A Gravity System Calculations

A.1 Introduction

Included in this Appendix are all the calculations completed for both the existing gravity system and the wood redesign gravity system. These calculations are provided to show more specifically what was done to reach the design choices and conclusions.

A.2 Existing Gravity System

Calculations determining loads in the existing gravity system follow. The methods and process used for determining the gravity loads is described in chapter 1.

Tech Report 2 | Roof Loads | Samantha deVries

Roof Dead Load

Penthouse Roof: Load (psf)

Joist/Beam Allowance	10
Roof Decking	10
Roofing System	7
	$\boxed{27\ psf}$

12^{th} Floor Terrace:

Concrete / Deck	37
Joist / Beam Allowance	10
4" rigid insulation	3
Drop Ceiling	5
MEP	15
Sprinklers	3
Pavers or Tiles	25
	$\boxed{98\ psf}$

Roof Live Load

Penthouse Roof:

Code minimum is 20 psf
(Table 4-1: Ordinary flat roofs)

Use $\boxed{30\ psf}$ (value used in design)

12^{th} Floor Terrace:

Table 4-1: Roofs used for assemby purposes

Use $\boxed{100\ psf}$ (same as design value)

* Note: drawings indicate that snow load must be used instead as the live load where it is the larger value

Figure A.1: Roof Load Calculations

Tech Report 2 | Snow Load | Samantha deVries

ASCE 7-05 : Chapter 7

Section 7.3 : Flat Roof Snow Loads

$p_f = 0.7\, C_e C_t I\, p_g$

$p_g = 25$ psf (Figure 7-1)

$C_e = 0.9$ (Table 7-2) Terrain Category B
Roof Fully Exposed

$C_t = 1.0$ (Table 7-3)

$I = 1.0$ (Table 7-4) Use w/ importance Category II

$p_f = 0.7(0.9)(1.0)(1.0)(25) = 15.8$ psf

min. where $p_g > 20$ psf
$p_f = 20\,(I) = 20\,(1.0)$

$\boxed{p_f = 20 \text{ psf}}$ (Design snow load = 20 psf)
< 30 psf LL on Penthouse Roof

Snow Drift Section 7.7 : Drifts on Lower Roofs

$\gamma = 0.13\,p_g + 14 = 0.13(25) + 14 = 17.25$

$h_b = p_s/\gamma = 15.8/17.25 = 0.916$

$h_c = 15' \rightarrow h_c/h_b = 16.4 > 0.2$ (must calc. drift)

$l_{upper\ roof} = 128'$ $l_{lower\ roof} = 40'$

leeward drift (Fig. 7-9 w/ 128')
$h_d = 3.75' \rightarrow$ use larger value
windward drift (Fig. 7-9 w/ 40')
$h_d = 2.0'$

$h_d < h_c = 15$, so $w = 4h_d = 4\,(3.75) = 15'$

$p_d = h_d\gamma = 3.75(17.25) = \boxed{64.7 \text{ psf}}$ < 100 psf LL on level 12

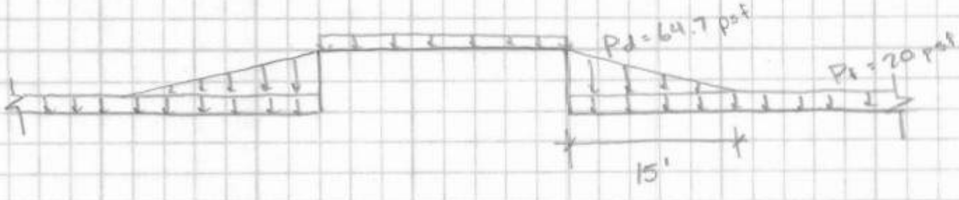


Figure A.2: Snow Load and Drift Calculations

Tech Report 2 | Floor Loads | Samantha deVries

Floor Dead Loads

Concrete Floor	Load (psf)
Drop Ceiling	5
MEP	15
Sprinklers	3
Concrete 6 1/2"	81.25
or 8" × 150 pcf	100

6 1/2" slab : $\boxed{105 \text{ psf}}$
8" slab : $\boxed{123 \text{ psf}}$

Steel Framed Floors	
Ceiling	5
MEP	15
Sprinklers	3
Beam / Joist Allowance	15
Concrete / Deck	37

$\boxed{75 \text{ psf}}$

Floor Live Loads

Area	Code Min. (psf)	Design Value
Residential	40	40
Lobbies / Stairs / Exits	100	100
Penthouse Floor	100	100
Lobby Floor	100	100
Corridors above 1st Floor	40	40
12th Floor Corridors	40	100
Parking	40	40

Note: Residential Areas also receive a 20 psf partition Allowance

Figure A.3: Floor Load Calculations

Tech Report 2 | Exterior Wall Loads | Samantha deVries

Typical Existing Building Wall Dead Load:

Applied as a line load at the edge of the slab

8" Brick Layer (assume hard brick)

130 pcf × 8/12 = 87 psf × 11' typ. = 957 plf

3/4" layer gypsum board

50 pcf × $^{0.75}/_{12}$ × 11' = 34.4 plf

Total = $\boxed{992 \text{ plf}}$

Typical Addition Wall Dead Load:

Composite Metal Panel

5 psf × 11' = 55 plf

CMU Infill (or Brick facade w/out metal panel)

29 psf (CMU) × 11' = 319 plf   or   38 psf (brick medium weight) × 11' = 418 plf

Water Membrane

2 psf × 11' = 22 plf

3/4" gypsum board = 34.4 plf

Fibrous glass insulation

1.1 psf × 11 = 12.1 plf

Total: at metal panels = $\boxed{443 \text{ plf}}$

at brick faces = $\boxed{487 \text{ plf}}$

Figure A.4: Exterior Wall Load Calculations

Tech Report 2 | Gravity Loads | Samantha deVries

Non-Typical Dead Loads

Floors & Roofs:

At 3/4" drop panels (7' x 7') existing building

3/4" x 150 pcf = $\boxed{9 \text{ psf}}$

Existing Building Perimeter Beams

12" x 150 pcf x 12" width (avg.) = $\boxed{150 \text{ plf}}$

16" depth = $\boxed{200 \text{ plf}}$

18" = $\boxed{225 \text{ plf}}$

24" = $\boxed{300 \text{ plf}}$

30" = $\boxed{375 \text{ plf}}$

(Note: there is a large variety of perimeter beam sizes, so this is a sample to provide a range of additional Load)

Figure A.5: Non-Typical Load Calculations

A.3 Wood Redesign

A.3.1 CLT Panel Calculations

Included below are the excel tables used to determine final CLT panel sizes. These calculations follow the methods and process described in chapter 3.

Typical CLT Floor Panel Design							
Strength Checks							
Level	**Span**	**Panel**	**FbSeff***	**D+L***	**Cd**	**M**	**Ok?**
Typical Level	20.8	5-ply	10400	76	1	4090.3	good
12th Level	20.8	7-ply	18375	140	1	7534.8	good
Penthouse Roof	20.8	5-ply	10400	66	1	3552.1	good
*9-ply would have higher FbSeff, however value was not tabulated and 7-ply value worked,							

Deflection Checks											
Level	**Span**	**Panel**	**EI**	**D**	**L**	**Defl L**	**Defl D+L**	**L limit**	**D+L limt**	**L OK?**	**D OK?**
Typical Level	20.8	5-ply	4.40E+08	36	40	0.38	1.03	0.69	1.04	good	good
12th Level	20.8	7-ply	1.09E+09	40	100	0.38	0.69	0.69	1.04	good	good
Penthouse Roof	20.8	5-ply	4.40E+08	36	30	0.28	0.97	0.69	1.04	good	good

Fire Design Check									
Level	**Span**	**Panel**	**Orig. h**	**Resid. H**	**Approx**	**FbSeff**	**D+L***	**M**	**OK?**
Typical Level	26	5-ply	9.625	7.125	5-ply	10400	43	3634	good
12th Level	26	7-ply	12.375	9.875	7-ply	18375	70	5915	good
Penthouse Roof	26	5-ply	9.625	7.125	5-ply	10400	39	3296	good
*D+L is reduced using the same assumptions as before									

Table A.1: CLT Panel Design for Typical bay

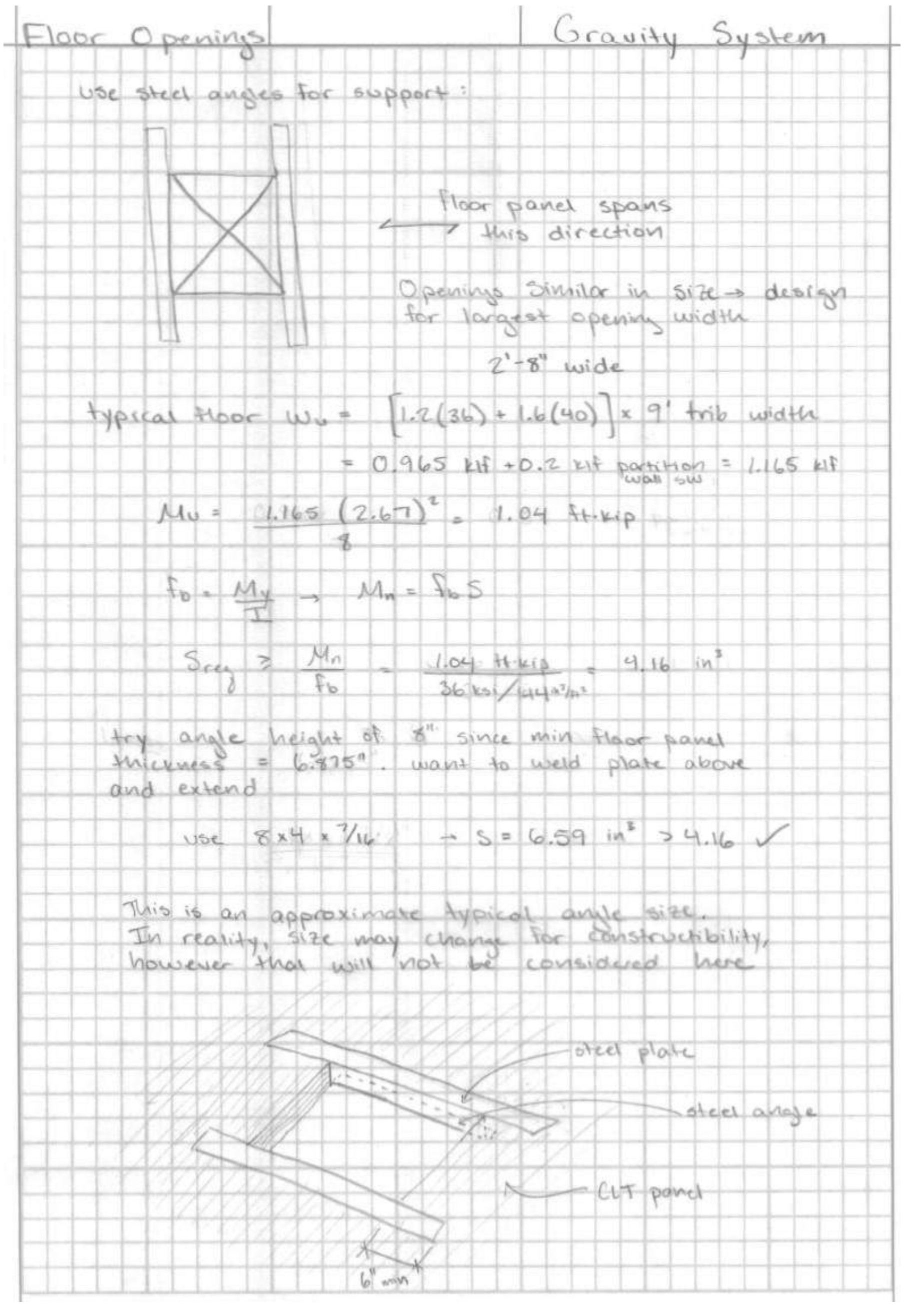
Non Typical CLT Floor Panel Design							
Strength Checks							
Level	**Span**	**Panel**	**FbSeff***	**D+L***	**Cd**	**M**	**Ok?**
Typical Level	26	7-ply	18375	80	1	6760	good
12th Level	26	9-ply	18375	144	1	12168	good
Penthouse Roof	26	7-ply	18375	70	1	5915	good
*D+L controlled over other combinations *9-ply would have higher FbSeff, however value was not tabulated and 7-ply value worked, so new FbSeff was not calculated to save time							

