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Structural Option

Advisor: Dr. Linda Hanagan
The 400: Bremerton, WA
October 5, 2005
AE 481W



Structural Technical Report 1

Structural Concepts / Structural Existing Conditions Report

The 400 is a condominium complex located in Bremerton, Washington, right across the bay from Seattle. The building consists of two levels of structured concrete parking below four stories of residential metal frame construction. The majority of the parking is above ground. Ground has recently been broken for construction of The 400, and updated plans are in the process of being developed. No Mechanical, Electrical, or Plumbing designs are readily available, but they should be available by mid-November, 2005.

This technical assignment consists of an analysis of the existing structural system. Many aspects must be taken into consideration while designing a structural system, many of which are included in this assignment. The loading (dead, live, and snow) are calculated to be used for analysis. These calculations are followed by detailed explanations of the structural and foundation system as well as lateral system. The 400 consists of four stories of residential metal frame construction (condominiums) above two stories of structural concrete parking. The metal construction is a non-composite system of steel floor joists supported by girders; steel studs are used for the walls. The parking consist of slab on grade wherever possible and a post-tensioned slab wherever slab on grades are not possible.

The analysis of the lateral system, both wind and seismic, includes diagrams with supporting calculations in Appendix A. The controlling lateral case, seismic, is then evaluated, further explained in Appendix B.

Finally, a typical RAM floorplan was created. A simplified rectangular floorplan was used for analysis in spot checking several members. While the members were relatively close in size, variation in sizes is most likely due to length variation throughout the actual floorplan. The same codes used to design the building were used in this RAM analysis. Detailed calculations supporting these spot checks are located in Appendix C. Sketches of plans and details are included throughout the assignment and in the Appendix to help clarify design process.

Introduction:

The 400 is a condominium complex located in Bremerton, Washington, right across the bay from Seattle. The building consists of two levels of structured concrete parking below four stories of residential metal frame construction. The majority of the parking is above ground. Ground has recently been broken for construction of The 400, and updated plans are in the process of being developed. No Mechanical, Electrical, or Plumbing designs are readily available, but they should be available by mid-November, 2005.

The 400 is located within walking distance from the ferry boat which connects directly to downtown Seattle, and the condominium complex is going to be built right along the Port Washington Narrows. Maximum use of the site is being implemented for design, and careful design considerations needed to be taken into account because the building is so close to the water. Due to the heavy seismic activity possible in the geographic location of The 400, additional design requirements need to be considered.

Overall Structural System:

The bottom two stories of The 400 consist of slab on grade or post-tensioned slab parking (generally 8" normal weight). Four stories of light gage steel residential frame construction are built above the two levels of parking. Wooden trusses are then used to frame the roof. The gross floor area for The 400 is approximately 124,000 square feet.

The 400 is built according to the 2003 International Building Code and locally to the Bremerton Municipal Code. ASCE 7-02, ACI 318-05, and the LRFD Manual of Steel Construction were used for analysis of the building both while being designed and in the following calculations.

The concrete strength in The 400 ranges from 3,000 psi for the pile caps and footings to 4,000psi for the slabs to 5,000 psi for the concrete shear walls and columns. Steel grades and strengths are outlined in the table below:

Member	ASTM SPECIFICATION	Fy
Wide flange and WT shapes	A992	50 ksi
Plates, angles, channels, and rods	A36	36 ksi
Pipe members	A53	35 ksi
Structural tubing	A500	46 ksi
Anchor bolts	A307	
Connection bolts	A325-N	

Table 1

National Codes/Design Codes:

- 2003 International Building Code
- ASCE 7-02
- ACI 318-05
- LRFD Manual of Steel Construction

Design Loads:

Live Loads (From ASCE 7-02 Table 4-1):

Roof live load (including snow)	25 psf
Floor live load (parking)	40 psf
Floor live load (corridors/lobbies)	100 psf
Floor live load (residential units and decks)	40 psf
Attic live load (non-simultaneous with roof live load; no storage or living)	10 psf
Stair live load	100 psf
Guardrails/balcony rails	50 plf / 200 lb

Table 2

Dead Loads:

Metal Roof Deck	2 psf
Trusses (roof)	20 psf
Ceiling	5 psf
Mechanical/Electrical/Plumbing	15 psf
Concrete (with metal deck)	30 psf
Concrete Slab (parking)	100 psf
Perimeter Wall	15 psf

Table 3

NOTE: Construction loads of at least 20 psf must be accounted for during construction procedures.

Snow Load:

Chapter 7 of ASCE 7-02 was used to calculate the snow loads for The 400.

The roof is considered to be flat; therefore, section 7-3 applies:

$$\rho_f = 0.7C_eC_tI\rho_g$$

Terrain Category = B (6.5.6.2)

$C_e = 0.9$ (Table 7-2)

$C_t = 1.0$ (Table 7-3)

Building Category = II (Table 1-1)

$I = 1.0$ (Table 7-4)

$\rho_g = 20\text{psf}$ (Figure 7-1)

$$\rho_f = 0.7(0.9)(1.0)(1.0)(20\text{psf}) = 12.6 \text{ psf}$$

Minimum ρ_f for ρ_g of 20psf or less: $\rho_g (I) = 20\text{psf} (1.0) = 20\text{psf}$

Therefore, $\rho_f = 20\text{psf}$

NOTE: Structural notes of building specify a snow load of 30psf was used; this is by convention to use a minimum of 30psf in the Bremerton area.

Typical Floor layout:



NOTE: While exact floor layout varies between floors, this floorplan is considered to be most typical and assuming a rectangular layout; overall floor area per floor generally changes by less than 5%.

Following is a more detailed description of what is specifically unique to each level:

Lower Parking Level – P2

A four inch slab-on-grade with one #4 for reinforcement at 18 inches on center each way is the primary slab, while a 6 inch slab with an $f'c$ of 4000 psf is used at the exterior edge on this 25,844 gross square feet level. This level is supported by 7'6" square pilecaps 48 inches deep and is connected with standard hooks on both top and bottom, and the footings reach a height of 51'0". Control joints must be spaced so as to not exceed 225 square feet, and the areas should be approximately square with no acute angles.

Twelve shear walls are located on the on this level, 12" thick. Most reinforcing for the shear walls is one #5 for reinforcement at 18" on center each way, each face. At critical points, reinforcement can reach up to nine #6 for reinforcement at 3" on center, each face, and the lap splices range from 16" to 132", depending on bar size and concrete strength.

Three foot wide square spread footings one foot deep are located throughout the P2 level. The depth, however, can reach up to three feet, depending on location. Eight inch concrete walls (not shear) are located throughout the lower parking level. The P2 level generally contains vertical tube steel 6x6x1/4 throughout the exterior concrete walls, and the only grade beams specified are 36" x 30".

Upper Parking Level - P1

Of the 13,685 gross square feet P1 level, approximately half of the level contains a 4" slab-on-grade just like the P2 level, and the other half contains an 8 1/2" post-tensioned slab. The post-tensioned slab contains 9'4" square drop panels 4 1/4" deep at each column, and four #4 are used continuously on the top and bottom; this is general reinforcement requirements for the P1 level.

