparison to the timber model, which does not include a podium. Refer to Figure 2.14.

The concrete tower model generates periods and modes of the existing concrete tower as well as base shear and interstory drift. The base shear and drift values are used as a baseline for comparison to the results generated by the timber tower model. No strength analysis or design was performed on the modified existing building.

2.2.4 Timber Tower Model Assumptions

A timber tower analysis model is also developed using ETABS (Figure 2.15). Twelve braced frames – six in each primary building direction – support floor plates represented by rigid diaphragms. The braced frames employ 34" square glulam columns below Level 10 and 30" square columns above. Eighteen-inch square upturned frame beams connect the columns at the floor levels and support steel-core bucking restrained braced frames oriented in a chevron configuration. All glulam members are modeled based on the strength and stiffness properties of grade 24F-1.8E.

2.2.5 Timber Tower Buckling-restrained Brace Frames

Buckling-restrained brace frames (BRBFs) are chosen as the lateral force resisting system for a number of reasons. The method of connecting timber elements to timber elements with steel connection plates benefits from the relative simplicity of the connections where forces are transferred primarily through axial load. Buckling-restrained braces are a very ductile element with the ability to dissipate large amounts of energy, demonstrated by the assignment of R = 8 in ASCE 7-10. Testing of the performance of a wood sheathed buckling-restrained brace connected to glulam columns and beams is recommended to justify and R value of 8. The stiffness of buckling-restrained braces can be modified by shortening or elongating the steel core of the braces, which allows the response of the braces to be tuned in the model to control interstory drift. Unlike special concentric braced frames, buckling-restrained braces do not impose a large vertical unbalanced load at brace frame beams when oriented in a chevron or "V" configuration.

Braces are oriented in a chevron configuration to achieve a more optimal angle of inclination of the braces than a single brace per bay due to the relatively low floor-to-floor height and column spacing. Refer to Figure 2.16   This configuration also moves brace to column connections, which includes a large

steel plate, outside of CLT panels at column connections, allowing the CLT panels to maintain fixity over the column joints. Instead, the braces interrupt CLT panels at the middle of CLT panels parallel to the direction of primary moment which can be more easily accommodated.

Braced frame beams are upturned to allow direct connection to the composite slabs that vary in depth and that would require blocking for downturned beams (Figure 2.17). The decision to upturn the beams is reinforced by its consistency with the upturned moment frame beams of the existing concrete tower. In both cases, the upturned beams allow for unobstructed eye-level views of the outside at the perimeter glazing.

The steel core of the buckling-restrained braces is designed with the following material properties defined in AISC 341: $F_{y,min} = 42$ ksi, $F_{y,max} = 48$ ksi, $\omega = 1.5$, and $\omega\beta = 1.7$ where the steel yield stress would be determined by coupon testing. The size of the glulam sheathing used to restrain the steel core from

buckling is determined by sizing the brace sheathing such that the adjusted brace strength in compression, $Pu = \omega\beta F_{y,max}A$, does not exceed the Euler buckling load of the sheathing. For example, the adjusted compressive strength of a 5 square inch steel core is 408 kips, which is restrained by a sheathing size of 12" square with an Euler buckling load of 1,762 kips at 11'-0" long. A cross section of the buckling-restrained braced frame member is shown in Figure 2.18.

Alternate lateral force resisting systems were considered for the timber tower, such as CLT shear walls. Ultimately, it was determined there was not enough code support for this approach – notably, the lack of in-plane shear capacity values and an R value – to justify using CLT shear walls. The use of CLT shear wall in lieu of the braced frames has been identified as an area of future study.
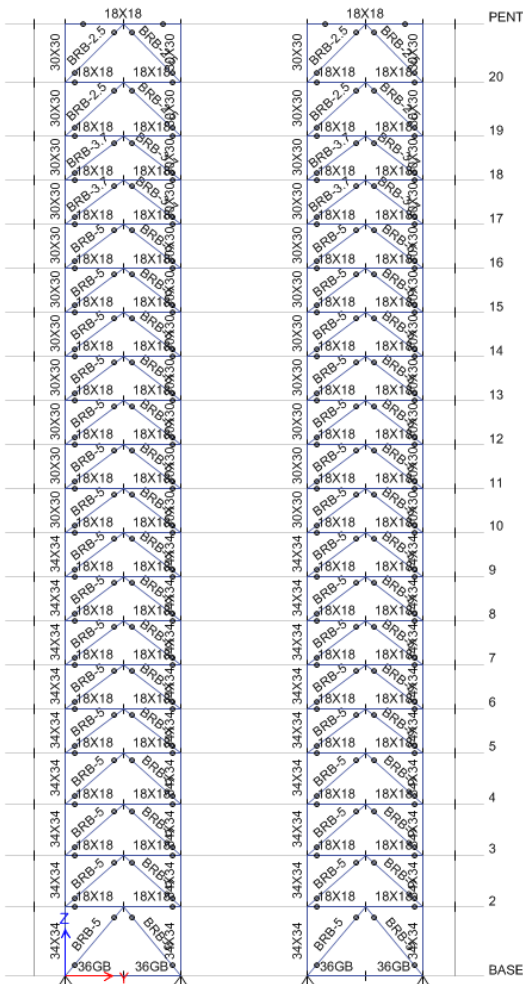