Deflection Checks											
Level	**Span**	**Panel**	**EI**	**D**	**L**	**Defl L**	**Defl D+L**	**L limit**	**D+L limt**	**L OK?**	**D OK?**
Typical Level	26	7-ply	1.09E+09	40	40	0.38	1.13	0.87	1.30	good	good
12th Level	26	9-ply	1.60E+09	44	100	0.64	1.21	0.87	1.30	good	good
Penthouse Roof	26	7-ply	1.09E+09	40	30	0.28	1.04	0.87	1.30	good	good

Fire Design Check									
Level	**Span**	**Panel**	**Orig. h**	**Resid. H**	**Approx**	**FbSeff**	**D+L***	**M**	**OK?**
Typical Level	26	7-ply	9.625	7.125	5-ply	10400	46	3887	good
12th Level	26	9-ply	12.375	9.875	7-ply	18375	73	6169	good
Penthouse Roof	26	7-ply	9.625	7.125	5-ply	10400	42	3549	good
*D+L is reduced using the same assumptions as before									

Table A.2: CLT Panel Design for 26' bay

Typical Opening Calculation


Floor Openings | Gravity System

use steel angles for support:

floor panel spans this direction

Openings similar in size → design for largest opening width

2'-8" wide

typical floor $w_u = \left[1.2(36) + 1.6(40)\right] \times 9'$ trib width

$= 0.965$ klf $+ 0.2$ klf partition wall sw $= 1.165$ klf

$M_u = \frac{1.165\,(2.67)^2}{8} = 1.04$ ft-kip

$f_b = \frac{My}{I} \rightarrow M_n = f_b S$

$S_{req} \geq \frac{M_n}{f_b} = \frac{1.04 \text{ ft-kip}}{36 \text{ ksi} / 144 \text{ in}^2/\text{ft}^2} = 4.16 \text{ in}^3$

try angle height of 8" since min floor panel thickness = 6.875". want to weld plate above and extend

use $8 \times 4 \times 7/16$ → $S = 6.59 \text{ in}^3 > 4.16$ ✓

This is an approximate typical angle size. In reality, size may change for constructibility, however that will not be considered here

Figure A.6: Typical Opening Calculations

A.3.2 Girder Calculations

	bw	bf	dc	dt	dtc	NA	I	St	Sb	EI
Normal Conditions	4	12	6.875	27	20.125	11.44	12758.8	820.1	1115.0	2.30E+10
	4	12	6.875	25.5	18.625	10.71	10549.8	713.3	985.1	1.90E+10
	4	12	6.875	24	17.125	9.98	8623.1	615.0	864.1	1.55E+10
	4	12	6.875	22.5	15.625	9.25	6958.3	525.2	752.1	1.25E+10
	4	12	6.875	21	14.125	8.53	5535.1	443.8	649.0	9.96E+09
	4	12	6.875	19.5	12.625	7.81	4333.2	370.7	554.8	7.80E+09
	4	12	6.875	18	11.125	7.10	3332.2	305.7	469.3	6.00E+09
	4	12	6.875	16.5	9.625	6.40	2511.8	248.7	392.5	4.52E+09
	4	12	6.875	15	8.125	5.71	1851.3	199.3	324.1	3.33E+09
	4	12	6.875	13.5	6.625	5.05	1330.0	157.4	263.5	2.39E+09
	4	12	6.875	12	5.125	4.42	927.0	122.2	209.9	1.67E+09
	4	12	6.875	10.5	3.625	3.85	620.3	93.2	161.3	1.12E+09
Residual Section during Fire	4	7	6.875	24.5	17.625	11.05	6676.6	496.2	604.5	1.20E+10
	4	7	6.875	23	16.125	10.32	5478.5	431.9	531.1	9.86E+09
	4	7	6.875	21.5	14.625	9.59	4438.1	372.6	462.8	7.99E+09
	4	7	6.875	20	13.125	8.87	3543.7	318.3	399.7	6.38E+09
	4	7	6.875	18.5	11.625	8.15	2783.4	268.9	341.6	5.01E+09
	4	7	6.875	17	10.125	7.44	2145.3	224.4	288.4	3.86E+09
	4	7	6.875	15.5	8.625	6.74	1617.4	184.6	240.1	2.91E+09
	4	7	6.875	14	7.125	6.05	1187.9	149.4	196.3	2.14E+09
	4	7	6.875	12.5	5.625	5.38	844.6	118.7	156.9	1.52E+09
	4	7	6.875	11	4.125	4.75	575.3	92.0	121.2	1.04E+09
	4	7	6.875	9.5	2.625	4.16	367.4	68.8	88.3	6.61E+08
	4	7	6.875	8	1.125	3.67	207.1	47.9	56.4	3.73E+08

Table A.3: Calculated Properties for Inverted T-Beam Girders

Typical Girder Redesign for Inverted T-Shape

	Level	Span	Gird. sw	Floor L+D	D+L**	M (in-lbs)	bf	bw	Depth	Cv	Sact	Sreq	OK?
Strength Design	Typical Level	20	26	700	726	435600	12	4	15	0.90	324.1	201.1	good
	12th Level	20	50	1400	1450	870000	12	4	18	0.89	469.3	409.0	good
	Penthouse Roof	20	50	660	710	426000	12	4	15	0.90	324.1	196.7	good

	Level	Span	L (plf)	D+L	EI	Defl. L	Defl. D+L	Lim. L	Lim. D+L	L OK?	D+L OK?
Defl. Design	Typical Level	20	400	726	3.33E+09	0.432	0.96	0.667	1.0	good	good
	12th Level	20	1000	1450	6.00E+09	0.600	1.00	0.667	1.0	good	good
	Penthouse Roof	20	300	710	3.33E+09	0.324	0.989	0.667	1.0	good	good

	Level	Span	D+L	Orig w	Orig h	Resid w	Resid h	Seff	Red. Load	M (in-lb)	Sreq	OK?
Fire/ Char Design	Typical Level	20	726	12	15	7	12.5	182.3	404.5	242700	70	good
	12th Level	20	1450	12	18	7	15.5	280.3	737.5	442500	130	good
	Penthouse Roof	20	710	12	15	7	12.5	182.3	427.5	256500	74	good

*Or along Grid 4 at 12th Level and Penthouse

Table A.4: Typical Girder Design for Inverted T-Shape

Typical Perimeter Girder in the E-W Direction

	Level	Span	Wall sw	Gird. Sw	D (plf)*	M (in-lbs)	bw	Depth	Cv	Sact	Sreq	OK?
Strength Design	Typical Level	21	450	50	500	330750	12	15	0.90	324.1	153.4	good
	12th Level parapet	21	200	50	250	165375	12	13	0.91	209.9	75.6	good
	12th Level penthouse	21	350	50	400	264600	12	15	0.90	324.1	122.7	good
	Penthouse Roof	21	200	50	250	165375	12	13	0.91	209.9	75.6	good

	Level	Span	D (plf)*	EI	Defl.	Defl. Lim.	OK?
Defl. Design	Typical Level	21	500	3.33E+09	0.985	1.05	good
	12th Level parapet	21	250	1.67E+09	0.983	1.05	good
	12th Level penthouse	21	400	3.33E+09	0.788	1.05	good
	Penthouse Roof	21	250	1.67E+09	0.983	1.05	good

	Level	Span	D (plf)*	Orig w	Orig h	Resid w	Resid h	Seff	Red. Load	M (in-lb)	Sreq	OK?
Fire/ Char Design	Typical Level	21	500	12	15	7	12.5	182.3	121.2	80201	23.3	good
	12th Level parapet	21	250	12	13	7	10.5	128.6	56.4	37308	10.7	good
	12th Level penthouse	21	400	12	15	7	12.5	182.3	88.3	58422	16.9	good
	Penthouse Roof	21	250	12	13	7	10.5	128.6	56.4	37308	10.7	good

*Dead Loads here include approx. girder self-weight and exterior wall load. Floor dead and live loads are assumed to be carried to the typical floor girders by the CLT panel and are not included. Therefore there is no live on carried by this girder type.

