WISCONSIN PLACE RESIDENTIAL

TECHNICAL ASSIGNMENT 1

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STRUCTURAL OPTION

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Executive Summary

Wisconsin Place Residential consists of 15 above stories and 2 below grade stories. The building is approximately 479,000 SF, stretching from 25 feet below grade to 142 feet above grade. The building consists of 432 units spread out over the 15 floors. The 13^{th} floor contains a 1,000 SF pool for all tenants of the building. The two levels below grade are set aside for residential parking and are integrated with the parking for the mixed use development.

This report introduces the structural conditions throughout a detailed description of the foundation, floor system, columns, and lateral systems. A preliminary analysis of seismic and wind lateral forces was researched along with spot checks of typical floor framing elements in gravity load areas as well as a simplified check of a typical shear wall.

ASCE 7-05 was used to determine all wind and seismic loads. The wind analysis was computed using the analytical procedure and the seismic design loads were computed using the equivalent lateral force procedure.

After completing the seismic and wind analysis I found the maximum base shear of the building to be controlled by the wind in the N-S direction (770 kips) and the maximum moment due to the seismic loads to control (67,171 ft-kips). The seismic forces distributed throughout the floors produced the largest force into the slabs (89 kips). Even though the seismic produced the largest forces overall, the wind produced much higher forces in the lower levels and gradually increased to the top of the building, whereas the seismic forces were low at the lower levels. All of these results are a preliminary analysis to gain a basic understanding of what is happening with the structure of the building and how the lateral forces are distributed throughout the floors.

Structural Systems

Foundations

The foundation shall be supported on spread footings. Column and wall footings supported by rock shall be designed for a bearing pressure of 40,000 PSF. A 4-inch gravel base shall be provided below floor slabs as a moisture barrier. Also, under-floor sub-drainage system shall be installed. All exterior footings shall be a minimum of 2'-6" below grade. All controlled compacted fill shall be compacted to not less than 95% of the maximum dry density determined in accordance with ASTM D-698.

Floor Systems

1st Floor:

Slab on grade.

2nd - 12th Floor:

Flat plate 7 ½" thick unbounded post-tension slabs, with a two-way bottom reinforcement mat of #4@24" continuous bars each way. Hooked bars at discontinuous ends are provided along with 2 #5 top and bottom additional bars along free slab edges. Concrete for slabs shall be normal weight concrete at 5000 psi. The post-tension cables consist of uniform tendons being pulled in the S-N direction and the banded tendons are in the pulled in the W-E direction of the building. The typical uniform cables are 15.0 klf and the banded cables range from approximately 50 - 400 kips.

13th Floor:

Floors are typically post-tensioned the same as the 2nd - 12th except in the pool area. The 12" and 15" slab areas require #5@24" O.C. each way continuous on top and bottom. The 23" slab area requires #6@12" O.C. each way continuous on top and bottom.

Pool House Roof:

7” slab with normal weight concrete and 60,000 psi reinforcing steel. A top and bottom mat of #4@12” O.C. continuous each way is required. Additional top reinforcing for column and middle strips is 6#5 top bars.

14^{th} and 15^{th} Floors:

Floors are typically post-tensioned the same as the 2^{nd} - 12^{th}.

Main Roof:

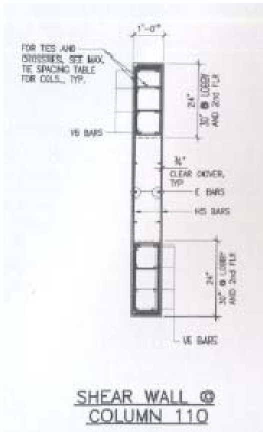
Slab is 8” thick unbounded post tensioned with a two-way bottom reinforcement of #4@24” continuous each way. For the 10” and 12” thick areas, #5@24” continuous mats are required as well as 2 #6 top and bottom additional bars along free slab edges.

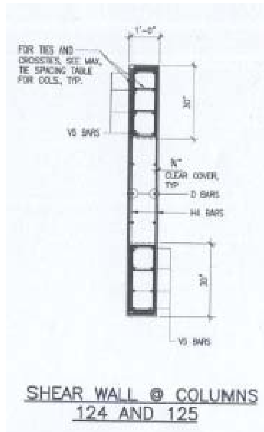
Columns

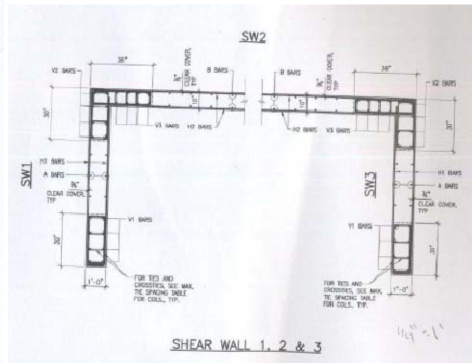
The columns in Wisconsin Place Residential are primarily standard reinforced concrete with varying sizes, shape, and reinforcement depending on their location and loads that are applied throughout the building. The most typical shapes are 16”x28” and 16”x32”. The reinforcement for the columns varies from floor to floor. The typical reinforcement is 8#7 or 8#8 bars, but varies throughout typical levels. The 12^{th} – 13^{th} floor reinforcement is typically #10 or #11 bars, due to the fact that they are supporting the pool. The loads vary greatly from column to column and are as large as 1380k and as small as 122k for dead loads and 293k to 17k for live loads at the top of the pad.

