Technical Assignment #1

Roosevelt Island Southtown Building No. 5, NY, NY

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Structural Option

10/5/07

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EXECUTIVE SUMMARY

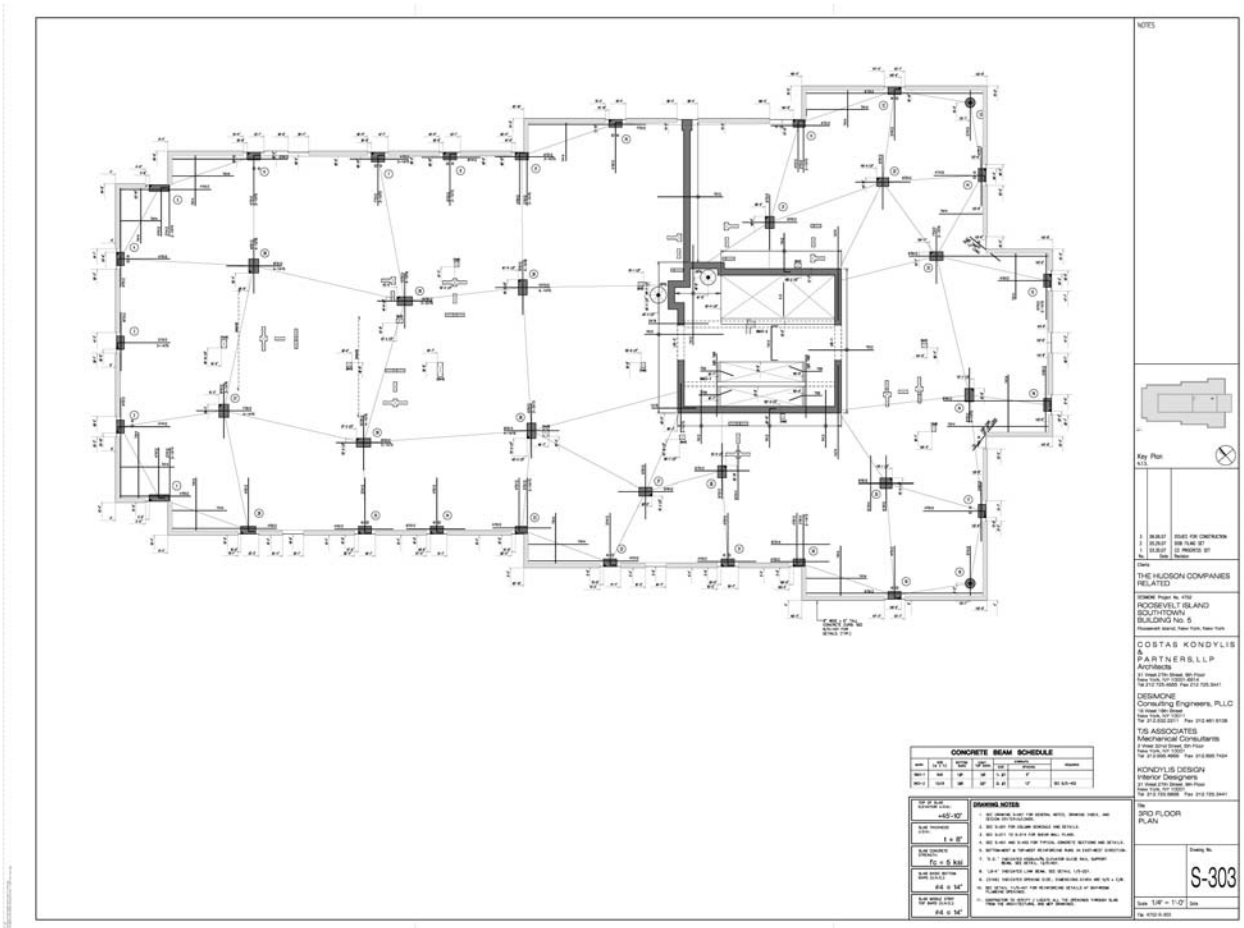
Southtown Building No. 5 is a luxury apartment building located in the center of Roosevelt Island in Manhattan's East River. It houses 123 apartments in 16 floors with an underground cellar which houses storage units as well as mechanical and electrical space. The building is the fifth out of nine apartment buildings in a development that is planning to revitalize the once industrial Roosevelt Island into a place in which people will live, work, and play. The apartment building also houses a full service lobby with concierge service and mail room, a health club, multi-purpose room, children's play area, party room, and rooftop terraces.

The purpose of this first technical report is to understand the existing conditions and design procedures for Roosevelt Island Southtown Building No. 5 in New York, NY. The structural systems of the building are introduced through detailed descriptions of the foundation, floor, column, and lateral systems. Preliminary design concepts were examined including an analysis of the lateral force resisting system, spot checks of gravity and lateral loads on the typical flooring system, columns, and shear wall.

To determine all wind and seismic loads, ASCE 7-05 was used in accordance with all reference standards from The Building Code of the City of New York. The appendix of this report contains the calculations performed to obtain loading conditions and detailed spot checks. A more complete and detailed set of calculations are available for further review upon request.

Panoramic View of Roosevelt Island as seen from Manhattan

TYPICAL FLOOR FRAMING PLAN



STRUCTURAL SYSTEMS

Foundations

Three types of foundation systems are used for Southtown Building No. 5. Individual footings are used for interior columns of the building. These footings range from 3'-0" x 3'-0" to 4'-6" x 4'-6". A mat footing is used at the base of the later force resisting shear walls. The mat is typically 42" thick with some step downs required for the elevator, boiler, and sump pits. Finally, a 12" thick foundation wall is used around the perimeter of the cellar. This system incorporates exterior concrete piers into the wall with footings at the base.

Floor System

The floor system of the apartment building is typically a 8" two-way normal weight concrete flat plate with varying size bays. At the cellar floor, a 6" concrete slab is used with W2.0 x W2.0 welded wire fabric. At the first floor, a 9" concrete slab is used to accommodate for higher occupancy loads. Typical reinforcement for the floor system is #4 @ 14" bottom steel and #4 @ 14" top steel. Middle strip reinforcement is used in the floor slab in some areas of higher stress. Due to the party room and lounge area on the main roof, a 10" concrete floor slab with #5 @ 12" top and bottom steel is used.

Columns

The columns in this New York building are typically rectangular reinforced concrete with varying sizes and reinforcement. The most common column size is 14" x 24" with 8 #6 bars as structural reinforcement. Column loads vary greatly within the building, especially as the elevation rises. The largest loads at the foundation level is 1056 kips of dead load and 139 kips of live load.

Lateral System

Reinforced concrete shear walls make up the lateral force resisting system of the building. The elevator and stairwell core in the center of the building have been assigned as the location of these shear walls. The shear walls rise from the cellar level of the building all the way to the elevator mechanical room. A 12" typical shear wall section consists of #4 @ 12" horizontal reinforcement and #5 @ 12" vertical reinforcement. Openings in the shear walls require link beams in order to transfer high shear forces from one side of the opening to the other.

