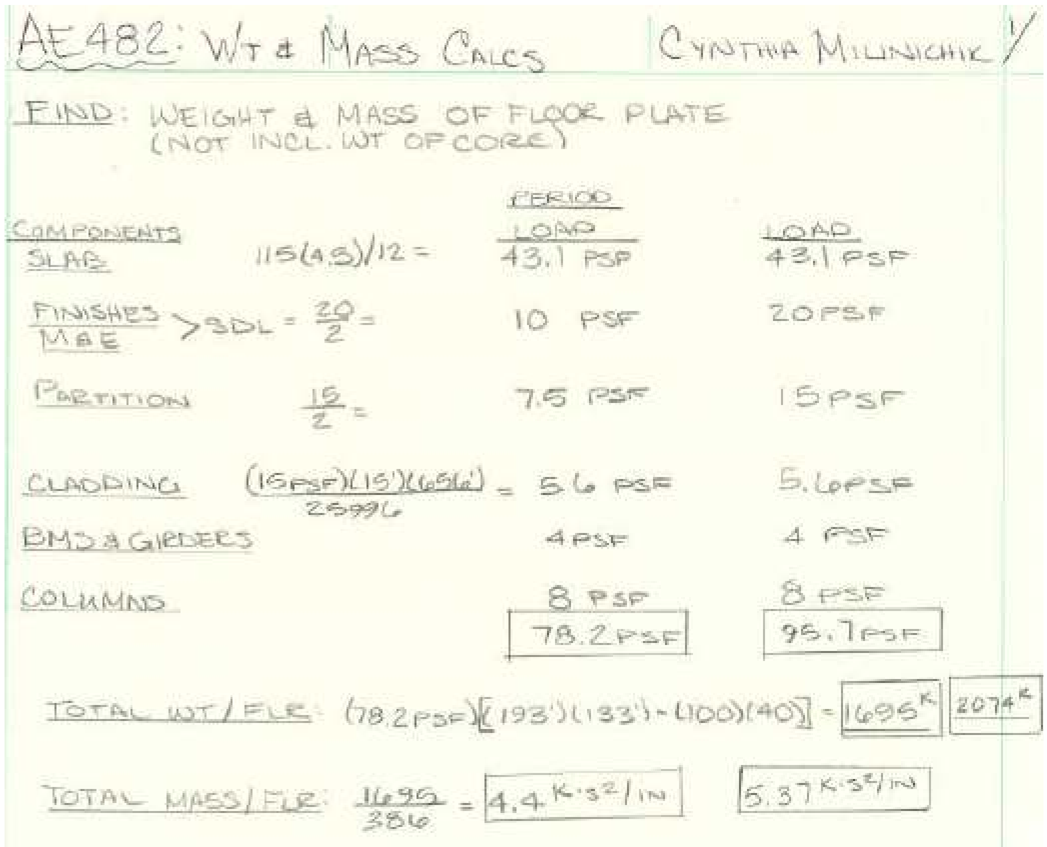
H. CALCULATION APPENDIX

H.1 GENERAL PROPERTY CALCULATIONS FOR PERIOD OF VIBRATION CALCULATIONS



AE482: WT & MASS CALCS — CYNTHIA MILINICHIK 1/

FIND: WEIGHT & MASS OF FLOOR PLATE (NOT INCL. WT OF CORE)

COMPONENTS		PERIOD LOAD	LOAD
SLAB	115(4.5)/12 =	43.1 PSF	43.1 PSF
FINISHES, MEE > SDL	= 20/2 =	10 PSF	20 PSF
PARTITION	15/2 =	7.5 PSF	15 PSF
CLADDING	(15PSF)(15')(656')/25996 =	5.6 PSF	5.6 PSF
BMS & GIRDERS		4 PSF	4 PSF
COLUMNS		8 PSF	8 PSF
		78.2 PSF	95.7 PSF

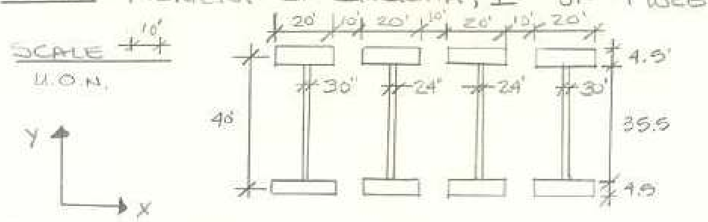
TOTAL WT/FLR: (78.2PSF)[(193')(133')-(100)(40)] = 1695^K | 2074^K

TOTAL MASS/FLR: 1695/386 = 4.4 K·s²/IN | 5.37 K·s²/IN

FIGURE H-1.1: WEIGHT AND MASS CALCULATIONS

AE 482: CYNTHIA MILINICHIK

FIND: MOMENT OF INERTIA, I OF 4 WEB



$I = I_{CM} + Ad^2 \qquad I_{CM} = \frac{bh^3}{12}$

FOR 24" WEB

$I_1 = \frac{(24)(35.5(12))^3}{12} = 154{,}617{,}552 \text{ IN}^4 = 7456 \text{ FT}^4$

$2 \cdot I_2 = 2\left[\frac{(20(12))(54)^3}{12} + (20)(12)(54)(240^2)\right] = 1{,}499{,}290{,}560 \text{ IN}^4$
$= 72304 \text{ FT}^4$

$I_{TOT_{24}} = 79{,}760 \text{ FT}^4$

FOR 30" WEB

$I_1 = \frac{(30)(35.5(12))^3}{12} = 193{,}271{,}940 = 9320 \text{ FT}^4$

$I_2 = 72304 \text{ FT}^4$

$I_{TOT_{30"}} = 81624 \text{ FT}^4$

FOR 4 WEB CONFIGURATION: $I_{4 WEB} = 322{,}768 \text{ FT}^4$

FOR 18" WEB

$I_3 = \frac{(18)(35.5)(12)^3}{12} = 115{,}963{,}164 \text{ IN}^4 = 5592 \text{ FT}^4$