Lateral System

Concrete shear walls make up the buildings lateral load resisting system. Two elevator cores serve as the main components of these elements and are connected from the 1^{st} Floor to the roof. There are also three other shear walls spread out on the west side of the building. Typically the shear wall reinforcement is #4@12" for horizontal reinforcement and #6 or #7 bars for vertical reinforcement. The typical reinforcement for ties and crossties cooresponds to the maximum spacing for columns.







Codes and Code Requirements

Codes Used for the building

The structural design of Wisconsin Place Residential used various codes for gravity and lateral load conditions. Some of the codes used are the "ACI 318-02 Building Code Requirements for Structural Concrete", "ASCE 7-02", and the 2003 International Building Code.

Codes Used for this Report

All of the information that I computed throughout this report took in consideration the most up-to-date codes. ACI 318-05, ASCE 7-05, and the 2006 IBC.   Also I referenced the "Design of Concrete Structures" 13th edition by Nilson, Darwin, and Dolan for structural designs.

Deflection Criteria

Maximum deflection of studs in exterior walls subject to wind shall be L/600 when used as a backup for masonry. For other materials, maximum deflection shall be L/360. Floor deck deflection shall not exceed L/360 under full live and superimposed loads. Dead load and a 20 PSF construction live load, shall not exceed L/180.

Materials

Concrete:

a. Formed Slab and Beams	5000psi
b. Columns & Shear Walls (1^{st} – 6^{th})	6000psi
Columns & Shear Walls (6^{th} – Roof)	5000psi
c. Concrete Topping on Metal Deck	3500psi
d. Walls and Piers	4000psi
e. Pea-Gravel Concrete (Grout)	2500psi
f. All Other Concrete	3000psi

Reinforcing Steel:

a. Rebar Steel	ASTM A615, Grade 60
b. Welded Wire Fabric	ASTM A-185
c. Reinforcing Bar Mats	ASTM A-184
d. Reinforcing bars in Balconies	ASTM A-775

Metal Deck

a. Welding at Supports	5/8” Diameter puddle welds at alternate ribs
b. Floor Deck	2” Deep and a minimum of 20 Gage Steel

Masonry

a. Concrete	Block ASTM C-90
b. Brick	ASTM C-62 & ASTMC-216
c. Mortar	ASTM C270

Gravity and Lateral Loads

The gravity and lateral loads were determined in accordance with ASCE 7-05. Live Loads were established using section 4 of ASCE 7-05. General assumptions for dead loads were made based on unit weights from ASCE 7-05. Instead of calculating every column and wall, I assumed an addition 10 PSF load on each floor.

Dead Loads:

<u>Construction Dead Loads:</u>

Concrete	150 PCF

<u>Superimposed Dead Loads:</u>

Partitions	20PSF
Finishes & Miscellaneous	5 PSF
MEP	10 PSF
Columns & Walls	10 PSF
Shear Walls	(SEE APPENDIX A)

Live Loads:

Floors Including Partition Load	60 PSF
Canopy	75 PSF
Slab-On-Grade	100 PSF
Storage	125 PSF
Public Rooms and Corridors	100 PSF
Balconies	100 PSF
Lobby, Corridors, Stairs and Pool Areas	100 PSF
Penthouse, Mechanical Room	150 PSF
Elevator Machine Room	125 PSF
Roof	30 PSF
Roof Snow Load	27 PSF

