# Low-Rise Commercial Mass Timber Design Example

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## Ackonowledgements

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# 2 Design Overview

The design of a mass timber building has many factors including, design requirements provided in the CSA standards and National Building Code for strength and serviceability requirements, Constructability requirements, and long-term service condition impacts, such as shrinkage. The intent of this design example is to provide an overview of key elements of a mass timber building for both the gravity system and the lateral system. A prototypical 2 storey mass timber office building is proposed with CLT floors supported on glulam purlins supported on glulam girders and columns. The lateral system is composed of CLT shearwalls. The concept building is designed to be located in Ottawa, Ontario.

## 2.1 Codes and Reference Documents

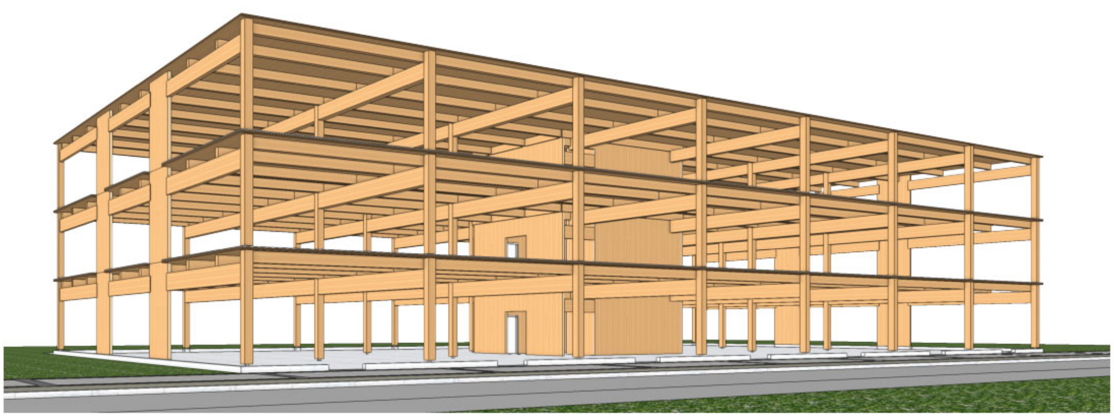
The loads for design of the building are taken from the National Building Code of Canada (NBCC 2015). The strength and serviceability design has been completed based on the following standards and reference documents

- Wood Design Standard CAN/CSA O86-19
- ANSI/APA PRG 320-19
- FP Innovations CLT Handbook

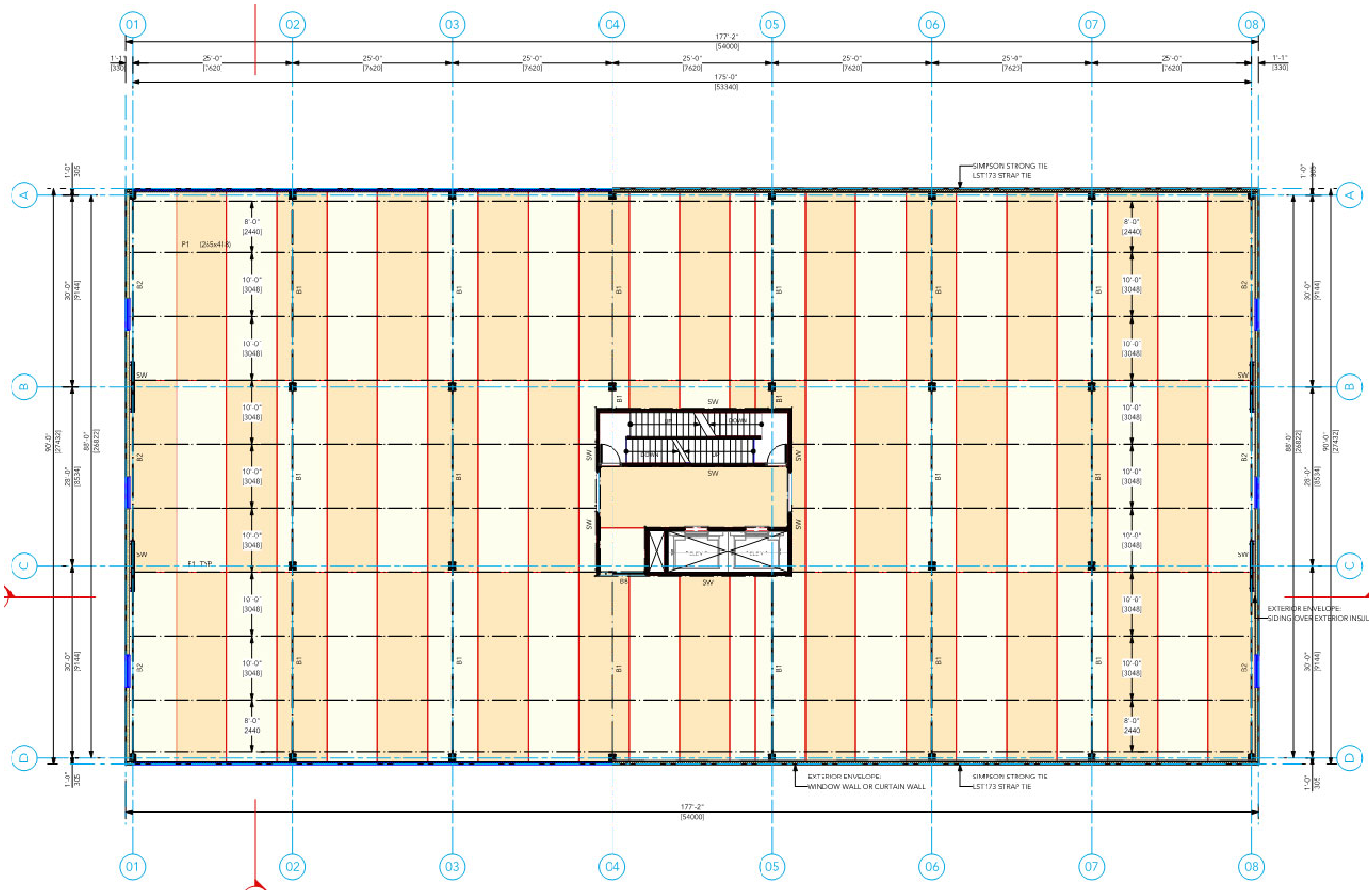
## 2.2 Design Considerations

### 2.2.1 BUILDING LAYOUT

The prototypical building layout represents a 3-storey office building with a central core and primarily open faces of the building to allow for glazing. The frame consists of CLT floors panels on glulam purlins, supported on glulam beams and glulam columns. CLT shearwalls are used for both gravity and lateral support at the center and ends of the building.



A grid spacing of 7.6m x 9.1m has been chosen; this bay size was chosen to maximize open floor plans, while working with common CLT panel widths (2.4m). The other direction has been chosen to optimize timber volumes; purlin spacing (10ft/3.05m) has been chosen to allow for a thin, efficient 3 ply floor/roof to minimize wood volume. Panel sizes vary by supplier and should be considered when choosing a grid layout. Optimizing the use of CLT is critical to an efficient design. Alternate systems without purlins would require either significantly thicker CLT panels, or narrower grid spacing which often results in higher overall wood volumes, but do provide a different, and sometimes preferred, architectural expression.


SIMPSON STRONG TIE
LST173 STRAP TIE
EXTERIOR ENVELOPE:
SIDING OVER EXTERIOR INSUL
EXTERIOR ENVELOPE:
WINDOW WALL OR CURTAIN WALL
SIMPSON STRONG TIE
LST173 STRAP TIE
177'-2"
[54000]
175'-0"
[53340]
25'-0"
[7620]
P1 (265x418)
P1 TYP
SW
B1
B2
ELEV
DOWN

### 2.2.2 CODE REFERENCE VALUES

The prototype is classified as normal importance and based on a location in Ottawa (City Hall), Ontario from the NBCC Appendix C.

#### 2.2.2.1 Wind

$q_{1/50} = 0.41\ kPa$

#### 2.2.2.2 Snow

$S_s = 2.4\ kPa$
$S_r = 0.4\ kPa$

#### 2.2.2.3 Seismic

| | | | | | | |
|---|---|---|---|---|---|---|
| Sa(0.2) | Sa(0.5) | Sa(1.0) | Sa(2.0) | Sa(5.0) | Sa(10.0) | PGA |
| 0.439 | 0.237 | 0.118 | 0.056 | 0.015 | 0.0055 | 0.281 |

### 2.2.3 LOADING

The calculation for the loading is not expanded on in detail but should be completed per the NBCC for all gravity loads including Dead, Snow, Wind, and Live Loads.

#### 2.2.3.1 Dead

The superimposed dead loads are assembled from the structural and architectural build-up o both the floors and the roofs. Partitions are included as per the requirements of the NBCC.

| **Floor Dead Loads (note: consider additional loads for finishes and other dead loads would be required for** | |
|---|---|
| 38mm concrete topping | 0.92 kPa |
| Mechanical and Electrical | 0.25 kPa |
| Carpet | |
| Partitions | 1.00 kPa |
| CLT | 0.43 kPa |
| Framing | 0.25 kPa |
| **Total** | **2.92 kPa** |

| **Roof Dead Loads** | |
|---|---|
| Roofing | 0.50 kPa |
| Mechanical and Electrical | 0.25 kPa |
| CLT | 0.50 kPa |
| Framing | 0.25 kPa |
| **Total** | **1.50 kPa** |

#### 2.2.3.2 Snow

The snow loads are calculated based on the NBCC 2015 section 4.1.6.

$S = I_s[S_s(C_b C_w C_s C_a + S_r] = 1.0[2.4\ kPa\ (0.8)(1.0)(1.0)(1.0) + 0.4KPa = 2.3\ KPa$

#### 2.2.3.3 Live

Live loads are determined based on the office occupancy of the buildings as per NBCC section 4.1.5.

$L = 2.4\ kPa$

### 2.2.3.4 Wind

The wind loads are calculated per the NBCC section 4.1.7.

$p = I_w q C_e C_t C_g C_p$

The external pressure coefficients are determined based on 4.1.7.6. For the flat roof we can establish the positive and negative wind loads on each face of the building. A detailed review of the wind design procedure is not included in this guide

$p = I_w q C_e C_t (C_{g1} C_{p1} - C_{g4} C_{p4}) = 1.0(0.41\ kPa)(1.0)(1.0)\big(0.75 - (-0.55)\big) = 0.533\ kPa$

$p_E = I_w q C_e C_t (C_{g1E} C_{p1E} - C_{g4E} C_{p4E}) = 1.0(0.41\ kPa)(1.0)(1.0)\big(1.15 - (-0.8)\big) = 0.800\ kPa$

The ends zones are determined based on the height and a least horizontal dimension of the building.

$z = min(0.1W, 0.4H) = min[0.1(27.4m), 0.4(11.8m)] = 2.74m$

$y = max(6m, 2z) = 6m$

The wind loads need to be assessed in both directions. For more details refer to the NBCC 2015.

$P_x = 0.533\ kPa\ (11.8m)(54m - 6m) + 0.800\ kPa(11.8m)(6m) = 359\ KN$

$P_y = 0.533\ kPa\ (11.8m)(27.4m - 2.74m) + 0.800\ kPa(11.8m)(2.74m) = 181\ KN$

### 2.2.3.5 Earthquake

The lateral loads in the building are calculated using the NBCC section 4.1.8 Equivalent Static Load Procedure. Based on an assumed site class C, a summary of the seismic loads follows:

| | **Sa(0.2)** | **Sa(0.5)** | **Sa(1.0)** | **Sa(2.0)** | **Sa(5.0)** | **Sa(10.0)** | **PGA** |
|---|---|---|---|---|---|---|---|
| **Sa(T)** | 0.439 | 0.237 | 0.118 | 0.056 | 0.015 | 0.0055 | 0.281 |
| **F(T)** | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| **S(Ta)** | max(0.439,0.237) | 0.237 | 0.118 | 0.056 | 0.015 | 0.0055 | 0.281 |

Shearwall System: $T_a = 0.05\ h_n^{3/4} = 0.05\ (11.8m)^{3/4} = 0.32s$

$S(T_a) I_e = (0.36g)(1.0)(1.0) = 0.36g$

Lateral System: CLT Shearwalls (O86-19 11) $R_d = 2.0; R_o = 1.5$

$$V_s = \frac{S(T_a) M_v I_E}{R_d R_o} W = \frac{0.36(1.0)(1.0)}{2.0x1.5} W = 0.12W = 0.12\ (12{,}798KN) = 1{,}536\ KN$$

→ Governs over wind

Table 2-1Vertical Distribution of Seismic Forces

| Level | Dead Load | Snow Load | Weight ($W_i$) | Height ($H_i$) | $W_i H_i$ | $\frac{W_x H_x}{\sum W_i H_i}$ | $F_s = V \frac{W_x H_x}{\sum W_i H_i}$ |
|---|---|---|---|---|---|---|---|
| **Roof** | 1.5 KPa | 0.25(2.3KPa) | 3,372 KN | 11.8m | 39,790 | 0.408 | 627 KN |
| **L3** | 2.9 KPa | | 4,713 KN | 8.0m | 37,704 | 0.387 | 594 KN |
| **L2** | 2.9 KPa | | 4,713 KN | 4.25m | 20,030 | 0.205 | 315 KN |
| **Totals** | | | **12,798 KN** | | **97,524** | | **1536 KN** |

Accidental torsion would also need to be included in the earthquake loads. For simplicity this will be ignored in this design example

### 2.2.4 MATERIAL PROPERTIES

The materials used in this design example are based on the standard materials provided in the wood standard CSA O86-19.

2.2.4.1 Glulam:

Generic SPF grades have been used for the design guide based on the O86-19 design standard. Always check with local suppliers for species and glulam grade availability and costs.

Table 2-2: Glulam grade excerpt from O86-19

**Table 7.2 (Concluded)**

| | Spruce-Lodgepole Pine-Jack Pine | | | | Hem Fir and Douglas Fir-Larch | |
|---|---|---|---|---|---|---|
| | 20f-E | 20f-EX | 14t-E | 12c-E | 24f-E | 24-EX |
| Bending moment (pos.), $f_b$ | 25.6 | 25.6 | 24.3 | 9.8 | 30.6 | 30.6 |
| Bending moment (neg.), $f_b$ | 19.2 | 25.6 | 24.3 | 9.8 | 23.0 | 30.6 |
| Longitudinal shear, $f_v$ | 1.75 | 1.75 | 1.75 | 1.75 | 1.75 | 1.75 |
| Compression parallel, $f_c$ | 25.2* | 25.2* | 25.2 | 25.2 | — | — |
| Compression parallel combined with bending, $f_{cb}$ | 25.2* | 25.2 | 25.2 | 25.2 | — | — |
| Compression perpendicular, $f_{cp}$ | 5.8 | 5.8 | 5.8 | 5.8 | 4.6 | 7.0 |
| Compression face bearing | | | | | | |
| Tension face bearing | 5.8 | 5.8 | 5.8 | 5.8 | 7.0 | 7.0 |
| Tension net section, $f_{tn}$ (see Clause 7.5.11) | 17.0* | 17.0 | 17.9 | 17.0 | 20.4* | 20.4 |
| Tension gross section, $f_{tg}$ | 12.7* | 12.7 | 13.4 | 12.7 | 15.3* | 15.3 |
| Tension perpendicular to grain, $f_{tp}$ | 0.51 | 0.51 | 0.51 | 0.51 | 0.83 | 0.83 |
| Modulus of elasticity, $E$ | 10 300 | 10 300 | 10 700 | 9 700 | 13 100 | 13 100 |

2.2.4.2 CLT:

For this guide we will use generic stress grades based on the standard stress grades published in O86-19. Always confirm with local suppliers to determine the stress grades available for a given project.

Table 2-3: CLT grade types table excerpt from O-86-19

**Table 8.1**
**Primary CLT grades**
(See Clause 8.2.3.)

| Stress grade | Species combinations and grades of laminations |
|---|---|
| E1 | 1950 $F_b$-1.7E Spruce-Pine-Fir MSR lumber in all longitudinal layers and No. 3/Stud Spruce-Pine-Fir lumber in all transverse layers |
| E2 | 1650 $F_b$-1.5E Douglas Fir-Larch MSR lumber in all longitudinal layers and No. 3/Stud Douglas Fir-Larch lumber in all transverse layers |
| E3 | 1200 $F_b$-1.2E Northern Species MSR lumber in all longitudinal layers and No. 3/Stud Northern Species lumber in all transverse layers |
| V1 | No. 1/No. 2 Douglas Fir-Larch lumber in all longitudinal layers and No. 3/Stud Douglas Fir-Larch lumber in all transverse layers |
| V2 | No. 1/No. 2 Spruce-Pine-Fir lumber in all longitudinal layers and No. 3/Stud Spruce-Pine-Fir lumber in all transverse layers |

Table 2-4: CLT specified strengths for longitudinal and transverse layers

**Table 8.2**
**Specified strengths and moduli of elasticity of laminations in primary CLT grades, MPa**
(See Clause 8.2.4.)

| Stress grade | Longitudinal layers | | | | | | Transverse layers | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $f_b$ | $E$ | $f_t$ | $f_c$ | $f_s$ | $f_{cp}$ | $f_b$ | $E$ | $f_t$ | $f_c$ | $f_s$ | $f_{cp}$ |
| E1 | 28.2 | 11 700 | 15.4 | 19.3 | 0.50 | 5.3 | 7.0 | 9000 | 3.2 | 9.0 | 0.50 | 5.3 |
| E2 | 23.9 | 10 300 | 11.4 | 18.1 | 0.63 | 7.0 | 4.6 | 10 000 | 2.1 | 7.3 | 0.63 | 7.0 |
| E3 | 17.4 | 8300 | 6.7 | 15.1 | 0.43 | 3.5 | 4.5 | 6500 | 2.0 | 5.2 | 0.43 | 3.5 |
| V1 | 10.0 | 11 000 | 5.8 | 14.0 | 0.63 | 7.0 | 4.6 | 10 000 | 2.1 | 7.3 | 0.63 | 7.0 |
| V2 | 11.8 | 9500 | 5.5 | 11.5 | 0.50 | 5.3 | 7.0 | 9000 | 3.2 | 9.0 | 0.50 | 5.3 |

Note that generic CLT panel properties can be calculated using the equations provided in O86-19, the same equations are provided in the appendix of the product standard PRG 320-19. It is also acceptable to use the overall section properties provided in the design tables in PRG 320-19.

#### 2.2.5 FIRE RATING

Fire-resistance ratings are determined based on the occupancy and building classification. It can range from cases where no fire-resistance rating is required to specific fire-resistance ratings ranging from 45min to 2-hour, where heavy timber minimum size requirements would be permitted in buildings required to have a fire-resistance rating not more than 45min. For this example a **1-hour** fire rating has been chosen for illustrative purposes, fire design is done using Annex B, from the O86-19.

### 2.3 Member Size Summary

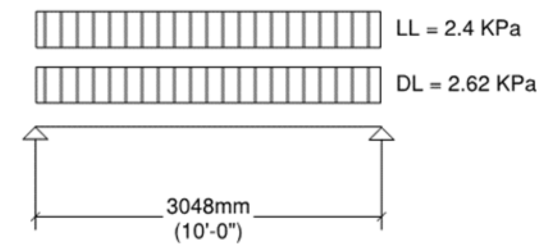
| | Floor | Roof |
|---|---|---|
| **CLT slab** | 105mm 3 ply E1 | 105mm 3ply V2 |
| **Glulam Purlins** | 365x380mm SPF 20f-E | 315x380mm SPF 20f-E |
| **Glulam Girders** | 265X876mm SPF 20f-E | 215X836mm SPF 20f-E |
| **Glulam Columns** | 365x342 SPF 16c-E | |
| **CLT Walls** | 245mm 7ply E1 | |

# 3 Gravity Design

## 3.1 CLT Floor Panel Design

Because the CLT panels in this design example are based on 2.4mx12.192m panels, and the purlins are spaced at 3.05m, the panels are multi-span-continuous, allowing for lower design moments, smaller deflections, and ease of erection. For simplicity, a conservative, simple span panel design is provided here. Ultimate Limit States (ULS) load combinations specified in the NBCC 2015 are considered. In the absence of alternate advice from a fire protection engineer, the fire case loads considered are per O86-19 Annex B Clause B.1.4.

| | **Load Combinations** | |
|---|---|---|
| | **Typical (1.25D+1.5L)** | **Fire (1.0D+1.0L)** |
| | $w_F = 6.88\ kN/m$ | $w_f = 5.02\ kN/m$ |




| | |
|---|---|
| $M_F = 7.99\ kNm/$m | $M_{F.fire} = 5.83\ kNm/m$ |
| $V_F = 10.49\ kN/m$ | $V_{F.fire} = 7.65\ kN/m$ |
| Note: dead load includes only the self-weight of the panel and the loads supported by the panel (framing weight is not included. | |

The design calculations are provided for the floor panel. The panel properties and strength calculations are per O86-19, chapter 8; other references for CLT panel properties include the FP Innovations CLT handbook and PRG 320-2019. Additionally, CLT manufacturers with PRG 320 certification provide panel properties for their CLT panels; these can be relied on for the design process,.

### 3.1.1 PANEL PROPERTIES

For the 3ply E1 rated panel we can establish the panel properties using the longitudinal and transverse lamination properties shown in Table 2-4. From here we can calculate the stiffness, and strength properties for the panel using O86-19 section 8.4.3.2. This design example focuses on the primary direction as the panels are uniformly supported in their secondary direction.

Table 3-1: 3 ply E1 Panel Stiffness Calculation in the Primary Direction

| Lam Thickness (mm) | Lam Material | $E_{i,y}$ (Mpa) | $I_{i,y}$ (mm$^4$/m) | $E_iI_{i,y}$ (Nmm$^2$/m) | $A_i$ (mm$^2$/m) | neutral axis, z (mm) | $E_{i,y}A_{i,y}z_{i,0}^2$ (Nmm$^2$/m) |
|---|---|---|---|---|---|---|---|
| **35** | 1950 MSR | 11,700 | 3572917 | 41.80 x10$^9$ | 35000 | 35 | 501.64 x10$^9$ |
| **35** | No.3 SPF | 9000 /30 = 300 | 3572917 | 1.07 x10$^9$ | 35000 | 0 | 0 |
| **35** | 1950 MSR | 11,700 | 3572917 | 41.80 x10$^9$ | 35000 | 35 | 501.64 x10$^9$ |
| **Total** | | | | **84.68x10$^6$** | | | **1003.28x10$^9$** |

$$EI_{eff} = \sum E_iI_i + \sum E_iA_iz_i^2 = 1087.95x10^9\ Nmm^2/m$$

### 3.1.2 PANEL STRENGTH

The panel strength is then calculated as per O86-19 chapter 8; as noted in the panel property section, this example is for the panel strength in the primary axis of the panel. Note that most strength calculations in O86-19 include a load duration factor ($K_D$) described in O86-19 section 5.3.2. In this case, the load duration needs to be calculated based on section 5.3.2.2 as the dead loads are greater than the live loads

**Primary Axis Bending Strength – O86-19 CL 8.4.3**

| | | | |
|---|---|---|---|
| $K_D = 1.0 - 0.5log\left(\frac{P_{Longterm}}{P_{short}}\right) = 1.0 - 0.5\log\left(\frac{2.62}{2.4}\right) => 0.98$ | | | CL 5.3.2.2 |
| $K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $S_{eff} = \frac{EI_{eff}}{E_{outer}(h/2)} = 1{,}771{,}373mm^3$ | CL 8.4.3.1 |
| $K_{rb,0} = 0.85$ | CL 8.4.3.1 | | |
| $\phi = 0.9$ | CL 8.4.3.1 | $M_r = \phi\ f_b(K_DK_HK_{sb}K_T)S_{eff}K_{rb}$ | CL 8.4.3.1a |
| $f_b = 28.2MPa$ | Table 8.2 | $= 0.9(28.2)(1.0)(0.98)(1{,}771{,}373)0.85$<br>$= 37.4\ KNm/m$ | $M_r > M_f$→ OK |

**Primary Axis Shear Strength – O86-19 CL 8.4.4**

| | | | |
|---|---|---|---|
| $K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $A_{g,0} = \sum A_i = 105{,}000\ mm^2$ | CL 8.4.4.2a |
| $\phi = 0.9$ | CL 8.4.4.2a | $V_r = \phi\ f_s(K_D K_H K_{sv} K_T)\dfrac{2\ A_{g,0}}{3}$ $= 0.9(0.5)(0.98)(1.0)\dfrac{2}{3}(105000) = 30.9\ KN/m$ | CL 8.4.4.2a $V_r > V_f$ → OK |
| $f_s = 0.5MPa$ | Table 8.2 | | |

3.1.3 DEFLECTION

Shear deformation can make up a significant portion of the deflection in the CLT due to shear deformations and specifically the deformations due to the rolling shear in the cross layers. The impacts of rolling shear in CLT are addressed in Annex A of O86-19. Direct consideration of the shear effects on deflection is considered by including a term accounting for shear deformation. Additionally, creep must be considered for CLT. Similar to concrete, creep represents permanent long-term deformations resulting long term loads; creep is discussed more in O86-19 Annex A, section A.8.5.3.

**Deflection including shear deformation– O86-19 A8.5.2**

| | |
|---|---|
| $\Delta = \frac{5}{384}\frac{\omega L^4}{EI_{eff}} + \frac{1}{8}\frac{\omega L^2}{GA_{eff}}$ = elastic Def. + shear def | CL A.8.5.2 |
| G = E/16 | CL 8.2.4l |
| $GA_{eff} = \dfrac{h - \frac{t_1}{2} - \frac{t_n}{2}}{\frac{t_1}{2G_1 b_Y} + \sum_{i=2}^{n-1}\frac{t_i}{G_i b_y} + \frac{t_n}{2G_n b_y}} = \dfrac{h - \frac{t_1}{2} - \frac{t_3}{2}}{\frac{1}{b_y}\left(\frac{t_1}{2G_{1950MSR}} + \frac{t_2}{G_{SPF-No.3}} + \frac{t_3}{2G_n}\right)}$ $= \dfrac{105 - \frac{35}{2} - \frac{35}{2}}{\frac{1}{1000}\left[\frac{35}{2\left(\frac{11700}{16}\right)} + \frac{35}{\frac{9000/30}{16}} + \frac{35}{2\left(\frac{11700}{16}\right)}\right]} = 8.06\ x10^6\ N/m$ | CL 8.4.3.2 |
| $\Delta = \frac{5}{384}\frac{\omega L^4}{EI_{eff}} + \frac{1}{8}\frac{\omega L^2}{GA_{eff}} =$ $\Delta_D = \dfrac{5}{384}\dfrac{wL^4}{EI_{eff}} + \dfrac{1}{8}\dfrac{wL^2}{GA_{eff}} = \dfrac{5}{384}\dfrac{(2.62)(3048)^4}{1087.95x10^9} + \dfrac{1}{8}\dfrac{(2.62)(3048)^2}{8.06x10^6} = 3.2mm$ $\Delta_L = \dfrac{5}{384}\dfrac{wL^4}{EI_{eff}} + \dfrac{1}{8}\dfrac{wL^2}{GA_{eff}} = \dfrac{5}{384}\dfrac{(2.4)(3048)^4}{1087.95x10^9} + \dfrac{1}{8}\dfrac{(2.4)(3048)^2}{8.06x10^6} = 2.9mm$ | CL A.8.5.2 L/1051 < L/360 OK |

**Deflection including Creep – O86-19 A8.5.3**

| | |
|---|---|
| $K_{Creep} = 2.0$ | CL A.8.5.2 |
| $\Delta_{total} = \Delta_{ST} + K_{creep}\Delta_{LT}$ $= 2.9mm + 2(3.2mm) = 10.3mm$ | CL A.8.5.2 L/295 < L/240 OK |

### 3.1.4 VIBRATION

Annex A in O86-19 provides guidance on CLT span limits for vibration. The section is based on physical testing of bare simple span panels, as can be seen in the notes in Clause A.8.5.3. Provisions allow for a 20% increase in maximum span length for multi-span floors. Additionally it is noted that for floors with concrete topping the equation may be used taking only the mass of the CLT provided the weight of the concrete is note greater than the weight of the CLT. These criteria are not met for this CLT meaning this vibration analysis is not applicable for this case. The calculations I completed for illustrative purposes here

**Vibration – O86-19 CL A.8.5.3**

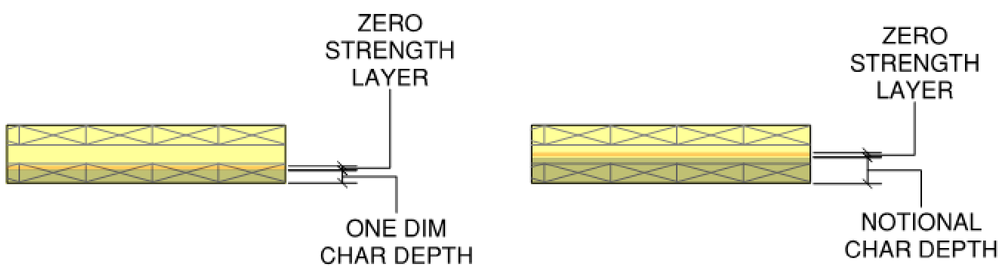
| | |
|---|---|
| $m_{CLT} = 46.2kg/m\ (CLT\ only)$<br>$m_{Concrete} = 2400kg/m^3 \times 0.038m \times 1m = \ 910.2kg/m(CLT\ only)$ | |
| $\iota_V = 0.11\dfrac{\left(EI_{eff}/10^6)\right)^{0.29}}{m^{0.12}} = 0.11\dfrac{(1087.95x10^3)^{0.29}}{4.91^{0.12}} = 3.91m$ | CL A.8.5.3 |

More detailed vibration analysis might be appropriate using FEM models and detailed analysis of accelerations. This approach is beyond the scope of this example. Refer to the U.S. Mass Timber Floor Vibration Design Guide.

| | |
|---|---|
| | |

### 3.1.5 FIRE DESIGN

CSA O86-19 Annex B provides guidance on fire design of CLT per clause B.4.6 if the char depth stays within a single lamination thickness ($t_n$), then *one-dimensional char* rate is appropriate. If the char depth exceeds the depth of one lamination then *Notional Char* rates should be used.




**Char Calculation – O86-19 CL B.4.3 & CL B.4.4**

| | | | |
|---|---|---|---|
| t = 60 min | | $x_{c,o} = \ \beta_o t = 0.65\ (60) = \ 39mm > t_n$ | CL B.4.3 |
| βo = 0.65 mm/min | Table B.2 | One-dimensional char not ok | CL B.4.6 |
| βn = 0.80 mm/min | Table B.2 | $x_{c,n} = \ \beta_n t = 0.80\ (60) = \ 48mm$ | CL B.4.4 |
| $x_t$ = 7mm (for t ≥ 20) | Cl B.5 | $h_{fire} = \ h - x_{c,n} - \ x_t = 105 - 48 - 7 = \ 55mm$ | |

Table 3-2: 3 ply Panel Layup after Char

| Lam thickness (mm) | Lam Material | Direction | Charred Depth | Remaining thickness (mm) |
|---|---|---|---|---|
| **35** | 1950 MSR | 0 | | 35 |
| **35** | No.3 SPF | 90 | 55-35=20 | 15 |
| **35** | 1950 MSR | 0 | 35 | |

The cross-section properties based on the charred layup must be calculated per O86-19 chapter 8, for fire cases it is not possible to use the provided section properties provided in PRG 320-19. In this case, the charred section has one remaining lamination in each direction; the strength calculations can be simplified to the single remaining lam in each direction.  The strength calculations for the strong direction are as follows:

**Bending Strength – CSA O86-19 Annex B**

| | | | |
|---|---|---|---|
| $K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $S_{eff} = \dfrac{E_1 I_{eff}}{E_1(h_{tot} - NA)} = \dfrac{35^3(1000)/12}{(35/2)}$ | CL 8.4.3.1 |
| $K_D = 1.15$ | CL B.3.3 | | |
| $K_{fi} = 1.25$ | CL B.3.9 | $= 204167 mm^3/m$ | |
| $K_{rb,0} = 0.85$ | CL 8.4.3.1 | $M_r = \phi\, f_b\left(K_D K_H K_{sb} K_T K_{fi}\right) S_{eff} K_{rb}$ | CL 8.4.3.1a |
| (No cross lams acting) | | $= 1.0(28.2)(1.15x1.0x1.25)(204167)0.85$ | |
| $\phi = 1.0$ | CL B.3.2 | $= 7.0\ KNm/m$ | $M_{r.fire} > M_{f.fire}$ |
| $f_b = 28.2MPa$ | Table 8.2 | | → OK |

**Shear Strength – CSA O86-19 Annex B**

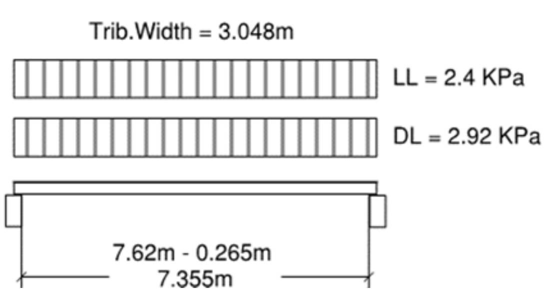
| | | | |
|---|---|---|---|
| $K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $A_{g,0} = 35{,}000\ mm^2$ | CL 8.4.4.2a |
| $K_D = 1.15$ | CL B.3.3 | $V_r = \phi\ f_s(K_D K_H K_{sv} K_T)\dfrac{2\ A_{g,0}}{3}$ | CL 8.4.4.2a |
| $K_{fi} = 1.25$ | CL B.3.9 | | |
| $\phi = 1.0$ | CL B.3.2 | $= 1.0(0.5)(1.15x1.0x1.25)\frac{2}{3}(35000)$ | $V_{r.fire} > V_{f.fire}$ |
| $f_s = 0.5MPa$ | Table 8.2 | $= 16.8\ KN/m$ | → OK |

## 3.2 Glulam Beam

Glulam beam design is completed as per Chapter 7 of O86-19.  The CWC *Wood Design Manual* also provides design examples for glulam beams.  Additional tables providing design strengths for typical glulam beam sizes are provided in the *Wood Design Manual*; the strengths provided are modified with design specific modifiers as outlined in the manual.  Refer to the *Wood Design Manual* for more information.

### 3.2.1 PURLIN STRENGTH AND PROPERTIES

The loads for the glulam purlins will be determined based on the loads described in section 1.2.3 and live load reduction factors per section 4.1.5.8 of the NBCC 2015. The strength and stiffness of the glulam purlins are taken from the *Wood Design Manual.*




**Factored Loads**

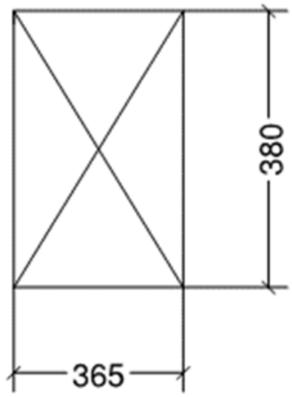
$A_{trib.purlin.} = 7.62m\ \times 3.048m = 23.2m^2 > 20m^2$

$$LL_{red.inner} = \left[0.3 + \sqrt{\frac{9.8}{A_{purlin}}}\right] 2.4\ KPa = 2.28\ KPa$$

| **Typical (1.25D+1.5L)** | **Fire (1.0D+1.0L)** |
|---|---|
| $w_F = 21.6\ KN/m$ | $w_f = 15.8 KN/m$ |
| $M_F = 145.7 KNm$ | $M_{F.fire} = 104\ KNm$ |
| $V_F = 89.3\ KN$ | $V_{F.fire} = 51.4\ KN$ |
| $W_F = 165.8\ KN$ | $W_{F.fire} = 114.6\ KN$ |

Note that the shear force provided here represents the full reaction load at the ends of the beam.  Clause 7.5.7.2.1 notes that the shear loads acting within a distance equal to the member depth away from the face of the support.

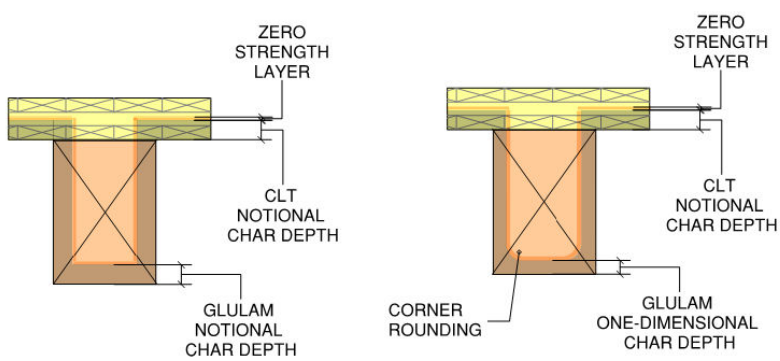
**Member Strength and Deflection – CSA O86-19 section 7.5 and CSA Wood Design Manual**

| | | | |
|---|---|---|---|
|   | $K_D = 1.0 - 0.5log\left(\frac{P_{Longterm}}{P_{short}}\right)$ $= 1.0 - 0.5\log\left(\frac{2.92}{2.4}\right) => 0.957$ | CL 5.3.2.2 | |
| | $K_L = 1.0$ | CL.7.5.6.4 | |
| | $K_{zbg} = \left(\frac{130}{b}\right)^{0.1}\left(\frac{610}{h}\right)^{0.1}\left(\frac{9100}{L}\right)^{0.1}$ $= \left(\frac{130}{365}\right)^{0.1}\left(\frac{610}{380}\right)^{0.1}\left(\frac{9100}{7620}\right)^{0.1} = 0.963$ | Cl 7.5.6.5.1 | |
| | $M_r = M_r' \min(K_{zbg}, K_L)K_x K_D = 202\ KNm\ (0.963)$ $= 195KNm$ | CL 7.5.6.5 $M_r > M_f$ | OK |
| Material Grade: 20f-E | $V_{f@d} = V_f \frac{(7.355/2 - .380)}{7.355/2} K_D$ $= 79.3KN(0.897)(1.0) = 146KN$ | CL 7.5.7.2.1 | OK |
| | $V_r = V_r' K_D = 146KN(1.0) = 146KN$ | CL 7.5.7.3a $V_r > V_f$ | |
| **CSA Wood Design Manual Design Strength Values** | $W_r = \frac{W_r L^{0.18}}{L^{0.18}} K_D = \frac{552KN}{7.62^{0.18}}(1.0) = 383KN$ | CL 7.5.7.3b $W_r > W_f$ | OK |
| $V_r{}' = 146\ KN$ | $\Delta_L = \frac{5}{384}\frac{(2.28x3m)(7355)^4}{17200 \times 10^9} = 15.2\ mm$ | NBCC L/483<L/360 | OK |
| $M_r' = 202KNm$ | $\Delta_D = \frac{5}{384}\frac{(2.92x3m)(7355)^4}{17200 \times 10^9} = 19.5\ mm$ | CL 5.4.3 L/377<L/360 | OK |
| $W_r L^{0.18} = 552\ KN$ | | | |
| EI = 17200 x $10^9$ $Nmm^2$ | $\Delta_{Tot} = \Delta_L + \Delta_D = 34.7\ mm$ | CL 5.4.2 L/211<L/180 | OK |

Note that the stability factor (KL) is set to 1.0 because it is continuously supported on it's compression face (top of beam) by the CLT. CSA O86-19 CL 7.5.6.4.2 noted that this allows the unsupported length to be taken as 0, resulting in a stability factor of 1.0.

