TECHNICAL REPORT 2
BUILDING CODES, SPECIFICATIONS, AND LOADS

SECOND & STATE BUILDING

HARRISBURG PA

JADOT A MOOSMAN

STRUCTURAL OPTION

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Revision... 1

TABLE OF CONTENTS

1. EXECUTIVE SUMMARY FROM TECHNICAL REPORT 1

The purpose of Technical Report 1 was to develop an understanding of the existing structural system of the Second & State Building, a five-story, 56,000 SF office building located in downtown Harrisburg, Pennsylvania.

The structural system consists of an ordinary, pin-connected steel superstructure supported by concrete caissons transferring building loads to bedrock. Floor and roof gravity loads are supported by a concrete slab on composite steel deck, resting on wide-flange beams and girders and transferred to the foundation by wide-flange columns. Lateral resistance is provided by a combination of perimeter moment connections and concentrically braced frames, with loads being collected and transferred via the floor diaphragm.

A typical 30' x 36' bay was analyzed in greater detail. Wide flange beams ranging from W21x44 to W24x84 span in the 36' direction. Interior beams are supported by W24x76 and W24x84 girders spanning in the 30' direction, and edge beams (and girders) are supported by W12x120 and W12x152 columns. Beam connections are pinned, with the exception of the edge beam-to-column connections, which are moment connections serving as part of the lateral load resisting system. Floor loads are transferred to the framing members through a 5" composite slab formed on 20 gage Vulcraft 1.5VLI steel deck oriented in the 30' direction.

The Second & State Building was designed under the 2009 version of the Pennsylvania Uniform Construction Code (PUCC 2009), which adopts the 2009 International Building Code (IBC 2009), and, by reference, ASCE 7-05 for design loads, AISC 350-05 for steel design, and ACI 318-08 for concrete design.

Reduced plans for a typical floor, roof, mechanical penthouse, and typical braced frames are included in the appendix to Technical Report 1.

This report (Technical Report 2) contains research and calculations identifying and quantifying building loads for use in subsequent reports.

2. BUILDING DESCRIPTION & LOCATION

Constructed in 2012 and located one block from the State Capitol Complex, the Second & State Building is a five-story, steel frame structure housing retail on the ground level with four stories of office space above, for a total leasable area of approximately 56,000 square feet.

The Second & State Building was developed and is owned by WCI Partners of Harrisburg, PA. Architectural design services were provided by Bernardon Haber Holloway of Kennett Square, PA, with Baker, Ingram & Associates of Lancaster, PA completing structural design work. Warfel Construction of East Petersburg, PA provided construction management and served as the general contractor for the project.