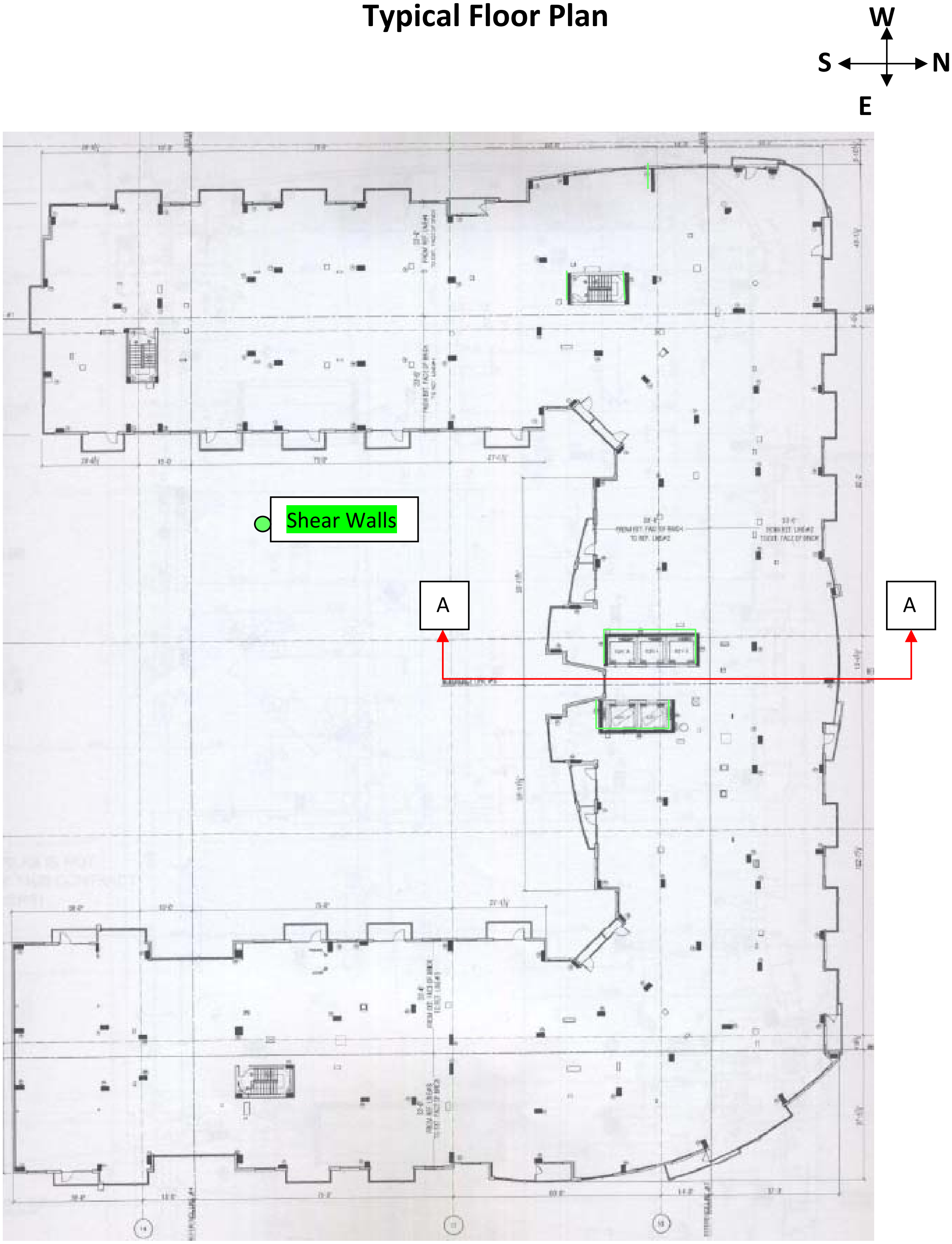
Lateral Loads

A summary of both wind and seismic load analyses are in the following section. Please refer to Appendices A and B for a more detailed description of wind and seismic procedures.