$I_{TOT_{18"}} = 5592 + 72304 = 77896 \text{ FT}^4$

FOR 6 WEB CONFIGURATION: $I_{6 WEB} = 471{,}104 \text{ FT}^4$

$T = \frac{2\pi}{\omega}$

$\omega = \sqrt{\frac{k}{m}}$

$k \sim EI$

FIGURE H-1.2: MOMENT OF INERTIA CALCULATIONS

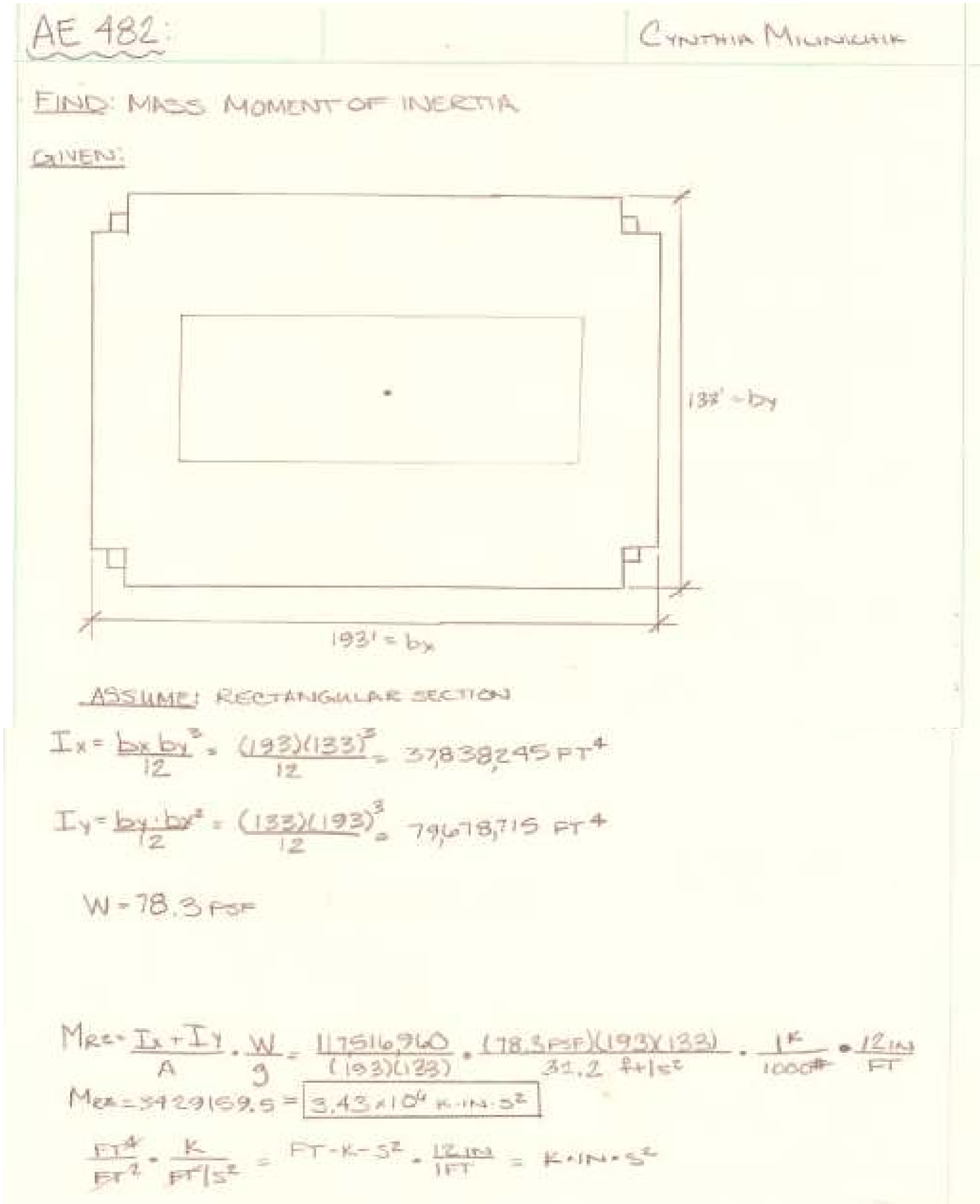


FIGURE H-1.3: MASS MOMENT OF INERTIA CALCULATION

H.2 PCA COLUMN INPUT CALCULATIONS

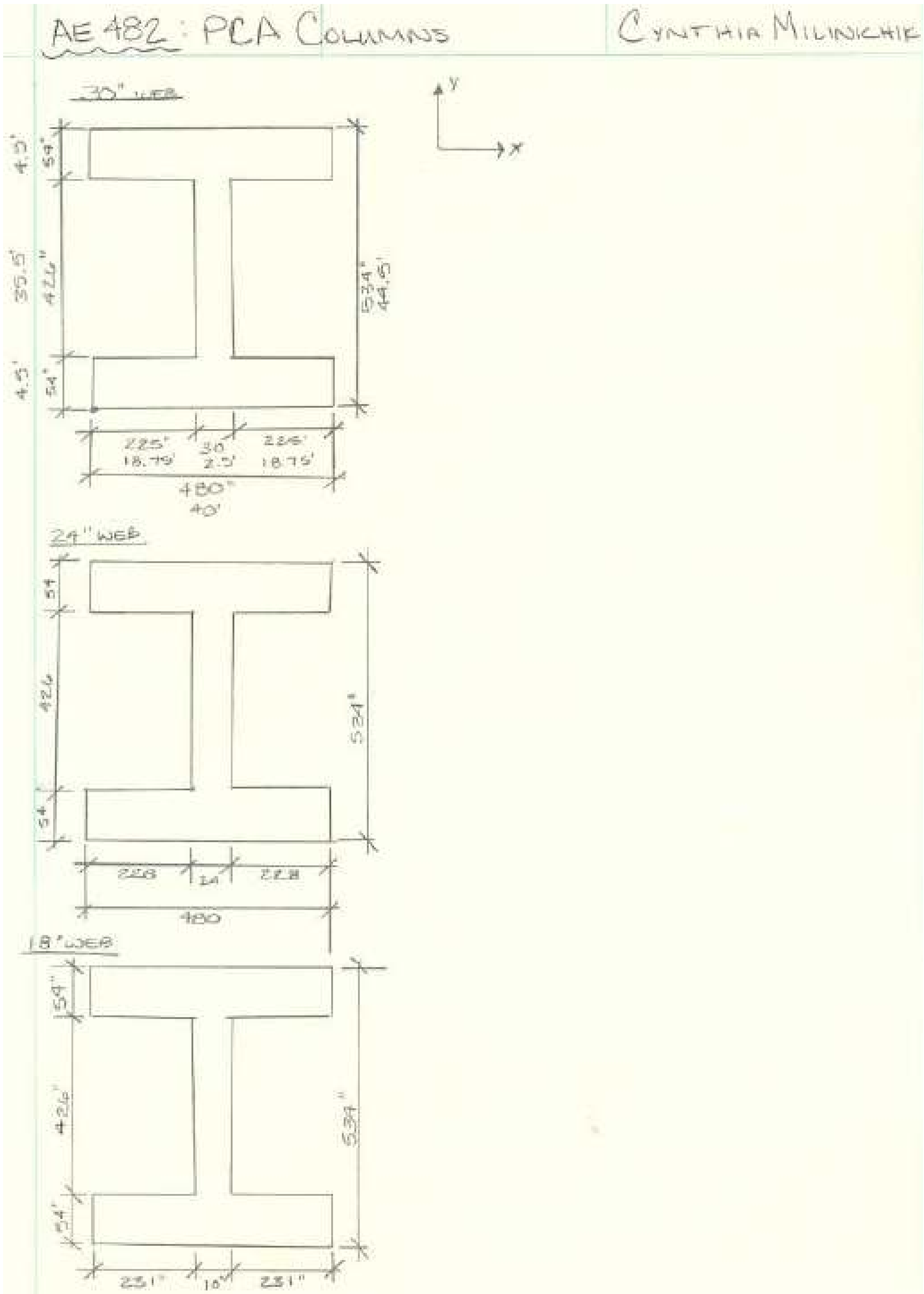
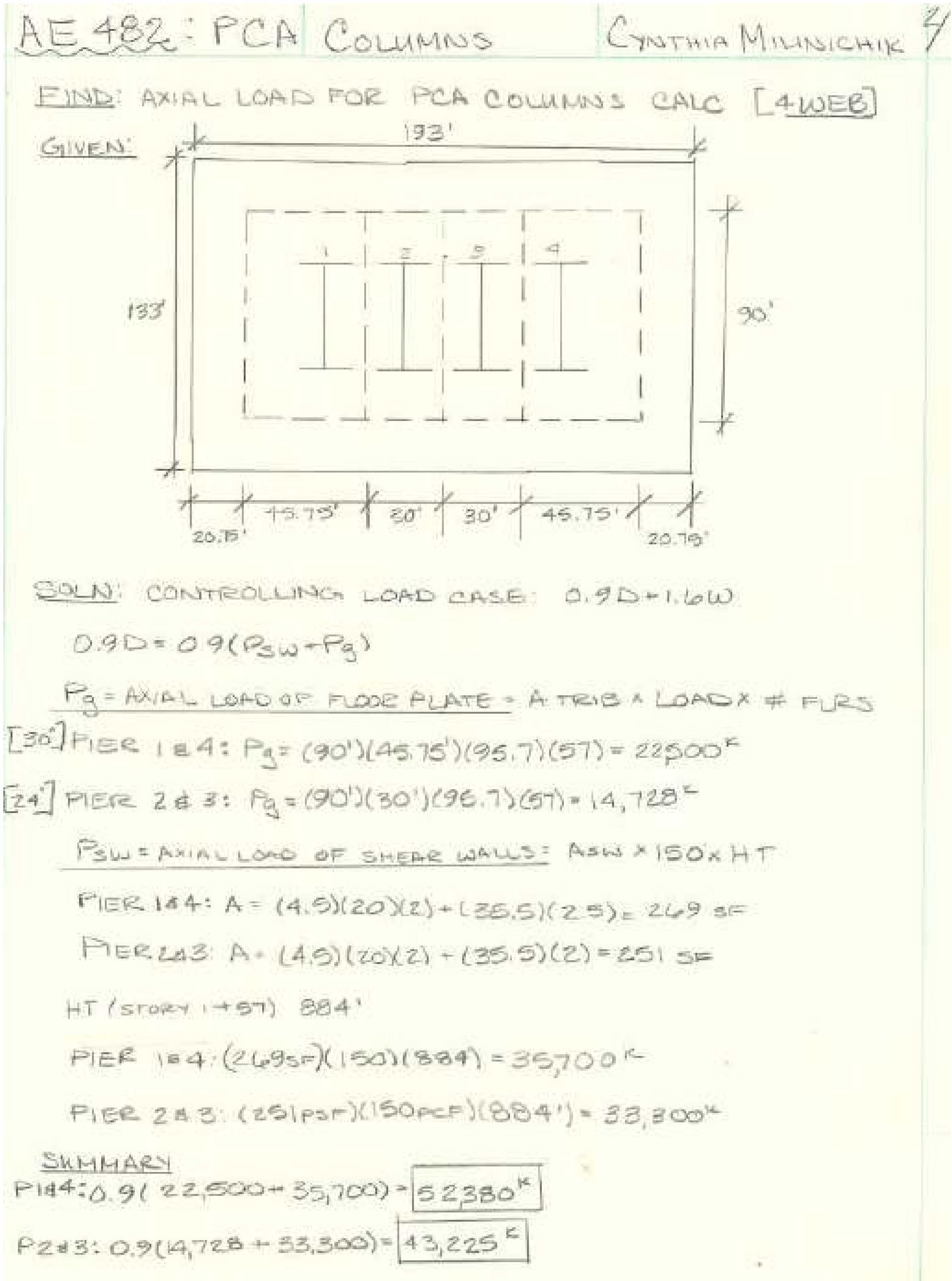