The P1 level contains 8" concrete walls. Just as in the P2 level, 6x6x1/4 tube steel is typically used vertically throughout the exterior concrete walls. One corner contains a 3'9"x3'9"x4 1/4" drop cap. This is because of the poor soil which is located in the southeast building corner, requiring additional load distribution. Unique to this level, however, is a three foot closure/pour strip and an extra stair well for access to the P2 parking level.

First Floor Framing –R1

The first residential level, approximately 228'x115', consists of a twelve inch post-tensioned slab with ten foot square drop panels 6 ¼" deep with ten #5 on the top each way for reinforcement. Generally, four #4 top and bottom are used continuously along the perimeter of the slab. Similar to the P1 level, this level also contains a 3' closure/pour strip, dividing the floor into almost two squares.

Residential Framing - R2, R3, R4

Each residential floor is approximately 21,000 gross square feet. The wall framing consists of metal studs spaced typically 2 feet on center. The typical deck slab is made up of ½" metal form deck with 2 ½" concrete. The floor joists typically used are generally 10TDW16 steel joists 24" on center, supported by W10x15 girders.

Roof Framing

The roof framing typically consists of C8x18.75 and W8x28 as struts throughout the roof framing plan. Then, 5/8" plywood sheathing is generally used with connector-plate trusses 24" on center.

Foundation Analysis:

The southeast building corner has extremely poor soil, requiring additional support. Considering the size of the building, the footings are relatively large, reflecting the relatively poor soil.

Lateral Analysis:

Shear walls comprise several major walls throughout the building, extending from bottom parking slab to the wooden trusses on the roof. Due to the fact that few, if any, vertical design elements are available, additional framing information is unknown.

The following lateral analyses of both wind and seismic were developed in accordance with Chapter 6 and Chapter 9 (respectively) of ASCE 7-02. More detailed and intermediate calculations are provided Appendix A. Worst case situations were evaluated laterally, and controlling lateral case, which turned out to be seismic, is explained in detail in Appendix B.

Calculating wind loads against building (psf):

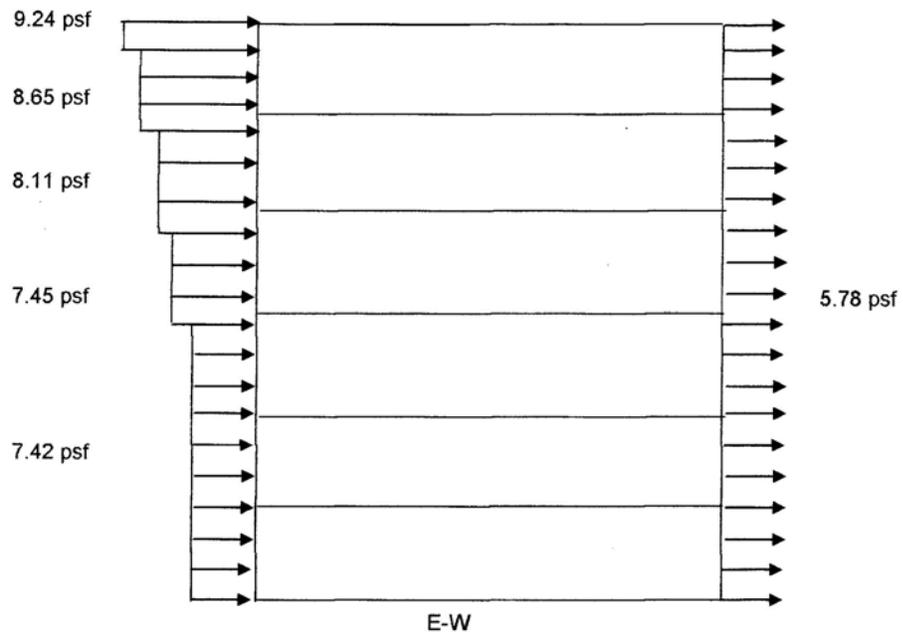
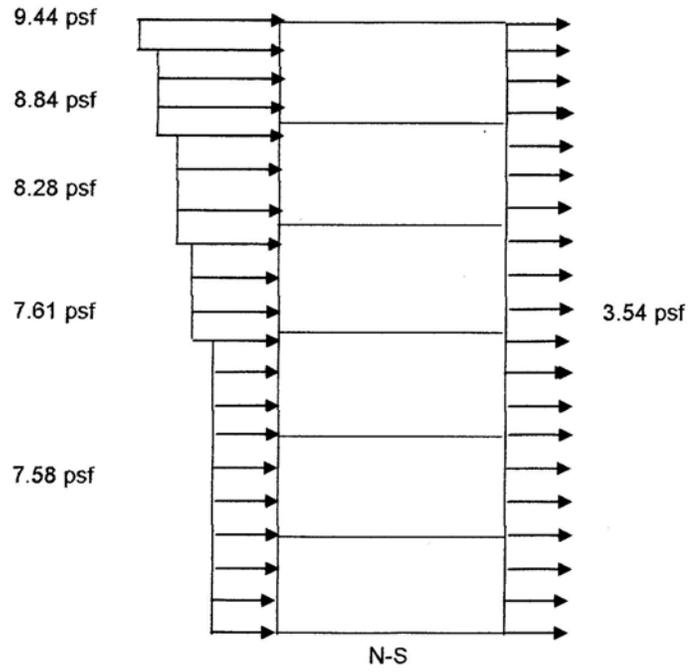
z(ft)	K _z (Table 6-3)	q _z (lb/ft ²)	q _h (lb/ft ²)	q _z C _p G (lb/ft ²) Windward	q _h C _p G (lb/ft ²) Leeward	q _z C _p G - q _h C _p G (lb/ft ²) Total
0	0.7	11.01	13.71	7.58	(3.54)	11.12
15	0.7	11.01	13.71	7.58	(3.54)	11.12
20	0.7	11.01	13.71	7.58	(3.54)	11.12
25	0.7	11.01	13.71	7.58	(3.54)	11.12
30	0.7	11.01	13.71	7.58	(3.54)	11.12
40	0.76	11.95	13.71	8.23	(3.54)	11.77
50	0.81	12.73	13.71	8.77	(3.54)	12.31
60	0.85	13.36	13.71	9.20	(3.54)	12.74
70	0.89	13.99	13.71	9.64	(3.54)	13.18
N-S						

z(ft)	K _z (Table 6-3)	q _z (lb/ft ²)	q _h (lb/ft ²)	q _z C _p G (lb/ft ²) Windward	q _h C _p G (lb/ft ²) Leeward	q _z C _p G - q _h C _p G (lb/ft ²) Total
0	0.7	11.01	13.71	7.42	(5.78)	13.20
15	0.7	11.01	13.71	7.42	(5.78)	13.20
20	0.7	11.01	13.71	7.42	(5.78)	13.20
25	0.7	11.01	13.71	7.42	(5.78)	13.20
30	0.7	11.01	13.71	7.42	(5.78)	13.20
40	0.76	11.95	13.71	8.06	(5.78)	13.83
50	0.81	12.73	13.71	8.59	(5.78)	14.36
60	0.85	13.36	13.71	9.01	(5.78)	14.79
70	0.89	13.99	13.71	9.43	(5.78)	15.21
E-W						

Calculating story forces and story shear (kip):

Level	Story Forces		Story Shear	
	N-S (kips)	E-W (kips)	N-S (kips)	E-W (kips)
Base	7.84	17.98		
2	14.94	34.53	7.84	17.98
3	14.28	33.24	22.78	52.51
4	13.47	31.67	37.06	85.75
5	13.43	31.59	50.53	117.42
6	13.43	31.59	63.95	149.01
			77.38	180.60

Wind Diagrams



NOTE: Table calculations interpolated for exact story height along the vertical side of the building.