### 3.2.2 FIRE DESIGN

CSA O86-19 Annex B provides guidance for calculating the remaining section properties after a fire. Two approaches are provided for members with char on multiplate faces: the first is the more precise combination of one-dimensional char and explicit inclusion of corner rounding, the alternate uses the notional char depth to implicitly account for corner rounding.




This design example takes the simplified notional char approach. For a 1hr fire-resistance rating the, the char depth can be calculated as follows:

**Char Calculation – O86-19 CL B.4.3 & CL B.4.4**

| | | | |
|---|---|---|---|
| t = 60 min | | $x_{c,n} = \beta_n t = 0.70\ (60) = 42mm$ | CL B.4.4 |
| βn = 0.70 mm/min | Table B.2 | $h_{fire} = h - (x_{c,n} + x_t) = 380 - (42 + 7) = 331mm$ | |
| $x_t$ = 7mm | Cl B.5 | $b_{fire} = b - 2(x_{c,n} + x_t) = 365 - 2(42 + 7) = 267mm$ | |

**Bending Strength – CSA O86-19 Annex B**

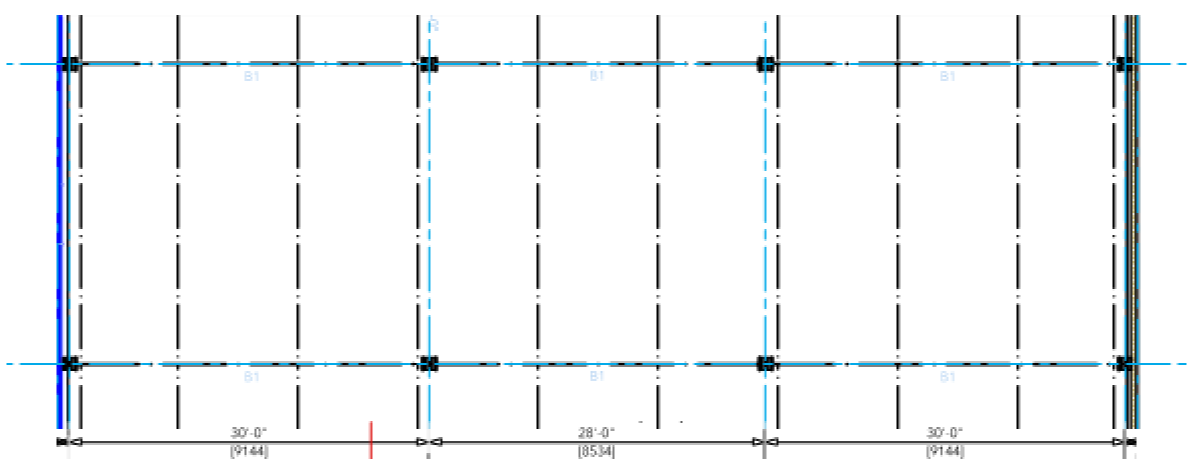
| | | | |
|---|---|---|---|
| $K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $S_x = \frac{h_{fire}^2 b_{fire}}{6} = \frac{331^2(267)}{6} = 4875.5 \times 10^3 mm^3$ | |
| $K_D = 1.15$ | CL B.3.3 | | |
| $K_L = 1.0$ | CL 7.5.6.4.4 | | |
| $K_{Zbg} = 0.953$ | CL B.3.5 | $M_r = \phi\ f_b(K_D K_H K_{sb} K_T)\min(K_L, K_{zbg})(K_{fi})S_{eff}$ | CL 7.5.6 |
| $K_{fi} = 1.35$ | CL B.3.9 | $= 1.0(25.6)(1.15)(0.953)(1.35)(4875.5 \times 10^3)$ | |
| $\phi = 1.0$ | CL B.3.2 | $= 184.7KNm$ | $M_{r.fi} > M_{f.fi}$ |
| $f_b = 25.6MPa$ | Table 8.2 | | OK |

**Shear Strength – CSA O86-19 Annex B**

| | | | |
|---|---|---|---|
| $K_H = K_{sv} = K_T = 1.0$ | CL 8.3 | $A_{g,0} = h_{fire} b_{fire} = 3319x267 = 98532\ mm^2$ | |
| $K_D = 1.15$ | CL B.3.3 | | |
| $K_{fi} = 1.35$ | CL B.3.9 | $V_r = \phi\ f_v(K_D K_H K_{sv} K_T)\frac{2\ A_{g,0}}{3}$ | CL 7.5.6 |
| $\phi = 1.0$ | CL B.3.2 | | |
| $f_s = 1.75MPa$ | Table 8.2 | $= 1.0(1.75)(1.15x1.0x1.35)\frac{2}{3}(98532) = 178.4KN$ | $V_{r.fi} > V_{f.fi}$ OK |

### 3.2.3 GIRDER STRENGTH AND PROPERTIES

The girder design is completed similarly to the purlin design discussed in section 2.2., including live load reduction factors. There are 2 bay cases as shown in the figure below: outer bays are 9.14m wide, supporting 4 purlins each as shown in the below image, inner bay 8.53m wide, supporting only 2 purlins.

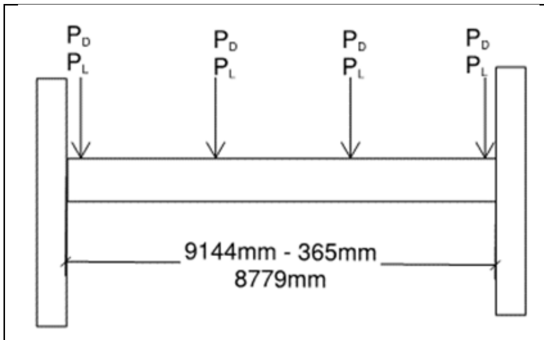



As a result, the tributary area associated with each girder varies, and the live load reduction factor must be calculated separately. Although he tributary area of the girders is based on the full bay width, it should also be noted that the beam frame in to the face of the columns, reducing the total length of the beam in to 8.78m and 8.17m respectively. The worst case of moments and shears are taken for the 2 different girder configurations and summarized below.

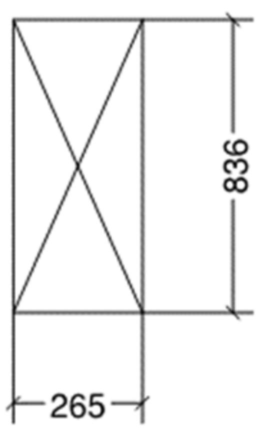
**Live Load Reduction per the NBCC CL 4.1.5.8**

| | |
|---|---|
| $A_{girder.outer} = 4\ A_{trib.purlin} = 3.5\ (3.048m \times 7.62m) = 81.3m^2 > 20m^2$ | |
| $LL_{red.inner} = \left[0.3 + \sqrt{9.8/92.9m^2}\right] 2.4\ KPa = 1.55\ KPa$ | |
| $A_{girder.inner} = 2\ A_{trib.purlin} = 2\ (7.62m \times 3.048m) = 46.5m^2 > 20m^2$ | |
| $LL_{red.inner} = \left[0.3 + \sqrt{9.8/46.5m^2}\right] 2.4\ KPa = 1.82\ KPa$ | |

**Factored Design Loads**

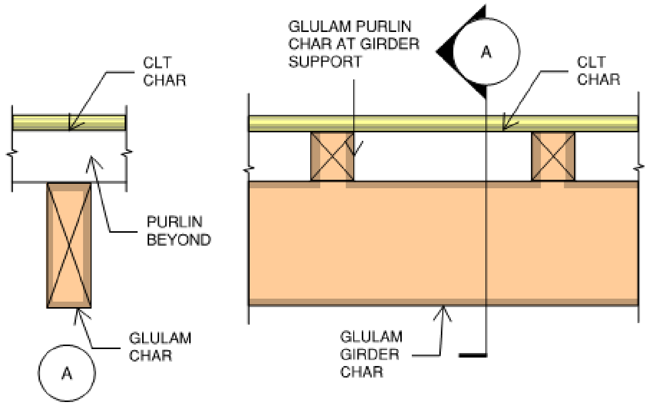
| | **Typical (1.25D+1.5L)** | **Fire (1.0D + 1.0L)** |
|---|---|---|
|   | $P_{f.outer} = 137.0\ KN$ | $P_{fi.outer} = 102.6\ KN$ |
| | $P_{f.inner} = 148.2\ KN$ | $P_{fi.inner} = 110.1\ KN$ |
| | $M_{f.max} = 435.5\ KNm$ | $M_{fi.max} = 323.5\ KNm$ |
| | $V_{f.\max.@d} = 148.2\ KN$ | $V_{fi.mas@d} = 110.1\ KN$ |
| | $R_F = 264.2\ KN$ | $R_{fi.max} = 197.7\ KN$ |
| | $W_F = 528.4\ KN$ | $W_{fimax} = 395.4\ KN$ |

**Member Strength and Deflection – CSA O86-19 section 7.5 and CSA Wood Design Manual**

| | | | |
|---|---|---|---|
|   | $K_D = 1.0 - 0.50 \log\left(\frac{P_{LT}}{P_{ST}}\right) = 1.0 - 0.5 \log\left(\frac{2.92}{1.5}\right) = 0.855$ | CL 5.3.2.2 | |
| | $K_L = 1.0$ | CL.7.5.6.4 | |
| | $K_{zbg} = \left(\frac{130}{b}\right)^{0.1} \left(\frac{610}{h}\right)^{0.1} \left(\frac{9100}{L}\right)^{0.1} = \left(\frac{130}{265}\right)^{0.1} \left(\frac{610}{836}\right)^{0.1} \left(\frac{9100}{8800}\right)^{0.1} = 0.905$ | Cl 7.5.6.5.1 | |
| | $M_r = M_r' \min(K_{zbg}, K_L) K_x K_D = 711\ KNm\ (0.905)(0.855) = 543\ KNm$ | CL 7.5.6.5 $M_r > M_f$ | OK |
| | $Vol = 0.265m(0.836m)(8.78m) = 1.94m^3 < 2.0m$ Ie. no need to consider calculation of Wr | CL 7.5.7.3.a) | |
| Material Grade: 20f-E | $V_r = V_r' K_D = 233\ KN(0.833) = 194\ KN$ | CL 7.5.7.3.b) $V_r > V_f$ | OK |
| **CSA Wood Design Manual Design Strength Values** | $\Delta_L = 3.73\ mm$ | NBCC L/1332<L/360 | OK |
| $V_r' = 243\ KN$ | | | |
| $M_r' = 777\ KNm$ | $\Delta_D = 6.59\ mm$ | CL 5.4.3 L/2509<L/360 | OK |
| $W_r L^{0.18} = 841\ KN$ | | | |
| EI = 152000 x $10^9$ Nmm² | $\Delta_{Tot} = \Delta_L + \Delta_D = 10.3\ mm$ | CL 5.4.2 L/870<L/180 | OK |
| | | | |

3.2.4 FIRE DESIGN

Girder design for fire is nearly identical to the purlin design except that stacked purlins allow airflow over tops of girders, allowing for exposure to fire on 4 faces as shown in the image below




We calculate the fire properties of the section using the same approach provided in section 3.2.2 for the purling fire design

**Char Calculation – O86-19 CL B.4.3 & CL B.4.4**

| | | | |
|---|---|---|---|
| t = 60 min | | $x_{c,n} = \beta_n t = 0.70\ (60) = 42mm$ | CL B.4.4 |
| βn = 0.70 mm/min | Table B.2 | $h_{fire} = h - 2(x_{c,n} + x_t) = 836 - 2(42+7) = 738mm$ | |
| $x_t$ = 7mm | Cl B.5 | $b_{fire} = b - 2(x_{c,n} + x_t) = 265 - 2(42+7) = 167mm$ | |

**Bending Strength – CSA O86-19 Annex B**

| | | | |
|---|---|---|---|
| $K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $S_x = \frac{h_{fire}^2 b_{fire}}{6} = \frac{738^2(167)}{6} = 15.16\ x\ 10^6 mm^3$ | |
| $K_D = 1.15$ | CL B.3.3 | | |
| $K_L = 1.0$ | CL 7.5.6.4.4 | | |
| $K_{Zbg} = 0.905$ | CL B.3.5 | $M_{r.fire} = \phi\ f_b(K_D K_H K_{sb} K_T)\min(K_L, K_{zbg})(K_{fi})S_{eff}$ | CL 7.5.6 |
| $K_{fi} = 1.35$ | CL B.3.9 | $= 1.0(25.6)(1.15x1.0)(0.905)(1.35)(15.16x10^6)$ | |
| $\phi = 1.0$ | CL B.3.2 | $= 545.2\ KNm$ | $M_{r.fi} > M_{f.fi}$ |
| $f_b = 25.6MPa$ | Table 8.2 | | OK |

**Shear Strength – CSA O86-19 Annex B**

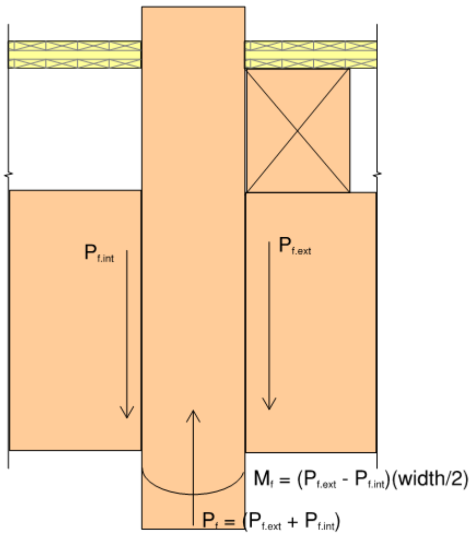
| | | | |
|---|---|---|---|
| $K_H = K_{sv} = K_T = 1.0$ | CL 8.3 | $A_{g,0} = h_{fire} b_{fire} = 738 \times 167 = 123246\ mm^2$ | |
| $K_D = 1.15$ | CL B.3.3 | | |
| $K_{fi} = 1.35$ | CL B.3.9 | $V_{r.fire} = \phi\ f_v(K_D K_H K_{sv} K_T)\frac{2\ A_{g,0}}{3}$ | CL 7.5.6 |
| $\phi = 1.0$ | CL B.3.2 | | |
| $f_s = 1.75MPa$ | Table 8.2 | $= 1.0(1.75)(1.15x1.0x1.35)\frac{2}{3}(123246)$ $= 223.2KN$ | $V_{r.fi} > V_{f.fi}$ OK |

## 3.3 Glulam Column

Glulam column design is completed as per Chapter 7 of O86-19. The CWC *Wood Design Manual* also provides design examples for Glulam columns. Additional tables for typical glulam column sizes and lengths are provided, refer to the *Wood Design Manual* for more information. The loads for the glulam columns will be determined based on the loads described in section 1.2.3 and live load reduction factors per section 4.1.5.8 of the NBCC 2015.

Note that the unbalanced loading of the beams will also impose a moment on the columns.  The axial load in the columns will be cumulative, but the because the columns can be assumed to be pinned at their base we can take the moment imposed from the unbalanced load at a single floor.

**Design Loads per the NBCC CL 4.1.5.8**




| | |
|---|---|
| $A_{col.interior.Live} = (2\ floors)\left(7.62m\right)\left(\frac{8.53m}{2}+\frac{9.144m}{2}\right)$<br>$= 134.7m^2 > 20m^2$ | |
| $LL_{red.inner} = \left[0.3+\sqrt{9.8/134.7m^2}\right]2.4\ KPa = 1.37\ KPa$ | |
| $P_S = 161.1\ kN$ | |
| $P_{D.roof} = 98.2\ kN$ | |
| $P_{D.floor} = 127.4\ kN + 67.8\ kN = 195.2\ kN$ | |
| $P_{l.floor} = 59.8\ kN + 31.8\ kN = 91.6\ kN$ | |
| **Typical (1.25D+1.5L+1.0S)** | **Fire (1.0D + 1.0L+1.0S)** |
| $P_f = 1086.9\ kN$ | $P_{f.fi} = 832.9\ kN$ |
| $M_f = (249.0-132.5)\left(\frac{0.365}{2}\right)$<br>$= 21.3\ kNm$ | $M_f$<br>$= (187.2-107.9)\left(\frac{0.365}{2}\right)$<br>$= 14.5\ kNm$ |

### 3.3.1 MEMBER STRENGTH AND PROPERTIES

The column strength must be compared for both moment and compression, and for combined loading as described in CSA O86 Clause 7.5

$$K_D = 1.0 - 0.50\log\left(\frac{P_{LT}}{P_{ST}}\right) = 1.0 - 0.5\log\left(\frac{477}{366}\right) = 0.942$$

**Compression strength per CSA O86-19 CL 7.5.8**

| | | | |
|---|---|---|---|
| $K_H = K_T = 1.0$ | CL 8.3 | $C_c = \frac{4250}{342} = 12.42 < 50$ | CL 7.5.8.2<br>OK |
| $K_{sc} = K_{se} = 1.0$ | CL 8.3 | | |
| $f_c = 25.2MPa$ | Table 8.2 | $K_{zcg} = 0.68\ Z^{-0.13} = 0.738$ | CL 7.5.8.5 |
| $E = 10300\ MPa$ | Table 8.2 | $K_c = \left(1.0+\frac{F_c K_{zcg} C_c^3}{35(0.87E)K_{se}K_T}\right)^{-1} = 0.898$ | CL 7.5.8.6 |
| | | $P_r = \phi\ f_c(K_D K_H K_{sc} K_T)K_{zcg}K_c A$<br>$= 0.8(25.2)(0.942)(0.738)(0.898)(365\times 342) =$<br>$1570.4\ KN$ | CL 7.5.8.5<br>$P_r > P_f$<br>OK |

**Moment strength per CSA O86-19 CL 7.5.6**

| | | | |
|---|---|---|---|
| $K_H = K_T = K_{sb} = 1.0$ | | $K_{zbg} = \left(\frac{130}{b}\right)^{0.1}\left(\frac{610}{d}\right)^{0.1}\left(\frac{9100}{L}\right)^{0.1} = 1.03$ | Cl 7.5.6.5.1 |
| $K_L = 1.0$ | CL.7.5.6.4 | | |
| $f_{b.plank} = 9.8MPa$ | Table8.2 | $S_y = \frac{342\times 365^2}{6} = 7593.8\times 10^3 mm^3$ | |
| | | | |
| | | $M_r = \phi\ f_b(K_D K_H K_{sc} K_T)\min(K_{zbg}K_c)\ S_x\ K_x$<br>$= 0.9(9.8)(0.942)(1.0)(7593.8\times 10^3) = 63.1\ KN$ | CL 7.5.8.5<br>$M_r > M_f$<br>OK |

**Combined Axial and Moment Interaction from CSA O86 CL 7.5.12**

| | |
|---|---|
| $I_y = \dfrac{342 \times 365^3}{12} = 1385.7 \times 10^6 mm^3$ | |
| $E = 9700\ Mpa$ | |
| $P_E = \dfrac{\pi^2(0.87E)(K_{se}K_T)(I_y)}{L_e^2} = \dfrac{\pi^2(8439)(1.0)(1385.7 \times 10^6)}{(4250)^2} = 6390.5\ kN$ | CL 7.5.12 |
| $\left(\dfrac{P_f}{P_r}\right)^2 + \dfrac{M_f}{M_r}\left(1 - \dfrac{P_f}{P_E}\right)^{-1} = \left(\dfrac{1086.9}{1570.4}\right)^2 + \left(\dfrac{21.3}{63.1}\right)\left(1 - \dfrac{1086.9}{6390.5}\right)^{-1} = 0.886 < 1.0$ | OK |

3.3.2 FIRE DESIGN

In general, columns in open space will be exposed to fire on all four faces.

**Char Calculation – O86-19 CL B.4.3 & CL B.4.4**

| | | | |
|---|---|---|---|
| t = 60 min | | $x_{c,n} = \beta_n t = 0.70\ (60) = 42mm$ | CL B.4.4 |
| βn = 0.70 mm/min | Table B.2 | $h_{fire} = h - 2(x_{c,n} + x_t) = 365 - 2(42 + 7) = 267\ mm$ | |
| $x_t$ = 7mm | Cl B.5 | $b_{fire} = b - 2(x_{c,n} + x_t) = 342 - 2(42 + 7) = 244\ mm$ | |

**Compression strength per CSA O86-19 CL 7.5.8**

| | | | |
|---|---|---|---|
| $K_H = K_T = 1.0$ | CL 8.3 | $C_c = \dfrac{4250}{244} = 17.41 < 50$ | CL 7.5.8.2<br>OK |
| $K_{sc} = K_{se} = 1.0$ | CL 8.3 | | |
| $K_D = 1.15$ | CL B.3.3 | $K_c = \left(1.0 + \dfrac{F_c K_{zcg} C_c^3}{35(0.87E_{05})K_{se}K_T}\right)^{-1}$ $= \left(1.0 + \dfrac{25.2(1.15)(1.35)(0.738)(17.4)^3}{35(8439)(1.0)}\right)$ $= 0.66$ | CL 7.5.8.6 |
| $K_{fi} = 1.35$ | CL B.3.9 | | |
| $\phi = 1.0$ | CL B.3.2 | | |
| $K_{Zcg} = 0.738$ | CL B.3.5 | | |
| $f_c = 25.2MPa$ | Table 8.2 | $P_r = \phi\ f_c(K_D K_H K_{sc} K_T)K_{zcg}K_c K_{fi} A$ $= 1.0(25.2)(1.15)(0.738)(0.66)(1.35)(267 \times 244) =$ $1241.5\ KN$ | CL 7.5.8.5<br>$P_r > P_f$<br>OK |
| $E_{05} = 9700\ MPa$ | Table 8.2 | | |
| | | | |

**Moment strength per CSA O86-19 CL 7.5.6**

| | | | |
|---|---|---|---|
| $K_H = K_T = K_{sb} = 1.0$ | | $S_y = \dfrac{244 \times 267^2}{6} = 2899.1 \times 10^3 mm^3$ | |
| $K_L = 1.0$ | CL.7.5.6.4 | | |
| $f_{b.plank} = 9.8MPa$ | Table8.2 | $M_r = \phi f_b(K_D K_H K_{sc} K_T)\min(K_L, K_{zbg})\,S_x\ K_x K_{fi}$ $= 1.0(9.8)(1.15)(1.0)(2899.1$ $\times 10^3)(1.35) = 44.1\ KN$ | CL 7.5.8.5<br>$M_r > M_f$<br>OK |
| $K_{zbg} = 1.03$ | CL B.3.5 | | |

**Combined Axial and Moment Interaction from CSA O86 CL 7.5.12**

| | |
|---|---|
| $I_y = \dfrac{244 \times 267^3}{12} = 387.0 \times 10^6 mm^3$ | |
| $E_{05} = 9700\ Mpa$ | |
| $P_E = \dfrac{\pi^2(0.87E)(K_{se}K_T)(I_y)}{L_e^2} = \dfrac{\pi^2(8439)(1.0)(387.0 \times 10^6)}{(4250)^2} = 1784.5\ kN$ | CL 7.5.12 |

| | |
|---|---|
| $\left(\frac{P_f}{P_r}\right)^2 + \frac{M_f}{M_r}\left(1 - \frac{P_f}{P_E}\right)^{-1} = \left(\frac{832.9}{1412.6}\right)^2 + \left(\frac{14.5}{44.1}\right)\left(1 - \frac{832.9}{1784.5}\right)^{-1} = 0.964 < 1.0$ | OK |

## 3.4 CLT Walls

CLT panels are typical designed per CSA O86-19, clause 8, as discussed in section 2.1 of this document. For more information refer to the *CLT Design Manual* from FP Innovations.

### 3.4.1 WALL PROPERTIES

The CLT properties are developed similarly to those in the slab section, except that only the laminations oriented in the primary direction are considered, as a result, the effective values provided in PRG 320 (or manufacturer specific published values) cannot be directly applied for in-plane compression. For the panel properties we are taking a 1.725m wide panel. The panel width is chosen to suit the CLT shearwall design, for more information on panel width, refer to chapter 3 of this document.

Table 3-3: CLT Wall Panel Lamination Properties

| Thickness (mm) | Lam Material | $E_{i,y}$ (Mpa) | $I_{i,y}$ ($mm^4/m$) | $E_iI_{i,y}$ ($Nmm^2/m$) | $A_i$ ($mm^2/m$) | z (mm) | $E_{i,y}A_{i,y}z_{i,0}^2$ x$10^9$ |
|---|---|---|---|---|---|---|---|
| **35** | 1950 MSR | 11700 | 3572917 | 72.11 x$10^9$ | 35000 | 105 | 4514.74 x$10^9$ |
| **35** | No.3 SPF | 0 | 3572917 | 0 | 0 | 70 | 0 |
| **35** | 1950 MSR | 11700 | 3572917 | 72.11 x$10^9$ | 35000 | 35 | 865.32 x$10^9$ |
| **35** | No.3 SPF | 0 | 3572917 | 0 | 0 | 0 | 0.00 |
| **35** | 1950 MSR | 11700 | 3572917 | 72.11 x$10^9$ | 35000 | 35 | 865.32 x$10^9$ |
| **35** | No.3 SPF | 0 | 3572917 | 0 | 0 | 70 | 0 |
| **35** | 1950 MSR | 11700 | 3572917 | 72.11 x$10^9$ | 35000 | 105 | 4514.74 x$10^9$ |
| **Total Compression (parallel only)** | | | | **288.4 x$10^9$** | **140.0x$10^3$** | | **10,032 x$10^9$** |

The panel properties then need to be determined for the full panel width and the specific values for buckling of panels need to be developed per O86-19 clause 8.4.5

**Panel Properties - In-plane Compression**

| | |
|---|---|
| $I_{eff} = \frac{\sum E_i I_i + \sum E_i A_i z_i^2}{E_{outer}} = \left(10321 \times 10^9 \frac{Nmm^2}{m}\right)(1.725\ m) = 1521.71\ mm^4$ | CL 8.4.5 |
| $A_{eff} = 140{,}000 \frac{mm^2}{m}(1.725\ m) = 241{,}500\ mm^2$ | CL 8.4.5 |
| $r_{eff} = \sqrt{\frac{I_{eff}}{A_{eff}}} = 78.9mm$ | CL 8.4.5 |

For bending, we can pull the panel properties from the generic PRG 320-2019 published values

TABLE A4

**LSD STIFFNESS AND UNFACTORED RESISTANCE VALUES[a] FOR BASIC CLT GRADES AND LAYUPS (FOR USE IN CANADA)**

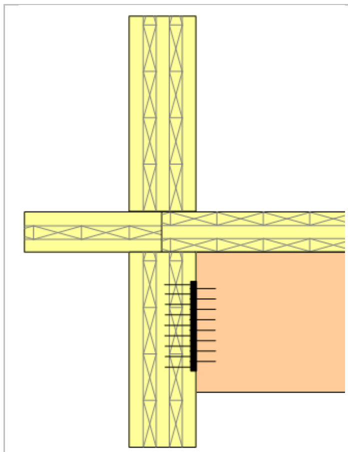
| | | Lamination Thickness (mm) in CLT Layup | | | | | | | Major Strength Direction | | | | Minor Strength Direction | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| CLT Grade | $t_p$ (mm) | = | ⊥ | = | ⊥ | = | ⊥ | = | $(f_bS)_{eff,f,0}$ ($10^6$ N-mm/m of width) | $(EI)_{eff,f,0}$ ($10^9$ N-mm²/m of width) | $(GA)_{eff,f,0}$ ($10^6$ N/m of width) | $v_{s,0}$ (kN/m of width) | $(f_bS)_{eff,f,90}$ ($10^6$ N-mm/m of width) | $(EI)_{eff,f,90}$ ($10^9$ N-mm²/m of width) | $(GA)_{eff,f,90}$ ($10^6$ N/m of width) | $v_{s,90}$ (kN/m of width) |
| E1 | 105 | 35 | 35 | 35 | | | | | 42 | 1,088 | 7.3 | 35 | 1.40 | 32 | 9.1 | 12 |
| | 175 | 35 | 35 | 35 | 35 | 35 | | | 98 | 4,166 | 15 | 58 | 12 | 837 | 18 | ~~35~~ |
| | 245 | 35 | 35 | 35 | 35 | 35 | 35 | 35 | 172 | 10,306 | 22 | 82 | 29 | 3,220 | 27 | 58 |

**Panel Properties from PRG 320-2019**

| |
|---|
| $EI_{eff} = (10306 \times 10^9\ Nmm^2/m)(1.725\ m) = 17777.9 \times 10^9\ Nmm^2$ |
| $F_bS_{eff} = \left(172 \times 10^6 \frac{Nmm}{m}\right)(1.725\ m) = 296.7\ kNm/m$ |
| $GA_{eff} = (22 \times 10^6\ Nmm^2/m)(1.725\ m) = 37.95 \times 10^6\ Nmm^2$ |

### 3.4.2 APPLIED LOADS

For wall panels supporting beams, the loading at a given floor is applied eccentrically to the centerline of the wall. For this portion of the example we will look at the walls at the interior supporting a purlin girder



| Cumulative Applied Axial Force | |
|---|---|
| $P_S = 26.8\ KN$ | roof |
| $P_D = 17.4KN$ | roof |
| $P_D = 32.3\ KN$11.8m(1.1Ka)(1.725M) | Floors |
| $P_D = 11.8$m(1.1Ka)(1.725M) = 22.4 kN | Wall (total) |
| $P_L = 2(25.2\ KN)$ | floors |
| **Typical (1.25D+1.5L+1.0S)** | **Fire (1.0D + 1.0L+1.0S)** |
| $P_f = 233\ KN$ | $P_{fire} = 182\ KN$ |
| $M_f = P_{floor}\left(\frac{th_{panel}}{2}\right)$ $= (1.25(32.3) + 1.5(25.2))\frac{245mm}{2}$ $= 9.58KNm$ | $M_f = P_{floor}\left(\frac{th_{panel}}{2}\right)$ $= (32.3 + 25.2)\frac{245mm}{2}$ $= 7.04KNm$ |

### 3.4.3 PANEL STRENGTH

Bending and Compression as well as the interaction are calculated per chapter 8 of the O86-19

**Panel Axial Strength – O86-19 CL 8.4.5**

| | | | |
|---|---|---|---|
| $K_D = K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $C_c = \frac{L_e}{\sqrt{12}r_{eff}} = \frac{4200}{\sqrt{12}(78.9mm)} = 15 < 43$ | CL 8.4.5.3 |
| $\phi = 0.8$ | CL 8.4.5.5 | | |
| $f_c = 19.3MPa$ | | | |
| $K_{zc} = 6.3\left(\sqrt{12 r_{eff} L}\right)^{-0.13} = 1.03 < 1.3$ | CL 8.4.5.5 | $P_r = \phi f_c(K_D K_{Sc} K_T K_H) A_{eff} K_{Zc} K_c$ $= 0.8(19.3\ MPa)(1.0)(241500mm^2)(1.03)(0.8)$ | CL 8.4.5.5 |

| | | | |
|---|---|---|---|
| $K_c = \left[1 + \frac{F_c K_{Zc} C_c^3}{35 E_{05} K_{SE} K_T}\right]^{-1} = 0.8$ | CL 8.4.5.5 | $= 3072.5\text{ kN}$ | $P_r > P_f$ OK |

**Bending Strength – O86-19 CL 8.4.3**

| | | | |
|---|---|---|---|
| $K_D = K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $M_r = \phi(K_D K_{Sc} K_T K_H) f_b S_{eff}'$ $= 0.9(1.0)\left(296.7\frac{KNm}{m}\right)(1.725m)$ $= 460.6\ kN$ | CL 8.4.3.1 $M_r > M_f$ OK |
| $K_{rb,0} = 0.85$ | CL 8.4.3.1 | | |
| $\phi = 0.9$ | CL 8.4.3.1 | | |
| $f_b = 28.2MPa$ | Table 8.2 | | |

**Combined Bending and Axial – O86-19 CL.8.5.6**

| | |
|---|---|
| $P_E = \frac{\pi^2 E_{05} K_{SE} K_T I_{eff}}{L_e^2} = \frac{\pi^2 (0.87 \times 11700MPa)(1.0)(1504 \times 10^6)}{(4200)^2} = 8072.4KN$ | CL 8.4.6 |
| $P_{E,v} = P_E\left(1 + \frac{k\ P_E}{GA_{eff}}\right)^{-1} = 8072.4KN\ \left(1 + \frac{1.2(8072.4)}{37.8 \times 10^6}\right)^{-1} = 6653.2\text{KN}$ | CL 8.4.6 |
| $\frac{P_f}{P_r} + \frac{M_f}{M_r}\left(1 - \frac{P_f}{P_{E,v}}\right)^{-1} = \frac{233}{3072.5} + \frac{9.6}{460.6}\left(1 - \frac{233}{6653.2}\right)^{-1} = 0.097 < 1.0$ | CL 8.4.6 OK |

### 3.4.4 FIRE DESIGN

CLT walls must be designed for fire ratings. For this example the walls will be taken as fire-separation elements, allowing us to calculate the fire-resistance rating from one face at a time. The depth of char is calculated as discussed in section 2.1.5 of this document; based on the post-fire panel layup, the section properties will need to be recalculated. To simplify the calculations, the additional laminations remaining after char that make the panel non-symmetrical in the direction of consideration can be ignored. For the primary direction, these laminations are shown in grey.

Table 3-4: CLT wall char calculation

| thickness (mm) | Lam Material | $E_{i,y}$ (Mpa) | $G_{i,y}$ (Mpa) | Centroid, X (mm) | NA – x = z (mm) |
|---|---|---|---|---|---|
| **35** | 1950 MSR | 11700 | 731 | 18 | 70 |
| **35** | No.3 SPF | 300 | 56 | 53 | 35 |
| **35** | 1950 MSR | 11700 | 731 | 88 | 0 |
| **35** | No.3 SPF | 300 | 56 | 123 | 35 |
| **35** | 1950 MSR | 11700 | 731 | 158 | 70 |
| **15** | No.3 SPF | 0 | 0 | 193 | 88 |
| **0** | 1950 MSR | 0 | 0 | | |
| | | **Total** | | | |

For Axial Properties they are calculated in the same manner previously provided.

Table 3-5: CLT Wall Panel Lamination Properties

| Thickness (mm) | Lam Material | $E_{i,y}$ (Mpa) | $I_{i,y}$ ($mm^4$/m) | $E_i I_{i,y}$ ($Nmm^2$/m) | $A_i$ ($mm^2$/m) | z (mm) | $E_{i,y} A_{i,y} z_{i,0}^2$ ($Nmm^2$/m) |
|---|---|---|---|---|---|---|---|
| **35** | 1950 MSR | 11700 | 3572917 | 72.11 x$10^9$ | 35000 | 70 | 2006.55 x$10^9$ |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **35** | No.3 SPF | 0 | 3572917 | 0 | 0 | 35 | 0 |
| **35** | 1950 MSR | 11700 | 3572917 | 72.11 x$10^9$ | 35000 | 0 | 0 |
| **35** | No.3 SPF | 0 | 3572917 | 0 | 0 | 35 | 0 |
| **35** | 1950 MSR | 11700 | 3572917 | 72.11 x$10^9$ | 35000 | 70 | 2006.55 x$10^9$ |
| **Total Compression** | | | | **216.33 x$10^9$** | **105x$10^3$** | | **4013 x$10^9$** |

| **Panel Properties - In-plane Compression (only highlighted rows)** | |
|---|---|
| $I_{eff} = \frac{\sum E_i I_i + \sum E_i A_i z_i^2}{E_{outer}} = 623.57 \times 10^6\ mm^4$ | CL 8.4.5 |
| $A_{eff} = 181{,}125\ mm^2$ | CL 8.4.5 |
| $r_{eff} = \sqrt{\frac{I_{eff}}{A_{eff}}} = 58.7mm$ | CL 8.4.5 |

In this case we can see that the remaining panel effectively represents a 5 ply panel. As such we can take the panel properties for a 5-ply panel directly from PRG 320-19 for bending properties. I cases where the depth of char results in a non-symmetrical panel it is important to calculate the neutral axis, which would not be at the center of the panel.

