Appendices

A. Wind Load Calculations for Proposed System

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Appendix A

Wind Loading for Proposed System
(Unchanged from Existing System)

Assumptions based on criteria listed on construction drawings and documents, and verified using the BOCA 1996 Building Code.

Coefficients and Categories

Exposure Category: B (BOCA 1996 1609.4)
Worst Case L/B Ratio: (73.5 ft)/(164 ft) = 0.448
Basic Wind Speed (V): 80 mph (Figure 1609.3 – Wilmington, DE)
Basic Velocity Pressure (P_v): 16.4 psf (Table 1609.7(3) based on V = 80 mph)
Wall Pressure Coefficients (C_p): For N-S Direction (Table 1609.7)
- Windward Walls: $C_p = 0.8$
- Leeward Walls: $C_p = -0.5$

Wall Pressure Coefficients (C_p): For W-E Direction (Table 1609.7)
- Windward Walls: $C_p = 0.8$
- Leeward Walls: $C_p = -0.3$

Importance Factor (I): 1.04 (Table 1609.5 and interpolation)
Internal Pressure Coefficients (GC_{pi}): +/- 0.25 (Table 1609.7(6))
Velocity Pressure Exposure (K_z and K_h): see below (Table 1609.7(4))
Gust Response Factors (G_h and G_z): see below (Table 1609.7(5))

Building Main Windforce-Resisting System:
- Windward wall design pressure, P

$P = (P_v)(I)[(K_z)(G_h)(C_p) - (K_h)(GC_{pi})]$

- Leeward wall design pressure, P

$$P = (P_v)(I)[(K_z)(G_h)(C_p) - (K_h)(GC_{pi})]$$

North-South Direction Wind Loading Data

Level	Elev. (ft)	K coeff	G coeff.	P (windward)	P (leeward)	Total P (psf)
Roof	279.22	1.84	1.09	27.51	-17.19	44.71
25	269.22	1.83	1.10	27.29	-17.19	44.49
24	259.39	1.81	1.10	27.07	-17.19	44.27
23	247.36	1.78	1.10	26.80	-17.19	44.00
22	236.00	1.76	1.10	26.47	-17.19	43.67
21	225.75	1.74	1.11	26.19	-17.19	43.38
20	215.50	1.71	1.11	25.89	-17.19	43.08
19	205.25	1.69	1.11	25.65	-17.19	42.84
18	195.00	1.67	1.11	25.33	-17.19	42.53
17	184.75	1.64	1.12	25.05	-17.19	42.24
16	174.50	1.62	1.12	24.76	-17.19	41.96
15	164.25	1.59	1.13	24.49	-17.19	41.68
14	154.00	1.56	1.13	24.15	-17.19	41.34
13	143.75	1.53	1.14	23.80	-17.19	40.99
12	133.50	1.50	1.15	23.43	-17.19	40.63
11	123.25	1.46	1.15	22.91	-17.19	40.10
10	113.00	1.43	1.16	22.54	-17.19	39.73
9	102.75	1.39	1.16	21.98	-17.19	39.18
8	92.50	1.35	1.17	21.52	-17.19	38.71
7	82.25	1.30	1.18	20.90	-17.19	38.09
6	72.00	1.25	1.19	20.26	-17.19	37.46
5	61.75	1.20	1.20	19.62	-17.19	36.81
4	51.50	1.14	1.21	18.81	-17.19	36.00
3	41.25	1.07	1.23	17.93	-17.19	35.12
2	31.00	0.99	1.26	16.98	-17.19	34.17
1	10.50	0.80	1.32	14.41	-17.19	31.60

North-South Direction Wind Loading Data (continued)

Level	Trib. Width	Trib Height	P (plf)	F (kips)
Roof	172.5	10.00	7711.74	77.12
25	172.5	9.83	7673.98	75.44
24	172.5	12.03	7636.05	91.86
23	172.5	11.36	7589.21	86.21
22	172.5	10.25	7532.70	77.21
21	172.5	10.25	7483.76	76.71
20	172.5	10.25	7432.01	76.18
19	172.5	10.25	7390.62	75.75
18	172.5	10.25	7335.71	75.19
17	172.5	10.25	7286.92	74.69
16	172.5	10.25	7237.52	74.18
15	172.5	10.25	7190.17	73.70
14	172.5	10.25	7131.55	73.10
13	172.5	10.25	7071.43	72.48
12	172.5	10.25	7008.58	71.84
11	172.5	10.25	6917.96	70.91
10	172.5	10.25	6853.59	70.25
9	172.5	10.25	6757.93	69.27
8	180.75	10.25	6996.77	71.72
7	180.75	10.25	6884.80	70.57
6	180.75	10.25	6770.36	69.40
5	180.75	10.25	6653.46	68.20
4	180.75	10.25	6507.11	66.70
3	180.75	10.25	6348.52	65.07
2	180.75	20.50	6177.04	126.63
1	180.75	10.50	5712.32	59.98
			Sum of F:	1960.35

East-West Direction Wind Loading Data

Level	Elev. (ft)	K coeff	G coeff.	P (windward)	P (leeward)	Total P (psf)
Roof	279.22	1.84	1.09	27.51	-10.32	37.83
25	269.22	1.83	1.10	27.29	-10.32	37.61
24	259.39	1.81	1.10	27.07	-10.32	37.39
23	247.36	1.78	1.10	26.80	-10.32	37.12
22	236.00	1.76	1.10	26.47	-10.32	36.79
21	225.75	1.74	1.11	26.19	-10.32	36.51
20	215.50	1.71	1.11	25.89	-10.32	36.21
19	205.25	1.69	1.11	25.65	-10.32	35.97
18	195.00	1.67	1.11	25.33	-10.32	35.65
17	184.75	1.64	1.12	25.05	-10.32	35.37
16	174.50	1.62	1.12	24.76	-10.32	35.08
15	164.25	1.59	1.13	24.49	-10.32	34.80
14	154.00	1.56	1.13	24.15	-10.32	34.46
13	143.75	1.53	1.14	23.80	-10.32	34.12
12	133.50	1.50	1.15	23.43	-10.32	33.75
11	123.25	1.46	1.15	22.91	-10.32	33.23
10	113.00	1.43	1.16	22.54	-10.32	32.85
9	102.75	1.39	1.16	21.98	-10.32	32.30
8	92.50	1.35	1.17	21.52	-10.32	31.83
7	82.25	1.30	1.18	20.90	-10.32	31.21
6	72.00	1.25	1.19	20.26	-10.32	30.58
5	61.75	1.20	1.20	19.62	-10.32	29.93
4	51.50	1.14	1.21	18.81	-10.32	29.12
3	41.25	1.07	1.23	17.93	-10.32	28.25
2	31.00	0.99	1.26	16.98	-10.32	27.30
1	10.50	0.80	1.32	14.41	-10.32	24.73

East-West Direction Wind Loading Data (continued)

Level	Trib. Width	Trib Height	P (plf)	F (kips)
Roof	72.1667	10.00	2729.92	27.30
25	72.1667	9.83	2714.12	26.68
24	72.1667	12.03	2698.25	32.46
23	72.1667	11.36	2678.66	30.43
22	72.1667	10.25	2655.02	27.21
21	72.1667	10.25	2634.54	27.00
20	72.1667	10.25	2612.89	26.78
19	72.1667	10.25	2595.57	26.60
18	72.1667	10.25	2572.60	26.37
17	72.1667	10.25	2552.19	26.16
16	72.1667	10.25	2531.53	25.95
15	72.1667	10.25	2511.71	25.75
14	72.1667	10.25	2487.19	25.49
13	72.1667	10.25	2462.04	25.24
12	72.1667	10.25	2435.75	24.97
11	72.1667	10.25	2397.83	24.58
10	72.1667	10.25	2370.90	24.30
9	72.1667	10.25	2330.88	23.89
8	104.417	10.25	3323.77	34.07
7	104.417	10.25	3259.09	33.41
6	104.417	10.25	3192.98	32.73
5	104.417	10.25	3125.45	32.04
4	104.417	10.25	3040.90	31.17
3	104.417	10.25	2949.29	30.23
2	104.417	20.50	2850.23	58.43
1	104.417	10.50	2581.77	27.11
			Sum of F:	756.34

Overturning Moments due to Controlling Wind Case: N-S			
Level	F (kips)	Elev (ft)	M (kip-ft)
Roof	77.12	279.22	21532.72
25	75.44	269.22	20308.67
24	91.86	259.39	23828.01
23	86.21	247.36	21325.75
22	77.21	236.00	18221.61
21	76.71	225.75	17316.96
20	76.18	215.50	16416.39
19	75.75	205.25	15548.48
18	75.19	195.00	14662.25
17	74.69	184.75	13799.16
16	74.18	174.50	12945.22
15	73.70	164.25	12105.09
14	73.10	154.00	11257.16
13	72.48	143.75	10419.3
12	71.84	133.50	9590.367
11	70.91	123.25	8739.549
10	70.25	113.00	7938.168
9	69.27	102.75	7117.37
8	71.72	92.50	6633.809
7	70.57	82.25	5804.313
6	69.40	72.00	4996.526
5	68.20	61.75	4211.223
4	66.70	51.50	3434.939
3	65.07	41.25	2684.235
2	126.63	31.00	3925.511
1	59.98	10.50	629.7831
Total Overturning Moment:			295,392.5 kip-ft

$T_{overturning} = C_{overturning}$ = Moment/Span = (295,392.5 kip/ft)/(182.72 ft) = 1,616.64 kip

C_{weight} = (Weight/2) = (42650.38 kips)/2 = 21,325.19 kips

Since $C_{weight} > C_{overturning}$, the weight of the building eliminates chance of overturning

Appendix B

Seismic Load Calculations for Proposed System
(Changes from Original System Noted Below)

Assumptions based on criteria listed on construction drawings and documents, and verified using the BOCA 1996 Building Code.

