HIGH RISE CONDO
SOHO, NEW YORK, NY

TECHNICAL ASSIGNMENT 1

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TABLE OF CONTENTS

EXECUTIVE SUMMARY

The Soho high rise condominium project consists of 13 above grade stories and two below grade stories. The building encompasses roughly 175,000 SF stretching from 28 feet below grade to 175 feet above grade. The first floor houses highly marketable retail spaces while the remaining 12 stories are condominium units. A sub-cellar level is set aside for resident parking and the cellar level contains a pool lounge, exercise facility, resident storage spaces and mechanical rooms. There are also roof terraces and Jacuzzi pools located at the 6^{th} Floor step back.

In the first technical report the existing structural conditions are introduced through a detailed description of the foundation, floor, column and lateral systems. Structural concepts were investigated including preliminary analysis of the lateral force resisting system. Spot checks of gravity loads and lateral loads were done on a typical floor bay, column and shear wall for discrepancies in design loads.

ASCE 7-05 was used to determine all wind and seismic loads. For wind loads Method 2 (analytical procedure) of ASCE 7-05 section 6 was used. Seismic design loads were established using the equivalent lateral force procedure set forth in ASCE 7-05.

STRUCTURAL SYSTEMS

Foundations

A Mat foundation ranging from 3-0" to 4'-0" provides the foundation system for the high rise. The 4'-0" thick section is located at the center of the sub-cellar around the elevator cores which make up the main lateral resisting system of the building. There are a few areas requiring the mat foundation to be stepped down to provide for sump and elevator pits. A 6" mud slab is located below the mat foundation.

Floor System

The floor system of the Soho high rise is typically a 10-1/2" two-way normal weight concrete flat plate with bays range in size from 13 feet by 21 feet to 25 feet by 25 feet. At the cellar level a 12" two-way slab is used to accommodate for mechanical equipment. Typical reinforcement is #4 @12" bottom steel and #5 @ 16" top steel. Additional reinforcement is located in some areas where the uniform steel was inadequate. This allows for a more economical rebar layout. At the tower transfer level or 6^{th} floor deep beams and areas of thickened slab transfer the overturning moment and tower column loads from the tower into the structure below.

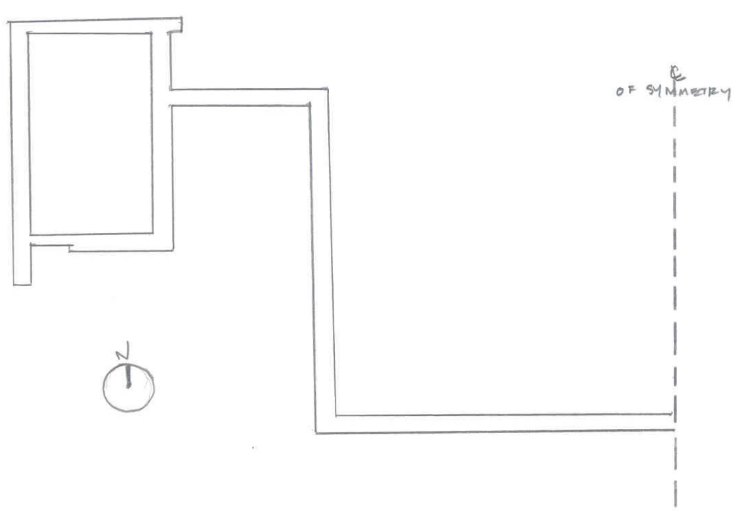
Between the ground level and sub-cellar a concrete ramp allows for vehicular traffic. The framing for the ramp differs to accommodate the varying requirements of the structure. It begins at the ground level with an 8” one-way slab between beams spanning from the 16” interior columns to the foundation wall. This is to accommodate the planter section located in the center of the ramp. The high aspect ratio would not allow for a two-way slab design. The next section is a 4” slab on top of the 12” cellar slab with the inter layer being taken up by a foam fill. The third section is a 14” deep two-way slab system. The remaining ramp is concrete infill providing a gradual slope to the sub-cellar level.

Columns

The columns in the Soho high rise are primarily standard reinforced concrete with varying sizes, shape and reinforcement depending on their location in the building. The most typical shapes are 20x14 and 12x19, both with 6-#9 bars as reinforcement. The total service loads vary greatly from column to column and are as large as 2000^k in one location and as low as 70^k in another at the foundation level. This may be a result of the atypical bay sizes, column heights and the load funneling effect of stiffer columns. There are also two concrete encased W21x201 w/ (2) 2 ½” web plates spanning between ground and third floor at the entrance lobby. This is most likely attributed to the architectural requirements of the designer for a narrower section than would be feasible with a concrete section.

