Pearl Condominiums

9th & Arch Street
Philadelphia, PA

Technical Assignment #1

Structural Concepts / Structural Existing Conditions Report

October 5, 2007

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http://www.engr.psu.edu/ae/thesis/portfolios/2008/jgl138/

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Executive Summary

Technical Assignment #1 is an existing conditions report describing the as-built structural system for Pearl Condominiums, Philadelphia, PA. Codes used in the original design are listed in this report. The structural system is described to the reader through the use of written description, plans of the building and diagrams. The design of gravity and lateral loads are described based on assumed loads based on the codes. The spot checks are performed on typical members including steel roof joist, wide flange column, metal stud from bearing wall, and masonry shear wall. The spot checks will compare the actual as-built members and the thesis designed members. The Appendix contains calculations to obtain loads and spot check members.

Building Description:

Pearl Condominiums is a mixed use development housing including 10 retail units on the ground floor and 90 condominium units on the upper floors. The gross floor area is 111,570 square feet and has 6 stories above grade.

The start of construction was March 30, 2006 and the finish date is October 2007. The zoning is C-4 Commercial. Design considerations for the site included the site location existing above a SEPTA commuter rail tunnel.

Structural Overview

The structural materials used in the construction of the building are steel, concrete, and metal. The foundation of the building is comprised of concrete drilled piers with the use of concrete grade beams and slab on grade to distribute the forces to the caissons. The columns on the first floor that support the upper floors are steel wide flange shapes. Also on the ends of the north and south sides of the building there are steel HSS tubes used as columns that extend from the first floor to the roof. For the upper floors, the bearing walls are composed of metal studs. The floor system is composed of precast planks with a covering of concrete. The lateral forces are resisted by the metal stud shear walls and the concrete masonry shear walls around the elevator and stairway towers. The roof system main elements are K- series steel joists.

Codes & Requirements

The codes used for the Original Design are as followed:

- Internal National Building Code 2003
- American Society of Civil Engineers 07-02 : Minimum Design Loads for building and Other Structures
- American Concrete Institute (ACI) 318-02: Building Code
- ACI 315 : Detailing Manual
- ACI 301: Specifications For Structural Concrete For Buildings
- Manual of Standard Practice: Concrete Reinforcing Steel Institute
- American Society of Testing and Materials (ASTM)
- Precast Concrete Institute (PCI)
- American Institute of Steel Construction (AISC)
- American Welding Society (AWI): Structural Welding Codes
- Steel Deck Institute: Design Manual
- Steel Joist Institute
- ACI 530: Building Code Requirement For Masonry Structures
- Brick Institute of America
- National Concrete Masonry Association
- Philadelphia Building Code
- SEPTA Recommendations For Building Over and Adjacent to the Tunnel Right of Way on Redevelopment Authority Lands

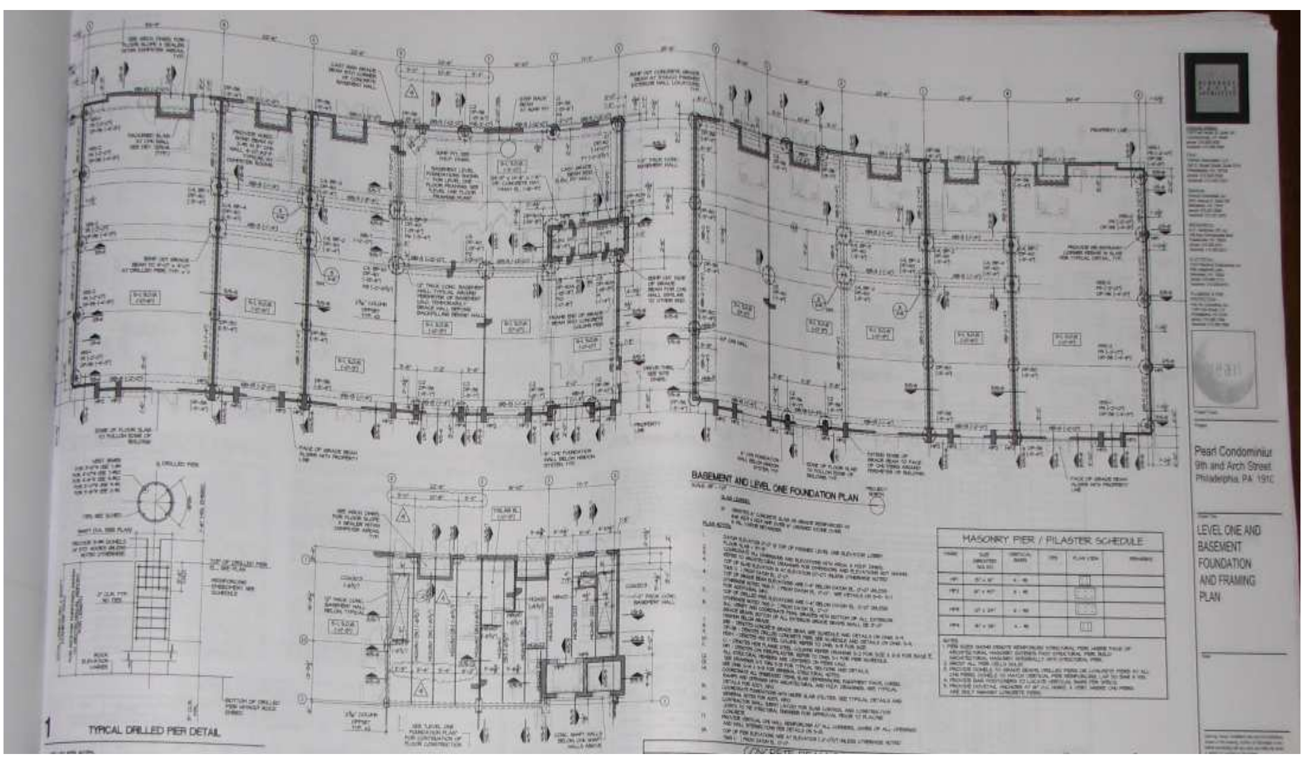
Structural Systems

Foundations:

The primary support for the foundation is the use of drilled piers. The drilled pier option was performed, so the loads from the building would be transferred from the pier to the soil below the SEPTA commuter train tunnel. If a shallow foundation system was chosen, special precautions to not disturb the area around the tunnel would have been needed to be performed. The drilled piers range in size of diameter from 3’-0” to 3’-6” to 4’-0”. They also range in depth depending on the rock elevations in the area as described in the geotechnical report.