Seismic Hazard Exposure Group: II (Table 1610.1.5 – Substantial occupancy building)
Effective Peak Velocity-related Acceleration: $A_v = 0.075$
(Wilmington, DE – Figure 1610.1.3(1): halfway between 0.05 and 0.10 regions)
Effective Peak Acceleration Coefficient: $A_a = 0.05$ (Wilmington, DE – Figure 1610.1.3(2))
Seismic Performance Category: B (Table 1610.1.7 since $0.05 < A_v < 0.10$)
Seismic Resisting System: Combination of concentric braced frames (at the staggered trusses) and ordinary moment frames
- Response Modification Factor (R): 5.0 *(Compare to R = 8.0 of original system)*
- Deflection Amplification Factor (Cd): 4.5 *(Compare to R = 6.5 of original system)*

Site Coefficient: S_4, 2.0 (Table 1610.3.1)

Use Equivalent Lateral Force Procedure (Section 1610.3.5.2)

$V = (C_s)(W)$

Seismic Design Coefficient (C_s): (Section 1610.4.1.1)
min of $C_s = (1.2A_vS) / (RT)^{(2/3)}$ = See below
…and $(2.5A_a)/(R) = (2.5)(0.05)/(5.0) = \mathbf{0.025}$

Approximate Fundamental Period (T_a):
$T_a = (C_T)(h_n)^{(3/4)}$
$C_T = 0.02$ (Section 1610.4.1.2.1: Seismic resisting systems with concentrically braced frames) *(Unchanged from original system)*
$h_n = 279.22$ ft (Section 1610.4.1.2.1: Height from base to highest level of building)
$T_a = (0.02)(279.22)^{(3/4)} = 1.366$ seconds

Coefficient for Upper Limit on Calculated Period (C_a): 1.7 (Table 1610.4.1.2)
Fundamental Period (T): $T = (C_a)(T_a)$
$T = (1.7)(1.366) = 2.322$ seconds

$C_s = [(1.2)(0.075)(2.0)]/[(5.0)(2.322)]^{(2/3)} = \mathbf{0.0351} > 0.025 \rightarrow$ Use $C_s = 0.025$

$V = (C_s)(W_{total}) = (0.025)(42,650.381 \text{ kips}) = \mathbf{1066.26\ kips}$

Story Drift Based on Requirements from BOCA 1996 Building Code

Story Drift: $\Delta_a = 0.015(h_{sx})$			
Level	**h_{sx} (ft)**	**Δ_a (ft)**	**Δ_a (in)**
25	279.22	4.19	50.26
24	269.22	4.04	48.46
23	259.39	3.89	46.69
22	247.36	3.71	44.52
21	236.00	3.54	42.48
20	225.75	3.39	40.64
19	215.50	3.23	38.79
18	205.25	3.08	36.95
17	195.00	2.93	35.10
16	184.75	2.77	33.26
15	174.50	2.62	31.41
14	164.25	2.46	29.57
13	154.00	2.31	27.72
12	143.75	2.16	25.88
11	133.50	2.00	24.03
10	123.25	1.85	22.19
9	113.00	1.70	20.34
8	102.75	1.54	18.50
7	92.50	1.39	16.65
6	82.25	1.23	14.81
5	72.00	1.08	12.96
4	61.75	0.93	11.12
3	51.50	0.77	9.27
2	41.25	0.62	7.43

Appendix C

Staggered Truss Structural Calculations for Selected Members

Hollow Core Slab System Selection

- Superimposed Dead Load:
    7 psf for ceiling/mechanical (presumed)
    5 psf for collateral (listed on drawings)

- Worst Case Live Load (typical floor): 70 psf

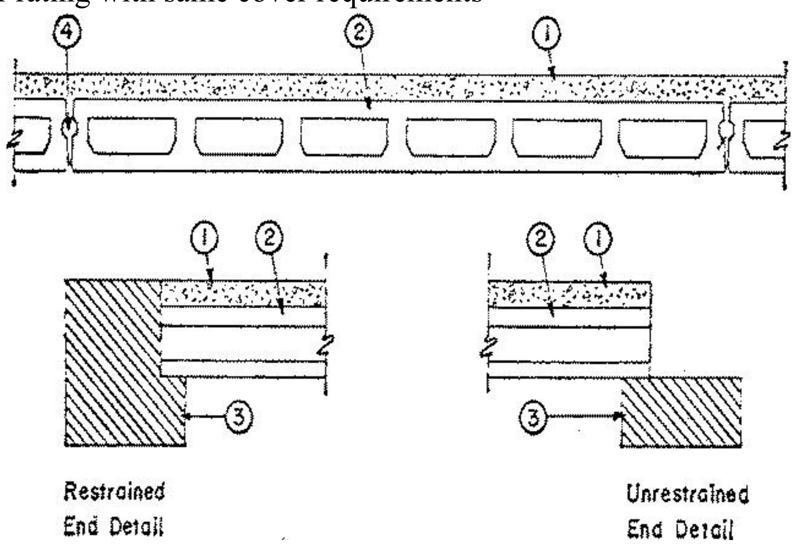
- **Total Superimposed Load (unfactored)** = 70 psf + 7 psf + 5 psf = **82 psf**
- Total Superimposed Load (factored) = 1.2(12 psf) + 1.4(70psf) = 114.8 psf

- *From Nitterhouse Concrete Products (see following data chart):*

    - Max Span = 28'-6" → 29' for design

    - Choose **8" × 4' Prestressed Concrete SpanDeck with 2" topping (U.L. J917)**
        - f'_c = 5000 psi at 28 days, 3000 psi at release
        - Precast density = 150 pcf (top and webs), 115 pcf (soffit)
        - Allowable Superimposed Load for 24' span = 112 psf (flexure) > 82 psf req'd
        - (4) ½" diameter, 270 ksi Low-Relaxation Strands at 2" height
        - Precast System Weight = 330 plf = 82.5 psf

Fire Rating from Underwriters' Laboratories
- Restrained end: 2 in. concrete cover (1 in. gypsum board) required for 2 hour fire rating
- Unrestrained end: 1 ½ hour rating with same cover requirements



U.L. Assembly Diagram Key

1.) Floor Topping (concrete, gypsum, or floor mat material)
2.) Precast Plank
3.) Min. 1.5" End Bearing Detail
4.) Grout: 3500 psi

Courtesy Nitterhouse Concrete Products, Inc.:
http://www.nitterhouse.com/DrawingSpecs/DrawingsSpecs.html

Prestressed Concrete
8" x 4' SpanDeck – U.L. – J952
(2" C.I.P. TOPPING)

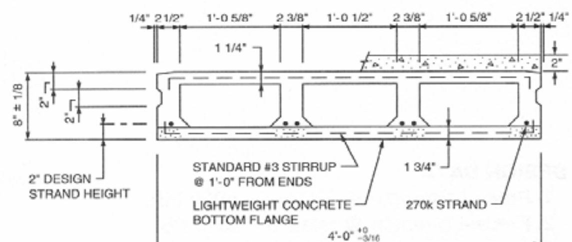
PHYSICAL PROPERTIES

Composite

A' = 295 in.^{2}	S'$_b$ = 468 in.^{3}
I' = 2624 in.^{4}	S'$_t$ = 1096 in.^{3} (At Top of SpanDeck)
Y$_{b'}$ = 5.61 in.	S'$_{tt}$ = 597 in.^{3} (At Top of Topping)
Y$_t$' = 2.39 in. (To Top of SpanDeck)	Wt.' = 330 PLF
Y$_{tt}$' = 4.39 in. (To Top of Topping)	Wt.' = 82.5 PSF

DESIGN DATA

1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Strength @ release = 3000 PSI.
3. Precast Density = 150 PCF (Top and Webs)
   = 115 PCF (Soffit)
4. Strand = 1/2"ø, 270K Lo-Relaxation.
5. Composite Strength = 3000 PSI.
6. Composite Density = 150 PCF.
7. Strand Height = 2.00 in.
8. Ultimate moment capacities (when fully developed) . . .
   4 – 1/2"ø, 270K = 88.3'K
   6 – 1/2"ø, 270K = 124.0'K
9. Maximum bottom tensile stress is $6\sqrt{f'c}$ = 424 PSI.
10. All superimposed load is treated as live load in the strength analysis of flexure and shear.
11. Flexural strength capacity is based on stress/strain strand relationships.
12. Shear values are the maximum allowable before shear reinforcement is required.
13. Deflection limits were not considered when determining allowable loads in this table.
14. Load values to the left of the solid line are controlled by ultimate strength. Load values to the right are controlled by service stress.
15. All loads shown refer to allowable loads applied after topping has hardened.



8" SPANDECK CROSS SECTION

UL FIRE RATED J952

8" SPANDECK W/2" TOPPING				ALLOWABLE SUPERIMPOSED LOAD (PSF)																					
STRAND PATTERN				SPAN (FEET)																					
				10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Flexure	4	–	1/2"ø	750	675	611	546	462	394	338	291	252	218	191	167	146	128	112	98	85	74	63	51	41	31	
Shear	4	–	1/2"ø	527	469	421	382	348	317	294	272	252	235	219	197	176	157	140	129	122	110	98	88	78	70	
Flexure	6	–	1/2"ø	1098	900	898	794	676	580	502	437	382	336	296	262	233	207	185	165	147	132	116	101	87	74	63
Shear	6	–	1/2"ø	542	483	434	393	359	329	303	280	261	243	227	212	199	188	178	167	152	137	124	112	101	91	86

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths.

NITTERHOUSE
CONCRETE PRODUCTS

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Chambersburg, PA 17201-0813
717-267-4505 • FAX: 717-267-4518

REVISED 12/93

Staggered Truss Design

Even Floor Centroids	
Truss	x_i (ft)
2	29.25
4	71.583
6	114.917
Sum =	215.75
x_{even} = (215.75)/3 = 71.92ft	

Odd Floor Centroids	
Truss	x_i (ft)
3	43.083
5	86.627
Sum =	129.71
x_{odd} = (129.71)/2 = 64.85 ft	

Torsional Rigidity, Even Floors		
Truss	$Xbar_i$ (ft)	$Xbar_i^2$ (ft^2)
2T	-42.67	1820.44
4T	-0.33	0.11
6T	43.00	1849.00
	Sum =	3669.56

Torsional Rigidity, Odd Floors		
Truss	$Xbar_i$ (ft)	$Xbar_i^2$ (ft^2)
3T	-21.77	474.00
5T	21.77	474.00
	Sum =	948.00

Shear Force in Each Truss Due to Lateral Loads (Bottom Floor)								
Truss	x_i (ft)	V_s (k)	T = 23227.71 (ft-k)		T = -8207.40 (ft-k)		Design V_i	Φ_{ecc}
			V_{tors}	V_i	V_{tors}	V_i		
3T	43.08	905.91	-533.44	372.47	188.49	1094.40	1094.40	1.00
5T	86.63	905.91	533.44	1439.35	-188.49	717.42	1439.35	1.32
			T = 10433.07 (ft-k)		T = -21002.03 (ft-k)			
2T	29.25	603.94	-121.31	482.63	244.19	848.14	848.14	1.40
4T	71.58	603.94	-0.95	602.99	1.91	605.85	605.85	1.00
6T	114.92	603.94	122.26	726.20	-246.10	357.84	726.20	1.20

(Assuming each truss has approximately equal shear rigidity (GA))

$x = (\sum x_i GA_i)/(\sum GA_i)$

Xbar = $x_{even} - x_i$ or $x_{odd} - x_i$

Using the centroid of lower levels (1-7th floors):

e_{even} = Centroid – x_{even} = 69 ft – 71.92 ft = -2.92 ft

e_{odd} = Centroid - x_{odd} = 69 – 64.85 = 4.15 ft

After adding accidental torsional eccentricity (5% of total width):

e_{even} = -2.92 ft ± (0.05*173.5 ft) = 5.76 ft or -11.59 ft

e_{odd} = 4.15 ft ± (0.05*173.5 ft) = 12.82 ft or -4.53 ft

Base torsion calculations: Using base shear V = 1811.82 k (see Appendix A)

$T = 1811.82*(5.76ft) = 10{,}433.07$ ft-k
$T = 1811.82*(-11.59ft) = -21{,}002.03$ ft-k
$T = 1811.82*(12.82ft) = 23{,}227.71$ ft-k
$T = 1811.82*(-4.53ft) = -8{,}207.40$ ft-k

$V_s = (1811.82\text{ k})/2 = 905.51\text{k}$ for odd floors
$V_s = (1811.82\text{k})/3 = 603.94$ k for even floors

Values in table above, where:

$V_{tors} = (T*Xbar_i)/\sum x_i^2$
$V_i = V_s + V_{tors}$

Transverse Shear in Diaphragm (Hollow Core Planks)