FIGURE H-2.1: DIMENSIONS FOR PCA COLUMN INPUT



AE 482: PCA COLUMNS CYNTHIA MILINICHIK 4

FIND: AXIAL LOAD FOR PCA COLUMNS CALC [4 WEB]

GIVEN:

SOLN: CONTROLLING LOAD CASE: 0.9D+1.6W

0.9D = 0.9(Psw + Pg)

Pg = AXIAL LOAD OF FLOOR PLATE = A TRIB × LOAD × # FLRS

[30"] PIER 1 & 4: Pg = (90')(45.75')(95.7)(57) = 22,500^K

[24"] PIER 2 & 3: Pg = (90')(30')(95.7)(57) = 14,728^K

Psw = AXIAL LOAD OF SHEAR WALLS: Asw × 150 × HT

PIER 1&4: A = (4.5)(20)(2) + (35.5)(2.5) = 269 SF

PIER 2&3: A = (4.5)(20)(2) + (35.5)(2) = 251 SF

HT (STORY 1→57) 884'

PIER 1&4: (269SF)(150)(884) = 35,700^K

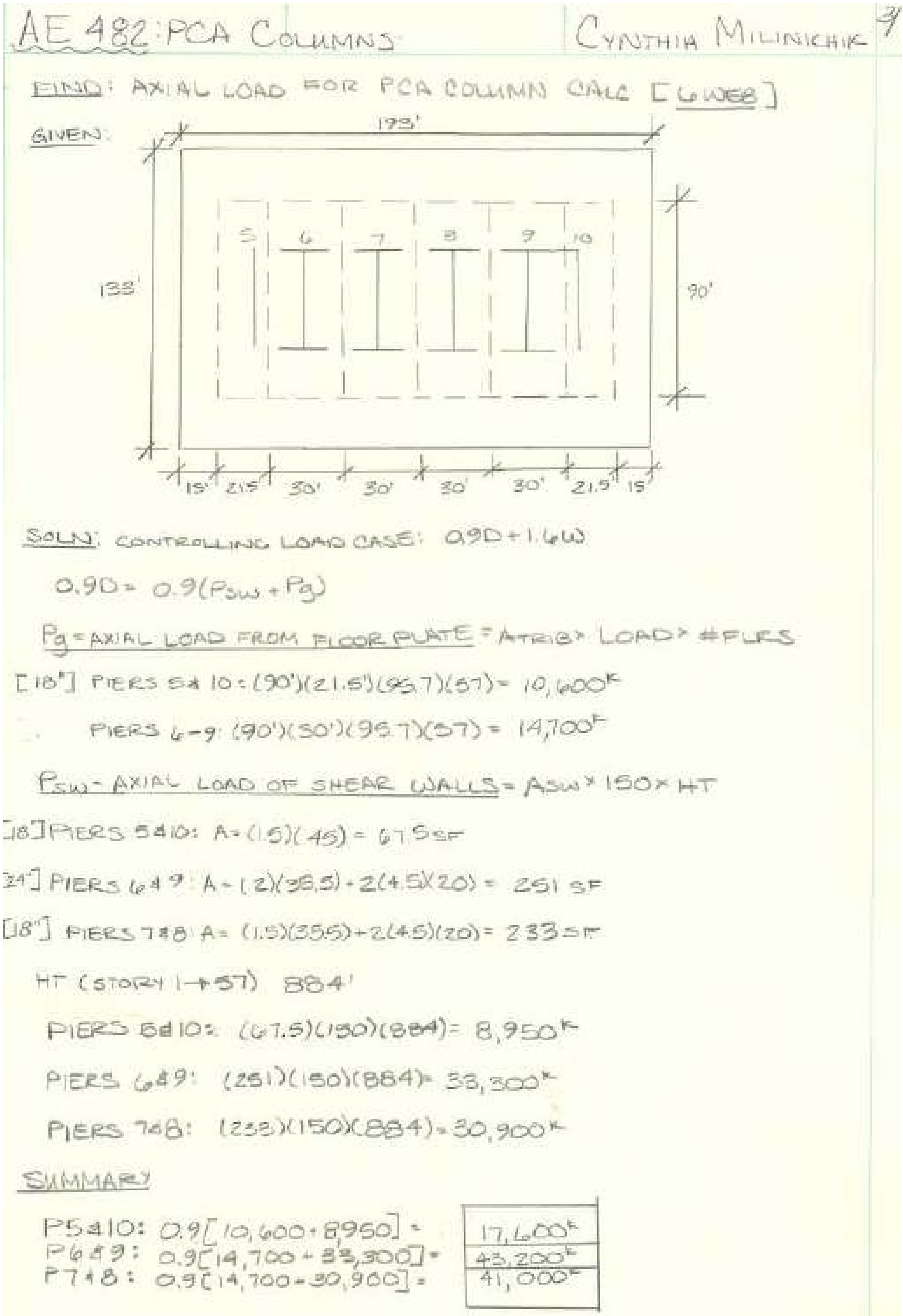
PIER 2&3: (251psf)(150pcf)(884') = 33,300^K

SUMMARY

P1&4: 0.9(22,500 + 35,700) = 52380^K

P2&3: 0.9(14,728 + 33,300) = 43,225^K

FIGURE H-2.2 AXIAL LOADS FOR 4 WEB SYSTEM FOR PCA COLUMN INPUT



AE 482: PCA COLUMNS — CYNTHIA MILINICHIK — 3

FIND: AXIAL LOAD FOR PCA COLUMN CALC [6 WEB]

GIVEN:

SOLN: CONTROLLING LOAD CASE: 0.9D + 1.6W

0.9D = 0.9(Psw + Pg)

Pg = AXIAL LOAD FROM FLOOR PLATE = ATRIB × LOAD × #FLRS

[18"] PIERS 5 & 10: (90')(21.5')(95.7)(57) = 10,600K

PIERS 6–9: (90')(30')(95.7)(57) = 14,700K

Psw = AXIAL LOAD OF SHEAR WALLS = Asw × 150 × HT

[18"] PIERS 5 & 10: A = (1.5)(45) = 67.5 SF

[24"] PIERS 6 & 9: A = (2)(38.5) + 2(4.5)(20) = 251 SF

[18"] PIERS 7 & 8: A = (1.5)(35.5) + 2(4.5)(20) = 233 SF

HT (STORY 1 → 57) 884'

PIERS 5 & 10: (67.5)(150)(884) = 8,950K

PIERS 6 & 9: (251)(150)(884) = 33,300K

PIERS 7 & 8: (233)(150)(884) = 30,900K

SUMMARY

P5 & 10:	0.9[10,600 + 8,950] =	17,600K
P6 & 9:	0.9[14,700 + 33,300] =	43,200K
P7 & 8:	0.9[14,700 + 30,900] =	41,000K

FIGURE H-2.3: AXIAL LOADS FOR 6 WEB SYSTEM FOR PCA COLUMN INPUT

AE 482: PCA COLUMNS CYNTHIA MILINICHIK 4

FIND FACTORED WIND LOAD FOR 24" THK WEB [4 WEBS]

$M_{1.0w_y} = 8{,}807{,}650^{"k} = \underline{733{,}970^{'k}} \times 0.3 = \underline{220{,}200^{'k}}$

$M_{1.0w_x} = 288{,}028^{"k} = \underline{24{,}000^{'k}} \times 0.3 = \underline{7{,}200^{'k}}$

CHECK MIN STEEL

ACI 10.5 EQN (10-3) MIN REINF FLEX.

$$A_{s_{MIN}} = \frac{3\sqrt{f'_c}}{f_y} b_w d$$

$$\rho_{MIN} = \frac{3\sqrt{f'_c}}{f_y} = \frac{3\sqrt{10{,}000}}{60{,}000} = 0.005 \leftarrow \text{CONTROLS} = 0.5\%$$

$$\frac{200}{f_y} = \frac{200}{60{,}000} = 0.003$$

CONCLUSION

(FROM PCA COLUMNS) $\phi M_n / M_u = 1.2 > 1.0$ ∴ OK

FOR A $\rho = 0.24\% < 0.5\%$ ∴ USE MIN STEEL

* FLEXURAL REINFORCEMENT

* STRENGTH DOES NOT CONTROL DESIGN, SERVICEABILITY (STIFFNESS DOES)

NOTE: LOAD CASE

FIGURE H-2.4: WIND MOMENTS, MIN FLEXURAL STEEL CALC, CONCLUSIONS FROM PCA COLUMN CALC

H.3 PCA COLUMN OUTPUT FOR FLEXURAL REINFORCEMENT

General Information:
=====================

File Name: T:\PCA_Columns\24.col
Project: Comcast
Column: 24
Code: ACI 318-02
Engineer: clm
Units: English

Run Option: Investigation
Run Axis: Biaxial
Slenderness: Not considered
Column Type: Structural

Material Properties:
=====================

f'c = 10 ksi
Ec = 5700.01 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.65
fy = 60 ksi
Es = 29000 ksi