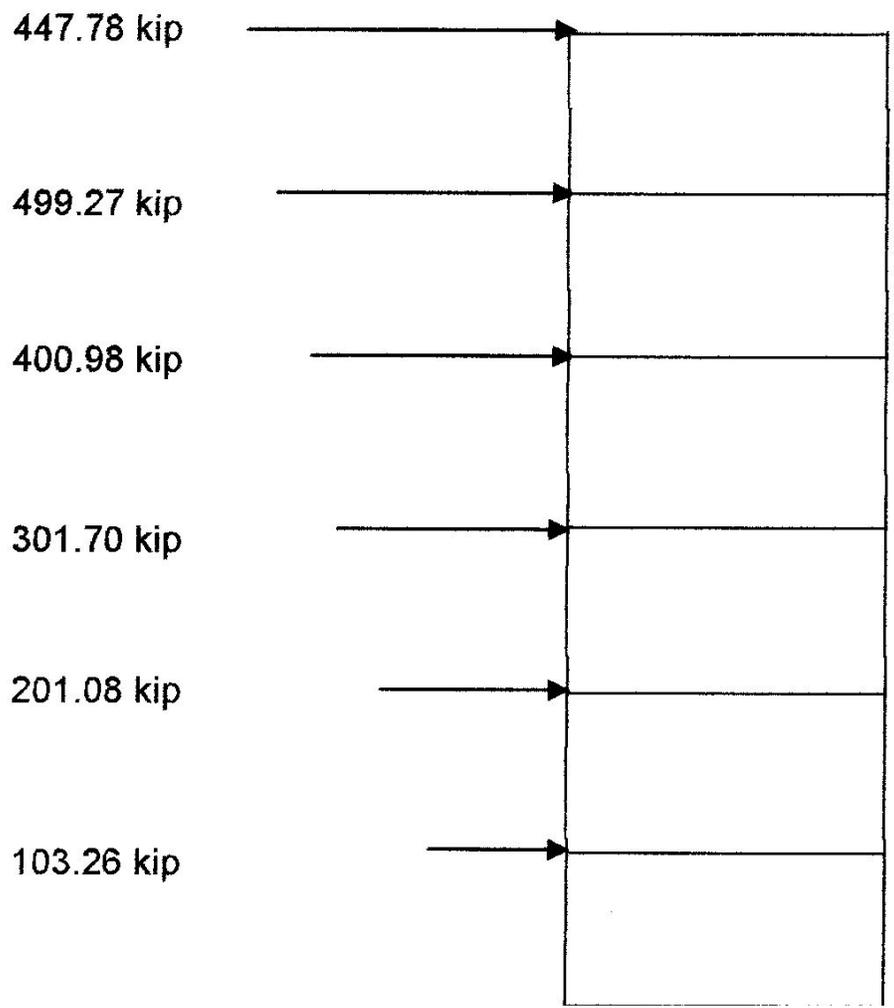
Seismic Design Summary:

Vertical Distribution

N-S: $k_{N-S} = 1 + (T_{N-S} - 0.5) / (2.5 - 0.5) = 0.96$

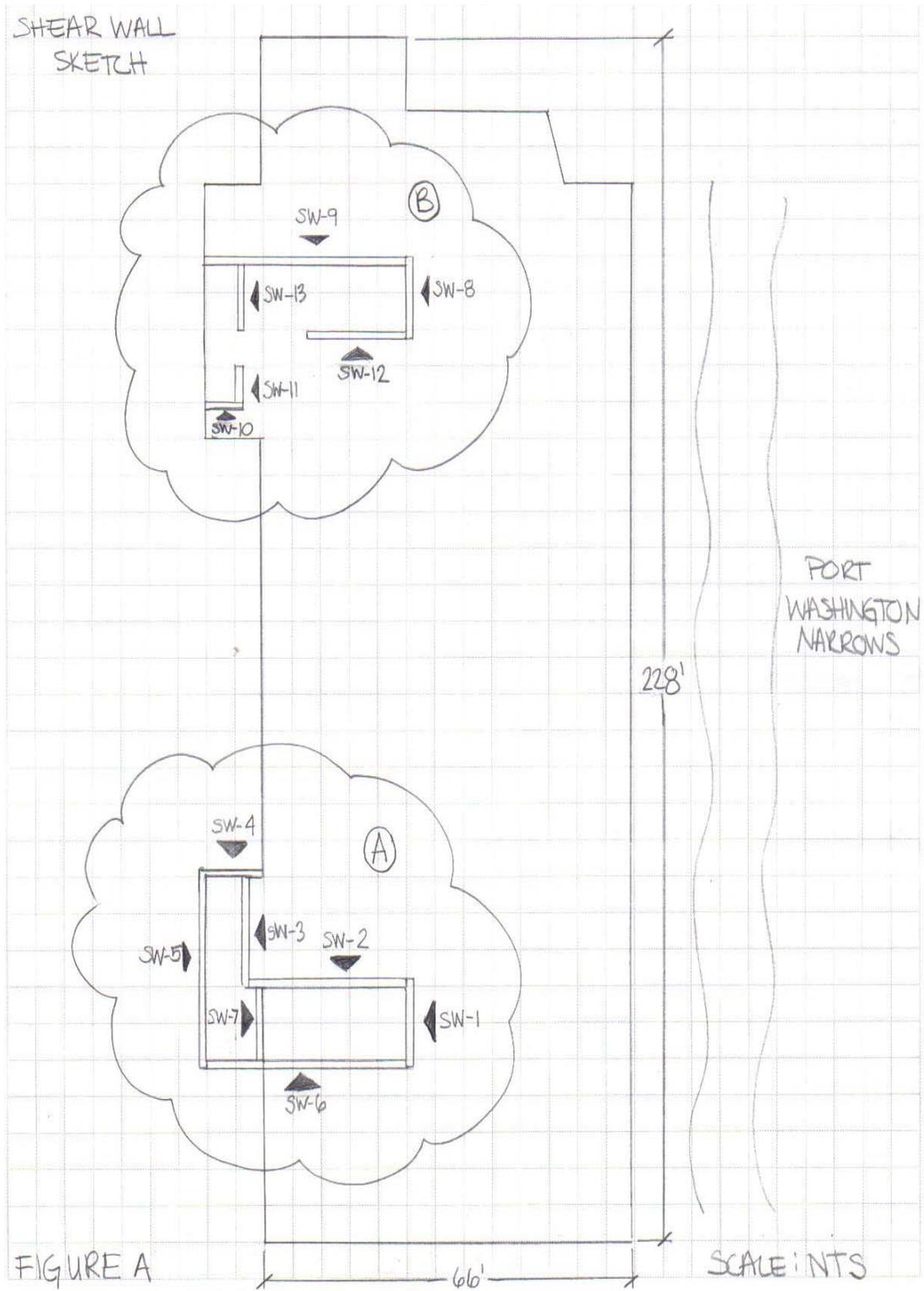
Level	w_x (kips)	h_x (ft)	$w_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)
Roof	1233.9	65.5	68810.4	0.2	447.8
6	1733.7	51.5	76722.0	0.3	499.3
5	1733.7	41.0	61617.8	0.2	401.0
4	1733.7	30.5	46362.3	0.2	301.7
3	1733.7	20.0	30899.1	0.1	201.1
2	1733.7	10.0	15867.1	0.1	103.3
1					
Sum:	9902.3		300278.8	1.0	1954.1