To help distribute the load to the drilled piers the use of grade beams was employed. They range in width from 12” to 40” and in depth from 18” to 30”. The slab on grade is 6” reinforced with 6x6 W2.9xW2.9 WWR over 6” crushed stone over 6 mil. Vapor retarder.

Application	Concrete Strength (f’c)
Drilled Piers	4,000 psi
Grade Beams	4,000 psi
Slab on Grade	4,000 psi



Basement and Level One Foundation Plan

Columns \ Load Bearing Walls:

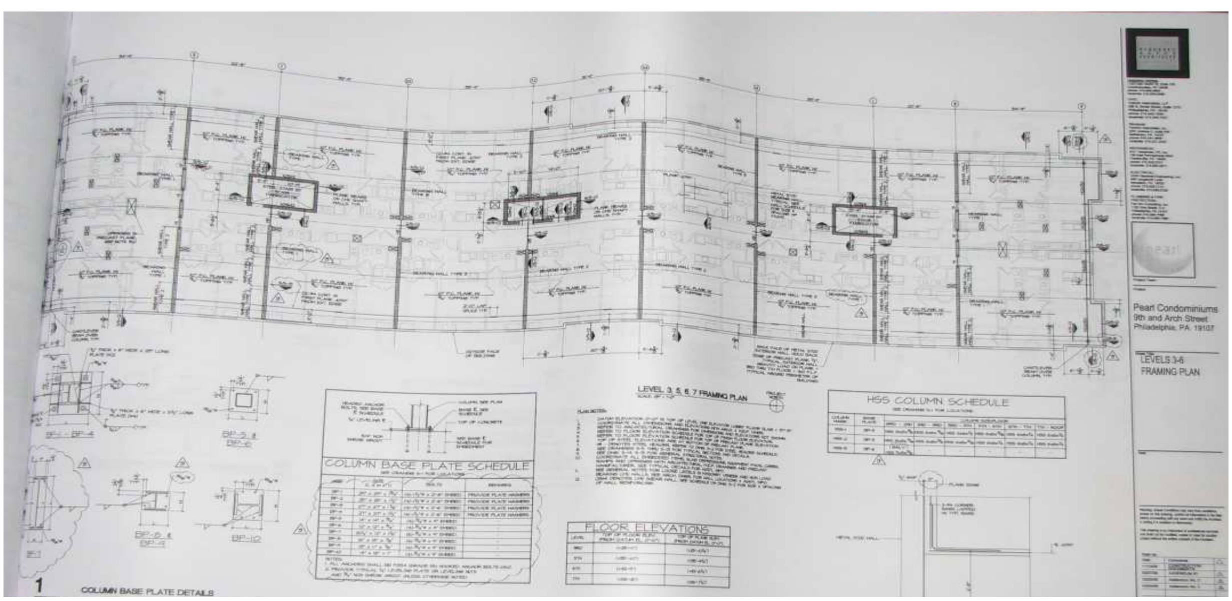
The columns in the building are HSS tube columns sizes of 6"x6" and 8"x8" with varying thickness. Wide flange shapes are also used in select spaces ranging from W10X39, W10X49, W12X53, W12X120 and W30X90. There are masonry and concrete piers present on the basement and level one. The load bearing walls are comprised of 8" metal stud at 16" and 12" on center.

Application	Strength (fy)
Wide Flange	50 ksi
HSS tubes	46 ksi
Metal Studs	33 ksi & 50 ksi

Floor System:

The floor system for level 2 thru 6 is comprised of a 10" Precast Concrete Plank with a ¾" concrete thick topping. The concrete strength of the precast plank is f'c equals 5,000 psi. The plank has a maximum span of 34'-9". The required reinforcing for the planks is at 4'-0" from the edge two number 4 bars continuous.

Level two acts as a transfer level, which requires the use of wide flange beam (W36) to be implemented around the area near the Open Entry Drive on the first floor. These beams help to distribute the load from this area and down into the foundation.