**Panel Properties from PRG 320-2019**

| |
|---|
| $EI_{eff} = (4166 \times 10^9\ Nmm^2/m)(1.725\ m) = 7186.4 \times 10^9\ Nmm^2$ |
| $F_b S_{eff} = \left(98 \times 10^6 \frac{Nmm}{m}\right)(1.725\ m) = 169.1\ kNm$ |
| $GA_{eff} = (15 \times 10^6\ Nmm^2/m)(1.725\ m) = 25.9 \times 10^6\ Nmm^2$ |

The wall strength is calculated using the same chapter 8 provisions as outlined previously with additional guidance from Annex B.

**Panel Axial Strength – O86-19 CL 8.4.5**

| | | | |
|---|---|---|---|
| $K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $C_c = \frac{L_e}{\sqrt{12} r_{eff}} = \frac{4200}{\sqrt{12}(58.7mm)} = 17 < 43$ | CL B.3.7 |
| $K_D = 1.15$ | CL B.3.3 | | |
| $K_{fi} = 1.25$ | CL B.3.9 | | |
| $K_{zc} = 1.03 < 1.3$ | CL B.3.5 | $K_c = \left[1 + \frac{F_c K_{Zc} C_c^3}{35 E_{ave} K_{SE} K_T}\right]^{-1} = 0.74$ | CL B.3.8 |
| $\phi = 1.0$ | CL B.3.2 | | |
| $f_c = 19.3MPa$ | | $P_{r.fi} = \phi f_c (K_D K_{Sc} K_T K_H) A_{eff} K_{Zc} K_c K_{fi}$<br>$= 1.0(19.3)(1.15)(181{,}125)(1.03)(0.74)(1.25)$<br>$= 3830.1\ kN$ | $P_{f.fi} < P_{r.fi}$<br>OK |
| | | | |
| | | | |

**Bending Strength – O86-19 CL 8.4.3**

| | | | |
|---|---|---|---|
| $K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $M_{r.fi} = \phi f_b S_{eff} K_D K_{fi} K_{rb}$<br>$= 1.0(169.1\ kNm)(1.15)(1.25)(0.85)$<br>$= 206.6\ kNm$ | CL 8.4.3.1a<br><br>$M_{f.fi} < M_{r.fi}$<br>OK |
| $K_D = 1.15$ | CL B.3.3 | | |
| $K_{fi} = 1.25$ | CL B.3.9 | | |
| $K_{rb,0} = 0.85$ | CL 8.4.3.1 | | |
| $\phi = 1.0$ | CL B.3.2 | | |
| $f_b = 28.2MPa$ | Table 8.2 | | |

**Combined Bending and Axial – O86-19 CL.8.5.6**

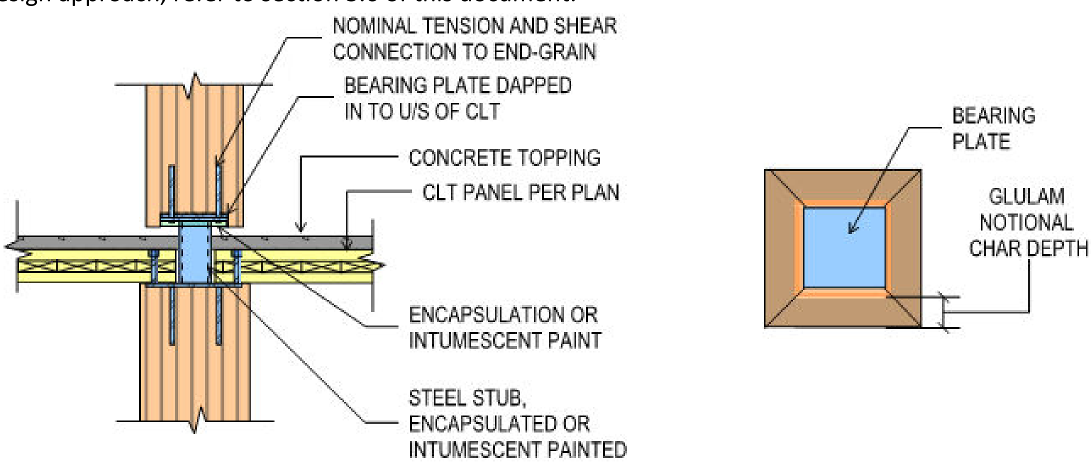
| | |
|---|---|
| $P_E = \dfrac{\pi^2 E_{05} K_{SE} K_T I_{eff}}{L_e^2} = \dfrac{\pi^2(0.87 \times 11700MPa)(1.0)(623.6 \times 10^6)}{(4200)^2} = 3551.5KN$ | CL 8.4.6 |
| $P_{E,v} = P_E\left(1 + \dfrac{k\,P_E}{GA_{eff}}\right)^{-1} = 8072.4KN\left(1 + \dfrac{1.2(8072.4)}{25.9 \times 10^6}\right)^{-1} = 3049.68\text{KN}$ | CL 8.4.6 |
| $\dfrac{P_f}{P_r} + \dfrac{M_f}{M_r}\left(1 - \dfrac{P_f}{P_{E,v}}\right)^{-1} = \dfrac{182}{3830.1} + \dfrac{7.04}{206.6}\left(1 - \dfrac{182}{3049.7}\right)^{-1} = 0.084 < 1.0$ | CL 8.4.6<br>OK |

## 3.5 Connections

Per O86-19 Annex B Clause 9, all connections supporting elements with a fire rating must have a fire rating meeting or exceeding that provided for the primary loading-bearing elements, referencing the AWC's Technical Report No.10 for more information. The Technical report notes that steel elements completely encased in wood that is not within the char depth or zero strength layer can be considered fire protected.

### 3.5.1 COLUMN-TO-COLUMN

It is good practice to maintain end-grain to end-grain connections for column-to-column connections between floors to avoiding high perp-to-grain forces resulting from cumulative column loads. This approach will also minimize perpendicular to grain shrinkage between erection and the final in-situ condition. Shrinkage is also minimized using this design approach, refer to section 3.6 of this document.




To achieve a fire rated solution, the steel column connector can be intumescent painted on the exposed surfaces or encased in gypsum. The plates should be held back the depth on char on all 4 sides of the columns to protect the sides. The end-grain axial capacity of the columns also needs to be checked.

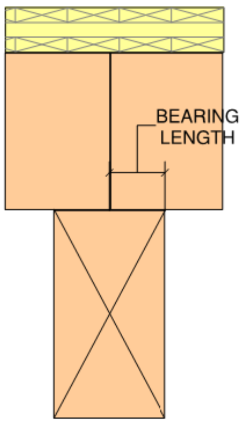
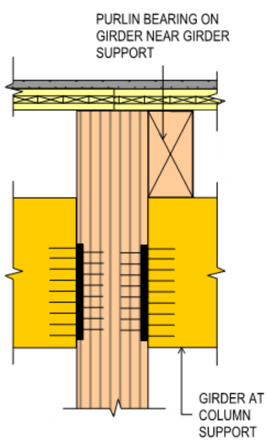
**Axial End Connection Compression Strength - CSA O86-19 CL 7.5.8**

| | | | |
|---|---|---|---|
| $K_H = K_{sc} = K_T = K_D = 1.0$ | CL 8.3 | $P_r = \phi\, f_c(K_D K_H K_{sc} K_T) A_b$<br>$= 0.8(25.2)(1.0)(215 \times 240) = 1040\ KN$ | Cl 7.5.8.5<br>$P_f < P_r$<br>OK |
| $\phi = 0.8$ | CL 7.5.8.5 | | |
| $f_b = 25.6MPa$ | Table8.2 | | |
| $K_C = 1.0$ | Cl 7.5.8.6 | | |
| $K_{Zcg} = 1.0$ | CL 7.5.8.5 | | |

### 3.5.2 PURLIN-TO GIRDER

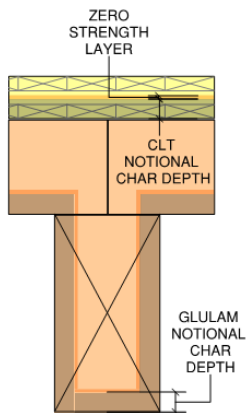
The purlin to girder connections are bearing connections; the bearing design is completed to support the reactions determined in the purlin design. The bearing on the top of the beam for purlins adjacent to the girder support must also be checked. Finally, the bearing condition in the fire case must also be reviewed.

**Compression Perpendicular to grain – CSA O86-19 CL 7.5.9**

<table>
<tr><td rowspan="12">
</td><td colspan="4">Applied Bearing Load on Purlins</td></tr>
<tr><td colspan="2">Typical (1.25D+1.5L+1.0S)</td><td colspan="2">Fire (1.0D + 1.0L+1.0S)</td></tr>
<tr><td colspan="2">$Q_f = 89.3\ kN$</td><td colspan="2">$Q_{f.fi} = 60.8\ kN$</td></tr>
<tr><td colspan="2"></td><td colspan="2"></td></tr>
<tr><td colspan="4">Purlin Support Bearing Strength – CSA O86-19 CL. 7.6.9.2</td></tr>
<tr><td>$F_{cp} = 5.8\ MPa$</td><td>Table 7.2</td><td rowspan="2">$A_b = 365\ \times \frac{265}{2} = 48363mm^2$</td><td rowspan="2"></td></tr>
<tr><td>$K_D = 1.0$</td><td></td></tr>
<tr><td>$K_{scp} = 1.0$</td><td></td><td rowspan="5">$Q_r = \phi f_{cp}\left(K_D K_{scp} K_T\right) A_b K_b K_{Zcp}$<br>$= 0.8(5.8MPa)(48363)(1.0)(1.15)$<br>$= 258.1\ kN$</td><td rowspan="5">$Q_f < Q_r$<br>OK</td></tr>
<tr><td>$K_T =\ 1.0$</td><td></td></tr>
<tr><td>$\phi = 0.8$</td><td></td></tr>
<tr><td>$K_B = 1.0$</td><td>CL 6.5.6.5</td></tr>
<tr><td>$K_{Zcp} = 1.15$</td><td>CL 6.5.6.4</td></tr>
<tr><td rowspan="11">
</td><td colspan="4">Applied Bearing Load on Girders</td></tr>
<tr><td colspan="2">Typical (1.25D+1.5L+1.0S)</td><td colspan="2">Fire (1.0D + 1.0L+1.0S)</td></tr>
<tr><td colspan="2">$V_F = 2\left(Q_{f.purlin}\right) = 2(89.3\ kN)$<br>$= 178.6\ kN$</td><td colspan="2">$V_F = 2\left(Q_{f.fi.purlin}\right) = 2(60.8\ kN)$<br>$= 121.6\ kN$</td></tr>
<tr><td colspan="4">Bearing on Girder near Support – CSA O86-19 CL. 7.6.9.3</td></tr>
<tr><td>$F_{cp} = 5.8\ MPa$</td><td>Table 7.2</td><td rowspan="2">$A_b = 365\ \times 265 = 92725mm^2$</td><td rowspan="2"></td></tr>
<tr><td>$K_D = 1.0$</td><td></td></tr>
<tr><td>$K_{scp} = 1.0$</td><td></td><td rowspan="5">$Q_r = \phi f_{cp}\left(K_D K_{scp} K_T\right) A_b K_b K_{Zcp}$<br>$= 0.8(5.8MPa)(1.0)(92725)(1.0)(1.15)$<br>$= 344.1\ kN$</td><td rowspan="5">$Q_f < Q_r$<br>OK</td></tr>
<tr><td>$K_T =\ 1.0$</td><td></td></tr>
<tr><td>$\phi = 0.8$</td><td></td></tr>
<tr><td>$K_B = 1.0$</td><td>CL 6.5.6.5</td></tr>
<tr><td>$K_{Zcp} = 1.15$</td><td>CL 6.5.6.4</td></tr>
</table>

The bearing area for fire

**Compression Perpendicular to grain – Fire Case – CSA O86-19 CL 7.5.9**

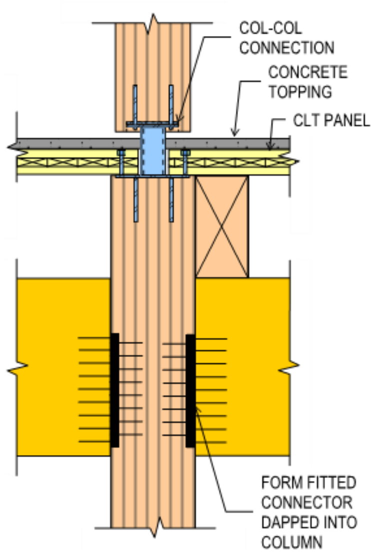


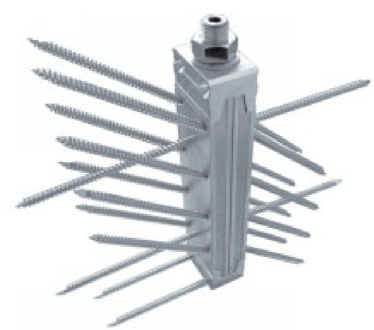

| Applied Bearing Load on Purlins | |
|---|---|
| **Typical (1.25D+1.5L+1.0S)** | **Fire (1.0D + 1.0L+1.0S)** |
| $Q_f = 89.3\ kN$ | $Q_{f.fi} = 60.8\ kN$ |
| | |

| Purlin Support Bearing Strength – CSA O86-19 CL. 7.6.9.2 | | |
|---|---|---|
| $F_{cp} = 5.8\ MPa$ | | $A_b = 267 \times \dfrac{167}{2} = 22294.5mm^2$ |
| $K_{scp} = 1.0$ | | |
| $K_T = 1.0$ | | $Q_r = \phi f_{cp}(K_D K_{scp} K_T) K_{fi} A_b K_b K_{Zcp}$ |
| $K_D = 1.15$ | | $= 1.0(5.8MPa)(1.15)(1.35)(22294.5)(1.15)(1.0)$ |
| $K_{fi} = 1.35$ | | $= 230.9\ KN$ |
| $\phi = 1.0$ | | |
| $K_B = 1.0$ | | $Q_{f.fi} < Q_{r.fi}$ |
| $K_{zcp} = 1.15$ | | OK |

### 3.5.3 GIRDERS TO COLUMNS

To maintain the column-column connections as previously shown, the girder to column connections must be provided into the face of the columns, without any exposed steel to ensure the fire rating is achieved. To achieve this, proprietary form fitted connectors can be used. In this example we will use the Megant connectors provided by MTC Solutions.



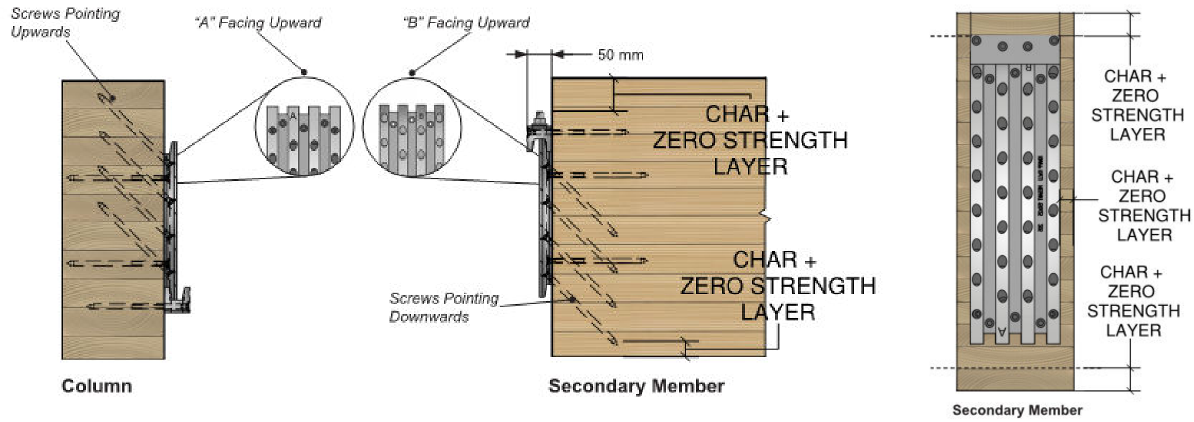



**MEGANT System**

| Reaction Loads | |
|---|---|
| **Typical (1.25D+1.5L+1.0S)** | **Fire (1.0D + 1.0L+1.0S)** |
| $R_f = 273.8\ KN$ | $R_{f.fire} = 205.2\ KN$ |

| Item # | Min. Beam Size [mm] | Relative Density [G] | Fasteners | | Threaded Rod | Factored Resistance, $N_r$ Down Load | | Uplift |
|---|---|---|---|---|---|---|---|---|
| | | | Type | Quantity | | kN | [lbs] | |
| MEGANT 730 x 150 170707301500200 | 190 x 840 | 0.42 (SPF) | VG CSK 8 x 160 | 104 | 3 pcs of M20 x 760 Grade 8.8 | 293 | [65,860] | See uplift design p. 51 |
| | | 0.49 (D.Fir) | VG CSK 8 x 160 | 104 | 3 pcs of M20 x 760 Grade 8.8 | 318 | [71,480] | |

Per O86-19 Annex B Clause 9, all connections supporting elements with a fire rating must meet or exceed the fire rating of the supported element. The AWC's Technical Report No.10 notes that steel elements completely encased in wood not within the char depth or zero strength layer can be considered fire protected. By maintaining enough timber around the Megant connector, we can ensure that the full strength of the connection is maintained in the fire case




MTC solutions has provided minimum beam sizes to maintain specific char ratings in the beam design guide

*Table 24.3 Suggested Cross Sections*

| Connector | Fire Resistance Rating | | | | | |
|---|---|---|---|---|---|---|
| | 1 hour | | | 2 hours | | |
| | Min. Beam Width (b) [mm] | Min. Beam Height (h) [mm] | $a_{sec}$ [mm] | Min. Beam Width (b) [mm] | Min. Beam Height (h) [mm] | $a_{sec}$ [mm] |
| **MEGANT 730x150** | 224 | 810 | 50 | 286 | 851 | 91 |

Notes:

1. All minimum beam requirements account for the corner effect rounding when beams are designed for three-sided fire exposure.
2. Beam Hanger Systems must be installed with fire rated caulking within the non charring area.

## 3.6 Shrinkage

O86-19 Annex A provides guidance on shrinkage provided that the moisture contents of wood can be determined in the initial and final condition. This is discussed in clause A.5.4.6.

$C_{perp} = 0.002$ $\quad S_{perp} = (d \text{ or } b)(M_{initial} - M_{final})c_{perp}$ $\quad$ CL A.5.4.6

$C_{para} = 0.00005$ $\quad S_{perp} = (d \text{ or } b)(15\% - 8\%)(0.002) = 1.4\%(d)$

$M_{initial} = 12\% \pm 3\%$ $\quad S_{para} = (L)(M_{initial} - M_{final})c_{perp}$

$M_{final} = 8\%$ (indoor) $\quad S_{para} = (L)(15\% - 8\%)(0.00005) = 0.035\%(L)$

From this we can determine the total expected shrinkage of the members and the building

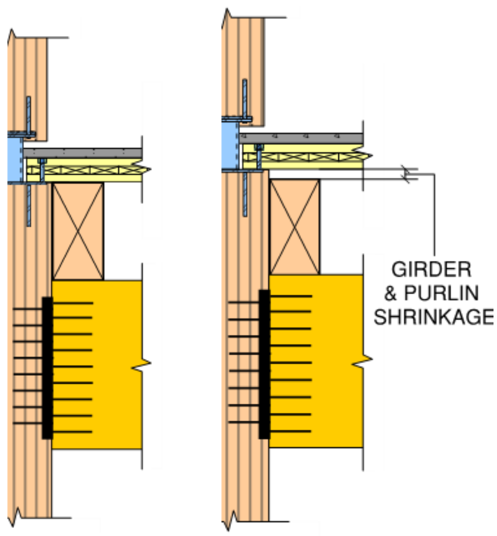
| Purlin | Girder | Column | CLT Floor |
|---|---|---|---|

| Member Dim | Dim. | Shrinkage | Dim. | Shrinkage | Dim. | Shrinkage | Dim. | Shrinkage |
|---|---|---|---|---|---|---|---|---|
| **B** | 365 | 5.11mm | 265 | 3.71mm | 315 | 4.41mm | 105mm | 1.47mm |
| **D** | 380 | 5.32mm | 876 | 12.26mm | 342 | 4.80mm | | |
| **L** | 7260 | | 9144 | | 4250 | 1.49mm | | |

It is important to look at locations of differential shrinkage to determine if there are any issues to reviewed.

### 3.6.1 FLOOR PLATE DIFFERENTIAL SHRINKAGE

Differential shrinkage within a floor plate can impose unintended forces in members or connections. Where purlins are supported on girders adjacent to columns is one example where this can occur.




Initial Condition Final Condition

Girders: Supported at their mid depth, but partially restrained again shrinkage due to screws in proprietary connection. As an estimate assume the top third will shrink down, and the bottom third will shrink upwards with the mid-depth of beam staying constant

Purlins: The purlins are supported in bearing at the base. They will shrink downward to maintain the bearing support

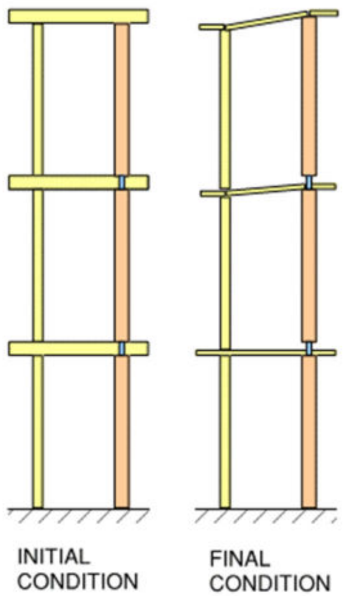
CLT: The CLT typically bears on the purlins, but at the columns it is framed into the top of the columns. the location of the bottom of the CLT at the column will be set by the column shrinkage

The total differential shrinkage between the u/s of the CLT and the top of the purlin would be as follows:

$$\Delta_{purlin} + \frac{1}{3}\Delta_{girder} = 5.32mm + \frac{1}{3}12.26mm = 9.41mm$$

The CLT will either need to slip around the column to accommodate this or accommodate lack of support by the purlins until both the purlins can deflect up, and the CLT can deflect down to meet each other.

### 3.6.2 DIFFERENTIAL SHRINKAGE OVER FULL BUILDING HEIGHT




Differential shrinkage over the height of the building can similarly impart unintended forces in members or result in unintended slopes or steps in floor plates.

In this example, one location of potential concern is at the columns adjacent to platform framed CLT walls. The columns are detailed to avoid perp-to-grain shrinkage, but the CLT walls panels are platform frames. An exaggeration of the shrinkage in the CLT floor plates clearly shows how the cumulative shrinkage in the perp-to-grain connections at the CLT walls that is not present in the columns.

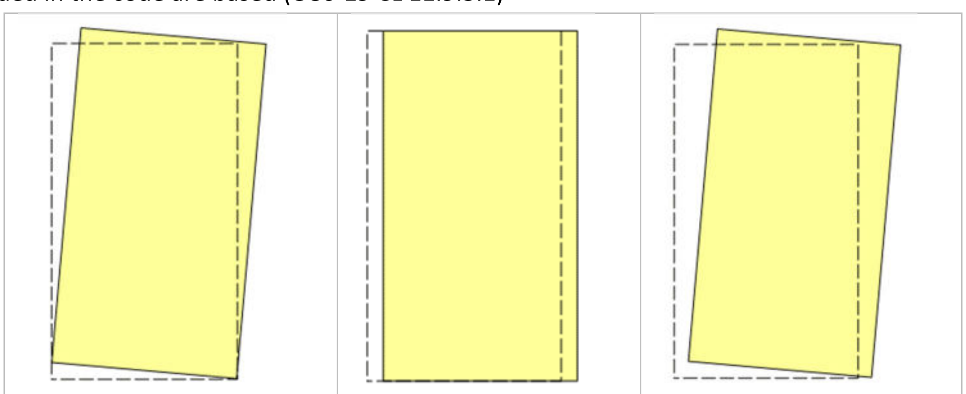
The shrinkage of the CLT floor at one level is limited to 1.47mm, but over 3 levels of CLT this will result in 4.4mm. On this three-storey building this does not constitute a large problem, but as the number of storeys increases, this can add up to detrimental cumulative shrinkage.

# 4 Lateral Design

A sample calculation is provided for the shearwall wall along grid C at the elevator core; first the loading on the wall needs to be established and basic requirements of CLT shearwalls must be outlined.

## 4.1 CLT Shearwall Concept

The in-plane strength of a CLT Shearwall is generally governed by its connections as opposed to in-plane shear or bending resistance of the CLT panels; similarly, the deformation in a CLT shearwall is generally dominated by the deformation in the connectors. Most codes and design guides note that the panels themselves can be modeled as rigid elements with the connections being the focus of design. For CLT shearwalls resisting seismic loads, ductility is typically developed by yielding the connections in a rocking wall system. This is the basis on which the ductility factors provided in the code are based (O86-19 CL 11.9.3.1)



| | | |
|---|---|---|
| Rocking Only - OK | Sliding Only – Not OK | Rocking & Sliding - OK |

For walls that do not achieve the desired behaviour, reduced ductility factors are provided, effectively designing for a fully elastic system

## 4.2 Code and Standard Limitations

CSA O86-19 Section 11.9 provides guidance on Shearwall Design corresponding to the upcoming provisions to be included in the NBCC 2020. There are several limitations to the provisions in the standard as a variety of requirements that must be met to provide a code compliant design:

| | |
|---|---|
| CLT shearwall provisions are limited to platform framed walls<br>i.e. multi-storey continuous panels are not premitted | CL 11.9.1.1 |
| A building using CLT shearwalls is required to avoid irregularities | CL 11.9.3.2.1 |
| Height limits below 20m or 30m depending on seismic category | CL 11.9.3.2.3 |
| Shearwall panels must be a minimum of 87mm thick | CL 11.9.3.5.1 |
| Shearwall panels with openings must be excluded from shearwalls | CL 11.9.3.5.4 |
| Shearwall flexural deformation should not exceed 30% of total drift<br>i.e. lateral deformations associated with panel flexural deformation and hold-down elongation at each floor shall not exceed 30% of the total drift | CL 11.9.3.6.2 |

The O86-19 standard provide ductility (Rd=2.0) and Overstrength (Ro=1.5) values for ductile design. To achieve these ductility values additional requirements and limitations are provided. If these limitations are not met, then a reduced ductility and overstrength value (RdRo = 1.3) must be used.

| | |
|---|---|
| Shearwall segments must have aspect ratios between 2H:1W and 4H:1W | CL 11.9.3.5.2, |
| Dissipative connections are designed so that a yielding mode governs | CL 11.9.3.3.1a |
| Dissipative connections are moderately ductile in directions associated with panel rocking | CL 11.9.3.3.1b |
| All Connections allow enough deformation capacity at displacement demands induced from rocking | CL 11.9.3.3.1c |

## 4.3 Energy Dissipating & Capacity Protected Elements

O86-19 gives some specific guidance on elements that must be capacity protected:

**Energy Dissipating Elements**

| | |
|---|---|
| Vertical joints between CLT panels/segments | CL 11.9.3.4.1a |
| Shear connections at base of walls in uplift only | CL.11.9.3.4.1b |
| Discrete hold-downs<br>hold-down must have a strength 20% greater than the forces developed with the vertical joints reach their design (ie. yielding) strength of the between panels | CL11.9.3.3.3.2 |

**Capacity Protected Elements**

| | |
|---|---|
| CLT panels in walls & diaphragms | CL 11.9.3.4.2 / CL 11.9.3.7.1 |
| Continuous Steel Rod Hold-downs | CL 11.9.3.3.3.3 |
| Diaphragms chords & struts/drags (including around openings) | CL 11.9.3.7.3 |

Note: Capacity protect elements are designed based on the probable capacity of the yielding element. For walls with vertical joints, these can be taken as the primary yielding element for which the probable capacity should be based.

Additionally, the Commentary clauses 11.9.2.4 and 11.9.2.4.2 goes on to state that "non-dissipative connection should not yield while the energy dissipative connections reach their maximum expected resistance…tolerate the displacement demands when the dissipative connections reach their target displacements". If this approach is used, there are several additional elements that should be capacity protected:

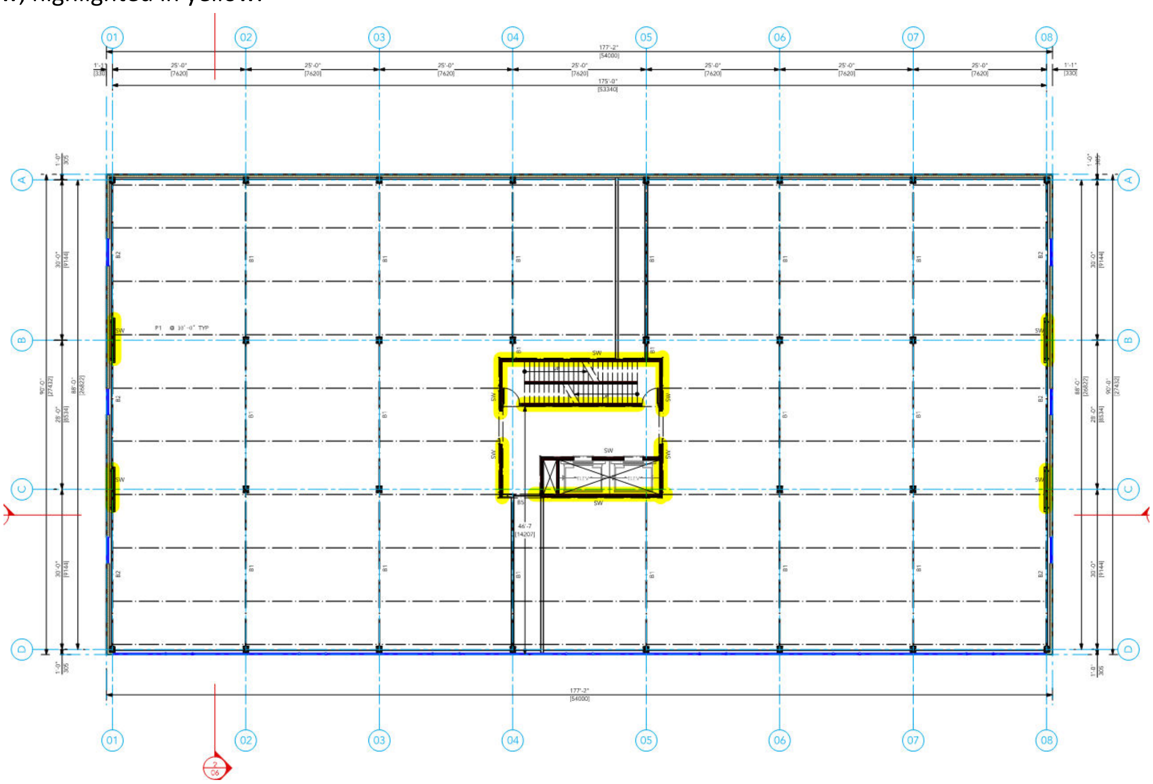
**Additional Capacity Protect Elements recommended in Commentary**

- Components of hold-downs not intended to dissipate energy
- Shear connections at base of wall in shear direction
- Splices between floor panels
- Connections between the floor and the wall below
- Bearing resistance of CLT panels supporting Shearwalls above
- Entire Concrete podium where applicable

Note: foundations would also be capacity protected. They should be designed for the worse of the capacity protected requirements as discussed here, as well as additional capacity protection requirements outlined in the NBCC. They are not discussed further herein

## 4.4 Lateral Force Resisting System (LFRS)

The LFRS used in this building consists of CLT shearwalls in both directions. These shearwalls are shown in the plan below, highlighted in yellow.




The building in question does not have any irregularities and is shorter than 20m, making it suitable for a CLT shearwall LFRS. For this example, we will review the design for the shearwall along grid C. Note that there are no provisions in the NBCC requiring elements of the LFRS that are not also part of the gravity load resisting system to be fire rated.

### 4.4.1 LOAD DISTRIBUTION

As with any building, the distribution of lateral forces to discrete LFRS elements (i.e. individual shearwalls in this example) is dependent on the relative stiffnesses of each vertical LFRS and the stiffness of the diaphragm that distributes the load. CLT diaphragms fall somewhere between a fully flexible (like plywood diaphragms) and a puely rigid (like concrete diaphragms); the drift associated with the CLT diaphragm will likely be in the same order of magnitude as the CLT shearwall drift at a single level. An enveloped approach to load distributions is a simplified approach to determine the load carried by each wall. This example will use a flexible and rigid diaphragm as the bounds of the envelope.

#### 4.4.1.1 Flexible Diaphragm Distribution

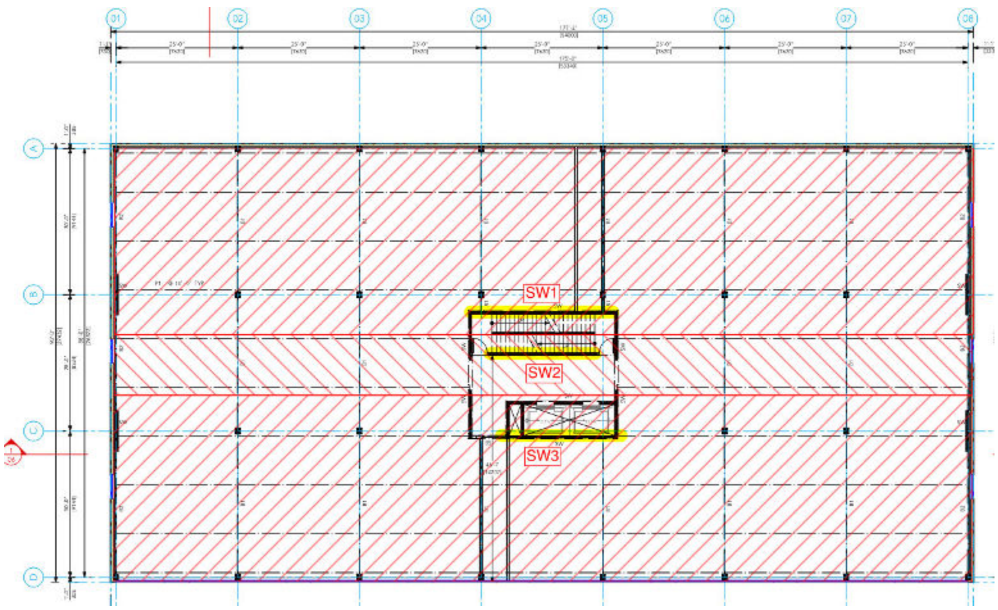



Table 4-1: Shear Load Distribtion between Shearwalls – Flexible Diaphragm Distribution

| Wall Element | Tributary Width (m) | Tributary Length (m) | Tributary Area (m²) | $\frac{V_{wall}}{V_{total}} = V\left(\frac{A_{trib.i}}{\sum A_{trib.i}}\right)$ | |
|---|---|---|---|---|---|
| **SW1** | 12.0 | 54 | 647.4 | 0.45 | GOVERNS |
| **SW2** | 4.3 | 54 | 234.5 | 0.16 | |
| **SW3** | 10.1 | 54 | 547.3 | 0.38 | GOVERNS |
| | | **Total** | **1429.2** | | |

#### 4.4.1.2 Rigid Diaphragm Distribution

If we assume the stiffness of the walls are directly proportional with their length, a common simplified approach for shearwalls suitable for hand calculations, we can distribute the loads based on their location from the center of mass (C.M.) and the center of rigidity (C.R.), and their lengths. Note that C.M. for this building is approximated to occur at the center of the floor plate. For simplicity, the C.R. is approximated as occurring at the same locations as the C.M.

Table 4-2: Shear Load Distribution between Shearwalls – Rigid Diaphragm Distribution

| Wall | Wall Height (m) | Wall Length (m) | Wall Stiffness (K) (KN/m) $E \times th\left(\left(\frac{H}{L}\right)^3 + 3\frac{H}{L}\right)^{-1}$ | Wall Participation $\frac{V_{wall}}{V_{total}} = V\left(\frac{K_x}{\sum K_x}\right)$ | |
|---|---|---|---|---|---|
| **SW1** | 3.8m | 9.1 | 750 (E)(th) | 0.40 | |
| **SW2** | 3.8m | 6.9 | 544 (E)(th) | 0.30 | GOVERNS |
| **SW3** | 3.8m | 6.9 | 544 (E)(th) | 0.30 | |
| | | Total | 1900 (E)(th) | | |

## 4.5 Initial Shearwall Design – SW3

This section outlines the initial shearwall design of a specific shearwalls as a means of illustrating the design process that needs to occur for all walls. For the shearwall wall along grid C (SW3) at the elevator core the flexible diaphragm approach is the governing load distribution for the wall in question.