NOTE: Analysis identical for N-S and E-W seismic



NOTE: Identical seismic analysis for both N-S and E-W direction

Shear Walls:



Lateral Load Distribution Procedure:

SW	L (ft)	h/L	T/S/O (*)	k / (Et/h)	x _i (ft)	y _i (ft)	k _x	k _y	k _i for x	k _i for y	d _i	k _i d _i ²
1	10	6.18	T	0.066		34		2.23		0.07	-28.19	52.10
2	21	2.94	O	0.562	50		28.10		0.56		-57.27	1843.32
3	13	4.75	T	0.144		58		8.35		0.14	-4.19	2.53
4	12	5.15	T	0.113	66		7.48		0.11		-41.27	192.97
5	28	2.21	O	1.259		66		83.11		1.26	3.81	18.28
6	32	1.93	O	1.810	40		72.42		1.81		-67.27	8192.92
7	8	7.72	T	0.034		54		1.81		0.03	-8.19	2.25
8	10	6.18	T	0.066		34		2.23		0.07	-28.19	52.10
9	32	1.93	O	1.810	180		325.89		1.81		72.73	9576.86
10	9	6.86	T	0.048	162		7.74		0.05		54.73	143.17
11	9	6.86	T	0.048		57		2.72		0.05	-5.19	1.29
12	18	3.43	T	0.382	172		65.77		0.38		64.73	1602.12
13	9	6.86	T	0.048		58		2.77		0.05	-4.19	0.84
Table 4							507.40	103.24	4.73	1.66		21680.75

SW	E-W		N-S		E-W		N-S	
	F _{iDirect, x}	F _{iDirect, y}	F _{iTorsion, x}	F _{iTorsion, y}	F _{i, x}	F _{i, y}		
1	1.24	1.24	4.86	-1.12	6.10	1.24		
2	2.16	2.16	84.68	-19.52	86.84	2.16		
3	2.73	2.73	1.59	-0.37	4.31	2.73		
4	0.44	0.44	12.30	-2.84	12.74	0.44		
5	23.84	23.84	-12.62	2.91	23.84	26.75		
6	6.97	6.97	320.42	-73.87	327.39	6.97		
7	0.64	0.64	0.72	-0.17	1.36	0.64		
8	1.24	1.24	4.86	-1.12	6.10	1.24		
9	6.97	6.97	-346.42	79.87	6.97	86.84		
10	0.18	0.18	-6.88	1.59	0.18	1.77		
11	0.90	0.90	0.65	-0.15	1.56	0.90		
12	1.47	1.47	-65.12	15.01	1.47	16.49		
13	0.90	0.90	0.53	-0.12	1.43	0.90		

Table 4 (continued)

height of wall: 61.75 ft

(*) Tall wall (T) with h/L > 3 / Short wall (S) with h/L < 0.3 / Other (O) with 0.3 < h/L < 3

For Tall Walls, $k = L^3 / (4h^2) \times (Et/h)$

For Short Walls, $k = L / 2.78 \times (Et/h)$

For Other Walls, $k = 1 / (4h^2 / L^3 + (2.78 / L)) \times (Et/h)$

x-CR 107.27 ft P_x 1954 kip e_x 6.73 ft M_{IE-W} -57039.01 ft-kip

y-CR 62.19 ft P_y 1954 kip e_y -29.19 ft M_{IN-S} 13150.82 ft-kip

These shear wall lateral values make sense because for the most part, the shear walls carry the most load in the direction in which they are running parallel to. In addition, the largest overall values are located the farthest away from the center of rigidity, while the smallest values are located closest to the center of rigidity.

Other Structural Elements:

The increase in wind loading also comes into effect when designing for uplift on the roof. The increased wind may mean an increased chance of failure due to uplift on the building. Lastly, the increased wind loading can create an overturning moment which will need to be taken into consideration while designing the structural system. This becomes very crucial when designing the foundation system for the building.

Because of the proximity to the water and increased wind loading, higher strength windows were required throughout the building. This, in turn, increased the cost of the building.

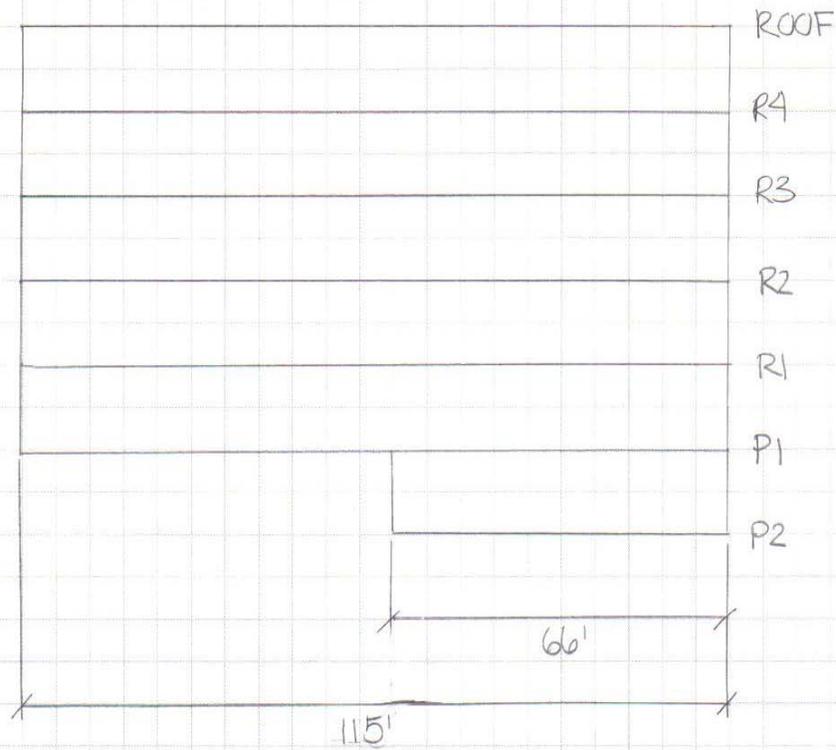
Spot Checking:

A RAM model of a typical floor was created to verify typical floor members and bays (see following page). The vertical members (parallel to the joists) according to the RAM model were chosen to be W8x10. These members were typically sized as W10x19 on the structural plans. The relative sizes, however make sense because the smaller members should be running parallel to the joists because the joists would be decreasing the tributary area of the girders. The variation in size of typical members is most likely due to the fact that the lengths of the bays were sometimes longer than the length used in the RAM model (due to alterations in building design for architectural reasons, such as the courtyard area).

NOTE: The structural members are also only partially complete and not construction drawings; verification of members is difficult. The structural plans do not currently contain enough information to provide a spot check for columns, so a column spot check was not yet performed. Once updated structural plans are obtained, adequate spot checks will be performed.