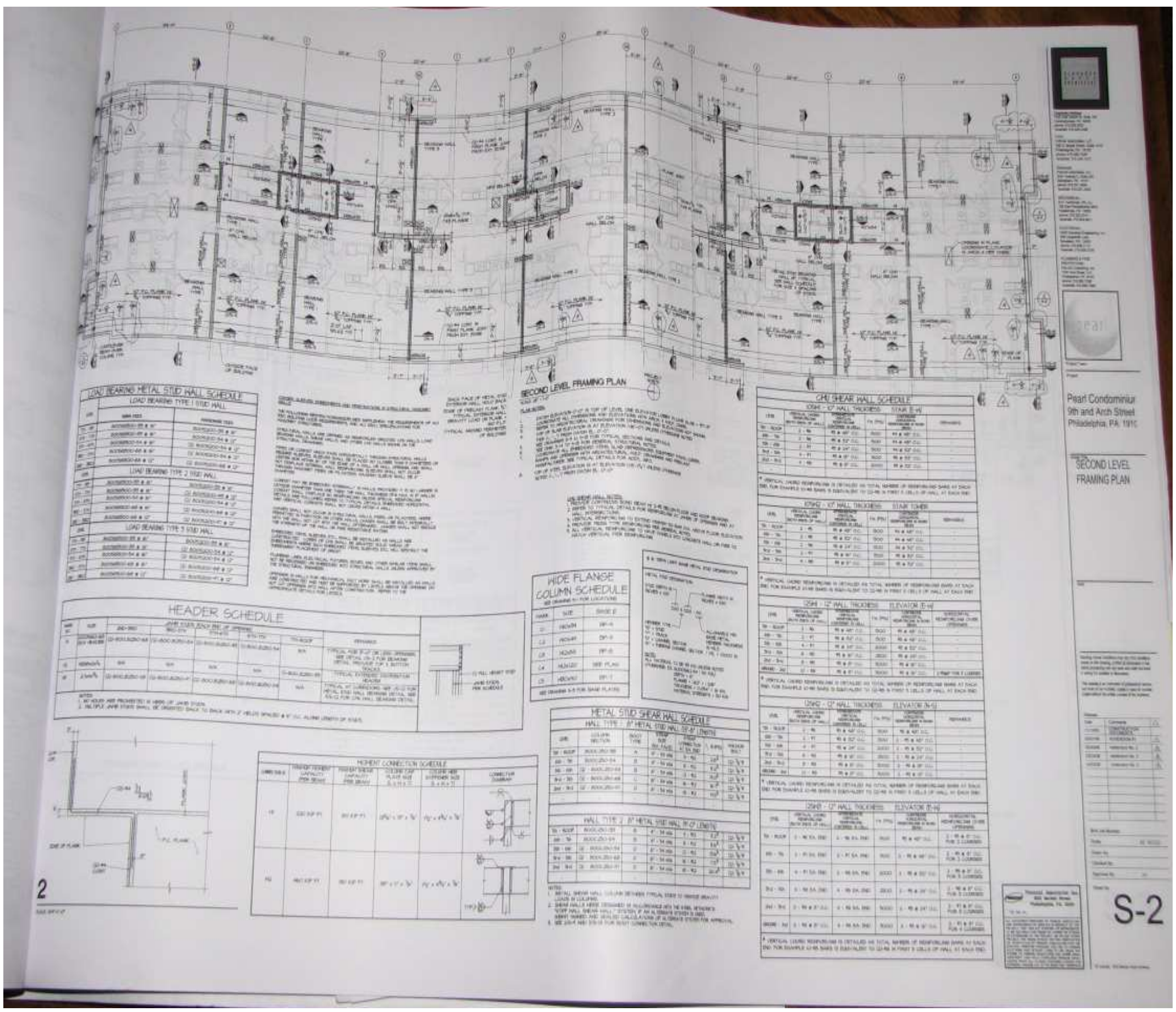


Level 3,4,5,6 Framing Plan

<u>Lateral Resisting System</u>:

The Lateral System in the building is comprised of two types: concrete masonry unit shear walls and metal stud shear wall. The concrete masonry unit shear walls are used around the elevator and stairway towers. These walls range from thickness of 10" in the stair areas and 12" in the elevators. The strength of the concrete masonry units (f'm) range from1500 psi to 2000psi and 3000psi depending on the area they are used in.

The metal stud shear walls are composed of 8" metal studs varying in thickness. The two heights of the studs are 13'-8" and 9'-0". Metal straps connected by #12 screws to the metal studs and 7/8" diameter anchor bolts connected through different boot types help to resist the lateral forces applied to the metal studs.