$V_u = 1.7*(\Phi_h)*(V)*(0.75) = 1.7*(1.0)*(726.2k)*(0.75) =$ **925.91 k**

Where: Max V_i = 726.6 k from above

$\Phi V_n = \Phi V_c + \Phi V_s$
$\Phi V_c = \Phi*(2*\sqrt{f'_c})*(bd) = (0.85*2*(\sqrt{5000})*6in*0.8*73.5ft*12in/ft*(1k/1000lb) =$ **508.91 kip**
$\Phi V_s = \Phi*A_{vf}*f_y*\mu = (0.85)*(7.92in^2)*(60\text{ ksi})*(1.4) =$ **565.49 kip**

Where: $\mu = 1.4$ (coefficient of friction)
No. of planks = 73.5'/4ft wide planks ≈ 19
No. of joints = 19 – 1 = 18
$A_{vf} = (18\text{ joints})*(0.44\text{ in}^2) = 7.92\text{ in}^2$ (using #6 bars)

$\Phi V_n = 508.91k + 565.49k = 1074.4k > 925.91k$ (OK)

Design of Truss Members

Gravity Loads on Typical Truss Members Using Method of Joints

Dead Loads for precast planks
- 8" precast plank with 2" topping = 82.5 psf
- Leveling compound = 5 psf
- Structural steel = 5 psf (estimate)
- Partitions / MEP = 12 psf

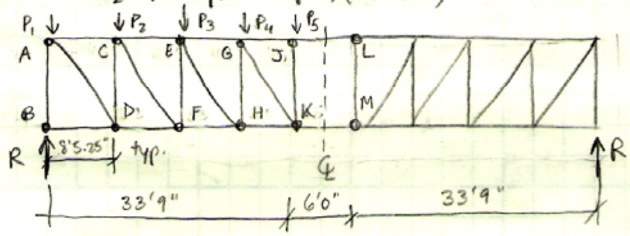
104.5 psf

Live Load (typical floor) 70 psf

Truss #2 ("regular" truss)

Trib width = $(28.4167' + 13.833')/2 = 21.125'$

$w_2 = (104.5\,psf + 70\,psf)(21.125') = 3.87$ kip/ft



$P_1 = (3.87\,klf)(8.354'/2) = 16.165$ kip

$P_2 = (3.87\,klf)(8.354') = 32.33$ kip

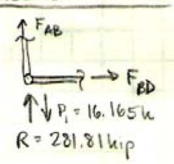
$P_3 = P_4 = 32.33$ kip

$P_5 = (3.87\,klf)\left(\frac{8.354'}{2} + \frac{6'}{2}\right) = 27.78$ kip

$R = (16.165k + 3(32.33k) + 27.75k) \times \underbrace{2}_{\text{top and bottom}} = 281.82$ kip

Method of Joints ($\Sigma F_x = 0;\ \Sigma F_y = 0$)

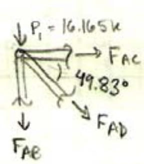
B



$F_{AB} = -281.81k + 16.165k = -265.65$ kip

$F_{BD} = 0$ kip

A

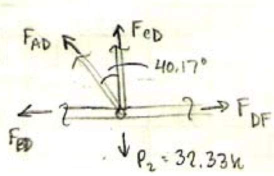


$F_{AC} = -F_{AD}\cos 49.83° = -0.645\,F_{AD} \rightarrow F_{AC} = -210.22$

$-16.165k - F_{AB} = F_{AD}\sin 49.83°$

$F_{AD} = \dfrac{-16.165 - (-265.65)}{0.764} = 325.92$ kip

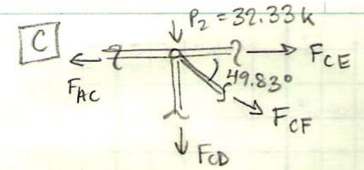
D



$F_{CD} = -F_{AD}\cos 40.17° + 32.33 = -216.67$ kip

$F_{DF} = \cancel{F_{BD}} + F_{AD}\sin 40.17°$

$= (325.92k)(0.645) = 210.22$ kip

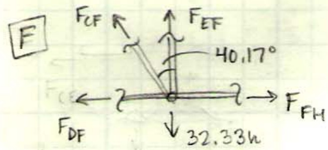


$-32.33k - F_{CD} = F_{CF} \sin 49.83°$

$F_{CF} = \dfrac{-32.33 - (-216.67)}{0.764} = \underline{241.28 \text{ kip}}$

$F_{AC} = F_{CE} + F_{CF} \cos 49.83°$

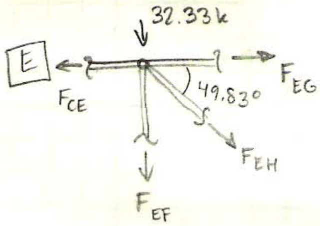
$F_{CE} = -(241.38)(0.645) + (-210.22k) = \underline{-365.91 \text{ kip}}$



$F_{EF} = -F_{CF} \cos 40.17° + 32.33 = \underline{-152 \text{ kip}}$

$F_{FH} = F_{DF} + F_{CF} \sin 40.17°$

$F_{FH} = (210.22k) + (241.28k)(0.645) = \underline{365.85 \text{ kip}}$



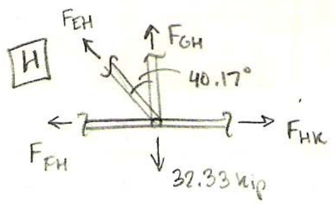
$F_{EF} + F_{EH} \sin 49.83° = -32.33k$

$F_{EH} = \dfrac{-32.33k - (-152.0)}{0.764} = \underline{156.64 \text{ kip}}$

$F_{CE} = F_{EG} + F_{EH} \cos 49.83°$

$F_{EG} = F_{CE} - F_{EH}(0.645) = -365.91 - (156.64)(0.645)$

$\underline{F_{EG} = -466.94 \text{ kip}}$

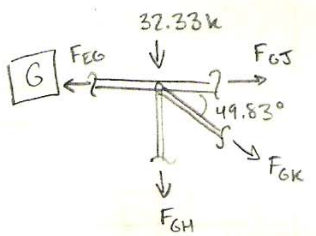


$F_{GH} = -F_{EH} \cos 40.17° + 32.33k$

$F_{GH} = -(0.764)(156.64k) + 32.33 = \underline{-87.34 \text{ kip}}$

$F_{HK} = F_{FH} + F_{EH} \sin 40.17°$

$= (365.85k) + (156.64k)(0.645) = \underline{466.88 \text{ kip}}$



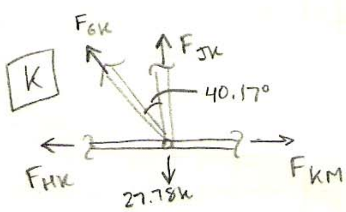
$F_{GK} \sin 49.83° = -32.33 - F_{GH}$

$F_{GK} = \dfrac{-F_{GH}}{0.764} - \dfrac{32.33}{0.764} = \dfrac{-(-87.34) - 32.33}{0.764} = \underline{72.0 \text{ kip}}$

$F_{EG} = F_{GJ} + F_{GK} \cos 49.83°$

$F_{GJ} = F_{EG} - F_{GK} \cos 49.83°$

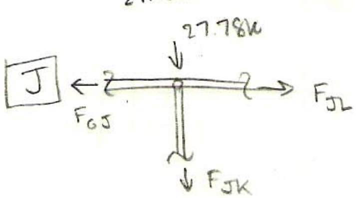
$F_{GJ} = (-466.88) - (72.0k)(0.645) = \underline{-513.32 \text{ kip}}$



$F_{JK} = -F_{GK} \cos 40.17° + 27.78k = \underline{-27.28 \text{ kip}}$

$F_{KM} = F_{HK} + F_{GK} \sin 40.17°$

$= (466.88k) + (72.0k)(0.645) = \underline{513.32 \text{ kip}}$



$F_{JK} = -27.78 \text{ kip}$ (close due to rounding)

$F_{JL} = F_{GJ} = \underline{-513.32 \text{ kip}}$

Lateral Loads on Typical Truss Members Using Method of Joints

Lateral Loads - Truss #2 - regular truss → 73.5' width, 10' tall

Design V_i = 848.14 kips (from spreadsheet)

∴ Horiz. reactions = Design V_i/2 = 424.07 kips

$R = (424.07k)(2)(10')/(73.5') = 115.39\ k$

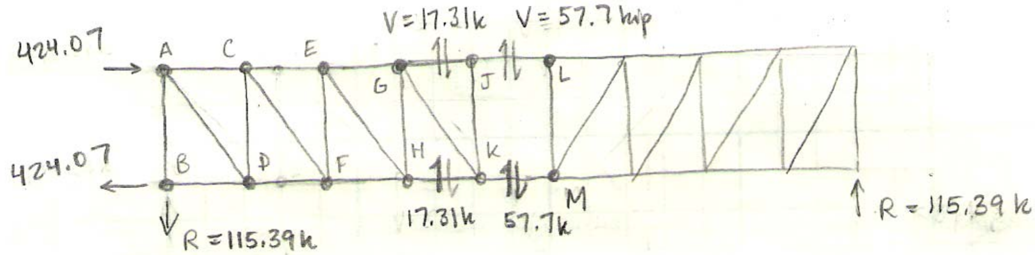
$V_{vierendeel} = \frac{1}{2}(424.07\ k)(10')/(\frac{73.5'}{2}) = \underline{57.70\ kip}$

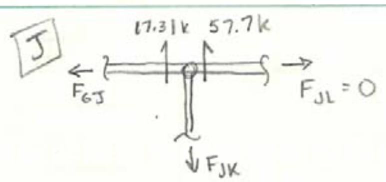
At joint J:

- $M = (57.70k)(6'/2) = 173.09$ ft·kip (6' = width of panel)
- In the panel GJ: (assuming M at other end = 0)

$V_{GJ} = (173.09\ ft\cdot k)/(10') = \underline{17.31\ kip}$

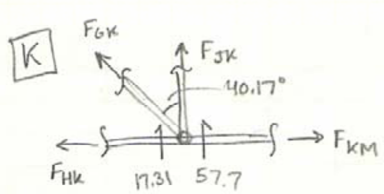
(Assume chord members in remaining members. and shear chord forces also = 0)





$F_{GJ} = F_{JL} = 0$

$F_{JK} = \underline{75.01k}$

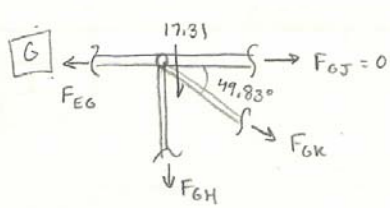


$F_{JK} = -F_{GK}\cos 40.17° - 75.01k$

$F_{GK} = \frac{-2(75.01k)}{0.764} = \underline{-196.35\ k}$

$F_{HK} = -F_{GK}\sin 40.17°$

$F_{HK} = -(-196.35k)(0.645) = \underline{126.64\ k}$

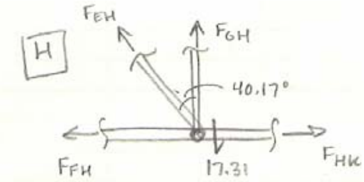


$F_{EG} = F_{GK}\cos 49.83°$

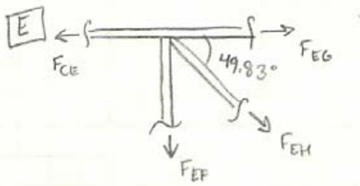
$F_{EG} = (0.645)(-196.35\ k) = \underline{-126.64k}$