Figure 2.17 Brace-to-Column Connection at Upturned Beam
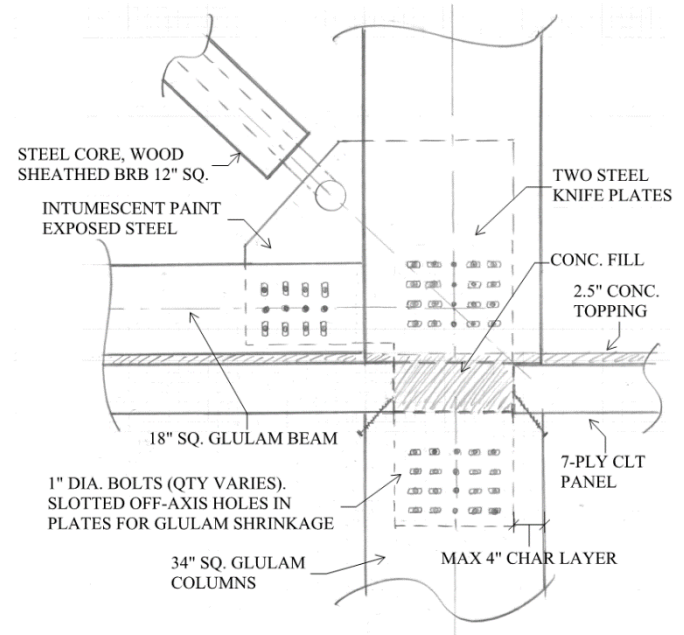

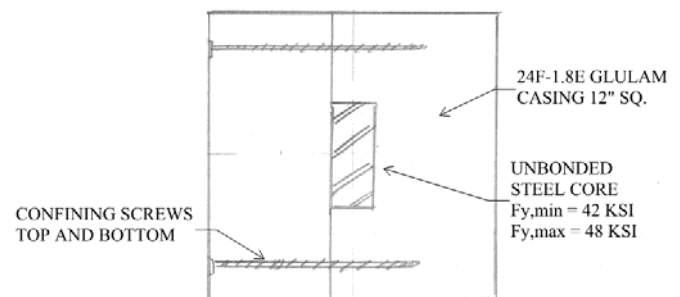**Figure 2.16 ETABS Model: Timber BRBFs**

Figure 2.18 Buckling-restrained Brace Section

12

2.2.6 Braced Frame Column Design

Braced frame columns are sized using foundation overturning as a limiting state in accordance with AISC 341 Section F4.3. The footing is sized to resist overturning using the load combination of 0.6D - 0.9E, where the 0.6 factor on the dead load equals $0.9 – 0.2S_{DS}$ and the 0.9 factor on earthquake loads represents a reduction in overturning effects in accordance with ASCE 7-10 Section 12.13.4. Using the resulting footing size, the braced frame overturning moment corresponding to the vertical load contributing load case 1.5D + 0.5L + E was calculated, where the 1.5 factor on the dead load equals $1.2 + 0.2S_{DS}$. A resulting overturning increase factor of 2.5 was determined, and the load case 1.5D + 0.5L + 2.5E was used to design the braced frame columns. Note the 2.5 increase factor is not the ASCE 7 omega factor, although the values are coincidentally the same. This design approach is taken to control the amount of conservatism included in the column design. Had the braced frame columns been designed assuming every connected brace in a 20 story tower was experiencing the expected compressive strength in accordance with AISC 341 Section F4.3, the required size of the glulam columns would be prohibitively large. For example, a typical frame column axial load due to unfactored earthquake forces is 857 kips. The axial load due to expected brace compression yielding would be on the order of 5,770 kips, or 6.7 times the forces determined by the code design earthquake. An overturning increase factor of 2.5 is a more appropriate over-strength factor given the unlikely event that all braces in a 20 story frame would be yielding at the same time. Further study including a non-linear time history analysis of the lateral structure is recommended to justify this assumption.