CODES AND DESIGN REFERENCES

Codes and References

1. "Building Code of the City of New York"
2. "The New York City Seismic Code: Local Law 17/95"
3. "Code Requirements for Structural Concrete" (ACI 318-99), American Concrete Institute as modified by subchapter 10, article 5 of the N.Y.C Building Code
4. "Building Code Requirements for Masonry Structures" (ACI 530-99), American Concrete Institute as modified by reference standard RS 10 -18 of the N.Y.C. Building Code

MATERIALS

Cast in Place Concrete

Foundations: 4 ksi
Foundation Walls: 5 ksi
Slabs on Ground: 4 ksi
Formed Slabs: 5 ksi
Shear Walls:
  Cellar – 2nd Floor: 7 ksi
  3rd Floor – 8th Floor: 6 ksi
  9th Floor – EMR Roof: 5 ksi
Columns
  Cellar: 4 ksi for Buttress, 7 ksi for Columns
  1st Floor – 2nd Floor: 7 ksi
  3rd Floor – 8th Floor: 6 ksi
  9th Floor – EMR Roof: 5 ksi

Reinforcement

ASTM A615, Grade 60
Welded Wire Fabric: ASTM A185
Welded Deformed Wire Fabric: ASTM A467, Grade 60

Structural Steel

All Rolled Shapes: ASTM A572 or A992, Grade 50
All Plates and Connection Material: ASTM A36
All Tubular Sections: ASTM A500, Grade B
All Pipe Sections: A53, Grade B
Anchor Bolts: ASTM F1554

Welding Electrodes: E70XX Low Hydrogen

Bolting Materials: ASTM A325 or A490

Light Gage Framing:

16 Gage and Heavier: ASTM A653, Grade 50
18 Gage and Lighter: ASTM A653, Grade 33

GRAVITY AND LATERAL LOADS

Dead Loads

1. Construction Dead Load
    a. Cellar Floor: 75 psf
    b. 1^{st} Floor: 113 psf
    c. 2^{nd} -16^{th} Floor: 100 psf
    d. Main Roof: 113 psf
    e. Mechanical Room Floor: 100 psf
    f. E.M.R Floor: 100 psf
    g. E.M.R Roof: 100 psf
2. Superimposed Dead Load
    a. Cellar Floor: 25 psf
    b. 1^{st} Floor: 30 psf
    c. 2^{nd} -16^{th} Floor: 20 psf
    d. Main Roof: 50psf
    e. Mechanical Room Floor: 25 psf
    f. E.M.R Floor: 25 psf
    g. E.M.R Roof: 25 psf

Live Loads

1. Cellar Floor:
    a. Equipment Rooms: 100 psf
    b. Offices: 50 psf
2. 1^{st} Floor:
    a. Public Area: 100 psf
    b. Residential: 40psf
3. 2^{nd} – 16^{th} Floor: 40 psf
4. Main Roof: 100 psf – Public Area, Mechanical, Storage
5. Mechanical Room Floor: 100 psf
6. E.M.R. Floor: 100 psf – Mechanical + Machine Weight
7. E.M.R. Roof: 30 psf

Lateral Loads

Wind and Seismic load analyses are summarized below. For a more detailed description of wind and seismic procedures, please refer to Appendix A and B, respectively.

1. Wind Loads

   The wind load for this building is calculated based on Method 2 of ASCE 7-05 Chapter 6. The factors used to determine the wind loads can be found in Appendix A. For ease of analysis, the building was modeled as a 200 ft. x 80 ft. rectangular box. Through an analysis of the building's fundamental period set forth in ASCE 7-05, the building was found to behave as a flexible structure. Due to the large north-south façade area, the wind loading was found to control in the N-S direction. Also, the calculated wind loading diagrams can be found in Appendix A. The wind distribution is mostly linear down the face of the building with some variance due to different floor-to-ceiling height located on the first three floors.

   Below is a table with calculated values for the load, shear, and moment created at each level for the wind loading.

Level	Load (k)		Shear (k)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
EMR Roof	26	15			4784	2793
EMR	57	33	26	15	10141	5921
Mech. Rm.	60	35	82	48	10029	5855
Roof	61	36	142	83	9597	5603
16	62	36	204	119	8940	5219
15	54	31	266	155	7180	4192
14	49	28	319	186	6040	3527
13	48	28	368	215	5495	3208
12	47	27	416	243	4957	2894
11	46	27	463	270	4432	2588
10	45	26	509	297	3919	2288
9	44	26	554	323	3416	1994
8	43	25	598	349	2923	1707
7	42	24	641	374	2448	1429
6	40	23	682	398	1986	1160
5	38	22	722	422	1543	901
4	36	21	761	444	1122	655
3	34	20	797	465	726	424
2	35	20	831	485	423	247
Totals:	865	505	**865**	**505**	**90100**	**52605**

2. Seismic Loads

Seismic loads for this building were found using the appropriate sections of ASCE 7-05. All calculations and assumptions can be found in Appendix B. The dead loads used for this section can be found in the General Notes section of the structural drawings provided for the building. These loads coincide with all applicable loads described in ASCE 7-05.

Below is a table displaying the final load, shear, and moment due to seismic loading. The base shear is found to be very close to the seismic loading calculated by the structural engineers of record. However, the seismic loading does not control in any direction of the building.

					Load	Shear	Moment
	w_x	h_x	$w_x(h_x)$^k	C_{vx}	F_x	V_x	M_x
Level	(kips)	(ft.)			(kips)	(kips)	(ft-kips)
EMR Roof	178	187.25	6,241,136	0.025	9.73		1,821.95
EMR Floor	258	178.25	8,197,450	0.033	12.78	9.09	2,278.03
Mech Room Floor	652	167	18,183,628	0.073	28.35	21.87	4,734.22
Main Roof	2295	156.58	56,267,195	0.225	87.72	50.22	13,735.44
16	1396	144.58	29,181,113	0.117	45.49	137.94	6,577.50
15	1396	133.58	24,909,688	0.100	38.83	183.43	5,187.53
14	1396	124.25	21,551,535	0.086	33.60	222.27	4,174.70
13	1396	114.91	18,433,214	0.074	28.74	255.87	3,302.25
12	1396	105.58	15,561,402	0.062	24.26	284.61	2,561.42
11	1396	96.25	12,932,631	0.052	20.16	308.87	1,940.61
10	1396	86.92	10,546,901	0.042	16.44	329.03	1,429.21
9	1396	77.58	8,402,044	0.034	13.10	345.47	1,016.22
8	1396	68.25	6,502,655	0.026	10.14	358.57	691.90
7	1396	58.92	4,846,307	0.019	7.56	368.71	445.17
6	1396	49.58	3,431,614	0.014	5.35	376.26	265.25
5	1396	40.25	2,261,607	0.009	3.53	381.61	141.92
4	1396	30.92	1,334,641	0.005	2.08	385.14	64.34
3	1396	21.58	650,112	0.003	1.01	387.22	21.87
2	1396	12.25	209,487	0.001	0.33	388.23	4.00
Totals	24323		249,644,363	1.000	389.20	389.20	50,393.54

CONCLUSION

In this first technical report, a preliminary study was conducted including a wind analysis, seismic analysis, gravity, lateral, and snow load check for Roosevelt Island Southtown Building No. 5. By following the guidelines of ASCE 7-05 and the Building Code of the City of New York, I was able verify some initial calculations that the engineer performed prior to the design of the building's structural system. Although most of the numbers that I obtained were similar to the numbers obtained by the structural engineer, there were some discrepancies.