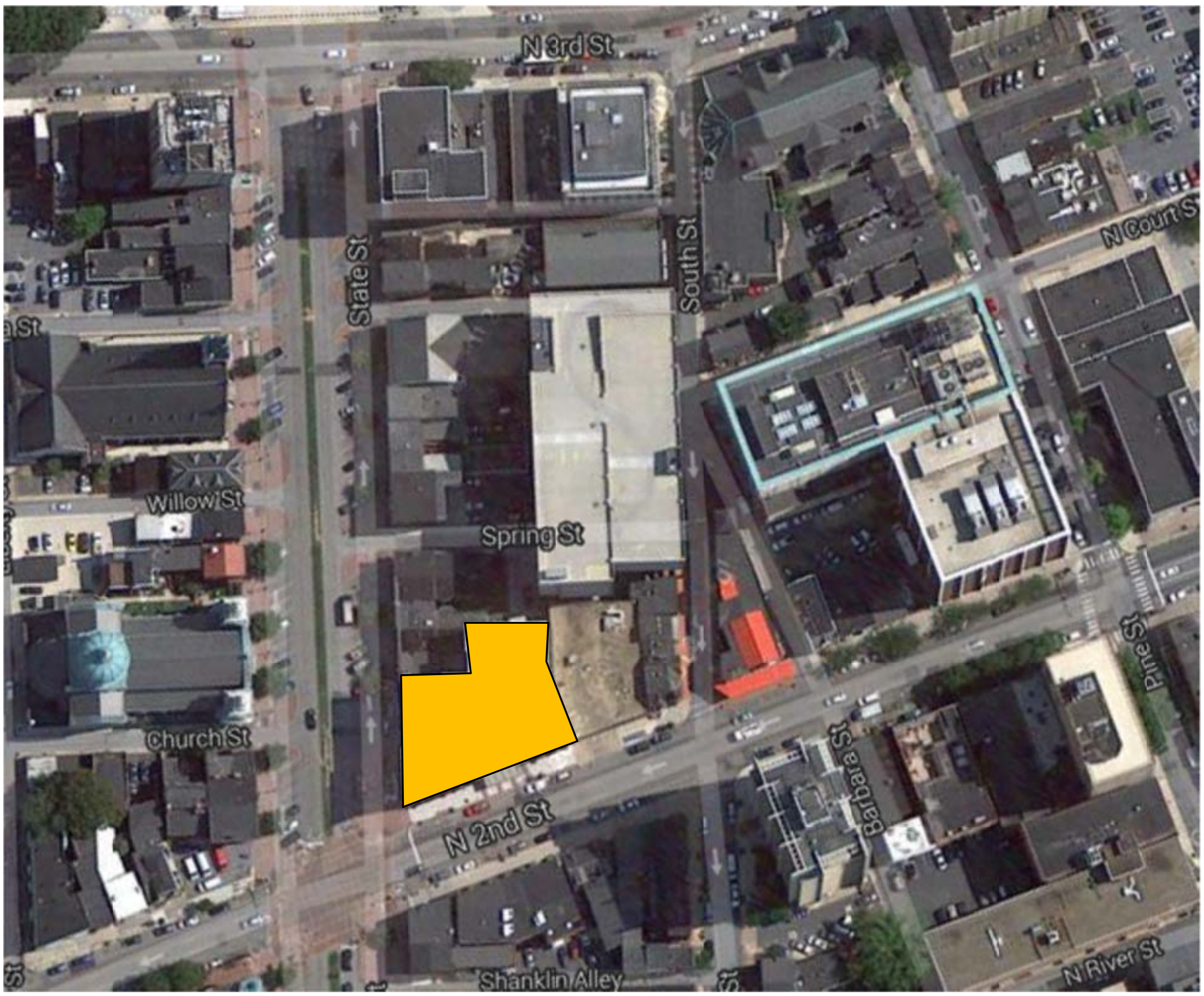


FIGURE 1 SATELLITE IMAGE OF BUILDING SITE (RETREIVED FROM GOOGLE MAPS 11 SEP 2013)

3. REFERENCED DOCUMENTS

Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)

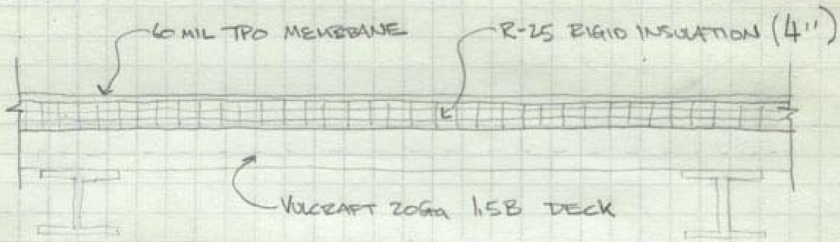
2006 International Building Code (IBC 2006)

Structural Load Determination Under 2006 IBC and ASCE/SEI 7-05 (Design Guide)

4. GRAVITY LOADS

ROOF LOADS 1/1

- SKETCH OF ROOF ASSEMBLY



- ROOF DEAD LOADS

60-MIL TPO REFLECTIVE MEMBRANE	0.31 PSF
R-25 RIGID INSULATION (4" @ 1.5 PSF/IN)	6.0 PSF
VULCRAFT 20Ga 1.5B ROOF DECK	3.54 PSF
CEILING & MISC MEP	15 PSF
TOTAL	24.8 PSF

- ROOF TYP. BAY FRAMING SELF-WEIGHT

W21×44 (6 @ 36'-0")	9504 #
W24×84 (1 @ 36'-0")	3024 #
W12×14 (4 @ 6'-0")	336 #
W24×76 (2 @ 30'-0")	4650 #
A325 BOLTS (APPROX 150 @ 0.95#)	142.5 #
MISC	250 #
	TOTAL = 17900
AREA = 30' × 36' = 1080 SF →	16.5 PSF

- ROOF LIVE LOAD

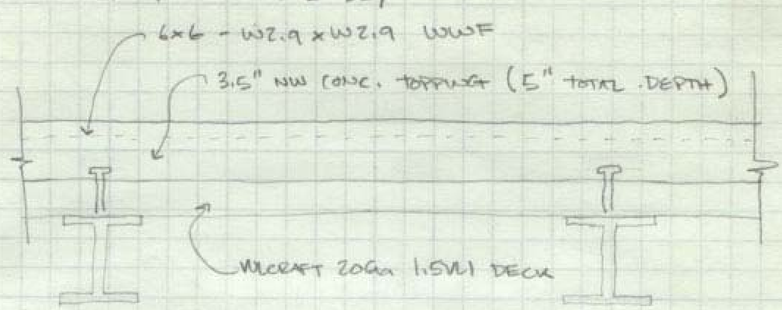
ASCE 7-05 CODE MINIMUM	20 PSF
VALUE USED IN DESIGN	30 PSF

- FLAT-ROOF SNOW LOAD

CALCULATED USING ASCE 7-05	21 PSF
└ MATCHES VALUE USED IN DESIGN	

FLOOR LOADS 1/1

- SKETCH OF FLOOR ASSEMBLY



- FLOOR DEAD LOADS

5" NORMAL WT SLAB ON 1.5VL1 DECK	51 PSF
VULCRAFT 20Ga 1.5VL1 FLOOR DECK	2.14 PSF
CARPET & PAD	2.0 PSF
CEILING & MISC MEP	15 PSF
TOTAL	70.2 PSF