Table A.5: Non-typical Girder Design

Perimeter Girder Along Grid 2* (West side)

	Level	Span	Wall sw	Gird. sw	Floor L+D	D+L**	M (in-lbs)	bf	Depth	Cv	Sact	Sreq	OK?
Strength Design	Typical Level	20	450	50	1092	1592	955200	12	19.5	0.88	554.8	452.7	good
	12th Level parapet	20	200	50	1872	2122	1273200	12	21	0.87	649.0	607.9	good
	12th Level penthouse	20	350	50	2880	3280	1968000	12	25.5	0.86	985.1	958.0	good
	Penthouse Roof	20	200	50	962	1212	727200	12	18	0.89	469.3	341.9	good

	Level	Span	L (plf)	D+L	EI	Defl. L	Defl. D+L	Lim. L	Lim. D+L	L OK?	D+L OK?
Defl. Design	Typical Level	20	520	1592	7.80E+09	0.240	0.982	0.667	1	good	good
	12th Level parapet	20	1300	2122	9.96E+09	0.470	0.915	0.667	1	good	good
	12th Level penthouse	20	2000	3280	1.90E+10	0.379	0.743	0.667	1	good	good
	Penthouse Roof	20	390	1212	6.00E+09	0.234	0.974	0.667	1	good	good

	Level	Span	D+L	Orig w	Orig h	Resid bf	Resid h	Resid bw	Seff	Red. Load	M (in-lb)	Sreq	OK?
Fire/ Char Design	Typical Level	20	1592	12	19.5	7	17	4	240.1	1012.0	607200	179.9	good
	12th Level parapet	20	2122	12	21	7	18.5	4	341.6	1136.5	681900	203.5	good
	12th Level penthouse	20	3280	12	25.5	7	23	4	531.1	1280.0	768000	233.7	good
	Penthouse Roof	20	1212	12	18	7	15.5	4	196.3	772.5	463500	136.2	good

*Or along Grid 4 at 12th Level and Penthouse

**D+L was the controlling case for other girders, and will therefore be the only case considered in non typical giders

Table A.6: Non-typical Girder Design

Perimeter Girder Along Grid 12* (East side)

	Level	Span	Wall sw	Gird. sw	Floor L+D	D+L**	M (in-lbs)	bf	Depth	Cv	Sact	Sreq	OK?
Strength Design	Typical Level	20	450	50	840	1340	804000	12	19.5	0.88	554.8	381.0	good
	12th Level parapet	20	200	50	1512	1762	1057200	12	19.5	0.88	554.8	501.0	good
	12th Level penthouse	20	350	50	2880	3280	1968000	12	25.5	0.86	985.1	958.0	good
	Penthouse Roof	20	200	50	735	985	591000	12	18	0.89	469.3	277.9	good

	Level	Span	L (plf)	D+L	EI	Defl. L	Defl. D+L	Lim. L	Lim. D+L	L OK?	D+L OK?
Defl. Design	Typical Level	20	420	1340	7.80E+09	0.194	0.831	0.667	1	good	good
	12th Level parapet	20	1050	1762	7.80E+09	0.485	0.978	0.667	1	good	good
	12th Level penthouse	20	2000	3280	1.90E+10	0.379	0.743	0.667	1	good	good
	Penthouse Roof	20	315	985	6.00E+09	0.189	0.792	0.667	1	good	good

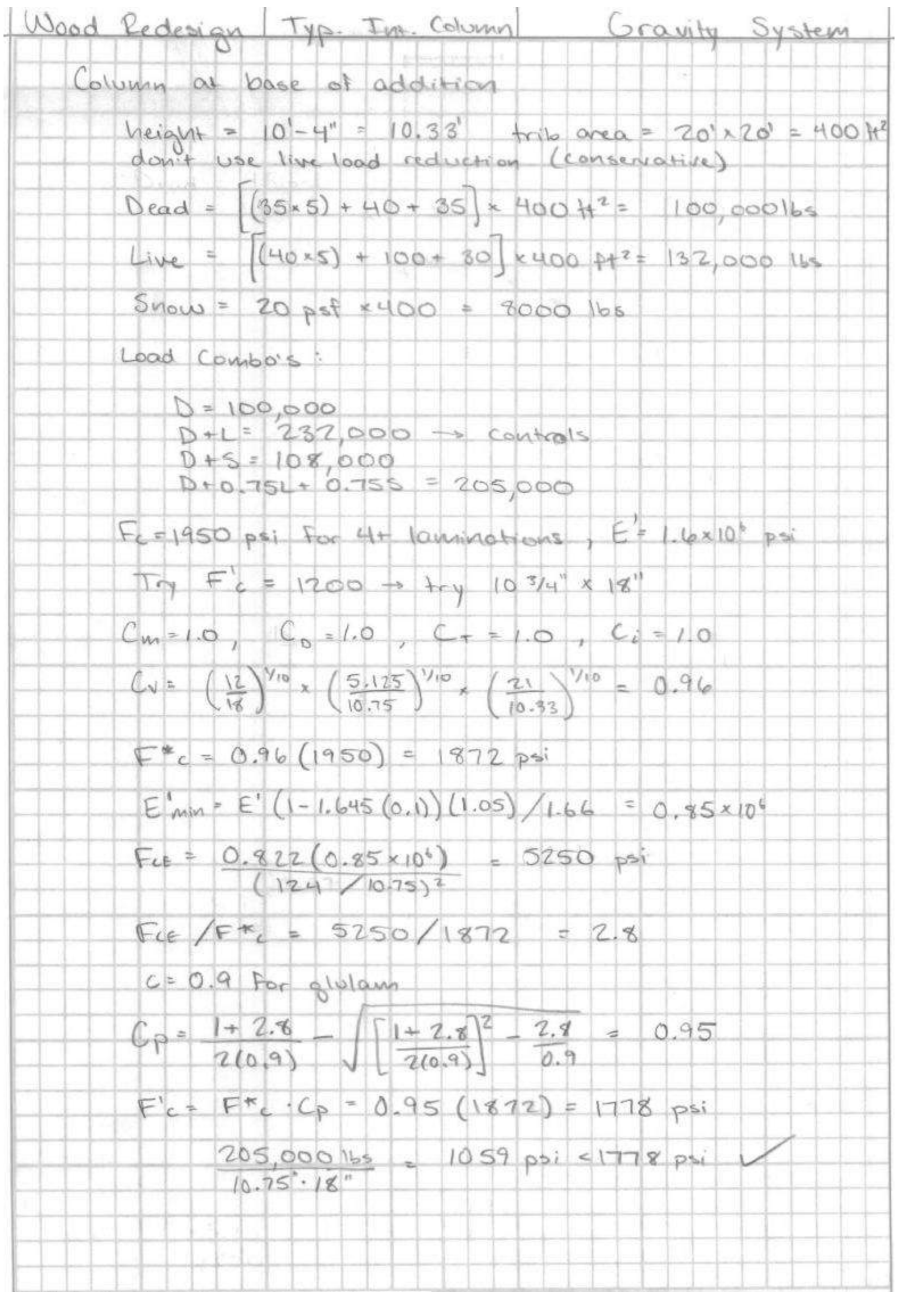
	Level	Span	D+L	Orig w	Orig h	Resid bf	Resid h	Resid bw	Seff	Red. Load	M (in-lb)	Sreq	OK?
Fire/ Char Design	Typical Level	20	1340	12	19.5	7	17	4	240.1	858.0	514800	152	good
	12th Level parapet	20	1762	12	19.5	7	17	4	288.4	954.0	572400	170	good
	12th Level penthouse	20	3280	12	25.5	7	23	4	531.1	1280.0	768000	234	good
	Penthouse Roof	20	985	12	18	7	15.5	4	196.3	628.5	377100	111	good

*Or along Grid 4 at 12th Level and Penthouse

**D+L was the controlling case for other girders, and will therefore be the only case considered in non typical giders

Table A.7: Non-typical Girder Design

A.3.3 Column Calculations



Wood Redesign | Typ. Int. Column | Gravity System

Column at base of addition

height = 10'-4" = 10.33'   trib area = 20'×20' = 400 ft²
don't use live load reduction (conservative)

Dead = $\left[(35\times5) + 40 + 35\right] \times 400\ ft^2 =$ 100,000 lbs

Live = $\left[(40\times5) + 100 + 30\right] \times 400\ ft^2 =$ 132,000 lbs

Snow = 20 psf ×400 = 8000 lbs

Load Combo's:

D = 100,000
D+L = 232,000 → controls
D+S = 108,000
D+0.75L+0.75S = 205,000

F_c=1950 psi for 4+ laminations, $E' = 1.6\times10^6$ psi

Try F'_c = 1200 → try 10 3/4" × 18"

$C_m=1.0,\ C_D=1.0,\ C_t=1.0,\ C_i=1.0$

$$C_V = \left(\frac{12}{18}\right)^{1/10} \times \left(\frac{5.125}{10.75}\right)^{1/10} \times \left(\frac{21}{10.33}\right)^{1/10} = 0.96$$

$F^*_c = 0.96(1950) = 1872$ psi

$E'_{min} = E'(1-1.645(0.1))(1.05)/1.66 = 0.85\times10^6$

$$F_{cE} = \frac{0.822(0.85\times10^6)}{(124/10.75)^2} = 5250 \text{ psi}$$

$F_{cE}/F^*_c = 5250/1872 = 2.8$

c = 0.9 for glulam

$$C_P = \frac{1+2.8}{2(0.9)} - \sqrt{\left[\frac{1+2.8}{2(0.9)}\right]^2 - \frac{2.8}{0.9}} = 0.95$$

$F'_c = F^*_c \cdot C_P = 0.95(1872) = 1778$ psi

$$\frac{205{,}000\text{ lbs}}{10.75"\cdot18"} = 1059 \text{ psi} < 1778 \text{ psi}$$ ✓

Figure A.7: Typical Column Calculations at Base of Addition

Wood Design | Typ. Int. Column | Gravity System

Column has extra capacity, however keep for now & try design for fire:

w/ single gypsum layer, eff. char = 2.5" after 2hrs. (30 min gyp. + 90 min char)

residual cross section ≈ 5.75" × 13"

reduced loads:

w/out assembly load

$0.75(100{,}000) + 0.5(108{,}000) = 129{,}000$ lbs

$\frac{129{,}000}{5.75 \times 13} = 1720$ psi < 1778 psi

∴ keep size for increased fire performance

use 10 3/4" × 18"

Typ Ext. Column

Loads are halved (new trib area = 200 ft^2)

$D+L = 116{,}000$ lbs

From APA design chart: use 10.75" × 12" column with C_D, allowable axial load = 125,000 @ 11'

Design for fire: residual section ≈ 5.75" × 7"

allowable axial $\frac{125000}{(10.75 \times 12)} = 970$ psi

reduced load =

$0.75(50{,}000) + 0.5(54{,}000) = 64{,}500$ lbs

$\frac{64{,}500}{5.75 \times 7} = 1602 > 970$

$\frac{64{,}500}{5.75 \times d} \leq 970 \rightarrow d = 11.5" + 5"$ char $= 16.5"$

Use 10 3/4" × 16 1/2"

Figure A.8: Typical Column Calculations at Base of Addition

Gravity Loads (psf, lbs for SW)			
Level	**Dead**	**Live**	**C. SW (per floor)**
Typical Level	36	40	415
12th Level	40	100	470
Roof	36	30	670