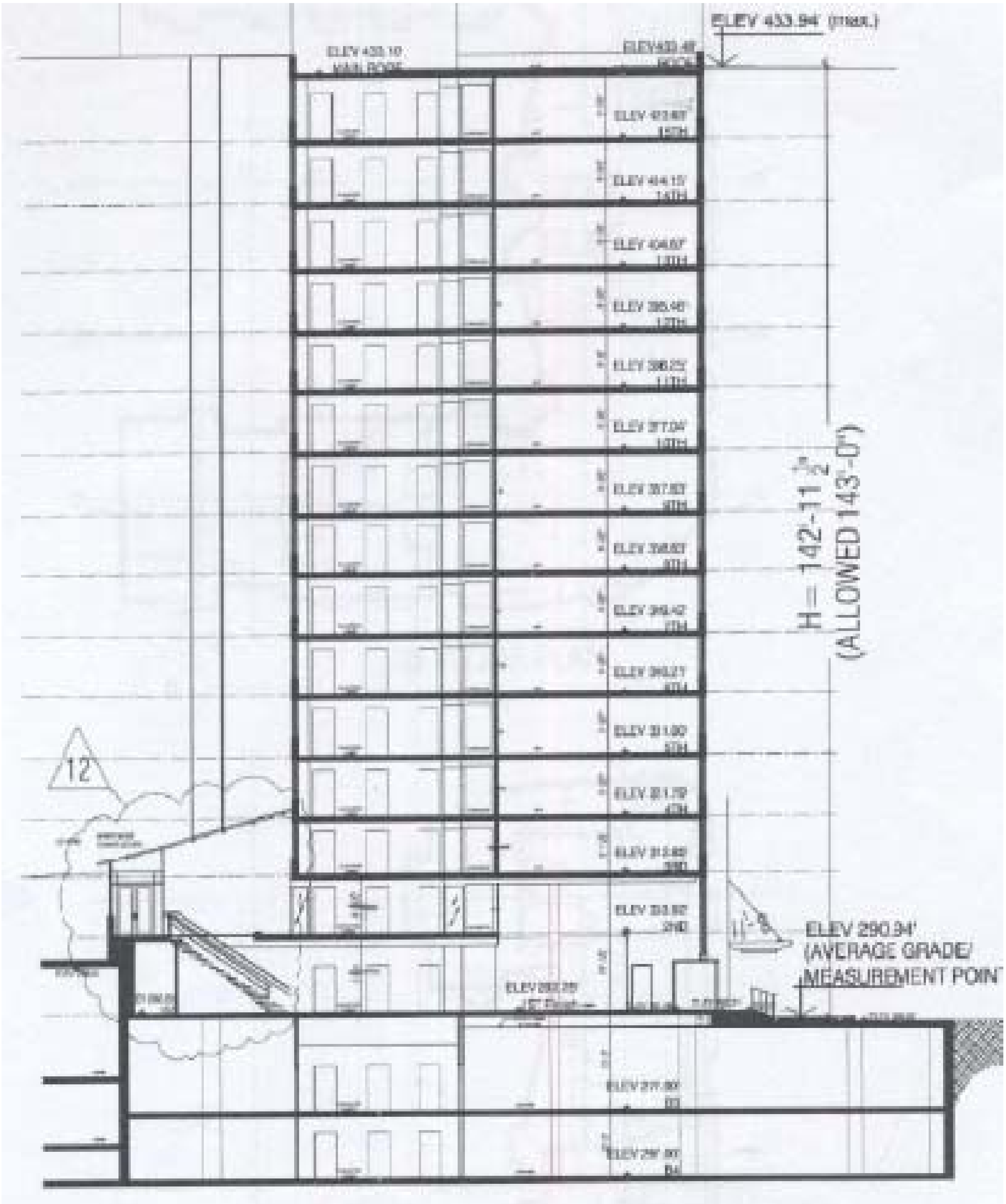
Wind

Wind loads were analyzed using section 6 of ASCE 7-05. For ease of the analysis I assumed the building to be a rectangle instead of a U-shape. I also ignored the cut-backs in different elevations of the building because they only occurred on one side of the U-shape. Therefore the wind would still be affecting either the East or West side regardless. I also did not take in consideration the Penthouse above the Main Roof or the canopy, because it would contribute minimally. The building also had many curves and undulations, so I assumed them to be straight. As you can see in the picture below of my actual building floor plan, there is a court yard formed within the U-shape. Please note that if a wind tunnel test was performed the cladding report would most likely show higher wind pressures on the South Elevation of the building because the legs of the “U” would act like a funnel, thus causing higher wind speeds which would increase pressures. Another assumption I made was that throughout my calculations I took the worst case of the K_z factor that applied to each floor because there were as many as three different K_z factors applying to each level and I was being conservative.

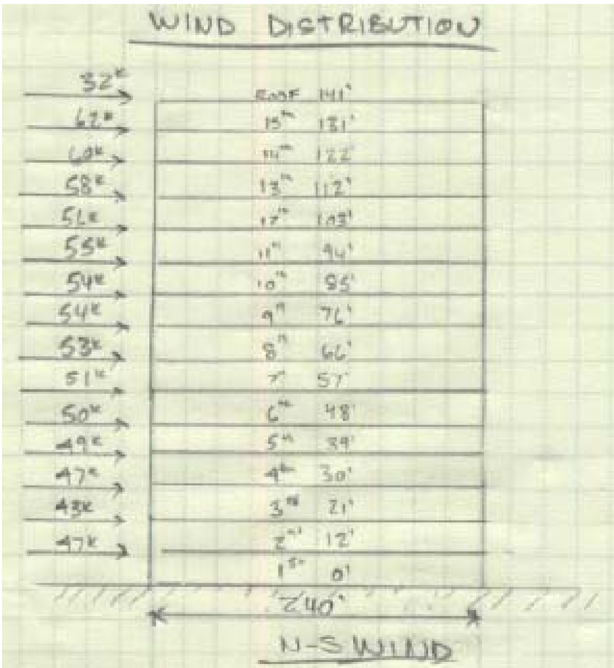
Typical Floor Plan



Building Section A-A



Factors Used in Wind Distribution Spread Sheet



Approximate Fundamental Period	0.819			
Topographic Factor K_{zt}	1			
Wind Directionality Factor K_d	0.85			
Basic Wind Speed V (mph)	90			
N-S Length of Building	240			
E-W Length of Building	296			
No. of Stories	15			
Typ. Story Height (ft)	9.21			
Building Height (ft)	141.23			
L/B in N-S Direction	0.81			
L/B in E-W Direction	1.23			
h/L in N-S Direction	0.588			
h/L in E-W Direction	0.477			
	$C_{p,\ windward}$	$C_{p,\ leeward}$	$C_{p,\ side\ wall}$	Gust Factor
N-S Direction	0.8	-0.5	-0.7	0.85
E-W Direction	0.8	-0.454	-0.7	0.85

Level	FL-FL Height	h_x	K_z	q_z	Pressures (PSF) N/S Wind Windward	Leeward	Sidewall	E/W Wind Windward	Leeward	Sidewall	Load (kips) N/S	E/W	Shear (kips) N/S	E/W	Moment (ft-k) N/S	E/W	Internal Pressure (PSF) +	-
Roof	9.85	141.23	1.13	19.92	13.54	-8.46	-11.85	13.54	-7.69	11.85	32	25	0	0	4,536	3,548	25	-25
15	9.48	131.38	1.09	19.21	13.06	-8.46	-11.85	13.06	-7.69	11.85	62	48	32	25	8,096	6,327	24	-24
14	9.48	121.9	1.09	19.21	13.06	-8.46	-11.85	13.06	-7.69	11.85	60	47	94	73	7,364	5,755	22	-22
13	9.21	112.42	1.04	18.33	12.46	-8.46	-11.85	12.46	-7.69	11.85	58	45	154	120	6,512	5,083	20	-20
12	9.21	103.21	1.04	18.33	12.46	-8.46	-11.85	12.46	-7.69	11.85	56	44	212	165	5,799	4,527	19	-19
11	9.21	94	0.99	17.45	11.87	-8.46	-11.85	11.87	-7.69	11.85	55	43	268	209	5,131	4,001	17	-17
10	9.21	84.79	0.96	16.92	11.51	-8.46	-11.85	11.51	-7.69	11.85	54	42	323	252	4,546	3,542	15	-15
9	9.21	75.58	0.96	16.92	11.51	-8.46	-11.85	11.51	-7.69	11.85	54	42	376	294	4,052	3,158	14	-14
8	9.21	66.37	0.93	16.39	11.15	-8.46	-11.85	11.15	-7.69	11.85	53	41	430	335	3,494	2,721	12	-12
7	9.21	57.16	0.89	15.69	10.67	-8.46	-11.85	10.67	-7.69	11.85	51	40	483	376	2,936	2,284	10	-10
6	9.21	47.95	0.85	14.98	10.19	-8.46	-11.85	10.19	-7.69	11.85	50	39	534	416	2,401	1,866	9	-9
5	9.21	38.74	0.81	14.28	9.71	-8.46	-11.85	9.71	-7.69	11.85	49	38	584	455	1,890	1,467	7	-7
4	8.93	29.53	0.76	13.40	9.11	-8.46	-11.85	9.11	-7.69	11.85	47	37	633	493	1,393	1,080	5	-5
3	8.93	20.6	0.66	11.63	7.91	-8.46	-11.85	7.91	-7.69	11.85	43	33	680	530	892	689	4	-4
2	11.67	11.67	0.57	10.05	6.83	-8.46	-11.85	6.83	-7.69	11.85	47	36	723	563	544	419	2	-2
Totals											770	599	770	599	59,587	46,464	205	-205