- FLOOR TYP BAY FRAMING SELF WEIGHT

SAME AS ROOF, PLUS:	17900 #
STUDS (APPROX 200 @ 0.6 #)	120 #
	19100#
AREA = 30' x 36' = 1080 SF	→ 17.8 PSF

- FLOOR LIVE LOADS

	ASCE MIN.	DESIGN
OFFICES (FLOOR ONLY)	50 PSF	80 PSF ←
PARTITIONS	15 PSF	20 PSF
OFFICES (FLOOR + PARTITION)	65 PSF	100 PSF
CORRIDORS ABOVE 1ST FLOOR	80 PSF	N/A →
STAIRS	100 PSF	100 PSF
STORAGE AREAS (NEAR ELEVATORS)	250 PSF	250 PSF

SNOW LOADS 1/3

FLAT-ROOF SNOW LOAD — REF. IN ASCE 7-05

- DETERMINE GROUND SNOW LOAD (P_g)

  BY MAP, $P_g = 30$ PSF (FIG 7-1)

- DETERMINE EXPOSURE FACTOR (C_e)

  URBAN AREA ∴ SURFACE ROUGHNESS = "B" (§ 6.5.6.2)

  PARAPETS & OBSTRUCTIONS ∴ PARTIALLY EXPOSED (TABLE 7-2)

  ∴ $C_e = 1.0$ (TABLE 7-2)

- DETERMINE THERMAL FACTOR (C_t)

  DOES NOT MEET EXCEPTIONS IN TABLE 7-3

  ∴ $C_t = 1.0$ (TABLE 7-3)

- DETERMINE SNOW LOAD IMPORTANCE FACTOR (I)

  NON-ESSENTIAL OFFICE ∴ CATEGORY II (TABLE 1-1)

  ∴ $I = 1.0$ (TABLE 7-4)

- CALCULATE ALTERNATIVE MINIMUM SNOW LOAD ($P_{f,min}$)

  $P_g = 30 \text{ PSF} > 20 \text{ PSF}$

  ∴ $P_{f,min} = 20I = 20(1.0) = 20$ PSF (§ 7.3)

- CALCULATE FLAT-ROOF SNOW LOAD (P_f)

  $P_f = 0.7 C_e C_t I P_g \geq P_{f,min}$ (EQ 7-1)

  $= 0.7(1.0)(1.0)(1.0)(30) \geq 20$

  $21 \geq 20$ ✓ OK

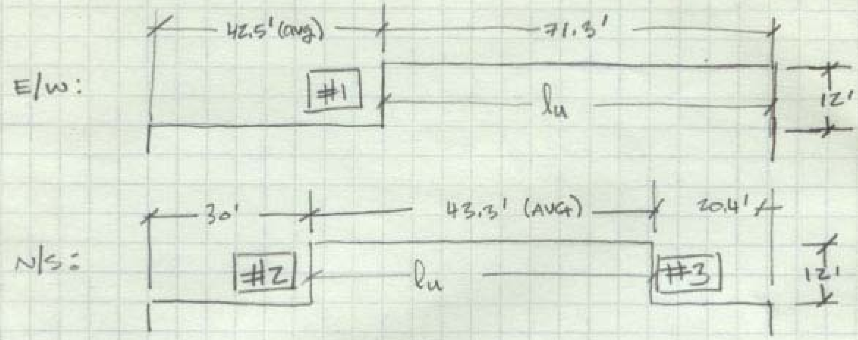
  ∴ $P_f = 21$ PSF

- COMPARE TO EXISTING DESIGN

  CALCULATED FLAT-ROOF SNOW LOAD MATCHES VALUE LISTED IN STRUCTURAL NOTES ✓ OK

SNOW LOADS 2/3

SNOW DRIFT LOADS



° DETERMINE SNOW DENSITY (γ)

$\gamma = 0.13\,P_g + 14 \le 30$ PCF (EQ 7-3)

$= (0.13)(30) + 14 = 17.9 \le 30$

$\therefore \gamma = 17.9$ PCF

° DETERMINE HEIGHT OF BALANCED SNOW LOAD (h_b)

$h_b = \frac{P_f}{\gamma} = \frac{21}{17.9} = 1.17'$ (§ 7.7.1)

° DETERMINE CLEAR HEIGHT ABOVE BALANCED SNOW LOAD (h_c)

$h_c = h_{penthouse} - h_b$ (§ 7.7.1)

$= (12) - (1.17)$

$\therefore h_c = 10.83'$

° DETERMINE SNOW DRIFT HEIGHTS (h_d)

#1) LEEWARD ($l_u = 71.3'$)

$h_d = 0.43 \sqrt[3]{l_u}\, \sqrt[4]{P_g + 10} - 1.5$ (FIG 7-9)

$= 0.43 \sqrt[3]{71.3}\, \sqrt[4]{30+10} - 1.5 = 2.98'$

WINDWARD ($l_u = 42.5'$)