Floor Heights (ft)	
Typical Level	10.33
12th Level	11.67
Roof	16.75

Wood Properties	
Fc (psi)	1950
E' (psi)	1.60E+06
Cm	1
Cd	1
Ci	1
Ct	1
E'min	8.50E+05

(a) *General Column Design Information*

Column and Ext. Wall Load Information				
Col. Type*	**Trib Area**	**Wall Load (lbs)**		
		Typ. Level	**12th Level**	**Roof**
Typ. Int.	415	0	0	4150
Typ. Ext.	208	9338	10686	-
A	285	9338	10686	4150
B	130	10350	11845	-
C	335	0	0	0
D	300	0	0	0
E	475	0	0	-
F	260	10350	11845	-

(b) *Exterior Wall Load Information*

Figure A.9: General Column Design Information

Column Design: Various Levels, Strength, Fire Performance (See Design Summary for Splicing and Final Sizing Choices)																
Lev	Type*	D+L (lbs)	width	depth	Cv	F*c	Fce	Fce/F*c	Cp	F'c	fc	str ok?	red. D+L	resid. A	fc (fire)	fire ok?
Level 7	Typ. Int.	250555	12	12.375	0.98	1917	6548	3.42	0.96	1844	1687	0.915	131061	51.6	2539	0.860
	Typ. Ext.	168079	12	12.375	0.98	1917	6548	3.42	0.96	1844	1132	0.614	113571	51.6	2200	0.745
	A	231749	12	12.375	0.98	1917	6548	3.42	0.96	1844	1561	0.846	149435	51.6	2895	0.981
	B	133740	10.5	12	1.00	1948	5013	2.57	0.95	1842	1061	0.576	99434	38.5	2583	0.877
	C	199525	10.5	12	1.00	1948	5013	2.57	0.95	1842	1584	0.860	102911	38.5	2673	0.907
	D	179015	10.5	12	1.00	1948	5013	2.57	0.95	1842	1421	0.771	92411	38.5	2400	0.815
	E	249545	12	12.375	0.98	1917	6548	3.42	0.96	1844	1680	0.911	125884	51.6	2438	0.826
	F	201340	12	12.375	0.98	1917	6548	3.42	0.96	1844	1356	0.735	133364	51.6	2583	0.875
Level 8	Typ. Int.	218600	10.5	12	1.00	1948	5013	2.57	0.95	1842	1735	0.942	112905	38.5	2933	0.995
	Typ. Ext.	142518	10.5	12	1.00	1948	5013	2.57	0.95	1842	1131	0.614	94978	38.5	2467	0.837
	A	200336	12	12.375	0.98	1917	6548	3.42	0.96	1844	1349	0.731	127531	51.6	2470	0.837
	B	113095	10.5	12	1.00	1948	5013	2.57	0.95	1842	898	0.487	83183	38.5	2161	0.733
	C	173650	10.5	12	1.00	1948	5013	2.57	0.95	1842	1378	0.748	88195	38.5	2291	0.777
	D	155800	10.5	12	1.00	1948	5013	2.57	0.95	1842	1237	0.671	79200	38.5	2057	0.698
	E	213030	10.5	12	1.00	1948	5013	2.57	0.95	1842	1691	0.918	105148	38.5	2731	0.927
	F	170815	10.5	12	1.00	1948	5013	2.57	0.95	1842	1356	0.736	111523	38.5	2897	0.983
Level 9	Typ. Int.	186645	10.5	12	1.00	1948	5013	2.57	0.95	1842	1481	0.804	94749	38.5	2461	0.835
	Typ. Ext.	116958	10.5	12	1.00	1948	5013	2.57	0.95	1842	928	0.504	76385	38.5	1984	0.673
	A	168924	10.5	12	1.00	1948	5013	2.57	0.95	1842	1341	0.728	105628	38.5	2744	0.931
	B	92450	8.5	12	1.02	1990	3285	1.65	0.89	1780	906	0.509	66931	24.5	2732	0.959
	C	147775	10.5	12	1.00	1948	5013	2.57	0.95	1842	1173	0.637	73479	38.5	1909	0.648
	D	132585	8.5	12	1.02	1990	3285	1.65	0.89	1780	1300	0.730	65989	24.5	2693	0.946
	E	176515	10.5	12	1.00	1948	5013	2.57	0.95	1842	1401	0.761	84411	38.5	2193	0.744
	F	140290	10.5	12	1.00	1948	5013	2.57	0.95	1842	1113	0.605	89681	38.5	2329	0.791
Level 10	Typ. Int.	154690	10.5	12	1.00	1948	5013	2.57	0.95	1842	1228	0.667	76593	38.5	1989	0.675
	Typ. Ext.	91397	8.5	12	1.02	1990	3285	1.65	0.89	1780	896	0.503	57792	24.5	2359	0.828
	A	137511	10.5	12	1.00	1948	5013	2.57	0.95	1842	1091	0.593	83724	38.5	2175	0.738
	B	71805	8.5	12	1.02	1990	3285	1.65	0.89	1780	704	0.396	50680	24.5	2069	0.726
	C	121900	8.5	12	1.02	1990	3285	1.65	0.89	1780	1195	0.672	58763	24.5	2398	0.842
	D	109370	8.5	12	1.02	1990	3285	1.65	0.89	1780	1072	0.602	52778	24.5	2154	0.757
	E	140000	8.5	12	1.02	1990	3285	1.65	0.89	1780	1373	0.771	63675	24.5	2599	0.913
	F	109765	8.5	12	1.02	1990	3285	1.65	0.89	1780	1076	0.605	67840	24.5	2769	0.972
Level 11	Typ. Int.	122735	8.5	12	1.02	1990	3285	1.65	0.89	1780	1203	0.676	58436	24.5	2385	0.838
	Typ. Ext.	65837	8.5	12	1.02	1990	3285	1.65	0.89	1780	645	0.363	39200	24.5	1600	0.562
	A	106099	8.5	12	1.02	1990	3285	1.65	0.89	1780	1040	0.584	61820	24.5	2523	0.886
	B	51160	8.5	12	1.02	1990	3285	1.65	0.89	1780	502	0.282	34429	24.5	1405	0.493
	C	96025	8.5	12	1.02	1990	3285	1.65	0.89	1780	941	0.529	44046	24.5	1798	0.631
	D	86155	8.5	12	1.02	1990	3285	1.65	0.89	1780	845	0.475	39566	24.5	1615	0.567
	E	103485	8.5	12	1.02	1990	3285	1.65	0.89	1780	1015	0.570	42939	24.5	1753	0.615
	F	79240	8.5	12	1.02	1990	3285	1.65	0.89	1780	777	0.437	45999	24.5	1878	0.659
Level 12	Typ. Int.	90780	8.5	10.5	1.02	1992	2574	1.29	0.84	1678	1017	0.606	40280	19.3	2092	0.779
	Typ. Ext.	40276	6.75	10.5	1.05	2039	1623	0.80	0.66	1355	568	0.419	20607	9.6	2141	0.988
	A	74686	8.5	10.5	1.02	1992	2574	1.29	0.84	1678	837	0.499	39916	19.3	2074	0.772
	B	30515	6.75	10.5	1.05	2039	1623	0.80	0.66	1355	431	0.318	18178	9.6	1889	0.871
	C	70150	8.5	10.5	1.02	1992	2574	1.29	0.84	1678	786	0.468	29330	19.3	1524	0.567
	D	62940	6.75	12	1.03	2012	1623	0.81	0.67	1349	777	0.576	26355	12.3	2151	0.997
	E	66970	6.75	12	1.03	2012	1623	0.81	0.67	1349	827	0.613	22203	12.3	1812	0.840
	F	48715	6.75	12	1.03	2012	1623	0.81	0.67	1349	601	0.446	24158	12.3	1972	0.914
Penthouse	Typ. Int.	32210	8.5	10.5	0.99	1922	1250	0.65	0.57	1102	361	0.328	20838	19.3	1082	0.614
	A	23630	8.5	10.5	0.99	1922	1250	0.65	0.57	1102	265	0.240	15768	19.3	819	0.465
	C	22780	6.75	12	1.00	1940	788	0.41	0.38	742	281	0.379	13568	12.3	1108	0.933
	D	20470	6.75	12	1.00	1940	788	0.41	0.38	742	253	0.341	12203	12.3	996	0.839

*Column Types are labeled on the following floor plan
Note: As long as "OK?" column values are less than 1.0, the size has passed design checks. (Value is ratio of fp/F'c)

Table A.8: Column Excel Calculations

B | Lateral System Redesign Calculations

B.1 Introduction

Included in this Appendix are all the calculations completed for both the existing lateral system and the wood redesign lateral system. These calculations are provided to show more specifically what was done to reach the design choices and conclusions.

B.2 Existing Lateral System

Sample excel calculations determining loads in the existing lateral system follow. The methods and process used for determining the lateral loads is described in chapter 1.

B.2.1 Wind Loads

Wind Load Calculations: Wind Perpendicular to Building
ASCE 7-05, Chapter 6.5: Method 2 - Analytical Procedure
Design Procedure from Section 6.5.3
Blue boxes are input boxes, all else are determined by equations

Building Information	B	214		
	L	60		
	h	153		
	z bar	145		
	Variable	**Value**	**Units**	**Comments**
1. Determine Basic Wind Speed and Directonality Factor				
Basic Wind Speed	V	90	mph	(Fig. 6-1)
Directionality Factor	k_d	0.85		(Table 6-4)
2. Determine Importance Factor				
Occupancy Category		II		(Table 1-1)
Importance Factor	I	1		(Table 6-1)
3 & 9. Exposure Category, Velocity Pressure Exposure Coefficient, and Velocity Pressure				
Exposure Category		B		From Structural Drawings
Velocity Pressure Exposure Coefficient				

Note: Use exposure B, case 2 for MWFRS
Values determined by Interpolation

Height (ft)	K_z	q_z or q_h
8	0.570	11.82
19	0.618	12.81
30	0.700	14.52
41	0.765	15.86
51	0.814	16.88
61	0.854	17.71
73	0.902	18.70
83	0.940	19.49
94	0.972	20.16
104	1.000	20.74
114	1.025	21.25
125	1.053	21.84
136	1.080	22.39
140	1.090	22.60
153	1.116	23.14
158	1.126	23.35

4. Determine Topographic Factor			
Topographic Factor	K_z	1	Value used by structural engineering firm

Table B.1: Wind Load Excel Calculations

5. Determine Gust Effect Factor

The following is based on a flexible building (Section 6.5.8.2)