2.2.7 Braced Frame Beam Design

Braced frame glulam beams are designed for the expected braced forces defined in AISC 341. Since the upturned beams do not support floor gravity loads, the only appreciable forces they are subjected to are generated by the braces. The beams are loaded with a significant axial force, generated by the horizontal components of the expected strength two braces, and a moment due to the vertical component of the braces which, although they act in opposite directions, are not balanced because of the difference in tension and compression strength of the confined steel cores.

2.2.8 Lateral Performance and Tower Comparison

The controlling load case of the timber tower parallel to the short side was wind which had a base shear of 1,500 kips compared to the seismic base shear of 1,012 kips. In the long direction, seismic load, which was the same as the short direction, controlled the frame design over wind which had a base shear of only 800 kips.

Ultimately the timber tower lateral design was governed by drift. Figure 2.19 shows the tower story drifts by load case. The levels that most closely approach the drift limit of 2% (or 0.4% elastic drift) set by ASCE 7 are at the top of the tower. Accordingly, the brace size of 5 in^2 was used in all stories below. The axial demand at the most highly loaded brace at the bottom of the tower is only 80% of the brace axial capacity, and as expected, the demand-capacity ratio only decreases at higher levels.
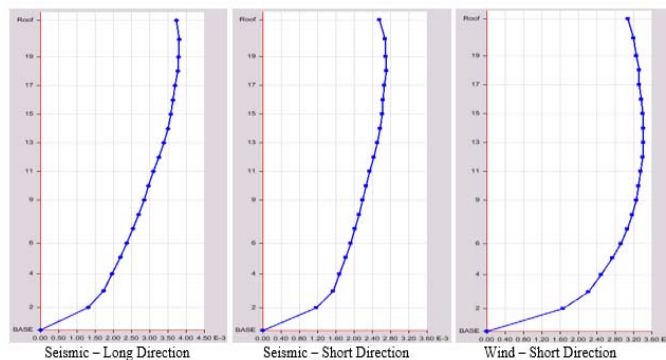
Figure 2.19 Story Drift by Load Case

Compared to the existing concrete tower, the timber tower has half of the mass and is roughly twice as flexible. The larger mass of the concrete tower and its relatively greater stiffness results in seismic loads of 2,377 kips which govern over wind load of 1,500 kips (short direction) as is to be expected of buildings this size in Los Angeles.

3. Fire Design

Fire design provisions for mass timber buildings beyond the prescriptive allowances of Type IV-HT Construction, are yet to be adequately codified or provided with appropriate guidance to allow authorities having jurisdiction to approve concepts that deviate significantly from the regular code.

The following section outlines both the prescriptive code provisions and an alternative solution concept utilizing performance based design methodology that could be extended upon to allow approval of a high-rise mass timber building.

3.1 Prescriptive Code Provisions

The model building is located in the City of Los Angeles, and therefore is subject to the 2014 City of Los Angeles Building Code, being based upon the 2013 California Building Code (2013 CBC), with local amendments.

The timber tower is residential occupancy (R-2), is 20-stories and in excess of 180-ft high and is therefore required to meet

Type I-A construction provisions (CBC Table 503), which permits unlimited building area and height. Note that the subject building is 194'-10 ¾" high, with typical floor plate of 11,926.5 sqft.

Per CBC 602.2 primary and secondary building elements for Type I construction are required to be non-combustible. In order to allow combustible construction it must be demonstrated that the material and method of construction is at least equivalent to the prescriptive requirements with respect to quality, strength, effectiveness, fire resistance, durability and safety, in accordance with CBC 104.11 - *Alternative materials, design and methods of construction and equipment.*

With the highest occupied floor in excess of 75-ft above the lowest level of Fire Department vehicle access, the building is subject to the high-rise building provisions of CBC 403, summarized as follows:

- Automatic sprinkler system throughout with secondary on-site water supply.
- Fire pump(s) shall be supplied by connections to no fewer than two water mains located on different streets.
- Area and duct automatic smoke detection, Fire Department communication (emergency responder radio coverage) system and emergency voice/alarm system.
- Class I standpipe system.
- Fire command center.
- Smoke control system and standby power.
- Emergency power system.
- Stairway door operation and communication system.
- Smoke proof exit enclosures including access vestibule at each level.
- Two Fire Service access elevators.
- Luminous egress path marking.

Additional general provisions:

- Per CBC 1007, two accessible means of egress are required per floor. Areas of refuge are not required where the building is sprinklered throughout, or an R-2 occupancy.
- Manual fire alarm system and smoke alarms in accordance with CBC 907.2.11.2.
- Systems such as exit signage, emergency lighting, etc., would meet minimum code provisions.