Lateral System

Concrete shear walls make up the buildings lateral load resisting system. The two elevator cores have been used as the main components of these elements and are connected up to the seventh floor where they become independent sections. Mechanical and architectural penetrations have been allowed in several areas, but require specially detailed link beams to transfer the shear forces. Typical shear wall reinforcement is #4 @ 12” o.c. each way, but increases in some areas to accommodate for axial load and increased shear forces that must be resisted.



Typical Shear Wall Layout at Building Core
(West Elevator Core section shown)

CODE AND DESIGN REQUIREMENTS

Codes and References

1. “The Building Code of the City of New York”.

2. “The New York City Seismic Code: Local Law 17/95”.

3. “Building Code Requirements for Structural Concrete (ACI 318-02)”, American Concrete Institute.

4. “Specification for the Design, Fabrication and Erection of Structural Steel for Buildings-Load and Resistance Factor Design,” Second Edition, and “Allowable Stress Design,” Ninth Edition, American Institute of Steel Construction.

5. “Minimum Design Loads for Buildings and Other Structures (ASCE 7-98)”, American Society of Civil Engineers.

Deflection Criteria

Slab deflection Criteria

L/240 Total and L/360 Live Load

L/600 or ½” max for curtain wall support

L/1666 max impact loads for elevator support beams

Lateral Deflection Criteria

Wind allowable drift (total building):	H/500
Wind inter-story drift:	H/400
Seismic allowable drift:	H/400

MATERIALS

1. Concrete:

a. Footings	F'c= 4000psi
b. Foundation walls	F'c= 4000psi
c. Slab on grade	F'c= 4000psi
d. Floor slabs	F'c= 4000psi
e. Shear walls	F'c= 4000psi
f. Columns	F'c= 4000psi

2. Reinforcing Steel:

a. Rebar	ASTM A-615, Grade 60
b. Welded Wire Fabric	ASTM A-185

3. Structural Steel:

a. Columns	ASTM A36 and A572- Grade 50
b. Beams	ASTM A36 and A572- Grade 50
c. Base Plates	ASTM A572- Grade 50 and 42 A588 Grade 42 and A36
d. Bolts	ASTM A-325 and A-490
e. Anchor Bolts	ASTM A-307/ ASTM A-325
f. Weld Electrodes	E70XX

4. Metal Deck and Shear Studs:

a. Composite Floor/Roof Deck	3" Galvanized
b. Studs	¾" dia. X 4.5"/6" headed stud

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 and interpretation of structural details and components. Gravity loads are as follows:

Dead Loads

Construction Dead Loads:

Concrete	150 PCF

Superimposed Dead Loads:

¼" Glass and Framing	20 PSF
Partitions	20 PSF
Finishes and Misc.	5 PSF
MEP	10 PSF
Roofing	20 PSF
Terrace (pavers, planters, etc.)	150 PSF

Live Loads

Typical Floor	40 PSF
Sub-Cellar	100 PSF
1st Floor & Cellar	100 PSF
6th Floor & Roof	100 PSF
Public Areas	75 PSF
Snow Load	30 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. Appendix A contains a detailed analysis of wind loads using the equations and factors set forth in ASCE. These factors are dependent on building location and characteristics as well as experimental data. For ease of analysis the high rise was modeled as two rectangular boxes, one on top of the other. The tributary width for the tower in the N-S is roughly half of the base. This was taken into account in determining the resultant forces, but its effect on other variables has been considered negligible. Through a generalized analysis of the buildings fundamental period set forth in ASCE 7-05 the high rise condo was found to behave as a rigid structure. (*See the seismic analysis located in appendix B for the building period calculation*) Because the building is more than twice as large in the E-W direction the total wind load resulting from wind in the N-S direction is much larger. The wind loading was found to control in the N-S direction. Also note that because story heights are not constant the wind distribution is not a perfect curve (i.e. at the first floor the story height is 19 feet while the typical building story height is between 12 and 13 feet).