Input Variables

	b bar	0.45	(Table 6-1)
	α bar	0.25	(Table 6-1)
	ϵ bar	0.33	(Table 6-1)
	l	320.00	(Table 6-1)
	c	0.30	(Table 6-1)
	β	1.50	(C6.5.8)

Output Variables

n_1	0.49			
N_1	2.987		R_n	0.070
η_h	4.012		R_h	0.218
η_B	5.611		R_B	0.162
η_L	5.267		R_L	0.172
$I_{z\ bar}$	0.23		g_q	3.40
$L_{z\ bar}$	524.125		g_r	4.02
$V\ bar_{z\ bar}$	86.000		g_v	3.40
Q	0.82			
R	0.03			

Gust Effect Factor G_f 0.83

6. Determine the Enclosure Classification

Building is considered enclosed (Section 6.5.9)

7. Determine the Internal Pressure Coefficient

	Gc_{pi}	0.18	(Figure 6-5)
	or	-0.18	

8. Determine External Pressure Coefficients

Windward Wall	C_p	0.8	(Figure 6-6)	use with q_z
Leeward Wall	C_p	-0.5	(Figure 6-6)	use with q_h
Side Wall	C_p	-0.7	(Figure 6-6)	use with q_h
Roof (0' to 60')	C_p	-0.9	(Figure 6-6)	

Table B.2: Wind Load Excel Calculations

Wind Pressure Chart (Wind Perpendicular to Building)								
Location	**z(ft)**	**q_z or q_h**	**C_p**	**G_f**	**Gc_{pi}**	**q_iGC_{pi} (psf)**	**Net Pressure (psf)**	
							$q_zG_fC_p$-q_i(+GC_{pi})	**$q_zG_fC_p$-q_i(-GC_{pi})**
Windward	8	11.82	0.8	0.83	0.18	2.13	5.70	**9.95**
	19	12.81	0.8	0.83	0.18	2.31	6.17	**10.79**
	30	14.52	0.8	0.83	0.18	2.61	6.99	**12.22**
	41	15.86	0.8	0.83	0.18	2.86	7.64	**13.35**
	51	16.88	0.8	0.83	0.18	3.04	8.13	**14.21**
	61	17.71	0.8	0.83	0.18	3.19	8.53	**14.91**
	73	18.70	0.8	0.83	0.18	3.37	9.01	**15.75**
	83	19.49	0.8	0.83	0.18	3.51	9.39	**16.41**
	94	20.16	0.8	0.83	0.18	3.63	9.71	**16.97**
	104	20.74	0.8	0.83	0.18	3.73	9.99	**17.46**
	114	21.25	0.8	0.83	0.18	3.83	10.24	**17.89**
	125	21.84	0.8	0.83	0.18	3.93	10.52	**18.38**
	136	22.39	0.8	0.83	0.18	4.03	10.79	**18.85**
	153	23.14	0.8	0.83	0.18	4.17	11.15	**19.48**
Leeward	All	23.35	-0.5	0.83	0.18	4.20	**-13.86**	-5.46
Side	All	23.35	-0.7	0.83	0.18	4.20	**-17.72**	-9.32
Roof (0' to 60')	153	23.35	-0.9	0.83	0.18	4.20	**-21.59**	-13.18
Low Parapet WW	140	22.60			1.5	33.90		**33.90**
High Parapet WW	158	23.35			1.5	35.02		**35.02**
High Parapet LW	158	23.35			-1.0	-23.35		**-23.35**

Table B.3: Wind Load Excel Calculations

Level	Floor Ht.	Story Ht. * Net Pressure	
		Perpendicular	Parallel
B2	8	79.6	81.4
B1	11	118.7	121.3
L1	11	134.4	137.4
L2	11	146.9	150.2
L3	10	142.1	145.3
L4	10	149.1	152.4
L5	12	189.0	193.1
L6	10	164.1	167.7
L7	11	186.6	190.8
L8	10	174.6	178.4
L9	10	178.9	182.9
L10	11	202.2	206.7
L11	11	207.4	212.0
L12	17	331.2	338.5
Base Shear (kips)		963.9	226.6

Table B.4: Wind Load Excel Calculations

B.2.2 Seismic Loads

Tech Report 2 | Seismic Loads | Samantha deVries

Seismic Load Calculations

ASCE 7-05, Chapter 12: Seismic Design Requirements for Building Structures

1. Exemptions (11.1.2)

   Building not exempt

2. Site Class (11.4.2)

   C (From structural documents)

   11.4.1 (Fig. 22-1 to 22-6)

   $S_s = 0.155\,g$ (from structural documents)
   $S_1 = 0.050\,g$

   11.4.3 Adjust for site class:

   Table 11.4-1, $S_s \le 0.25$, $F_a = 1.2$
   Table 11.4-2, $S_1 \le 0.1$, $F_v = 1.7$

   Eqn 11.4-1 $S_{MS} = F_a S_s = 1.2(0.155) = 0.186\,g$
   Eqn 11.4-2 $S_{M1} = F_v S_1 = 1.7(0.050) = 0.085\,g$

   11.4.4 Design Parameters:

   Eqn. 11.4-3, $S_{DS} = \frac{2}{3} S_{MS} = (2/3)(0.186) = \underline{0.124\,g}$
   Eqn. 11.4-4, $S_{D1} = \frac{2}{3} S_{M1} = (2/3)(0.085) = \underline{0.057\,g}$

3. Seismic Design Category (11.6)

   Table 11.6-1 $S_{DS} < 0.167 \rightarrow A$ $\therefore \boxed{\text{SDC A}}$
   Table 11.6-2 $S_{D1} < 0.067 \rightarrow A$

4. Select Analysis Procedure (use 11.7)

   Eq. 11.7-1 $F_x = 0.01 w_x$

5. Calculate effective total seismic weight (w)

   Roof: DL + 20% SL
   Floors: DL

   $W_{RF} = (\text{penthouse}) = (125')(46')(27 + 0.2(20)) + 2(125 + 46')(38)(5)$
   $= 178{,}250 + 49{,}020$
   $= \underline{228{,}000 \text{ lbs}}$

Figure B.1: Seismic Load Calculations

$W_{ST\,FL} = (60')(214')(75\text{ psf}) + 2(60+214)(490\text{ plf})$
$= 963{,}000 + 268{,}520$
$= \underline{1{,}232{,}000\text{ lbs}}$

$W_{CONC\,FL} = (60')(214')(105\text{ psf}) + 2(60+214)(992)$
6½" typ. $= 1{,}348{,}200 + 543{,}616$
$= \underline{1{,}892{,}000\text{ lbs}}$

Total Load =

$W = W_{RF} + 6(W_{ST\,FL}) + 7(W_{CONC\,FL})$
$= 228\text{ k} + 6(1{,}232\text{ k}) + 7(1{,}892\text{ k})$
$\underline{W = 20{,}864\text{ k}}$

6. Other Factors

Basic Seismic Force-Resisting System: Ordinary Concrete Moment Frames and Steel Moment Frames

Response Modification Factor, $R = 3$ (Table 12.2-1)

7. Calculate Seismic Base Shear (V)

Eqn. 12.8-1 $V = C_s W$

$C_s = S_{DS} / \left(\frac{R}{I}\right)$ Eq. 12.8-2

$= 0.124 / (3/1) = 0.042$

$V = 0.042(20{,}864) = \boxed{876\text{ k}}$

$T_a = 0.1N = 0.1(14) = 1.4\text{ s}$ (Eqn. 12.8-8)

$T_L = 5.5\text{ s}$ (Fig. 22-2)

$\therefore C_s$ need not exceed $\dfrac{S_{D1} T_L}{T^2 (R/I)} = 0.116 > 0.042$ ✓

8. Vertical Distribution of Seismic Forces (F_x)

$F_x = C_{vx} V = \dfrac{w_x h_x^k}{\sum w_i h_i^k} V$

$k = 1.5$ (using linear interpolation)