The first discrepancy I encountered was the shear force due to the seismic loading. I received a 10% higher shear force than that of the structural engineer. Even though I followed the gravity loads assigned by the engineer, I may have accounted for too much dead load in the structure. If the building weight came down, the shear force due to the seismic load would follow.

In most buildings found in New York City, it is assumed that the wind load is the controlling factor for base shear and overturning moment. My calculations validate this assumption. In the North-South direction of the building, the controlling base shear is 865 kips compared to 389 kips of shear for Seismic base shear.

The gravity load check included a two-way flat plate floor slab, punching shear, and column check. All of these calculations have been found to be very similar to those calculated by the structural engineer. A 26' x 24', 8" floor slab was evaluated and was found to resist slab moments resulting from dead and snow loads with the chosen reinforcement. By providing the extra mid-strip bars at the column strips, the shear capacity was also met. However, for most flat slabs, it is very common for punching shear to be the controlling factor. After completing the calculations for a typical 28" x 16" column, it was found that the allowable concrete shear strength was found to be sufficient without the addition of drop panels. Additionally this same size column was checked using PCA Column. This column was found adequate with the given reinforcement to resist the axial load at the 1^{st} floor level.

The lateral load resisting system for this building consisted of reinforced cast-in-place shear walls. Two of these shear walls were reviewed in the N-S direction to determine if they were sufficient to resist the calculated wind loads. Under my calculations, I assumed that the shear wall resisted only lateral loads without the addition of axial force. In later studies, I will review the addition of gravity loads. These increased loads will likely result in an increase in rebar size.

A more in depth analysis of the interaction between gravity and lateral loads in the structural frame will be reviewed in a later report. A complex study of serviceability requirements will also be evaluated.

APPENDIX

APPENDIX A: WIND ANALYSIS

Building specific information for wind analysis

Exposure Class	C
Importance Factor I	1
Topographic Factor K_{zt}	1
Wind Directionality Factor K_d	0.85
Basic Wind Speed V (mph)	110
N-S Length of Building	144
E-W Length of Building	80

No. of Stories	16
Typical Story Height (ft.)	9.33
Building Height (ft.)	187.25
L/B in N-S Direction	1.8
L/B in E-W Direction	0.56
h/L in N-S Direction	1.3
h/L in E-W Direction	2.34

	Cp,windward	Cp, leeward	Cp, side wall	Gust Factor
N-S Direction	0.8	-0.34	-0.7	0.648
E-W Direction	0.8	-0.5	-0.7	0.681

	N-S	E-W
L	80	144
B	144	80
n1	0.56	0.56
TYPE	FLEXIBLE	FLEXIBLE
g_R	4.05	4.05
V_z	126.62	126.62
Lz	638.8	638.8
N1	2.83	2.83
Q	0.254	0.302
Rn	0.072	0.072
Iz	0.163	0.163
Rh	0.228	0.228
Rb	0.283	0.433
Rl	0.167	0.097
R	0.238	0.286
Gf	0.648	0.681

Wind Analysis of Building broken down by story

Level Heights (ft.)	Level	hx	Kz	qz	qh
	EMR Roof	187.25	1.44	38.03	38.03
9.00	EMR	178.25	1.43	37.64	38.03
11.25	Mech. Rm.	167.00	1.41	37.12	38.03
10.42	Roof	156.58	1.39	36.62	38.03
12.00	16.00	144.58	1.37	36.01	38.03
11.00	15.00	133.58	1.35	35.42	38.03
9.33	14.00	124.25	1.32	34.88	38.03
9.33	13.00	114.91	1.30	34.31	38.03
9.33	12.00	105.58	1.28	33.71	38.03
9.33	11.00	96.25	1.26	33.06	38.03
9.33	10.00	86.92	1.23	32.35	38.03
9.33	9.00	77.58	1.20	31.59	38.03
9.33	8.00	68.25	1.17	30.75	38.03
9.33	7.00	58.92	1.13	29.81	38.03
9.33	6.00	49.58	1.09	28.75	38.03
9.33	5.00	40.25	1.04	27.51	38.03
9.33	4.00	30.92	0.99	26.03	38.03
9.33	3.00	21.58	0.92	24.13	38.03
9.33	2.00	12.25	0.81	21.42	38.03

North - South Pressure					
NS windward (+/-)		NS leeward (+/-)		NS side wall (+/-)	
26.56	12.87	-1.53	-15.22	-24.10	-10.41
26.36	12.66	-1.53	-15.22	-24.10	-10.41
26.09	12.40	-1.53	-15.22	-24.10	-10.41
25.83	12.14	-1.53	-15.22	-24.10	-10.41
25.51	11.82	-1.53	-15.22	-24.10	-10.41
25.21	11.52	-1.53	-15.22	-24.10	-10.41
24.93	11.24	-1.53	-15.22	-24.10	-10.41
24.63	10.94	-1.53	-15.22	-24.10	-10.41
24.32	10.63	-1.53	-15.22	-24.10	-10.41
23.98	10.29	-1.53	-15.22	-24.10	-10.41
23.62	9.93	-1.53	-15.22	-24.10	-10.41
23.22	9.53	-1.53	-15.22	-24.10	-10.41
22.79	9.09	-1.53	-15.22	-24.10	-10.41
22.30	8.61	-1.53	-15.22	-24.10	-10.41
21.75	8.06	-1.53	-15.22	-24.10	-10.41
21.11	7.42	-1.53	-15.22	-24.10	-10.41
20.34	6.65	-1.53	-15.22	-24.10	-10.41
19.35	5.66	-1.53	-15.22	-24.10	-10.41
17.95	4.26	-1.53	-15.22	-24.10	-10.41

East - West Pressure					
EW windwall (+/-)		EW leeward (+/-)		EW side wall (+/-)	
13.87	27.56	-6.10	-19.79	-24.97	-11.28
13.66	27.35	-6.10	-19.79	-24.97	-11.28
13.38	27.07	-6.10	-19.79	-24.97	-11.28
13.11	26.80	-6.10	-19.79	-24.97	-11.28
12.77	26.47	-6.10	-19.79	-24.97	-11.28
12.45	26.14	-6.10	-19.79	-24.97	-11.28
12.16	25.85	-6.10	-19.79	-24.97	-11.28
11.85	25.54	-6.10	-19.79	-24.97	-11.28
11.52	25.21	-6.10	-19.79	-24.97	-11.28
11.16	24.85	-6.10	-19.79	-24.97	-11.28
10.78	24.47	-6.10	-19.79	-24.97	-11.28
10.36	24.06	-6.10	-19.79	-24.97	-11.28
9.91	23.60	-6.10	-19.79	-24.97	-11.28
9.40	23.09	-6.10	-19.79	-24.97	-11.28
8.82	22.51	-6.10	-19.79	-24.97	-11.28
8.14	21.83	-6.10	-19.79	-24.97	-11.28
7.33	21.03	-6.10	-19.79	-24.97	-11.28
6.30	19.99	-6.10	-19.79	-24.97	-11.28
4.82	18.51	-6.10	-19.79	-24.97	-11.28