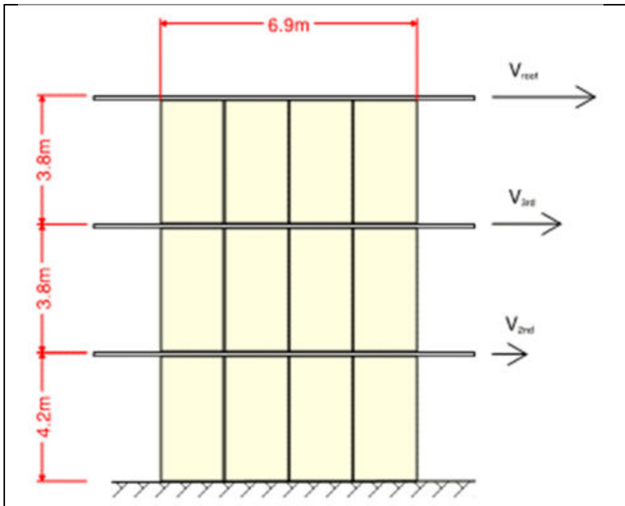


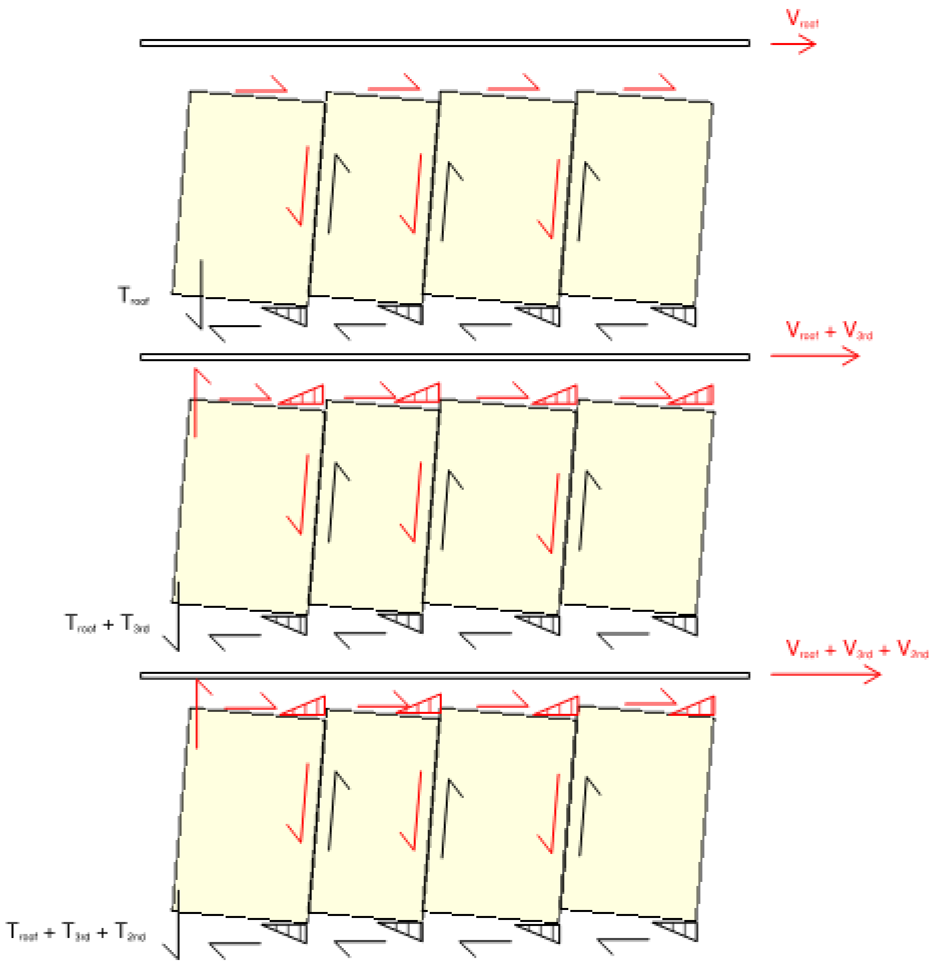

Table 4-3: Storey Shears for SW3

| Level | Storey Shear (KN) | Cumulative Storey Shear (KN) | Wall Shear Load (KN) |
|---|---|---|---|
| **Roof** | 611 | 611 | 232.2 |
| **Level 3** | 579 | 1190 | 452.2 |
| **Level 2** | 307 | 1497 | 568.9 |

The wall is 6.9m long, to meet the aspect ratio requirements, the wall panels must be not more than 1.9m wide at the upper floors; the wall is divided into 4 equal panels. The panels will be designed with dissipative elements included:

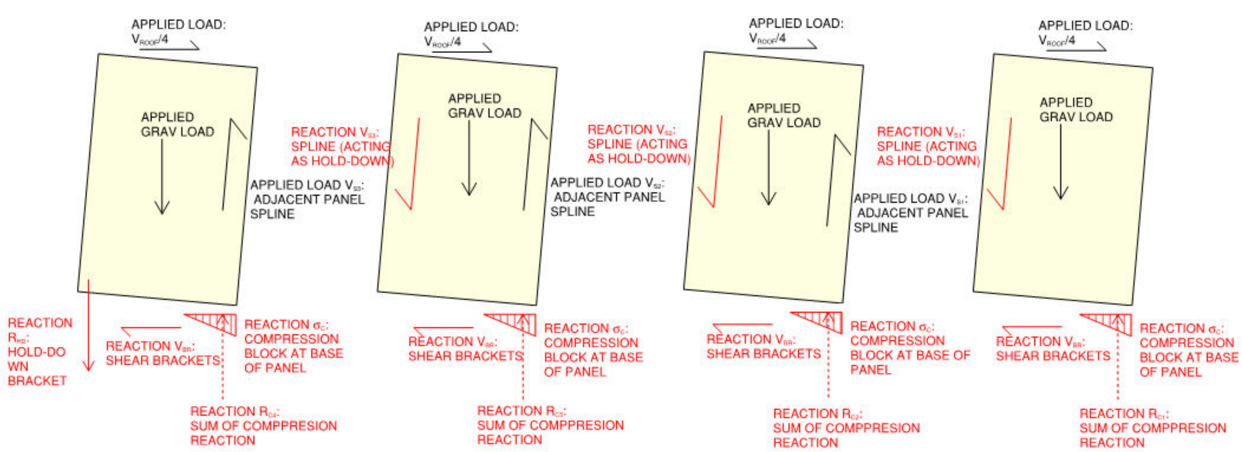
1. Vertical splines between panels
2. Hold-downs
3. Base Shear Connections

A Schematic exploded free-body diagram of the shearwalls shows the compression at the end of each panel, the cumulative hold-down forces, and the shear between individual panels. The exact load distribution in each spline and the hold-downs is an indeterminate problem. This example includes a simplified approach to the loading and design of a CLT shearwall.




## 4.5.1 INITIAL FORCE DISTRIBUTION IN SHEARWALL

The first step in designing a rocking CLT shearwall involves create free-body diaphragms for each panel and distribute the shear load as a line load along each panel. Starting with a simplified assumption of a compression reaction occurring at the corner, the sum of the moments about the corner of the panel at the compression side to meet equilibrium as follows (ΣM= 0). Similarly, if we assume that the spline load applies along the same line as the bearing, we can simplify the loading as follows:



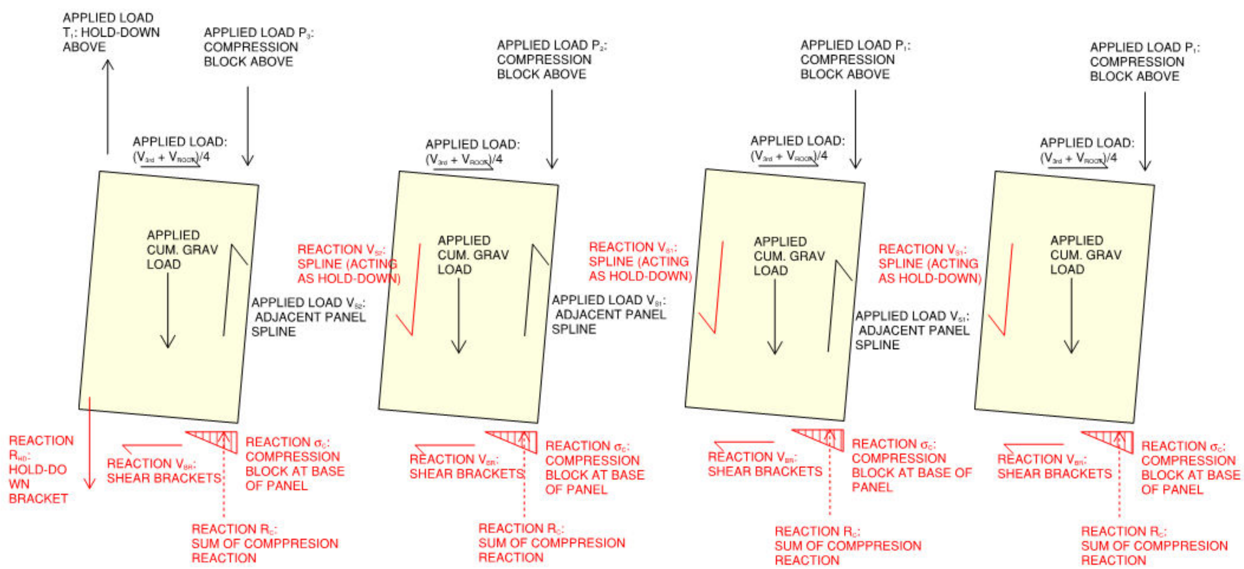



Table 4-4: Panel Loads for "n" panels

| | **Acting Loads x Moment Arm** | **Resisting Loads x Moment Are** |
|---|---|---|
| **Panel 1 to Panel n-1** | $V_{f.i}(H_{panel.i})$ | $V_{spline.i}(L_{panel,i}) + W_{panel.i}\left(\frac{L_{panel,i}}{2}\right)$ |
| **Panel n** | $V_{f.4}(H_{panel.4})$ | $T_{HD}(L_{panel,i}) + W_{panel.4}\left(\frac{L_{panel,i}}{2}\right)$ |

From this we can establish that for panels of equal width, carrying equal load, the loads in the spline and the load in the hold-down will all be roughly equal. Note that this is determined based on the elastic response of the connectors.

$$V_{f.spline}(L_{panel}) = V_{f.panel}(H_{Panel}) - W_{panel}\left(\frac{L_{panel}}{2}\right)$$

$$v_{f.spline}(L_{Panel})(H_{Panel}) = v_{f.panel}(L_{Panel})(H_{Panel}) - w_{panel}L_{panel}\left(\frac{L_{panel}}{2}\right)$$

$$v_{f.spline} = \left[v_{f.panel}(H_{Panel}) - \left(\frac{W_{panel}L_{Panel}}{2}\right)\right]\left[v_{f.panel}(H_{Panel}) - \left(\frac{W_{panel}L_{Panel}}{2}\right)\right]/H_{panel}$$

For hold-downs acting at the end of a panel, the loads carried will be effectively the same as the spline load. Lastly, it should be noted that the spline loads is dependant only on the shear resisted by the shearwall at that level; the compression bearing at each panel and tension in the hold-down must account for the cumulative overturning in addition to the shears in the wall in question.

$$T_{f.hold-down} = v_{spline}(H_{panel})$$

$$T_{f.hold-down} = v_{spline}(H_{panel}) + T_{f.hold-down.above}$$

Compression loads at the ends of panels can be determined using the same engineering principles of equilibrium by summing the forces in the vertical direction.

$$P_{f.panel} = W_{panel.i} + V_{spline}$$

$$P_{f.panel} = W_{panel,i} + \left[V_{f.panel}(H_{Panel}) - W_{panel}\left(\frac{L_{panel}}{2}\right)\right]$$

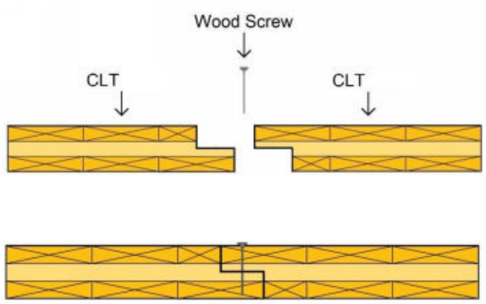
$$P_{f.panel.end} = W_{panel}\left(1 - \frac{L_{panel}}{2}\right) + V_{f.panel}(H_{Panel}) + P_{f.panel\ above.end}$$

Table 4-5: Shearwall System and Component Loading

| Level | Floor Height | Panel Length | Gravity Load | Wall Shear Load | Wall Shear Load | Spline Load | | Hold-down Load | Panel end Axial Load |
|---|---|---|---|---|---|---|---|---|---|
| **(kN)** | **(m)** | **(m)** | **(kN/m)** | **(kN)** | **(kN/m)** | **(kN/m)** | **(kN)** | **(kN)** | **(kN)** |
| **Level 3** | 3.8 | 1.725 | 7.3 | 232 | 33.6 | 32.0 | 122 | 122 | 134 |
| **Level 2** | 3.8 | 1.725 | 16.0 | 452 | 65.5 | 61.9 | 235 | 357 | 397 |
| **Ground** | 4.2 | 1.725 | 25.1 | 569 | 82.4 | 77.3 | 325 | 681 | 765 |

As you can see, the cumulative spline loads down the height of the building are equal the calculated hold-down loads.

## 4.5.2 SPLINE DESIGN



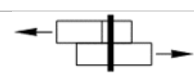
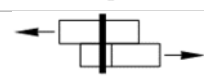
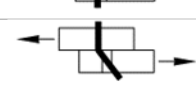
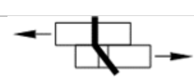

A lapped CLT spline with Proprietary self-tapping (STS) screws between the wall panels is proposed for these shear walls. There are numerous suppliers for these screws and the specific design values for the screws should be used for the design. (reference commentary for STS design per lag screws). The design is completed based on O86-19 section 12.6 for lag screws. Note that clause 12.6.2.1 notes that the grain orientation for CLT should be considered the grain orientation of the face laminations.

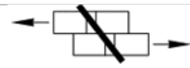
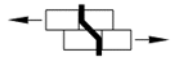
**Bolt Design Properties – CSA O86-19 CL 12.6.5.1.2**

| **10Ø x 240 STS screw** | **Side Member - CLT** | **Main Member - CLT** |
|---|---|---|
| d = 10mm | t1 = 245/2 = 122mm | t2 = 240-122 = 118mm |
| $d_{shank}$ = 7.2mm | G = 0.42 | G = 0.42 |
| $f_{y.screw}$ = 1000 MPa | Parallel to grain | Parallel to grain |
| | $f_2 = 50G(1-0.01d_s)J_x$<br>$= 50(0.42)\big(1-0.01(7.2mm)\big)(0.9)$<br>$= 17.5\ MPa$ | $f_2 = 50G(1-0.01d_s)J_x$<br>$= 50(0.42)\big(1-0.01(7.2mm)\big)(0.9)$<br>$= 17.5\ MPa$ |

The O86-19 commentary clause 11.9.2.2 provides guidance on approaches for moderately ductile connections stating that ductility can be met "…where fasteners develop a single or two plastic hinges…". Our lap spline connection will use screws chosen to develop two plastic hinges in the connector.

**Fastener Unit Lateral Strength Calculation - CSA O86-19 CL 12.6.5.1.2**

| | | | | |
|---|---|---|---|---|
| a) | $f_1 d_F t_1$ |  | 15.47 KN/screw | |
| b) | $f_2 d_F t_2$ |  | 14.84 KN/screw | |
| c) | $f_1 d_F^2\left(\sqrt{\frac{1}{6}\frac{f_2}{(f_1+f_2)}\frac{f_y}{f_1}}+\frac{1}{5}\frac{t_1}{d_F}\right)$ |  | 5.08 KN/screw | |
| d) | $f_1 d_F^2\left(\sqrt{\frac{1}{6}\frac{f_2}{(f_1+f_2)}\frac{f_y}{f_1}}+\frac{1}{5}\frac{t_2}{d_F}\right)$ |  | 4.95 KN/screw | |

| | | | | |
|---|---|---|---|---|
| e) | $f_1 d_F^2 \frac{1}{5}\left(\frac{t_1}{d_F}+\frac{f_2}{f_1}\frac{t_2}{d_F}\right)$ |  | 6.06 KN/screw | |
| f) | $f_1 d_F^2 \sqrt{\frac{2}{3}\frac{f_2}{(f_1+f_2)}\frac{f_y}{f_1}}$ |  | 3.96 KN/screw | GOVERNS |

$$Nr \;=\; \phi\, n_u\, K_D K_{st} K_T = 0.6\left(3.96\frac{KN}{screw}\right)(1.15)(1.0)(1.0) = 2.73\frac{KN}{screw}$$

After determining the strength per screw, we can determine the necessary screw spacing and resulting spline strength.

Table 4-6: Spline Screw strength and Spacing

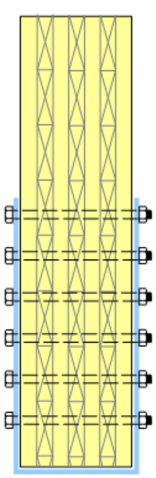
| Level | Panel Height (m) | Spline Load (KN/m) | Screw Spacing (mm) | Spline Strength (KN/m) |
|---|---|---|---|---|
| **Level 3** | 3.8 | 32.1 | 1 row @ 75 | 36.4 |
| **Level 2** | 3.8 | 62.7 | 1 row @ 40 | 68.3 |
| **Ground** | 4.25 | 78.5 | 2 rows @ 65 | 84.0 |

### 4.5.3 HOLD-DOWN DESIGN

The hold-downs must resist relatively high loads; proprietary hold-downs are not suitable to withstand the design loads. The proposed custom hold-down uses steel bolts with steel side plates. The guidance provided in the O86-19 commentary clause 11.9.2.2 notes that "mild steel dowel-type fasteners such as bolts or dowels driven perpendicular to the face of the CLT panel, that use inserted or side steel plates, and fail in fastener yielding modes (d), (e), or (g) as specified in Clause 12.4.4.3 can be considered moderately ductile and have a ductility ratio of 3.0 or more if the fasteners have a slenderness ration t/df ≥ 10. In the ratio, t is the CLT member thickness for the three member connections where the fasteners develop one-or more plastic hinges per shear plate…in all cases the fastener diameter should be 19mm or less".

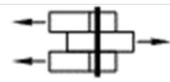
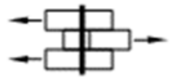
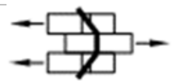
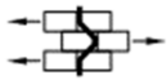
Bolt Diameter limit t2/10 = 24.5mm > 19mm

**Bolt Design Properties – CSA O86-19 CL 12.4.4.3.3**

| | | | |
|---|---|---|---|
|  | 19Ø ASTM A307 bolts<br>d= 19mm | $t_2/d \;=\; 245/19 = 12.9 > 10$ | OK |
| | $f_y \;=\; 310\ Mpa$ | | |
| | $f_u \;=\; 400\ MPa$ | | |
| | $f_{y-design} = average\ (fy, fu) = 355\ MPa$ | | |
| | **Side Member – 300W** | **Main Member - CLT** | |
| | t1 = 4.8mm | t2 = 245mm | |
| | fu = 450 MPa | G = 0.42 | |
| | Steel side plate | Parallel to grain | |
| | $f_{steel} \;=\; K_{sp}\frac{\phi_s}{\phi_{wy}} f_u \;=\; 1350\ MPa$ | $f_2 = \; 50G(1-0.01d_s)J_x$<br>$= 50(0.42)\big(1-0.01(7.2mm)\big)(0.9)$<br>$=\; 17.5\ MPa$ | |

The design is completed based on O86-19 section 12.4 for bolts. Note that clause 12.4.3.1 notes that the grain orientation for CLT should be considered based on the orientation of the load relative to the face grain of the CLT.

**Fastener Unit Lateral Strength Calculation - CSA O86-19 CL 12.4.4.3.2**

| | | | | |
|---|---|---|---|---|
| a) | $f_1 d_F t_1$ |  | 123.44 KN/shear plane / bolt | |
| c) | $\frac{1}{2} f_2 d_F t_2$ |  | 17.46 KN/shear plane / bolt | |
| d) | $f_1 d_F^2\left(\sqrt{\frac{1}{6}\frac{f_2}{(f_1+f_2)}\frac{f_y}{f_1}}+\frac{1}{5}\frac{t_1}{d_F}\right)$ |  | 32.30 KN/shear plane / bolt | |
| g) | $f_1 d_F^2\sqrt{\frac{2}{3}\frac{f_2}{(f_1+f_2)}\frac{f_y}{f_1}}$ |  | 15.23 KN/shear plane / bolt | GOVERNS |

$$Nr = \phi\, n_u\, K_D K_{st} K_T n_s = 0.8\left(15.23\frac{KN}{screw}\right)(1.15)(1.0)(1.0)(2\ shear\ planes) = 26.12\frac{KN}{screw}$$

O86-19 clause 11.9.3.3.3.2 notes that discrete hold-down must resist an additional 20% above the forces it observed forces in the system.

Table 4-7: Hold-down fastener design

| Level | Hold-down Load (KN) | Hold-down Design Load (KN) | Minimum number of Bolts Required |
|---|---|---|---|
| **Level 3** | 122 | 146 | 6 |
| **Level 2** | 360 | 432 | 17 |
| **Ground** | 690 | 828 | 32 |

The minimum bolt spacing requirements of O86-19 are:

**Table 12.15**
**Placement of bolts and dowels in a connection loaded parallel to grain**
(See Clause 12.4.3.1.)

| Symbol | Dimension* | Minimum spacing |
|---|---|---|
| $S_P$ | Spacing parallel to grain | $4d_F$ |
| $S_Q$ | Spacing perpendicular to grain | $3d_F$ |
| $a$ | Unloaded end distance parallel to grain | Maximum {$4d_F$, 50 mm} |
| $a_L$ | Loaded end distance parallel to grain | Maximum {$5d_F$, 50 mm} |
| $e_P$ | Unloaded edge distance perpendicular to grain | Maximum {$1.5d_F$, 1/2 $S_Q$} |

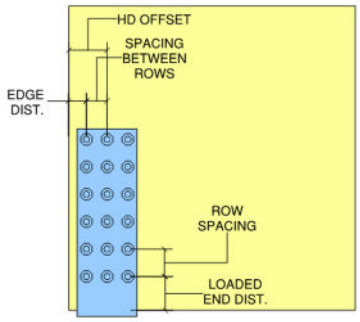
* *$d_F$ = nominal diameter of fastener.*

O86-19 clauses 12.4.4.4 and clause 12.4.4.5 note that row shear and group tear out of connectors are not required for CLT. It is noted in the commentary that "it is good engineering practice to use larger fastener spacing in connections to help avoid stress concentrations in small areas of the CLT panel." The chosen layout for the bolts is as follows.

Table 4-8: Hold-down Fastener Sspacings

| Level | Rows parallel to load | Bolts per row | Stiffness of Hold-down Bolts (KN/mm) | Row Spacing & End Dist. (mm) | Spacing btw Rows (mm) | Edge Dist. (mm) | Wall Edge Offset (mm) |
|---|---|---|---|---|---|---|---|
| **Level 3** | 2 | 3 | 183 | 100 | 60 | 30 | 90 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Level 2** | 3 | 6 | 549 | 100 | 60 | 30 | 120 |
| **Ground** | 3 | 11 | 1006 | 100 | 60 | 30 | 120 |




It should be noted, in the absence of test data demonstrating the necessary ductility where steel tension or bending is included, all steel elements should be capacity protected. Additionally, bolts connection discrete hold-downs between floors, and anchors into concrete foundations should also be capacity protected. The steel plate and anchors have not been designed in this example. The steel elements would be designed following guidance provided in S16 and the concrete anchorage at the base of the connection would be designed per A23.3.

### 4.5.4 BASE SHEAR CONNECTION

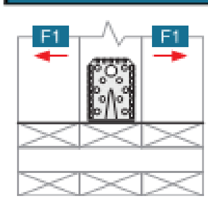
O86-19 Clause 11.9.3.4.1.b notes that energy dissipative connections should be found at “...connections of shearwalls to foundations, and shearwalls to floors beneath, in uplift only”. As discussed in section 3.3 of this document, the shear connection is not explicitly noted as requiring capacity protection, further, the commentary notes that “wall-to-foundation” or “wall-to-floor below” connection can form part of the basis of the dissipating system. This is discussed in the same section that discusses the use of discrete hold-downs as dissipating connections. Despite no specific code recommendation, a 20% overstrength has been applied to these discrete base shear connections in keeping with capacity protection philosophy outlined in O86-19.

Table 4-9: Shearwall Base Shear Design Forces

| Level | Design Wall Shear Load (KN) | Panel Shear (KN/panel) | Design Panel Shear (KN/panel) |
|---|---|---|---|
| **Level 3** | 232 | 58 | 70 |
| **Level 2** | 452 | 113 | 136 |
| **Ground** | 569 | 143 | 171 |

One typical approach to shear connections at the base of CLT shearwalls is to provide angle bracket connections with either screws or nails. Custom brackets can be designed in much the same approach outlined in the hold-down design section, designing fasteners, steel plates, and anchorage where applicable. For this example, we will review proprietary MTB brackets provided by MTC Solutions. In this case the combined behaviour of the steel plate and fasteners has been reviewed in physical testing.

Table 1.1, F1 - Factored Lateral Resistance in CLT

| Configuration | | | Fasteners | | Factored Resistance [kN] | | Estimated Slip Modulus [kN / mm] |
|---|---|---|---|---|---|---|---|
| | | | | | F1 - Lateral Resistance | | |
|  F1 F1 | Angle Bracket | Relative Density | Type | Quantity | Standard Loading [$K_D$ = 1.0] | Short Term Loading [$K_D$ = 1.15] | |
| | 90 | 0.42 (SPF) | Ecofast 4.5 x 50 | 20 | 6.7 | 7.7 | 2.1 |
| | 105 | | | 26 | 6.8 | 7.9 | 3.1 |

It is also important to ensure that the base shear connectors can tolerate the displacement associated with the rocking behaviour shown in section 4.5.1. The conservative approach taken in this initial shearwall design assumes the entire load is resisted by the hold-downs whereas in reality, the brackets often have significant uplift resistance.

| Level | Design Shear Per Panel (KN) | Brackets per Panel | Spacing (mm o/c) | Offset from panel edge (mm) |
|---|---|---|---|---|
| **Level 3** | 70 | 9 | 150 | 275 |
| **Level 2** | 136 | 18 | 150 both faces | 275 |
| **Ground** | 171 | 22 | 150 both faces | 125 |

Determining the vertical load in the brackets is very dependant on the relative stiffness between the splines, the hold-downs, and the brackets themselves.

### 4.5.5 CLT PANEL DESIGN

The CLT panels needs to resist both the in-plane loads as well as the compression at the ends of the panels. It should be noted that Clause 11.9.3.4.2 specifically notes that CLT panels in shearwalls should be capacity designed. This can be done using either "…the 95$^{th}$ percentile of the ultimate resistance of the energy dissipative connections or the seismic design force may be determined using $R_dR_o \leq 1.3$". The wood design standard does not provide guidance on determining the 95$^{th}$ percentile of the ultimate strength of any connector types, and the commentary provides guidance on testing procedures suitable for determining this, but no guidance on actual numbers. In the absence of test data, or proprietary connectors where this testing has been completed, an approach using the lower $R_dR_o$ limit can be used. An effective overstrength factor (OS) can be determined as follows:

$$OS = \frac{(R_d)(R_o)}{R_dR_o \leq 1.3} = \frac{(2.0)(1.5)}{1.3} = 2.3$$

With this overstrength factor, shear loads, and compression loads can be determined based on the initial force distribution noted in section 3.4.1.

Table 4-10: Shearwall CLT panel loading

| Level | Design Wall Shear Load (KN) | Wall Shear Load w/ Overstrength (KN) | Overstrength Shear Per Panel (KN) | Design Compression (KN) | Overstrength Compression (KN) |
|---|---|---|---|---|---|
| **Roof** | 232 | 534 | 133.5 | 129 | 297 |
| **Level 3** | 452 | 1040 | 260.0 | 380 | 877 |
| **Level 2** | 569 | 1308 | 327.1 | 734 | 1701 |

#### 4.5.5.1 CLT Axial Design

Compression loads are determined in much the same manner as hold-down loads, except with the OS factor applied in place of the factor of 1.2 specified for hold-downs

The compression of the panel must be checked for both the bearing and the buckling of the panel. A simple first check of the bearing can be completed based on a triangular load distribution over half the width of the panel. At the base level, the panels rest on the foundation and the parallel to grain bearing governs. For CLT panels, all axial in-plane loading considers only the resistance of the laminations parallel to grain only.

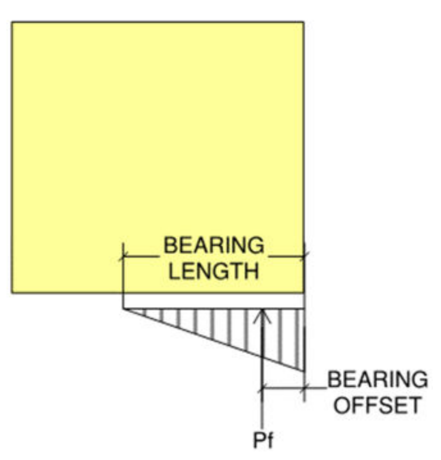
**Panel Axial Strength – O86-19 CL 8.4.5**

| | | | |
|---|---|---|---|
| $K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $K_{zc} = 6.3\left(\sqrt{12r_{eff}L}\right)^{-0.13} \leq 1.3 = 1.3$ | CL 8.4.5.5 |
| $K_D = 1.15$ | CL 8.3 | | |
| $\phi = 0.8$ | CL 8.4.5.5 | $K_c = \left[1 + \frac{F_c K_{Zc} C_c^3}{35E_{05}K_{SE}K_T}\right]^{-1} = 1.0$ | CL 8.4.5.5 |
| $f_c = 19.3MPa$ | | | |
| $L = 1\,mm$ | | $p_r = \phi f_c(K_D K_{Sc} K_T K_H) A_{eff} K_{Zc} K_c$ $= 0.8(19.3\,MPa)(1.15)\left(\frac{140mm^2}{mm}\right)(1.3)(1.0)$ $= 3.231\,kN/mm$ | |
| $C_c = \frac{L_e}{\sqrt{12r_{eff}}} = 0.0004$ | CL 8.4.5.3 | | |

**Perp-to-grain Bearing Strength – O86-19 CL 8.4.7**

| | | | |
|---|---|---|---|
| $K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $q_r = \phi f_{cp}(K_D K_H K_{Sc} K_T) A_{gross} K_B K_{Zcp}$ $= 0.8\,(5.3\,MPa)(1.15)\left(\frac{245mm^2}{mm}\right)(1.0)(1.0)$ $= 1.195\,kN/mm$ | CL 8.4.7.2 |
| $K_D = 1.15$ | CL 8.3 | | |
| $\phi = 0.8$ | CL 8.4.5.5 | | |
| $f_{cp} = 5.3MPa$ | | | |
| $K_{Zcp} = 1.0$ | CL 8.6.7.2 | | |
| $K_b = 1.0$ | CL 6.5.6.5 | | |

To determine the bearing width over which the bearing occurs we can assume a triangular distribution. It is important to ensure that the length of bearing does not exceed the distance from the compression face to the neutral axis. This can conservatively be taken as the mid-point of the panel unless cumulative tension forces are present.




From the initial force distribution described in section 4.5.2, we can estimate the bearing profile at the end of each panel to refine the loads in the splines and hold-downs:

$$P_r, Q_r = \frac{1}{2}(p_r, q_r)\left(\frac{W_{panel}}{2}\right)$$

A simple first check of the bearing can be completed similarly to the approach provided in section 3.4.2 of this document. Note that although O86-19 does not specifically require the perp-to-grain bearing of the floor panels to be capacity protected, it is good practice to ensure that the length of bearing for a capacity protected load does not exceed the length of the panel.

Table 4-11: Design Panel Overturning Bearing Strength

| Level | Design Wall Shear Load (KN) | Overstrength Compression (KN) |
|---|---|---|

| | | | |
|---|---|---|---|
| **Roof** | 1031 | 297 | OK |
| **Level 3** | 1031 | 877 | OK |
| **Level 2** | 2787 | 1701 | OK |

O86-19 does not provide guidance on effective widths for consideration when reviewing the buckling of CLT panels in compression, but it is noted the panel in compression should be capacity protected. An initial check can be completed based on half the width of the panel, sim the maximum bearing length in consideration. Using the CLT properties established in section 2.5 we can determine the buckling resistance of the panel in overturning.

**Panel Axial Strength – O86-19 CL 8.4.5**

| | | | |
|---|---|---|---|
| $K_H = K_{sb} = K_T = 1.0$ | CL 8.3 | $C_c = \frac{L_e}{\sqrt{12}r_{eff}} = \frac{4200}{\sqrt{12}(78.9mm)}$<br>$= 15 < 43$ | CL 8.4.5.3 |
| $K_D = 1.15$ | CL 8.3 | | |
| $\phi = 0.8$ | CL 8.4.5.5 | | |
| $f_c = 19.3MPa$ | | | |
| $K_{zc} = 6.3\left(\sqrt{12r_{eff}L}\right)^{-0.13}$<br>$= 1.03 < 1.3$ | CL 8.4.5.5 | $P_r = \phi f_c(K_D K_{Sc} K_T K_H)A_{eff} K_{Zc} K_c$<br>$= 0.8(19.3\ MPa)(1.15)(140000mm^2)(1.03)(0.8)$<br>$= 2048.5$ kN<br>$P_{r-half-panel} = P_r \frac{L_{panel}}{2}$<br>$= 2048.5\ kN/m\left(\frac{1.725m}{2}\right)$<br>$= 1766.9\ kN$ | CL 8.4.5.5<br>$P_r > P_f$<br>OK |
| $K_c = \left[1 + \frac{F_c K_{Zc} C_c^3}{35 E_{05} K_{SE} K_T}\right]^{-1}$<br>$= 0.8$ | CL 8.4.5.5 | | |

4.5.5.2 CLT Shear Design

The in-plane shear strength of a CLT panel should be determined based on the in-plane shear strength values provided by suppliers in their product specifications. Table 3 of Structurlam's PRG 320 product specification provides the in-plane shear strength for a similar panel the walls panels in this example. The tabulated values are intended for use with the full thickness of the CLT panel with the shear direction 0 or 90 used in the direction of applied load. It is important to check both the floors panel and the wall panel to ensure review if the length of the connection between the wall and the floor needs to exceed the length of the wall.

**In-Plane Shear Strength – Wall Panel**

| | | |
|---|---|---|
| $F_{v,,e,0} = 3.4\ MPa$ | `$V_{r-CLT} = \phi f_v(K_D K_{Sv} K_T)\frac{2}{3}(L_{panel} t_{panel})$<br>$=0.9(3.7MPa)(1.15)\left(\frac{2}{3}\right)(1725 \times 245) = 1078.9\ KN$ | |
| $F_{v,e,90} = 3.7\ MPa$ | | |
| $K_D = 1.15$ | | |
| $L_{panel} = 1725mm$ | | |
| $t_{panel} = 245\ mm$ | | |

**In-Plane Shear Strength – Floor Panel**

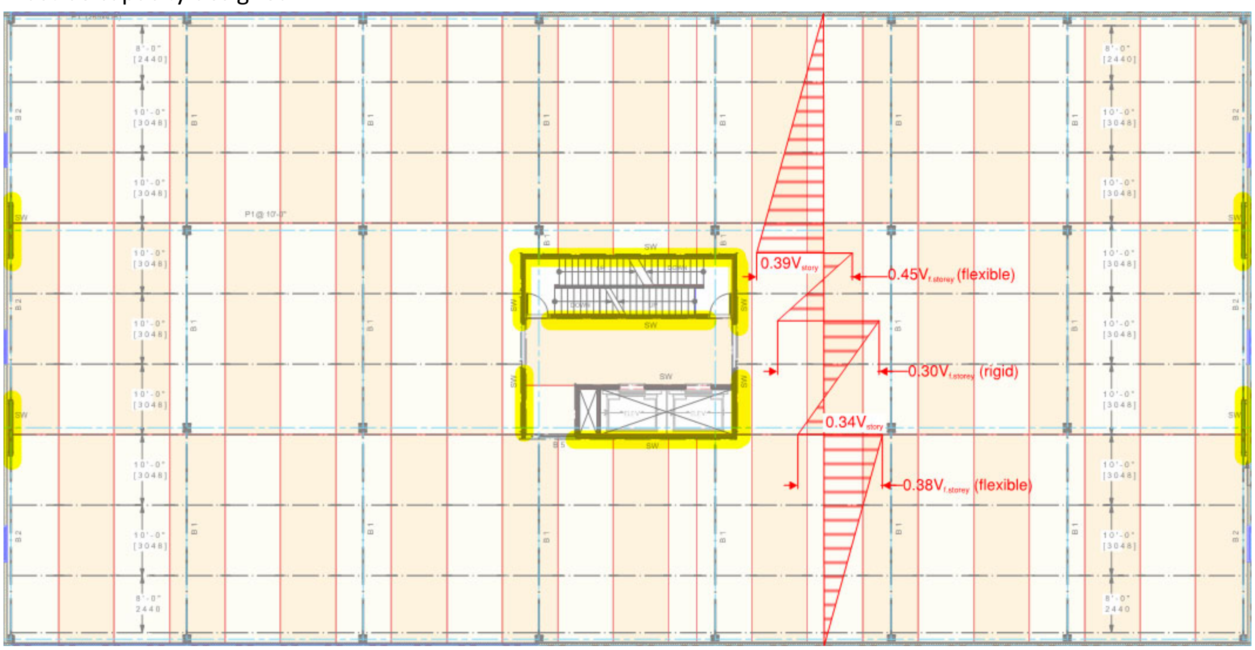
| | | |
|---|---|---|
| $F_{v,,e,0} = 2.5\ MPa$ | `$V_{r-CLT} = \phi f_v(K_D K_{Sv} K_T)\frac{2}{3}(L_{panel} t_{panel})$<br>$= 0.9(3.7\ MPa)(1.15)\left(\frac{2}{3}\right)(1725 \times 105) = 462.4\ KN$ | |
| $F_{v,e,90} = 3.7\ MPa$ | | |
| $K_D = 1.15$ | | |
| $L_{panel} = 1725mm$ | | |
| $t_{panel} = 105\ mm$ | | |

Table 4-12: Shearwall Panel shear strength review

| Level | Overstrength Shear Per Panel (kN/panel) | Shear Strength of Wall Panel (kN/panel) | Shear Strength of Floor Panel (kN/panel) | Shear Strength greater than Shear Load |
|---|---|---|---|---|
| **Roof** | 133.5 | 1078.9 | 462.4 | OK |
| **Level 3** | 260.0 | 1078.9 | 462.4 | OK |
| **Level 2** | 327.1 | 1078.9 | 462.4 | OK |

## 4.6 Diaphragm Design

Like the shearwall design, the diaphragm design is done using the envelope distribution determined in section 3.4.1. Taking the governing loads determined for both the flexible and rigid diaphragms we can establish an approximate diaphragm load distribution diagram. From here we can design the diaphragm splines, chords, and drags, all of which must be capacity designed.