Level	Load (k)		Shear (k)		Moment (ft-k)	
	N/S	E/W	N/S	E/W	N/S	E/W
Roof	82	14	0	0	14,367	2,384
12	71	12	82	14	11,379	1,880
11	71	12	153	25	10,529	1,739
10	69	11	224	37	9,471	1,557
9	69	11	293	48	8,639	1,420
8	67	11	363	60	7,593	1,241
7	65	11	430	71	6,593	1,070
6	64	10	496	81	5,706	922
5	98	32	560	92	7,540	2,427
4	65	21	658	124	3,780	1,201
3	64	20	723	144	2,859	902
2	61	19	787	164	1,967	615
1	81	25	848	184	1,547	470
Totals	930	208	930	208	91,970	17,828

Seismic

Seismic loads were found using the applicable sections of ASCE 7-05. All factors and accelerations were found using the tables and equations contained in ASCE and can be found in Appendix B. All dead loads used are based on ASCE 7-05 and are listed in the gravity loads section of this report. Because the high rise condo is narrow in the N-S direction relative to the E-W direction the seismic design was found to control over wind in the E-W direction.

					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)
Roof	785	184.67	534,398	0.096	59		10,899
13	980	172.67	613,405	0.110	68	59	11,698
12	975	160.67	557,729	0.100	62	121	9,897
11	975	148.67	506,155	0.090	56	188	8,311
10	975	136.67	455,613	0.081	50	244	6,877
9	975	124.67	406,169	0.073	45	295	5,592
8	975	112.67	357,903	0.064	40	339	4,454
7	975	100.67	310,906	0.056	34	379	3,457
6	3,890	76.67	882,534	0.158	97	413	7,473
5	2,480	58	396,951	0.071	44	511	2,543
4	2,480	45	289,046	0.052	32	555	1,437
3	2,480	32	188,751	0.034	21	587	667
2	2,355	19	93,419	0.017	10	607	196
Totals			5,592,979	1.000	618	618	73,500

CONCLUSIONS

In the first technical report the existing structural conditions were introduced through a detailed description of the foundation, floor, column and lateral systems. Structural concepts were investigated including preliminary analysis of the lateral force resisting system. Spot checks of gravity loads and lateral loads were done on a typical floor bay, column and shear wall for discrepancies in design loads. An interior bay of the 10 ½" two-way flat plate slab was found adequate to resist slab moments resulting from typical uniform dead and live loads. Punching shear in a flat plate system is often a controlling factor in design and was analyzed at a 22 inch square column. The allowable concrete shear was found sufficient to resist this failure mechanism without the addition of drop plates. An interior column was checked for axial compression at the 7^{th} floor level and found to be larger than required for axial compression alone. A more in depth analysis containing the interaction of the gravity and lateral loads in the structural frame as well as addressing serviceability requirements will be contained in a later report. I believe this column may be larger due to the increased stiffness required in the tower to reduce drift, the addition of moment at the column as a result of the rigidity of a concrete system and dimensions required to avoid punching shear as a failure mechanism. A shear wall section was also checked at the ground floor level. Self weight was assumed to be the only axial load action on the wall. This assumption allows for a preliminary check on the shear wall for lateral loads alone. Gravity loads can later be added in and vertical bars can be increased to resist the added axial load that results. The results of the gravity and lateral load spot checks have been presented in Appendices C&D.

ASCE 7-05 was used to determine all wind and seismic loads. For wind loads Method 2 (analytical procedure) of ASCE 7-05 section 6 was used. Seismic design loads were established using the equivalent lateral force procedure set forth in ASCE 7-05. The results of these analyses have been presented in Appendices A&B.

APPENDIX

Exposure Class	B
Importance Factor I	1
Topographic Factor K_{zt}	1
Wind Directionality Factor K_d	0.85
Basic Wind Speed V (mph)	100
N-S Length of Bldg.	80
E-W Length of Bldg.	200
Ct factor in the N-S Direction	0.02
Ct factor in the E-W Direction	0.02