Level	Load (k)		Shear (k)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
EMR Roof	26	15			4784	2793
EMR	57	33	26	15	10141	5921
Mech. Rm.	60	35	82	48	10029	5855
Roof	61	36	142	83	9597	5603
16	62	36	204	119	8940	5219
15	54	31	266	155	7180	4192
14	49	28	319	186	6040	3527
13	48	28	368	215	5495	3208
12	47	27	416	243	4957	2894
11	46	27	463	270	4432	2588
10	45	26	509	297	3919	2288
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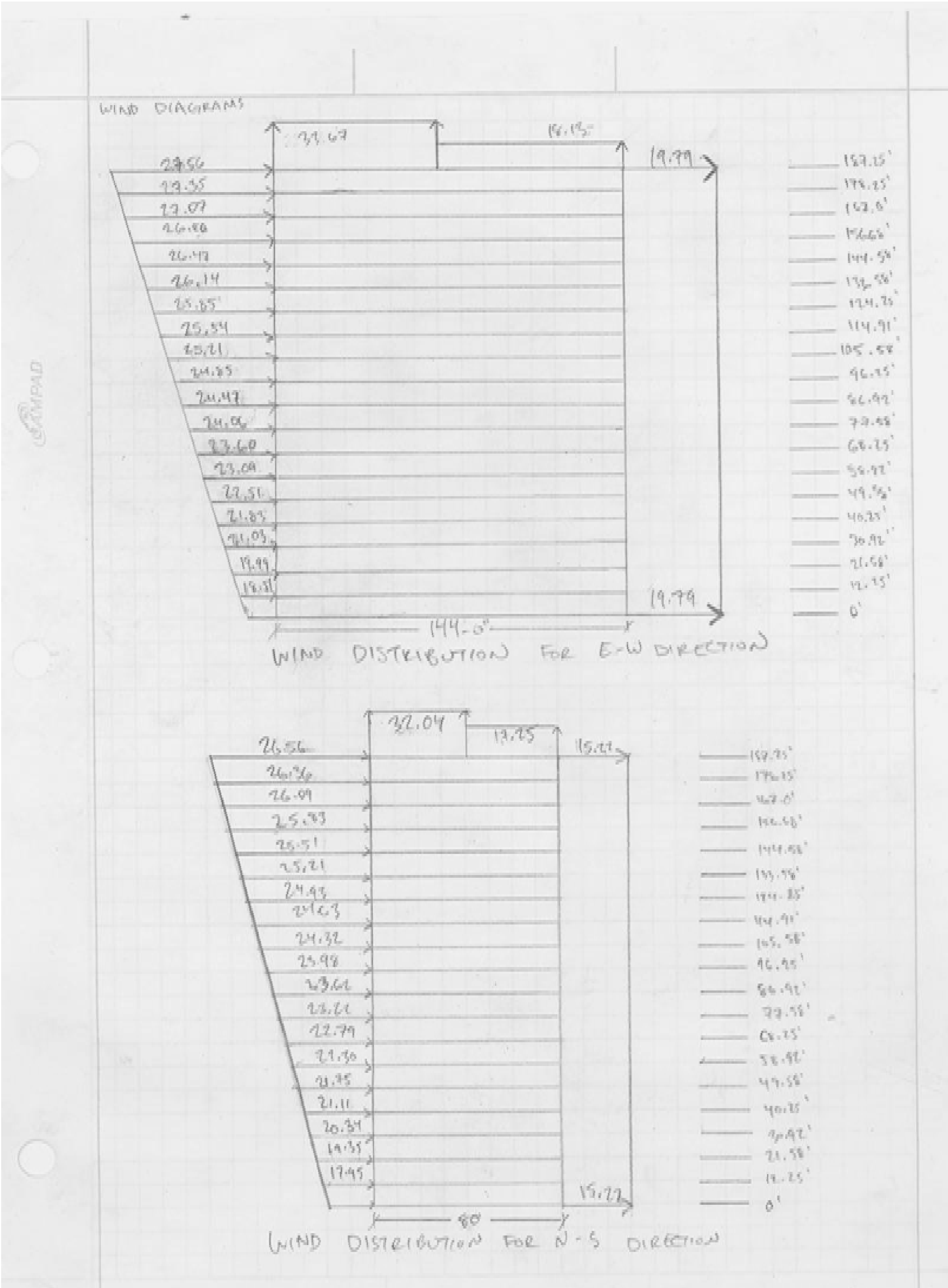
5	38	22	722	422	1543	901
4	36	21	761	444	1122	655
3	34	20	797	465	726	424
2	35	20	831	485	423	247
Totals:	865	505	865	505	90100	52605

Roof Wind Analysis

Roof Calculations		
qh	38.03	
h/L>1.0	Cp1	Cp2
0 to h/2	-1.3	-0.18
>h/2	-0.7	-0.18

Horizontal Distance	N-S				E-W			
	Pw/Cp1		Pw/Cp2		Pw/Cp1		Pw/Cp2	
0 to 93.625	-32.04	±6.85	-4.44	±6.85	-33.67	±6.85	-4.66	±6.85
> 93.635	-17.25	±6.85	-4.44	±6.85	-18.13	±6.85	-4.66	±6.85


WIND DIAGRAMS
144'-0"
WIND DISTRIBUTION FOR E-W DIRECTION
80'
WIND DISTRIBUTION FOR N-S DIRECTION

APPENDIX B: SEISMIC ANALYSIS

1/2

SEISMIC DESIGN

OCCUPANCY CATEGORY : II (STANDARD OCCUPANCY STRUCTURE)
IMPORTANCE FACTOR: I = 1.0
SITE CLASS : C

$S_S = 0.36$
$S_1 = 0.07$

$F_a = 1.6 + (0.36 - 0.25)\frac{1.4 - 1.6}{.5 - .25} = 1.512$ - TABLE 11.4-1

$F_v = 2.4$ - TABLE 11.4-2

$S_{MS} = F_a S_S = 1.512(0.36) = 0.544$
$S_{M1} = F_v S_1 = 2.4(0.07) = 0.168$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.544) = 0.363 \Rightarrow$ C TABLE 11.6-1
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.168) = 0.112 \Rightarrow$ B TABLE 11.6-2

R = 4 - ORDINARY REINFORCED CONCRETE SHEAR WALLS

$$C_s = \min \begin{cases} \frac{S_{DS}}{(R/I)} = \frac{0.363}{(4/1)} = 0.091 \\ \frac{S_{D1}}{T(R/I)} = \frac{0.112}{1.79(4/1)} = 0.016 \\ \frac{S_{D1} T_L}{(T^2 \cdot R/I)} = \frac{0.112(6)}{1.79^2 (4/1)} = 0.052 \end{cases}$$

$T = C_t h_n^x$
$= 0.016 (189.25)^{0.9} = 1.79$

$T_L = 6$

FLOOR AREAS & WEIGHTS

FLOOR	AREA (ft^2)	WT.
2-16	8926	8926 (160) = 1396 (15) = 20942^k
MAIN ROOF	8926	8926 (263) = 2295^k
MECH ROOM FLOOR	2897	2897 (225) = 652^k
EMR FLOOR	1149	1149 (225) = 258^k
EMR ROOF	1149	1149 (155) = 178^k