Seismic

Weight of the Building

Floor	Net Floor Area (SF)	Loads (PSF)	Weight of Shear Walls (kips)	Dead Load (excluding Shear Walls) (kips)	Total Dead Load (kips)
2	22,988	139	122	3,195	3,317
3	30,510	139	105	4,241	4,346
4	30,507	139	107	4,240	4,347
5	40,789	139	107	5,670	5,777
6	40,789	139	107	5,670	5,777
7	40,789	139	107	5,670	5,777
8	33,283	139	107	4,626	4,733
9	32,974	139	107	4,583	4,690
10	32,980	139	107	4,584	4,691
11	32,980	139	107	4,584	4,691
12	32,980	139	107	4,584	4,691
13	23,329	139	110	3,243	3,353
13	2,500	233	0	583	583
14	25,373	139	112	3,527	3,639
15	25,373	139	114	3,527	3,641
Roof	25,373	145	58	3,679	3,737
Sum	473,517		1584	66,206	67,790

Factors Used in Seismic Distribution Spread Sheet

Site Classification	B
$S_{s\ (g)}$	0.154
$S_{1\ (g)}$	0.05
F_a	1
F_v	1
$S_{ms\ (g)}$	0.154
$S_{m1\ (g)}$	0.05
$S_{DS\ (g)}$	0.103
$S_{D1\ (g)}$	0.033
R	4.5
Seismic Importance factor	1
Occupancy Category	II
Seismic Design Category	A
T (s)	1.39
$T_{L\ (s)}$	8
C_S	0.01
$h_{n\ (ft)}$	141.23
V (k)	678
M (ft-kips)	67,171

Seismic Distribution

						Load	Shear	Moment
Level	w_x	FL-FL Height	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)	(ft)			(kips)	(kips)	(ft-kips)
Roof	3,737	9.85	141.23	4,970,121	0.131	89	0	12,559
15	3,641	9.48	131.38	4,359,570	0.115	78	89	10,248
14	3,639	9.48	121.9	3,907,918	0.103	70	167	8,523
13	3,936	9.21	112.42	3,757,775	0.099	67	237	7,558
12	4,691	9.21	103.21	3,955,516	0.104	71	304	7,304
11	4,691	9.21	94	3,453,188	0.091	62	375	5,808
10	4,691	9.21	84.79	2,972,694	0.078	53	437	4,510
9	4,690	9.21	75.58	2,514,769	0.066	45	490	3,401
8	4,733	9.21	66.37	2,101,172	0.055	38	535	2,495
7	5,777	9.21	57.16	2,064,227	0.054	37	572	2,111
6	5,777	9.21	47.95	1,599,146	0.042	29	609	1,372
5	5,777	9.21	38.74	1,173,001	0.031	21	638	813
4	4,347	8.93	29.53	594,953	0.016	11	659	314
3	4,346	8.93	20.6	352,484	0.009	6	670	130
2	3,317	11.67	11.67	117,815	0.003	2	676	25
Totals				37,894,348	1.00	678	**678**	**67,171**

Conclusion

After completing the seismic and wind analysis I found the maximum base shear of the building to be controlled by the wind in the N-S direction (770 kips) and the maximum moment due to the seismic loads to control (67,171 ft-kips). The seismic forces distributed throughout the floors produced the largest force into the slabs (89 kips). Even though the seismic produced the largest forces overall, the wind produced much higher forces in the lower levels and gradually increased to the top of the building, whereas the seismic forces were low at the lower levels. I was very surprised that the wind controlled over seismic, because the building is so heavy. All of these results are a preliminary analysis to gain a basic understand of what is going on with the structure of the building and how the lateral forces are distributed throughout the floors.

Appendix A

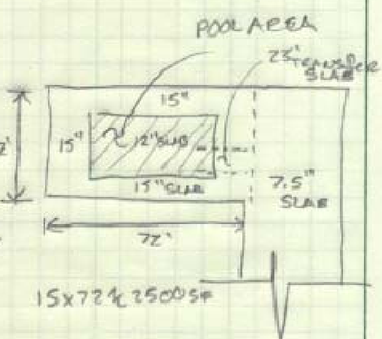
DEAD LOAD CALCULATIONS

TYPICAL LEVEL $2^{nd} - 12^{th}$ & $14^{th} - 15^{th}$

PARTITIONS	20PSF
MISC & FINISHES	5 PSF
MEP	10 PSF
7½" SLAB	94 PSF
COLUMNS & WALLS	10 PSF
	139PSF

13^{th} LEVEL (AREA SUPPORTING POOL)