3.1.1 Fire-Resistance Ratings

Per CBC 601 and 403.2.1.1 the following fire-resistance-ratings are required to the applicable structural elements:

Building Element	Type I-A Construction
Primary structure	3-hour (1 hour for structure supporting roof only)
Non-bearing exterior walls	0 hour (fire separation distance > 30* ft.)
	1 hour (fire separation distance < 30* ft.)
Non-bearing interior walls	0 hour (1-hr for unit separations)
Floors	2 hour
Roofs	1 hour

Fire-resistance ratings for interior enclosures are summarized as follows:

Interior Enclosure Type	Fire-Resistance Rating	CBC Section
Exit Stairway	2 hours	1022.2
Mechanical Shaft	2 hours	713.4
Elevator Shaft	2 hours	713.4

Fire-resistance ratings are prescriptively achieved through fire testing in accordance with CBC Section 703.2. CLT and glulam fire test data and research literature forms a significant component in demonstrating the fire resisting performance of the mass timber building in accordance with CBC Section 703.3 - *Alternative methods for determining fire resistance.*

3.1.2 Combustibility

Limited combustible materials are permitted within Type I-A construction in accordance with CBC 603. The building primary structural system is to feature glulam columns and CLT composite floors, with glulam beams and braces for lateral load support. These elements are not permitted exceptions of CBC 603, and therefore an alternative materials, design and methods of construction and equipment is required in accordance with CBC 104.11, to demonstrate equivalence to prescriptive code. Currently fire-retardant treatment of glulam elements and CLT panels has not undergone sufficient testing nor listing by treatment manufacturers.

Exit enclosures, common corridors, stair and elevator vestibules are lightweight non-combustible construction with any glulam or CLT wood elements encapsulated with non-combustible construction (i.e gypsum board).

Smoke (soot) and carbon monoxide (CO) are the primary hazards to occupants and fire fighters. High smoke production rates, or high levels of soot, can cause respiratory distress, limited visibility and impact safe egress. It can also cause disorientation of fire fighters in confined building spaces. For
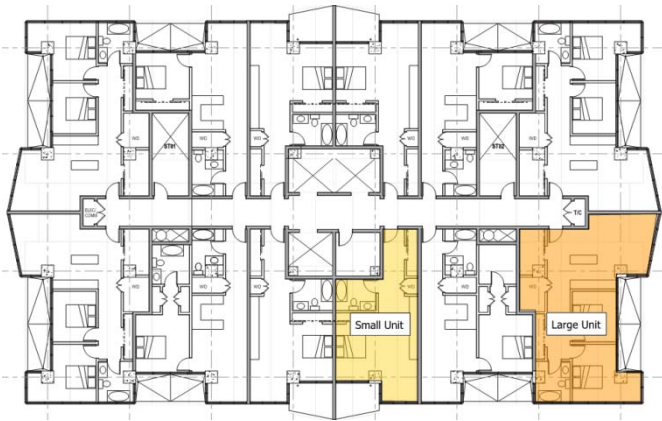
under-ventilated conditions, where the fire is expected to be producing combustion products at its peak release rate, wood exhibits a yield of 0.015 kg of soot per kg of material burnt, whereas under the same conditions, PVC-Nylon exhibits a yield of 0.115 kg of soot per kg of material burnt (Tewarson, 2008). Many polyurethane foams exhibit a soot yield of 0.130 kg/kg or higher. This suggests that exposed wood, if burning, would contribute approximately $1/10^{th}$ the soot of many common synthetic materials used in furniture in residential buildings.

3.1.3 Interior Surface Finishes

Interior finish classifications are specified in CBC Table 803.9, which requires interior wall and ceiling finishes for all spaces within a sprinkler protected R-2 occupancy to achieve Class C flame spread and smoke development benchmarks. Surface classifications are defined in CBC Section 803.1.1, with applicable criteria reproduced below.

Class C: Flame Spread Index   76-200
        Smoke Developed Index   0-450
The CLT ceiling and glulam columns are exposed and meet Class C material, per American Wood Council (2010).

3.2 Fire Engineering Assessment

The following assessment documents the proposed use of CLT floors and glulam columns as an alternative material and method of construction in relation to the applicable aspects of the fire performance characteristics. The assessment proposed is not a complete analysis; however, it outlines an approach that can be built upon to develop an approvable solution to high-rise wood construction.

3.2.1   Fire-Resistance Ratings (Floors)

Limited fire testing of CLT structural elements has been undertaken within the United States. The following table (Figure 3.1) summarizes current assemblies that have been tested to the ASTM E119 time temperature curve, with assigned fire performance levels.

Of note are the results of Tests #3 and #5, being unprotected 5 and 7 ply CLT floor systems, achieving 96-min and 178-min fire-resistance, respectively. The subject building utilizes similar CLT floor elements; however, in this case the wood system is composite with the concrete topping. Thermal-finite element modeling of the composite floor system to predict fire-resistance of the hybrid system is a study beyond this paper.