TOTAL $W_T = 24323$

$V = C_s W_T = 0.016(24323^k) = 389.2^k$

2/2

TYP FLOOR LOAD

CONSTRUCTION DEAD LOAD: 100 psf
SUPERIMPOSED DEAD LOAD: 20 psf
LIVE LOAD: 40 psf
160 psf

MAIN ROOF

CDL: 113 psf
SDL: 50 psf
LL 100 psf
263 psf

MECHANICAL ROOM FLOOR

CDL: 100 psf
SDL: 25 psf
LL: 100 psf
225 psf

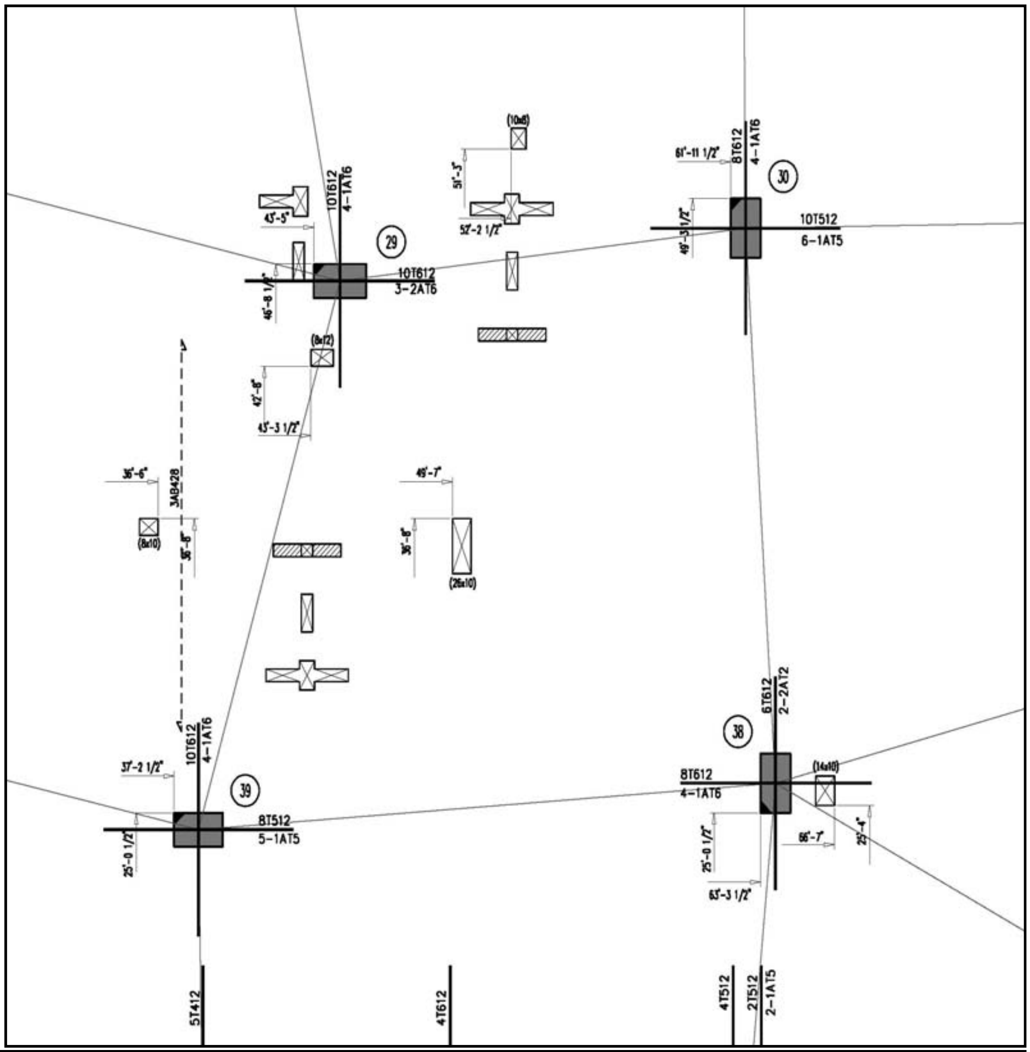
EMR FLOOR

CDL: 100 psf
SDL: 25 psf
LL: 100 psf
225 psf

EMR Roof

CDL: 100 psf
SDL: 25 psf
LL: 30 psf
155 psf

APPENDIX C: GRAVITY LOAD CHECK



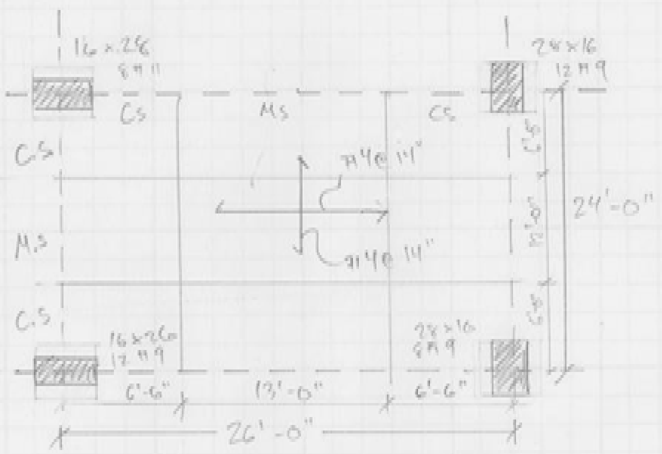
Typical Bay Used for Gravity Load Check

2- Way Flat Plate Floor System Spot Check

1/4

FLAT PLATE FLOOR SLAB SPOT CHECK

WORST CASE SCENARIO : 26'-0" x 24'-0" w/ 8" SLAB INT. SPAN



SLAB: t = 8"
f'_c = 5 ksi

REINF: BOTTOM; #4 @ 14"
TOP: #4 @ 14"

COLS: f'_c = 6 ksi
10 ADD'L #C BARS @ 12" (BOTH WAYS) @ SUPPORT

* #4 @ 14 MIDSTRIP BARS BETWEEN COLUMN SUPPORTS.