PARTITIONS	20PSF
MISC & FINISHES	5PSF
MEP.	10PSF
ASSUME 15" SLAB (CONSERVATIVE)	186 PSF
COLS & WALLS	10PSF
	233PSF

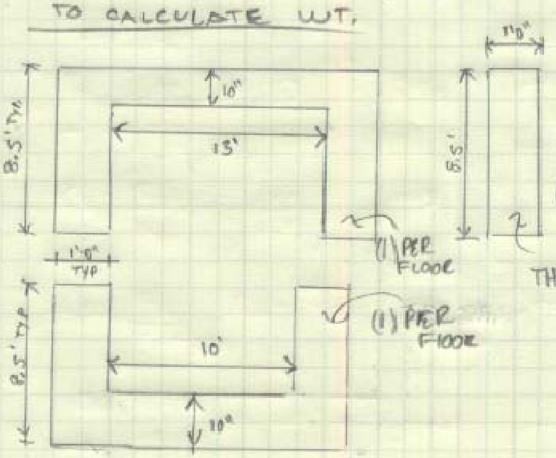


15x72 ≈ 2500 SF

ROOF LEVEL

PARTITIONS	20PSF
MISC & FINISHES	5PSF
MEP	10PSF
8" SLAB	100PSF
COLS + WALLS	10PSF
	145PSF

SHEARWALL DIMENSIONS USED TO CALCULATE WT.



IN ORDER TO CALCULATE WEIGHT OF SHEAR WALLS I SAID THERE ARE (7) - (8.5' x 1'0") PIECES AND 1 = 13'(10") PIECE, 1 - 10'(10") PIECE

THERE ARE (3) OF THESE PER FLOOR

WEIGHT OF SHEAR WALLS

2rd FLOOR

$(8.5ft \times 1.0ft) = 8.5ft^2 \times \underbrace{\left(\frac{11.67}{2} + \frac{8.93}{2}\right)}_{10.3'} = 87.55ft^3 \times 150lb/ft^3 = 13.13^k$

$(13')(10/12') = 10.83ft^2 \times 10.3' = 111.55ft^3 \times 150lb/ft^3 = 16.73^k$

$(10')(10/12) = 8.33ft^2 \times 10.3' = 85.80ft^3 \times 150lb/ft^3 = 12.87^k$

$$\text{TOTAL} = (7) \times 13.13 = 91.91 + 16.73 + 12.87 = \boxed{121.51^k}$$

3rd FLOOR

$8.5ft^2 \times \underbrace{\left(\frac{8.93}{2} + \frac{8.93}{2}\right)}_{8.93} = 75.91ft^3 \times 150lb/ft^3 = 11.39^k$

$10.83ft^2 \times (8.93) = 96.71ft^3 \times 150lb/ft^3 = 14.51^k$

$8.33ft^2 \times (8.93) = 74.39ft^3 \times 150lb/ft^3 = 11.16^k$

$$\text{TOTAL} = (7) \times 11.39 = 79.70 + 14.51 + 11.16 = \boxed{105.37^k}$$

4th - 12th FLOOR

$8.5ft^2 \times \underbrace{\left(8.93/2 + \frac{9.21}{2}\right)}_{9.07} = 77.10ft^3 \times 150lb/ft^3 = 11.56^k$

$10.83ft^2 \times 9.07 = 98.23ft^3 \times 150lb/ft^3 = 14.73^k$

$8.33ft^2 \times 9.07 = 75.55ft^3 \times 150lb/ft^3 = 11.33^k$

$$\text{TOTAL} = (7) \times 11.56 = 80.92 + 14.73 + 11.33 = \boxed{106.98^k}$$

13th FLOOR

$$8.5\,ft^2 \times \overbrace{\left(\frac{9.21}{2}+\frac{9.48}{2}\right)}^{9.35} = 79.43\,ft^3 \times 150\,lb/ft^3 = 11.91^k$$

$$10.83\,ft^2 \times (9.35) = 101.26\,ft^3 \times 150\,lb/ft^3 = 15.19^k$$

$$8.33\,ft^2 \times (9.35) = 77.89\,ft^2 \times 150\,lb/ft^3 = 11.68^k$$

$$\text{TOTAL} = (7) \times 11.91 = 83.37 + 15.19 + 11.68 = \boxed{110.24}$$

14th FLOOR

$$8.5\,ft^2 \times \overbrace{\left(\frac{9.48}{2}+\frac{9.48}{2}\right)}^{9.48} = 80.58\,ft^3 \times 150 = 12.09$$

$$10.83 \times 9.48 = 102.67\,ft^3 \times 150\,lb/ft^3 = 15.4$$

$$8.33 \times 9.48 = 78.97\,ft^3 \times 150\,lb/ft^3 = 11.85$$

$$\text{TOTAL} = (7) \times 12.09 = 84.63 + 15.4 + 11.85 = \boxed{111.88^k}$$

15th FLOOR

$$8.5\,ft^2 \times \overbrace{\left(\frac{9.48}{2}+\frac{9.85}{2}\right)}^{9.67} = 82.20\,ft^3 \times 150\,lb/ft^3 = 12.33$$

$$10.83 \times 9.67 = 104.73\,ft^3 \times 150\,lb/ft^3 = 15.71$$

$$8.33 \times 9.67 = 80.55\,ft^3 \times 150\,lb/ft^3 = 12.08$$

$$\text{TOTAL} = (7) \times 12.33 = 86.31 + 15.71 + 12.08 = \boxed{114.10^k}$$

ROOF

$$8.5 \times \overbrace{\left(\frac{9.85}{2}\right)}^{4.93} = 41.86\,ft^3 \times 150\,lb/ft^3 = 6.279$$

$$10.83 \times 4.93 = 53.39\,ft^3 \times 150\,lb/ft^3 = 8.00$$

$$8.33 \times 4.93 = 41.07\,ft^3 \times 150\,lb/ft^3 = 6.16$$

$$\text{TOTAL} = (7) \times 6.279 = 43.95 + 8.0 + 6.16 = \boxed{58.11^k}$$

Appendix B

SEISMIC DESIGN

BUILDING ON SITE CLASS B IS 141.23'