$-17.31 = F_{GH} + F_{GK}\sin 49.83°$

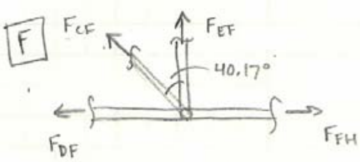
$F_{GH} = -17.31 - (-196.35k)(0.764) = \underline{132.7k}$



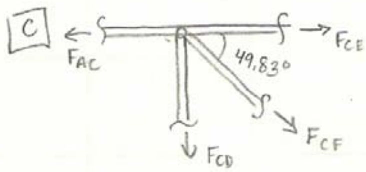
$$17.31 = F_{EH}\cos 40.17^\circ + F_{GH}$$
$$F_{EH} = \frac{17.31 - (132.7k)}{0.764} = \underline{-151.04\,k}$$
$$F_{FH} + F_{EH}\sin 40.17^\circ = F_{HK}$$
$$F_{FH} = (126.64k) - (-151.04k)(0.645) = \underline{224.06k}$$



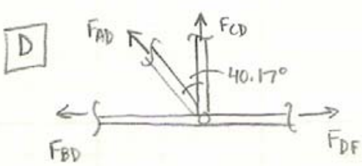
$$F_{CE} = F_{EG} + F_{EH}\cos 49.83^\circ$$
$$F_{CE} = (-126.64k) + (-151.04k)(0.645) = \underline{-224.06\,k}$$
$$F_{EH}\sin 49.83^\circ = -F_{EF}$$
$$F_{EF} = -(-151.04k)(0.764) = \underline{115.39\,k}$$



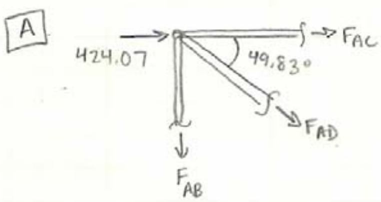
$$F_{EF} = -F_{CF}\cos 40.17^\circ$$
$$F_{CF} = \frac{-(115.39k)}{0.764} = \underline{-151.04\,k}$$
$$F_{DF} + F_{CF}\sin 40.17^\circ = F_{FH}$$
$$F_{DF} = 224.06 - (-151.04)(0.645) = \underline{321.48\,k}$$



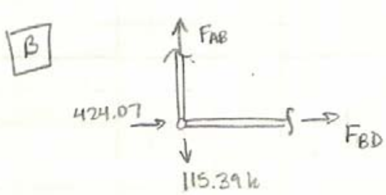
$$F_{AC} = F_{CE} + F_{CF}\cos 49.83^\circ$$
$$F_{AC} = -224.06k + (-151.04)(0.645) = \underline{-321.48k}$$
$$F_{CD} = -F_{CF}\sin 49.83^\circ = -(-151.04)(0.764) = \underline{115.39k}$$



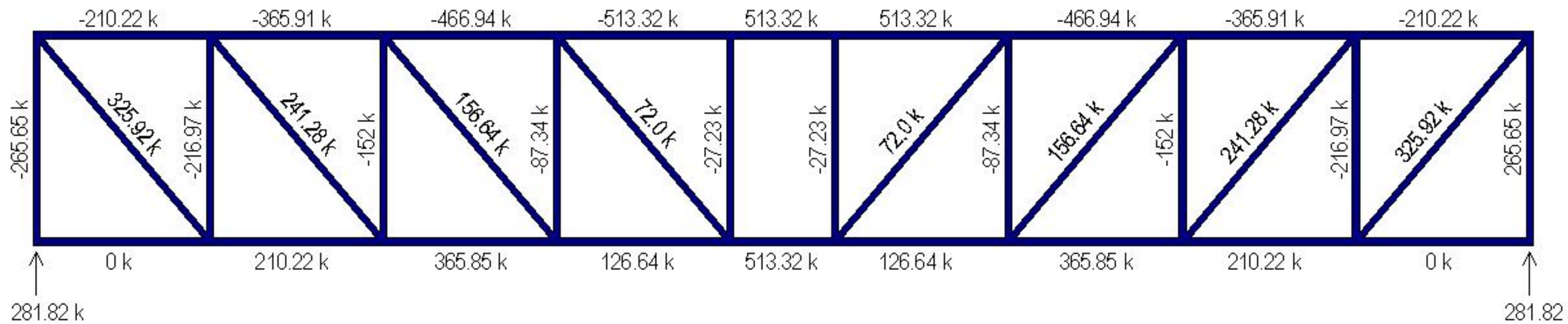
$$F_{CD} = -F_{AD}\cos 40.17^\circ$$
$$F_{AD} = \frac{-115.39k}{0.764} = \underline{-151.04k}$$
$$F_{BD} + F_{AD}\sin 40.17^\circ = F_{DF}$$
$$F_{BD} = 321.48k - (-151.04k)(0.645) = \underline{418.9k}$$



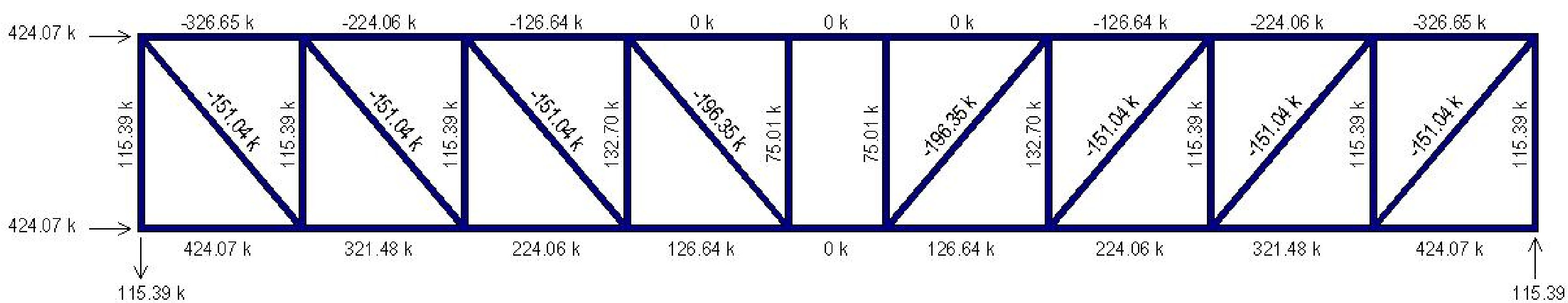
$$-424.07 = F_{AC} + F_{AD}\cos 49.83^\circ$$
$$-424.07 = -321.48 + (-151.04)(0.645)$$
$$-424.07 \approx -419 \quad (\text{rounding error})$$
$$F_{AB} = -F_{AD}\sin 49.83^\circ$$
$$F_{AB} = -(-151.04)(0.764) = \underline{+115.4\,k}$$



$$F_{AB} = +115.4\,k = 115.4\,k$$
$$F_{BD} = 424.07k \approx 418.9k \quad (\text{rounding error})$$



Typical Truss Member Force Diagram: Under Gravity Loading



Typical Truss Member Force Diagram: Under Lateral Loading

Design of Diagonal Member D1 of Truss 3										
Floor	WIND, kips		SEISMIC, kips		LOAD COMBINATIONS, kips					
	Φ_h	$\Phi_{ecc}\ \Phi_h\ F_w$	Φ_h	$\Phi_{ecc}\ \Phi_h\ F_E$	1	2	3	4	Governing Load	Member Sizes
25	0.04	6.56	0.10	5.27	273.84	339.04	277.82	272.59	339.04	HSS 10x4x1/2
24	0.09	13.13	0.20	10.23	273.84	339.04	288.32	277.55	339.04	HSS 10x4x1/2
23	0.13	19.55	0.29	14.79	273.84	339.04	298.59	282.11	339.04	HSS 10x4x1/2
22	0.18	27.37	0.37	19.01	273.84	339.04	311.10	286.33	339.04	HSS 10x4x1/2
21	0.23	34.70	0.44	22.83	273.84	339.04	322.84	290.15	339.04	HSS 10x4x1/2
20	0.27	41.27	0.51	26.38	273.84	339.04	333.36	293.70	339.04	HSS 10x4x1/2
19	0.32	47.80	0.58	29.61	273.84	339.04	343.80	296.93	343.80	HSS 10x4x1/2
18	0.36	54.25	0.63	32.57	273.84	339.04	354.12	299.89	354.12	HSS 10x4x1/2
17	0.40	60.65	0.69	35.25	273.84	339.04	364.36	302.57	364.36	HSS 10x4x1/2
16	0.44	67.01	0.73	37.68	273.84	339.04	374.53	305.00	374.53	HSS 10x4x1/2
15	0.49	73.32	0.78	39.85	273.84	339.04	384.63	307.17	384.63	HSS 10x4x1/2
14	0.53	79.59	0.81	41.79	273.84	339.04	394.67	309.11	394.67	HSS 10x4x1/2
13	0.57	85.81	0.85	43.50	273.84	339.04	404.62	310.82	404.62	HSS 10x4x1/2
12	0.61	91.98	0.88	45.01	273.84	339.04	414.49	312.33	414.49	HSS 10x4x1/2
11	0.65	98.10	0.90	46.32	273.84	339.04	424.27	313.64	424.27	HSS 10x4x1/2
10	0.69	104.13	0.92	47.44	273.84	339.04	433.93	314.76	433.93	HSS 10x4x1/2
9	0.73	110.11	0.94	48.40	273.84	339.04	443.50	315.72	443.50	HSS 10x4x1/2
8	0.77	116.01	0.96	49.20	273.84	339.04	452.93	316.52	452.93	HSS 10x4x1/2
7	0.81	122.11	0.97	49.85	273.84	339.04	462.69	317.17	462.69	HSS 10x4x1/2
6	0.85	128.11	0.98	50.37	273.84	339.04	472.30	317.69	472.30	HSS 10x4x1/2
5	0.89	134.02	0.99	50.77	273.84	339.04	481.75	318.09	481.75	HSS 10x4x1/2
4	0.93	139.83	0.99	51.06	273.84	339.04	491.04	318.38	491.04	HSS 10x4x1/2
3	0.96	145.50	1.00	51.27	273.84	339.04	500.12	318.59	500.12	HSS 10x4x1/2
2	1.00	151.04	1.00	51.41	273.84	339.04	508.98	318.73	508.98	HSS 10x4x1/2
Ground		151.04		51.41	273.84	339.04	267.32	267.32	339.04	HSS 10×4×1/2

Load Factors and Combinations and Truss Chord Design

DL = 105 psf
LL = 70 psf
RLL = 35 psf (due to 50% LL reduction)

$\Phi_W = 1.0$ for typical truss

$\Phi_{ecc} = 1.0$ for truss 3 and 4
2.81 for truss 5
1.04 for truss 2
1.56 for truss 6

(For Level 9)
$\Phi_{h\,wind} = 0.72$
$\Phi_{h\,seismic} = 0.85$

$$\Phi_{L1} = 1.4D = 1.4\left(\frac{105\text{ psf}}{105+70}\right) = 0.84$$

$$\Phi_{L2} = 1.2D + 1.6L = 1.2\left(\frac{105}{175}\right) + 1.6\left(\frac{35}{175}\right) = 1.04$$

$$\Phi_{L3} = 1.2D + 0.5L = 1.2\left(\frac{105}{175}\right) + 0.5\left(\frac{35}{175}\right) = 0.82$$

Load combos (for Diag. Member d_1 of Truss 3 at Level 2)

from Method of Joints (Gravity)