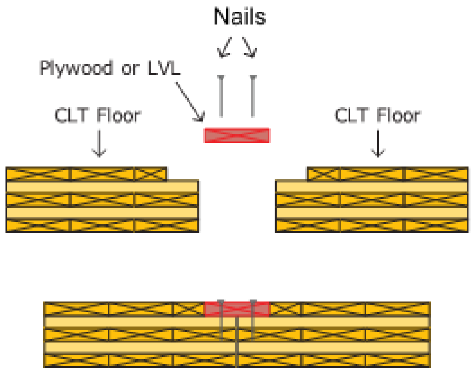



From this we can determine worst case diaphragm loads based on the storey shears previously established

Table 4-13: Diaphragm Spline Design Loads

| Level | Storey Shear $F_s = V\frac{W_x H_x}{\sum W_i H_i}$ (kN) | NBCC CL 4.1.8.15.1b) $F_s$ (kN) | Shear at SW3 $V_{fx\text{-}diaph\text{-}max}$ (kN) | Diaphragm Length $L_{x\text{-}diaph}$ (m) | Diaphragm Shear $v_{fx\text{-}diaph\text{-}max}$ (kN/m) |
|---|---|---|---|---|---|
| Roof | **611** | 1497/3=499 | 238 | 54 | 4.4 |
| Level 3 | **579** | 499 | 226 | 54 | 4.2 |
| Level 2 | 307 | **499** | 195 | 54 | 3.6 |

### 4.6.1 SPLINES




The diaphragm will be designed using plywood splines with nails in shear. The design is completed based on O86-19 section 12.9 for nails and spikes. Note that the calculation for the lateral strength of these small diameter fasteners are not impacted by the grain orientation of the connection. The strength of the nails can be calculated using similar lateral strength resistance calculations per clause 12.9.3.2 or the nail selection tables in the Wood Design Manual. Here the nail selection tables are used.

**Nail & Side Plate**

| | | |
|---|---|---|
| d = 5.26mmØ | $N'_r n_s = 0.816\ KN/nail$ | $N_r = N'_r n_s K' J_f =$ <br> $= (0.816\ KN/nail)(1.15)(1.3)$ <br> $= 1.22 KN/nail$ |
| L = 4.5in (114.3mm) | $J_E = J_A = J_B = 1.0$ | |
| 18.5mm DFP plywood | $J_D = 1.3$ | |
| 245mm S-P-F CLT | $K_D = 1.15$ | |
| Penetration = 93.8mm > min pen | $K_{Sf} = K_T = 1.0$ | |

From there we can determine the nail spacing required to provide a capacity protected diaphragm spline connection. We will also provide minimum nail spacings like the diaphragm options outlined in the plywood diaphragm in the Diaphragm Selection Tables in the Wood Design Manual.

Table 4-14: Diaphragm Spline Connector Design

| Level | $v_{fx\text{-}diaph}$ (KN/m) | $v_{fx\text{-}diaph\text{-}OS}$ (KN/m) | Max Nail Spacing (mm) | Chosen Nail Spacing (mm) |
|---|---|---|---|---|
| Roof | 4.4 | 10.1 | 120mm | 100mm |
| Level 3 | 4.2 | 9.6 | 125mm | 100mm |
| Level 2 | 3.6 | 8.3 | 135mm | 100mm |

### 4.6.2 CHORDS

The chord forces can be determined by treating the Diaphragm like a deep beam and determining the Moment and therefore tension and compression forces at the extremities. Once we know the tension and compression forces, we can design the chord and its connection to the CLT diaphragm

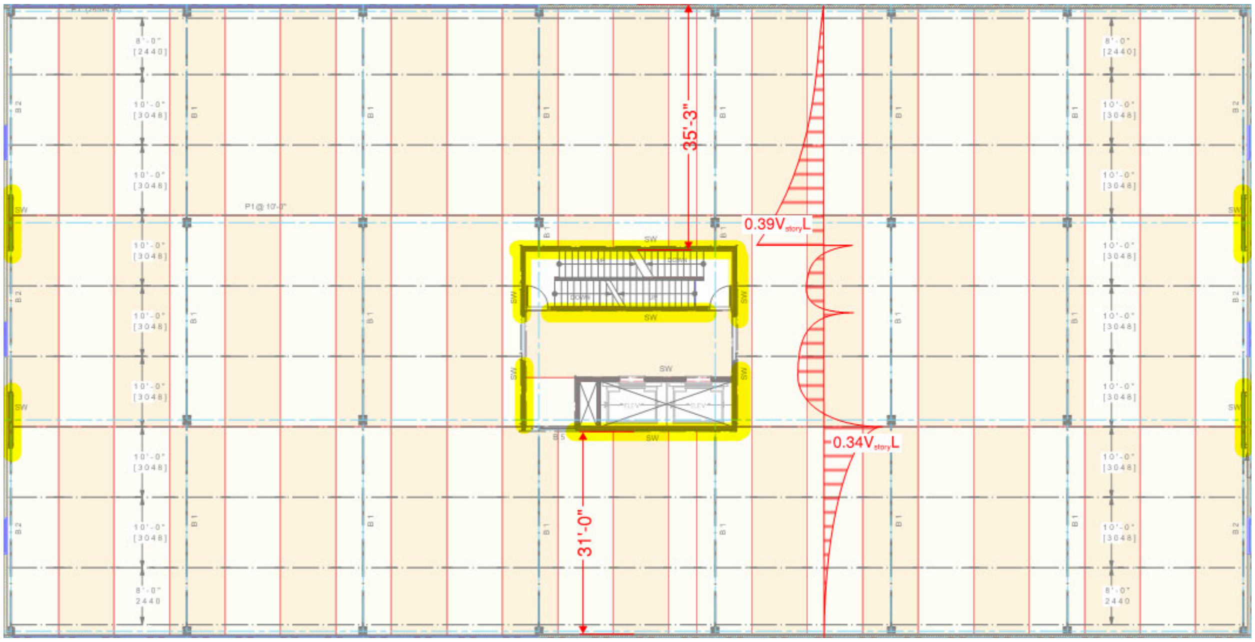
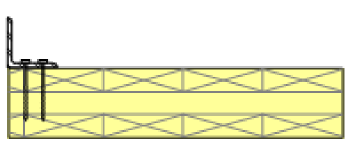


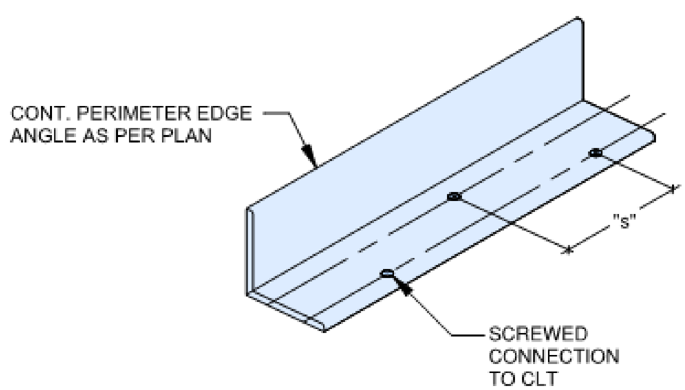

Table 4-15: Diaphragm Chord Design Loads

| Level | Peak Diaph Shear $V_{fx}$ (KN) | Length in X-dir $L_x$ (m) | Diaphragm Moment $M_{fx} = \frac{V_{fx}L_x^2}{6}$ (KNm) | Length in y-dir $d_y$ (m) | Chord Force $C_{fx}, = M_{fx}/d_y$ (KN) | Overstrength Chord Force $C_{fx.OS}, T_{fx.OS}$ (KN) | conn. Force (KN/m) |
|---|---|---|---|---|---|---|---|
| **Roof** | 238 | 10.75 | 4584.0 | 54 | 84.9 | 195.3 | 18.2 |
| **Level 3** | 226 | 10.75 | 4352.9 | 54 | 80.6 | 185.4 | 17.2 |
| **Level 2** | 195 | 10.75 | 3755.8 | 54 | 69.6 | 160.0 | 14.9 |

Steel perimeter angles will be used as the chord members. We can assume the chords are fully braced for compression due to continuous screwed connections along the length of the angle. The steel angle would likely also require splicing along it's length to accommodate shipping and fabrication; the design of theses steel splices would be completed with welds or bolts per S16-19; the design of these steel elements is not reviewed here.

The Angle size will likely be governed by the screw size requirements as well as whatever depth is required to provide closure to concrete toppings poured on the CLT. The strength of the angle is not reviewed here, but would be designed per the steel design standard, S16. For the purpose of the calculations, a L76X76X6.4 has been chosen.





The screwed connection to the steel is designed in the same manner as any other screwed connection. In this case we will look at typical wood screws as discussed in O86-19 section 12.11. As these connections are capacity protected, there is no need to ensure a specific yield mode for the screws. As the screws are connected in to CLT, there is no need to review row shear or group tear out in the wall as discussed in section 3.5.6 of this document. For simplicity, the *Screw Selection Tables* in the *Wood Design Manual* have been used to select the screws.

**Wood Screw & Side Plate**
10 Ga Screw x 3" long
4.8mm thick A36 steel
245mm E1 CLT
Penetration = 71.4mm > min pen

$N'_r n_s = 1.57\ KN/screw$
$K_{Sf} = K_T = 1.0$
$K_D = 1.15$

$N_r = N'_r n_s K'$
$= (1.57\ KN/screw)(1.15)$
$= 1.81 KN/screw$

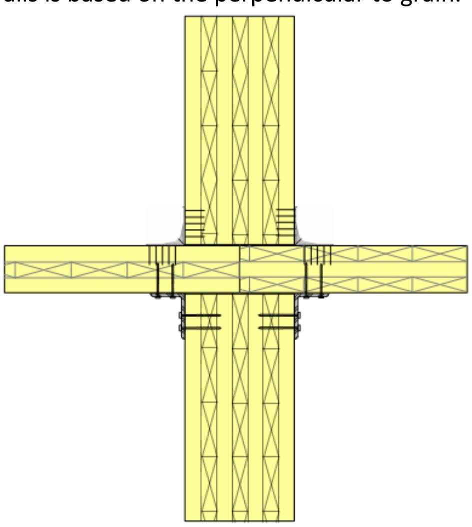
Table 4-16: Diaphragm Screwed Connection

| Level | $t_f/c_{fx\text{-}diaph\text{-}capacity}$ (KN/m) | Max Screw Spacing (mm) | Chosen Screw Spacing (mm) |
|---|---|---|---|
| Roof | 18.2 | 100mm | 100mm |
| Level 3 | 17.2 | 105mm | 100mm |
| Level 2 | 14.9 | 122mm | 100mm |

### 4.6.3 DIAPHRAGM-TO-SHEARWALL CONNECTIONS

Diaphragm-to-shearwall connections and drag connection in diaphragms are also similarly required to be capacity protected. Refer to the shearwall design section 4.5 for the calculation of the capacity protected load per panel

For this example, we will use custom angles with screws, like the diaphragm chord connection. We will look to use larger diameter SFS screws to achieve the relative high capacity protected design loads. The connection into the walls is based on the perpendicular to grain.



**Screw – 10Ø x 160 STS screw**
d = 10mm
$d_{shank}$ = 7.2mm
$f_{y.screw}$ = 1000 MPa

**Side Member – Steel Plate**
t1 = 4.8mm

$$f_1 = 3\left(\frac{\phi_s}{\phi_w}\right) f_u = 1800\ MPa$$

**Main Member – CLT**
G = 0.42
t2 = 155
Perp-to grain

$$\boldsymbol{f_2 = 22G(1 - 0.01d_s) = 8.6\ MPa}$$

Using the same lateral strength calculations discussed in section 3.5.4 of this design example. The results are as follows:

$$n_u = f_1 d_f^2 \sqrt{\left(\frac{2}{3}\frac{f_2}{f_1 + f_2}\frac{f_y}{f_1}\right)} = 3.91\ KN/screw$$

$$Nr = \phi\, n_u\, K_D K_{st} K_T = 0.6\left(3.91\frac{KN}{screw}\right)(1.15)(1.0)(1.0) = 2.70\frac{KN}{screw}$$

We also need to consider minimum screw spacings based on both the lag screw provisions and the proprietary screw spacing requirements from the supplier. From this we can determine the design connection between the actual screwed connection.

Table 4-17: Diaphragm to Shearwall Connection Design

| Level | Capacity Protected Panel Shear (KN/panel) | Min Screw Spacing (mm) | Actual Screw Spacing (mm) |
|---|---|---|---|
| **Roof** | 133.5 | 1 side, 2 rows @ 69mm | 1 side, 2 rows @ 65mm |
| **Level 3** | 260.0 | Both sides, 2 rows @ 71mm | Both sides, 2 rows @ 65mm |
| **Level 2** | 327.1 | Both sides, 2 rows @ 57mm | Both sides, 2 rows @ 50mm |

Note that where connections are required on both sides of the walls a drag element would be required where the wall is adjacent to an opening (like an elevator opening). In these cases, an angle could be connected to the wall within the opening, and then extend beyond the wall on either side to accommodate a connection to the diaphragm beyond.

### 4.6.4 DIAPHRAGM DEFLECTION

The Wood Standard O86-19 commentary discusses diaphragms and notes that for large diaphragms diaphragm deflections would likely need to be determined as CLT diaphragms do not act as purely rigid. Although the diaphragm spans in this building are relatively well below the 100ft spans mentioned in the commentary, they will be commented on here.

Both the O86-19 diaphragm deflection calculation method blocked plywood shearwalls which is outlined in CSA O86-19 CL 11.7.2 as follows:

| $\Delta_{diaph} =$ | $\frac{5vL^3}{96EAW}$ | $\frac{vL}{4B_v}$ | $0.00061Le_n$ | $\frac{\sum(x\Delta_c)}{2W}$ |
|---|---|---|---|---|
| | Chord Elongation | Panel Deformation | Fastener Slip | Chord Joint Slip |

Note that this provision is based on a simple span diaphragm. A similar equation is provided for shearwall deflection in CSA O86-19 CL 11.7.1 as follows:

| $\Delta_{diaph} =$ | $\frac{2vH_s{}^3}{3EAL_s}$ | $\frac{vL}{B_v}$ | $0.00025Le_n$ | $\frac{H_s}{L_s}\sum(x\Delta_c)$ |
|---|---|---|---|---|
| | Chord Elongation | Panel Deformation | Fastener Slip | Chord Joint Slip |

The American Technical Council (ATC) 7 Guidelines for the Design of Horizontal Wood Diaphragms further breaks down each of the components of the diaphragm specifically outlining the approach for determining the panel slip based on typical plywood panel sizes.

$$C = \frac{\left(\frac{1}{P_L}+\frac{1}{P_w}\right)}{2} = \frac{\left(\frac{1}{1219}+\frac{1}{2438}\right)}{2} = 0.00061$$

These can be modified for to be used for calculation of CLT diaphragm deflections. WoodWorks USA has published diaphragm deflection calculation methods based on this approach.

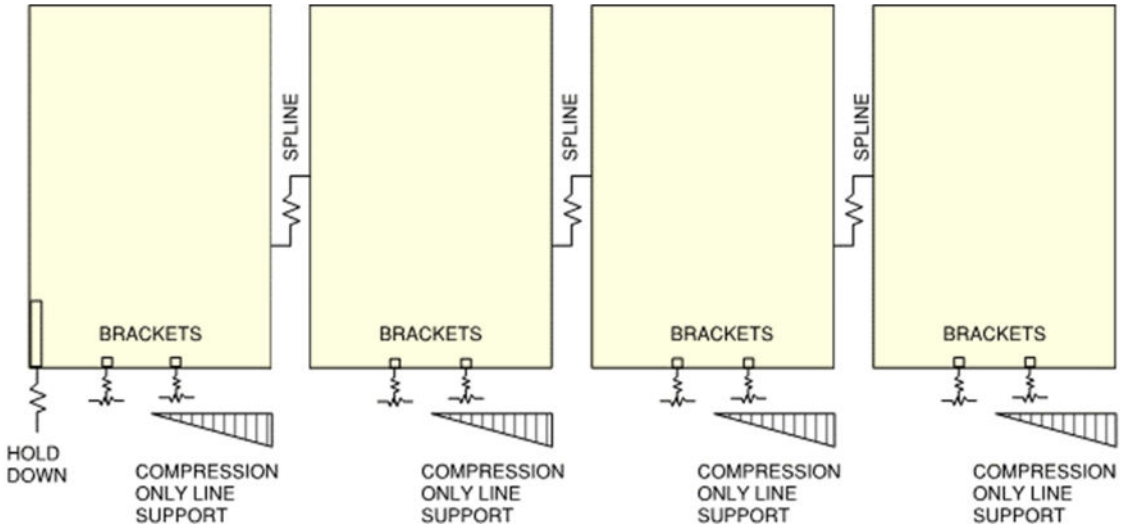
| | | | | |
|---|---|---|---|---|
| $\Delta_{diaph.simple}$ | $\frac{5vL^3}{96EAW}$ | $\frac{vL}{4G_{eff}t_v}$ | $\frac{\left(\frac{1}{P_L}+\frac{1}{P_w}\right)}{2}L\left(\frac{0.013vs}{d_f^2}\right)^2$ | $\frac{\Sigma\left(x\frac{PL}{EA}\right)}{2W}$ |
| $\Delta_{diaph.cantlv}$ | $\frac{2vL^3}{3EAW}$ | $\frac{vL}{2G_{eff}t_v}$ | $\frac{\left(\frac{1}{P_L}+\frac{1}{P_w}\right)}{2}L\left(\frac{0.013vs}{d_f^2}\right)^2$ | $\frac{\Sigma\left(x\frac{PL}{EA}\right)}{W}$ |
| | Chord Elongation based on the stiffness of the outer foot of the panel | Panel Deformation. Stiffness provided by the supplier | Fastener Slip, based on the nailing in the splines (O86-19 CL A.11.7 & ATC-7) | Chord Joint Slip. Elongation in the steel chord + slip in bolted connections per joint and the total number of CLT joints |

Note that is this case there are continuous panels over the length of the cantilevered diaphragm allowing for the use of the greater of either the panel elongation or the steel chord elongation. The calculation of the deflection of the cantilevered deflection is as follows:

| | | |
|---|---|---|
| Panel Elongation | $\frac{2vL^3}{3EAW} = \frac{2(4.4\ N/mm)(10750mm)^3}{3(11700\ MPa)(300x35x2)(54000mm)} = 0.275mm$ | GOVERNS |
| Chord Elongation | $\frac{2vL^3}{3EAW} = \frac{2(4.4\ N/mm)(10750mm)^3}{3(200000\ MPa)(927)(54000mm)} = 0.364mm$ | |
| Panel Shear deformation | $\frac{vL}{2G_{eff}t_v} = \frac{(4.4\ N/mm)(10750mm)}{2(200MPa)(105mm)} = 1.13mm$ | |
| Fastener Slip | $\frac{\left(\frac{1}{P_L}+\frac{1}{P_w}\right)}{2}L\left(\frac{0.013vs}{d_f^2}\right)^2 = \frac{\left(\frac{1}{3000}+\frac{1}{10750}\right)}{2}10750\left(\frac{0.013(10.1)(100)}{5.26^2}\right)^2 = 0.52mm$ | |
| Chord Slip | $\frac{\Sigma\left(x\frac{PL}{EA}\right)}{2W} = 0mm$ | no CLT joints in cantilevered portion of the diaphragm in question |
| $\Delta_{diaph} = 1.93\ mm$ | | |

## 4.7 CLT Shearwall Modeling

The hand calculation method provided includes several simplifications, including summation of the moment at the tip of the panel. Modeling the shearwall can be used to verify the connection design. It is also a critical element to determine the drift of the lateral system, as detailed calculations including all the deformations of each element would be very complex by hand. It can also be used to refine the connection design in the splines, hold-downs, and brackets. It is critical to model all the dissipating elements and any elements required to deform elastically as springs.




#### 4.7.1 CONNECTION STIFFNESS

Determining the spring stiffnesses of each connector is critical as most of the drift in the system is determined by the deformation in the connectors, as the panels are comparatively very stiff. Proprietary connectors will generally provide stiffnesses or expected deformations that can be used in modeling. For custom connections like the hold-down or the splines in this example, the stiffnesses can be determined based on the provisions laid out in the CLT guide from FP Innovations.

**Table 1 Elastic stiffness of timber-to-timber and wood-based panel-to-timber connections per shear plane per fastener (N/mm)**

| Fastener type | $K_{ser}$ | K |
|---|---|---|
| Dowels<br>Bolts with or without clearance [a]<br>Screws<br>Nails (with pre-drilling) | $\frac{\rho_m{}^{1.5}d}{23}$ | $1470G^{1.5}d$ |
| Nails (without pre-drilling) | $\frac{\rho_m{}^{1.5}d^{0.8}}{30}$ | $1125G^{1.5}d^{0.8}$ |
| Split-ring connectors type A according to EN 912<br>Shear-plate connector type B according to EN 912 | $\frac{\rho_m d_c}{2}$ | $520Gd_c$ |

[a] The clearance should be added separately to the deformation.

$$\rho_m = \sqrt{\rho_{m1} \cdot \rho_{m2}}$$

For steel-to-timber or concrete-to-timber connections, the slip modulus, $K_{ser}$, should be based on $\rho_m$ for the wood member and may be multiplied by 2.0.

##### 4.7.1.1 Spline Stiffness

The spline stiffnesses in this system are developed based on the density of the CLT panels, and the diameter of the screws used as outlined in Table 1 from the FP Innovations guide.

$$K_{screw} = 1470\, G^{1.5} d_S = 1470\, (0.42)^{1.5} (7.2) = 2880\; N/mm/screw$$

Table 4-18: Spline Stiffness Summary

| **Level** | **Panel Height** (m) | **Spline Load (KN/m)** | **Screw Spacing (mm)** | **Screw Stiffness (N/mm / m)** |
|---|---|---|---|---|
| **Level 3** | 3.8 | 32.1 | 1 row @ 75 | 38,400 |
| **Level 2** | 3.8 | 62.7 | 1 row @ 40 | 72,000 |
| **Ground** | 4.25 | 78.5 | 2 rows @ 65 | 88,615 |

### 4.7.1.2 Hold-down Stiffness

Using the connector stiffness equations previously provided, a summary of the stiffness of the dowel connections at each level are provided based on the same fastener stiffness calculations provided for the spline stiffness. Note that because the face plates are steel in this case, calculation is multiplied by 2.

$$K_{bolt} = 2(1470\ G^{1.5} d_S n_s) = 2(1470\ (0.42)^{1.5}(7.2)(2\ shear\ planes)) = 30489\ N/mm/bolt$$

Table 4-19: Hold-down Stiffness

| **Level** | **Rows** | **Bolts per row** | **Bolt Stiffness $K_{bolts}$ (KN/mm)** |
|---|---|---|---|
| **Level 3** | 2 | 3 | 183 |
| **Level 2** | 3 | 6 | 549 |
| **Ground** | 3 | 12 | 1098 |

Table 4-20

Additionally, the behaviour of the steel plates and anchors will also contribute to the stiffness of the hold-down and should considered. The stiffness contribution of these elements should be considered based on engineering mechanics with the overall stiffness established using the combined springs in series:

$$K_{holddown} = \left[\frac{1}{K_{bolts}} + \frac{1}{K_{plate\ elogation}} + \frac{1}{K_{plate.bending}} + \frac{1}{K_{anchor.elongation}}\right]^{-1}$$

### 4.7.1.3 Bracket Stiffness

Given the base connection brackets are proprietary, the supplier has provided the stiffness of the connector in shear and uplift. Although the initial Design outlined in section 4.5 ignores the uplift resistance strength of the brackets, they should be accounted for in the more detailed model. It is important to review the load and/or deflection of the brackets in uplift to ensure that they can tolerate the deformation of the system.

Table 1.1, F1 - Factored Lateral Resistance in CLT

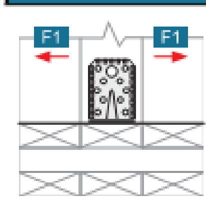
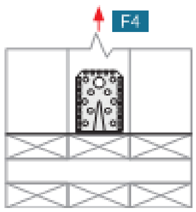
| Configuration | | | Fasteners | | Factored Resistance [kN] | | Estimated Slip Modulus [kN / mm] |
|---|---|---|---|---|---|---|---|
| | Angle Bracket | Relative Density | Type | Quantity | F1 - Lateral Resistance: Standard Loading [$K_D$ = 1.0] | F1 - Lateral Resistance: Short Term Loading [$K_D$ = 1.15] | |
|   | 90 | 0.42 (SPF) | Ecofast 4.5 x 50 | 20 | 6.7 | 7.7 | 2.1 |
| | 105 | 0.42 (SPF) | Ecofast 4.5 x 50 | 26 | 6.8 | 7.9 | 3.1 |

Table 2.1, F4 - Factored Uplift Resistance in CLT




| Configuration | | Fasteners | | Factored Resistance [kN] | | Estimated Slip Modulus [kN / mm] |
|---|---|---|---|---|---|---|
| | | | | F4 - Uplift Resistance | | |
| Angle Bracket | Relative Density | Type | Quantity | Standard Loading [$K_D$ = 1.0] | Short Term Loading [$K_D$ = 1.15] | |
| 90 | 0.42 (SPF) | Ecofast 4.5 x 50 | 20 | 6.3 | 7.2 | 5.4 |
| 105 | | | 26 | 6.0 | 6.9 | 4.3 |

Form this, and the bracket spacing information provided in section 3.5.5 of this document we can model each bracket in the system or smear the brackets over a line to create a line spring.

Table 4-21: Bracket Connection Stiffness

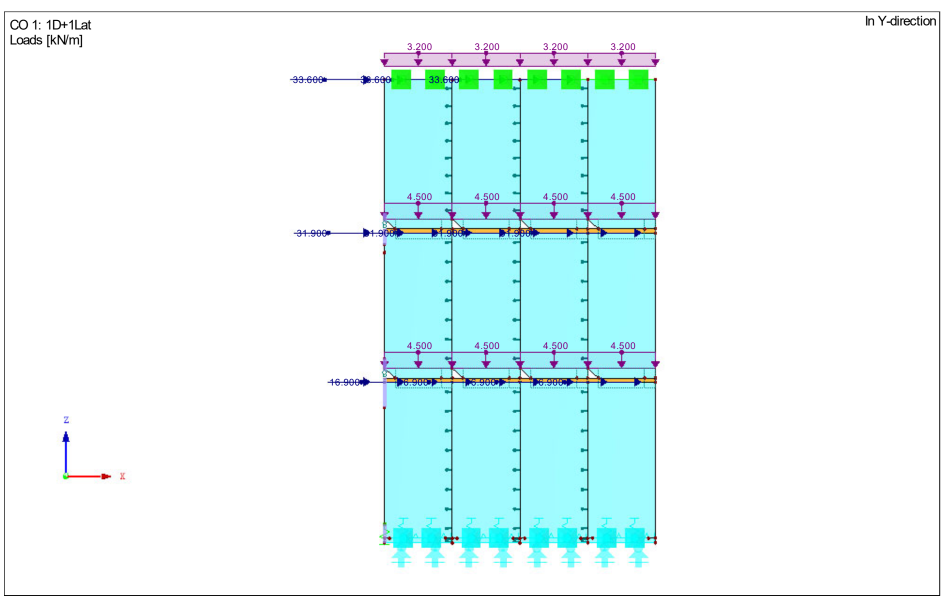
| Level | Panel Length (mm) | Brackets per Panel @ base | Offset from panel edge (mm) | Shear Stiffness of Brackets (kN/mm/m) | Uplift Stiffness of Brackets (kN/mm/m) |
|---|---|---|---|---|---|
| **Roof** | 1725 | 9 | 275 | 23.7 | 32.9 |
| **Level 3** | 1725 | 18 | 275 | 47.4 | 63.8 |
| **Level 2** | 1725 | 22 | 125 | 57.9 | 78.0 |

4.7.2 MODEL IMPLIMENTATION

Finite Element Modeling is a useful tool to check the load distribution does not exceed any of the design forces established, check the deflection of the system, and where desired, iterate to a more efficient design. The panels are modeled as shell elements with the in-plane stiffness properties associated with the 7 ply panels in use. The initial loads provide a starting point for design, but it should be noted that several things will affect the actual load distribution. The resultant model can be expected to have somewhat different load distributions than the initial design forces determined in section 3.5.2.

| | |
|---|---|
| Shearwall Panels | Shear panels are modeled as orthotropic laminated shell elements with the properties associated with the panels in question. It is often acceptable to model these as rigid shell elements as discussed in the O86-19 commentary |
| Floor Panels between levels | Floor panels are modeled as isotropic shell elements with properties based on the perpendicular to grain stiffnesses of CLT ($E_{parallel}$/30) |
| Splines | Splines are modeled as releases between panel elements with a spring stiffness along the length of the line |
| Shear Brackets at base | Shear brackets at the base as a non-linear line support that is rigid in compression (perpendicular to the line) and assigned a linear stiffness associated with the brackets in use in tension (perpendicular to the line) and shear (along the line) |
| Shear Brackets between floors | Shear brackets between floors are modeled as a non-linear line release that is rigid in compression (perpendicular to the line) and assigned a linear stiffness associated with the brackets in use in tension (perpendicular to the line) and shear (along the line). Note that this should include the brackets at the base of the walls to the floors below, and the brackets from the floors to the walls below as outlined in the diaphragm design section (section 4.7.3) |
| Hold-downs at base | Hold-downs at the base of the wall are modeled as a point support with a linear spring stiffness per the established spring stiffness. |
| Hold-downs between levels | Hold-downs between floors are modeled as steel elements with axial releases with linear springs corresponding to the hold-down stiffness established at each end. |

Gravity and lateral line loads are applied at the top of each wall panel. The loads represent the line loads for the wall in question determined in section 3.4 of this document



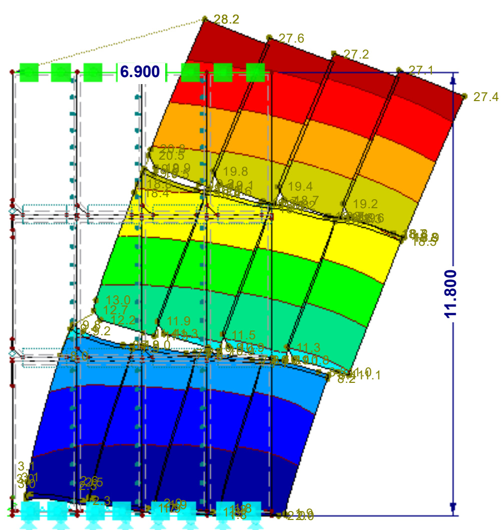
For this model, the non-linear line springs associated with the bracket connections include both the stiffness of the brackets as well as the stiffness of the capacity protected connection to minimize the number of non-linear connections in the model (ie. simplifying the model). These are calculated using springs in series approach, similar to the discussion in the Hold-down stiffness section.

$$K_{shear.conn} = \left[\frac{1}{K_{wall.base.brackets}} + \frac{1}{K_{top.fo.wall.connection}}\right]^{-1}$$

### 4.7.3 MODEL RESULTS

After running the model, we can look at the deformed shape as well as the overall deflection to determine if the wall is behaviour as it should, and to determine if the deflection is within allowable limits. The model clearly shows rocking behaviour between panels as well as uplift at panel ends between floors. This type of review is crucial to confirm that the model behaviour is as we would expect.

#### 4.7.3.1 Drift and Deformation Checks



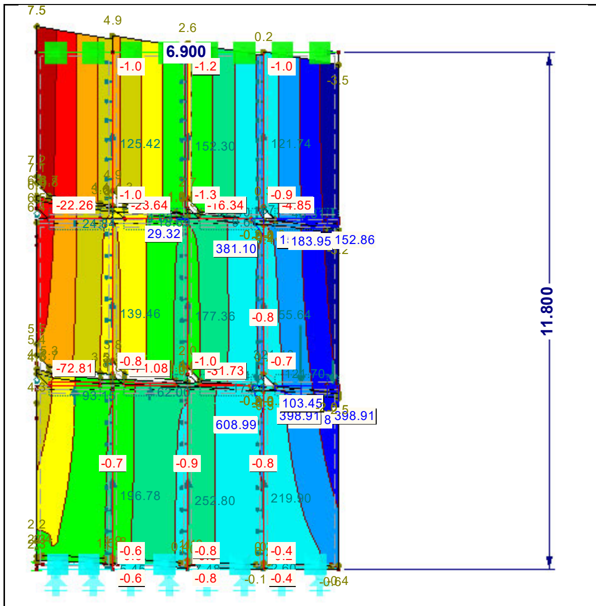

We can also review the drift of the building. The drift shown above is the total movement, not the horizontal movement. Exporting the horizontal movement, we can see a horizontal drift of 20.5mm, which is well within the limits for drift outlined in the NBCC.

$$\Delta_{drift}\left(\frac{R_D R_o}{I_e}\right) = 27.2mm(3.0) = 81.6mm$$

$$\Delta_{limit} = 2.5\%(H) = 2.5\%(11800mm) = 295mm$$

$$\Delta_{drift} = 82\ mm < \Delta_{limit} = 295\ mm$$

The global drift of the system clearly shows uplift at the entirety of the outermost panel. This can be worth exploring where modeling shows uplift in entire panels, which can limit the effectiveness of the CLT brackets or when the flexural limit of 30% of the total drift as discussed in section 4.2 is exceeded. In this case the total vertical deformation can be reviewed to determine the deformation associate with flexural behaviour of the wall.




From this we can see the cumulative elongation of the wall is 7.5mm. Taking the shearwall deflection formulation provided in O86-19 Clase 11.7.1.2, we can approximate the cumulative elongation of the wall with the portion of the deflection associated with the hold-down elongation (da)

$$\Delta_{Flexural} = \frac{H_s}{L_s} d_a = \frac{11.8m}{6.9m}(7.5mm) = 1.7(7.5mm)$$
$$= 12.75mm$$

$$\frac{\Delta_{Flexural}}{\Delta_{drift}} = \frac{12.75mm}{27.2mm} = 0.46 > 0.30$$

Fails flexural requirements.

### 4.7.3.2 Spline and Hold-down Review

We can also review the reactions at the base of the panels, the loads in the splines and hold-downs, and the deformation requirements for the base shear brackets to ensure they are all within design limits for the system

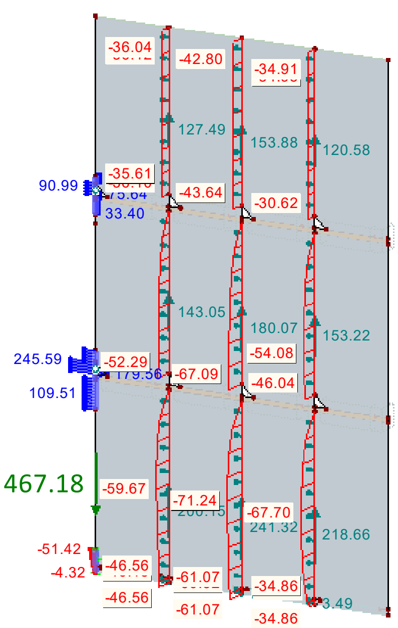



Table 4-22: Spline Strength Comparison

| Level | Spline Strength (KN/m) | Spline Strength (KN) | Spline Model Load (KN) | |
|---|---|---|---|---|
| **Level 3** | 36.4 | 140 | 153.9 | 9% over |
| **Level 2** | 68.3 | 260 | 180.1 | OK |
| **Ground** | 84.0 | 353 | 241.3 | OK |

The splines at the upper level will need to be re-designed for the higher load case, but all the other splines are within design limits.

Table 4-23: Hold-down strength Comparison

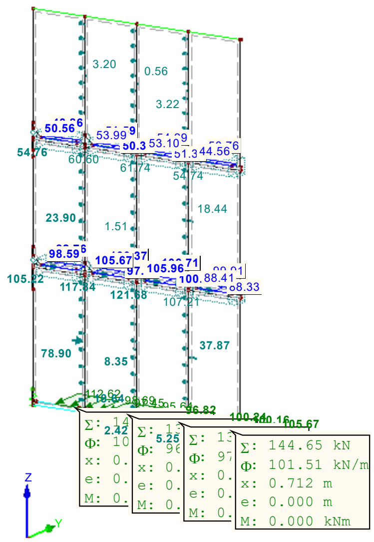
| Level | Hold-down Load (KN) | Hold-down Design Load (KN) | Hold-down Model Load (KN) | |
|---|---|---|---|---|
| **Level 3** | 122 | 146 | 91 | OK |
| **Level 2** | 360 | 432 | 246 | OK |
| **Ground** | 690 | 828 | 467 | OK |

The hold-downs are within design limits

Once the re-design of the splines at the upper levels is complete, it would be possible to re-iterate the spline and hold-down design to reduce the over capacity currently in the design. This would require the model the be updated for the new stiffnesses to verify the final conditions in the model.

#### 4.7.3.3 Bracket Review

The base shear connections also need to be reviewed for both the shear load distribution between panels, as well as the uplift deformation to ensure that the brackets are not failing in uplift. We also need to review and confirm the shear loads in the brackets are within tolerances:




By taking the shear loads observed in the model and increasing them by 20% we can determine if the strength of the shear brackets is sufficient

Table 4-24: Bracket Shear Design Comparison

| Level | Brackets per Panel | Panel Bracket Shear Strength (KN) | Model Panel Shear (KN) | Design Panel Shear (KN) | |
|---|---|---|---|---|---|
| **Roof** | 9 | 71.1 | 61.7 | 74.1 | 4% over |
| **Level 3** | 18 | 142.2 | 121.6 | 145.9 | 3% over |
| **Level 2** | 22 | 173.8 | 147.8 | 177.4 | 2% over |

Based on the load distribution between panels, some of the panel shear connections are slightly under designed and could have additional brackets added to accommodate the model loading.