LATITUDE: 38.96
LONGITUDE: -77.09

$S_s = .154_g$ $\quad$ $T_L = 8\,sec$, CHEVY CHASE MARYLAND
$S_1 = .050$

$F_a = 1.0$
$F_v = 1.0$

$S_{MS} = F_a \cdot S_s = 1.0(.154) = .154g$
$S_{M1} = F_v \cdot S_1 = 1.0(.050) = .050g$

$S_{DS} = 2/3\, S_{MS} = 2/3(.154) = .103g$
$S_{D1} = 2/3\, S_{M1} = 2/3(.050) = .033g$

DUAL SYSTEMS W/ INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES

$R \Rightarrow 4\frac{1}{2}$

IMPORTANCE FACTOR, $I = 1.0$
OCCUPANCY CATEGORY = II

SDC = A

$V_b = C_s \cdot W$

$T_a = C_t\, h_n^{x}$

$T_a = (.02)(141.23)^{(.75)} = .819s$

$T = C_u \cdot T_a = 1.7(.819) = 1.39s$

$C_s = \dfrac{S_{DS}}{R/I} = \dfrac{.103}{4.5/1.0} = .023$

$C_s = \dfrac{S_{D1}}{T(R/I)}$ for $T \le T_L$ $= \dfrac{.033}{1.39(4.5/1)} = .005$

$C_s \ge \dfrac{S_{D1} \cdot T_L}{T^2(R/I)} \Rightarrow$ NOT APPLICABLE

min $C_s = .01$ therefore

$C_s = .01$

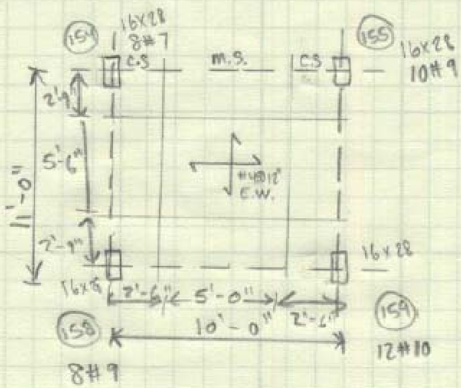
$V_b = .01(67{,}790^k)$

$\boxed{V_b = 678^k}$

Appendix C

FLAT PLATE SPOT CHECK

3rd FLOOR WEST FRAMING PLAN



7½" THICK SLAB
#4@24 E.W TOP & BOT
2#5 Additional AT SLAB EDGES
f'_c = 5'000 psi

LOADS

DEAD

PARTITIONS	20PSF
FINISHES & MISC	5PSF
MEP	10PSF
COLUMNS+WALLS	10PSF
7½ SLAB	94PSF
	139PSF

LIVE LOADS

RESIDENTIAL 40PSF

$1.4(139) = 194.6$

$W_u = 1.2(139) + 1.6(40) = 231\,PSF$ GOVERNS

MIN REINF. $= .0018\,A_g = .0018(12'')(7.5'') = .162\,in^2/ft$

$\#4 \rightarrow .20$ EACH WAY $.2 \times 2 = .4 > .162$ ✓OK

FRAMING IS FLATPLATE W/o EDGE BEAMS

DISTRIBUTION OF M_o

	INTERCONNECTED SPAN
M_{int}^-	.65
$M+$	.35

INTERIOR PANEL

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{.231(11')(11' - 16''/12)^2}{8} = 29.68'^k$$

LOCATION	STRIP		MOMENT	WIDTH	Mu/WIDTH
SUPPORT 0.65M_o	C_s	75%	14.47	5'-6"	2.63
	M_s	25%	4.83	5'-6"	.88
MIDSPAN 0.35M_o	C_s	60%	6.23	5'-6"	1.13
	M_s	40%	4.15	5'-6"	.75
			29.68 ✓		

COL. STRIP

$d = 7.5" - .75 - .3125" = 9.44"$

$$M_n = \frac{M_u}{\phi} = \frac{14.47}{.9} = 16'^k$$

$$R = \frac{M_n}{bd^2} = \frac{16'^k(12)}{(66)(9.44)^2} \times 1000 = 32.64\text{psi}$$

ρ table A.5a

$\rho = .0010$

$A_s = \rho bd = .0010(66)(9.44) = .623 in^2$

$A_{s_{min}} = .0020bt = .002(66)(7.5) = .99$ GOVERNS

$$N = \frac{A_s}{.31} = \frac{.99}{.31} = 3.19 \text{ bars} \quad 4 \text{ bars}$$

$$N_{min} = \frac{5.5'(12)}{2(7.5)} = 4.4 \text{ bars} \quad 5 \text{ bars}$$

ACTUAL DESIGN #4@24

MIDSPAN #4@24"

$M_n = \frac{6.23}{.9} = 6.92^{1k}$ $R = \frac{6.92(12)}{(66)(9.44)^2} = .014 \times 1000 = 14.11$