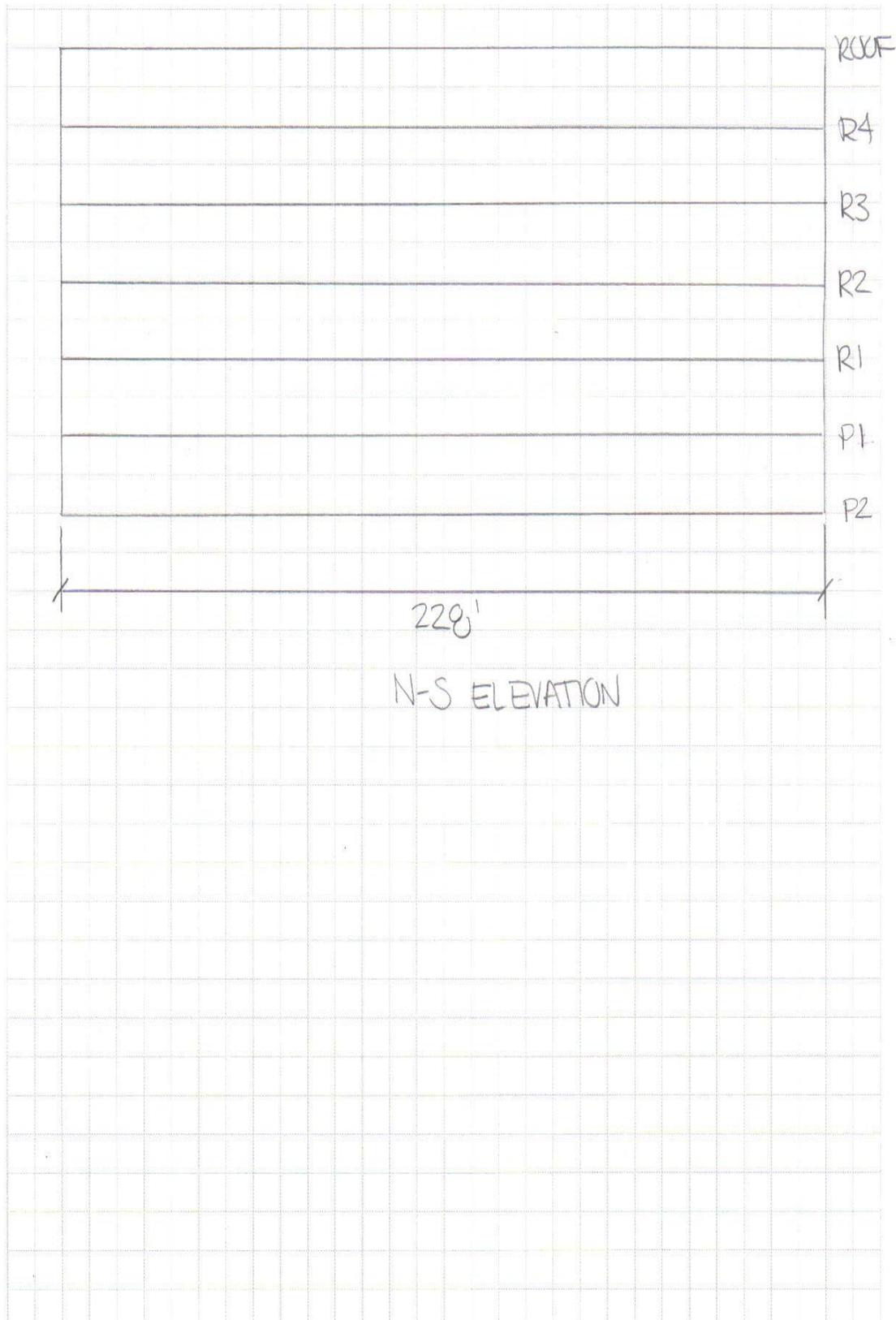
APPENDIX

APPENDIX A
Lateral Design



E-W ELEVATION

*FOR WIND ANALYSIS, ASSUME 115' FT
LENGTH FOR ALL FLOORS



Wind Analysis

Dimension	N-S	228.00	ft	
	E-W	115.00	ft	
	Total Height, h	58.50	ft	
	Inter-story Height, h _s	10.50	ft	
Velocity Pressure	K _z t	1.00		(Table 6-4) Assumed area is flat
	K _d	0.85		(Table 6-4)
	V	85.00	mph	(Figure 6-1)
	Group	II		(Table 1-1)
	Importance Factor, I	1.00		(Table 6-1)
	Exposure	C		Assumed area is flat
	q _z /K _z = 0.00256K _z tK _d V ² /I =	15.72		

z(ft)	K _z (Table 6-3)	q _z (lb/ft ²)	q _h (lb/ft ²)	q _z C _p G (lb/ft ²) Windward	q _h C _p G (lb/ft ²) Leeward	q _z C _p G - q _h C _p G (lb/ft ²) Total
0	0.7	11.01	13.71	7.58	(3.54)	11.12
15	0.7	11.01	13.71	7.58	(3.54)	11.12
20	0.7	11.01	13.71	7.58	(3.54)	11.12
25	0.7	11.01	13.71	7.58	(3.54)	11.12
30	0.7	11.01	13.71	7.58	(3.54)	11.12
40	0.76	11.95	13.71	8.23	(3.54)	11.77
50	0.81	12.73	13.71	8.77	(3.54)	12.31
60	0.85	13.36	13.71	9.20	(3.54)	12.74
70	0.89	13.99	13.71	9.64	(3.54)	13.18
N-S						

z(ft)	K _z (Table 6-3)	q _z (lb/ft ²)	q _h (lb/ft ²)	q _z C _p G (lb/ft ²) Windward	q _h C _p G (lb/ft ²) Leeward	q _z C _p G - q _h C _p G (lb/ft ²) Total
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50	0.81	12.73	13.71	8.59	(5.78)	14.36
60	0.85	13.36	13.71	9.01	(5.78)	14.79
70	0.89	13.99	13.71	9.43	(5.78)	15.21
E-W						

Calculating story forces and story shear (kip):

Level	Story Forces		Story Shear	
	N-S (kips)	E-W (kips)	N-S (kips)	E-W (kips)
Base	7.84	17.98		
2	14.94	34.53	7.84	17.98
3	14.28	33.24	22.78	52.51
4	13.47	31.67	37.06	85.75
5	13.43	31.59	50.53	117.42
6	13.43	31.59	63.95	149.01
			77.38	180.60

Wind Analysis (continued)

Gust Factor Calculation
E-W

B	115 ft	L/B = 1.98
L	228 ft	
h	58.5 ft	
Table 9.5.5.3.2 C_t	0.02	
Table 9.5.5.3.2 x	0.75	
$f=1/C_t h^x$	2.36 (Rigid)	
6.5.6.3	Exposure C	
Table 6-2 z_{min}	15	
Table 6-2 c	0.2	
Table 6-2 i	500	
$RL = 1/hL - (1-e^{-2hL})/2hL2 =$	n.a.	
$N1 = h1/Lz/Vz =$	n.a.	
$Rn = 7.47N1/(1+10.3N1)^{5/3} =$	n.a.	
$R = [(RnRhRB(0.53+0.47RL)/b)0.5 =$	n.a.	
$gR = (2\ln(3600n1))0.5+0.577/(2\ln(3600n1))0.5 =$	n.a.	
g_{RR}	0.000	
$G = 0.925[1+1.7Iz((g_{QQ})^2+(g_{RR})^2)0.5]/(1+1.7g_vIz) =$	0.843	