$h_d = \frac{3}{4}\, 0.43 \sqrt[3]{l_u}\, \sqrt[4]{P_g + 10} - 1.5$

$= \frac{3}{4}\, 0.43 \sqrt[3]{42.5}\, \sqrt[4]{30+10} - 1.5 = 1.33'$

$\therefore$ LEEWARD CONTROLS $\rightarrow h_{d\#1} = 2.98'$

SNOW LOADS 3/3

#2) LEEWARD ($l_u = 43.3'$)

$h_d = 0.43 \sqrt[3]{43.3} \sqrt[4]{30+10} - 1.5 = 2.30'$

WINDWARD ($l_u = 30'$)

$h_d = \frac{3}{4} 0.43 \sqrt[3]{30} \sqrt[4]{30+10} - 1.5 = 1.86'$

∴ LEEWARD CONTROLS → $h_{d\#2} = 2.30'$

#3) LEEWARD ($l_u = 43.3'$)

$h_d = 0.43 \sqrt[3]{43.3} \sqrt[4]{30+10} - 1.5 = 2.30'$

WINDWARD ($l_u = 20.4'$)

$h_d = \frac{3}{4} 0.43 \sqrt[3]{20.4} \sqrt[4]{30+10} - 1.5 = 0.586'$

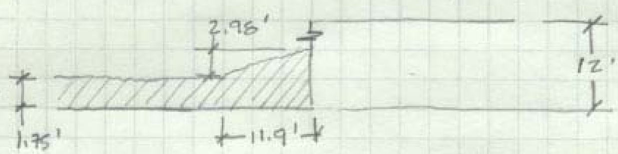
∴ LEEWARD CONTROLS → $h_{d\#3} = 2.30'$

• DETERMINE SNOW DRIFT WIDTHS (w)

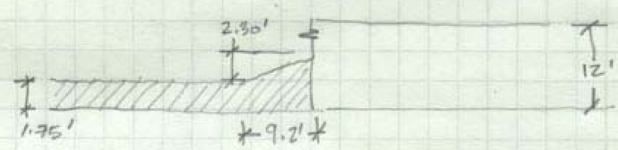
IN ALL CASES, $h_d < h_c = 10.83'$

∴ $w = 4h_d$ (§ 7.7.1)

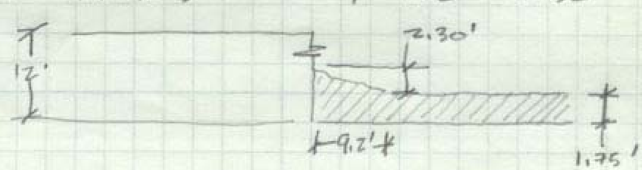
#1) $w = 4(2.98) = 11.9'$, $h_d = 2.98'$



#2) $w = 4(2.30') = 9.20'$, $h_d = 2.30'$

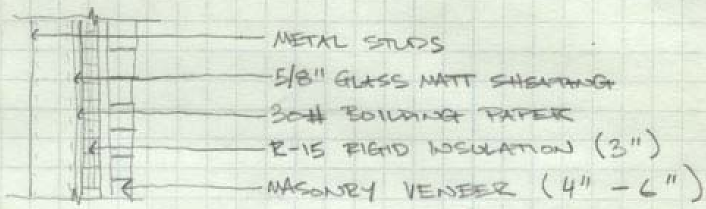


#3) $w = 4(2.30') = 9.20'$, $h_d = 2.30'$



WALL LOADS 1/1

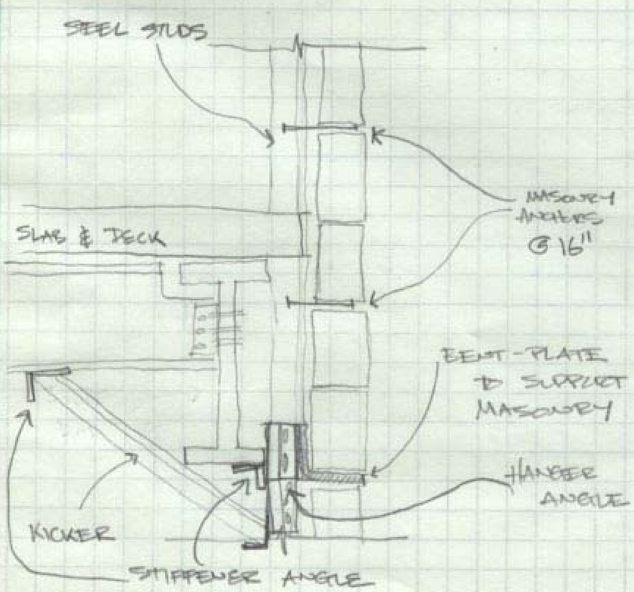
- SKETCH OF TYPICAL EXTERIOR WALL ASSEMBLY



- WALL DEAD LOADS

METAL STUDS @ 16" OC	2 PSF
5/8" GLASS MATT SHEATHING	2.8 PSF
30# BUILDING PAPER	0.3 PSF
R-15 RIGID INSULATION (3" @ 1.5 PSF/IN)	4.5 PSF
MASONRY VENEER (AVG 5" @ 15 PSF/IN)	75 PSF
MISC LINTELS, KICKERS, PLATES, ETC	10 PSF
TOTAL	94.6 PSF