1) $1.4D = \Phi_{L1} F_G = (0.84)(326\text{ kip}) = \underline{274\text{ k}}$

2) $1.2D + 1.6L = \Phi_{L2} F_G = (1.04)(326\text{ k}) = \underline{339\text{ k}}$

3) $1.2D + 1.6W + 0.5L = \Phi_{L3} F_G + \Phi_{ecc}\Phi_h F_W (1.6)$

from Method of Jts (Lat)

$= (0.82)(326\text{k}) + (1.0)(1.6)(1.00)(151.04\text{k}) = \underline{508.98\text{ k}}$

4) $1.2D + 1.0E + 0.5L = \Phi_{L3} F_G + \Phi_{ecc}\Phi_h F_E$

$$F_E = \underbrace{(151.04\text{k})}_{\text{force in chord}}\left(\frac{712.67}{1811.82\text{ k}}\right) = \underline{59.41\text{ k}}$$

Seismic story shear @ 2nd floor
wind shear at Lv. 2
(only want seismic portion of lat. load)

$= (0.82)(326\text{ k}) + (1.0)(1.0)(59.41\text{k}) = \underline{326.73\text{ k}}$

5) $\underline{0.9D} + (\underline{1.3W}$ or $\underline{1.0E}) <$ Case 3, 4 (Will not control)

See Excel for each floor calc

Truss Chords

$$\frac{P_u}{\Phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\Phi_b M_{nx}}\right)(1.0)$$

$\Phi = 0.9$ for tension
$\Phi = 0.85$ for compression
$\Phi_b = 0.9$ for bending

Gravity $\quad w = 3.87$ kip/ft

$\Phi_w = 1.0$

$$M = \frac{w\ell^2}{10} = \frac{(3.87\text{ kip/ft})(10')^2}{10} = 38.7\text{ kip ft}$$

P = 513.32 kip (worst case from Method of Joints)

Truss Chord Design (Continued) and Column Design

Truss Chords (cont.)

$\Phi_{L1} = 0.84$
$\Phi_{L2} = 1.04 \leftarrow$ controls
$\Phi_{L3} = 0.82$

Gravity (cont.)

2) $1.2D + 1.6L$

$P_u = \Phi_{L2} P = (1.04)(513.32\,k) = \underline{533.85\,k}$

$M_u = \Phi_{L2} M = (1.04)(38.7\,kip\cdot ft) = \underline{40.25\,k\cdot ft} = M_{u_G}$ (every story)

3) $1.2D + \cancel{1.6W} + 0.5L$

$P_u = \Phi_{L3} P = (0.82)(513.32\,k) = \underline{420.92\,k}$

$M_u = \Phi_{L3} M = (0.82)(38.7\,k\cdot ft) = \underline{31.73\,ft\cdot k}$

Wind

$M = 58.47\,ft\cdot kip$

$\Phi_{ecc} M = (1.0)(58.47\,ft\cdot kip) = \underline{58.47\,ft\cdot kip}$

$M_u = 1.6(58.47\,ft\cdot kip) = \underline{93.55\,ft\cdot kip}$

Combos

Case #2

$P_u = \underline{533.85\,k}$

$M_u = 40.25\,ft\cdot k$

Case #3

$P_u = 420.92\,k$

$M_u = 31.73\,ft\cdot k + 93.55\,ft\cdot k = \underline{125.28\,ft\cdot kip}$

Column Design

- Exterior Column at truss 4 (greatest trib. area)
- Assume 50% LL reduction
- Carries load from 2 floors (3 when used with a hanger/post)

Axial Force

$\text{Trib. Area} = \left(\frac{28.5' + 28.5'}{2}\right)\left(\frac{73.5'}{2}\right) = \underline{1047.38\,ft^2}$

$DL_1 = \left[(82.5\,psf)_{\text{hollow core}} + (5\,psf)_{\text{structural steel}}\right](1047.38\,ft^2)\left(\frac{1\,kip}{1000\,lb}\right) = \underline{91.65\,k}$

$DL_2 = 104.5\,psf\,(1047.38\,ft^2)\left(\frac{1\,kip}{1000\,lb}\right) = \underline{109.45\,kip}$

$RLL = 35\,psf\,(1047.38\,ft^2)\left(\frac{1\,kip}{1000\,lb}\right) = \underline{36.66\,kip}$

$DL_2 + RLL = 109.45\,k + 36.66\,k = \underline{146.11\,kip}$

Two Floors

$DL_1 = 2(91.65\,k) = 183.3\,kip$

$DL_2 + RLL = 2(146.11\,k) = 292.22\,kip$

(Arch. cladding is self supporting)

Column Design (continued)

$M_{column} = M_{translation} + M_{rotation}$

$$M_{col} = \frac{6EI(\Delta_t + \Delta_b)}{\ell_c^2} + \left(\frac{-3EI\theta}{\ell_c}\right)$$

where $\theta = \dfrac{2\Delta_{TS}}{L}$

$$\therefore M_{col} = \frac{6EI}{\ell_c}\left(\frac{\Delta_t + \Delta_b}{\ell_c} - \frac{\Delta_{TS}}{L}\right)$$

Assume $\Delta_{TS} = 3/4''$ (deflection of truss at midspan due to weights of planks and structural steel)

$L = 73.5'$
$\ell_c = 10'$

$$\Delta_t = \frac{\Sigma P_i L_i}{EA_i}$$

With a top and bottom chord of W10×54 for example:

$$\Delta_t = \left(\frac{87.5\text{ psf}}{104.5 + 70\text{ psf}}\right)\frac{(8.44')(12^{in}/ft)}{(29000\text{ ksi})(15.8\text{ in}^2)}\left(\frac{513.22}{2} + 513.22 + 466.94 + 365.91 + 210.22\right)$$

$\Delta_t = 0.201''$

$$\Delta_b = \left(\frac{87.5\text{psf}}{104.5 + 70\text{psf}}\right)\left[\frac{(8.44')(12^{in}/ft)}{(29000\text{ksi})(15.8\text{in}^2)}\right]\left(0 + 210.22 + 365.85 + 466.88 + \frac{513.32}{2}\right)$$

$\Delta_b = 0.144''$

Try W12×65 column

$$M_{col} = \frac{6(29000\text{ ksi})(174\text{ in}^4)}{(10')(12^{in}/ft)}\left[\frac{0.201'' + 0.144''}{10'(12^{in}/ft)} - \frac{0.75''}{73.5'(12^{in}/ft)}\right] = 510.82\ \text{in}\cdot\text{k}$$

$M_{col} = 42.56\ \text{k}\cdot\text{ft} \sim 43\ \text{kip}\cdot\text{ft}$

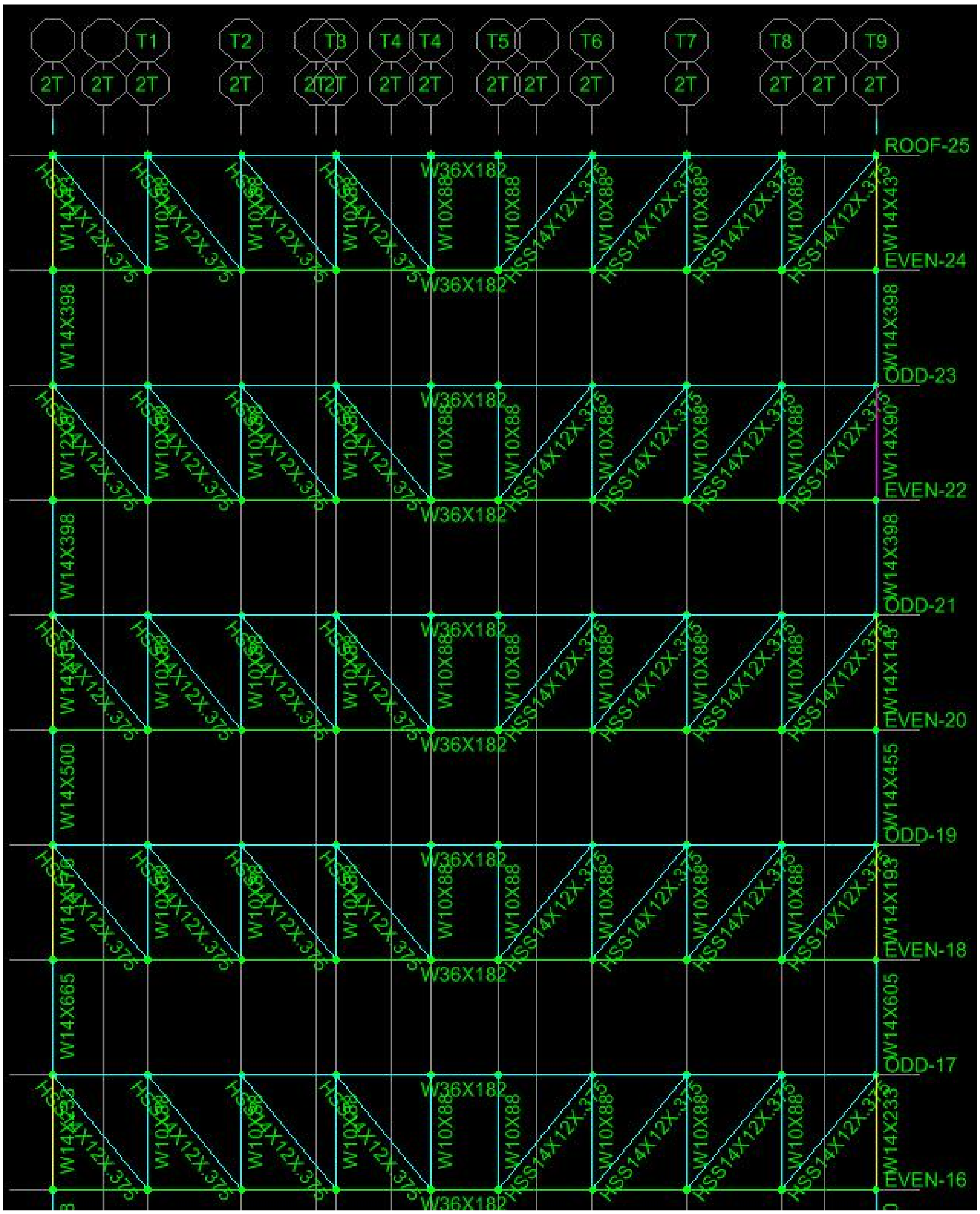
Design of Staggered Truss Chords - Truss 3						
Floor	Φ_h	M_{uw}	Total M_u	P_u	Section	LRFD Eq. 1-1a
25	0.04	4.07	44.32	533.85	W10x54	0.95
24	0.09	8.13	48.38	533.85	W10x54	0.97
23	0.13	12.11	52.36	533.85	W10x54	0.98
22	0.18	16.95	57.20	533.85	W10x54	1.00
21	0.23	21.49	61.74	533.85	W10x54	1.01
20	0.27	25.56	65.81	533.85	W10x60	0.92
19	0.32	29.61	69.86	533.85	W10x60	0.94
18	0.36	33.60	73.85	533.85	W10x60	0.95
17	0.40	37.56	77.81	533.85	W10x60	0.96
16	0.44	41.50	81.75	533.85	W10x60	0.97
15	0.49	45.41	85.66	533.85	W10x60	0.99
14	0.53	49.30	89.55	533.85	W10x60	1.00
13	0.57	53.15	93.40	533.85	W10x60	1.01
12	0.61	56.97	97.22	533.85	W10x68	0.90
11	0.65	60.76	101.01	533.85	W10x68	0.91
10	0.69	64.50	104.75	533.85	W10x68	0.92
9	0.73	68.20	108.45	533.85	W10x68	0.93
8	0.77	71.85	112.10	533.85	W10x68	0.94
7	0.81	75.63	115.88	533.85	W10x68	0.95
6	0.85	79.35	119.60	533.85	W10x68	0.96
5	0.89	83.01	123.26	533.85	W10x68	0.97
4	0.93	86.60	126.85	533.85	W10x68	0.98
3	0.96	90.12	130.37	533.85	W10x68	0.99
2	1.00	93.55	133.80	533.85	W10x68	1.00
25	0.04	4.07	44.32	533.85	W10x54	0.95
Ground						