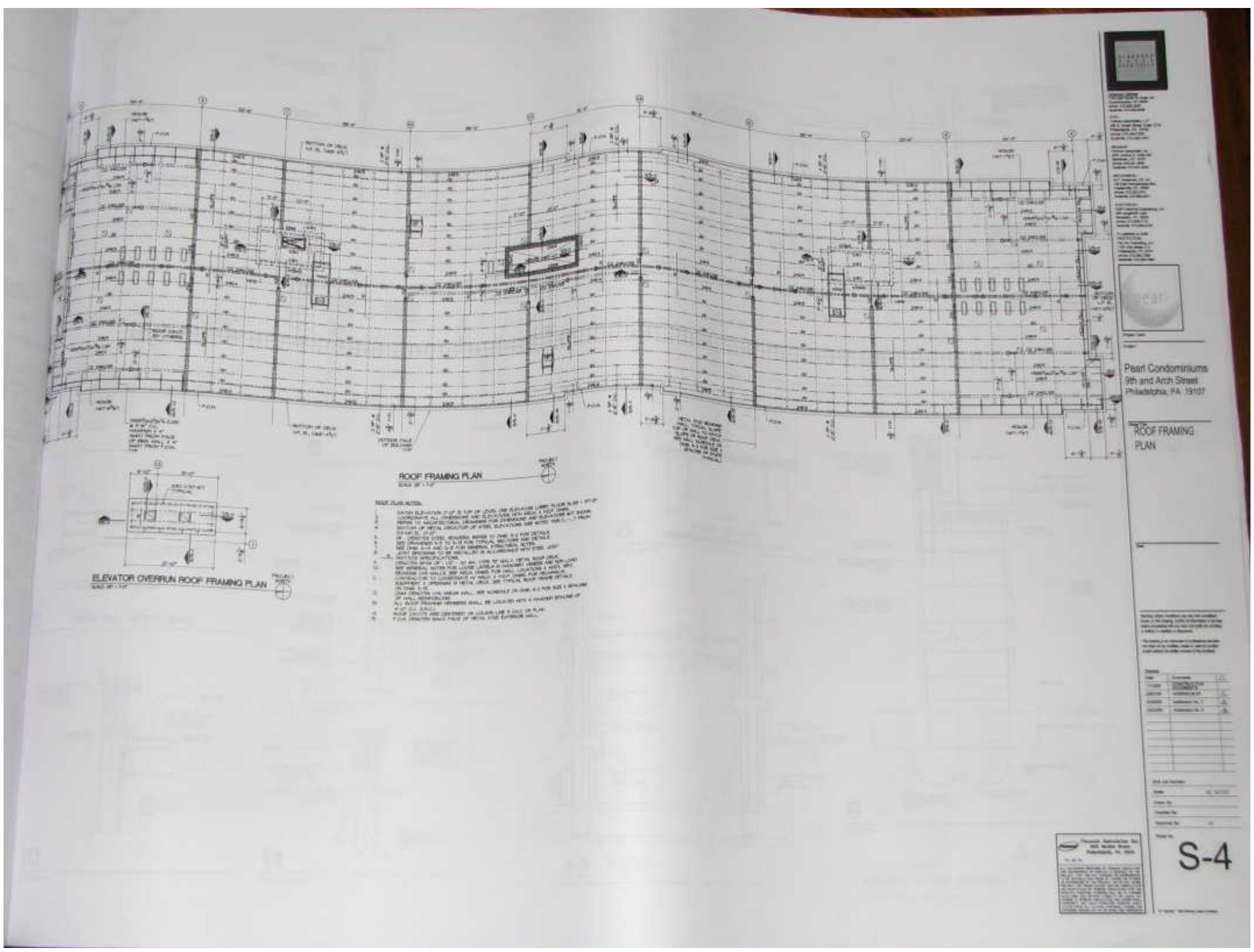


Second Level Framing Plan

Roof System:

The main structural element in the roof system is the use of 24” deep steel joists at 48” on center. On the two ends N-S the use of wide flange beams to transfer the load to HSS columns is implemented. The steel joists bear on the metal stud walls and the concrete masonry walls of the sixth floor. The roof assembly is composed of:

- Single-Ply Membrane
- 5/8” Protection Board
- R-30 Rigid Insulation
- 5/8” Gypsum Wall Board
- 1-1/2” Min Steel Deck
- Steel Roof Joists
- Steel Bridging
- 5/8” Gypsum Wall Board On Suspended Ceiling Panel



Roof Framing Plan

Gravity Loading

Floor Live Loads	
Occupancy or Use	**Uniform Live Load (psf)**
Condominium Units w\ Partitions	60
Retail Units (first floor)	100
Stairs	100
Corridor above first floor	80
Corridor at first floor	100
Roof	30

Floor Dead Loads	
Occupancy or Use	**Uniform Dead Load (psf)**
Concrete Precast Plank	66
Roof	30

Superimposed Floor Dead Loads	
Occupancy or Use	**Uniform Dead Load (psf)**
Roof	20
Condominium Units w\ Partitions	25
Corridor above first floor	25
Corridor at first floor	25
Retail Units	25

Snow Loading	
Item	**Value**
Ground Snow Load (Pg)	25 psf
Exposure Factor	B
Roof Exposure	Fully Exposed
Exposure Factor (Ce)	0.9
Thermal Factor (Ct)	1.0
Occupancy Category	II
Importance Factor (Is)	1.0
Flat-Roof Snow Load Pf = 0.7CeCtIsPg	16 psf

Wind Loading

The wind load calculation for this building is based on Method 2 Analytical Procedure of the ASCE 07-05 Chapter 6. The assumption is that the building acts as a rigid structure. These Calculations are for the N-S direction. For the E-W direction calculations and detailed calculations of N-S see the Appendix for added information.

Windward Calculations							
Level	**z**	**Kz**	**qz**	**qh**	**GCp**	**GCpi**	**P windward**
1	0	0.57	10.047	15.863	0.68	+/- 0.18	9.69
2	16	0.62	10.928	15.863	0.68	+/- 0.18	10.29
3	25.92	0.70	12.338	15.863	0.68	+/- 0.18	11.25
4	35.83	0.76	13.395	15.863	0.68	+/- 0.18	11.96
5	45.75	0.81	14.277	15.863	0.68	+/- 0.18	12.56
6	55.67	0.85	14.982	15.863	0.68	+/- 0.18	13.04
Roof	72.3	0.90	15.863	15.863	0.68	+/- 0.18	13.64

Leeward Calculations						
Level	**z**	**Kz**	**qh**	**GCp**	**GCpi**	**P leeward**
1	0	0.57	15.863	-0.17	+/- 0.18	-5.55
2	16	0.62	15.863	-0.17	+/- 0.18	-5.55
3	25.92	0.70	15.863	-0.17	+/- 0.18	-5.55
4	35.83	0.76	15.863	-0.17	+/- 0.18	-5.55
5	45.75	0.81	15.863	-0.17	+/- 0.18	-5.55
6	55.67	0.85	15.863	-0.17	+/- 0.18	-5.55
Roof	72.3	0.90	15.863	0.68	+/- 0.18	-5.55

Total	
Level	**P total (PSF)**
1	15.24
2	15.84
3	16.8
4	17.51
5	18.11
6	18.59
Roof	19.19