Gust Factor Effect Calculation
N-S

B	115 ft	L/B = 1.98
L	228 ft	
h	58.5 ft	
Table 9.5.5.3.2 C_t	0.02	
Table 9.5.5.3.2 x	0.75	
$f=1/C_t h^x$	2.36 (Rigid)	
6.5.6.3	Exposure C	
Table 6-2 z_{min}	15	
Table 6-2 c	0.2	
Table 6-2 i	500	
$RL = 1/hL - (1-e^{-2hL})/2hL2 =$	n.a.	
$N1 = h1/Lz/Vz =$	n.a.	
$Rn = 7.47N1/(1+10.3N1)^{5/3} =$	n.a.	
$R = [(RnRhRB(0.53+0.47RL)/b)0.5 =$	n.a.	
$gR = (2\ln(3600n1))0.5+0.577/(2\ln(3600n1))0.5 =$	n.a.	
g_{RR}	0.000	
$G = 0.925[1+1.7Iz((g_{QQ})^2+(g_{RR})^2)0.5]/(1+1.7g_vIz) =$	0.861	

Seismic Design:

Design Parameters			
ASCE Reference	Location	Bremerton, WA	
	Number of stories	N	6
	Story Height	h_s	10.5 ft
	Mean building height	h_n	58.5 ft
	Building Width	W	115 ft
	Building Length	L	228 ft
	Approximate Floor Area	A	26220 ft ²
Table 9.1.3	Seismic Use Group	I	I
	Occupancy Importance Factor		1
Table 9.4.1.2.1	Site Classification		C
Figure 9.4.1.1a	0.2 Acceleration	S_s	1.480 g-s
Figure 9.4.1.1b	1 s Acceleration	S_1	0.500 g-s
Table 9.4.1.2.4a	Site Class Factor	F_a	1.000
Table 9.4.1.2.4b	Site Class Factor	F_v	1.300
	Adjusted Accelerations	S_{MS}	1.480 g-s
		S_{M1}	0.650 g-s
	Design Spectral Response Accelerations	S_{DS}	0.987 g-s
		S_{D1}	0.433 g-s
Table 9.4.2.1a/b	Seismic Design Category		D
	Equivalent Lateral Load Method can be used		

9.5.3.2 Seismic Base Shear Coefficient

N-S Direction			
Table 9.5.2.2	Response Modification Factor	R_{N-S}	5
Equation 9.5.3.2.1-1	Seismic Response Coefficient	$C_{s, N-S}$	0.197
Table 9.5.5.3.2		$C_{T, N-S}$	0.020
Table 9.5.5.3.2		x	0.750
	Period of Structure	T_{N-S}	0.423
Equation 9.5.3.2.1-2		$C_{S \max N-S}$	0.205
Equation 9.5.3.2.1-3		$C_{S \min}$	0.043
		$C_{s, N-S}$	0.197

E-W Direction			
Table 9.5.2.2	Response Modification Factor	R_{E-W}	5
Equation 9.5.3.2.1-1	Seismic Response Coefficient	$C_{s, E-W}$	0.197
Table 9.5.5.3.2		$C_{T, E-W}$	0.020
Table 9.5.5.3.2		x	0.750
	Period of Structure	T_{E-W}	0.423
Equation 9.5.3.2.1-2		$C_{S \max E-W}$	0.205
Equation 9.5.3.2.1-3		$C_{S \min}$	0.043
		$C_{s, E-W}$	0.197

Loading

Roof	
Dead	
Trusses	20 psf
Ceiling	5 psf
MEP	20 psf
Total	45 psf

Parking	
Dead	
Slab	100 psf
MEP	15 psf
Total	115 psf

Residential	
Dead	
Metal roof deck	2 psf
Concrete	30 psf
Ceiling	5 psf
MEP	15 psf
Total	52 psf

Live	
Moveable Partition ASCE 7-02 9.5.3.2)	10 psf

Perimeter Wall	
Dead	15 psf

Snow Load	25 psf
-----------	--------

Do not include because does not exceed 30 psf

$$w_{\text{roof}} = A \times (q_{\text{roof}}) + 2(W+L) \times .5 \times h_s \times q_{\text{wall}} \quad 1233.92 \text{ kips}$$

$$w_{\text{per floor}} = A \times (q_{\text{floor}}) + 2(W+L) \times h_s \times q_{\text{wall}} \quad 1733.69 \text{ kips}$$

$$w_{\text{all floors}} = (N-1) \times w_{\text{per floor}} \quad 8668.43 \text{ kips}$$

$$\text{Total Building Weight, } W \quad 9902.35 \text{ kips}$$

$$\text{Seismic Base Shear, } V_{N-S} = C_{s, N-S} W \quad 1954.06 \text{ kips}$$

$$\text{Seismic Base Shear, } V_{E-W} = C_{s, E-W} W \quad 1954.06 \text{ kips}$$

Vertical Distribution

N-S: $k_{N-S} = 1 + (T_{N-S} - 0.5) / (2.5 - 0.5) = 0.96$

Level	w_x (kips)	h_x (ft)	$w_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)
Roof	1233.9	65.5	68810.4	0.2	447.8
6	1733.7	51.5	76722.0	0.3	499.3
5	1733.7	41.0	61617.8	0.2	401.0
4	1733.7	30.5	46362.3	0.2	301.7
3	1733.7	20.0	30899.1	0.1	201.1
2	1733.7	10.0	15867.1	0.1	103.3
1					
Sum:	9902.3		300278.8	1.0	1954.1

NOTE: Seismic analysis is identical for N-S and E-W distribution

APPENDIX B

Controlling Lateral Case:

FIGURE 000.242.9024 • FOX / 04.327.3140

DETERMINING CONTROLLING LATERAL CASE (N-S)

WIND:

$$\begin{aligned}P_{\text{roof}} &= 9.44 \text{ psf} (5.5') + 8.84 \text{ psf} (1.5') = 65.2 \text{ plf} \\P_5 &= 8.28 \text{ psf} (3.75') + 8.84 \text{ psf} (8.5') = 106.2 \text{ plf} \\P_4 &= 7.61 \text{ psf} (4.25') + 8.28 \text{ psf} (6.25') = 84.1 \text{ plf} \\P_3 &= 7.58 \text{ psf} (4.75') + 7.61 \text{ psf} (5.75') = 79.8 \text{ plf} \\P_2 &= 7.58 \text{ psf} (10.25') = 77.7 \text{ plf} \\P_1 &= 7.58 \text{ psf} (10') = 75.8 \text{ plf} \\P_{\text{base}} &= 7.58 \text{ psf} (5') = 37.9 \text{ plf}\end{aligned}$$

* TO COMPARE WITH SEISMIC, CONVERT TO KIP FORCE*

$$\begin{aligned}P_{\text{roof}} &= 65.2 \text{ plf} (228') = 14.9 \text{ k} \\P_5 &= 106.2 \text{ plf} (228') = 24.2 \text{ k} \\P_4 &= 84.1 \text{ plf} (228') = 19.2 \text{ k} \\P_3 &= 79.8 \text{ plf} (228') = 18.2 \text{ k} \\P_2 &= 77.7 \text{ plf} (228') = 17.7 \text{ k} \\P_1 &= 75.8 \text{ plf} (228') = 17.3 \text{ k} \\P_{\text{base}} &= 37.9 \text{ plf} (228') = 8.6 \text{ k}\end{aligned}$$