Design of Column 4A										
	Axial Force				Moment	Load Combinations				Section
	Floor		Total			1.4D		1.2D + 1.6L		
	DL_1	DL_2 + RLL	DL_1	DL_2 + RLL	DL_1	P_u	M_u	P_u	M_u	
25	183.3	292.22	274.95	438.33	42.56761	384.93	59.59466	526.00	51.08114	W12x72
24			274.95	438.33		384.93		526.00		W12x72
23	183.3	292.22	458.25	730.55	84.4013	641.55	118.1618	876.66	101.2816	W12x120
22			458.25	730.55		641.55		876.66		W12x120
21	183.3	292.22	641.55	1022.77	108.1113	898.17	151.3559	1227.32	129.7336	W12x152
20			641.55	1022.77		898.17		1227.32		W12x152
19	183.3	292.22	824.85	1314.99	138.8509	1154.79	194.3913	1577.99	166.6211	W12x210
18			824.85	1314.99		1154.79		1577.99		W12x210
17	183.3	292.22	1008.15	1607.21	173.1454	1411.41	242.4036	1928.65	207.7745	W12x252
16			1008.15	1607.21		1411.41		1928.65		W12x252
15	183.3	292.22	1191.45	1899.43	219.5685	1668.03	307.3959	2279.32	263.4822	W12x305
14			1191.45	1899.43		1668.03		2279.32		W12x305
13	183.3	292.22	1374.75	2191.65	204.3103	1924.65	286.0345	2629.98	245.1724	W12x305
12			1374.75	2191.65		1924.65		2629.98		W12x305
11	183.3	292.22	1558.05	2483.87	372.5659	2181.27	521.5923	2980.64	447.0791	W14x370
10			1558.05	2483.87		2181.27		2980.64		W14x370
9	183.3	292.22	1741.35	2776.09	405.1869	2437.89	567.2617	3331.31	486.2243	W14x496
8			1741.35	2776.09		2437.89		3331.31		W14x496
7	183.3	292.22	1924.65	3068.31	494.4654	2694.51	692.2515	3681.97	593.3585	W14x455
6			1924.65	3068.31		2694.51		3681.97		W14x455
5	183.3	292.22	2107.95	3360.53	557.9905	2951.13	781.1866	4032.64	669.5885	W14x550
4			2107.95	3360.53		2951.13		4032.64		W14x550
3	183.3	292.22	2291.25	3652.75	631.8169	3207.75	884.5436	4383.30	758.1803	W14x550
2			2291.25	3652.75		3207.75		4383.30		W14x550
Ground	183.3	292.22	2474.55	3944.97	42.568					

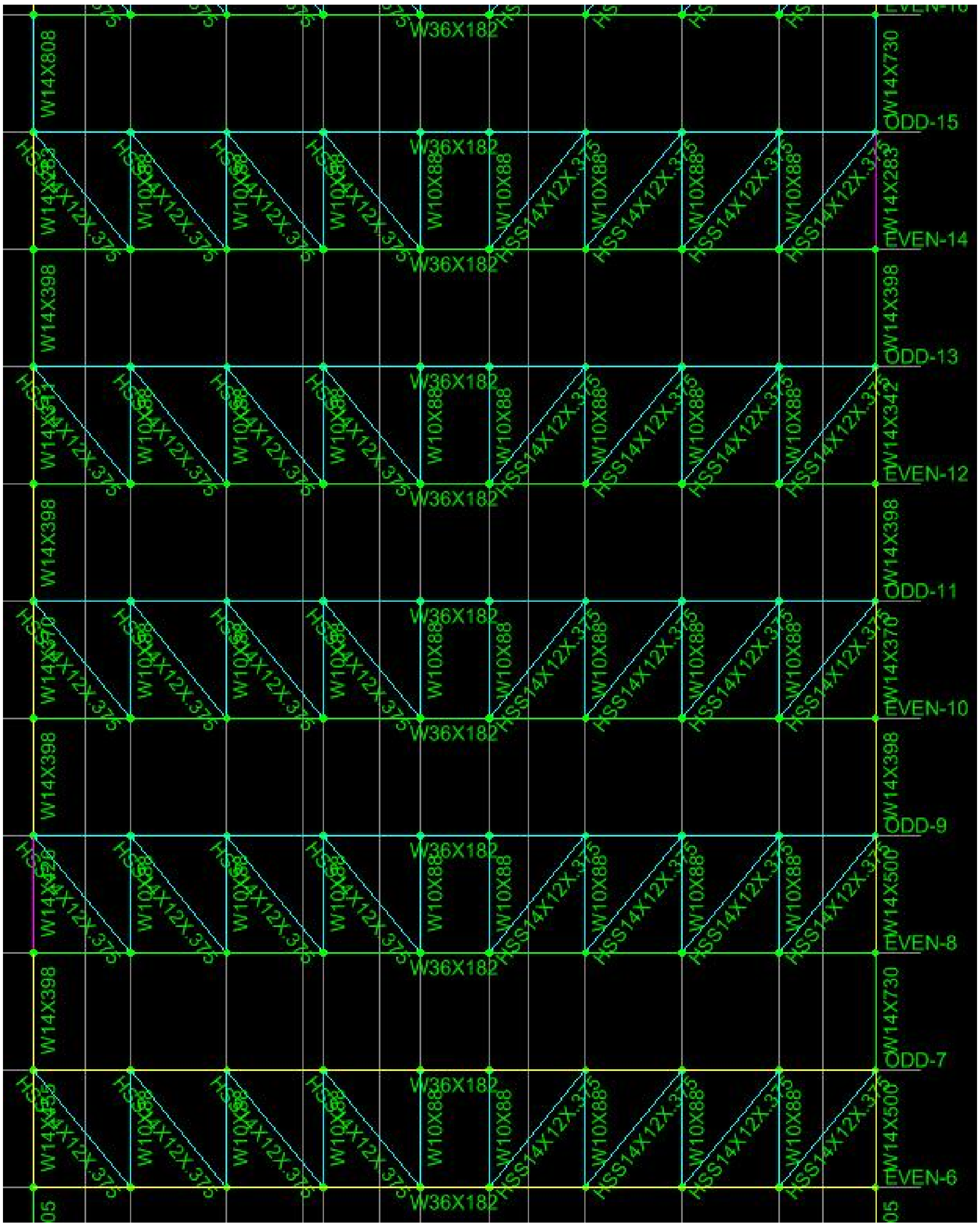
Appendix D

ETABS Output Data

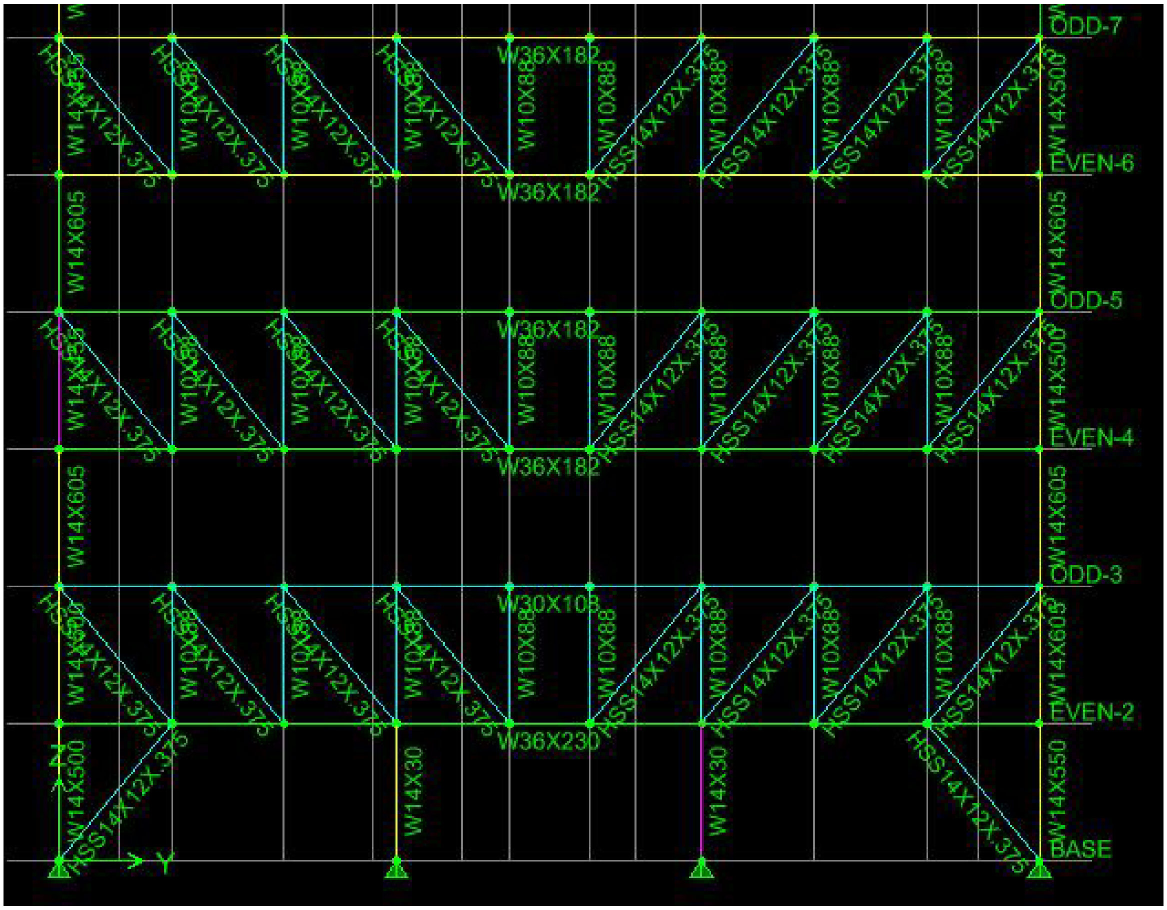
Sample Elevation: Truss 2 (16th Level to Roof)



Sample Elevation: Truss 2 (6th Level to 15th Level)



Sample Elevation for Truss 2 (Base to 7th Floor):