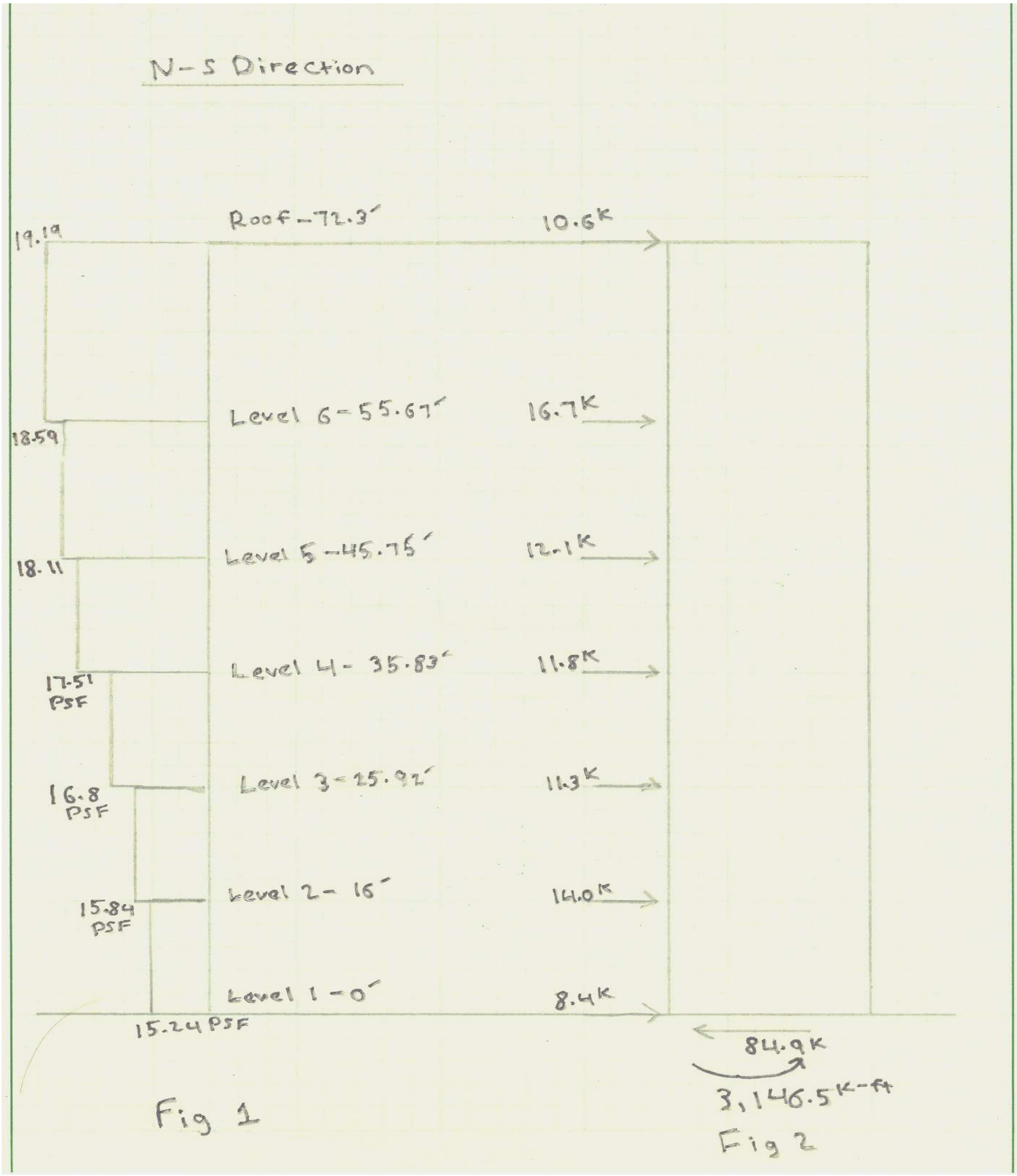


Fig. 1 – Wind Load Vertical Distribution
Fig 2 – Wind Load Base Shear and Overturning Moment

Seismic Loading

The base shear for seismic analysis was calculated using the total dead load of the building as the weight.

Seismic Design Category: B

Seismic Base Shear:

V = Cs*W

W = 11796 k

Cs = 0.0352

R = 5 ½ (Reinforced Masonry Shear Wall)

V = 415.2k

Vertical Distribution of Forces:

Fundamental Period:

Ta = 0.496 sec

K = 1.0

Level	**wx**	**hx**	**wx*hx^k**	**Cvx**	**Fx**	**Mx**
2	2171	16.000	34736	0.0749	31.1	497.60
3	2080	25.917	53907.36	0.1162	48.2	1249.20
4	2064	35.833	73959.312	0.1594	66.2	2372.14
5	2064	45.750	94428	0.2035	84.5	3865.88
6	2115	55.667	117735.705	0.2538	105.4	5867.30
Roof	1302	68.500	89187	0.1922	79.8	5466.30
				Overturning Moment =		19,318.42