FOR FACTORING LOADS, MUST CONSIDER LOAD COMBINATIONS. FROM ACI 9.2, POSSIBLE COMBINATIONS:

$$\begin{aligned}U &= 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W) \\U &= 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \\U &= 1.2D + 1.0E + 1.0L + 0.2S \\U &= 0.9D + 1.6W + 1.6H \\U &= 0.9D + 1.0E + 1.6H\end{aligned}$$

∴ ONLY NEED TO CONSIDER 1.6W OR 1.0E

BY INSPECTION, SEISMIC CONTROLS FOR N-S

WIND: (E-W)

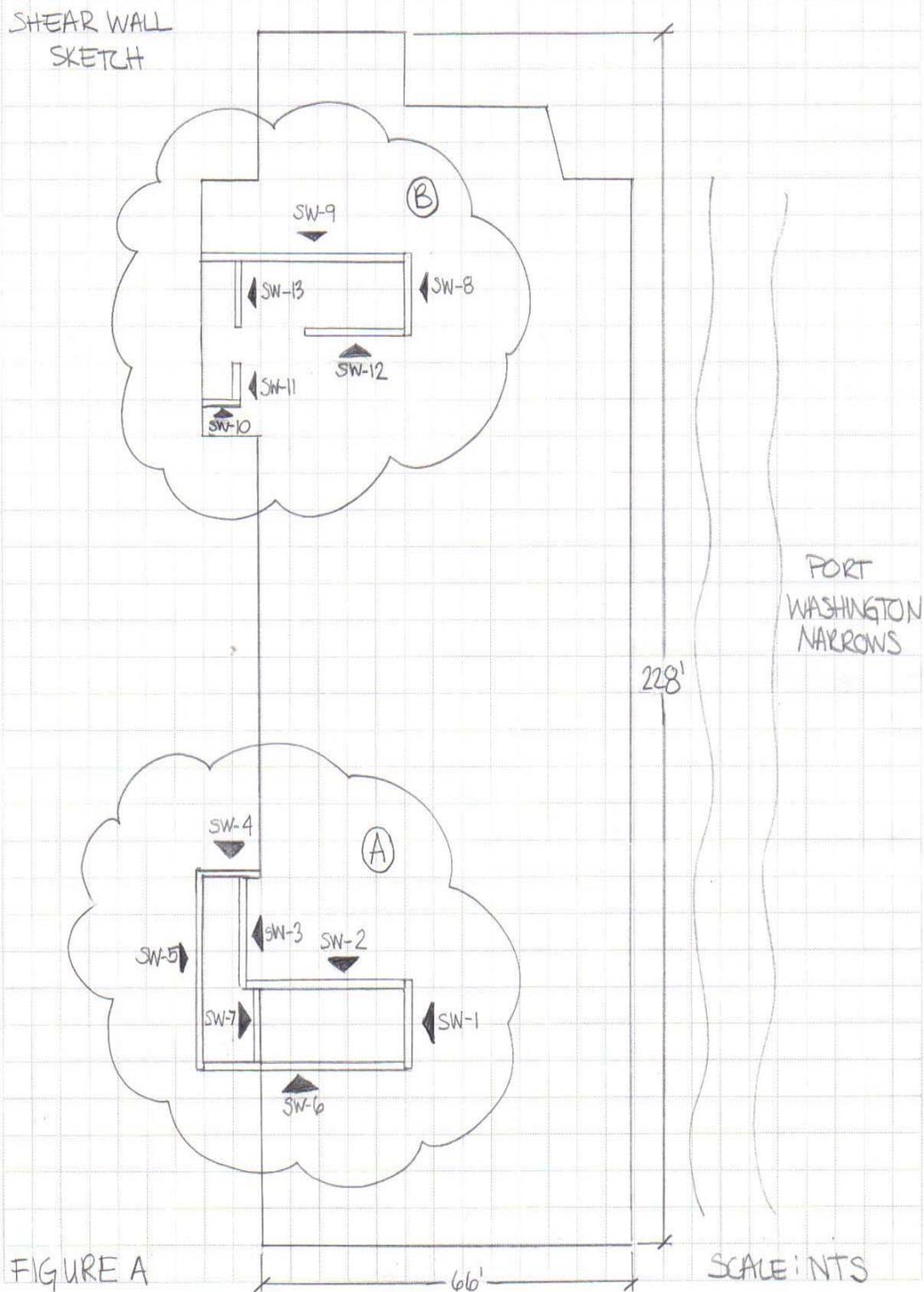
$$\begin{aligned}P_{\text{roof}} &= 9.24 \text{ psf} (5.5') + 8.65 \text{ psf} (1.5') = 63.8 \text{ plf} \\P_5 &= 8.11 \text{ psf} (3.75') + 8.65 \text{ psf} (8.5') = 103.9 \text{ plf} \\P_4 &= 7.45 \text{ psf} (4.25') + 8.11 \text{ psf} (6.25') = 82.4 \text{ plf} \\P_3 &= 7.42 \text{ psf} (4.75') + 7.45 \text{ psf} (5.75') = 78.1 \text{ plf} \\P_2 &= 7.42 \text{ psf} (10.25') = 76.1 \text{ plf} \\P_1 &= 7.42 \text{ psf} (10') = 74.2 \text{ plf} \\P_{\text{base}} &= 7.42 \text{ psf} (5') = 37.1 \text{ plf}\end{aligned}$$

* CONVERT TO KIP FORCE:

$$\begin{aligned}P_{\text{roof}} &= 63.8 \text{ plf} (114') = 7.3 \text{ k} \\P_5 &= 103.9 \text{ plf} (114') = 11.8 \text{ k} \\P_4 &= 82.4 \text{ plf} (114') = 9.4 \text{ k} \\P_3 &= 78.1 \text{ plf} (114') = 8.9 \text{ k} \\P_2 &= 76.1 \text{ plf} (114') = 8.7 \text{ k} \\P_1 &= 74.2 \text{ plf} (114') = 8.5 \text{ k} \\P_0 &= 37.1 \text{ plf} (114') = 4.2 \text{ k}\end{aligned}$$