	Test #	# of Plies	Thickness (mm)	Type X Gypsum Board Protection	Load	Load Ratio	Failure Mode	Fire Resistance (min)
WALLS	2	3	114	2 x 12.7 mm (½")	333 kN/m	76% P_r	Structural	106
	4	5	175	Unprotected	333 kN/m	37% P_r	Structural	113
	8	5	105	Unprotected	72 kN/m	34% P_r	Structural	57
FLOORS	1	3	114	2 x 12.7 mm (½")	2.70 kPa	36% M_r	No failure	77 *
	3	5	175	Unprotected	11.75 kPa	59% M_r	Integrity	96
	5	3	105	1 x 15.9 mm (⅝")	2.40 kPa	75% M_r	Integrity	86
	6	5	175	1 x 15.9 mm (⅝")	8.10 kPa	101% M_r	Integrity	124
	7	7	245	Unprotected	14.58 kPa	101% M_r	Structural	178

* The first test conducted was stopped due to laboratory equipment safety concerns. Failure was not reached.

Figure 3.1 CLT Fire Test Results, NRC-CNRC Preliminary CLT Fire Resistance Testing Report, Osbourne, etc., 2012

However, based upon the sacrificial char methodology discussed in Section 2, the following outlines the expected fire exposure to the structural floor system.

3.2.2 Time Equivalent Fire Severity

The glulam column fire resistance is assessed based upon 3-hours exposure to the ASTM E119 standard time temperature curve, for the purpose of the primary structural system being resilient and avoiding collapse, well beyond a 'realistic' fire exposure. Fire exposure to the CLT floor system, is based on the same fire curve, with the exception of the exposure duration being a function of the expected fuel load (FLED), compartment geometry and available ventilation.

The time equivalent fire severity value is calculated from the methodology of Eurocode 1 - *Actions on structures exposed to fire*. The following input parameters are used.

- FLED 400 MJ/m^2 for residential units, can be compared to US data where the average value is 320 MJ/m^2 (Parkinson, Kodur & Sullivan, 2009)
- 50% of vertical opening area (windows) assumed open in a fire event. Large unit vertical opening area: 330 sqft (30.7 m^2); Small unit vertical opening area: 86.0 sqft (7.99 m^2).
- Conversion factor kb = 0.065 for normal weight concrete ceiling and floors.
- Firecell height: 9'- 0 1/8" (2.74 m); Large unit firecell floor area: 888 sqft (82.5 m^2); Small unit firecell floor area: 528 sqft (49.1 m^2)
- Sprinkler operation is not accounted for in the assessment.

The above input parameters result in a time equivalence value of 22 mins for the larger unit and 30 mins for the smaller unit.

Figure 3.2 Floor Plan Identifying Unit Designation for Fire Severity.

Charring of the timber ceiling and columns is calculated with the above time equivalence values providing the contribution of the wood structure to the fire:

-   Wood: Douglas Fir (SFPE Handbook); Density 500 kg/m^3; Calorific value: 20 MJ/kg.
-   Char rate: 0.60 mm/min (SFPE Handbook)
-   Larger unit exposed timber (ceiling and columns): 1178 sqft (109.5 m^2); Smaller unit exposed timber (ceiling and columns): 594.5 sqft (55.23 m^2).

This results in a modified FLED for each of the units due to the additional wood fuel load. The modified FLED for the large unit is 575 MJ/m^2 and 603 MJ/m^2 for the small unit. This is an increase of 44 % and 51 % for the larger and smaller units respectively.

Time to burnout is then re-calculated, but with a conservative FLED of 800 MJ/m^2 based on the above results. The time equivalence values therefore double to 44 mins for the larger unit and 60 mins for the smaller unit. With these burnout times the total charring of the timber is calculated as 27.0 mm for the larger unit and 36.0 mm for the smaller unit. Therefore based upon the expected fuel load within a unit and contribution of the combustible construction, the CLT floor could lose up to 2 plys, when exposed to fire. The structural assessment confirms that the fire limit state loading case can be supported by the CLT composite floor, where the two bottom plys are assumed to no longer contribute to capacity.

It is noted that the fire severity calculation does not account for automatic sprinkler or manual fire-fighting intervention. It also assumes that the fire exposure to the wood structure continues with the same severity for at least 30 mins, after the unit fuel load is consumed.

3.2.3 Building System Performance

An NFPA Study (Hall, 2013) analyzed U.S. fire data from 2007-2011 and reported that wet pipe sprinklers operated 92% of the time when a fire was large enough for activation. When activated, they were effective in extinguishing or controlling the fire 96% of the time. Therefore, it is reasonably expected that fires will be controlled to the compartment of origin within the subject building. To further improve the reliability of the sprinkler system and its effectiveness to control and suppress a fire, the sprinkler system could be upgraded from Light Hazard (LH) to Ordinary Hazard Class 1 (OH-1) in accordance with NFPA 13. This increases the sprinkler reliability, by reducing the potential failures of insufficient water discharge, water not reaching the seat of fire, or the fire growth overwhelming the sprinkler system.