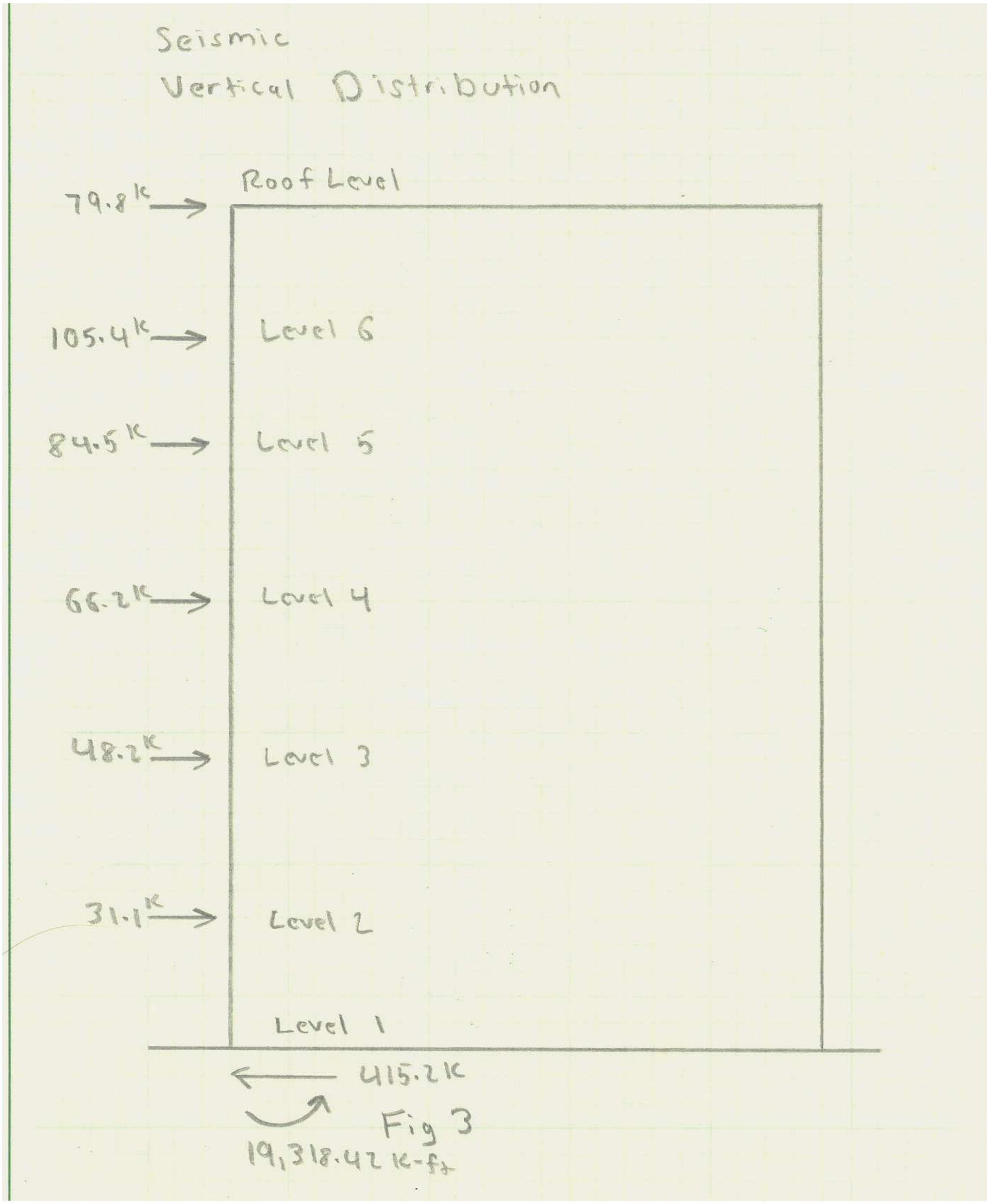


Fig 3– Seismic Load Base Shear and Overturning Moment

Spot Checks

During the process of spot checking a few of the structural elements, I realized that some of the structural components that were used in the building were both larger and smaller then the ones that I had calculated. I believe this to be the result of different loads calculated by myself and the engineer that designed the components. The steel joist that I found was smaller than the original and the metal studs required was larger than what was used. The column that I checked came out to be the same size that was placed in the building. It is possible that the rough estimate that I performed is in the proximity of the calculation that was done for the original design of the column. The shear wall design was based on a simplification of distributing the forces equally to the shear walls.

Conclusion

There are additional calculations that will have to be performed more in depth. The lateral force resisting system will have to be delved into more to understand how the forces are distributed between the two systems. Also the roof will have to be looked at for the possibility of uplift from the wind in both directions of the building. Finally the foundations components will have to calculate for the forces of bearing pressure, uplift, sliding and overturning.

Appendix

Wind Load Calculations:

Wind Force (Analytical Procedure

Main Wind Force Resisting System

$$p = qGC_p - q_i(GC_{pi})$$

ASCE 7-05 Table 6-3

Level	Z	K_z/K_h
1	0	0.57
2	16	0.62
3	25.92	0.70
4	35.83	0.76
5	45.75'	0.81
6	55.67'	0.85
Roof	72.3'	0.90

N-S Length of Bldg 66'-6"

E-W Length of Bldg 283'-8"

Design Wind Speed — 90 MPH

Wind Importance Factor — 1.0

Wind Exposure Category — B

Topographic Factor $K_{zt} = 1.0$

Wind Directionality Factor, $K_d = 0.85$

$T = 0.1 *$ # stories

$0.1(6) = 0.6$

$n_1 = 1/T = 1/0.6 = 1.67 > 1$ therefore Rigid

Gust Factor (G) 0.85 or $0.925\left(\frac{1+1.7 g_Q I_{\bar z} Q}{1+1.7 g_v I_{\bar z}}\right)$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(33/43.38\right)^{\wedge 1/6} = 0.287$$

$z_{min} = 30 \text{ ft}$

$c = 0.30$

$\bar{z} = 0.6(h) = 0.6(72.3') = 43.38'$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$$

	B	L
N-S	66'-6"	283'-8"
E-W	283'-8"	66'-6"

$h = 72.3'$

$\ell = 320$

$\bar{\epsilon} = 1/3.0$

$g_Q = 3.40$

$g_v = 3.40$

$$L_{\bar{z}} = 320\left(\frac{43.38}{33}\right)^{1/3.0} = 350.54$$

$$Q_{\perp_{N-S}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{66'-6" + 72.3'}{350.54}\right)^{0.63}}} = 0.86$$

$$Q_{\perp_{E-W}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{283'-8" + 72.3'}{350.54}\right)^{0.63}}} = 0.78$$

$$G_{\perp N\text{-}S} = 0.925\left(\frac{1+1.7(3.4)(0.287)(0.86)}{1+1.7(3.4)(0.287)}\right) = 0.844$$

$$G_{\perp E\text{-}W} = 0.925\left(\frac{1+1.7(3.4)(0.287)(0.78)}{1+1.7(3.4)(0.287)}\right) = 0.80$$

C_p -

	Windward	Leeward	Side Wall
N-S	0.80 (q_z)	-0.2 (q_h)	-0.7 (q_h)
E-W	0.80 (q_z)	-0.5 (q_h)	-0.7 (q_h)

	L/B
N-S	4.3
E-W	0.23

$GC_{pi} = \pm 0.18$

$$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$$
$$q_z = 17.6256\, K_z$$

N-S Windward

Level	q_z	q_h	$G \cdot C_p$	GC_{pi}	$P_{windward}$
1	10.047	15.863	0.68	+/- 0.18	9.69
2	10.928	15.863	0.68	+/- 0.18	10.29
3	12.338	15.863	0.68	+/- 0.18	11.25
4	13.395	15.863	0.68	+/- 0.18	11.96
5	14.277	15.863	0.68	+/- 0.18	12.56
6	14.982	15.863	0.68	+/- 0.18	13.04
Roof	15.863	15.863	0.68	+/- 0.18	13.64

N-S Leeward

Level	z	K_z	q_h	$G \cdot C_p$	GC_{pi}	$P_{Leeward}$
1	0	0.57	15.863	-0.17	+/-0.18	-5.55
2	16	0.62	15.863	-0.17	+/-0.18	-5.55
3	25.92	0.70	15.863	-0.17	+/-0.18	-5.55
4	35.83	0.76	15.863	-0.17	+/-0.18	-5.55
5	45.75	0.81	15.863	-0.17	+/-0.18	-5.55
6	55.67	0.85	15.863	-0.17	+/-0.18	-5.55
Roof	72.3	0.90	15.863	-0.17	+/-0.18	-5.55