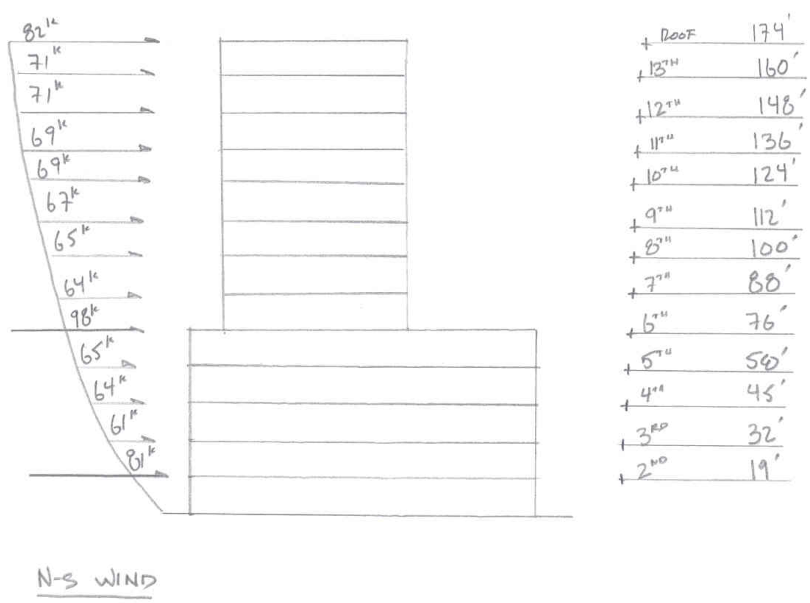
x= 0.75

No. of Stories	13			
Typ. Story Height (ft)	12			
Builidng Height (ft)	174			
L/B in N-S Direction	0.40			
L/B in E-W Direction	2.50			
h/L in N-S Direction	2.18			
h/L in E-W Direction	0.87			
	$C_{p,windward}$	$C_{p,leeward}$	$C_{p,side\ wall}$	Gust Factor
N-S Direciton:	0.80	-0.50	-0.70	0.91
E-W Direciton:	0.80	-0.28	-0.70	0.91

GUST FACTOR		
	N-S	E-W
L	80.00	200.00
B	200.00	80.00
n_1	1.04	1.04
TYPE	RIGID	RIGID
z_{min}	30.00	30.00
c	0.30	0.30
I_z	0.25	0.25
h	174.00	174.00
L_z	469.76	469.76
l	320.00	320.00
z	104.40	104.40
epsilon hat	0.33	0.33
Q	0.97	0.98
g_Q	3.40	3.40
g_v	3.40	3.40
G	0.91	0.91

Level Heights(ft)	Level	hx	Kz	qz	Pressures						Load (k)		Shear (k)		Moment (ft-k)	
					NS windward	NS leeward	NS side wall	EW windward	EW leeward	EW side wall	N/S	E/W	N/S	E/W	N/S	E/W
13.67	Roof	174.34	1.17	25.46	18.55	-11.59	-15.68	18.61	-6.40	-15.73	82	14	0	0	14,367	2,384
12	12	160.67	1.13	24.59	17.91	-11.59	-15.68	17.97	-6.40	-15.73	71	12	82	14	11,379	1,880
12	11	148.67	1.13	24.59	17.91	-11.59	-15.68	17.97	-6.40	-15.73	71	12	153	25	10,529	1,739
12	10	136.67	1.09	23.72	17.28	-11.59	-15.68	17.34	-6.40	-15.73	69	11	224	37	9,471	1,557
12	9	124.67	1.09	23.72	17.28	-11.59	-15.68	17.34	-6.40	-15.73	69	11	293	48	8,639	1,420
12	8	112.67	1.04	22.63	16.49	-11.59	-15.68	16.54	-6.40	-15.73	67	11	363	60	7,593	1,241
12	7	100.67	0.99	21.54	15.70	-11.59	-15.68	15.75	-6.40	-15.73	65	11	430	71	6,593	1,070
12	6	88.67	0.96	20.89	15.22	-11.59	-15.68	15.27	-6.40	-15.73	64	10	496	81	5,706	922
18.67	5	76.67	0.93	20.24	14.74	-11.59	-15.68	14.79	-6.40	-15.73	98	32	560	92	7,540	2,427
13	4	58	0.85	18.50	13.48	-11.59	-15.68	13.52	-6.40	-15.73	65	21	658	124	3,780	1,201
13	3	45	0.81	17.63	12.84	-11.59	-15.68	12.88	-6.40	-15.73	64	20	723	144	2,859	902
13	2	32	0.76	16.54	12.05	-11.59	-15.68	12.09	-6.40	-15.73	61	19	787	164	1,967	615
19	1	19	0.62	13.49	9.83	-11.59	-15.68	9.86	-6.40	-15.73	81	25	848	184	1,547	470
Totals											930	208	930	208	91,970	17,828

Wind Distribution for Wind in the E-W Direction



Wind Distribution in the N-S Direction

SEISMIC DESIGN 1/2

OCCUPANCY CATEGORY → II → I=1.0

SITE CLASS D → STIFF CLAYS → N=35 BLOWS/FT (FROM GEOTECH REPORT)