It is also critical to review the uplift experienced by the brackets and to complete and interaction check of the brackets. It should be noted that the fact that entire panel lengths appear to be in uplift is an early warning that the uplift capacity of the brackets may not be sufficient.

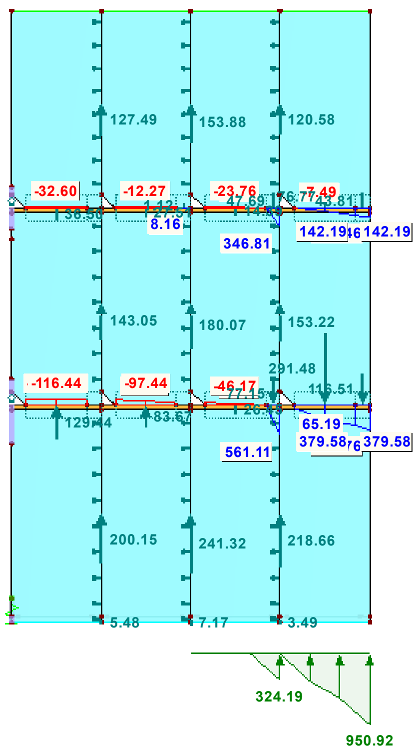
Table 4-25: Bracket uplift design comparison

| Level | Panel Bracket Uplift Strength (kN) | Model Panel Uplift (KN) | Panel Bracket Sheat Strength (kN) | Model Panel Shear (KN) | Tf/Tr + Vf/Vr |
|---|---|---|---|---|---|
| **Roof** | 62.1 | 36.5 | 71.1 | 61.7 | 145% |
| **Level 3** | 124.2 | 129.4 | 142.2 | 121.6 | 190% |
| **Level 2** | 151.8 | 233.4 | 173.8 | 147.8 | 239% |

Note that providing brackets with vertically slipped connections is one way to avoid the over-capacity found in the above model. Alternately, increasing the number of shear connection can achieve the same behaviour.

#### 4.7.3.4 Panel Review

It is important to also review the strength of the panels in compression for both buckling and bearing. Provided that the shear distribution was not significantly different than that found in the original design, it should not be needed to review the panel shear specifically. The model can provide us with the gravity loads in the releases and support.



Based on the load distribution we can see that in this load combination the outermost panel is taking the majority of the bearing over the height of the building, with limited or no bearing, depending on the level, or panels at the uplift end of the wall. This highlights that we may have a concern for compression in bearing or buckling.

If we first review the bearing per section 3.5.6 of this document

$P_{r.parallel} = \phi f_c(K_D K_H K_{Sc} K_T)A_{parallel} = 2472.96KN/m$

$P_{f.base} \ = \ 950.92\ KN/m$

$P_{f.OS-base} = 2.3(P_{f.base}) = 2187\ KN/m < P_{r.parallel}$ OK

$P_{r.perp} = \phi f_c(K_D K_H K_{Sc} K_T)A_{gross}K_B = 1373.8\ KN/m$

$P_{f.base} \ = \ 467.8\ KN/m$

$P_{f.OS-base} = 2.3(P_{f.base}) = 1076\ KN/m < P_{r.perp}$ OK

Buckling per section 3.5.6 of this document also outlines the buckling strength

$P_r \ = \ \phi f_c(K_D K_{Sc} K_T K_H)A_{eff}K_{Zc}K_c = 2048.5$ KN/m

$P_{f-edge\ panel} \ = \ 801\ KN$

$P_{f-OS} = 2.3(P_{f-edge\ panel}) = 1842\ KN/panel \ < \ Pr(LPanel)$

Note: bearing is distributed over the full length of the panel, so we have taken the full width in this case

#### 4.7.3.5 Next Steps

The modeled results clearly show that although the initial design provides an initial estimate of the design of the system, a detailed model is required to accurately establish that all the criteria outlined in Clause 11.9 of CSA O86-19 are met. In this example further iteration of the design is required to meet the flexural deformation limits as well as the bracket shear and tension interaction.

To accommodate the flexural requirements and the bracket strength requirements, a first step might be to increase the strength and stiffness of the hold-downs; this would decrease the overall vertical deformation in the system, both decreasing the flexural deformation, and decreasing the uplift demand on the brackets. Additionally, it may be possible to achieve the a change in behaviour by decreasing the stiffness of the splines. Typically, decreasing spline stiffnesses (and associated strengths) will allow more rocking behaviour and a better distribution of loads. This tuning exercise is not undertaken in this design example.

**Mass Timber Low-Rise Commercial**

**Block #1 Project Identification**

| | | | |
|---|---|---|---|
| **Project Name:** | | **Useage:** | First Storey Retail, Second and Third storey Office |
| **Code Classifications:** | Group E (Merchantile)<br>Group D (Office / Personal Services) | | |
| **Number of Storeys:** | 3 | **Footprint:** | 15,945 sf 1,481 m2 |
| **Structural System:** | Glulam Post and Beam, CLT Lateral System, CLT Floor System | | |

**Block #2 Project Description**

| | | | |
|---|---|---|---|
| **Building Size:** | 47,835 sf 4,443 m2 | **Building height:** | 3 Storeys |
| **Fire Rating:** | 45 min | **Maximum footprint:** | Unsprinklered: 1,500 m2<br>Sprinklered: 4,800 m2 |
| **Roof loading and deflection:** | Dead Load=1.5 KPa<br>Snow Load=2.3 Kpa<br>Snow Load Deflection=L/240<br>Total Load Deflection=L/180 | **Floor loading and deflection:** | Dead Load=3.0 KPa<br>Live Load=2.4 Kpa<br>Live Load Defledtion=L/420<br>Total Load Deflection=L/180 |
| **Structual system description:** | Panel-Purlin System, 25' x 30' typical bay | | |
| **SFRS Description** (indicate RdRo factor used): | CLT Shearwalls - Rd=2.0, Ro=1.5<br>Site Class | | |

**NBC 2015 Analysis**

**3.2.2.60 Group D, Up To 3 Storeys**

| | | |
|---|---|---|
| Sprinklered (Y/N): | No | |
| Storeys: | 3 | |
| Max Building Area: | Facing 1 Street: | 1,600 m2 |
| | Facing 2 Streets: | 2,000 m2 |
| | Facing 3 Streets: | 2,400 m2 |
| Construction: | Combustible | |
| Floor Assemblies: | 45 min FRR | |
| Mezzanines: | 45 min FRR | |
| Roof Assemblies: | 45 min FRR | |
| Loadbearing Walls, Columns and Arches: | 45 min FRR or noncombustible const. | |

**3.2.2.66 Group E, Up To 3 Storeys**

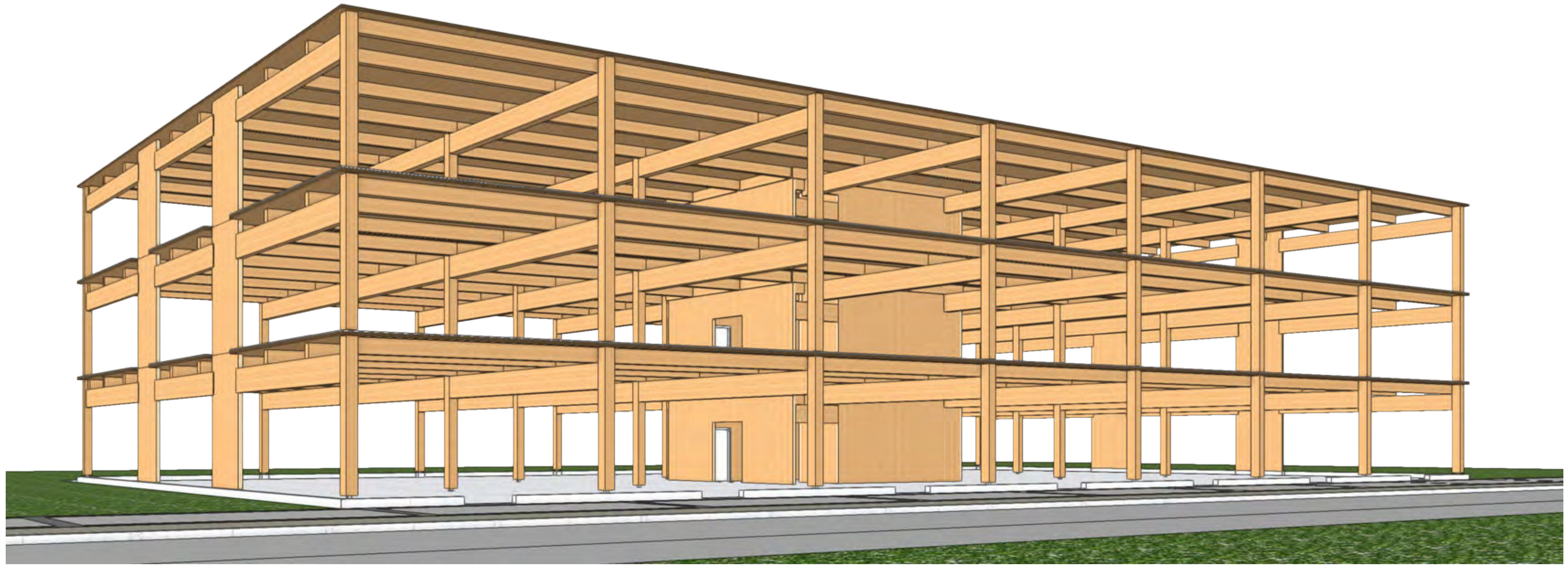
| | | |
|---|---|---|
| Sprinklered (Y/N): | No | |
| Storeys: | 3 | |
| Max Building Area (m2): | Facing 1 Street: | 800 m2 |
| | Facing 2 Streets: | 1,000 m2 |
| | Facing 3 Streets: | 1,500 m2 |
| Construction: | Combustible | |
| Floor Assemblies: | 45 min FRR | |
| Mezzanines: | 45 min FRR | |
| Roof Assemblies: | 45 min FRR | |
| Loadbearing Walls, Columns and Arches: | 45 min FRR or noncombustible const. | |

**3.2.2.61 Group D, Up To 3 Storeys, Sprinklered**

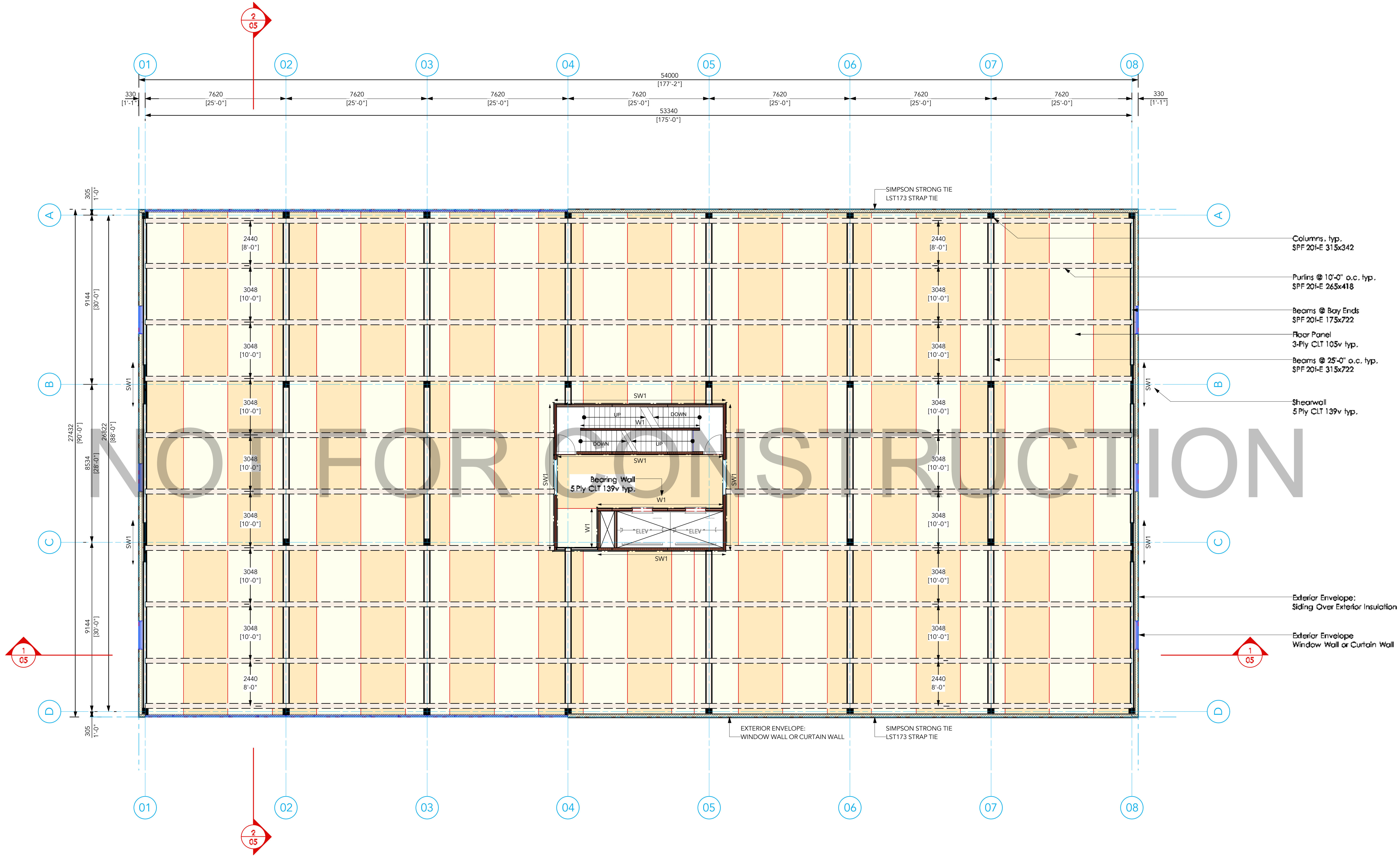
| | |
|---|---|
| Sprinklered (Y/N): | Yes |
| Storeys: | 3 |
| Max Building Area (m2): | 4,800 m2 |
| Construction: | Combustible |
| Floor Assemblies: | 45 min FRR |
| Mezzanines: | 45 min FRR |
| Loadbearing Walls, Columns and Arches: | 45 min FRR or noncombustible const. |

**3.2.2.67 Group D, Up To 3 Storeys, Sprinklered**

| | |
|---|---|
| Sprinklered (Y/N): | Yes |
| Storeys: | 3 |
| Max Building Area (m2): | 4,800 m2 |
| Construction: | Combustible |
| Floor Assemblies: | 45 min FRR |
| Mezzanines: | 45 min FRR |
| Loadbearing Walls, Columns and Arches: | 45 min FRR or noncombustible const. |








aka architecture + design

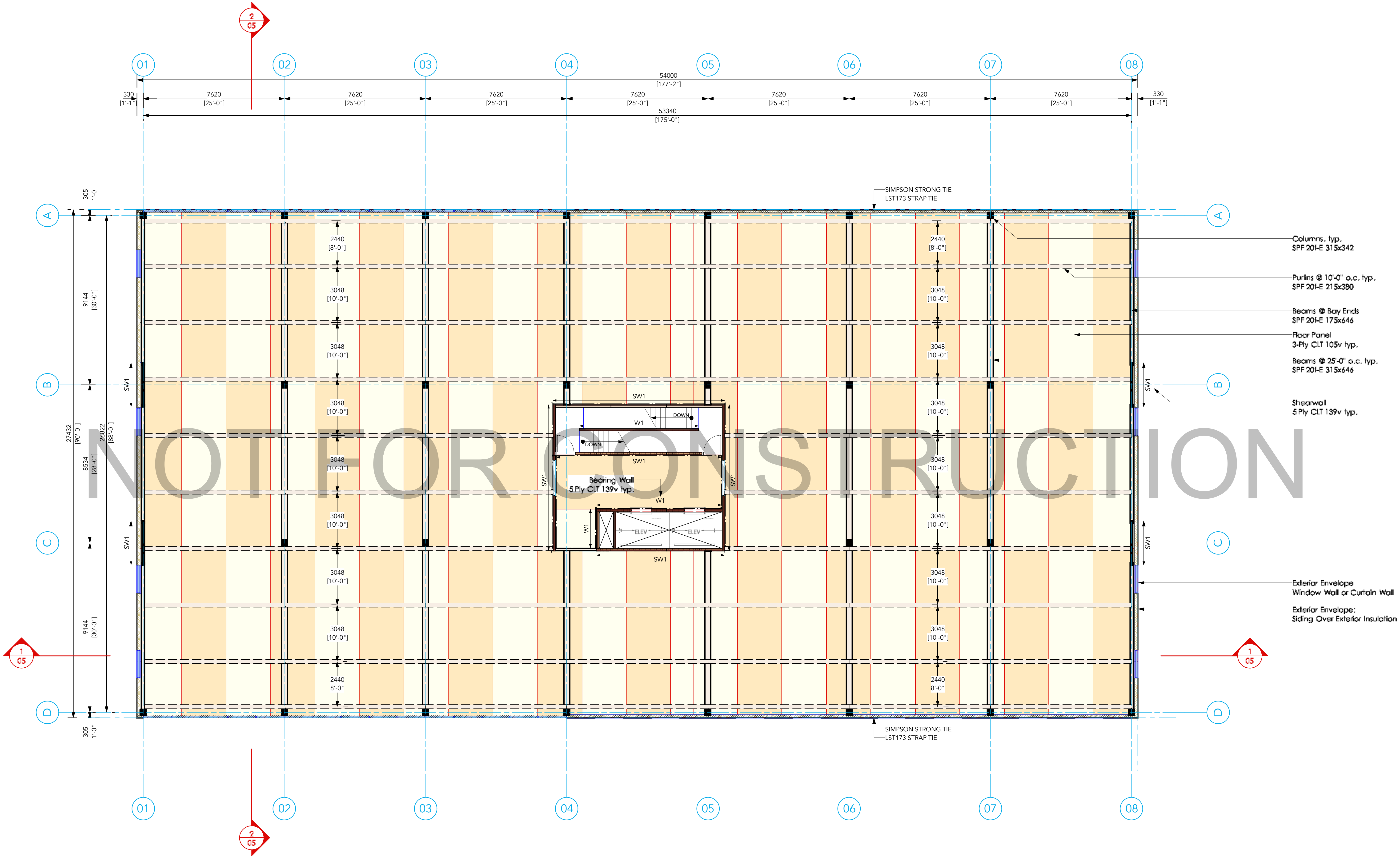
**Fast + Epp**

CWC Low-Rise Commercial Wood Building Initiative

***Mass Timber System No.1: Panel Floor / Purlin / Beam***

Review 2019-08-23

**Plan showing Typ. Floor Framing Above**




a|k|a architecture + design

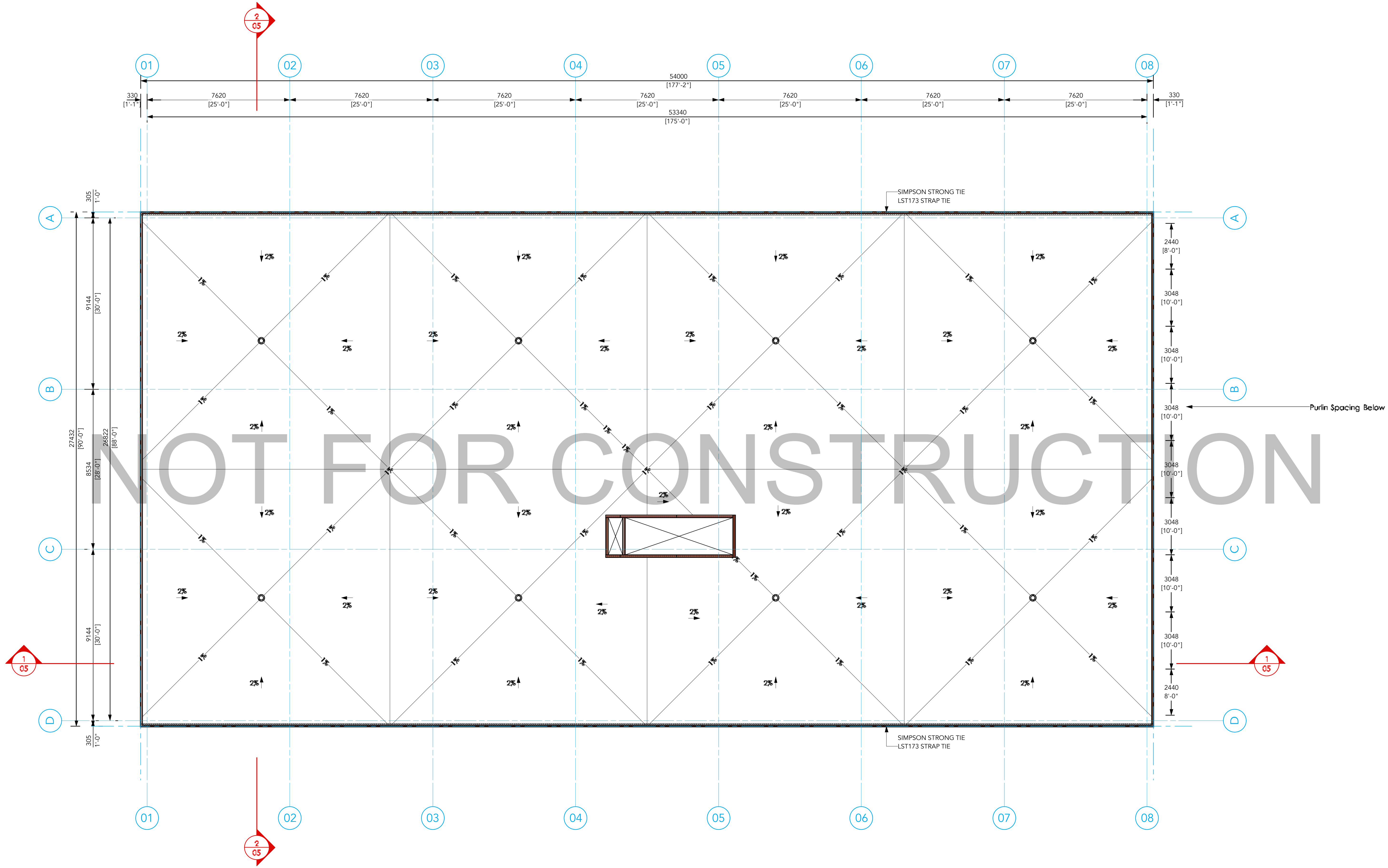
**Fast + Epp**

CWC Low-Rise Commercial Wood Building Initiative

***Mass Timber System No.1: Panel Floor / Purlin / Beam***

Review 2019-08-23

**Plan Showing Roof Framing Above**




a|k|a architecture + design

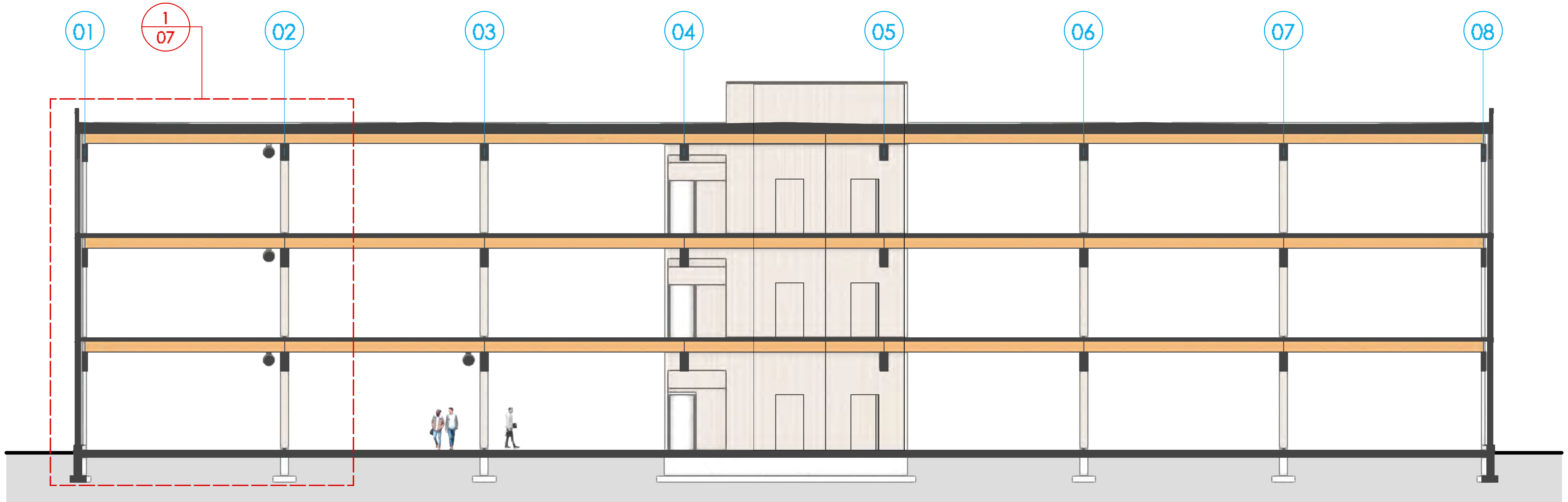
**Fast + Epp**

CWC Low-Rise Commercial Wood Building Initiative

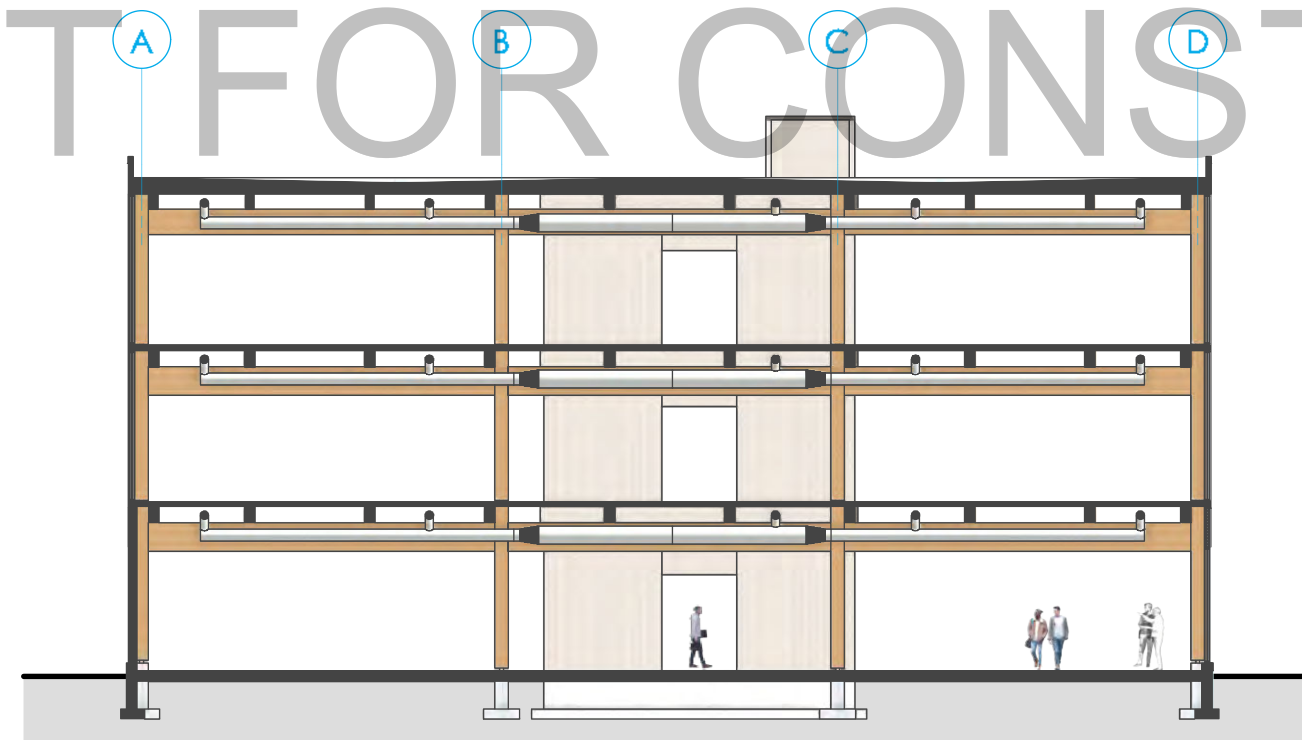
***Mass Timber System No.1: Panel Floor / Purlin / Beam***

Review 2019-08-23

**Roof Plan**

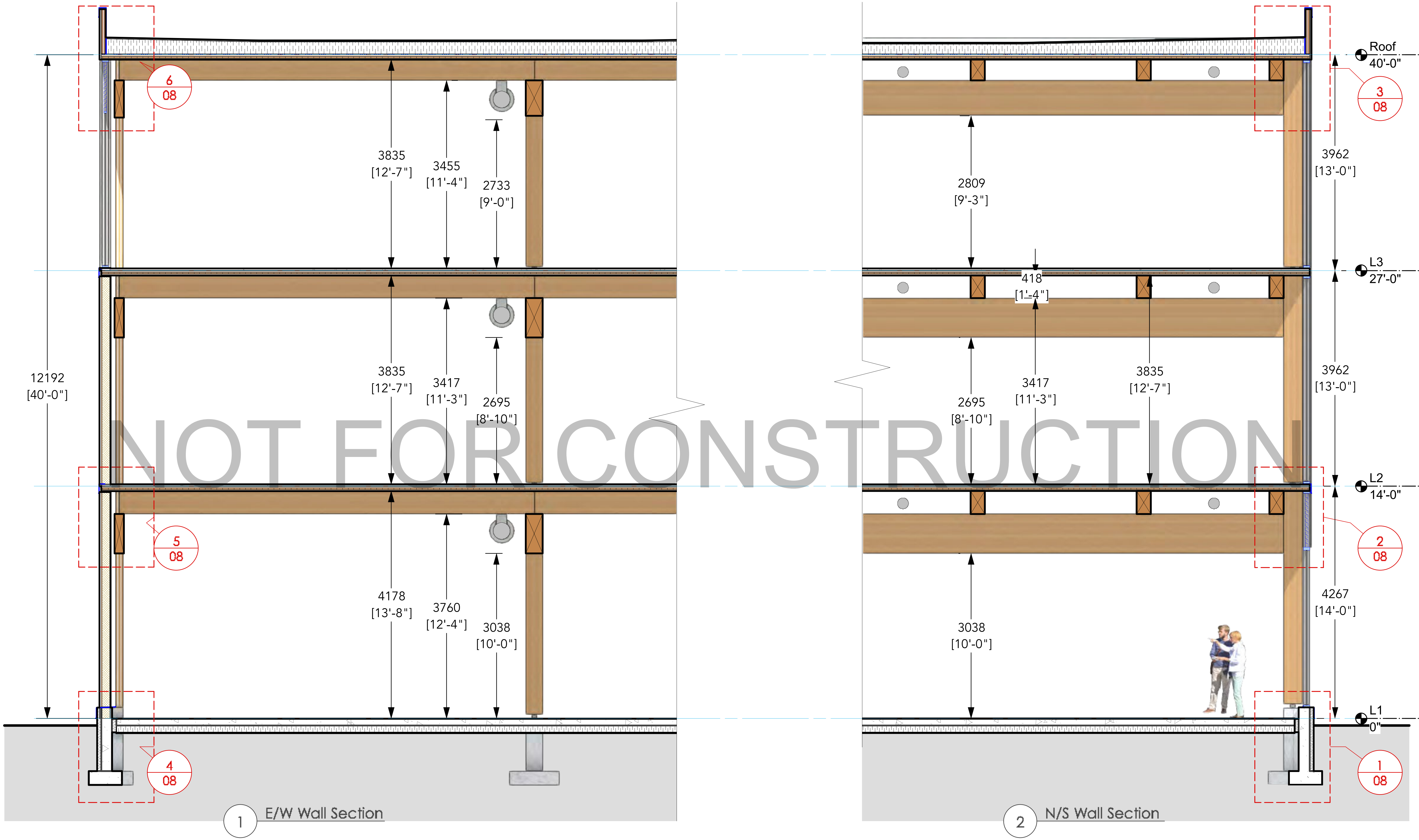



7 E/W Building Section




2 N/S Building Section



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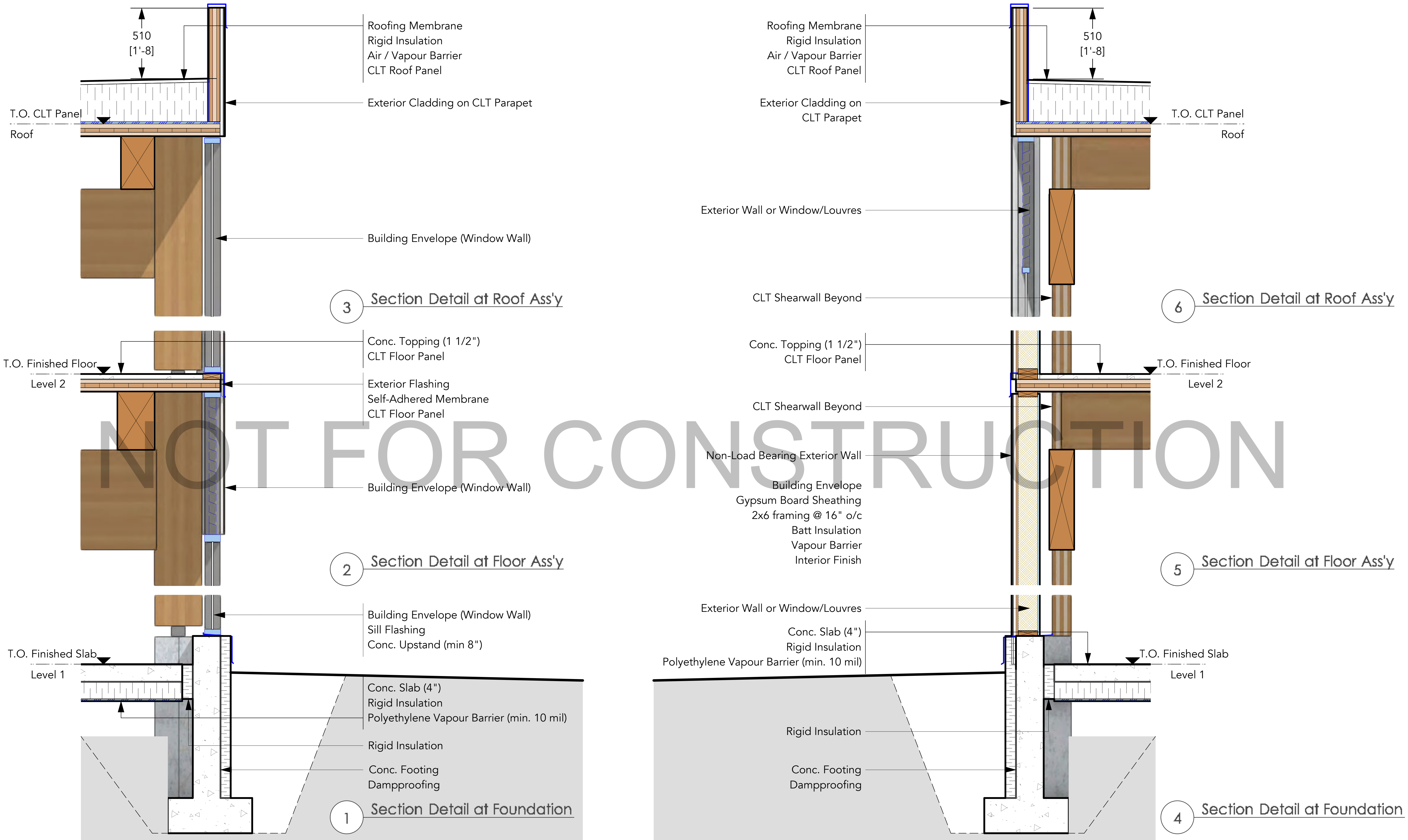
**Fast + Epp**