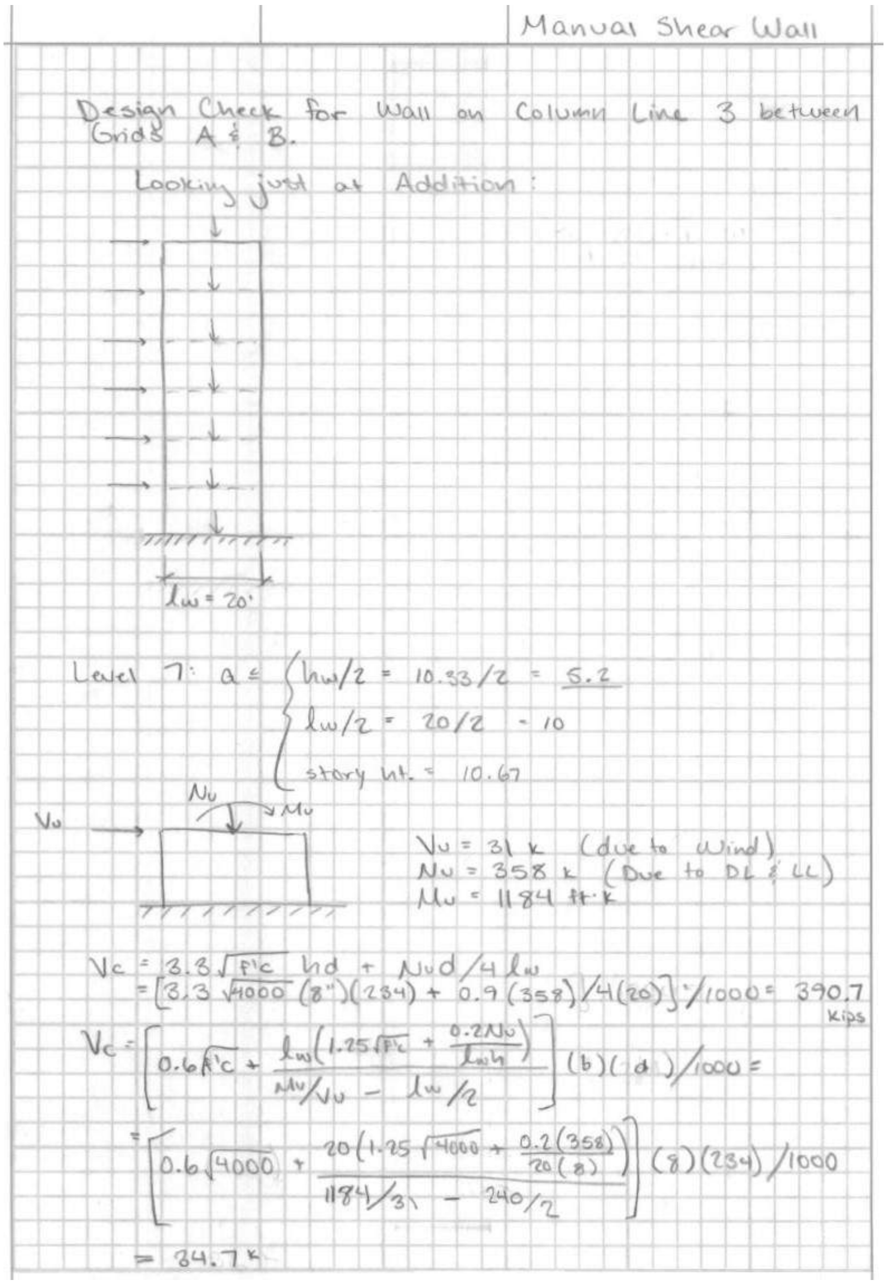
Figure B.2: Seismic Load Calculations

Level	h_x (ft)	w_x (k)	$w_x h_x^k$	C_{vx}	F_x (k)	V_x (k)	hx * Fx (ft*k)
Penthouse	153	228	526737	0.010	8.8	8.8	1353
12	136	1232	5881051	0.113	98.7	107.6	13426
11	125	1232	5405378	0.104	90.7	198.3	11342
10	114	1232	4929705	0.094	82.8	281.1	9434
9	104	1232	4497275	0.086	75.5	356.6	7851
8	94	1232	4064844	0.078	68.2	424.8	6414
7	83	1232	3589171	0.069	60.3	485.0	5001
6	73	1892	6007649	0.115	100.8	585.9	7362
5	61	1892	5020090	0.096	84.3	670.2	5141
4	51	1892	4197125	0.080	70.5	740.6	3593
3	41	1892	3374159	0.065	56.6	797.3	2322
2	30	1892	2468897	0.047	41.4	838.7	1243
1	19	1892	1563635	0.030	26.2	864.9	499
B1	8	1892	658373	0.013	11.1	876.0	88
Sum		20864	52184088	1.000	876.0		**75070**
							=OTM

Table B.5: Seismic Load Calculations

B.3 Wood Redesign

Included below are the various hand calculation spot checks of the software and any relevant software output for the redesigned lateral system.


Manual Shear Wall

Design Check for Wall on Column Line 3 between Grids A & B.

Looking just at Addition:

$l_w = 20'$

Level 7: $a \leq \begin{cases} h_w/2 = 10.33/2 = \underline{5.2} \\ l_w/2 = 20/2 = 10 \\ \text{story ht.} = 10.67 \end{cases}$

V_u, N_u, M_u

$V_u = 31$ k (due to Wind)
$N_u = 358$ k (Due to DL & LL)
$M_u = 1184$ ft·k

$V_c = 3.3\sqrt{f'c}\,hd + N_u d/4 l_w$
$= [3.3\sqrt{4000}(8'')(234) + 0.9(358)/4(20)]/1000 = 390.7$ kips

$V_c = \left[0.6\sqrt{f'c} + \dfrac{l_w\left(1.25\sqrt{f'c} + \dfrac{0.2N_u}{l_w h}\right)}{M_u/V_u - l_w/2}\right](b)(d)/1000 =$

$= \left[0.6\sqrt{4000} + \dfrac{20\left(1.25\sqrt{4000} + \dfrac{0.2(358)}{20(8)}\right)}{1184/31 - 240/2}\right](8)(234)/1000$

$= 84.7^k$

Figure B.3: Shear Wall Spot Check

or $V_c = 2\sqrt{f'c}\, hd = 2\sqrt{4000}\,(8)(234)/1000 = 237^k$

$V_u > 0.5\,\phi V_c = 0.5(0.75)(237) = 88.9$

$34.7 < 88.9$, $\therefore$ no shear reinf. req.

Include to meet min reinf. req's

Horizontal : $\rho_t \geq 0.0025 = 8''(240'')(0.0025) = 4.8\,in^2$

$S \leq \min \begin{cases} l_w/5 = 20/5 = 4' = 48'' \\ 3h = 3(8) = 24'' \\ 18'' \rightarrow \text{controls} \end{cases}$

#4 bar $\rightarrow$ $A = 0.2\ in^2/bar$ 24 bars

2 curtains, 12 #4's each side @ 10" O.C.

Vertical :

$\rho_l \geq \min \left|\ 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025) \right.$
$= 0.0025 + 0.5(2.5 - (10.33/20))(0.0025 - 0.0025)$
$= 0.0025$

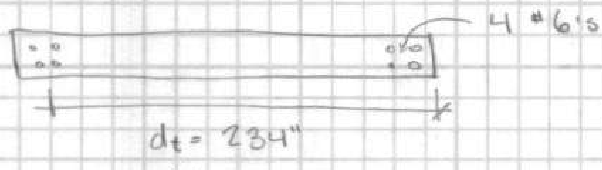
$8''(240'')(0.0025) = 4.8\ in^2 \rightarrow 24$ bars

2 curtains, 12 #4's each side @ 18" O.C.

$S \leq \min \begin{cases} l_w/3 = 20'/3 = 6.67' \\ 3h = 24'' \\ 18'' \rightarrow \text{controls} \end{cases}$

Flexural Design (Level 7 : Base Level)

$M_u = 1184$ ft·k check Etabs design



assume case 1:

$M_n = A_s f_y jd = 4(0.44)(60)(0.9)(19.5) = 1853$ ft·k

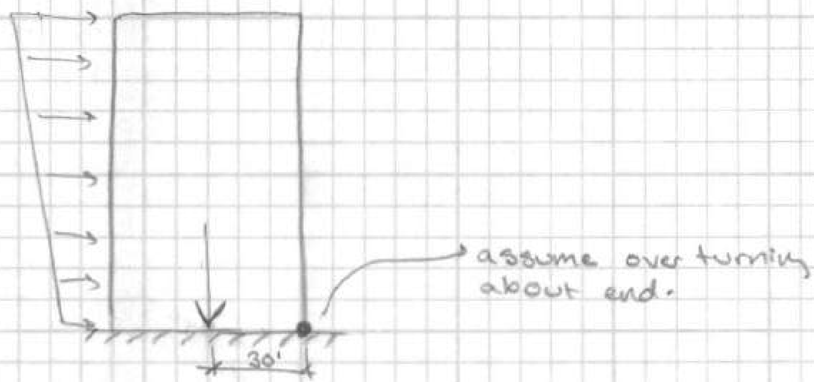
$M_u \geq \phi M_n \rightarrow 1184 \leq 1667$ ft·k ✓ ok

could go to #5's, but should be 2 sizes up from #4's

Figure B.4: Shear Wall Spot Check

Overturning Moment

From Wind Analysis in ETABS, overturning Moment = 34,000 ft·kip



Dead Load of Building =

110 psf (7 floors) (12,800 sf)
+ 36 psf (7 floors) (12,800 sf) = 13,080 kips

Resisting w/ moment arm of half building width = 30'

13,080 kips (30') = 392,000 ft·k

$M_R > M_{ov}$

392,000 >> 34,000

∴ building can resist overturning moment due to wind

Other direction has much lower overturning moment due to significantly less surface area ∴ will not control.

Figure B.5: Overturning Moment Check

In-Plane Diaphragm Deflections

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \underbrace{\frac{0.25vL}{1000G_a}}_{0} + \underbrace{\frac{\Sigma(x\Delta_c)}{2W}}_{0}$$

G_a = apparent stiffness from nail slip
glued product, ∴ does not apply

Δ_c = diaphragm chord splice slip, DNA

$$\therefore \delta_{dia} = \frac{5vL^3}{8EAW}$$

$v = 4.8$ kif $= 4800$ plf

$L = 26'$

$E = 1.5 \times 10^6$ psi

$A = 3000$ in^2

$W = 30'$ (Bay act individually)

$$\delta_{dia} = \frac{5(4800)(26)^3(1728)}{8(1.5E6)(3000)(30)} = \underline{\underline{0.675 \text{ in}}}$$

From ETABS: $\delta_{dia} = 0.823$

calculated $l/360$ limit $= 0.877$, ∴ acceptable

The hand calc is just a bit lower,
most likely b/c the equation couldn't
account for shear controlled deflection.
∴ hand calc may be low, ETABS may
be conservative.

Figure B.6: In-plane deflection spot check

C Design Tables

C.1 Introduction

Included in this Appendix are the design value tables used in the gravity system redesign. The purpose of this appendix is to provide the specifically referenced tables used in this thesis.

CLT Grade	Laminations in the Major Strength Direction of the CLT						Laminations in the Minor Strength Direction of the CLT					
	$f_{b,0}$ (psi)	E_0 (10^6psi)	$f_{t,0}$ (psi)	$f_{c,0}$ (psi)	$f_{v,0}$ (psi)	$f_{s,0}$ (psi)	$f_{b,90}$ (psi)	E_{90} (10^6psi)	$f_{t,90}$ (psi)	$f_{c,90}$ (psi)	$f_{v,90}$ (psi)	$f_{s,90}$ (psi)
E1	4,095	1.7	2,885	3,420	425	140	1,050	1.2	525	1,235	425	140
E2	3,465	1.5	2,140	3,230	565	190	1,100	1.4	680	1,470	565	190
E3	2,520	1.2	1,260	2,660	345	115	735	0.9	315	900	345	115
E4	4,095	1.7	2,885	3,420	550	180	1,205	1.4	680	1,565	550	180
V1	1,890	1.6	1,205	2,565	565	190	1,100	1.4	680	1,470	565	190
V2	1,835	1.4	945	2,185	425	140	1,050	1.2	525	1,235	425	140
V3	2,045	1.6	1,155	2,755	550	180	1,205	1.4	680	1,565	550	180

For SI: 1 psi = 6.895 kPa

[a] The characteristic values may be obtained from the published allowable design values for lumber in the United States as follows:

$f_{b,0}$ = 2.1 x published allowable bending stress (F_b), $f_{t,0}$ = 2.1 x published allowable tensile stress (F_t), $f_{c,0}$ = 1.9 x published allowable compressive stress parallel to grain (F_c), $f_{v,0}$ = 3.15 x published allowable shear stress (F_v), and $f_{s,0}$ = 1/3 x calculated $f_{v,0}$.

Table C.1: CLT Material Design Values Table. Source: CLT Handbook.