$S_s = .35$ Fig 22.1 – 22.14

$S_1 = .07$

$S_{MS} = F_a S_s = 1.52(.35) = .532$

$S_{M1} = F_v S_1 = 2.4(.07) = .168$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(.532) = .355 \longrightarrow \underline{\underline{C}}$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(.168) = .112 \longrightarrow B$

R=4 ORDINARY REINFORCED CONCRETE SHEAR WALLS
Cd=4

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{.355}{(4/1)} = .089$ $\quad T = C_t h_n^x = (174)^{.75} .02 = 0.96$

$T_L = 6$ Fig. 22-15

$\leq \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{.112}{.96\left(\frac{4}{1}\right)} = \underline{\underline{.029}}$ $\quad k = 1.25$ (12.8.3)

$V = C_s W_T = .029(21300) = 617.7^k$

FLOOR AREAS

FLOOR	AREA	TERRACE AREA
2	13,300 SF	—
3-5	14,000 SF	—
6	5,500 SF	8,500 SF
7-12	5,500 SF	—
13	4,700 SF	500 SF
R	4,700 SF	—

WEIGHTS

FLOOR	WT.	
2	$W_x = (13,300)\overbrace{(20+5+10+132+10)}^{177}$	$= 2355^k$
3-5	$W_x = (14,000)(177)$	$= 2480^k$
6	$W_x = (5500)(220) + 8500(315)$	$= 3890^k$
7-12	$W_x = (5500)(177)$	$= 975^k$
13	$W_x = (4700)(177) + 500(147+150)$	$= 980^k$
R	$W_x = (4700)(10+132+20+5)$	$= 785^k$
		$W_T = 21,300^k$

SEISMIC DESIGN 2/2

TYP FLOOR LOAD

PARTITIONS	20 PSF
FIN. & MISC.	5 PSF
MEP	10 PSF
10½" SLAB	132 PSF
COLUMNS	10 PSF
	177 PSF

TYP. TERRACE LOAD

TERRACE	150 PSF
FIN & MISC.	5 PSF
MEP	10 PSF
10½" SLAB	132 PSF

6TH FLOOR

PARTITIONS	20 PSF
FIN. & MISC.	5 PSF
MEP	10 PSF
12" SLAB	150 PSF
BEAMS & SLAB ADD.	25 PSF
COLUMNS	10 PSF
	220 PSF

6th FLOOR TERRACE

TERRACE	150 PSF
12" SLAB	150 PSF
MEP	10 PSF
FIN & MISC.	5 PSF
	315 PSF

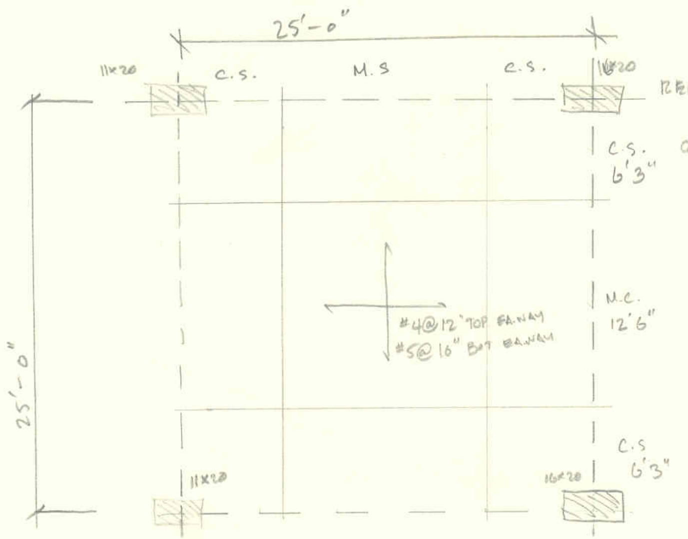
ROOF

MEP	10 PSF
ROOF	20 PSF
SLAB	132 PSF
MISC.	5 PSF
	167 PSF

1/

FLAT PLATE SPOT CHECK

SINCE A TYPICAL BAY IS NOT PRESENT THROUGHOUT THE HIGH RISE THE WORST CASE SCENARIO WILL BE USED TO CHECK GRAVITY LOADS. THE LARGEST BAY IS APPROX. 25'-0" x 25'-0" & 10½" THICK.



SLAB:
$t = 10\frac{1}{2}"$
$f'c = 4^{ksi}$

REINF: #4@12" EA.WAY @ top.
#5@16" EA.WAY @ bot.