BY INSPECTION, SEISMIC CONTROLS FOR E-W

∴ USE P FORCE FROM SEISMIC FOR LATERAL DESIGN



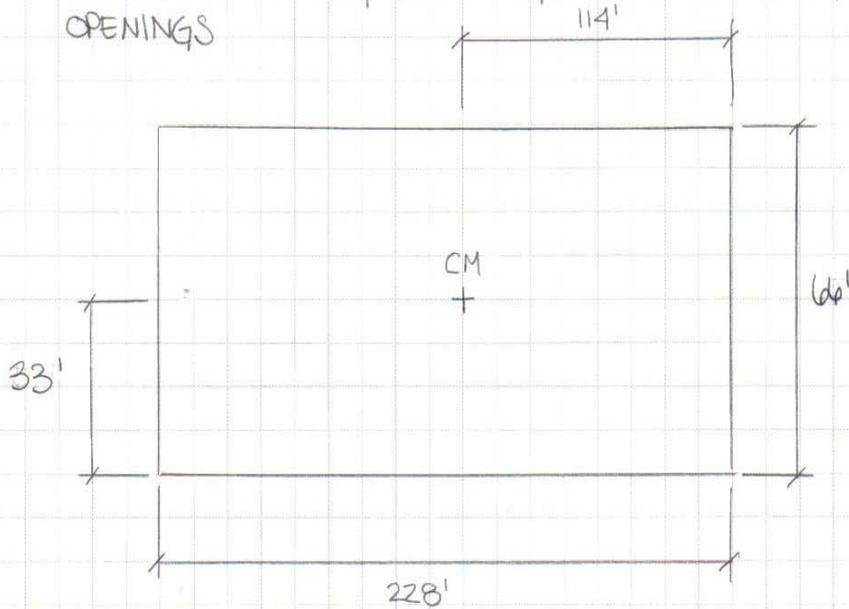
Note: While typical floorplan consists of similar protrusions, a rectangular floorplan for above floors (115 ft x 228 ft) is used for wind analysis

LATERAL LOAD DISTRIBUTION

- BUILDING < 7 STORIES
- ASSUME 1 TYPE OF LATERAL SYSTEM
- ASSUME RIGID DIAPHRAGM

DETERMINE THE CENTER OF MASS (CM)

- ASSUME BUILDING RECTANGULAR WITH SYMMETRICAL OPENINGS



DETERMINE STIFFNESSES OF EACH WALL / FRAME (SEE FIGURE A FOR LOCATION AND NOTATION)

NOTE: HEIGHT OF EACH SHEAR WALL IS 61.75'

* REFER TO TABLE 4 FOR VALUES *

DETERMINE THE CENTER OF RIGIDITY (CR)

$$X_{SW-2} = 50'$$

$$X_{SW-4} = 66'$$

$$X_{SW-6} = 40'$$

$$X_{SW-9} = 180'$$

$$X_{SW-10} = 162'$$

$$X_{SW-12} = 172'$$

$$Y_{SW-1} = 34'$$

$$Y_{SW-3} = 58'$$

$$Y_{SW-5} = 66'$$

$$Y_{SW-7} = 54'$$

$$Y_{SW-8} = 34'$$

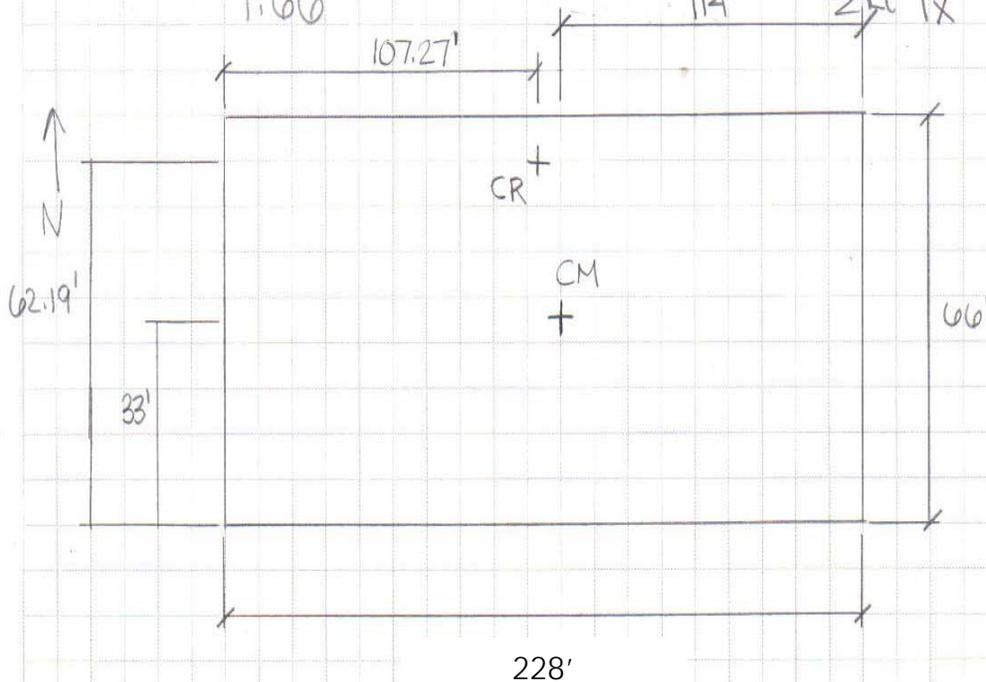
$$Y_{SW-11} = 57'$$

$$Y_{SW-13} = 58'$$

* REFER TO TABLE 4 FOR SUMMARY OF K, X, AND Y VALUES *

$$X_{CR} = \frac{507.40}{4.73} = 107.27 \text{ FEET} = \frac{\sum(K_i X_i)}{\sum K_i} \Big|_y$$

$$Y_{CR} = \frac{103.24}{1.66} = 62.19 \text{ FEET} = \frac{\sum(K_i Y_i)}{\sum K_i} \Big|_x$$



DETERMINE ECCENTRICITIES:

$$e_x = X_{cm} - X_{cr} = 114' - 107.27' = 6.73'$$

$$e_y = Y_{cm} - Y_{cr} = 33' - 62.19' = -29.19'$$

* NEGLECT ACCIDENTAL TORSION *

DETERMINE COORDINATES FOR EACH WALL / FRAME FROM CR TO PERPENDICULAR WALL / FRAME - d_i

NOTE: CR IS NOW (0,0) ON NEW COORDINATE SYSTEM

* REFER TO TABLE 4 FOR CALCULATION OF d_i 'S *

DETERMINE THE POLAR MOMENT OF INERTIA

$$J = \sum (k_i d_i^2) = 21680.75$$

* REFER TO TABLE 4 FOR CALCULATION AND SUM *

DETERMINE DIRECT SHEAR IN EACH FRAME / WALL IN X-DIRECTION (IF LATERAL FORCE IN X-DIRECTION)

$$F_{i\text{direct}} = \frac{k_i}{\sum k_i} P_x$$

* REFER TO TABLE 4 FOR CALCULATION *

DETERMINE DIRECT SHEAR IN FRAME/WALL IN Y-DIRECTION (IF LATERAL FORCE IS IN Y-DIRECTION)

NOTE: BASE SHEARS ARE IDENTICAL FOR N-S & E-W, SO $F_{c,direct}$ ARE THE SAME TOO

$$F_{c,direct} = \frac{k_i}{\sum k_i} P_y$$

REFER TO TABLE 4 FOR CALCULATION

DETERMINE TORSIONAL MOMENT

$$M_t = P_y * e_x + P_x * e_y$$

REFER TO TABLE 4 FOR CALCULATION

DETERMINE TORSIONAL SHEAR IN EACH FRAME/WALL

$$F_{torsion} = \frac{k_{idc}}{J} M_t$$

REFER TO TABLE 4 FOR CALCULATION

DETERMINE THE TOTAL SHEAR IN EACH FRAME/WALL

$$F_c = F_{c,direct} + F_{torsion}$$

NOTE: NEGLECT TORSIONAL EFFECT ($F_{torsion}$) IF IT REDUCES THE LOAD TAKEN BY THE WALL