Section:
========

Exterior Points

No.	X (in)	Y (in)	No.	X (in)	Y (in)	No.	X (in)	Y (in)
1	0.0	0.0	2	480.0	0.0	3	480.0	54.0
4	252.0	54.0	5	252.0	480.0	6	480.0	480.0
7	480.0	534.0	8	0.0	534.0	9	0.0	480.0
10	228.0	480.0	11	228.0	54.0	12	0.0	54.0

Gross section area, Ag = 62064 in^2
Ix = 3.1532e+009 in^4
Iy = 9.95818e+008 in^4
Xo = 240 in
Yo = 267 in

Reinforcement:
===============

Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Pattern: Irregular

Total steel area, As = 152.00 in^2 at 0.24%

Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)
2.00	10.0	27.0	2.00	24.0	27.0	2.00	38.0	27.0
2.00	52.0	27.0	2.00	66.0	27.0	2.00	80.0	27.0
2.00	94.0	27.0	2.00	108.0	27.0	2.00	122.0	27.0
2.00	136.0	27.0	2.00	150.0	27.0	2.00	164.0	27.0
2.00	178.0	27.0	2.00	192.0	27.0	2.00	206.0	27.0
2.00	220.0	27.0	2.00	234.0	27.0	2.00	248.0	27.0
2.00	262.0	27.0	2.00	276.0	27.0	2.00	290.0	27.0
2.00	304.0	27.0	2.00	318.0	27.0	2.00	332.0	27.0
2.00	346.0	27.0	2.00	360.0	27.0	2.00	374.0	27.0
2.00	388.0	27.0	2.00	402.0	27.0	2.00	416.0	27.0
2.00	430.0	27.0	2.00	444.0	27.0	2.00	458.0	27.0
2.00	472.0	27.0	2.00	10.0	507.0	2.00	24.0	507.0
2.00	38.0	507.0	2.00	52.0	507.0	2.00	66.0	507.0
2.00	80.0	507.0	2.00	94.0	507.0	2.00	108.0	507.0

```
        2.00    122.0    507.0        2.00    136.0    507.0        2.00    150.0    507.0
        2.00    164.0    507.0        2.00    178.0    507.0        2.00    192.0    507.0
        2.00    206.0    507.0        2.00    220.0    507.0        2.00    234.0    507.0
        2.00    248.0    507.0        2.00    262.0    507.0        2.00    276.0    507.0
        2.00    290.0    507.0        2.00    318.0    507.0        2.00    332.0    507.0
        2.00    346.0    507.0        2.00    360.0    507.0        2.00    374.0    507.0
        2.00    388.0    507.0        2.00    402.0    507.0        2.00    416.0    507.0
        2.00    430.0    507.0        2.00    444.0    507.0        2.00    458.0    507.0
        2.00    472.0    507.0        1.00    240.0     66.0        1.00    240.0     90.0
        1.00    240.0    114.0        1.00    240.0    138.0        1.00    240.0    162.0
        1.00    240.0    186.0        1.00    240.0    210.0        1.00    240.0    234.0
        1.00    240.0    258.0        1.00    240.0    282.0        1.00    240.0    306.0
        1.00    240.0    330.0        1.00    240.0    354.0        1.00    240.0    378.0
        1.00    240.0    402.0        1.00    240.0    426.0        1.00    240.0    450.0
        1.00    240.0    474.0

 Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
 =========================================================
             Pu          Mux          Muy         fMnx          fMny
   No.       kip         k-ft         k-ft         k-ft          k-ft    fMn/Mu
   --- ----------- ----------- ----------- ----------- ----------- --------
     1     43225.0     24000.0    220200.0      96675.5     886998.8     4.028
     2     43225.0     24000.0   -220200.0      96652.6    -886786.1     4.027
     3     43225.0    -24000.0    220200.0     -96723.3     887436.8     4.030
     4     43225.0    -24000.0   -220200.0     -96691.8    -887149.9     4.029
     5     43225.0      7200.0    733970.0       8784.9     895532.5     1.220
     6     43225.0     -7200.0    733970.0      -8788.6     895909.4     1.221
     7     43225.0      7200.0   -733970.0       8783.0    -895329.1     1.220
     8     43225.0     -7200.0   -733970.0      -8786.6    -895701.1     1.220

   *** Program completed as requested! ***
```

FIGURE H-3.1: PCA COLUMNS OUTPUT FOR FLEXURAL REINFORCEMENT

H.4 MOMENT DUE TO WIND LOAD CALCS

4 Web (Moment 1.0W)

	Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
30"	STORY1	PIER1	WINDXX	Top	19080	-145	-675	8376	-137376	-128486
	STORY1	PIER1	WINDXX	Bottom	19080	-145	-675	8377	-248785	-152418
	STORY1	PIER1	WINDYY	Top	-1	1495	0	-1849	4	8889972
	STORY1	PIER1	WINDYY	Bottom	-1	1495	0	-1849	-3	9136694
24"	STORY1	PIER2	WINDXX	Top	5122	-34	-1196	16104	-90715	-45383
	STORY1	PIER2	WINDXX	Bottom	5122	-34	-1196	16104	-288028	-51062
	STORY1	PIER2	WINDYY	Top	1	1350	0	-1849	5	8584861
	STORY1	PIER2	WINDYY	Bottom	1	1350	0	-1849	-4	8807650
24"	STORY1	PIER3	WINDXX	Top	-5122	34	-1196	16104	-90716	45553
	STORY1	PIER3	WINDXX	Bottom	-5122	34	-1196	16104	-288027	51235
	STORY1	PIER3	WINDYY	Top	1	1351	0	2249	-4	8586125
	STORY1	PIER3	WINDYY	Bottom	1	1351	0	2249	3	8809028
30"	STORY1	PIER4	WINDXX	Top	-19080	145	-675	8376	-137376	128662
	STORY1	PIER4	WINDXX	Bottom	-19080	145	-675	8377	-248784	152598
	STORY1	PIER4	WINDYY	Top	-1	1498	0	2054	-5	8893541
	STORY1	PIER4	WINDYY	Bottom	-1	1498	0	2054	4	9140775

FIGURE H-4.1: ETABS OUTPUT FOR 4 WEB SYSTEM MOMENTS DUE TO UNFACTORED WIND LOAD

6 Web (Moment 1.0W)

	Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
18"	STORY1	PIER5	WINDXX	Top	3951	-48	-13	22	20	-4859
	STORY1	PIER5	WINDXX	Bottom	3951	-48	-13	22	-154	-5514
	STORY1	PIER5	WINDYY	Top	-393	929	0	102	3	86884
	STORY1	PIER5	WINDYY	Bottom	-393	929	0	102	6	99655
24"	STORY1	PIER6	WINDXX	Top	12025	-74	-861	1160	-21689	-27424
	STORY1	PIER6	WINDXX	Bottom	12025	-74	-861	1160	-33523	-28444
	STORY1	PIER6	WINDYY	Top	-892	983	29	751	1633	675729
	STORY1	PIER6	WINDYY	Bottom	-892	983	29	751	2029	689245
18"	STORY1	PIER7	WINDXX	Top	3586	-24	-942	1473	-20309	-17371
	STORY1	PIER7	WINDXX	Bottom	3586	-24	-942	1473	-33265	-17697
	STORY1	PIER7	WINDYY	Top	363	858	16	-45	1692	630348
	STORY1	PIER7	WINDYY	Bottom	363	858	16	-45	1910	642143
18"	STORY1	PIER8	WINDXX	Top	-2358	8	-954	1476	-20195	-8861
	STORY1	PIER8	WINDXX	Bottom	-2358	8	-954	1476	-33315	-8750
	STORY1	PIER8	WINDYY	Top	1653	878	2	-448	1802	625999
	STORY1	PIER8	WINDYY	Bottom	1653	878	2	-448	1832	638072
24"	STORY1	PIER9	WINDXX	Top	-8986	52	-972	-1282	-19953	-20649
	STORY1	PIER9	WINDXX	Bottom	-8844	46	-628	5602	-29579	-22801
	STORY1	PIER9	WINDYY	Top	3535	1090	-477	-11192	2801	664091
	STORY1	PIER9	WINDYY	Bottom	3554	1085	478	8544	2903	678670
18"	STORY1	PIER10	WINDXX	Top	-8217	78	38	890	-1671	4344
	STORY1	PIER10	WINDXX	Bottom	-8359	83	-306	-3138	-2358	7062
	STORY1	PIER10	WINDYY	Top	-4267	903	441	5819	22427	170647
	STORY1	PIER10	WINDYY	Bottom	-4286	907	-513	-5814	22173	183303