COLUMN:
11x20 – 6#9
16x20 – 6#9
$f'c = 4^{ksi}$

*ADDL. 8#5 bars located @ supports
ADDL. 2#5 bars located @ Col. Strip. MIDSPAN

LOADS

DEAD		LIVE	
PARTITIONS	20 PSF	RESIDENTIAL	40 PSF
FIN & MISC	5 PSF		
MEP	10 PSF		
10½" SLAB	132 PSF		
COLUMNS	10 PSF		
	177 PSF		

$W_u = 1.2(177\text{ PSF}) + 1.6(40\text{ PSF})$
$= \underline{276.5\text{ PSF}}$
$1.4(177) = 247.8\text{ PSF}$

MIN. REINF. $= 0.0018A_g = 0.0018(12")(10.5") = 0.2268\ in^2/ft$

$\#4 \rightarrow 0.2in^2$ #4 both ways $= A_s = 0.4in^2 > 0.23in^2$ ok
$\#5 \rightarrow 0.31in^2$

FRAMING SYSTEM IS FLATE PLATE W/o EDGE BEAMS

DISTRIBUTION OF Mo		INTERCONNECTED SPAN
	M_{INT}	0.65
	M^+	0.35

INTERIOR PANEL

$$M_o = \frac{W_u l_2 l_n^2}{8} = \frac{.276.5(25.0')(25' - 11''/12)^2}{8} = 501'^k$$

LOCATION	STRIP	MOMENT	WIDTH	MU/WIDTH
SUPPORT 0.65 Mo	C.S. 75%	244'k	12'6"	19.5 'k/ft
	M.S. 25%	81.5'k	12'6"	6.5 'k/ft
MIDSPAN 0.35 Mo	C.S. 60%	105'k	12'6"	8.4 'k/ft
	M.S. 40%	70'k	12'6"	5.6 'k/ft

COLUMN STRIP

SUPPORT: #5 @ 16" o.c. → $A_s = 0.24 in^2/ft$ $d = 10.5'' - .75'' - .3125'' = 9.44''$

$$M_n = \frac{M_u}{\phi} = \frac{244'^k}{0.9} = 271'^k \qquad R = \frac{M_n}{bd^2} = \frac{271'^k(12)}{(9.44)^2(150)} = 243.3\ psi$$

ρ from table A.5a 0.00425

$A_s = \rho bd = 0.00425(150)(9.44) = 6 in^2$

$A_{s\,min} = 0.0020bt = 0.002(150)(10.5) = 3.15 in^2$

$N = A_s/0.31 = 6/0.31 = 19.35$ bars

$N_{min} = \frac{12.5(12)}{2(10.5)} = 7.14$ bars (SAME FOR ALL)

ACTUAL DESIGN
#5@16" o.c. → 10 bars + 8 = 18 bars < 19.35 bars

MIDSPAN: #4 @ 12" o.c. $d = 10.5'' - .75'' - .25'' = 9.5''$

$$M_n = \frac{105'^k}{0.9} = 117'^k \qquad R = \frac{117(12)}{(150)(9.5)^2} = 104$$

ρ from table A.5a 0.00175

$A_s = 0.00175(150)(9.5) = 2.5 in^2$

$A_{s\,min} = 0.002bt = 0.002(150)(10.5) = 3.15 in^2$

$N = 3.15in^2 - 2(0.31) = \frac{2.53}{.2} = 12.65$ bars

Actual design
#4@12" o.c. → 12.5 bars ≈ 12.65 bars

MIDDLE STRIP

SUPPORT

$M_n = \frac{M_u}{\phi} = \frac{81.5'^k}{.9} = 90.6'^k$ $\quad R = \frac{M_n}{bd^2} = \frac{90.6(12)}{(150)(9.44)^2} = 81.3^{psi}$

ρ from table A.5a 0.0015

$A_s = \rho bd = 0.0015(9.44)(150) = 2.12 in^2$

$A_{s\,min} = 0.002(150)(10.5) = 3.15 in^2$ (met b/c 2 curtains of steel present)

$N = 3.15/.31 = 10.2$ bars

ACTUAL DESIGN

#5 @ 16" o.c. ⟶ 10 bars > 10.2 bars

MIDSPAN

$M_n = \frac{M_u}{\phi} = \frac{70'^k}{0.9} = 77.8'^k$ $\quad R = \frac{M_n}{bd^2} = \frac{77.8(12)}{(150)(9.5)^2} = 68.9^{psi}$