CWC Low-Rise Commercial Wood Building Initiative

***Mass Timber System No.1: Panel Floor / Purlin / Beam***

Review 2019-08-23

## Wall Sections

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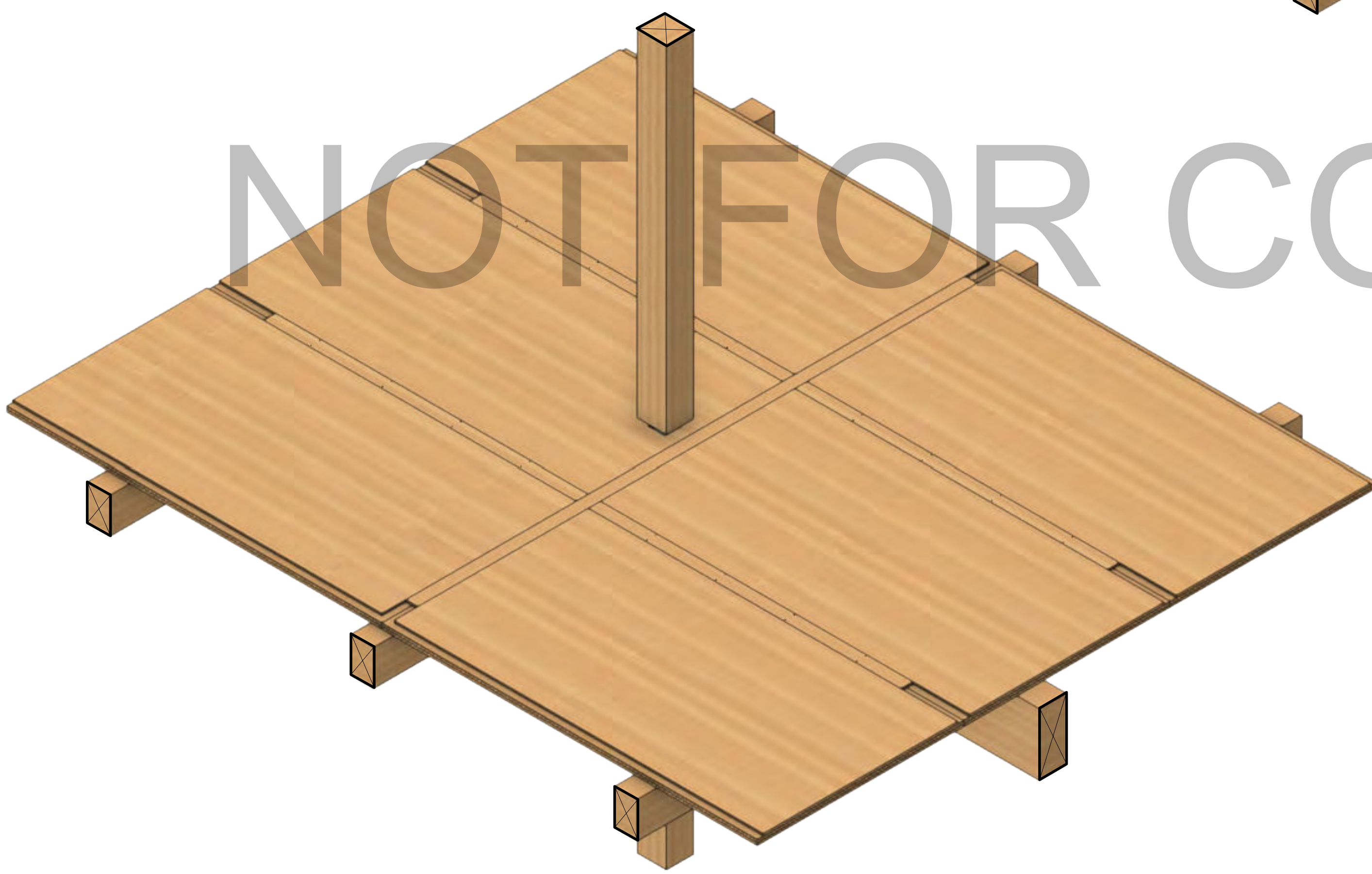
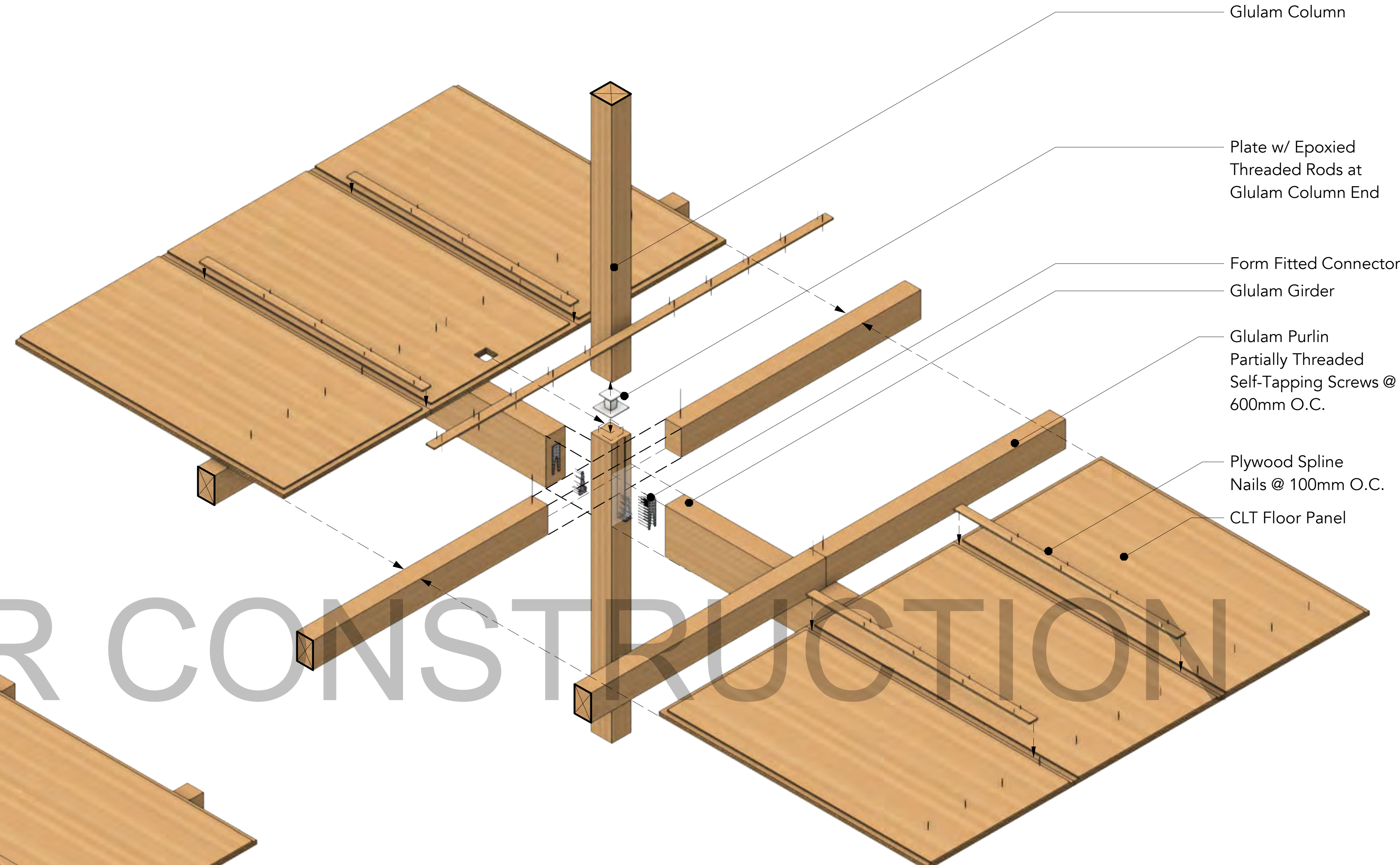
***Mass Timber System No.1: Panel Floor / Purlin / Beam***

Review 2019-08-23

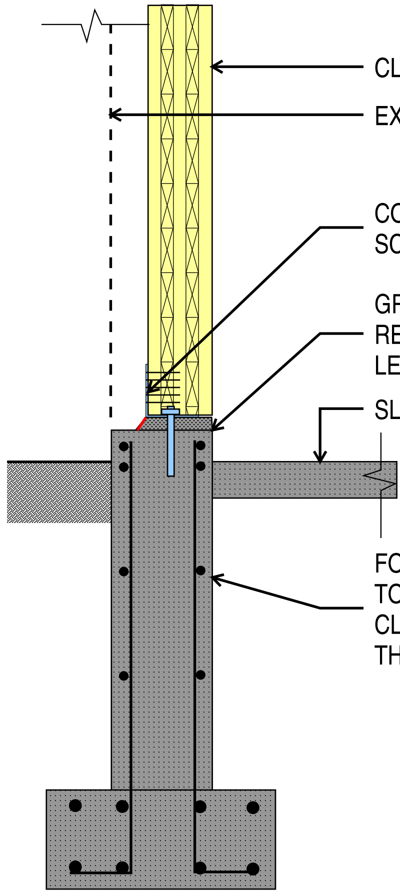
**Details**



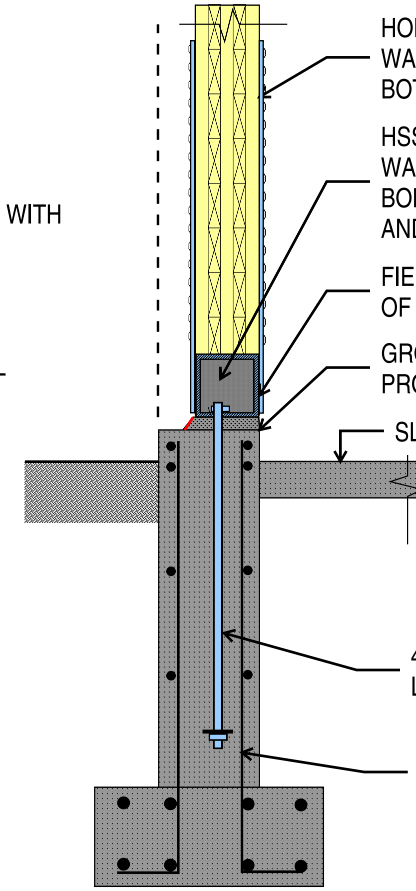
1 Isometric Detail at Floor Ass'y



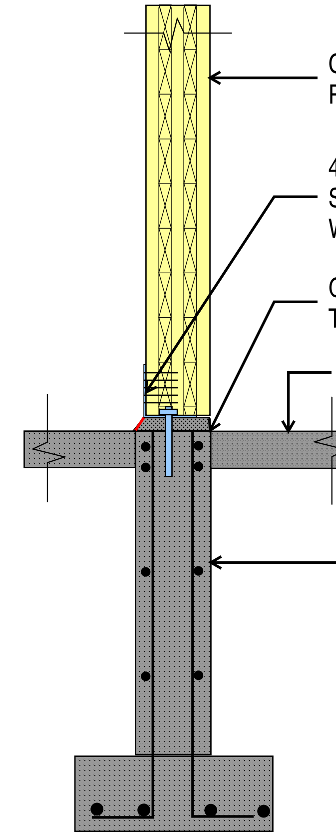

2 Exploded Isometric Detail at Floor Ass'y



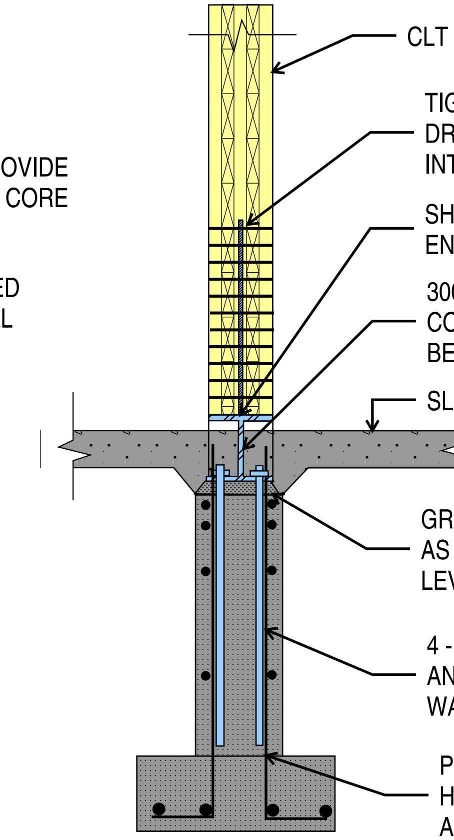

1 - TYPICAL EXTERIOR WALL BASE




2 - TYPICAL EXTERIOR WALL HOLD-DOWN

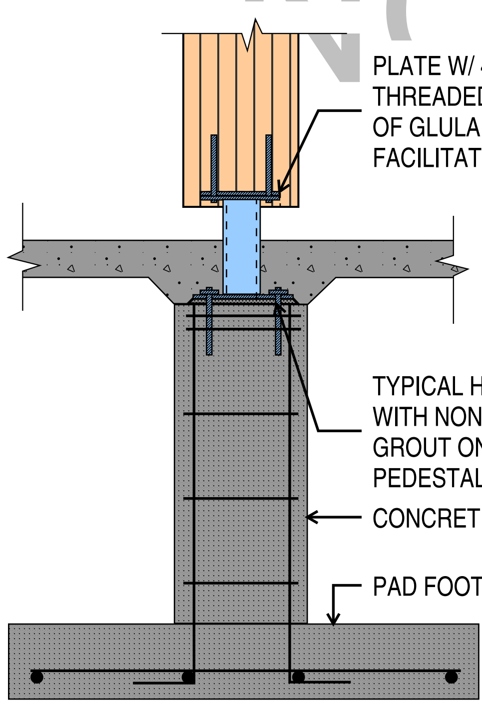



3 - TYPICAL CORE WALL BASE

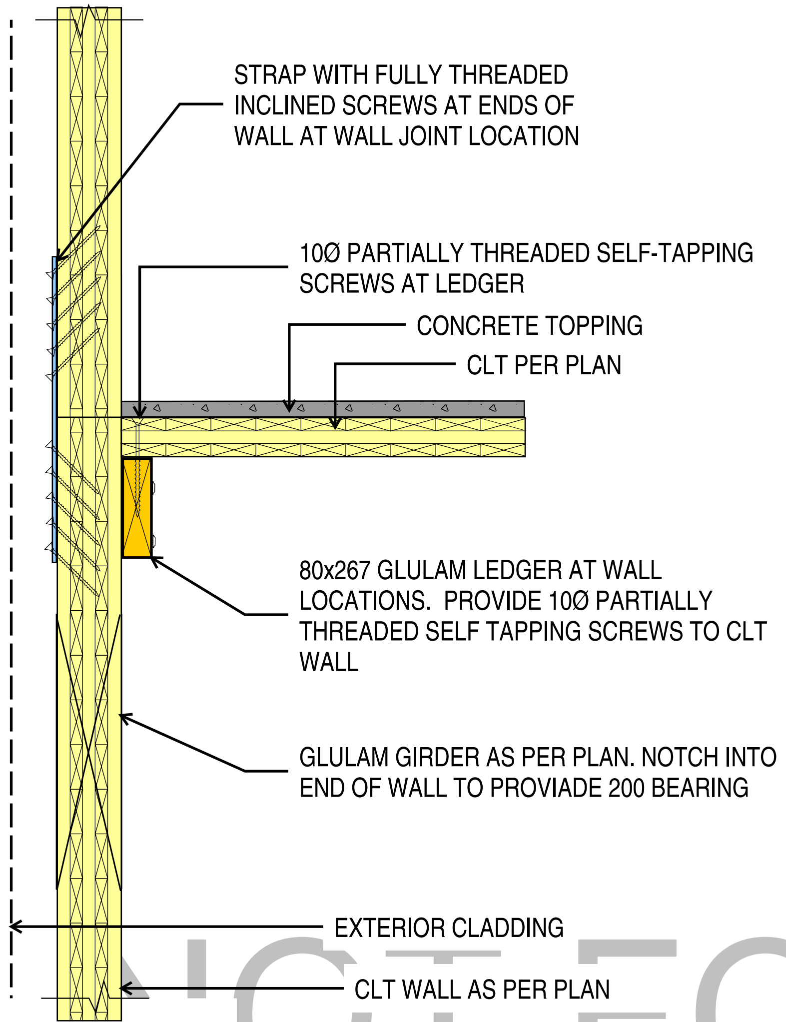



4 - TYPICAL CORE WALL HOLD-DOWN

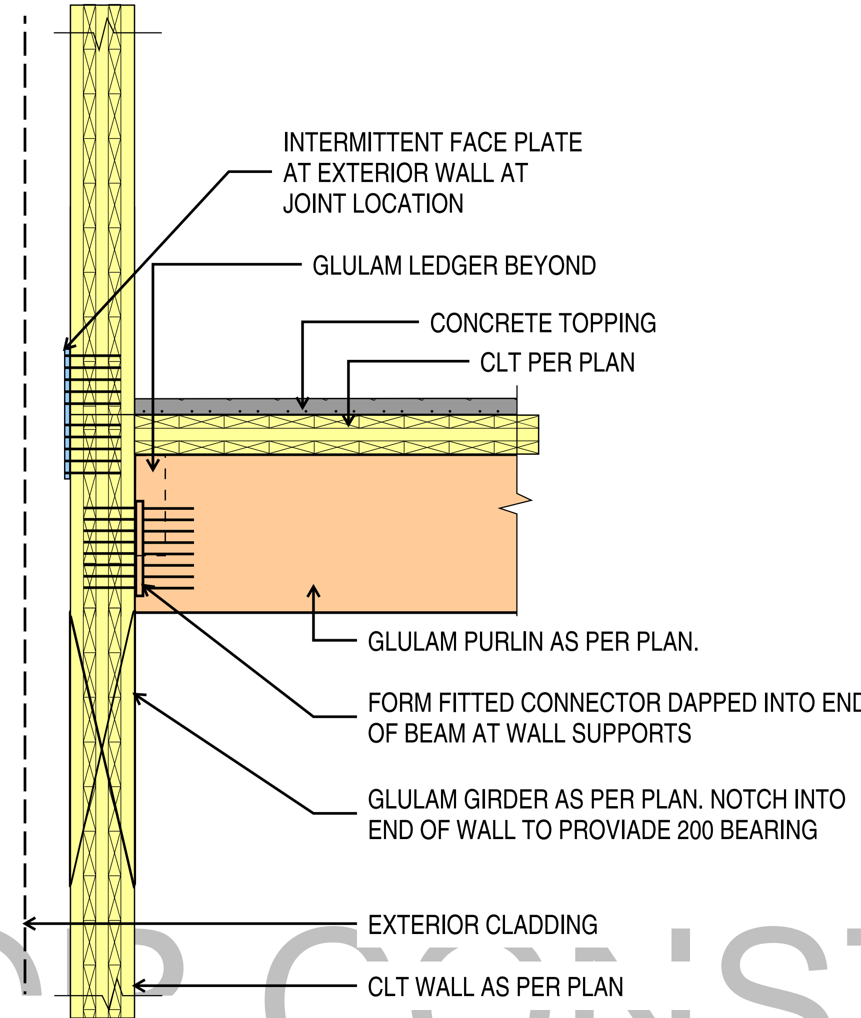




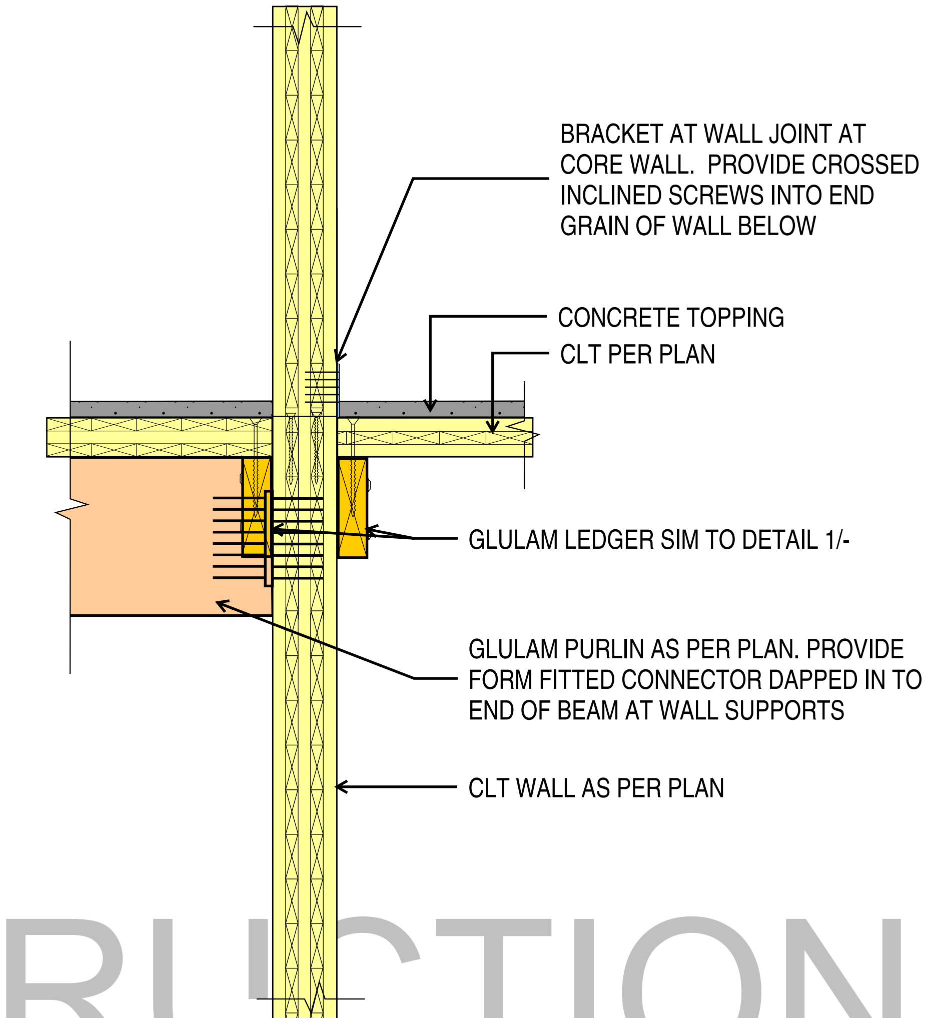

5 - TYPICAL COLUMN BASE



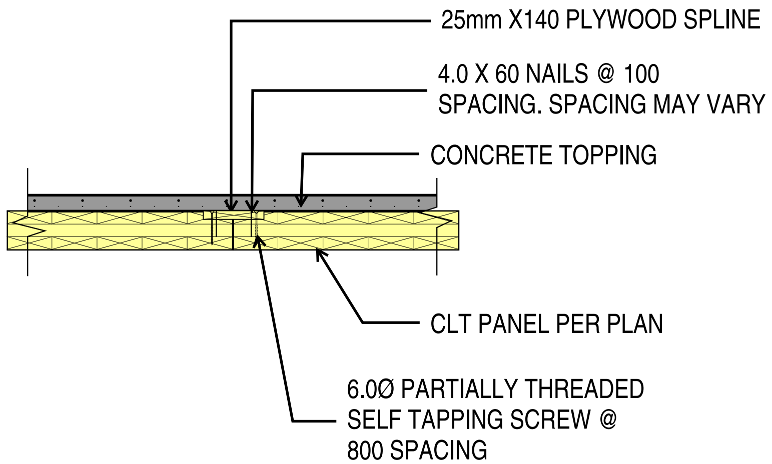

1
-
TYPICAL WALL JOINT HOLD-DOWN AT EXTERIOR WALL



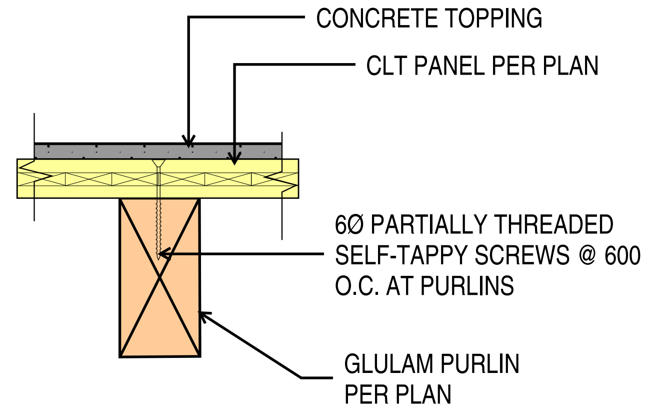

2
-
TYPICAL WALL JOINT CONNECTION AT EXTERIOR WALL



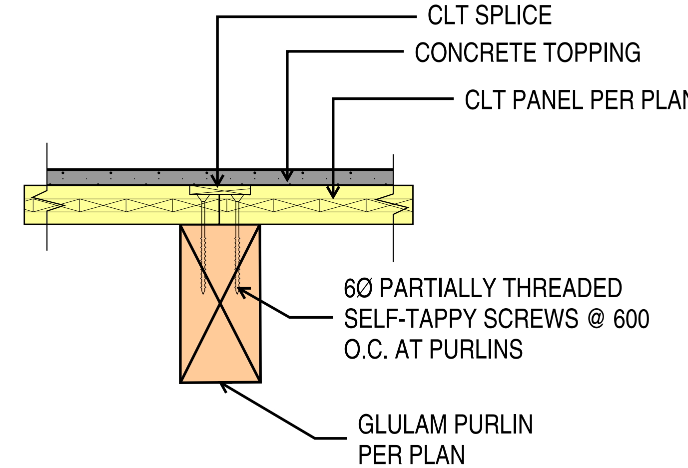

3
-
TYPICAL WALL JOINT CONNECTION AT CORE WALL



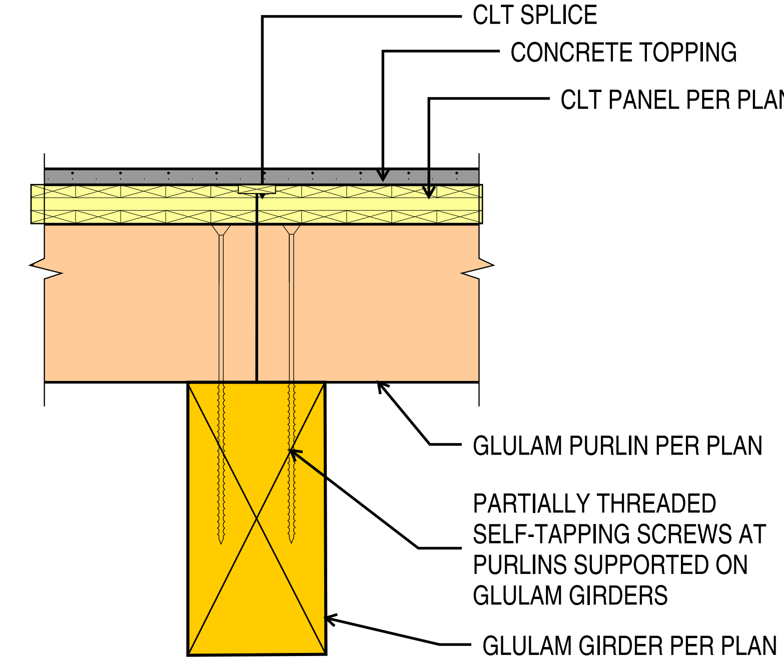

1
-
TYPICAL PANEL SPLINE CONNECTION




2
-
TYPICAL PANEL-PURLIN CONNECTION

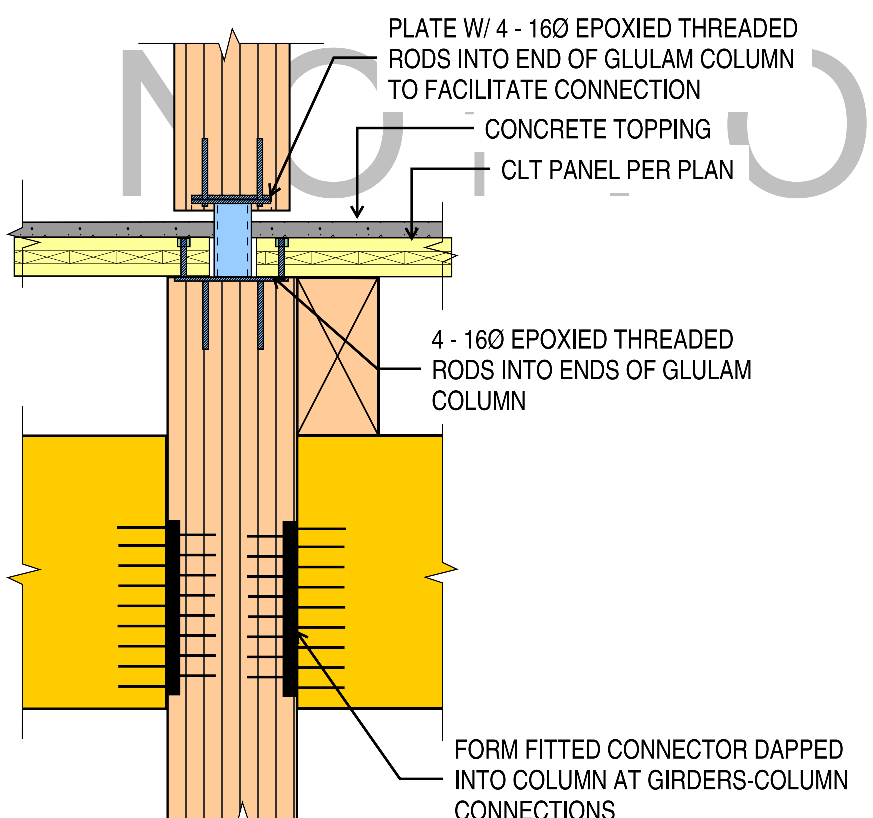



3
-
TYPICAL PANEL-PURLIN CONNECTION AT SPLINES




4
-
TYPICAL PURLIN-GIRDER CONNECTION






5
-
TYPICAL GIRDER-COLUMN CONNECTION

# Mass Timber Western Project

**Mass Timber Low-Rise Commercial**

**Block #1 Project Identification**

| | | | |
|---|---|---|---|
| **Project Name:** | | **Useage:** | First Storey Retail, Second and Third storey Office |
| **Code Classifications:** | Group E (Merchantile)<br>Group D (Office / Personal Services) | | |
| **Number of Storeys:** | 3 | **Footprint:** | 15,945 sf 1,481 m2 |
| **Structural System:** | Glulam Post and Beam, CLT Lateral System, CLT Floor System | | |

**Block #2 Project Description**

| | | | |
|---|---|---|---|
| **Building Size:** | 47,835 sf 4,443 m2 | **Building height:** | 3 Storeys |
| **Fire Rating:** | 45 min | **Maximum footprint:** | Unsprinklered: 1,500 m2<br>Sprinklered: 4,800 m2 |
| **Roof loading and deflection:** | Dead Load=1.5 KPa<br>Snow Load=2.3 Kpa<br>Snow Load Deflection=L/240<br>Total Load Deflection=L/180 | **Floor loading and deflection:** | Dead Load=3.0 KPa<br>Live Load=2.4 Kpa<br>Live Load Defledtion=L/420<br>Total Load Deflection=L/180 |
| **Structual system description:** | Panel-Purlin System, 25' x 30' typical bay | | |
| **SFRS Description (indicate RdRo factor used):** | CLT Shearwalls - Rd=2.0, Ro=1.5<br>Site Class | | |

**NBC 2015 Analysis**

**3.2.2.60 Group D, Up To 3 Storeys**

| | | |
|---|---|---|
| Sprinklered (Y/N): | No | |
| Storeys: | 3 | |
| Max Building Area: | Facing 1 Street: | 1,600 m2 |
| | Facing 2 Streets: | 2,000 m2 |
| | Facing 3 Streets: | 2,400 m2 |
| Construction: | Combustible | |
| Floor Assemblies: | 45 min FRR | |
| Mezzanines: | 45 min FRR | |
| Roof Assemblies: | 45 min FRR | |
| Loadbearing Walls, Columns and Arches: | 45 min FRR or noncombustible const. | |

**3.2.2.66 Group E, Up To 3 Storeys**

| | | |
|---|---|---|
| Sprinklered (Y/N): | No | |
| Storeys: | 3 | |
| Max Building Area (m2): | Facing 1 Street: | 800 m2 |
| | Facing 2 Streets: | 1,000 m2 |
| | Facing 3 Streets: | 1,500 m2 |
| Construction: | Combustible | |
| Floor Assemblies: | 45 min FRR | |
| Mezzanines: | 45 min FRR | |
| Roof Assemblies: | 45 min FRR | |
| Loadbearing Walls, Columns and Arches: | 45 min FRR or noncombustible const. | |

**3.2.2.61 Group D, Up To 3 Storeys, Sprinklered**

| | |
|---|---|
| Sprinklered (Y/N): | Yes |
| Storeys: | 3 |
| Max Building Area (m2): | 4,800 m2 |
| Construction: | Combustible |
| Floor Assemblies: | 45 min FRR |
| Mezzanines: | 45 min FRR |
| Loadbearing Walls, Columns and Arches: | 45 min FRR or noncombustible const. |

**3.2.2.67 Group D, Up To 3 Storeys, Sprinklered**

| | |
|---|---|
| Sprinklered (Y/N): | Yes |
| Storeys: | 3 |
| Max Building Area (m2): | 4,800 m2 |
| Construction: | Combustible |
| Floor Assemblies: | 45 min FRR |
| Mezzanines: | 45 min FRR |
| Loadbearing Walls, Columns and Arches: | 45 min FRR or noncombustible const. |

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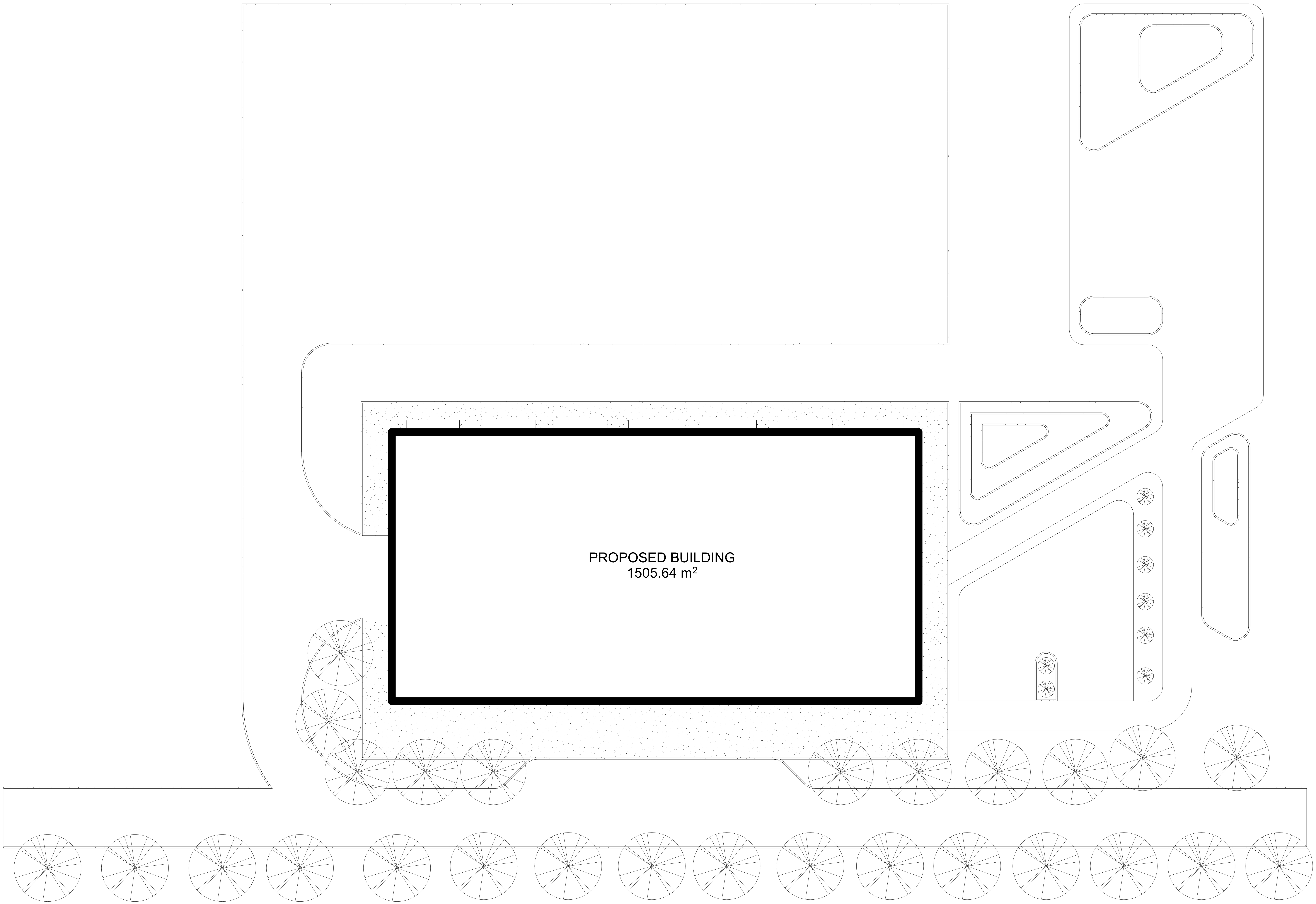
MASS TIMBER WESTERN PROJECT

COVER SHEET

| | |
|---|---|
| Project number | 0001 |
| Date | JULY 5, 2020 |
| Checked by | Hoda Ganji |

A100

Scale




1 SITE PLAN
1 : 200

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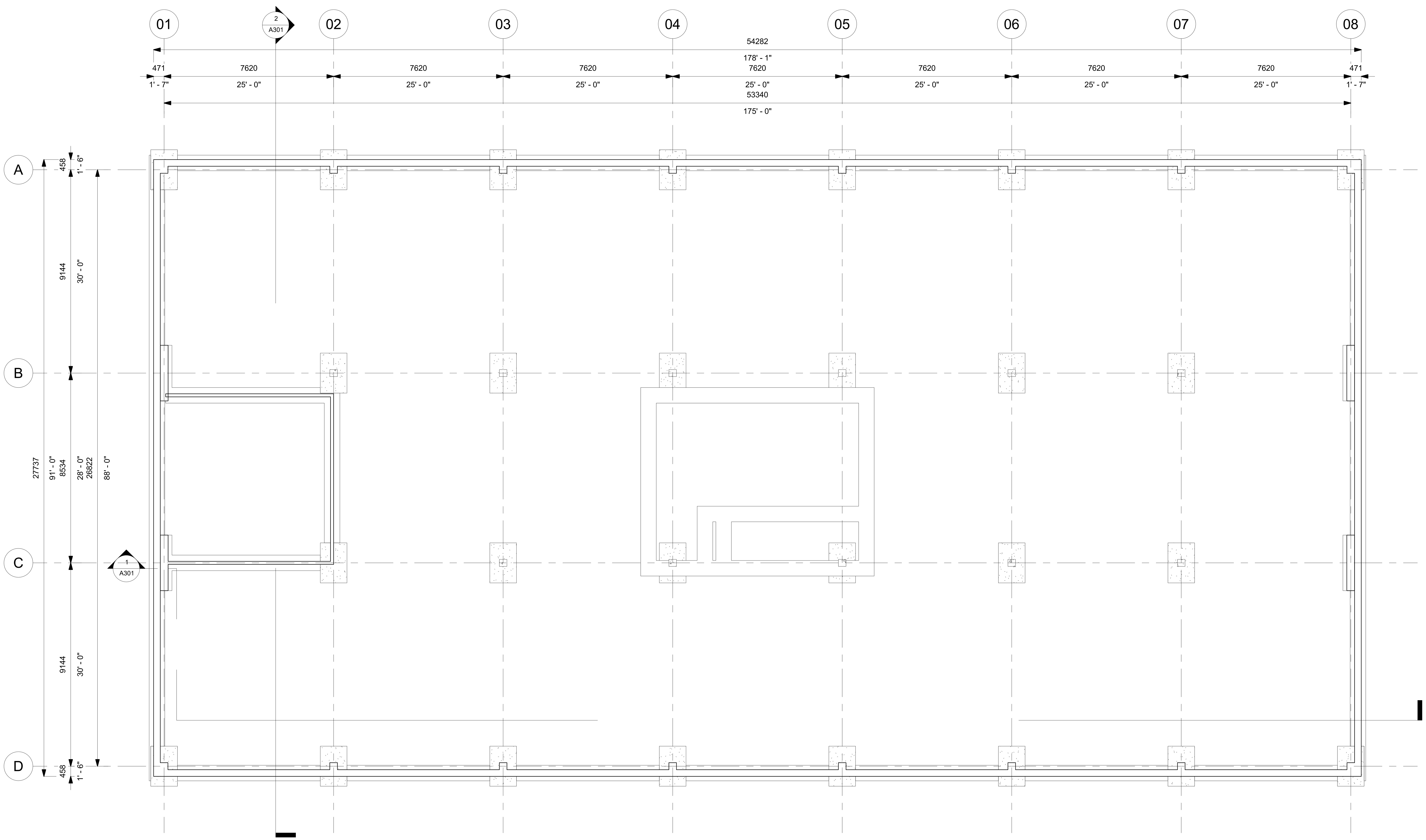
MASS TIMBER WESTERN PROJECT