Partition walls separating residential units and common corridors are to be non-combustible 2-hr fire-resistance assemblies with 90-min doors, upgraded from minimum code of 1-hr fire separation and 45-min doors. Therefore, providing increased compartmentation of fire and smoke.

Unit plumbing and electrical services are to be run through shaft walls and furred spaces in non-combustible conduit, with rated outlet boxes. Vertical services are to be contained in listed 2-HR shaft enclosures. Penetrations into units shall be through gypsum wall assemblies with listed fire-rated penetration systems.

The unit smoke alarm system could be upgraded to a full automatic smoke detection system utilizing a very early detection system (i.e. aspirating smoke detection with multi-level sensitivity), to minimize nuisance alarms within the unit and facilitate early notification of building occupants and Fire Department.

3.3 Fire Modeling Results (Glulam and CLT)

In order to obtain a better understanding of the thermal performance of the structure under fire conditions, a critical section of the structure is analyzed using the finite element software SAFIR (Franssen, 2011).

3.3.1 Numerical Analysis

SAFIR is a general purpose finite element program developed at the University of Liege, Belgium, for analyzing the behavior of structures under ambient or elevated temperatures. It consists of an integrated thermal and structural analysis program for carrying out 2D and 3D analyses of steel, concrete, timber and composite structures in fire conditions. A variety of finite elements may be utilized such as beam, truss,

solid and shell elements, for modelling a variety of engineering problems. The thermal and mechanical properties of steel, concrete and timber follow guidance from the Eurocodes, however alternate properties may be specified by the user.

The section of the structure analyzed is a 26 inch square timber column at a generic part of the floor plate, on a typical floor. The impact of a fire is assessed for the column and the underside of the CLT floor slabs it supports. The thermal analysis is run for 3 hours, in accordance with the requirements of Type 1A construction, on the slab-column joint interface of the 26 inch timber column. The model considers the protection of the steel bolts via 3 inch wood plugs. A schematic of the assembly studied and components of the model is shown in Figures 3.3 and 3.4.

The thermal impact of the ASTM E119 fire is considered on the underside of the CLT timber slab and on the sides of the timber column on the lower floor. An ambient temperature free convective surface condition is assumed on the opposing concrete slab surface and internal column surface on the upper
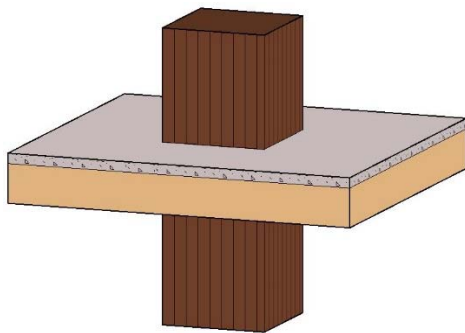
floor. The cross section below is analyzed, with a number of conservative assumptions made to simplify the analysis:
- The steel bolts/dowels are assumed to be 1 inch thick, continuing directly through the member.
- The steel plate is assumed to be 1 inch thick, aligned with the centerline of the column.
- Standard Eurocode properties are assumed for the heat transfer analysis (softwood, mild steel and siliceous concrete).
- As a redundancy measure, the impact of sprinkler systems or any other active fire protection measures are not included.

3.3.2 Steel Temperatures

The thermal contour of the beam-column joint cross section is shown at 3 hours fire exposure in Figure 3.5. The temperature profile of Section A-A' of the beam-column joint is depicted in Figure 3.6.