Elements for Levels 17 to 25

Story	ElementType	Material	TotalWeight	FloorArea	NumPieces
ROOF-25	Column	STEEL	46.788	1578105	74
ROOF-25	Beam	STEEL	119.066	1578105	55
ROOF-25	Brace	STEEL	21.839	1578105	28
ROOF-25	Floor	CONC	1643.755	1578105	
EVEN-24	Column	STEEL	81.172	1578105	56
EVEN-24	Beam	STEEL	113.451	1578105	54
EVEN-24	Brace	STEEL	12.48	1578105	16
EVEN-24	Floor	CONC	1643.755	1578105	
ODD-23	Column	STEEL	37.435	1578105	58
ODD-23	Beam	STEEL	126.739	1578105	55
ODD-23	Brace	STEEL	18.72	1578105	24
ODD-23	Floor	CONC	1643.755	1578105	
EVEN-22	Column	STEEL	82.229	1578105	56
EVEN-22	Beam	STEEL	111.908	1578105	54
EVEN-22	Brace	STEEL	12.48	1578105	16
EVEN-22	Floor	CONC	1643.755	1578105	
ODD-21	Column	STEEL	44.268	1578105	58
ODD-21	Beam	STEEL	124.541	1578105	55
ODD-21	Brace	STEEL	18.72	1578105	24
ODD-21	Floor	CONC	1643.755	1578105	
EVEN-20	Column	STEEL	85.222	1578105	56
EVEN-20	Beam	STEEL	112.692	1578105	54
EVEN-20	Brace	STEEL	12.48	1578105	16
EVEN-20	Floor	CONC	1643.755	1578105	
ODD-19	Column	STEEL	55.513	1578105	58
ODD-19	Beam	STEEL	122.823	1578105	55
ODD-19	Brace	STEEL	18.72	1578105	24
ODD-19	Floor	CONC	1643.755	1578105	
EVEN-18	Column	STEEL	91.34	1578105	56
EVEN-18	Beam	STEEL	110.266	1578105	54
EVEN-18	Brace	STEEL	12.48	1578105	16
EVEN-18	Floor	CONC	1643.755	1578105	
ODD-17	Column	STEEL	57.543	1578105	58
ODD-17	Beam	STEEL	125.166	1578105	55
ODD-17	Brace	STEEL	18.72	1578105	24
ODD-17	Floor	CONC	1643.755	1578105	

Elements for Levels 8 to 16

Story	ElementType	Material	TotalWeight	FloorArea	NumPieces
EVEN-16	Column	STEEL	96.389	1578105	56
EVEN-16	Beam	STEEL	110.346	1578105	54
EVEN-16	Brace	STEEL	12.48	1578105	16
EVEN-16	Floor	CONC	1643.755	1578105	
ODD-15	Column	STEEL	64.244	1578105	58
ODD-15	Beam	STEEL	124.805	1578105	55
ODD-15	Brace	STEEL	18.72	1578105	24
ODD-15	Floor	CONC	1643.755	1578105	
EVEN-14	Column	STEEL	91.765	1578105	56
EVEN-14	Beam	STEEL	110.117	1578105	54
EVEN-14	Brace	STEEL	12.48	1578105	16
EVEN-14	Floor	CONC	1643.755	1578105	
ODD-13	Column	STEEL	71.819	1578105	58
ODD-13	Beam	STEEL	124.706	1578105	55
ODD-13	Brace	STEEL	18.72	1578105	24
ODD-13	Floor	CONC	1643.755	1578105	
EVEN-12	Column	STEEL	95.659	1578105	56
EVEN-12	Beam	STEEL	109.454	1578105	54
EVEN-12	Brace	STEEL	12.48	1578105	16
EVEN-12	Floor	CONC	1643.755	1578105	
ODD-11	Column	STEEL	78.956	1578105	58
ODD-11	Beam	STEEL	124.382	1578105	55
ODD-11	Brace	STEEL	18.72	1578105	24
ODD-11	Floor	CONC	1643.755	1578105	
EVEN-10	Column	STEEL	97.514	1578105	56
EVEN-10	Beam	STEEL	109.206	1578105	54
EVEN-10	Brace	STEEL	12.48	1578105	16
EVEN-10	Floor	CONC	1643.755	1578105	
ODD-9	Column	STEEL	92.295	1578105	58
ODD-9	Beam	STEEL	124.236	1578105	55
ODD-9	Brace	STEEL	18.72	1578105	24
ODD-9	Floor	CONC	1643.755	1578105	
EVEN-8	Column	STEEL	96.865	1578105	55
EVEN-8	Beam	STEEL	113.072	1578105	59
EVEN-8	Brace	STEEL	25.967	1578105	21
EVEN-8	Floor	CONC	1643.755	1578105	

Elements for Base to Level 7

Story	ElementType	Material	TotalWeight	FloorArea	NumPieces
ODD-7	Column	STEEL	83.85	1651465	49
ODD-7	Beam	STEEL	116.972	1651465	40
ODD-7	Brace	STEEL	18.72	1651465	24
ODD-7	Floor	CONC	1643.755	1578105	
EVEN-6	Column	STEEL	88.603	1651465	47
EVEN-6	Beam	STEEL	101.824	1651465	39
EVEN-6	Brace	STEEL	21.792	1651465	17
EVEN-6	Floor	CONC	1643.755	1578105	
ODD-5	Column	STEEL	82.08	1651465	49
ODD-5	Beam	STEEL	101.651	1651465	40
ODD-5	Brace	STEEL	19.862	1651465	27
ODD-5	Floor	CONC	1643.755	1578105	
EVEN-4	Column	STEEL	87.851	1651465	47
EVEN-4	Beam	STEEL	91.63	1651465	39
EVEN-4	Brace	STEEL	17.667	1651465	19
EVEN-4	Floor	CONC	1643.755	1578105	
ODD-3	Column	STEEL	99.77	1651465	65
ODD-3	Beam	STEEL	98.218	1651465	39
ODD-3	Brace	STEEL	29.869	1651465	30
ODD-3	Floor	CONC	1643.755	1578105	
EVEN-2	Column	STEEL	79.914	1651465	35
EVEN-2	Beam	STEEL	110.359	1651465	39
EVEN-2	Brace	STEEL	8.83	1651465	13
EVEN-2	Floor	CONC	1643.755	1578105	
BASE	Floor	CONC	423.597	406679.6	

System Summary

Story	ElementType	Material	TotalWeight	FloorArea	NumPieces
SUM	Column	STEEL	1889.085	38721365	1333
SUM	Beam	STEEL	2737.631	38721365	1222
SUM	Brace	STEEL	414.139	38721365	499
SUM	Floor	CONC	40332.17	38721365	
TOTAL	All	All	53572.24	38721365	3054

Material List by Section

Section	ElementType	NumPieces	TotalLength	TotalWeight
W10X45	Column	22	2706	10.185
W10X49	Column	11	1353	5.514
W10X88	Column	107	13161	96.466
W12X87	Column	29	3567	25.842
W12X106	Column	22	2706	23.893
W12X120	Column	24	2952	29.49
W14X22	Column	233	28659	52.637
W14X22	Beam	9	533.25	0.906
W14X22	Brace	4	685.211	1.259
W14X26	Column	21	2583	5.621
W14X30	Column	129	15867	39.74
W14X30	Brace	6	946.477	2.371
W14X34	Column	12	1476	4.177
W14X34	Beam	5	1243.75	3.43
W14X38	Column	10	1230	3.899
W14X38	Beam	2	497.5	1.545
W14X43	Column	63	7749	27.631
W14X43	Beam	14	3627.75	12.468
W14X48	Column	15	1845	7.362
W14X48	Beam	3	862.75	3.354
W14X53	Column	6	738	3.258
W14X61	Column	63	7749	39.254
W14X68	Column	17	2091	11.835
W14X74	Column	12	1476	9.106
W14X74	Brace	1	184.859	1.14
W14X82	Column	11	1353	9.19
W14X90	Column	16	1968	14.759
W14X99	Column	23	2829	23.298
W14X109	Column	22	2706	24.506
W14X120	Column	19	2337	23.346
W14X132	Column	27	3321	36.466
W14X145	Column	28	3444	41.618
W14X159	Column	47	5781	76.402
W14X176	Column	48	5904	86.549
W14X176	Brace	1	184.859	2.71
W14X193	Column	29	3567	57.337
W14X211	Column	10	1230	21.582
W14X233	Column	50	6150	119.221
W14X233	Brace	2	369.719	7.167

Material List by Section (continued)

Section	ElementType	NumPieces	TotalLength	TotalWeight
W14X257	Column	14	1722	36.842
W14X283	Column	7	861	20.297
W14X311	Column	5	615	15.908
W14X342	Column	8	984	28.126
W14X370	Column	12	1476	45.53
W14X398	Column	14	1722	57.017
W14X426	Column	61	7503	265.419
W14X426	Brace	1	184.859	6.539
W14X455	Column	17	2091	79.295
W14X500	Column	15	1845	76.754
W14X500	Brace	1	184.859	7.69
W14X550	Column	11	1353	62.03
W14X605	Column	39	4797	241.644
W14X605	Brace	1	184.859	9.312
W14X665	Column	1	123	6.823
W14X730	Column	2	246	14.968
W14X808	Column	1	123	8.25
W18X50	Beam	703	141914.8	551.644
W18X60	Beam	46	14340.31	68.791
W21X55	Beam	47	12404.92	54.025
W21X62	Beam	18	6193.104	30.948
W21X68	Beam	21	6165.068	33.115
W24X62	Beam	5	1206	5.857
W24X76	Beam	86	27287.64	165.65
W24X84	Beam	26	7296.074	48.763
W27X94	Beam	34	11071.41	83.638
W27X102	Beam	21	8463.482	70.153
W27X114	Beam	1	882	8.197
W30X108	Beam	4	1766.964	15.697
W30X116	Beam	2	1221.482	11.774
W30X124	Beam	10	3049.287	30.22
W33X130	Beam	5	2648.964	28.168
W33X141	Beam	4	1633.482	18.822
W36X150	Beam	2	643.232	7.846
W36X170	Beam	14	11672	162.277
W36X182	Beam	140	88700.09	1320.342
HSS14X12X.375	Brace	482	76788.76	375.95
WALL1	Wall			8199.209
PLANK1	Floor			37609.53

Story Drift Summary for Controlling Lateral Load Case, Wind in N-S Direction

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY
EVEN-24	Max Drift X	WINDY	60	1379	303.75	2829	0	
EVEN-24	Max Drift Y	WINDY	60	1379	303.75	2829		0
ODD-23	Max Drift X	WINDY	10	1723	0	2706	0.000041	
ODD-23	Max Drift Y	WINDY	1123	2046.088	466.452	2706		0.000051
EVEN-22	Max Drift X	WINDY	10	1723	0	2583	0.000041	
EVEN-22	Max Drift Y	WINDY	1123	2046.088	466.452	2583		0.000051
ODD-21	Max Drift X	WINDY	10	1723	0	2460	0.000029	
ODD-21	Max Drift Y	WINDY	1123	2046.088	466.452	2460		0.000046
EVEN-20	Max Drift X	WINDY	10	1723	0	2337	0.000029	
EVEN-20	Max Drift Y	WINDY	1123	2046.088	466.452	2337		0.000046
ODD-19	Max Drift X	WINDY	10	1723	0	2214	0.000024	
ODD-19	Max Drift Y	WINDY	1123	2046.088	466.452	2214		0.000042
EVEN-18	Max Drift X	WINDY	10	1723	0	2091	0.000024	
EVEN-18	Max Drift Y	WINDY	1123	2046.088	466.452	2091		0.000042
ODD-17	Max Drift X	WINDY	10	1723	0	1968	0.000021	
ODD-17	Max Drift Y	WINDY	1123	2046.088	466.452	1968		0.000039
EVEN-16	Max Drift X	WINDY	10	1723	0	1845	0.000021	
EVEN-16	Max Drift Y	WINDY	1123	2046.088	466.452	1845		0.000039
ODD-15	Max Drift X	WINDY	10	1723	0	1722	0.000019	
ODD-15	Max Drift Y	WINDY	1123	2046.088	466.452	1722		0.000036
EVEN-14	Max Drift X	WINDY	10	1723	0	1599	0.000019	
EVEN-14	Max Drift Y	WINDY	1123	2046.088	466.452	1599		0.000036
ODD-13	Max Drift X	WINDY	10	1723	0	1476	0.000017	
ODD-13	Max Drift Y	WINDY	1123	2046.088	466.452	1476		0.000032
EVEN-12	Max Drift X	WINDY	10	1723	0	1353	0.000017	
EVEN-12	Max Drift Y	WINDY	1123	2046.088	466.452	1353		0.000032
ODD-11	Max Drift X	WINDY	10	1723	0	1230	0.000012	
ODD-11	Max Drift Y	WINDY	1123	2046.088	466.452	1230		0.000031
EVEN-10	Max Drift X	WINDY	10	1723	0	1107	0.000012	
EVEN-10	Max Drift Y	WINDY	1123	2046.088	466.452	1107		0.000031
ODD-9	Max Drift X	WINDY	1200	1723	882	984	0.000035	
ODD-9	Max Drift Y	WINDY	67	-64.25	362	984		0.000066
EVEN-8	Max Drift X	WINDY	93	1955.078	439.328	861	0.001673	
EVEN-8	Max Drift Y	WINDY	93	1955.078	439.328	861		0.009267
ODD-7	Max Drift X	WINDY	2	194	0	738	0.000039	
ODD-7	Max Drift Y	WINDY	67	-64.25	362	738		0.000065
EVEN-6	Max Drift X	WINDY	49	1379	283	615	0.000029	
EVEN-6	Max Drift Y	WINDY	8	1585	0	615		0.000042