CLT Grade	CLT Thick-ness (in.)	Lamination Thickness in CLT Lay-up (in.)							Major Strength Direction			Minor Strength Direction		
		=	⊥	=	⊥	=	⊥	=	$F_bS_{eff,0}$ (lb.-ft. /ft.)	$EI_{eff,0}$ (10^6 lb.-in.^{2}/ft.)	$GA_{eff,0}$ (10^6 lb. /ft.)	$F_bS_{eff,90}$ (lb.-ft. /ft.)	$EI_{eff,90}$ 10^6 lb.-in.^{2}/ft.)	$GA_{eff,90}$ (10^6 lb. /ft.)
E1	4 1/8	1 3/8	1 3/8	1 3/8					4,525	115	0.46	160	3.1	0.61
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			10,400	440	0.92	1,370	81	1.2
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	18,375	1,089	1.4	3,125	309	1.8
E2	4 1/8	1 3/8	1 3/8	1 3/8					3,825	102	0.53	165	3.6	0.56
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			8,825	389	1.1	1,430	95	1.1
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	15,600	963	1.6	3,275	360	1.7
E3	4 1/8	1 3/8	1 3/8	1 3/8					2,800	81	0.35	110	2.3	0.44
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			6,400	311	0.69	955	61	0.87
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	11,325	769	1.0	2,180	232	1.3
E4	4 1/8	1 3/8	1 3/8	1 3/8					4,525	115	0.53	180	3.6	0.63
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			10,425	441	1.1	1,570	95	1.3
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	18,400	1,090	1.6	3,575	360	1.9
V1	4 1/8	1 3/8	1 3/8	1 3/8					2,090	108	0.53	165	3.6	0.59
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			4,800	415	1.1	1,430	95	1.2
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	8,500	1,027	1.6	3,275	360	1.8
V2	4 1/8	1 3/8	1 3/8	1 3/8					2,030	95	0.46	160	3.1	0.52
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			4,675	363	0.91	1,370	81	1.0
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	8,275	898	1.4	3,125	309	1.6
V3	4 1/8	1 3/8	1 3/8	1 3/8					2,270	108	0.53	180	3.6	0.59
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			5,200	415	1.1	1,570	95	1.2
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	9,200	1,027	1.6	3,575	360	1.8

For SI: 1 in. = 25.4 mm; 1 ft. = 304.8 mm; 1 lb. = 4.448 N

(a) This table represents one of many possibilities that CLT could be manufactured by varying lamination grades, thicknesses, orientations, and layer arrangements in the lay-up.
(b) Custom CLT grades that are not listed in this table are permitted in accordance with ANSI/APA PRG 320.
(c) The allowable properties can be converted to the characteristic properties by multiplying the tabulated F_bS by 2.1, and EI and GA by 1.0.

Table C.2: CLT Panel Design Table. Source: CLT Handbook.

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES

F_b = 2,400 psi, E = 1.80 x 10^6 psi, F_v = 265 psi

3-1/8-INCH WIDTH

Depth (in.)	**6**	**7-1/2**	**9**	**10-1/2**	**12**	**13-1/2**	**15**	**16-1/2**	**18**	**19-1/2**	**21**	**22-1/2**	**24**	**25-1/2**	**27**
Beam Weight (lb/ft)	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.75	23.44	28.13	32.81	37.50	42.19	46.88	51.56	56.25	60.94	65.63	70.31	75.00	79.69	84.38
S (in.^{3})	18.75	29.30	42.19	57.42	75.00	94.92	117.2	141.8	168.8	198.0	229.7	263.7	300.0	338.7	379.7
I (in.^{4})	56.25	109.9	189.8	301.5	450.0	640.7	878.9	1170	1519	1931	2412	2966	3600	4318	5126
EI (10^6 lb-in.^{2})	101.3	197.8	341.7	542.6	810.0	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)	3750	5859	8438	11480	15000	18980	23440	28360	33750	39610	45940	52730	60000	67730	75940
Shear Capacity (lb)	3313	4141	4969	5797	6625	7453	8281	9109	9938	10770	11590	12420	13250	14080	14910

3-1/2-INCH WIDTH

Depth (in.)	**6**	**7-1/2**	**9**	**10-1/2**	**12**	**13-1/2**	**15**	**16-1/2**	**18**	**19-1/2**	**21**	**22-1/2**	**24**	**25-1/2**	**27**
Beam Weight (lb/ft)	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.00	26.25	31.50	36.75	42.00	47.25	52.50	57.75	63.00	68.25	73.50	78.75	84.00	89.25	94.50
S (in.^{3})	21.00	32.81	47.25	64.31	84.00	106.3	131.3	158.8	189.0	221.8	257.3	295.3	336.0	379.3	425.3
I (in.^{4})	63.00	123.0	212.6	337.6	504.0	717.6	984.4	1310	1701	2163	2701	3322	4032	4836	5741
EI (10^6 lb-in.^{2})	113.4	221.5	382.7	607.8	907.2	1292	1772	2358	3062	3893	4862	5980	7258	8705	10330
Moment Capacity (lb-ft)	4200	6563	9450	12860	16800	21260	26250	31760	37800	44360	51450	59060	67200	75860	85050
Shear Capacity (lb)	3710	4638	5565	6493	7420	8348	9275	10200	11130	12060	12990	13910	14840	15770	16700

5-1/8-INCH WIDTH

Depth (in.)	**12**	**13-1/2**	**15**	**16-1/2**	**18**	**19-1/2**	**21**	**22-1/2**	**24**	**25-1/2**	**27**	**28-1/2**	**30**	**31-1/2**	**33**
Beam Weight (lb/ft)	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.50	69.19	76.88	84.56	92.25	99.94	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123.0	155.7	192.2	232.5	276.8	324.8	376.7	432.4	492.0	555.4	622.7	693.8	768.8	847.5	930.2
I (in.^{4})	738.0	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11530	13350	15350
EI (10^6 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10630	12750	15130	17800	20760	24030	27630
Moment Capacity (lb-ft)	24600	31130	38440	46510	55350	64960	75340	86480	98400	111100	124500	138800	153800	169500	186000
Shear Capacity (lb)	10870	12220	13580	14940	16300	17660	19010	20370	21730	23090	24450	25800	27160	28520	29880

5-1/2-INCH WIDTH

Depth (in.)	**12**	**13-1/2**	**15**	**16-1/2**	**18**	**19-1/2**	**21**	**22-1/2**	**24**	**25-1/2**	**27**	**28-1/2**	**30**	**31-1/2**	**33**
Beam Weight (lb/ft)	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.00	74.25	82.50	90.75	99.00	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132.0	167.1	206.3	249.6	297.0	348.6	404.3	464.1	528.0	596.1	668.3	744.6	825.0	909.6	998.3
I (in.^{4})	792.0	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12380	14330	16470
EI (10^6 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11400	13680	16240	19100	22280	25790	29650
Moment Capacity (lb-ft)	26400	33410	41250	49910	59400	69710	80850	92810	105600	119200	133700	148900	165000	181900	199700
Shear Capacity (lb)	11660	13120	14580	16030	17490	18950	20410	21860	23320	24780	26240	27690	29150	30610	32070

6-3/4-INCH WIDTH

Depth (in.)	**18**	**19-1/2**	**21**	**22-1/2**	**24**	**25-1/2**	**27**	**28-1/2**	**30**	**31-1/2**	**33**	**34-1/2**	**36**	**37-1/2**	**39**
Beam Weight (lb/ft)	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	364.5	427.8	496.1	569.5	648.0	731.5	820.1	913.8	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11070	13020	15190	17580	20210	23100	26240	29660	33370
EI (10^6 lb-in.^{2})	5905	7508	9377	11530	14000	16790	19930	23440	27340	31650	36390	41580	47240	53390	60060
Moment Capacity (lb-ft)	72900	85560	99230	113900	129600	146300	164000	182800	202500	223300	245000	267800	291600	316400	342200
Shear Capacity (lb)	21470	23250	25040	26830	28620	30410	32200	33990	35780	37560	39350	41140	42930	44720	46510

8-3/4-INCH WIDTH

Depth (in.)	**24**	**25-1/2**	**27**	**28-1/2**	**30**	**31-1/2**	**33**	**34-1/2**	**36**	**37-1/2**	**39**	**40-1/2**	**42**	**43-1/2**	**45**
Beam Weight (lb/ft)	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840.0	948.3	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12090	14350	16880	19690	22790	26200	29940	34020	38450	43250	48440	54020	60020	66450
EI (10^6 lb-in.^{2})	18140	21760	25830	30380	35440	41020	47170	53900	61240	69210	77860	87190	97240	108000	119600
Moment Capacity (lb-ft)	168000	189700	212600	236900	262500	289400	317600	347200	378000	410200	443600	478400	514500	551900	590600
Shear Capacity (lb)	37100	39420	41740	44060	46380	48690	51010	53330	55650	57970	60290	62610	64930	67240	69560

Notes:

(1) Beam weight is based on density of 35 pcf.

(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.

(3) Moment and shear capacities are based on a normal (10-year) duration of load and should be adjusted for the design duration of load per the applicable building code.

Table C.3: Glulam Beam Design Table. Source: APA - The Engineered Wood Association

D Breadth Calculations

D.1 Introduction

Included in this appendix are additional breadth tables and calculations.

D.1.1 Construction Management Breadth

Structurlam Products Ltd Budget Pricing for CrossLam (Cross Laminated Timber Panels) CDN$

			1	2	3			
Panel Type	# of Laminations	Panel Thickness	Blank Panel $/Sq. Ft	Hand Framing (Floor/Roof) $/Sq. Ft	5 Axis Robotic Framing (Walls) $/Sq. Ft	Fasterner, Hardware, Shop Drawings Floor/Roof $/Sq. Ft.	Walls $/Sq. Ft.	Visual Grade $/Sq. Ft
SLT3	3	99mm	5.80	6.05	7.02	2.50	3.00	1.00
SLT5	5	169mm	9.68	9.93	11.21	2.50	3.00	1.00
SLT7	7	239mm	13.77	14.02	15.93	3.00	3.50	1.00
SLT9	9	309mm	17.53	17.97	19.90	3.00	3.50	1.00

Note: it's columns 1 or 2 or 3... not 1 + 2 or 1 +3 or 1 + 2 + 3

Table D.1: Structurlam CLT costs given in Canadian dollars. Source: Michael Green's Presentation on How to Develop a CLT Project