FIGURE H-4.2: ETABS OUTPUT FOR 6 WEB SYSTEM MOMENTS DUE TO UNFACTORED WIND

6 Web System: Moment Resulting from Factored Wind Loading

	Story	Pier	Load	Loc	M2 ('k)	M3('k)
18"	STORY1	PIER5	9D16WX	Top	39	-8585
	STORY1	PIER5	9D16WX	Bottom	-249	-9727
	STORY1	PIER5	9D16WY	Top	12	138204
	STORY1	PIER5	9D16WY	Bottom	6	158544
24"	STORY1	PIER6	9D16WX	Top	-34107	-50986
	STORY1	PIER6	9D16WX	Bottom	-52972	-52631
	STORY1	PIER6	9D16WY	Top	3208	1074059
	STORY1	PIER6	9D16WY	Bottom	3912	1095671
18"	STORY1	PIER7	9D16WX	Top	-31910	-35262
	STORY1	PIER7	9D16WX	Bottom	-52590	-35801
	STORY1	PIER7	9D16WY	Top	3292	1001090
	STORY1	PIER7	9D16WY	Bottom	3690	1019943
18"	STORY1	PIER8	9D16WX	Top	-31738	-22504
	STORY1	PIER8	9D16WX	Bottom	-52668	-22330
	STORY1	PIER8	9D16WY	Top	3457	993272
	STORY1	PIER8	9D16WY	Bottom	3568	1012586
24"	STORY1	PIER9	9D16WX	Top	-30454	-41689
	STORY1	PIER9	9D16WX	Bottom	-45992	-45062
	STORY1	PIER9	9D16WY	Top	5952	1053894
	STORY1	PIER9	9D16WY	Bottom	5979	1077291
18"	STORY1	PIER10	9D16WX	Top	-3875	4329
	STORY1	PIER10	9D16WX	Bottom	-4931	8640
	STORY1	PIER10	9D16WY	Top	34683	270414
	STORY1	PIER10	9D16WY	Bottom	34318	290627

FIGURE H-4.3: ETABS OUTPUT FOR 6 WEB SYSTEM MOMENTS DUE TO FACTORED WIND

4 Web System: Moment Resulting from Factored Wind Loading

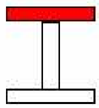
Story	Pier	Load	Loc	M2 ('k)	M3 ('k)
STORY1	PIER1	9D16WX	Top	-18748	-11597
STORY1	PIER1	9D16WX	Bottom	-33328	-14772
STORY1	PIER1	9D16WY	Top	-431	1190864
STORY1	PIER1	9D16WY	Bottom	-157	1223777
STORY1	PIER2	9D16WX	Top	-12406	-659
STORY1	PIER2	9D16WX	Bottom	-38685	-1401
STORY1	PIER2	9D16WY	Top	-310	1150040
STORY1	PIER2	9D16WY	Bottom	-282	1179760
STORY1	PIER3	9D16WX	Top	-12355	11504
STORY1	PIER3	9D16WX	Bottom	-38723	12276
STORY1	PIER3	9D16WY	Top	-260	1150247
STORY1	PIER3	9D16WY	Bottom	-319	1179982
STORY1	PIER4	9D16WX	Top	-18463	22797
STORY1	PIER4	9D16WX	Bottom	-33609	26005
STORY1	PIER4	9D16WY	Top	-147	1191448
STORY1	PIER4	9D16WY	Bottom	-438	1224428

FIGURE H-4.4: ETABS OUTPUT FOR 4 WEB SYSTEM MOMENTS DUE TO FACTORED WIND

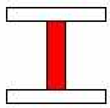
H.5 Shear Due to Wind Load Calcs

4 Web: Summary of Wall Shear Forces

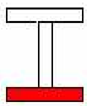
Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
STORY1	PIER11	9D16WX	Top	949	**502**	0	1	-3	111725
STORY1	PIER11	9D16WX	Bottom	755	502	0	1	-3	194593
STORY1	PIER11	9D16WY	Top	-34429	-4	-3	0	108	1960
STORY1	PIER11	9D16WY	Bottom	-35110	-4	4	0	131	1329
STORY1	PIER12	9D16WX	Top	590	-231	0	1	-2	-18256
STORY1	PIER12	9D16WX	Bottom	565	-231	0	1	-3	-30956
STORY1	PIER12	9D16WY	Top	-10200	**2397**	0	0	0	2167353
STORY1	PIER12	9D16WY	Bottom	-10395	2388	0	0	0	2313031
STORY1	PIER13	9D16WX	Top	446	**558**	0	2	4	113250
STORY1	PIER13	9D16WX	Bottom	145	558	0	1	-2	205343
STORY1	PIER13	9D16WY	Top	16083	-16	0	0	32	3210
STORY1	PIER13	9D16WY	Bottom	16440	-16	1	0	56	558
STORY1	PIER21	9D16WX	Top	-6475	**902**	-1	1	27	76055
STORY1	PIER21	9D16WX	Bottom	-6634	902	1	1	27	224860
STORY1	PIER21	9D16WY	Top	-34499	5	-3	0	107	1256
STORY1	PIER21	9D16WY	Bottom	-35142	5	4	0	133	2092
STORY1	PIER22	9D16WX	Top	-5799	-54	0	0	-1	-45
STORY1	PIER22	9D16WX	Bottom	-5939	-54	0	0	-2	-3097
STORY1	PIER22	9D16WY	Top	-8305	**2164**	0	0	0	1721905
STORY1	PIER22	9D16WY	Bottom	-8466	2156	0	0	0	1846695
STORY1	PIER23	9D16WX	Top	-6507	**1009**	1	1	-26	72813
STORY1	PIER23	9D16WX	Bottom	-6691	1009	-1	1	-27	239359
STORY1	PIER23	9D16WY	Top	15828	-7	0	0	30	2460
STORY1	PIER23	9D16WY	Bottom	16150	-7	1	0	57	1291
STORY1	PIER31	9D16WX	Top	-12560	**905**	-2	1	51	75753
STORY1	PIER31	9D16WX	Bottom	-12723	905	2	1	51	225084
STORY1	PIER31	9D16WY	Top	-34629	-5	-3	0	107	2196
STORY1	PIER31	9D16WY	Bottom	-35273	-5	4	0	134	1292
STORY1	PIER32	9D16WX	Top	-10924	56	0	0	-1	16611
STORY1	PIER32	9D16WX	Bottom	-11106	56	0	0	-2	19798
STORY1	PIER32	9D16WY	Top	-8417	**2165**	0	0	0	1722205
STORY1	PIER32	9D16WY	Bottom	-8578	2157	0	0	0	1847052
STORY1	PIER33	9D16WX	Top	-12054	**1013**	2	1	-50	72510
STORY1	PIER33	9D16WX	Bottom	-12192	1013	-2	1	-49	239584
STORY1	PIER33	9D16WY	Top	15706	10	0	0	29	929
STORY1	PIER33	9D16WY	Bottom	16029	10	1	0	56	2530



STORY1	PIER41	9D16WX	Top	-19916	**523**	-3	1	81	110016
STORY1	PIER41	9D16WX	Bottom	-20045	523	3	1	80	196280
STORY1	PIER41	9D16WY	Top	-34826	4	-3	0	109	1468
STORY1	PIER41	9D16WY	Bottom	-35508	4	4	0	133	2075



STORY1	PIER42	9D16WX	Top	-21415	233	0	1	-2	38989
STORY1	PIER42	9D16WX	Bottom	-21782	233	0	1	-3	51848
STORY1	PIER42	9D16WY	Top	-10627	**2402**	0	0	0	2168375
STORY1	PIER42	9D16WY	Bottom	-10823	2393	0	0	0	2314312

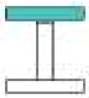
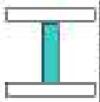
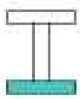
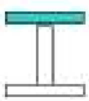
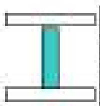
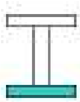
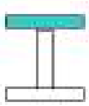
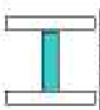
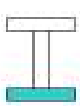
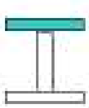
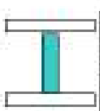


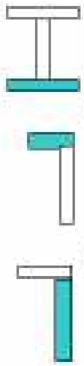
STORY1	PIER43	9D16WX	Top	-18938	**579**	3	2	-80	111539
STORY1	PIER43	9D16WX	Bottom	-18961	579	-3	1	-74	207030
STORY1	PIER43	9D16WY	Top	15711	17	0	0	31	295
STORY1	PIER43	9D16WY	Bottom	16070	17	1	0	54	3177

FIGURE H-5.1: ETABS OUTPUT FOR 4 WEB SYSTEM SHEAR DUE TO WIND

Note: The load case 0.9D + 1.6W was used to calculate the shear. The V2 column lists the shear experienced along the member where as the shear perpendicular to the member is listed under V3. Member perpendicular to the shear force resisted the force with only 10% of the actual width. This was a conservative assumption for calculating the shear experienced in a member and is the reason that all the V3 forces are zero or close to zero.