Windward and Leeward

Level	P_{total} (PSF)
1	15.24
2	15.84
3	16.8
4	17.51
5	18.11
6	18.59
Roof	19.19

E-W Windward Same as N-S Windward

E-W Leeward

Level	z	Kz	G·Cp	qh	GCpi	$P_{Leeward}$
1	0	0.57	-0.425	15.863	+/- 0.18	9.60
2	16	0.62	-0.425	15.863	+/- 0.18	9.60
3	25.92	0.70	-0.425	15.863	+/- 0.18	9.60
4	35.83	0.76	-0.425	15.863	+/- 0.18	9.60
5	45.75	0.81	-0.425	15.863	+/- 0.18	9.60
6	55.67	0.85	-0.425	15.863	+/- 0.18	9.60
Roof	72.3	0.90	-0.425	15.863	+/- 0.18	9.60

E-W

Windward and Leeward

Level	P total (PSF)
1	19.29
2	19.89
3	20.85
4	21.56
5	22.16
6	22.64
Roof	23.24

Wind Load Vertical Distribution (L = 66'-6") N-S

E-W 283'-8" = 283.67'

Roof Level

$19.19\text{ psf}(8.315)(66.5) = 10.6^{k}$ N-S
54.8^{k} E-W

Level 6

$19.19\text{ psf}(8.315)(66.5) + 18.59(4.96)(66.5) = 16.7^{k}$ N-S
86.7^{k} E-W

Level 5

$18.59(4.96)(66.5) + 18.11(4.96)(66.5) = 12.1^{k}$ N-S
63.0^{k} E-W

Level 4

$18.11(4.96)(66.5) + 17.51(4.96)(66.5) = 11.8^{k}$ N-S
61.5^{k} E-W

Level 3

$17.51(4.96)(66.5) + 16.8(4.96)(66.5) = 11.3^{k}$ N-S
60.0^{k} E-W

Level 2

$16.8(4.96)(66.5) + 15.84(8.0)(66.5) = 14.0^{k}$ N-S
74.5^{k} E-W

Level 1

$15.84(8.0)(66.5) = 8.4^{k}$ N-S
45.1^{k} E-W

Base Shear = 84.9^{k} N-S
445.6^{k} E-W

Overturning Moment

N-S

$$M = 10.6^{k}(72.3') + 16.7^{k}(55.67') + 12.1^{k}(45.75') + 11.8^{k}(35.83') + 11.3^{k}(25.92') + 11.3^{k}(16')$$
$$= 3{,}146.5^{k\text{-}ft}$$

E-W

$$M = 54.8^{k}(72.3') + 86.7^{k}(55.67') + 63.0^{k}(45.75') + 61.5^{k}(35.83') + 60.0^{k}(25.92') + 74.5^{k}(16')$$
$$= 16{,}621.6^{k\text{-}ft}$$

Seismic Load Calculations:

Pearl Condominiums
9th and Arch Street
Philadelphia, PA 19107

Site Class D

From USGS Website

$S_S = \underline{0.270g}$

$S_1 = \underline{0.060g}$

USING ASCE 7-05 OPTION

Determine F_a, F_v

$F_a = \underline{1.585}$ (Table 11.4-1, Site Class D)

$F_v = \underline{2.4}$ (Table 11.4-2, Site Class D)

Determine S_{MS}, S_{M1}

$S_{MS} = F_a \cdot S_S = 1.585(0.270) = \underline{0.430g}$

$S_{M1} = F_v \cdot S_1 = 2.4(0.060) = \underline{0.144g}$

Determine S_{DS}, S_{D1}

$S_{DS} = 2/3 \cdot S_{MS} = \underline{0.287}$

$S_{D1} = 2/3 \cdot S_{M1} = \underline{0.096}$

Two Options Used In Conjunction As Lateral Force-Resisting System

Option I: Reinforced Masory Shear Walls ($\underline{R = 5\frac{1}{2}}$)

Option II: Cold Form Metal Stud Shear Walls ($\underline{R = 4}$)

Option I: $C_d = 4$ Option II: $C_d = 3\frac{1}{2}$

Importance Factor

$I = \underline{1.0}$, per Table 11.5-1 (Occupancy Category II)

Determine Seismic Design Category

Seismic Design Category = $\underline{B}$

$0.167 \leq S_{DS} = 0.287 < 0.33$ Table 11.6-1

Seismic Design Category = $\underline{B}$

$0.067 \leq S_{D1} = 0.096 < 0.133$ Table 11.6-1

Approximate Fundamental Period

$T_a = 0.02\,(72'' - 4'')^{0.75} = 0.496$ sec

$C_u = 1.7$

$S_{D1} = 0.096 \leq 0.1$

$C_u \cdot T_a = 1.7(0.496) = 0.8432$

Equivalent Lateral Force Method — Option I — Option II

$$C_s \geq \begin{cases} S_{DS}/(R/I) & 0.287/(5.5/1) = 0.052 \quad\quad 0.287/(4/1) = 0.07175 \\ S_{D1}/[T(R/I)] & 0.096/0.8432(5.5/1) = 0.0207 \quad\quad 0.096/0.8432(4/1) = 0.0285 \end{cases}$$