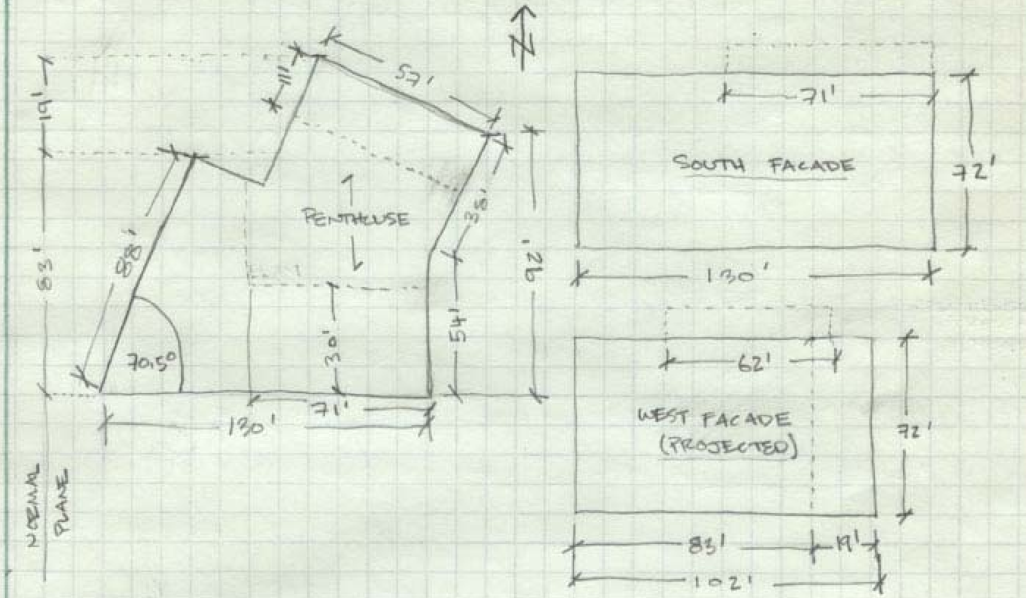
- WALL DEAD LOAD PATH



5. WIND LOADS

WIND LOADS 1/6

NOTE: USE ANALYTICAL PROCEDURE OF ASCE 7-05 § 6.5



- FOR WIND FROM SOUTH:

$$L_{(AVG)} = \frac{83 + 102 + 92}{3} = 92.3'$$

B = 130'

h = 72'

A = 9360 SF

- FOR WIND FROM WEST:

$$L_{(AVG)} = \frac{130 + 119 + 71}{3} = 106.7'$$

B = 102'

h = 72'

A = 7340 SF

WIND LOADS 2/6

REF. IN ASCE 7-05 ↓

- DETERMINE BASIC WIND SPEED (V)

  BY MAP, V = 90 MPH (FIG. 6-1)

- DETERMINE IMPORTANCE FACTOR (I)

  NON-ESSENTIAL OFFICE ∴ CATEGORY II (TABLE 1-1)

  V = 90 MPH ∴ I = 1.0 (TABLE 6-1)

- DETERMINE EXPOSURE CATEGORY

  URBAN AREA ∴ SURFACE ROUGHNESS = "B" (§6.5.6.2)

  ROUGHNESS "B" CONTINUES FOR MILES TO WEST, BUT RIVER IS ONE BLOCK (<2600') TO THE SOUTH. (§6.5.6.3)

  ∴ SOUTH = EXPOSURE "C"
  WEST = EXPOSURE "B"

- DETERMINE RIGIDITY OF STRUCTURE

  APPROXIMATE FUNDAMENTAL FREQUENCY (n_1)

  METHOD 1: $n_1 = \frac{22.2}{H^{0.8}} = 0.73$ Hz (FLEXIBLE) (EQ. C6-14)

  METHOD 2: $n_1 = \frac{100}{H} = 1.34$ Hz (RIGID) (EQ C6-17)

  $n_1 = \frac{75}{H} = 1.04$ Hz (") (EQ C6-18)

  METHOD 3: $n_1 = \frac{150}{H} = 2.08$ Hz (") (EQ C6-19)

  METHOD 4: $n_1 = \frac{164}{H} = 2.28$ Hz (") (EQ C6-21)

  RIGID OR FLEXIBLE? THE MORE-CONSERVATIVE METHODS FROM THE COMMENTARY (C6-14 - 18) YEILD SPLIT RESULTS. I WILL CLASSIFY THE STRUCTURE AS "RIGID", AS DID THE PROJECT ENGINEERS.