6 Web: Summary of Shear Forces (0.1t in Bending)

	Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
	STORY1	PIER52	9D16WX	Top	-667	-84	0	1	3	-67528
	STORY1	PIER52	9D16WX	Bottom	-778	-84	0	1	-4	-81424
	STORY1	PIER52	9D16WY	Top	-7485	**347**	0	-3	0	1265251
	STORY1	PIER52	9D16WY	Bottom	-7596	347	0	-3	0	1322462
	STORY1	PIER61	9D16WX	Top	-1454	**444**	0	0	8	73165
	STORY1	PIER61	9D16WX	Bottom	-1582	444	0	0	7	146421
	STORY1	PIER61	9D16WY	Top	-34002	28	-3	-1	105	-18784
	STORY1	PIER61	9D16WY	Bottom	-34672	28	4	-1	133	-14228
	STORY1	PIER62	9D16WX	Top	-2387	-361	0	1	-1	-71645
	STORY1	PIER62	9D16WX	Bottom	-2446	-361	0	1	-1	-91280
	STORY1	PIER62	9D16WY	Top	-9155	**2282**	0	0	0	1617528
	STORY1	PIER62	9D16WY	Bottom	-9320	2274	0	0	0	1748266
	STORY1	PIER63	9D16WX	Top	-3686	**539**	0	2	-11	71858
	STORY1	PIER63	9D16WX	Bottom	-3981	539	-1	2	-19	160838
	STORY1	PIER63	9D16WY	Top	13406	-61	0	-1	20	-14813
	STORY1	PIER63	9D16WY	Bottom	13758	-61	1	-1	50	-24918
	STORY1	PIER71	9D16WX	Top	-6399	**696**	-1	1	28	50553
	STORY1	PIER71	9D16WX	Bottom	-6542	696	1	1	26	165448
	STORY1	PIER71	9D16WY	Top	-32731	17	-3	-1	99	-17799
	STORY1	PIER71	9D16WY	Bottom	-33336	17	4	-1	131	-15060
	STORY1	PIER72	9D16WX	Top	-4854	-109	0	0	-1	-42552
	STORY1	PIER72	9D16WX	Bottom	-4962	-109	0	0	-1	-48460
	STORY1	PIER72	9D16WY	Top	-6224	**1922**	0	0	0	1192626
	STORY1	PIER72	9D16WY	Bottom	-6346	1914	0	0	0	1296476
	STORY1	PIER73	9D16WX	Top	-8101	**837**	1	1	-32	45019
	STORY1	PIER73	9D16WX	Bottom	-8295	837	-1	1	-34	183161
	STORY1	PIER73	9D16WY	Top	14080	-66	0	-1	22	-14099
	STORY1	PIER73	9D16WY	Bottom	14361	-66	1	-1	54	-26021
	STORY1	PIER81	9D16WX	Top	-10306	**740**	-2	1	42	46610
	STORY1	PIER81	9D16WX	Bottom	-10483	740	2	1	43	168723
	STORY1	PIER81	9D16WY	Top	-31578	15	-3	-1	95	-17659
	STORY1	PIER81	9D16WY	Bottom	-32175	15	4	-1	126	-15206
	STORY1	PIER82	9D16WX	Top	-7258	69	0	0	-1	-31537
	STORY1	PIER82	9D16WX	Bottom	-7384	69	0	0	-1	-28265
	STORY1	PIER82	9D16WY	Top	-5567	**1871**	0	0	0	1185313
	STORY1	PIER82	9D16WY	Bottom	-5684	1864	0	0	0	1286384
	STORY1	PIER83	9D16WX	Top	-11459	**853**	2	1	-47	43498
	STORY1	PIER83	9D16WX	Bottom	-11602	853	-2	1	-46	184306
	STORY1	PIER83	9D16WY	Top	14913	-34	0	-1	26	-17133
	STORY1	PIER83	9D16WY	Bottom	15180	-34	1	-1	58	-22681
	STORY1	PIER91	9D16WX	Top	-14610	**1441**	-2	1	55	48423
	STORY1	PIER91	9D16WX	Bottom	-14701	411	3	4	66	141900
	STORY1	PIER91	9D16WY	Top	-30267	1009	-2	-5	93	-53222
	STORY1	PIER91	9D16WY	Bottom	-30856	-825	4	4	120	-46290
	STORY1	PIER92	9D16WX	Top	-13307	815	0	-7	0	-46098
	STORY1	PIER92	9D16WX	Bottom	-13446	815	0	5	-1	-15183
	STORY1	PIER92	9D16WY	Top	-6187	**2158**	0	-12	1	1583469
	STORY1	PIER92	9D16WY	Bottom	-6319	2150	0	11	1	1705220



STORY1	PIER93	9D16WX	Top	-15538	**524**	3	2	-68	73354
STORY1	PIER93	9D16WX	Bottom	-15494	524	-2	2	-56	159757
STORY1	PIER93	9D16WY	Top	16244	12	0	0	33	-21284
STORY1	PIER93	9D16WY	Bottom	16556	12	2	0	61	-19294
STORY1	PIER101	9D16WX	Top	-9353	-135	-2	-84	41	23028
STORY1	PIER101	9D16WX	Bottom	-9678	**895**	1	66	34	40245
STORY1	PIER101	9D16WY	Top	-14763	-944	-1	-57	45	27747
STORY1	PIER101	9D16WY	Bottom	-15051	890	1	71	49	28147
STORY1	PIER102	9D16WX	Top	-13567	-350	0	15	-1	-5036
STORY1	PIER102	9D16WX	Bottom	-13934	-353	0	-11	-3	-1740
STORY1	PIER102	9D16WY	Top	-3830	**531**	0	22	-2	1238493
STORY1	PIER102	9D16WY	Bottom	-3810	529	0	-19	-3	1294486