$\rho = .0005$ from tab 6A.5A

$A_s = .0005(66)(9.44) = .311 \text{ in}^2$

$A_{s_{min}} = .002bt = .002(66)(7.5) = .99$ - governs

$N = \frac{A_s}{.31} = .99/.31 = 3.19 \text{ bars} = 4 \text{ bars}$

MIDDLE STRIP

SUPPORT

$M_n = \frac{M_u}{\phi} = \frac{4.83}{.9} = 5.37^{k'}$ $R = \frac{M_n}{bd^2} = \frac{5.37(12)}{(66)(9.44^2)} \times 1000 = 10.95$

$\rho = .0005$

$A_s = \rho bd = (.0005)(66)\, 9.44 = .311 \text{ in}^2$

$A_{s_{min}} : .002bt = .002(66)(7.5) = .99$ governs

$N = \frac{.99}{.31} = 3.19 \text{ bars} = 4 \text{ bars}$

ACTUAL DESIGN

#4@24"

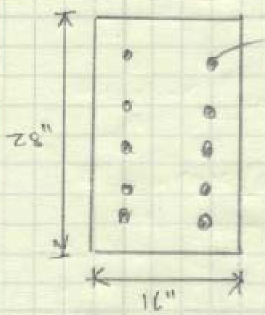
MIDSPAN

$M_n = \frac{M_u}{\phi} = 4.15 \Rightarrow$ SAME RESULT 4 bars.
BASED ON MIN N req'd.

COLUMN GRAVITY CHECK

@ LVL 1

ASSUME LOAD IS PURE AXIAL



COL. 153 10#10 bars

$DL = 796^k$
$LL = 181^k$
977

ASSUMING LOADS ARE FACTORED

$P_o = .85f'_c(bh - \Sigma A_s) + \Sigma A_s f_y$

$= .85(6)(16 \times 28 - 10(1.27)) + 10(1.27)(60)$

$2220 + 76.2 = 2296^k$

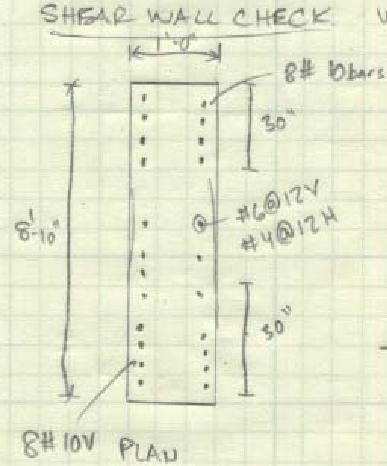
$\phi P_o = .65(2296) = 1492.53$

$.8\phi P_o = .8(1492.53) = 1194.02 > 977$

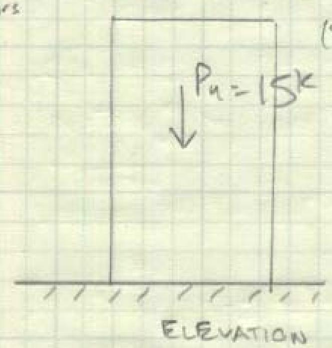
This COLUMN IS ADEQUATE ✓ OK

SHEAR WALL CHECK WORST CASE FOR WIND

SHEAR WALL TAKES E-W WIND



PLAN



ELEVATION

$(8.5 \times 1.0) \times 11.67' = 100 \times 15\$ = 15^k$

TOTAL $M_u = 419$ k'

TOTAL $V_u = 563$

SW@COL. 124+125

ASSUMING S.W. TAKES NO GRAVITY LOADING. $P_u = 15^k$ SELF WT.

A.) BE, NEEDED?

$f'_c > .2f'_c$

$A_g = (1.0)(8.83') = 8.83 ft^2$

$I_g = \frac{(1.0)(8.83)^3}{12} = 57.37 ft^4$

$\frac{P_u}{A_g} = \frac{15^k}{8.83} = 1.7^k/ft^2$

$\frac{M_u \frac{h_w}{2}}{I_g} = \frac{419\left(8.83/2\right)}{57.34} = 32.26 k/ft^2 / 144 = .224$

$32.26 + 1.7 = 33.96$ KSF

$.2f'_c = .2(6ksi) = 1.2 in$

$.224 < 1.2 in$ ∴ NO NEED FOR B.E.

B.) LONGITUDINAL & TRANSVERSE REINFORCEMENT

$V_n \geq 2 A_{cv} \sqrt{f'_c}$ ∴ need 2 curtains

$(2)(12'')(12'')(\sqrt{6}) = 405.5 > 563$

∴ 2 CURTAIN REQ'D,

$A_{cv} = 144\ in^2/ft$

$A_s\ long = (.0025)(144) = .36\ in^2/ft$

#4 HORIZONTAL

$A_{sl} = 2(.20) = .40\ in^2/s\text{-}space$

$$\frac{.36}{12} = \frac{.40}{S}$$

$S = 13.33 < 18''$ OK

NOMINAL SHEAR CAPACITY

$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_t f_y)$

$\frac{h_w}{l_w} > 2 \quad \alpha_c = 2.0$

$A_{cv} = (12'')(8.33 \times 13 \times 12)$

$= 923.37$

$$\rho_t = \frac{2(.2)}{(12)(12)} = .0028$$

$V_n = (923)(2\sqrt{6{,}000} + .0028 \times 60{,}000)/_{1000} = 226.56^k$

$\phi V_n = .6(226.56)^k = 135.94^k$

135.94 < 563 because there is MORE THAN ONE SW ON THE FLOOR, IT DEPENDS ON THE STIFFNESS OF THE WALL TO CALCULATE THE LOAD TRANSFERED. THERE ARE 7 OF THESE 8'-10'x1'-0" WALLS. ✓ OK.