- DETERMINE GUST EFFECT FACTOR

  RIGID STRUCTURE ∴ G = 0.85 (§6.5.8.1)

- DETERMINE ENCLOSURE CLASSIFICATION

  NO OPERABLE WINDOWS OR OTHER OPENINGS

  ∴ ENCLOSED BUILDING (§6.2)

WIND LOADS 3/6

- DETERMINE TOPOGRAPHIC FACTOR (K_{zt})

SITE CONDITIONS DO NOT MEET § 6.5.7.1

$\therefore K_{zt} = 1.0$ (§6.5.7.2)

- DETERMINE WIND DIRECTIONALITY FACTOR (K_d)

BUILDING $\therefore K_d = 0.85$ (TABLE 6-4)

- CALCULATE VELOCITY PRESSURES (q_z)

<u>WEST</u> (EXPOSURE "B")

$\alpha = 7.0 \quad z_g = 1200'$ (TABLE 6-2)

EXAMPLE CALCULATION AT 2ND FLOOR ($z = 16.5'$):

$K_{16.5} = 2.01\left[\frac{z}{z_g}\right]^{2/\alpha} = 2.01\left[\frac{16.5}{1200}\right]^{2/7} = 0.591$ (TABLE 6-3)

$q_{16.5} = 0.00256 K_z K_{zt} K_d V^2 I$ (EQ. 6-15)

$= 0.00256(0.591)(1.0)(0.85)(90)^2(1.0)$

$= 10.4$ PSF

<u>SOUTH</u> (EXPOSURE "C")

$\alpha = 9.5 \quad z_g = 900'$

EXAMPLE CALCULATION AT 2ND FLOOR ($z = 16.5'$)

$K_{16.5} = 2.01\left[\frac{z}{z_g}\right]^{2/\alpha} = 2.01\left[\frac{16.5}{900}\right]^{2/9.5} = 0.867$

$q_{16.5} = 0.00256 K_z K_{zt} K_d V^2 I$

$= 0.00256(0.867)(1.0)(0.85)(90)^2(1.0)$

$= 15.3$ PSF

VELOCITY PRESSURES FOR SUBSEQUENT LEVELS CALCULATED USING SAME METHOD & TABULATED WITH OTHER RESULTS AT END

WIND LOADS 4/6

- DETERMINE INTERNAL PRESSURE COEFFICIENT (GC_{pi})

ENCLOSED ∴ $GC_{pi} = \pm 0.18$ PSF (FIG 6-5)

- DETERMINE EXTERNAL PRESSURE COEFFICIENTS (C_p)

	WEST	SOUTH	
L/B	1.05	0.71	
WINDWARD WALL	0.80	0.80	(FIG 6-6)
LEEWARD WALL	-0.49	-0.50	(FIG 6-6)
SIDE	-0.70	-0.70	(FIG 6-6)
h/L	0.67	0.78	
ROOF (0 to h/2)	-1.04	-1.12	(FIG 6-6)
(> h/2)	-0.44	-0.52	(FIG 6-6)

- CALCULATE DESIGN WIND PRESSURES (P_z)

WEST (WINDWARD)

ENCLOSED ∴ $q_i = q_h$ ($h = 72'$) (§6.5.12.2)

$K_{72'} = 2.01\left[\frac{72}{1200}\right]^{2/7} = 0.899$

$q_{72} = 0.00256(0.899)(1.0)(0.85)(90)^2(1.0) = 15.8 \text{ PSF} = q_i$

EXAMPLE CALCULATION AT 2ND FLOOR ($z = 16.5'$)

$P_{16.5} = qGC_p - q_i(GC_{pi})$ (EQ 6-17)

$= (10.4)(0.85)(0.8) \pm (15.8)(0.18)$

$= 4.23$ PSF

SOUTH (WINDWARD)

$K_{72} = 2.01\left[\frac{72}{900}\right]^{2/9.5} = 1.18$

$q_{72} = 0.00256(1.18)(1.0)(0.85)(90)^2(1.0) = 20.8 \text{ PSF} = q_i$

EXAMPLE CALCULATION AT 2nd FLOOR ($z = 16.5'$)

$P_{16.5} = (15.3)(0.85)(0.8) \pm (20.8)(0.18)$

$= 6.67$ PSF

WIND LOADS 5/6

- DESIGN WIND PRESSURES ON WEST FACADE

	FT $h=z$	K_z	PSF q_z	PSF P_z (WW)	PSF P_z (LW)	SF A_T	K F	ft·k M_{OT}
L1	0'-0"	0.575	10.1	9.71	-9.56	1683	32.4	0
L2	16'-6"	0.591	10.4	9.92		1357	26.4	436.1
L3	29'-10"	0.699	12.3	11.2		1357	28.2	840.8
L4	43'-2"	0.777	13.7	12.3		1357	29.5	1272
L5	56'-6"	0.839	14.8	12.9		1357	30.5	1723
LR	71'-6"	0.898	15.8	13.6		1530	35.4	2532
PH	84'-5"	0.942	16.6	14.1	-9.56	806	19.1	1612
							202	8416