FIGURE H-5.2: ETABS OUTPUT FOR 6 WEB SYSTEM SHEAR DUE TO WIND

H.6 SEISMIC LOADS

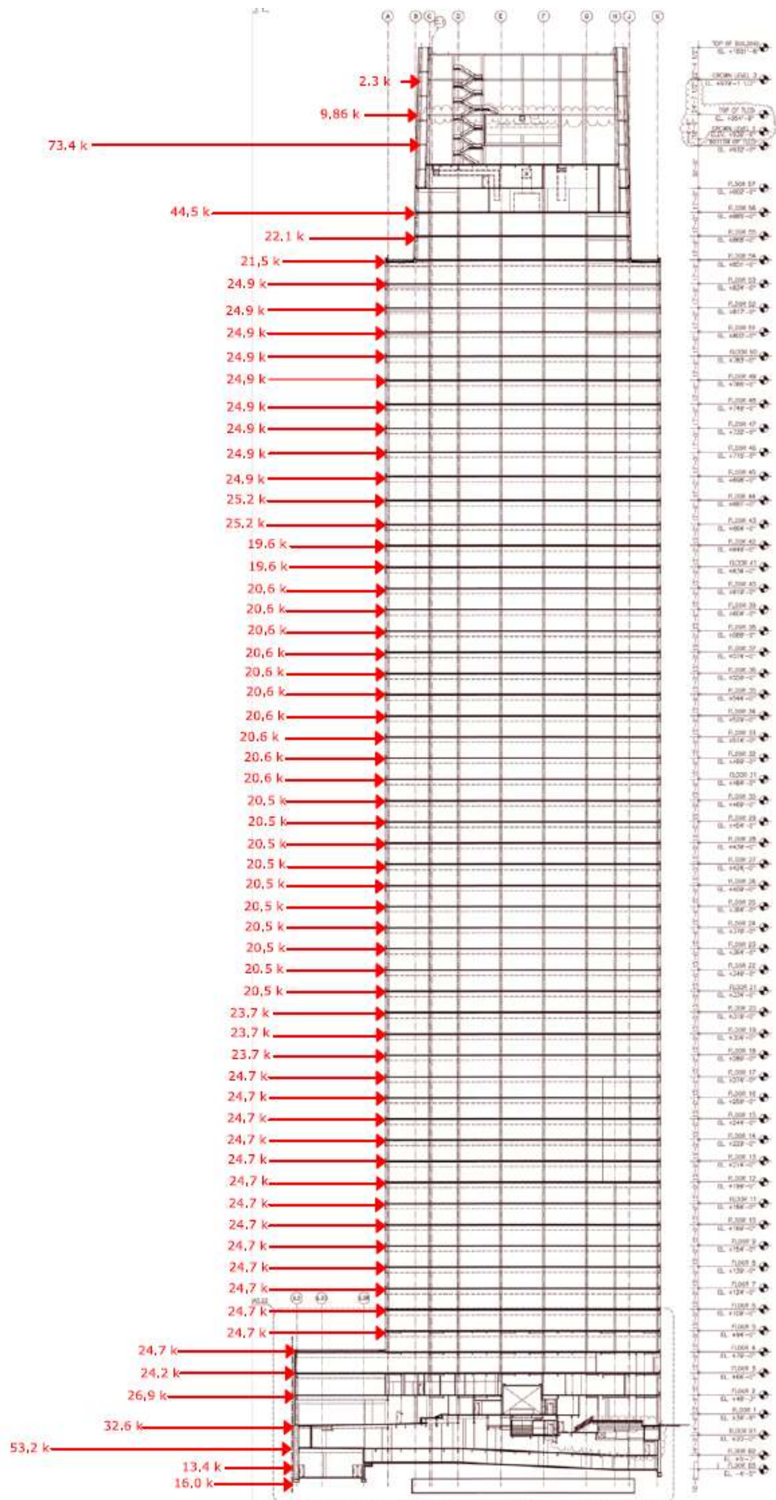


FIGURE H-6.1: SEISMIC FORCES PER FLOOR

H.7 TYPICAL FLOOR PLAN

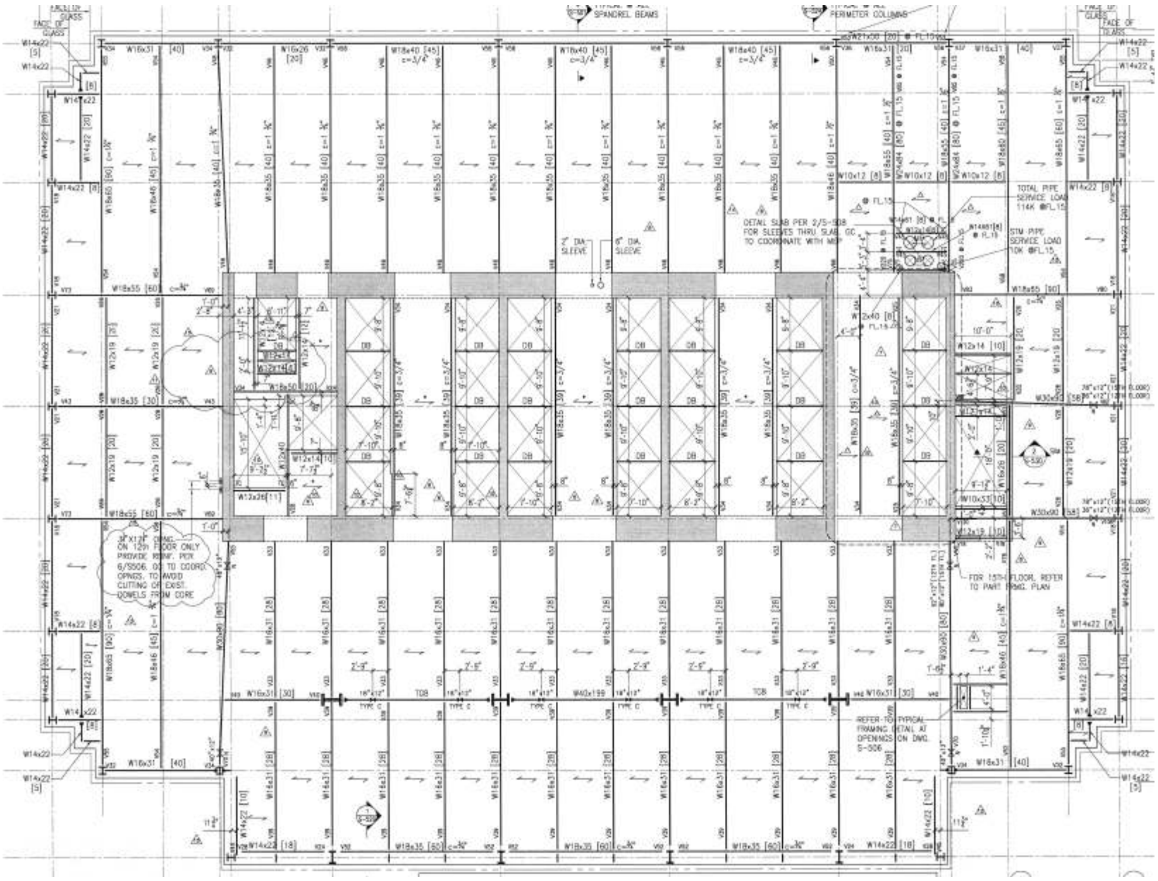


FIGURE H-7.1: TYPICAL FLOOR PLAN

H.8 BUILDING ELEVATIONS

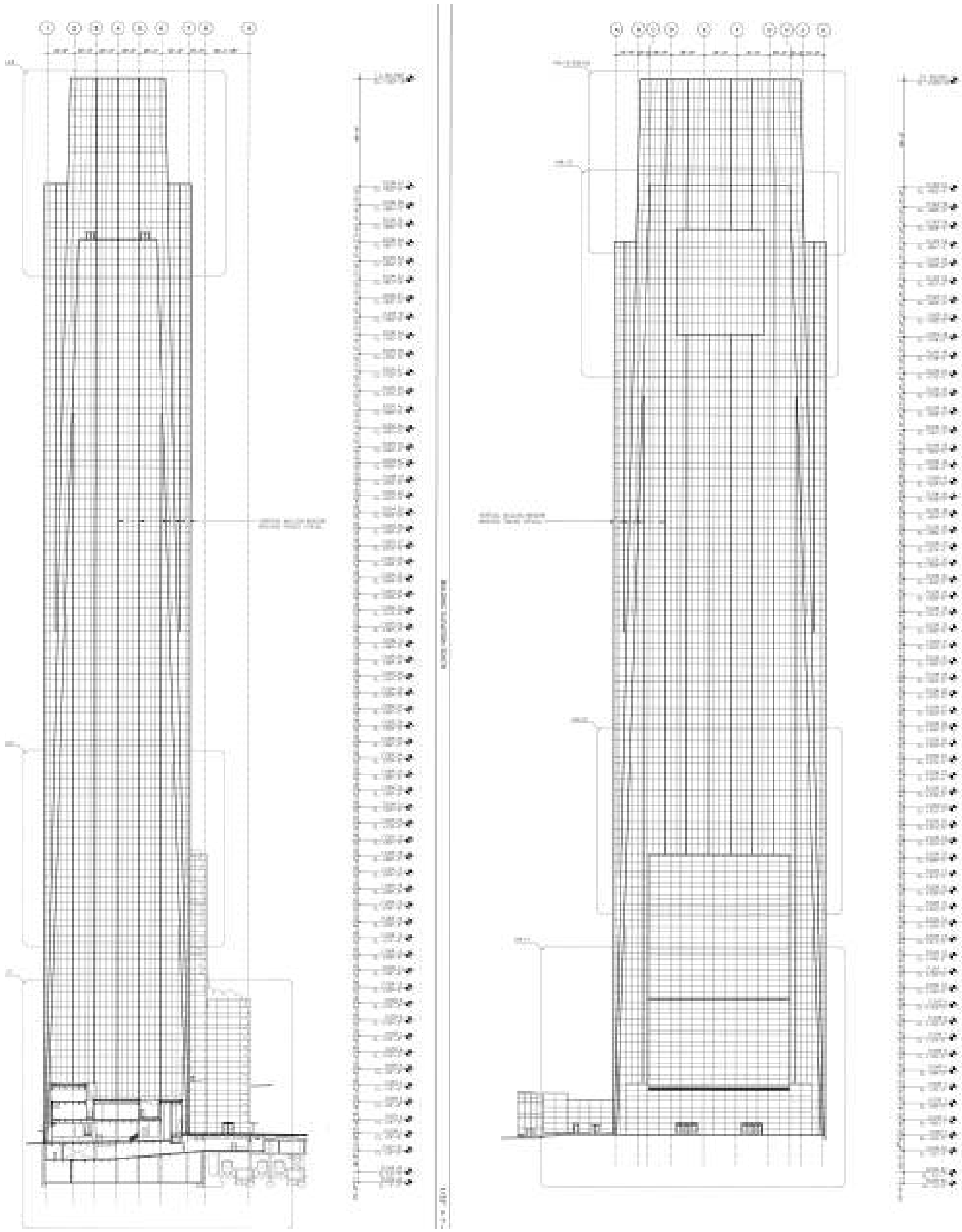
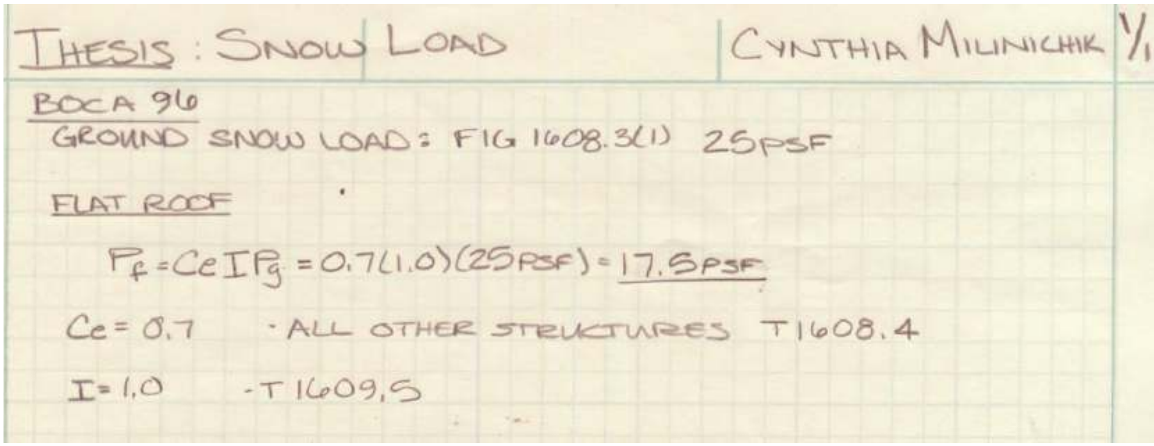