SITE PLAN

| | |
|---|---|
| Project number | 0001 |
| Date | JULY 5, 2020 |
| Checked by | Hoda Ganji |

A101

| | |
|---|---|
| Scale | 1 : 200 |




1 FOUNDATION PLAN
1 : 100

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FOUNDATION PLAN

| | |
|---|---|
| Project number | 0001 |
| Date | JULY 5, 2020 |
| Checked by | Hoda Ganji |
| | A102 |
| Scale | 1 : 100 |


1 L1 PLAN
1 : 100

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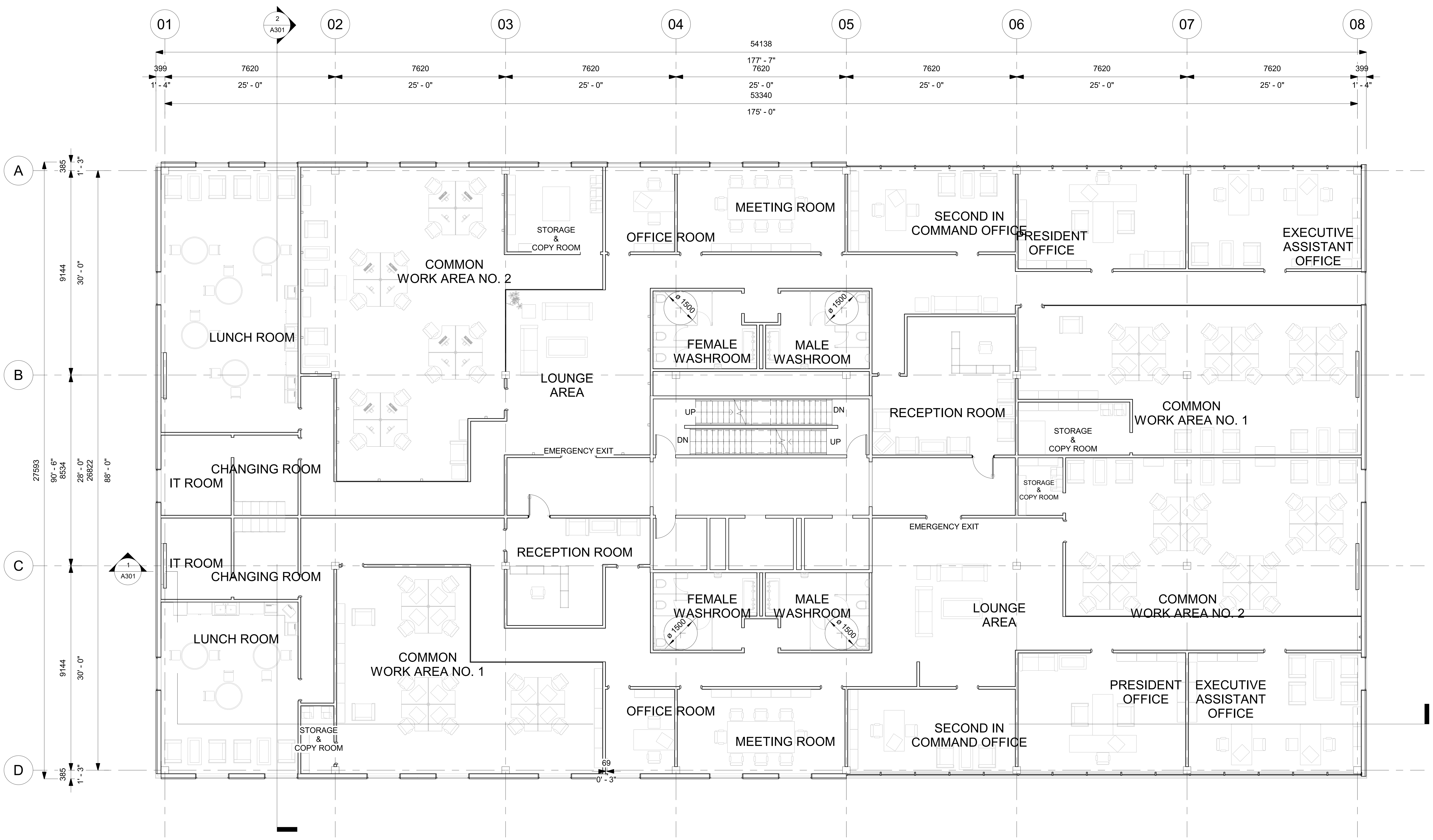
MASS TIMBER WESTERN PROJECT

FIRST FLOOR PLAN

| | |
|---|---|
| Project number | 0001 |
| Date | JULY 5, 2020 |
| Checked by | Hoda Ganji |
| | A103 |
| Scale | 1 : 100 |




1 L2 PLAN
1 : 100

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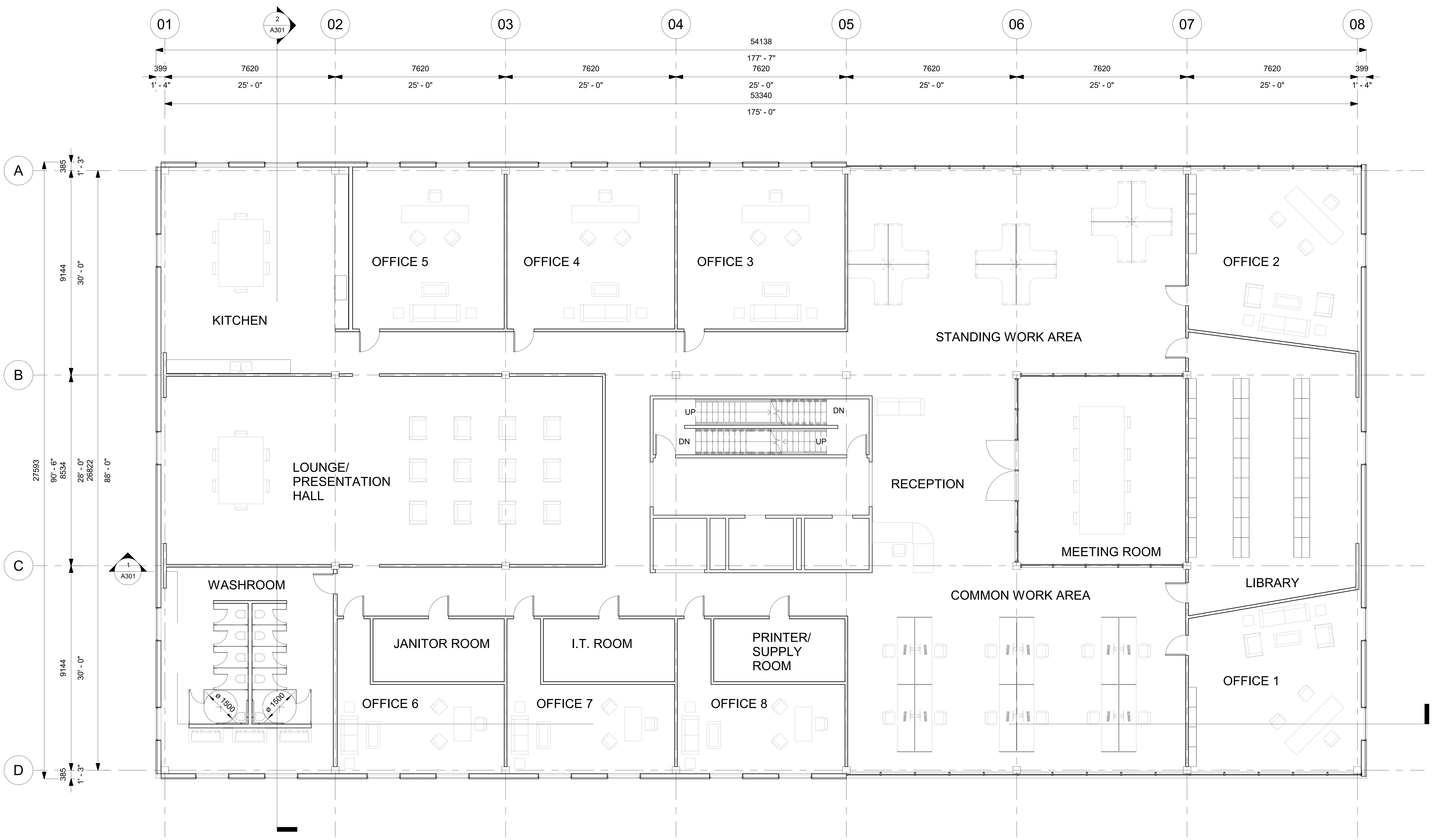
MASS TIMBER
WESTERN
PROJECT

SECOND FLOOR PLAN

| | |
|---|---|
| Project number | 0001 |
| Date | JULY 5, 2020 |
| Checked by | Hoda Ganji |
| A104 | |
| Scale | 1 : 100 |




1 L3 PLAN
1 : 100

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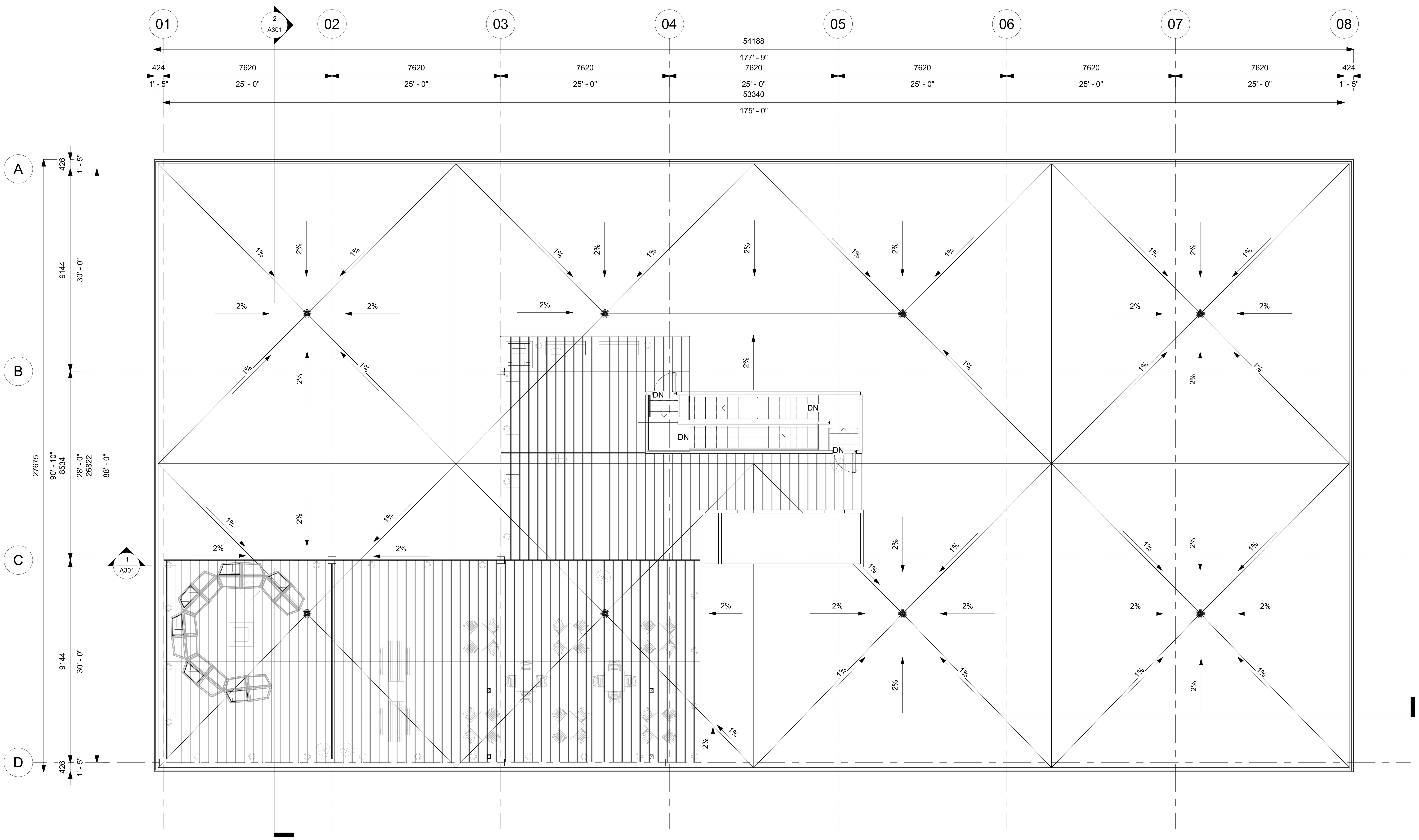
# MASS TIMBER WESTERN PROJECT

## THIRD FLOOR PLAN

| | |
|---|---|
| Project number | 0001 |
| Date | JULY 5, 2020 |
| Checked by | Hoda Ganji |
| | A105 |
| Scale | 1 : 100 |




1 ROOF PLAN
1 : 100

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Architectural Technology Program (T109)

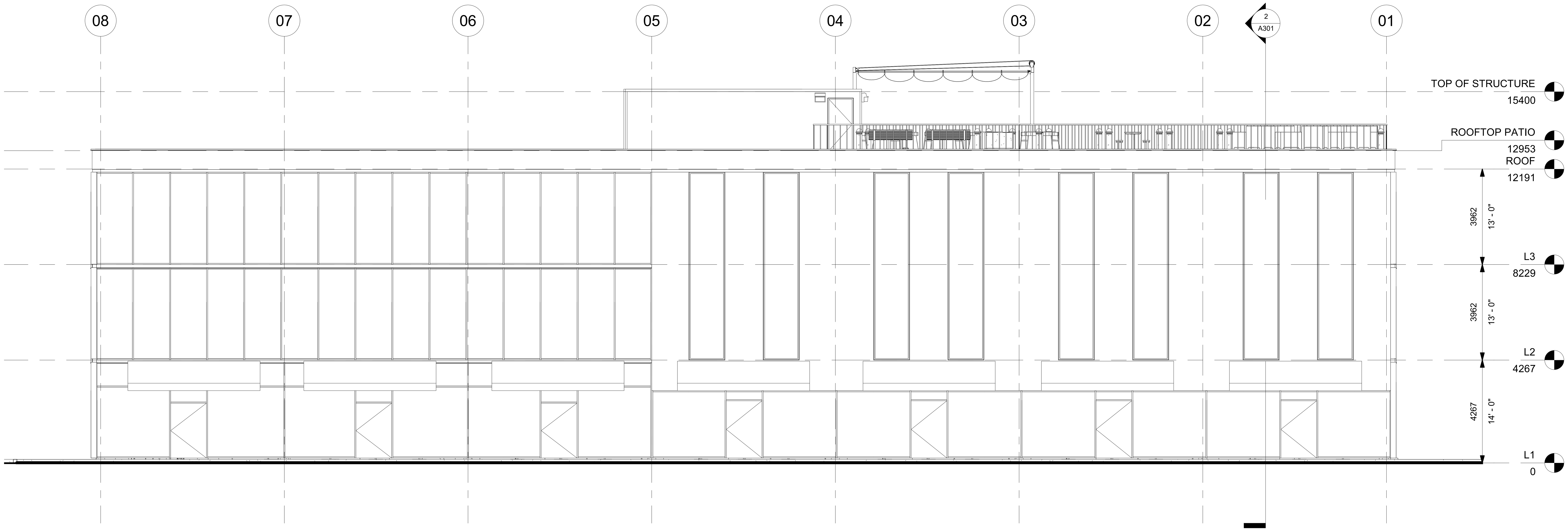
CADE3002 Spring 2020

Class of 2021
Co-op Students

MASS TIMBER WESTERN PROJECT

ROOF PLAN

| | |
|---|---|
| Project number | 0001 |
| Date | JULY 5, 2020 |
| Checked by | Hoda Ganji |
| | A106 |
| Scale | 1 : 100 |




1 NORTH ELEVATION
1 : 100


2 SOUTH ELEVATION
1 : 100

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MASS TIMBER WESTERN PROJECT

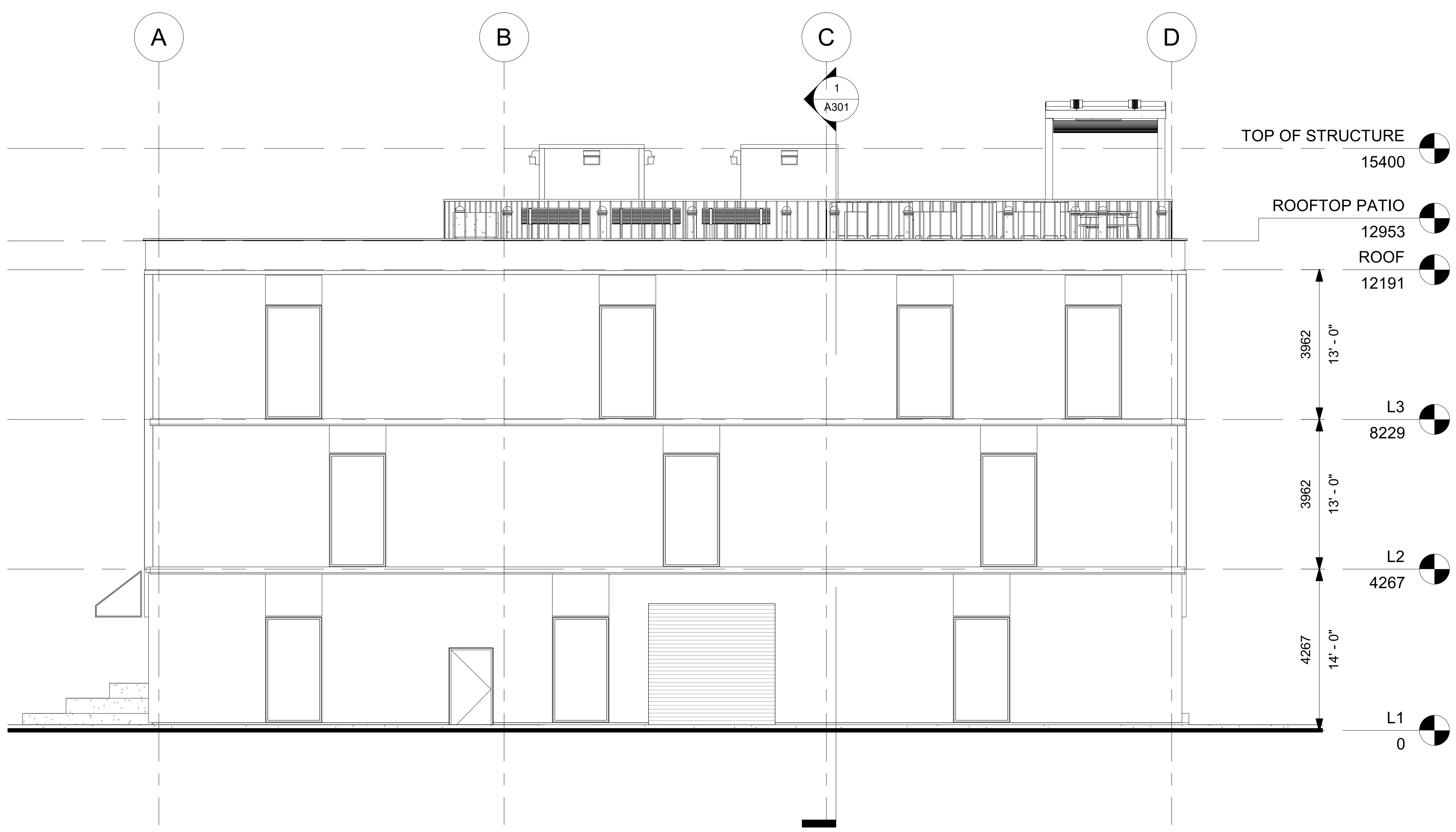
ELEVATIONS

| | |
|---|---|
| Project number | 0001 |
| Date | JULY 5, 2020 |
| Checked by | Hoda Ganji |

A201

| | |
|---|---|
| Scale | 1 : 100 |


1 EAST ELEVATION
1 : 100




2 WEST ELEVATION
1 : 100

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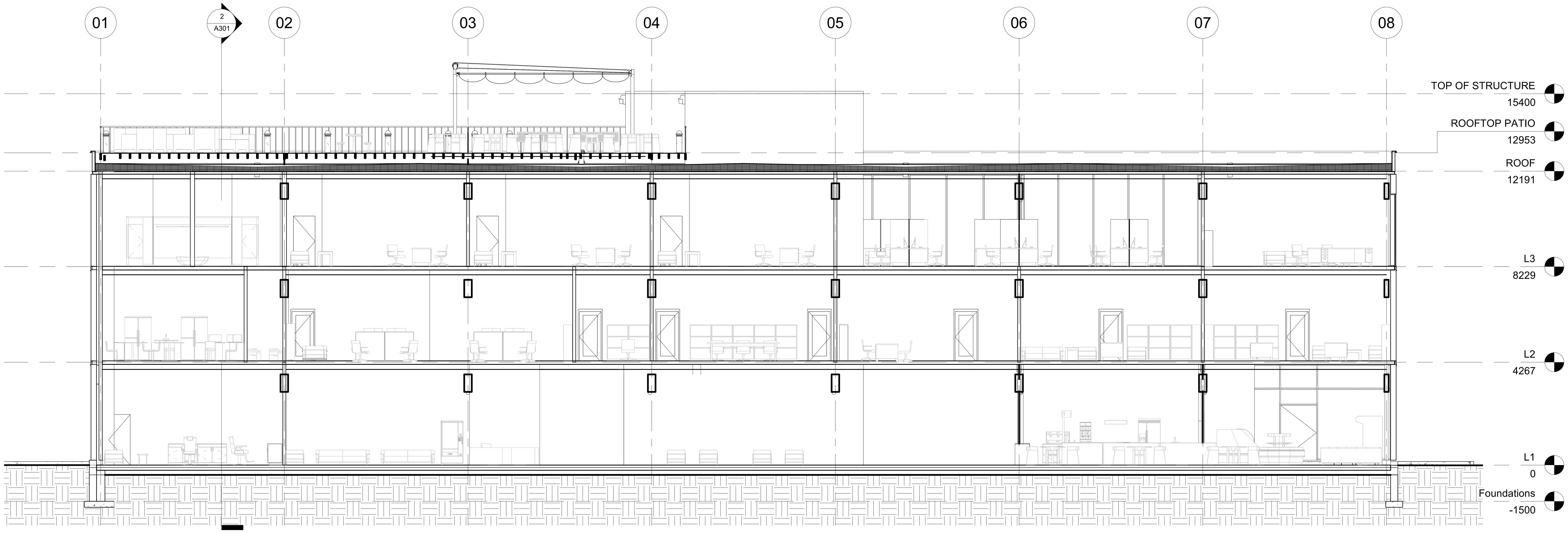
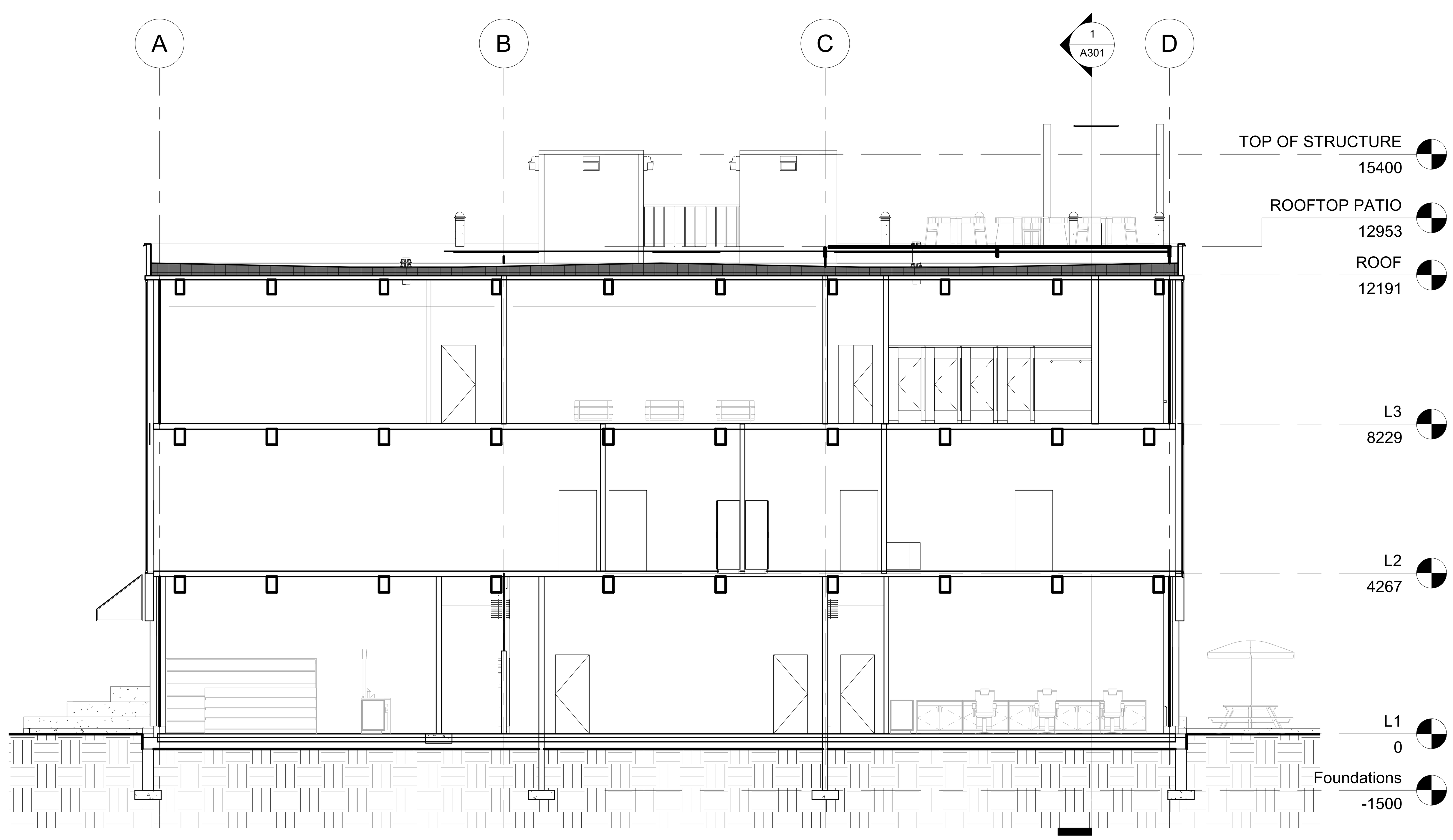
MASS TIMBER WESTERN PROJECT

ELEVATIONS

| | |
|---|---|
| Project number | 0001 |
| Date | JULY 5, 2020 |
| Checked by | Hoda Ganji |
| A202 | |
| Scale | 1 : 100 |




1 E/W BUILDING SECTION
1 : 100




2 N/S BUILDING SECTION
1 : 100

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MASS TIMBER WESTERN PROJECT

BUILDING SECTIONS

| | |
|---|---|
| Project number | 0001 |
| Date | JULY 5, 2020 |
| Checked by | Hoda Ganji |
| | A301 |
| Scale | 1 : 100 |

NOT FOR CONSTRUCTION

EXTERIOR VIEW FROM SOUTH-EAST

EXTERIOR VIEW FROM SOUTH-WEST

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EXTERIOR 3D VIEWS

| | |
|---|---|
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A501

Scale

# NOT FOR CONSTRUCTION

EXTERIOR VIEW FROM NORTH

EXTERIOR VIEW FROM WEST

BIRD'S EYE VIEW OF THE BUILDING

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EXTERIOR 3D VIEWS

| | |
|---|---|
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A502

Scale

NOT FOR CONSTRUCTION


L1 CAFETERIA VIEW

**LINKS TO PANORAMA IMAGES:**

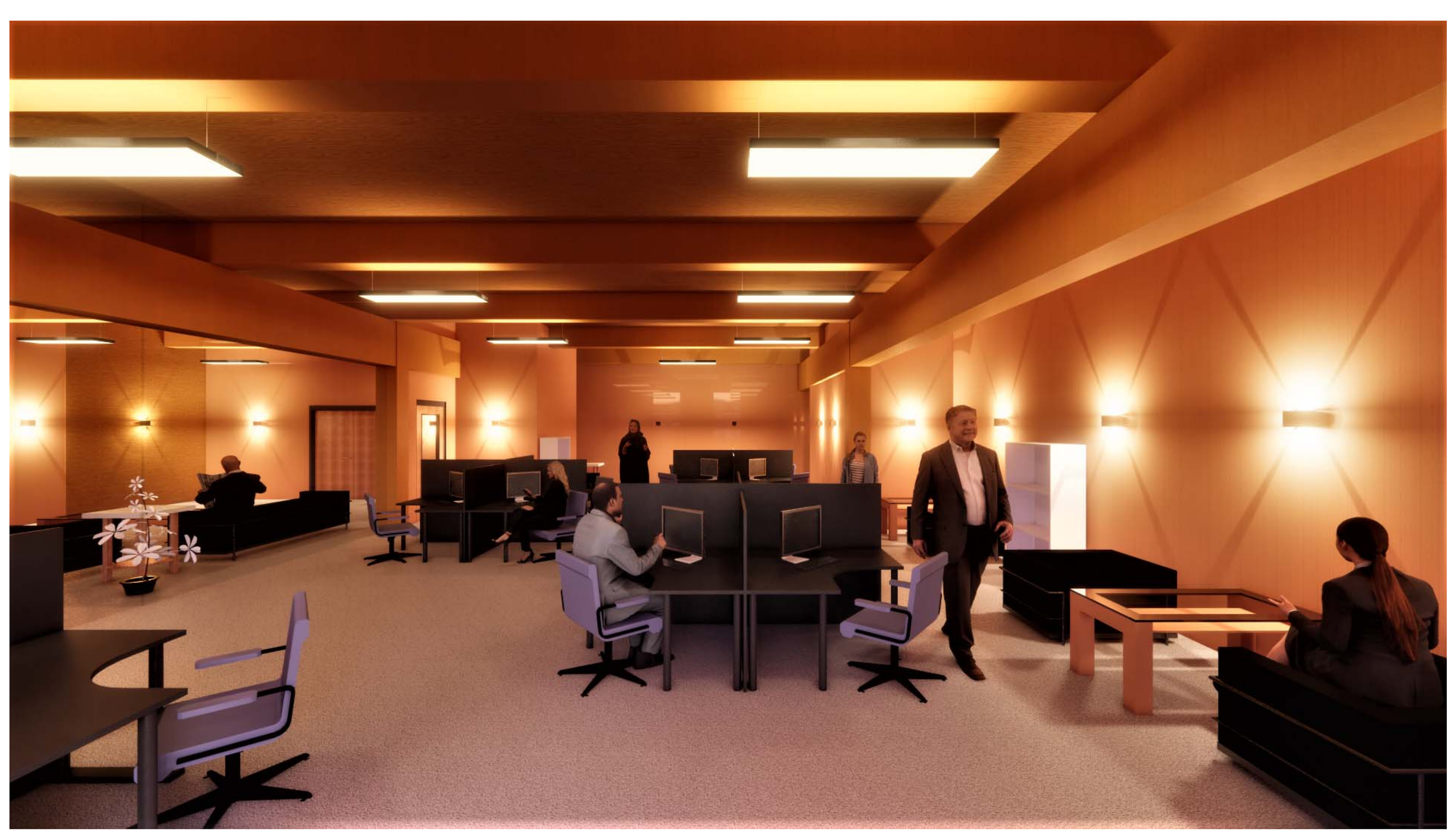
https://pano.autodesk.com/pano.html?mono=jpgs/b2d324a1-0b96-47b8-aaf5-2d79a6122ce9&version=2

https://pano.autodesk.com/pano.html?url=jpgs/1a8f482e-ce4b-4bf2-b0e8-1bb8ac9e59d0&version=2

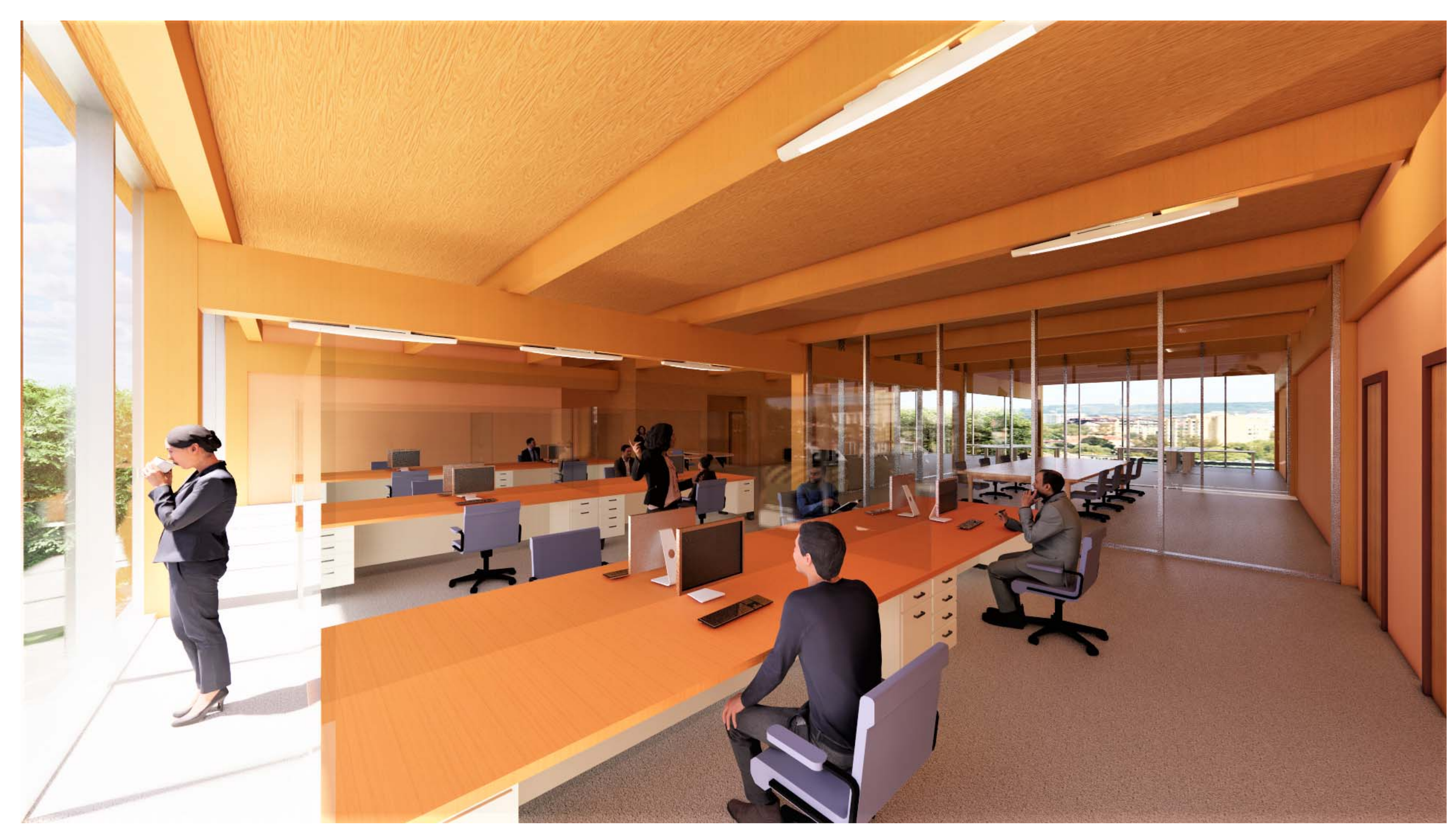
https://pano.autodesk.com/pano.html?url=jpgs/dcfec427-42ff-47fd-a279-187ddfdcd332&version=2

L1 LOBBY

L1 CAFETERIA



L3 OFFICE



L3 OFFICE

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MASS TIMBER WESTERN PROJECT

INTERIOR 3D VIEWS

| | |
|---|---|
| Project number | 0001 |
| Date | JULY 5, 2020 |
| Checked by | Checker |

A503

Scale

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MASS TIMBER WESTERN PROJECT

DRAWN BY

| | |
|---|---|
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X001

Scale