LOADS

DEAD

8" SLAB	100 psf
COLUMNS	10 psf
MEP	10 psf
PARTITIONS	20 psf
FIN & MISC	5 psf
	145 psf

LIVE

RESIDENTIAL 40 psf

$$W_u = \max \begin{cases} = 1.2(145\text{ psf}) + 1.6(40\text{ psf}) = \underline{238\text{ psf}} \\ = 1.4(145\text{ psf}) = 203\text{ psf} \end{cases}$$

MIN REINFORCEMENT = $0.0018\,A_g = 0.0018(12")(8") = 0.1728\ \text{in}^2/\text{ft}$
$= 0.204\ \text{in}^2/14"$

$\#4 = 0.2\ \text{in}^2$

#4 @ 14" BOTH WAYS ∴ $A_s = 0.4\ \text{in}^2 > 0.204\ \text{in}^2$

OK

7/4

FRAMING SYSTEM IS FLATE PLATE W/ OUT EDGE BEAMS

DISTRIBUTION OF M_0 — INTERIOR/UNRESTRAINED SPAN

M_{int}^{-}: 0.65

M^{+}: 0.35

$$M_0 = \frac{W_u \ell_2 \ell_n^2}{8}$$

① $M_0 = \frac{(238)(24')(24' - 24''/12'')^2}{8} = 335.2^{\text{1k}}$

② $M_0 = \frac{(238)(24')(24' - 16''/12'')^2}{8} = 366.8^{\text{1k}}$

③ $M_0 = \frac{(238)(26)(26 - 21''/12'')^2}{8} = \underline{455^{\text{1k}}}$ ← CONTROL

LOCATION	STRIP	MOMENT	WIDTH	M_u/WIDTH
SUPPORT .65M_0	C_s 75%	222^{1k}	12'-0"	18.5 1k/ft
	M_s 25%	74^{1k}	12'-0"	6.2 1k/ft
MIDSPAN .35M_0	C_s 60%	95.5^{1k}	13'-0"	7.3 1k/ft
	M_s 40%	63.7^{1k}	13'-0"	4.9 1k/ft

COLUMN STRIP

SUPPORT: $M_n = \frac{M_u}{\phi} = \frac{222^{\text{1k}}}{0.9} = 247^{\text{1k}}$

$d = 8'' - 0.75'' - 0.25'' = 7''$ (0.75" = clear cover)

$R = \frac{M_n}{bd^2} = \frac{247^{\text{1k}}(12'')}{144''(7'')^2} = 420 \text{ psi}$

FROM TABLE A.5.a $\rho = 0.0074$

$A_s = \rho bd = 0.0074(144)(7'') = 7.5 \text{ in}^2$

$A_{s,min} = 0.0018\,bt = 0.0018(144'')(8'') = 2.074 \text{ in}^2$

3/4

$N = \frac{A_s}{0.31} = \frac{7.5 \text{ in}^2}{0.31} = 24.2$ bars

ACTUAL DESIGN

#4 @ 14" o.c.

FOR COLUMN STRIP; ADD'L #4@14" MIDSTRIP BARS

ACTUAL: 22 BARS + 11 ADDL MIDSTRIP BARS = 33 BARS > 24 OK

MIDSPAN:

$M_n = \frac{95.5^{\text{ik}}}{0.9} = 106^{\text{ik}}$

$d = 8" - .75" - .25" = 7"$

$R = \frac{106\,(12)}{(144)(7)^2} = 180$ psi

FROM TABLE A.5 a $\rho = 0.0031$

$A_s = 0.0031(144)(7") = 3.13$

$A_{s,min} = 0.0018\,bt = 0.0018\,(144)(8) = 2.074$ in²

$N = 2.074 - 2(0.31) = \frac{1.454}{.2} = 7.27$ bars → 8 BARS

ACTUAL DESIGN

#4 @ 14" o.c.

ACTUAL: 10 BARS + 5 ADDL MIDSTRIP BARS = 15 BARS > 8 ok

MIDDLE STRIP

SUPPORT

$M_n = \frac{M_u}{\phi} = \frac{74^{\text{ik}}}{0.9} = 82.2^{\text{ik}}$

$R = \frac{M_n}{bd^2} = \frac{82.2^{\text{ik}}(12)}{(144)(7)^2} = 139.8$ psi

$\rho = 0.00245$

$A_s = \rho bd = 0.00245\,(144)(7) = 2.47$ in²

$A_{s,min} = 0.0018\,(144)(8) = 2.074$ in²

4/4

$N = 2.074 / .31 = 6.69$ bars ≈ 7 bars

ACTUAL DESIGN
#4 @ 14"
12 bars USED > 7 bars OK

MIDSPAN

$M_n = \frac{M_u}{\phi} = \frac{63.7^{'k}}{0.9} = 70.78^{'k}$

$R = \frac{M_n}{bd^2} = \frac{70.78(12)}{(144)(7)^2} = 120.37$ psi

$\rho = 0.0020$

$A_s = \rho bd = 0.0020(144)(7) = 2.016 \text{ in}^2$

$A_{s,min} = 0.0018(144)(8) = 2.0741 \text{ in}^2$

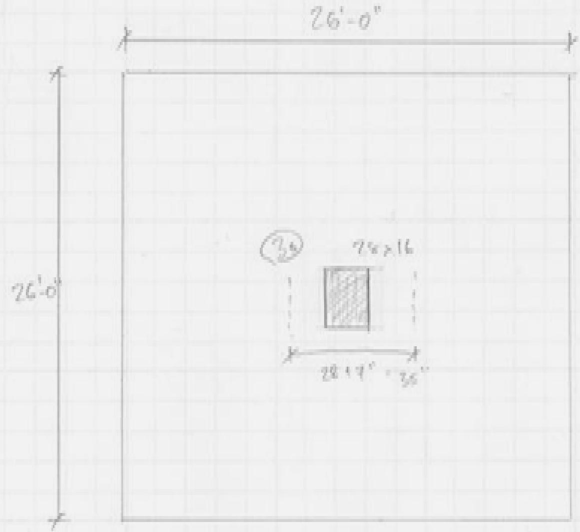
$N = 2.016 \text{ in}^2 / 0.2 = 10.08$ bars

ACTUAL DESIGN:
#4 @ 14"
12 BARS USED > 10.08 bars OK

Punching Shear Spot Check

1/1

PUNCHING SHEAR



W_u: 238 psf

f'_c: 5000 psi

f_y: 60,000 psi

b_o: $(28 + 7") \cdot 4 = \underline{140 \text{ in}}$

or : $(16 + 7") \cdot 4 = 92$ in

$V_u = 238_{psf}\left(26^2 - \frac{35^2}{144}\right) = 159^k$

$\frac{b_o}{d} = \frac{140 \text{ in}}{7 \text{ in}} = 20.$

$V_c = 4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}\,(140)(7) = 277^k$

$V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d$

$= (2 + 4)\sqrt{5000}\,(140)(7) = 416^k$

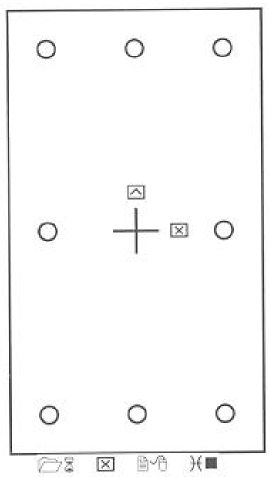
$V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d$