The thermal contour shows the temperatures predicted by the finite element model throughout the section after 3 hours fire exposure. As wood is an excellent thermal insulator whilst steel is a good thermal conductor, heat from the fire at the edge of the steel bolts is transferred directly into the core of the column and dissipated to the surrounding wood in contact with the steel. Thus at the exposed edge the steel remains cooler than the nearby wood, which is more locally heat affected.
The temperature profile through the steel plate, bolt and outer wood plug is shown in Figure 3.6 for Section A-A'. This shows that although the wood has completely burned away, overall the steel maintains relatively low temperatures. The steel plate within the center of the column is on average 36°C after 3 hours fire exposure, while the steel bolt at the outer edge of the column is approximately 61°C. This is due primarily to the
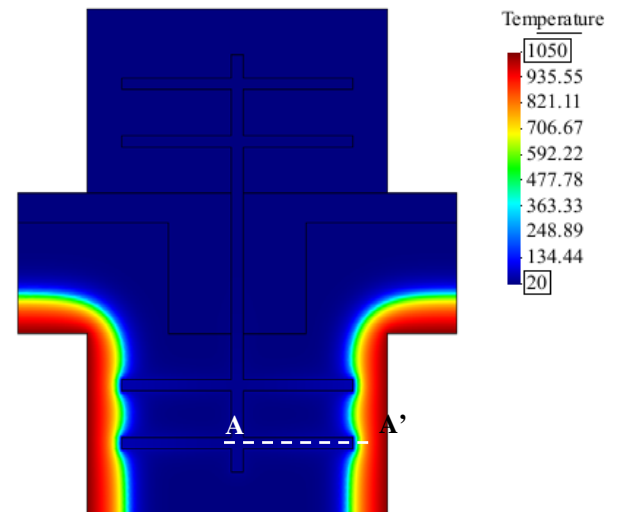

Figure 3.3 Assembly Selected for Fire Analysis

Figure 3.4   Square Column to Slab Connection

Figure 3.5   Thermal Contour of Slab-Column Joint at 3 Hours – Centerline of Column Identified

protection of the steel from the fire via 3 inch wood plugs at the column surface, and the dissipation of heat throughout the interior of the column by the steel itself.
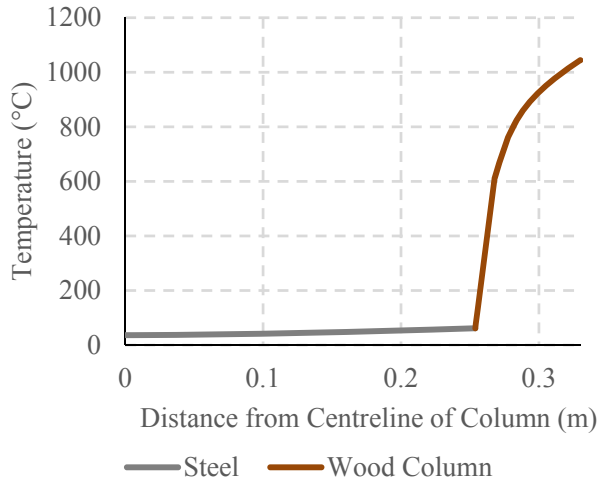

Figure 3.6 Temperature Profile of Section A-A' of the Beam-Column Joint at 3 Hours

3.3.3 Timber Char Damage

A contour limit of 300°C (572°F) is applied to the beam-column joint section, which is a generally accepted upper limit temperature for wood to turn to char. This gives a profile of the remaining wood section, with the char layer represented in black, as shown in Figures 3.7 and 3.8.
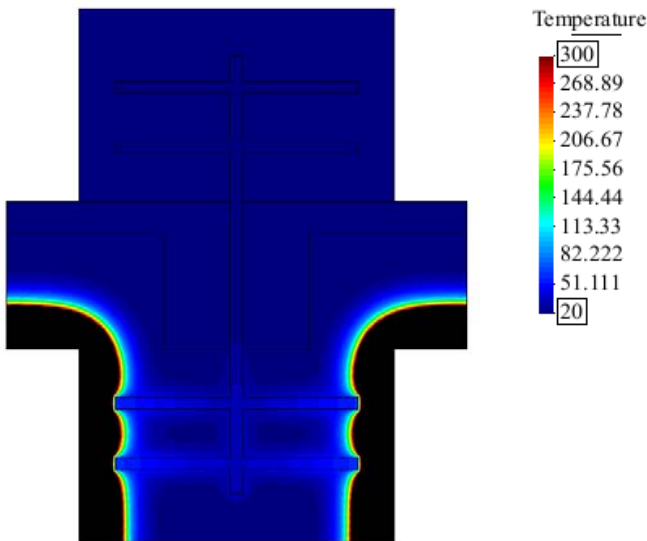
Figure 3.7 Char Profile of the Beam-Column Joint at 3 Hours

Plan views of the heat affected region of column are shown through the centerline of the steel bolts, and through a clear region of the wood column far from the steel in Figure 3.8.
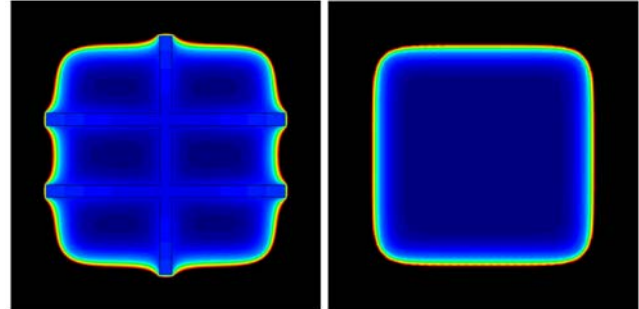

Figure 3.8 Char Profile of Plan Section of the Column at 3 Hours

Approximately 50% of the timber column cross sectional area remains after the fire, with the majority of this being able to fully carry building loads. The char layer is assumed to carry no mechanical load.