ρ from table A.5a 0.00125

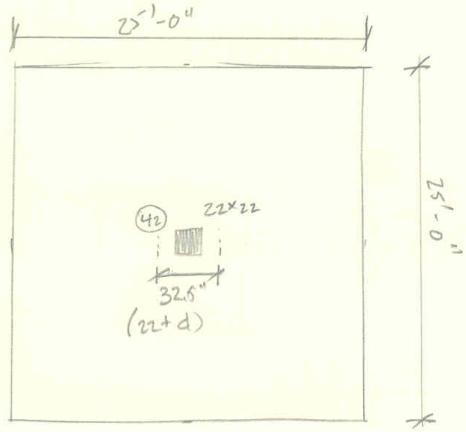
$A_s = \rho bd = 0.00125(9.5)(150) = 1.78 in^2$

$A_{s\,min} = 0.002(150)(10.5) = 3.15 in^2$

$N = \frac{1.78}{.2} = 8.9$ bars Actual design #4 @ 12" o.c. ⟶ 12.5 bars.

PUNCHING SHEAR

$W_u = 276.5$ PSF
$f'_c = 4000$ psi



$V_u = .2765\left[25' \times 25' - \left(\frac{32.5^2}{144}\right)\right] = 171^k$

$b_o = 4(32.5) = 130"$ $\qquad \frac{b_o}{d} = \frac{130"}{10.5"} = 12.4$

$V_c = 4\sqrt{f'_c}\, b_o d = 4\sqrt{4000}(130)(10.5)\frac{1}{1000} = 345^k$

$V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = (2+4)\sqrt{4000}(130)(10.5)\frac{1}{1000} = 518^k$ ×

$V_c = \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d = \left(\left(\frac{40}{130/10.5}\right) + 2\right)\sqrt{4000}(130)(10.5)\frac{1}{1000} = 451.6^k$ ×

$\phi V_c = 0.75(345^k) = 258.8^k > 171^k$ ok

Floor	Tributary Area ft^2	Dead Load PSF	L_o PSF	Influence Area ft^2	Reduction	Live Load Kips	Dead Load Kips
Roof	271.875	167	30	1087.5	1	8	45
13	271.875	132	40	1087.5	0.704859	16	81
12	543.75	132	40	2175	0.571634	20	117
11	815.625	132	40	3262.5	0.512613	25	153
10	1087.5	132	40	4350	0.477429	29	189
9	1359.375	132	40	5437.5	0.453419	33	224
8	1631.25	132	40	6525	0.435695	36	260
7	1903.125	132	40	7612.5	0.42192	40	296

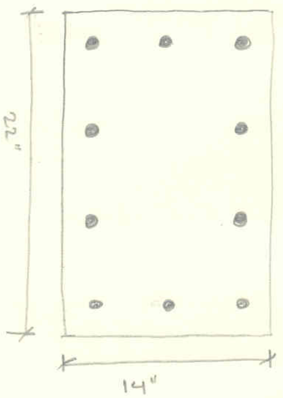
Column 68

L_1	18.75		
L_2	14.5	1.2D+1.6L	420

COLUMN GRAVITY CHECK

1/1

$P_u = 420^k$ ASSUME LOAD IS PURE AXIAL



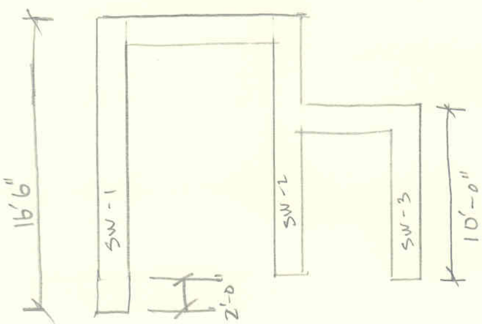
10-#9 bars

$$P_o = 0.85f'_c(bh - \Sigma A_s) + \Sigma A_s f_y$$
$$= 0.85(4)((22 \times 14) - 10(1.0)) + 10(1.0)(60)$$
$$= 1613^k$$
$$\phi P_o = 0.65(1613^k) = 1048^k$$
$$0.8\phi P_o = 0.8(1048) = \underline{\underline{838^k}} > 420^k$$

THE LARGE SIZE OF THIS COLUMN IS MOST LIKELY DUE TO THE INCREASED STIFFNESS REQUIRED IN THE TOWER TO LIMIT THE DRIFT OF THE STRUCTURE AS WELL AS AVOIDING A PUNCHING SHEAR FAILURE.