- DESIGN WIND PRESSURES ON SOUTH FACADE

	FT $h=z$	K_z	PSF q_z	PSF P_z (WW)	PSF P_z (LW)	SF A_T	K F	ft·k M_{OT}
L1	0'-0"	0.849	14.9	13.9	-12.6	2145	56.8	0
L2	16'-6"	0.867	15.3	14.1		1729	46.2	763.1
L3	29'-10"	0.981	17.3	15.5		1729	48.6	1450
L4	43'-2"	1.06	18.7	16.5		1729	50.2	2169
L5	56'-6"	1.12	19.7	17.1		1729	51.4	2905
LR	73'-6"	1.19	21.0	18.0		1950	59.7	4389
PH	84'-5"	1.22	21.5	18.4	-12.6	923	28.6	2412
							342	14090

- DESIGN WIND PRESSURES ON ROOF

WIND FROM WEST

$q_h = 15.8$ PSF ; $\frac{h}{2} = 36'$

0 TO 36' FROM EAVE ($c_p = -1.04$)

$P_{roof} = (15.8)(0.85)(-1.04) \pm (15.8)(0.18) = -16.8$ PSF

36' TO 130' FROM EAVE ($c_p = -0.44$)

$P_{roof} = (15.8)(0.85)(-0.44) \pm (15.8)(0.18) = -8.74$ PSF

WIND FROM SOUTH

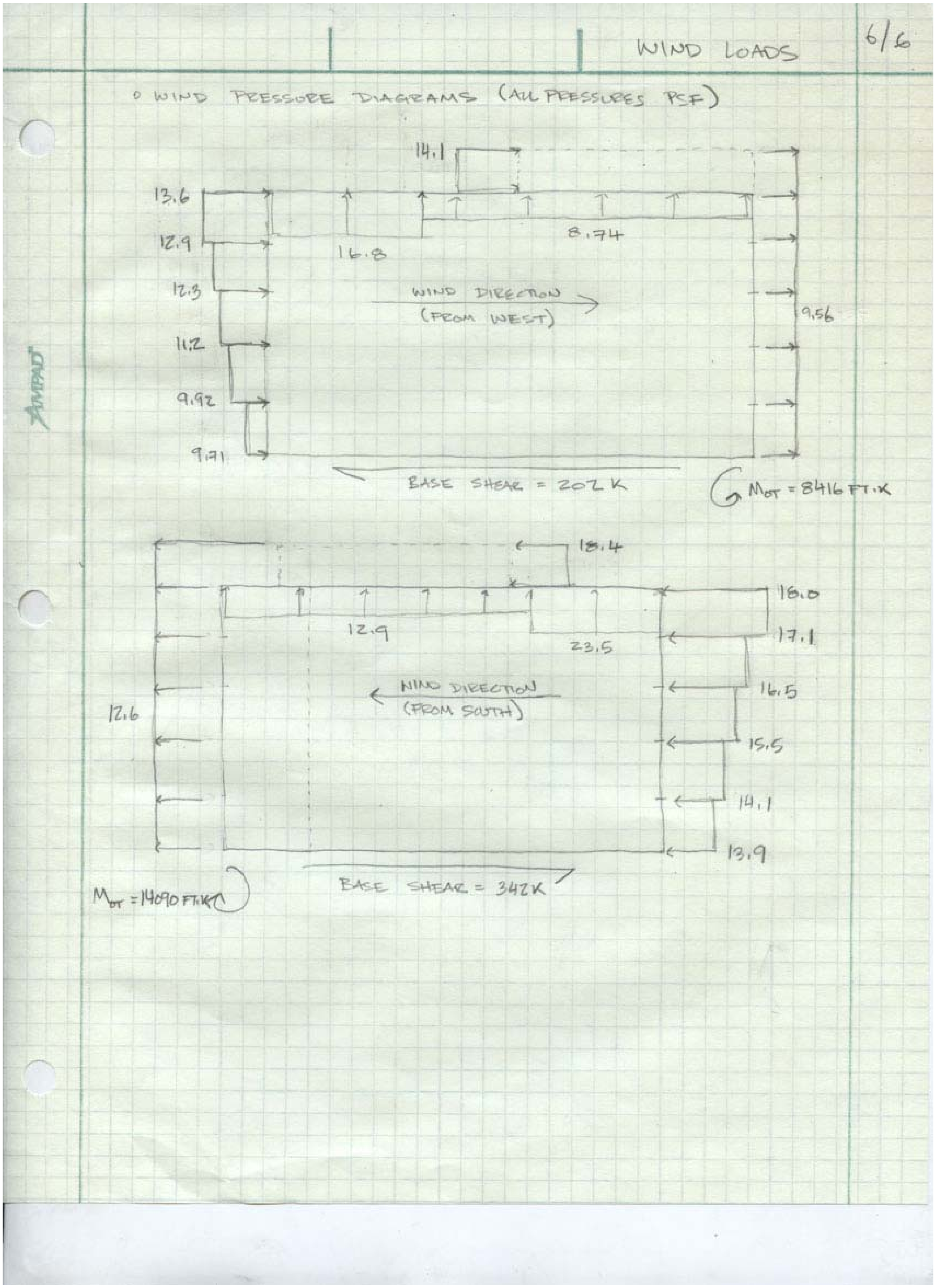
$q_h = 20.8$ PSF ; $\frac{h}{2} = 36'$

0 TO 36' FROM EAVE ($c_p = -1.12$)

$P_{roof} = (20.8)(0.85)(-1.12) \pm (20.8)(0.18) = -23.5$ PSF

36' TO 102' FROM EAVE ($c_p = -0.52$)