$C_s = \underline{0.0207}$ $C_s = \underline{0.0285}$

$C_s = 0.0207$ w/ $T = C_u \cdot T_a$

$C_s = 0.0352$ w/ $T = T_a$

Level	Wall Weight	Floor/Roof Weight
2	50psf(700.3')(12.958') = 454K	(66psf + 25psf) 18,863 = 1717K
3	50psf(700.3')(10.375') 363K	1717K
4	50psf(700.3')(9.9167') 347K	1717K
5	50psf(700.3)(9.9167') 347K	1717K
6	50psf(700.3')(11.375') 398K	1717K
Roof	50psf(700.3')(10.247') 359K	(30psf + 20psf) 18,863 = 943 K

perimeter = 2(283'-8") + 2(66'-6") = 700.3ft

Floor/Roof Area = 18,863 sqft

Total Weight = 11796K

Base Shear

$V = C_s \,(w/T = C_u \cdot T_a)\; W$

$= 0.0207\,(11796^K) = \underline{244.2^K}$

$V = C_s\,(w/T = T_a)\; W$

$= 0.0352\,(11796^K) = \underline{415.2^K}$

Period T= Approximate Period Ta

V= 415.2^K

K= 1.0

Level	Weight(k)	Story Height h	h^K	Wx*hx^K
2	2171	16.000	16.00	34736
3	2080	25.917	25.917	53907.36
4	2064	35.833	35.833	73959.312
5	2064	45.750	45.750	94428
6	2115	55.667	55.667	117735.705
Roof	1302	68.500	68.500	89187
Total	11796			463953.377

Level	Cvx	Fx	Fx hx
2	0.0749	31.1	497.60
3	0.1162	48.2	1249.20
4	0.1594	66.2	2372.14
5	0.2035	84.5	3865.88
6	0.2538	105.4	5867.30
Roof	0.1922	79.8	5466.30
Total	1	415.2^K	19318.42 K-ft

Over-Turning Moment = Σ Fxhx = 19,318.42^{K-ft}

Period $T = \text{Max } C_u * T_a$

$V = 244.2^k$

$K = 1.17$

Level	Weight (k)	Story Height h	h^k	$w_x * h_x$^k
2	2171	16.000	25.634	55651.414
3	2080	25.917	45.071	93747.68
4	2064	35.833	65.843	135899.952
5	2064	45.750	87.631	180870.384
6	2115	55.667	110.243	233163.945
Roof	1302	68.500	140.526	182964.852
Total	11796			882298.227

Level	C_{vx}	F_x	$F_x h_x$
2	0.0631	15.41	246.56
3	0.1063	25.96	672.81
4	0.1540	37.61	1347.68
5	0.2050	50.06	2290.25
6	0.2643	64.54	3592.73
Roof	0.2074	50.65	3469.53
Total	1	244.23^k	11619.56 k-ft

Over Turning Moment $= \sum F_x h_x = \underline{11,619.56}$ k-ft

Spot Check Calculations:

Steel Joist

Spot Checks

Steel Roof Joist

Dead Load

Tributary width - 4'-0"

Worst Case Span - 34'-9"

- Single Ply membrane 0.7 psf
- 5/8 Protection Board 1 psf
- R-30 Rigid Insulation 3 psf
- 5/8" Gypsum Wall Board 2.5 psf
- 1 1/2" Min Steel Deck 3 psf
- Steel Roof Joists 4 psf
- Steel Bridging 1 psf
- 5/8" Gypsum Wall Board on Suspended Ceiling Panel 4.5 psf
- MEP - 5 psf
- Collateral - 5 psf

Total = 30 psf

Superimposed Dead Load - 20 psf

Live Loads - 30 psf

Total Load ASD

= 30 + 20 + 30 = 80 psf

80 psf (4') = 320 plf

From ASD Standard Load Table for open web Steel Joists K-Series from New Columbia Joist Company

Select 24K8 w = 360 plf > 320 plf

Original Design Selection 24K9

Steel Wide Flange Column

Spot Check

Wide Flange Column

Tributary Area =

$$\left[22'-6''/2 + \left(\frac{15'-10'' + 6'-1''}{2}\right)\right] \times \left(\frac{66'-6''}{2}\right)$$

= 755 sq ft

$P_u = 150 \text{ psf}\,[755 \times 5] + 80\,[755]$

$566.25^K + 60.4^K = 626.65^K$

Try W30x90

Length - 16'

$r_y = 2.09$ in

$K = 1.0$

$$\frac{KL}{r_y} = \frac{16(12)}{2.09} = 91.87 < 4.71\sqrt{E/F_y} = 113.4$$

$$F_e = \frac{\pi^2\, 29000}{(91.87^2)} = 33.91$$

$$F_{cr} = 0.658^{(F_y/F_e)}\,F_y = 0.658^{(50/33.91)}\,50$$

$= 26.97$ ksi

$\phi P_n = 0.9\,(26.97 \text{ ksi})(26.4 \text{ in}^2) =$

$= 648.1^K > P_u = 626.65^K$

USE W30x90

Original Design W30x90

Metal Stud in Load Bearing Wall

Spot Check

Metal Stud in Load Bearing Wall (Bearing Wall Type 3)

Level 5

Tributary Area = 33'-9" x [66'-6"/2] = 1131 sqft

[150 psf + 80 psf] 1130 sqft = 259.9 K

259.9 K / ((66'-6")/2) = 7.82 K/ft

7.82 K/ft (12/12) = 7.82 K

From Marion\Ware Light weight Steel Framing Systems

SSMA

800S200-97 8" member t = 0.1017 in

V_a = 5.938 x2 @ 12"

= 11.876 K

Original Design 8" member

SSMA t = 0.0566 in

800S200-54

Difference in Load factoring

Masonry Shear Wall

Spot Check

Masonry Shear Wall (6th Floor) 10SW1

$\frac{M}{Vd} = \frac{h}{2d} = \frac{16.63'}{2(20')} = 0.42 < 1$

Assumption shear force divide equally between masonry towers

$F_v = \frac{1}{2}(4 - 0.42)(1500)^{1/2}$ $n = 4$

$F_v = 1342.5 \text{ psi} < 120 - 45(0.42) = 101.1$

Steel Resisting All Shear – Assumption

10 in CMU

$F_v (\text{Area}_{CMU}) = 1342.5 \text{ psi} (47.40 \text{ in}^2)$

$= 63.6 \text{ kips}$

$63.6 \text{ kips } F_v < F_x = 105.4^k / 2 \text{ walls}$

$= 52.7^k$

If assumptions hold true 10SW1

6th Floor – 10" w/ #5 @ 48" O.C.

$f'_m = 1500 \text{ psi}$

Then Calculations are Correct.