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY
ODD-5	Max Drift X	WINDY	39	1379	202.5	492	0.000027	
ODD-5	Max Drift Y	WINDY	49	1379	283	492		0.00004
EVEN-4	Max Drift X	WINDY	1174	351	780.75	369	0.000029	
EVEN-4	Max Drift Y	WINDY	50	1585	283	369		0.000063
ODD-3	Max Drift X	WINDY	1192	194	882	246	0.000024	
ODD-3	Max Drift Y	WINDY	67	-64.25	362	246		0.000068
EVEN-2	Max Drift X	WINDY	2	194	0	123	0.000061	
EVEN-2	Max Drift Y	WINDY	1136	10	519	123		0.000159

Appendix E

Fire Protection Calculations

Thickness Check for Sample Truss Column: W12×72

Spray-Applied Fire Resistant Materials						
Section:		R	W/D	(W/D from LRFD Table 1-36: Case B)		
W12×72		3	1.02	where R = [C1(W/D)+C2]*h		
R = Fire Resistance Rating (hrs)				h = thickness of application		
	Grace MK6	Isolatek 800	Isolatek 280	Isolatek 280	Isolatek D-C/F	Isolatek D-C/F
C1	1.05	0.86	1.25	1.25	1.01	0.95
C2	0.61	0.97	0.53	0.25	0.66	0.45
h	1.785	1.624	1.662	1.967	1.775	2.114
Min./Max. W/D Requirements	OK	OK	OK	OK	OK	N/A
	OK	OK	OK	OK	OK	OK
Rank	4	1	2	5	3	6

Minimum thickness required for W12×72 Section: 1.662" ≈ 1.75" for R = 3 hr fire rating

Gypsum Wallboard Type X Board			
R = 130*[h(W'/D)/2]^0.75			
Section	W12x72		
Section Properties	bf =	12	in
	d =	12.3	in
	W =	72	plf
Gypsum Thickness to Check	h =	1.5	in
Weight of Column and Wallboard	W' =	97.3125	plf
Inside Perimeter of Wallboard	D =	48.6	in
Assembly Fire Rating	**R =**	**176.36**	**min**

1 ½ inch wallboard does not provide enough for R = 3 hrs = 180 min for W12×72 Section

Try 2 in thickness:

Gypsum Wallboard Type X Board			
R = 130*[h(W'/D)/2]^0.75			
Section	W12x72		
	bf =	12	in
Section Properties	d =	12.3	in
	W =	72	plf
Gypsum Thickness to Check	h =	2	in
Weight of Column and Wallboard	W' =	105.75	plf
Inside Perimeter of Wallboard	D =	48.6	in
Assembly Fire Rating	R =	232.90	min

2 inch thickness of wallboard gives R =3 (nearly 4) for W12×72 Section

Concrete Protection for Columns: Full Encasement			
Rough Concrete Encasement Dimensions	Width	13.5	in
	Depth	13.5	in
Column Section Properties	Section	W12x72	
	bf =	12	in
	d =	12.3	in
	As =	21.1	in2
	W =	72	plf
	tw =	0.43	in
	tf =	0.67	in
	T =	9.125	in
Interior Perimeter of Encasement (from Table 1-36)	D =	70.3	in
Average thickness of Conc. Encasement	h =	1.35	in
Average Interior Dimension of One Side of Square Conc. Box Protection	L =	12.15	in
Conc Density	Rhoc =	145	pcf
Ambient Spec Heat of Conc	cc =	0.2	Btu/lbF
Moisture Content of Concrete by Volume	m =	4	% volume
Thermal Capacity of Steel Column	H =	25.58835	
Fire endurance rating at zero moisture	Ro =	167.422	min
		2.790367	hrs
Fire endurance rating at actual moisture condition	R =	187.5127	
		3.125211	hrs

13.5" square concrete encasement around W12×72 Section provides R = 3 hr fire rating
(Average thickness of 1.35" around steel section)

Appendix F

Cost Analysis Data and Calculations

Existing Design: Building Statistics

- Building Name: River Tower at Christina Landing
- Location and Site: 115 Christina Landing, Wilmington, DE, 19801
- Building Occupant Name: The Buccini Pollin Group
- Occupancy or Function Type:
- **Primary Occupancy:** Condominium Building
- **Accessory Occupancy:** Enclosed Parking Garage interfaced in lower eight stories

- Size (Total Sq. Ft.): Approximately 433,200 Sq. Ft.
- Number of Stories Above Grade: 25 stories
- **Dates of Construction:**
  - **Planned:** May 1, 2005 to November, 2006 (18 months)
  - **Actual:** September 1, 2005 to April 1, 2007 (18 months)

- Costs:

Original estimates:

- **Overall Project:** $55.5 million
- **Building Cost:** $46 million
- **Soft Costs:** $4 million (CM fees, permits, site services, laborers) and an additional $5 million of environmental remediation (contamination due to buried oil tanks from old tannery/oil storage yard)

- Project Delivery Method: Originally: Design-Bid-Build
After redesign: Fast Track/Design-Build

Cost Analysis Calculations

Existing Concrete System Estimates: Using R.S. Means Cost Data: Unit Costs								
Concrete Column Estimate: Using 36" square columns, max reinf.							Column CY =	129.08
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
C14A	17.80	11.22	CY	$ 400.00	$ 365.00	$ 48.50	$ 813.50	$ 1,075.00
	7.25	1415.16		$ 51,633.54	$ 47,115.60	$ 6,260.57	$ 105,009.71	$ 138,765.14
Shear Wall estimates (taken from Grade Walls, 15" thick, interpolated between 8' and 12' high)							Shear Wall CY =	442.100823
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
C14B	65.75	3.20	CY	$ 98.25	$ 103.00	$ 13.78	$ 215.03	$ 283.50
	1415.16	442.10		$ 43,436.41	$ 45,536.38	$ 6,092.15	$ 95,064.94	$ 125,335.58
Cast In Place Concrete 5000 psi Ready-Mix							Total Concrete CY =	1416.657735
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
			CY	$ 71.00			$ 71.00	$ 78.00
				$ 100,582.70			$ 100,582.70	$ 110,499.30
Prestressed Concrete (Large Job)							Total Concrete CY =	6686.364
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
C17B	10	8.2	CY	$ 530.00	$ 350.00	$ 30.00	$ 910.00	$ 1,150.00
	668.6641975	54830.46	0	$ 3,543,920.25	$ 2,340,324.69	$ 200,599.26	$ 6,084,844.20	$ 7,689,638.27
Totals								
	2091.08	56687.73		$ 3,739,572.89	$ 2,432,976.68	$ 212,951.98	$ 6,385,501.55	$ 8,064,238.29
							Add 5% waste:	$ 8,467,450.21

Proposed Steel System Estimates: Using R.S. Means Cost Data: Unit and Assembly Costs								
Steel projects: Apartments over 15 stories						Member	Weight (k) =	Tonnage =
						Column	1889.085	944.54
						Beam	2737.631	1368.82
						Brace	414.139	207.07
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
E6	13.9	9.209	TON	$ 1,900.00	$ 390.00	$ 129.00	$ 2,419.00	$ 2,900.00
Column	67.95	9.21	TON	$ 1,794,630.75	$ 368,371.58	$ 121,845.98	$2,284,848.31	$ 2,739,173.25
Beam	98.48	8698.29	TON	$ 2,600,749.45	$ 533,838.05	$ 176,577.20	$3,311,164.69	$ 3,969,564.95
Brace	14.90	12605.42	TON	$ 393,432.05	$ 80,757.11	$ 26,711.97	$ 500,901.12	$ 600,501.55
TOTALS:	181.33	1906.90		$ 4,788,812.25	$ 982,966.73	$ 325,135.15	$6,096,914.12	$ 7,309,239.75

<u>Accounting for Differential in Sizing:</u>

Approximately 200 plf differential in each exterior truss column:

(10 truss columns per floor)*(200 plf)*(10.25 ft height)*(24 floors) = 492 kip reduction in column weight

Approximately 150 plf differential in truss chords on each floor:

(5 chords per floor)*(150 plf)*(73.5 ft width)*(24 floors) = 1323 kip reduction in beam weight

Approximate adjustment on ETABS output = $2,630,000 (see next page)

Adjusted Steel System Estimates to Account for Sizing Discrepancy								
Steel projects: Apartments over 15 stories						Member	Weight (k) =	Tonnage =
						Column	1397.09	698.54
						Beam	1414.63	707.32
						Brace	414.14	207.07
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
E6	13.9	9.209	TON	$ 1,900.00	$ 390.00	$ 129.00	$ 2,419.00	$ 2,900.00
Column	50.25	6432.88	TON	$ 1,327,230.75	$ 272,431.58	$ 90,111.98	$1,689,774.31	$ 2,025,773.25
Beam	50.89	6513.66	TON	$ 1,343,898.50	$ 275,852.85	$ 91,243.64	$1,710,994.99	$ 2,051,213.50
Brace	14.90	1906.90	TON	$ 393,432.05	$ 80,757.11	$ 26,711.97	$ 500,901.12	$ 600,501.55
TOTALS:	116.04	618.89		$ 3,064,561.30	$ 629,041.53	$ 208,067.58	$3,901,670.41	$ 4,677,488.30

Assembly Cost for Precast Plank Flooring						
Total Area	Span	Total Depth	Superimposed Load	Materials	Installation	Total/SF
270,809 SF	30 ft	8 in	82 psf	$6.75	$3.75	$10.50
			Costs =	$1,827,960.75	$1,015,553.75	$2,843,494.50

Total Structure =	**$7,520,952.80**
Add 5% waste	$7,897,000.44
Add 10% Connections and Fabrication Costs	$8,649,095.72

Staggered Truss System is **$181,645.51 more expensive**

Appendix G

List of Resources

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