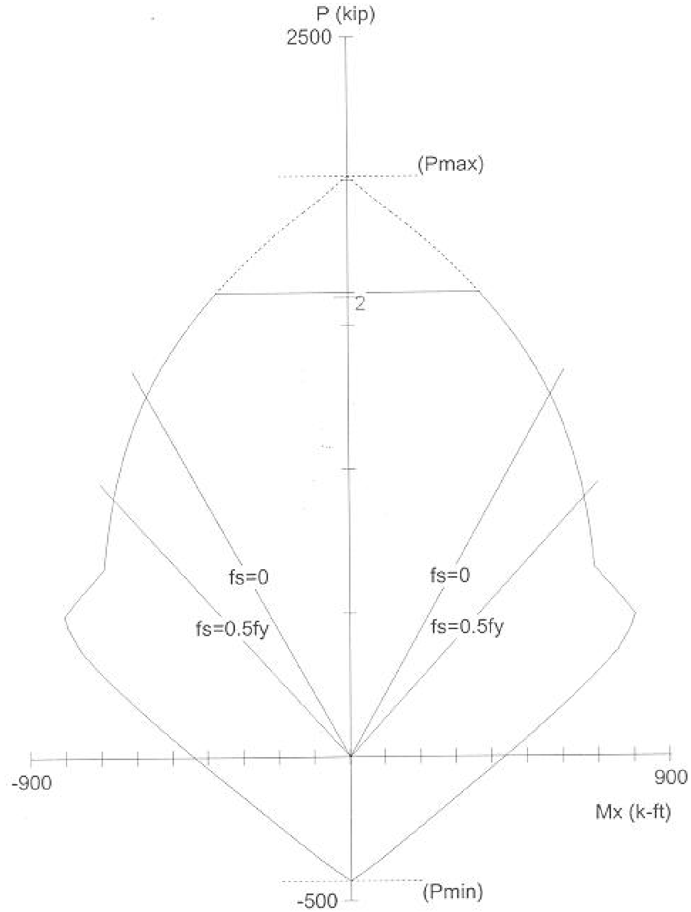
$= \left(\frac{40(7)}{140} + 2\right)\sqrt{5000}\,(140)(7) = 277^k$

$\phi V_c = 0.75(277^k) = 207.75^k > 159^k$ ok

Column Check (using PCA Column)

Column 38: 28” x 16”





Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Design

Slenderness: Considered

Column type: Structural

Bars: ASTM A615

Date: 10/01/07

Time: 18:17:39

pcaColumn v3.64. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D

File: F:\Thesis\Column 38 at base.col

Project:

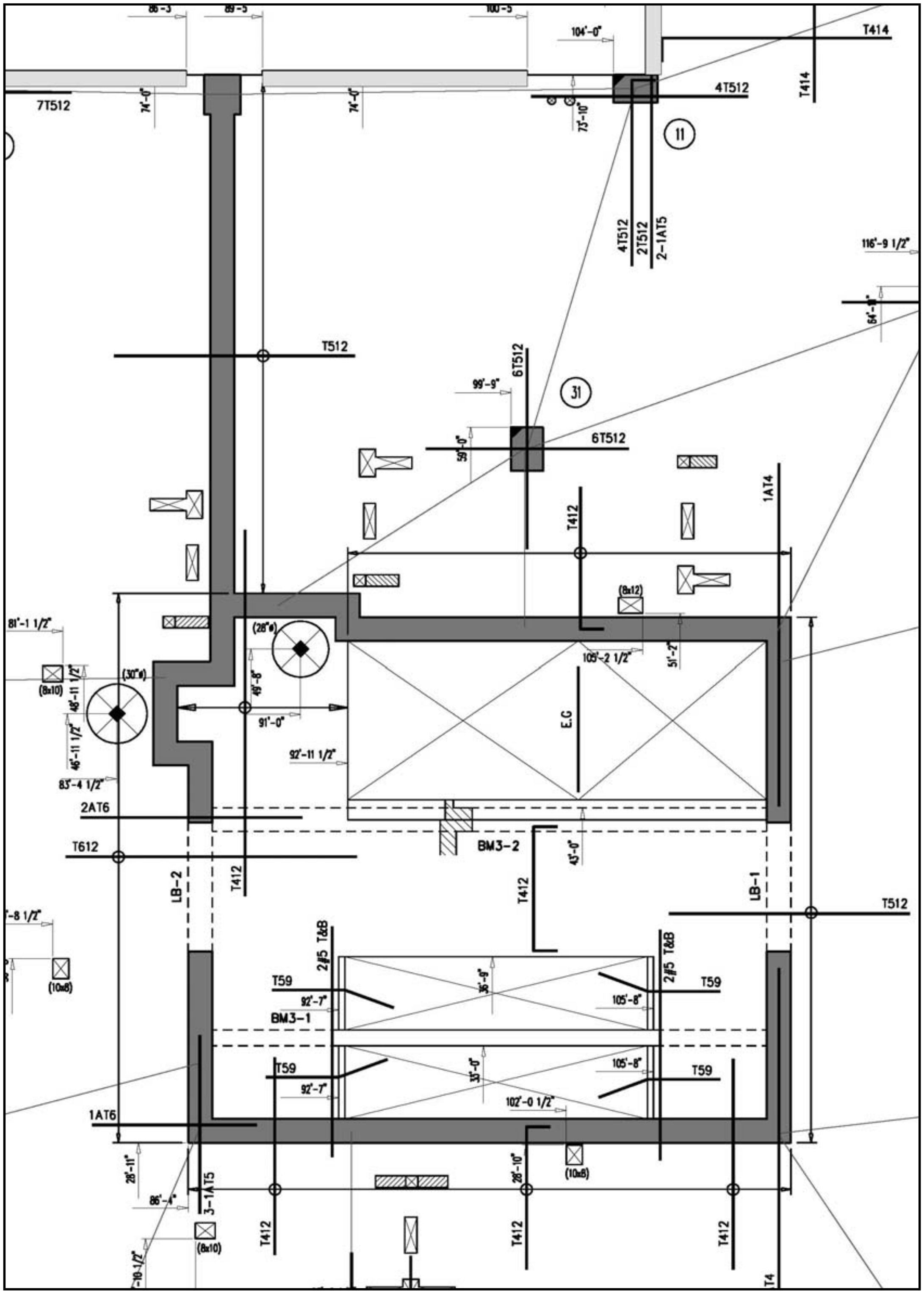
Column: c1-c45 | Engineer:

f'c = 7 ksi	fy = 60 ksi	Ag = 448 in^2	8 #9 bars
Ec = 4769 ksi	Es = 29000 ksi	As = 8.00 in^2	Rho = 1.79%
fc = 5.95 ksi	fc = 5.95 ksi	Xo = 0.00 in	Ix = 29269.3 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 9557.33 in^4
Beta1 = 0.7		Clear spacing = 4.43 in	Clear cover = 1.88 in