$P_{roof} = (20.8)(0.85)(-0.52) \pm (20.8)(0.18) = -12.9$ PSF


WIND LOADS
6/6
o WIND PRESSURE DIAGRAMS (ALL PRESSURES PSF)
14.1
13.6
12.9
12.3
11.2
9.92
9.71
16.8
8.74
WIND DIRECTION (FROM WEST)
9.56
BASE SHEAR = 202 K
M_OT = 8416 FT.K
18.4
16.0
17.1
16.5
15.5
14.1
13.9
12.9
23.5
12.6
WIND DIRECTION (FROM SOUTH)
BASE SHEAR = 342 K
M_OT = 14090 FT.K

6. SEISMIC LOADS

SEISMIC LOADS 1/2

- SITE CLASSIFICATION & LOCATION

SITE CLASS = "B" (ROCK, PER GEOTECH REPORT)

OCCUPANCY CATEGORY = "II" (NON-ESSENTIAL OFFICE)

LOCATION: 40.26 N 76.88 W

IMPORTANCE FACTOR = 1.0 (TABLE 11.5-1)

- SPECTRAL RESPONSE ACCELERATIONS/COEFFICIENTS

(USGS VALUES FROM USGS US SEISMIC DESIGN MAPS)

	USGS	DESIGN	
S_S	0.188g	0.190g	
S_1	0.053g	0.053g	
$S_{MS} = F_a S_s$	0.188g	—	$F_a = 1.0$
$S_{M1} = F_v S_1$	0.053g	—	$F_v = 1.0$
$S_{DS} = \frac{2}{3} S_{MS}$	0.126g	0.127	
$S_{D1} = \frac{2}{3} S_{M1}$	0.035g	0.035	

NOTE: USGS VALUES USED IN CALCULATIONS

- DETERMINE APPROXIMATE FUNDAMENTAL PERIOD (T_a)

$T_a = C_t h_n^x$ (EQ 12.8-7)

where $C_t = 0.028$ (STEEL MOMENT FRAME) (TABLE 12.8-2)

$h_n = 84.4'$ (PENTHOUSE ROOF HT)

$x = 0.8$ (STEEL MOMENT FRAME) (TABLE 12.8-2)

$T_a = (0.028)(84.4)^{0.8} = 0.973$

- DETERMINE SEISMIC DESIGN CATEGORY

$S_1 = 0.053 < 0.75$ (§11.6)

CHECK $T_a < 0.8 T_s$: $0.973 < 0.8 \frac{0.035}{0.126} = 0.278$ ✗ NO GOOD

∴ USE TABLE 11.6-2 ∴ CATEGORY = "A"

SEISMIC LOADS 2/2

- DETERMINE DEAD LOAD & FORCE PER LEVEL

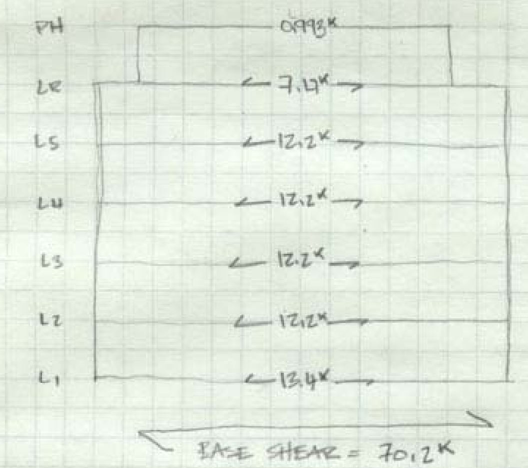
	FT	SF			KIPS		FT·K
	h_x	A_{FLOOR}	A_{ROOF}	A_{FACADE}	W_x	F_x	M_{OT}
L1	0'-0"	10076		6683	1340	13.4	0
L2	16'-6"	10076		5386	1217	12.2	200.8
L3	29'10"	10076		5386	1217	12.2	363.0
L4	43'2"	10076		5386	1217	12.2	525.3
L5	56'-6"	10076		5386	1217	12.2	687.5
LR	73'6"	4005	6071	3015	716.9	7.17	526.9
PH	84'-5"		4005		99.3	0.993	83.63
BASE					7023	70.23	2387

NOTES: FLOOR DL = 70.2 PSF P_{FLOORS} = 405'
ROOF DL = 24.8 PSF P_{PENTHS} = 201'
WALL DL = 94.6 PSF

FORCES PER ASCE 7-05 § 11.7.2 ($F_x = 0.01\, W_x$)

CALCULATED BASE SHEAR (70.2 K) IS SUBSTANTIALLY LARGER THAN DESIGN VALUE (57 K), SUGGESTING DEAD LOAD VALUES MAY BE TOO CONSERVATIVE.

- SEISMIC LOADING VS HEIGHT



M_{OT} = 2387 FT·K