Item Quantities					
Gravity System Items	Unit	Quantity Per Level			Total
		Typ. Level	12th Level	Penthouse	
Steel Columns	L.F.	455	513	270	3058
Steel Columns	Ton	10.9	12.34	11.43	78.27
W 12x22	L.F.	336	336	0	2016
W10x33	L.F.	798	798	0	4788
W16x26	L.F.	0	0	625	625
W14x22	L.F.	0	0	310	310
W12x30	L.F.	105	105	0	630
Open Web Joist 12K3	L.F.	2700	2700	0	16200
Open Web Joist 16K3	L.F.	0	0	1100	1100
Floor Deck	S.F.	12840	12840	0	77040
Roof Deck	S.F.	0	0	4300	4300
Moment Connection Weld	L.F.	82	82	30	522
Shear Connection Weld	L.F.	207	207	138	1380
Bolts	Ea	1250	1250	830	8330
Connection Angle	L.F.	294	294	196	1960
Welded Wire Fabric	C.S.F.	12840	12840	4300	813.4
Concrete deck topping	CY	12840	12840	4300	81340
Partitions	L.F.	750	750	130	4630
Shear Wall System Items	Unit	Quantity Per Level			Total
		B2	B1	Typ	
CMU	S.F.	1650	1510	1140	8860
Rebar (#5's @ 24" O.C.)	Ton	0.51	0.47	0.36	2.78

Table D.2: Quantities found for Steel Addition

Item Quantities					
Gravity System Items	Unit	Quantity Per Level			Total
		Typ. Level	12th Level	Penthouse	
5-ply CLT Panels (including visual grading)	S.F.	10780	0	5500	59400
7-ply CLT Panels (including visual grading)	S.F.	1560	10780	0	18580
9-ply CLT Panels (including visual grading)	S.F.	0	1560	0	1560
Double 3-ply Partitions	S.F.	6600	7400	1990	42390
Wall Insulation	S.F.	5980	6704	1803	38405
Studs 2" x 3", pneumatic nailed	MBF	9	10	3	56
Sound Attenuation for Floor	S.F.	12340	12340	5500	79540
Glulam Typ Beams	Ea	27	27	18	180
Glulam Perimeter Beams	Ea	20	20	12	132
Glulam Columns	MBF	3640	4110	3760	26070
Shear Wall System Items	Unit	Quantity Per Level			Total
		Existing Typ.	Addition Typ.	Penthouse	
Cast in Place Concrete	C.Y.	50	50	64	714
Rebar (#4's @ 18" O.C.)	Ton	0.51	0.51	0.705	7.335

Table D.3: Quantities found for Wood Addition

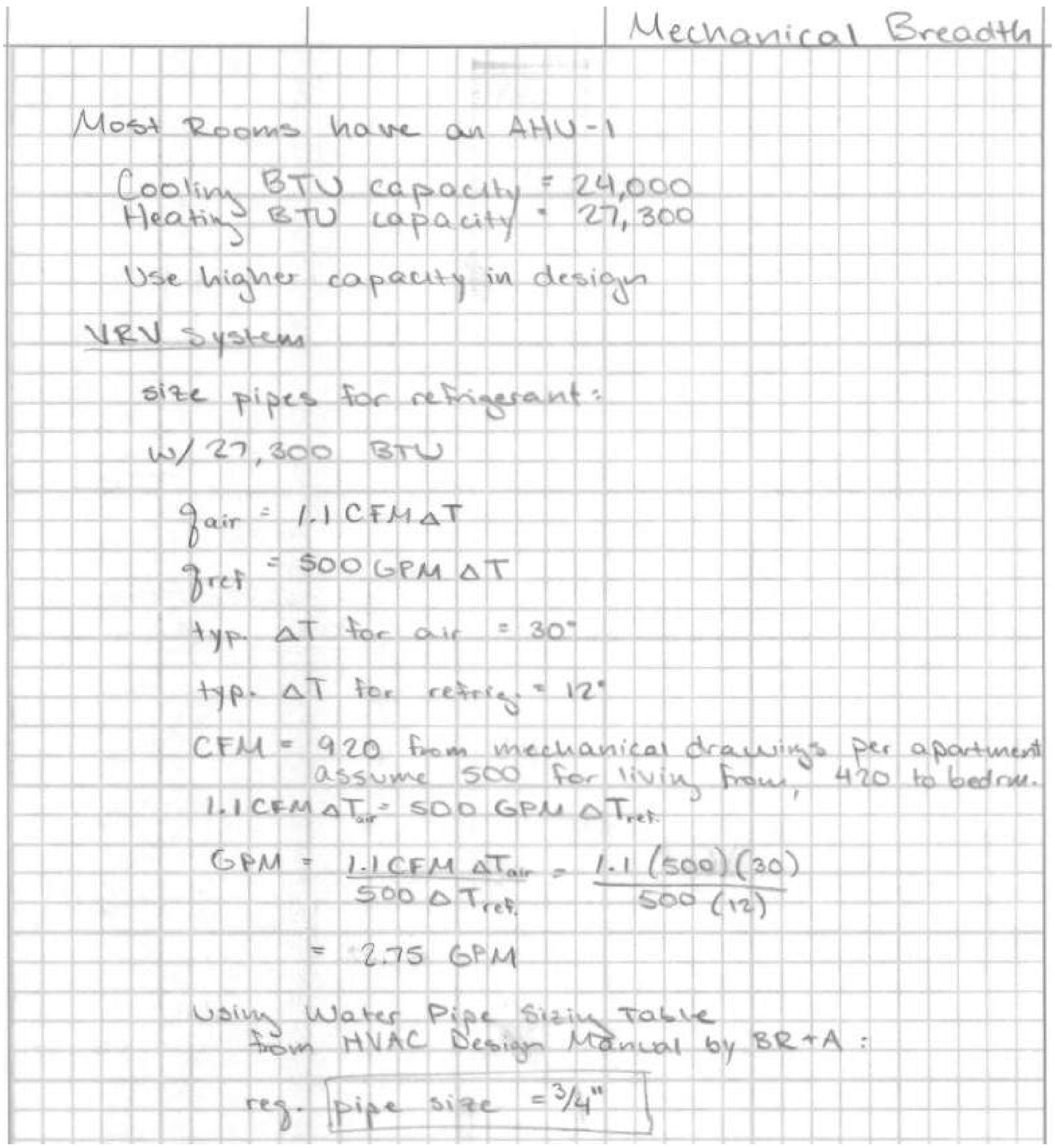
Schedule Analysis: 11141 Georgia Ave Existing Addition						
Item	Qty	Crew Type	# on Crew	Daily Output	Labor Hours	Hrs per item
W10x49	3058	E-2	8	550	0.102	39.0
W12x22	2016	E-2	8	880	0.064	16.1
W10x33	4788	E-2	8	550	0.102	61.0
W12x35	625	E-2	8	810	0.069	5.4
W16x26	310	E-2	8	1000	0.056	2.2
W14x22	630	E-2	8	990	0.057	4.5
Open Web Joist 12K3	16200	E-7	13	1500	0.053	66.0
Open Web Joist 16K3	1100	E-7	13	1800	0.044	3.7
Floor Decking, Composite decking, 1.5" deep, 20 ga.	77040	E-4	8	3800	0.008	77.0
Roof Decking, under 50 squares, 1.5" deep, 22 ga.	4300	E-4	8	4500	0.007	3.8
Weld, 4 passes, 1/2" thick plus avg 150% for half overhead	522	E-14	2	22	0.364	95.0
Weld, 4 passes, 1/2" thick + 20% for vertical	1380	E-14	2	22	0.364	251.2
3/4" diameter bolts 2" long	8330	1 Sswk	1	120	0.067	558.1
Angles, 3"x3"	1960	2 Sswk	2	500	0.032	31.4
Welded Wire Fabric 6x6 W2.1xW2.1	813.4	2 Rodm	2	31	0.516	209.9
Elevated Slab, regular 4000 psi conc., 2-1/2" thick floor fill	81340	C-8	8	2685	0.022	223.7
Framing, stud walls, 10' high, 6" wide, studs 12" O.C.	4630	2 Carp	2	51	0.314	726.9
8" CMU solid grouted reinforced altenate cources	8860	D-8	5	355	0.113	200.2
Reinforcing in place, walls, #3 to #7	2.78	4 Rodm	4	3	10.667	7.4
					Total (days)	322.8
					Weeks (5 d/wk)	64.6
					Months (4 wk/m)	16.1

Table D.4: Scheduling time found for Steel Addition

Schedule Analysis: 11141 Georgia Ave Wood Addition Redesign						
Item	Qty	Crew Type	# on Crew	Daily Output	Labor Hours	Hrs per item
03 41 13.50 Precaset Slab Planks (5-ply CLT)	59400	C-11	10	2400	0.03	178.2
03 41 13.50 Precaset Slab Planks (7-ply CLT)	18580	C-11	10	2800	0.026	48.3
03 41 13.50 Precaset Slab Planks (9-ply CLT)	1560	C-11	10	3200	0.023	3.6
03 47 13.40 Tilt-up walls (Double 3-ply Partitions)	42390	C-14	19	1600	0.09	200.8
Mineral Wool Wall Insulation	38405	1 Carp	1	1600	0.005	192.0
2x3 Studs in wall	56	2 Carp	2	22.222	0.72	20.3
Sound Attenuation for Floor	79540	1 Caro	2	1600	0.0005	19.9
Straight Glulam Beam, 20' span, 6.75" x 15" (Typ Beams)	180	F-3	6	29	1.379	41.4
Straight Glulam Beam, 20' span, 6.75" x 18" (Perim. Beams)	132	F-3	6	28	1.429	31.4
Alternate Pricing, columns including hardware	26.07	F-3	6	2	20	86.9
Wall, free-standing, 8" thick	714	C-14D	27	45.83	4.364	115.4
Reinforcing in place, walls, #3 to #7	7.335	4 Rodm	4	3	10.667	19.6
					Total (days)	119.7
					Weeks (5 d/wk)	23.9
					Months (4 wk/m)	6.0

Table D.5: Scheduling time found for Wood Addition

D.1.2 Mechanical Breadth



Mechanical Breadth

Most Rooms have an AHU-1

Cooling BTU capacity = 24,000
Heating BTU capacity = 27,300

Use higher capacity in design

<u>VRV System</u>

size pipes for refrigerant:

w/ 27,300 BTU

$q_{air} = 1.1\,CFM\,\Delta T$

$q_{ref} = 500\,GPM\,\Delta T$

typ. ΔT for air = 30°

typ. ΔT for refrig. = 12°

CFM = 920 from mechanical drawings per apartment
assume 500 for living room, 420 to bedrm.

$1.1\,CFM\,\Delta T_{air} = 500\,GPM\,\Delta T_{ref.}$

$$GPM = \frac{1.1\,CFM\,\Delta T_{air}}{500\,\Delta T_{ref.}} = \frac{1.1(500)(30)}{500(12)}$$

$= 2.75$ GPM

Using Water Pipe Sizing Table
from HVAC Design Manual by BR+A:

req. $\boxed{\text{pipe size} = 3/4''}$

Figure D.1: Mechanical Equipment Sizing Calculations