Confinement: Tied phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

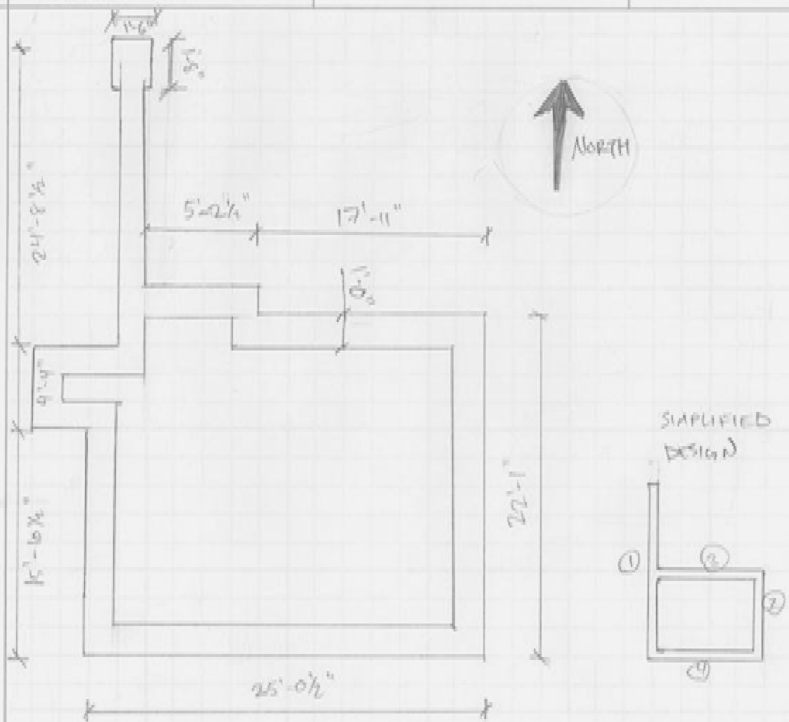
kx(braced) = 1, kx(sway) = N/A

APPENDIX D: SHEAR WALL CHECK



Typical Shear Wall

SHEAR WALL CHECK 1/4



1st FLOOR: f'c = 7 ksi
t = 1'-0"

CHECK N-S WIND LOAD (SHEAR WALLS 1 AND 2) SINCE IT PRODUCES LARGEST BASE SHEAR.
ASSUME THAT THE DISTRIBUTION OF FORCES IS BASED ON THE OVERALL LENGTH OF THE WALL

WALL	LENGTH	STIFFNESS	DISTRIBUTION
SW 1	44'-11"	44'-11"/44'-11" = 1.0	65%
2	22'-1"	22'-1"/44'-11" = 0.5	35%

REINFORCEMENT = #4 @ 12" O.C. HORIZ. EACH FACE
SW 1 #8 @ 12" O.C. VERT. EACH FACE

2/4

FROM WIND DESIGN

SHEAR WALL ①

$V = 831^k \Rightarrow$ TAKES 65% : $.831(.65) = 540.15^k$

$M = 90{,}100\,(.65) = 58565^{1k}$

$P = (44.9')(1')(187.25')(150\text{ pcf}) = 1261^k$

$C_u = \frac{P_u}{2} + \frac{M_u}{d} = \frac{1261^k}{2} + \frac{58565^{1k}}{44.9} = 1935^{1} = P_{u_{BE}}$

BOUNDARY ELEMENTS

$A_g = (44.9')(1'\text{-}0") = 44.9\text{ ft}^2$

$I_g = \frac{(44.9')^3(1)}{12} = 7543.2\text{ ft}^4$

$f_c = \frac{P_u}{A_g} + \frac{M_u \frac{h_w}{2}}{I_g} = \frac{1261}{44.9} + \frac{58565\left(\frac{44.9}{2}\right)}{7543.2} = 202.4\text{ ksf} = 1.41\text{ ksi}$

$.2(7\text{ ksi}) = 1.4\text{ ksi} \leq 1.41\text{ ksi}$ NO NEED FOR BOUNDARY ELEMENTS

REINFORCEMENT

HORIZ: $\rho_l, \rho_t = \frac{A_{st}}{A_{cv}} \geq 0.0025$ $\quad A_{cv} = 12" \times (12") = 144\text{ in}^2/\text{ft}$

$A_{s,req'd} = (0.0025)(144) = 0.36\text{ in}^2/\text{ft}$ MIN

ASSUME #4 IN 2 CURTAINS

$A_{s,l} = 2(0.2) = 0.4\text{ in}^2/\text{s}$ $\quad$ #4 @ 12" ok

$\frac{h_w}{l_w} = \frac{187.25}{44.9} = 4.17 > 2 \quad \alpha_c = 2.0$

3/4

VERT

$A_{cv} = (12)(44.9' \times 12) = 6465.6 \text{ in}^2$

$\rho_t = \frac{2(.79)}{(12'')(12'')} = 0.01097 \Rightarrow$ 2 rows #8 @ 12"

$V_n = (6465.6)(2\sqrt{7000} + 0.01097 \cdot 60{,}000)/1000 = 5337.6^k$

$\phi V_n = 0.6(5337.6) = 3202.5^k > V_u = 540.15^k$ ok

#4 @ 12" HORIZ. EACH FACE ok

#8 @ 12" VERT EACH FACE ok

SW (2)

#4 @ 12" OC. HORIZ. REINF EACH FACE

#7 @ 12" OC VERT REINF. EACH FACE

$V = 0.35(831) = 290.8^k$

$M = 90{,}100\,(.35) = 31{,}535^{ft\text{-}k}$

$P = (22.08')(1')(187.25')(150\text{ pcf}) = 620.17^k$

$C_u = \frac{P_u}{2} + \frac{M_u}{d} = \frac{620.17}{2} + \frac{31535}{22.08} = 1738.3$ ft-k

BOUNDARY ELEMENTS

$A_g = (22.08')(1'-0'') = 22.08 \text{ ft}^2$

$I_g = \frac{(22.08')^3(1')}{12} = 897.05 \text{ ft}^4$

$f_c = \frac{P_u}{A_g} + \frac{M_u \frac{h_w}{2}}{I_g} = \frac{620.17}{22.08} + \frac{31535\left(\frac{22.08'}{2}\right)}{897.05} = 414.96 \text{ ksf} = 2.88 \text{ ksi}$

$.2f'_c = .2(7 \text{ ksi}) = 1.4 \text{ ksi} < 2.88 \text{ ksi}$, NO NEED FOR BOUNDARY ELEMENT.

4/4

REINFORCEMENT

HORIZ:

$\rho_{l}, \rho_{t} = \frac{A_{sl}}{A_{cv}} \geq 0.0025$ $\quad A_{cv} = 12''(12'') = 144\ in^2/ft$

$A_{s,req'd} = (0.0025)(144) = 0.36\ in^2/ft$ MIN

Assume #4 in 2 curtains

$A_{sl} = 2(0.2) = 0.4\ in^2/s$ $\quad$ #4 @ 12" OK.

$\frac{h_w}{l_w} = \frac{187.25}{22.01} = 8.51 > 2$ $\quad \alpha_c = 2.0$

VERT:

$A_{cv} = (12)(22.01 \times 12) = 3169.44\ in^2$

$\rho_t = \frac{2(.6)}{(12'')(12'')} = 0.0083 \Rightarrow$ #7 @ 12"

$V_n = (3169.44)(2\sqrt{7000} + 0.0083 \cdot 60{,}000)/1000 = 2108.7^k$

$\phi V_n = 0.6(2108.7) = 1265.24^k > 290.8^k$ OK

#7 @ 12" OK

APPENDIX E: SNOW LOAD CHECK

1/1

SNOW LOAD

ASCE 7-05

$p_g = 20$ psf (Fig 7.1)

$C_e = 1.1$, EXPOSURE C, SHELTERED

$C_t = 1.0$, HEATED STRUCTURE

$I = 1.0$, CATEGORY II

$$p_f = 0.7 C_e C_t I p_g$$

$$= 0.7(1.1)(1.0)(1.0)(20) = 15.4 \leq 20$$ ($p_f = I p_g$ when p_g is 20psf or less)