1/2

SHEAR WALL



* SYMMETRICAL
@ OTHER ELEVATOR CORE

ASSUME DISTRIBUTION BASED ON LENGTH OF WALL

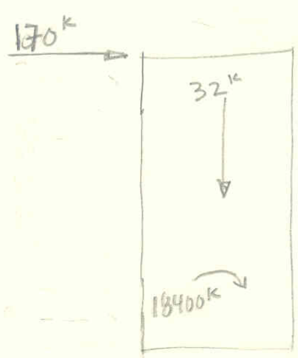
WALL	LENGTH	STIFFNESS	DISTRIBUTION
SW-1	16.5'	16.5/16.5 = 1.0	40%
SW-2	14.5'	14.5/16.5 = .879	35%
SW-3	10'	10/16.5 = .61	25%

SW-1 40% OF LOAD TO THESE WALLS → 20% EACH

CHECK SHEAR WALL @ 1ST FLOOR LEVEL

SW-1
1'-0" THICK
f'c = 4000psi

REINFORCEMENT
#4 @ 12" o.c. HORIZONTAL EA FACE
#6 @ 12" o.c. VERTICAL EA. FACE (ADDED LATER ORIG. #4@12)



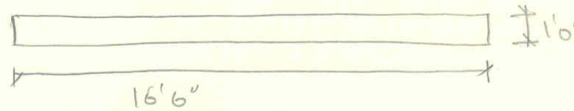
1ST FLOOR $V = 848(.2) = 169.6^k$
$M = 91,970(.2) = 18394'^k$

$P = (16.5)'(1')(174'0")(150\ PCF) \approx 430^k$
(ASSUME ONLY SW)

$C_v = \frac{P_u}{2} + \frac{M_u}{d} = \frac{430^k}{2} + \frac{18400}{16.5'} = 1330.5 = P_{u\,BE}$

2/2

BOUNDARY ELEMENTS



$A_g = (16.5')1'0" = 16.5 ft^2 \quad I_g = \frac{(16.5)^3(1)}{12} = 374.3 ft^4$

$f_c = \frac{P_u}{A_g} + \frac{M_u \frac{h_w}{2}}{I_g} = \frac{430^k}{16.5'} + \frac{18394\left(\frac{16.5}{2}\right)}{374.3} = 443.4 \text{ KSF} = 3.08^{ksi}$

$0.2(4^{ksi}) = .8^{ksi} < 3.08^{ksi}$ need boundary elements
(WILL BE INVESTIGATED IN LATER TECH. REPORT)

REINFORCEMENT

HORIZ.

$\rho_l, \rho_t = \frac{A_{sl}}{A_{cv}} \geq 0.0025$, $A_{cv} = 12" \times 12" = 144 in^2/ft$

$A_{s\,reqd.} = (0.0025)(144) = 0.36 in^2/ft$

ASSUME #4 in 2 curtains

$A_{sl} = 2(0.2) = 0.4 in^2/s \rightarrow 12"$ o.c.

ACTUAL DESIGN

#4 @ 12" o.c. OK

$\frac{h_w}{l_w} = \frac{174'}{16.5'} = 10.5 > 2 \quad \alpha_c = 2.0$

VERT.

$A_{cv} = (12)(16.5 \times 12) = 2376 in^2$

$\rho_t = \frac{2(.2)}{(12)(14)} = .00238$

$V_n = (2376)(2\sqrt{4000} + .00238 \times 60{,}000) \div 1000 = 639.8^k$

$\phi V_n = 0.6(639.8^k) = 383.9^k > V_u = 170^k$

2 - #4 @ 12" o.c. FOR BOTH DIRECTIONS

THIS MATCHES THE ACTUAL DESIGN REINFORCEMENT EXCEPT VERT. BARS WERE CHANGED TO #6 @ 12" o.c. I ASSUME THIS IS B/C OF AN INCREASE IN AXIAL LOAD.

1/1

SNOW LOAD ASCE 7-05

$P_g = 20$ PSF (Fig. 7.1)
$C_e = 1.0$ (TABLE 7-2) EXPOSURE B AND PARTIALLY EXPOSED
$C_t = 1.0$ (TABLE 7-3) HEATED STRUCTURE
$I = 1.0$

$P_f = 0.7 C_e C_t I p_g$
$0.7(1.0)(1.0)(1.0)\,20 \leq 20$

$P_f = 20$ PSF

ORIG. DESIGN USED 30 PSF

USE 30 PSF FOR CALCS

(* GROUND SNOW LOAD PER NYC BUILDING CODE IS HIGHER THAN THAT SET OUT IN ASCE 7-05)