3.3.4 Discussion

The overall impact of the 3 hour fire exposure is that the loadbearing capacity and fixity of the steel bolts and plate are not significantly affected, as the steel can be considered to retain full strength (normally stiffness and strength begins to significantly degrade beyond approximately 400°C for hot rolled and cold worked reinforcing steel). A significant residual section of viable wood remains, such that building loads are still able to be carried by the columns. Thus the expected performance of the unprotected timber columns is that they will survive a 3 hour fire event and not suffer catastrophic collapse for standard building designs and loads.

4. Conclusions

New knowledge in the realm of wood building design is growing rapidly. Design options are expanding and ongoing national and international testing of mass timber assemblies is paving the way for tall wood buildings to meet code requirements, even for Type 1A construction. Although no modern high-rise wood buildings have been completed to date in the United States, this study demonstrates that a performance based, code-compliant high-rise wood building is possible, even in a challenging environment such as Los Angeles, California.

The wood tower demonstrates competitive performance with the reinforced concrete tower in the following areas: vibration, deflection, lateral force resistance, thermal performance, acoustic performance and compliance with modern code

requirements. With appropriate testing as noted in the report, it is expected that the mass timber design proposed can meet all code requirements for a Type 1A building.

The fire analysis demonstrates that the wood tower can withstand the 'realistic' fire event (based on fuel load, compartment geometry and available ventilation), without compromising the structure. It is noted that after a fire event, all charred wood can be removed and replaced with new wood that will carry load and protect the remaining wood in future fire events.

The following list is a summary comparison between the wood tower and reinforced concrete tower:

- Column sizes (at lowest level): wood = 26x26 non-frame, 34x34 frame; concrete = 40x40 non-frame, 32x40 frame
- Number of columns: wood = 32 ; concrete = 40 columns
- Lateral system: wood = brbf; concrete = ductile moment frame
- Floor thickness: wood = 12 1/8"; concrete = 8"
- Height: wood =   ; concrete = 190 feet
- Max Interstory drift: wood = 1.9%; concrete = 0.6%
- Base shear: wood = 1,012 kips; concrete = 2,377 kips
- Governs floor design: wood = fire load case; concrete = service load case
- Governs lateral design: wood = deflection; concrete = assumed strength given the low maximum interstory drift
- Building height: wood = 194-10 ¾"; concrete = 190'-1"
- Building area: wood = 11,926.5sqft; concrete = 11,926.5sqft

The widespread use of wood as the main structural elements in tall buildings is inhibited as wood is a combustible material and commonly thought to behave poorly in fires. Due to the nature of the non-combustible materials of steel and concrete, their performance in fires can be relatively simple to predict and design for. When considering wood elements, the loss of cross-section due to charring can complicate what has traditionally been the prescriptive exercise of evaluating fire resistance for materials such as steel and concrete.

Despite this, each material provides its own unique challenges with regards to fire performance. For example, thermal expansion and the protection of steel reinforcement can raise major issues for concrete structures. Similarly, steel structures can suffer catastrophic collapse under the effects of both thermal expansion and contraction. Massive timber structures have an inherent resistance to fire due to the properties of the

wood and the development of a char layer, such that unprotected wood members may still carry building loads for extended durations of fire exposure.

In comparison to a similar steel structure, all exposed steel loadbearing members critical to the performance of the structure are likely to require extensive applied passive protection to avoid exhibiting excessive deflections and premature collapse in a fire. In comparison to a concrete structure, a wood structure can achieve equivalent code requirements for fire resistance while providing additional benefits in terms of erection time, reduced overall building weight and improved sustainability.

4.1 Necessary testing and research

Physical lab testing is needed in the following areas to validate the results of this study:
-Composite interaction verification of 2.5" topping over CLT slab.
-Buckling-restrained brace needs testing to validate expected performance and develop backbone curve.
-A thermodynamic analysis of the building shell is recommended to study the performance of exposed wood buildings with window walls relative to exposed concrete buildings with window walls.
-Testing of the acoustic performance of floor assemblies.

Further investigations into the fire modeling effort are recommended to develop a better understanding of the overall building performance in a fire:
- Refinements to incorporate the effects of char layer ablation and an appropriate design fire for the space.
- Future thermal modeling to investigate the performance of the CLT floor slabs, glulam members and connection systems throughout the building. Structural modeling of these elements and assemblies to determine their displacement and limit state behaviors.

Acknowledgments

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