FIGURE H-8.1: BUILDING ELEVATIONS

H.9 GRAVITY LOAD CALCS


THESIS : SNOW LOAD — CYNTHIA MILINICHIK 1/1

BOCA 96

GROUND SNOW LOAD: FIG 1608.3(1) 25PSF

FLAT ROOF

$P_f = C_e I P_g = 0.7(1.0)(25PSF) = 17.5PSF$

$C_e = 0.7$ · ALL OTHER STRUCTURES T1608.4

$I = 1.0$ - T 1609.5

FIGURE H-9.1: SNOW LOAD CALCULATION

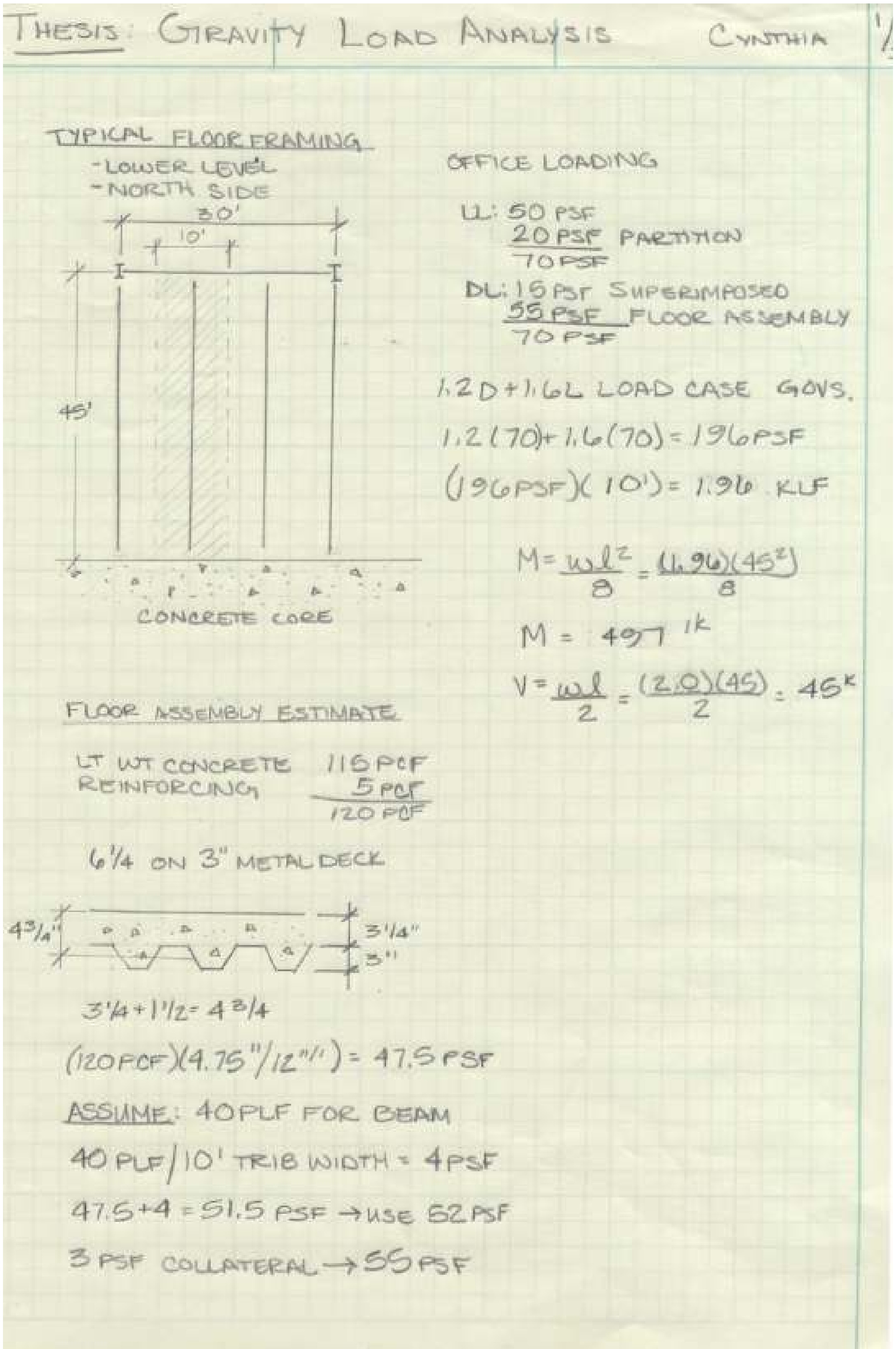


FIGURE H-9.2: GRAVITY LOAD CALC

THESIS: GRAVITY LOAD ANALYSIS — CYNTHIA

COMPOSITE BEAM DESIGN

6 1/4" CONCRETE SLAB
ON 3" METAL DECK

3/4" φ STUDS

ASSUMING $a = 2"$

$Y2 = Y_{con} - a/2 = 6.25 - 2/2 = 5.25"$

$M_u = 497^{'k}$ TRY W18×35 → $\phi M_n = 505^{'k}$

$Y1 = 0$ $\Sigma Q_N = 515^k$

CHECK: ASSUMED Y2

$b_{min} = \left| \begin{array}{l} 1/4 L = 1/4(45') = 11.25' \\ s = 10' \leftarrow \text{USE} \quad 120" \end{array} \right.$

$a_{req} = \dfrac{\Sigma Q_N}{0.85 f'_c b} = \dfrac{515}{0.85(4)(120)} = 1.26"$

$Y2 = Y_{con} - \dfrac{a}{2} = 6.25 - \dfrac{1.26}{2} = 5.62$ USE 5.5

SHEAR STUDS REQ'D

$\dfrac{2\Sigma Q_N}{Q_N} = \dfrac{2(516)}{26.1} = 39.5 \rightarrow 40$ STUDS

$Q_N = 26.1$ 3/4" φ STUDS 4 KSI

ASSUME: UNSHORED CONSTRUCTION
CHECK: STEEL CAPACITY

(48 PSF)(10') = 480 PLF
1.4(480) = 672 PLF

1.2(480) + 1.6(20 PSF)(10) = 896 PLF ← GOVERNS

$M_u = \dfrac{wl^2}{8} = \dfrac{(896)(45')^2}{8} = 226.8^{'k}$

$V_u = \dfrac{wl}{2} = \dfrac{896(45)}{2} = 20.2^k =$

FIGURE H-9.3: GRAVITY LOAD CALC

THESIS : GRAVITY LOAD ANALYSIS | CYNTHIA MILINICHIK 3/3

ASSUME: METAL DECK PROVIDES LATERAL STABILITY FOR BEAMS

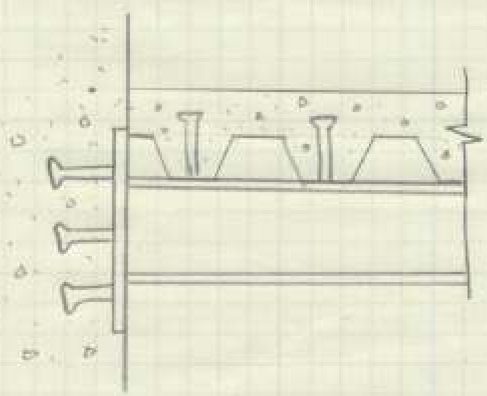
TABLE 5-3

$\phi M_n = 249^{'k} < 226.8^{'k} \therefore$ OK

NOTE I USE LRFD TO DESIGN/ CHECK THIS AND SINCE AND JUDGING BY THE CLOSENESS OF THE NOMINAL MOMENT OF THE SHAPE THEY PICKED AND THE FACTORED MOMENT I CALCULATED IT APPEARS THAT THEY LIKELY ALSO USED LRFD.

SHEAR CONNECTION @ CONCRETE CORE

$\phi V_n > V_u = 20.2^k$



CHECK: DEFLECTION $I = 510 \text{ in}^4$

$\Delta_{conc} = \frac{5wl^4}{384EI} = \frac{5(55)(45^4)(1728)}{384(29{,}000)(510)} = 0.3431''$ DEFLECTION OF STEEL SHAPE UNDER WET CONCRETE

$l/360 = \frac{45(12)}{360} = 1.5$

$\Delta_{comp} = \frac{5wl^4}{384EI} = \frac{5(500)(45^4)(1728)}{384(29{,}000{,}000)} = 1.061''$

$\Delta_{TOTAL} = \Delta_{CON} + \Delta_{COMP} \leq l/360 = 1.5''$
$= 0.3431 + 1.061 = 1.404 < 1.5'' \therefore$ OK

FIGURE H-9.4: GRAVITY LOAD CALC

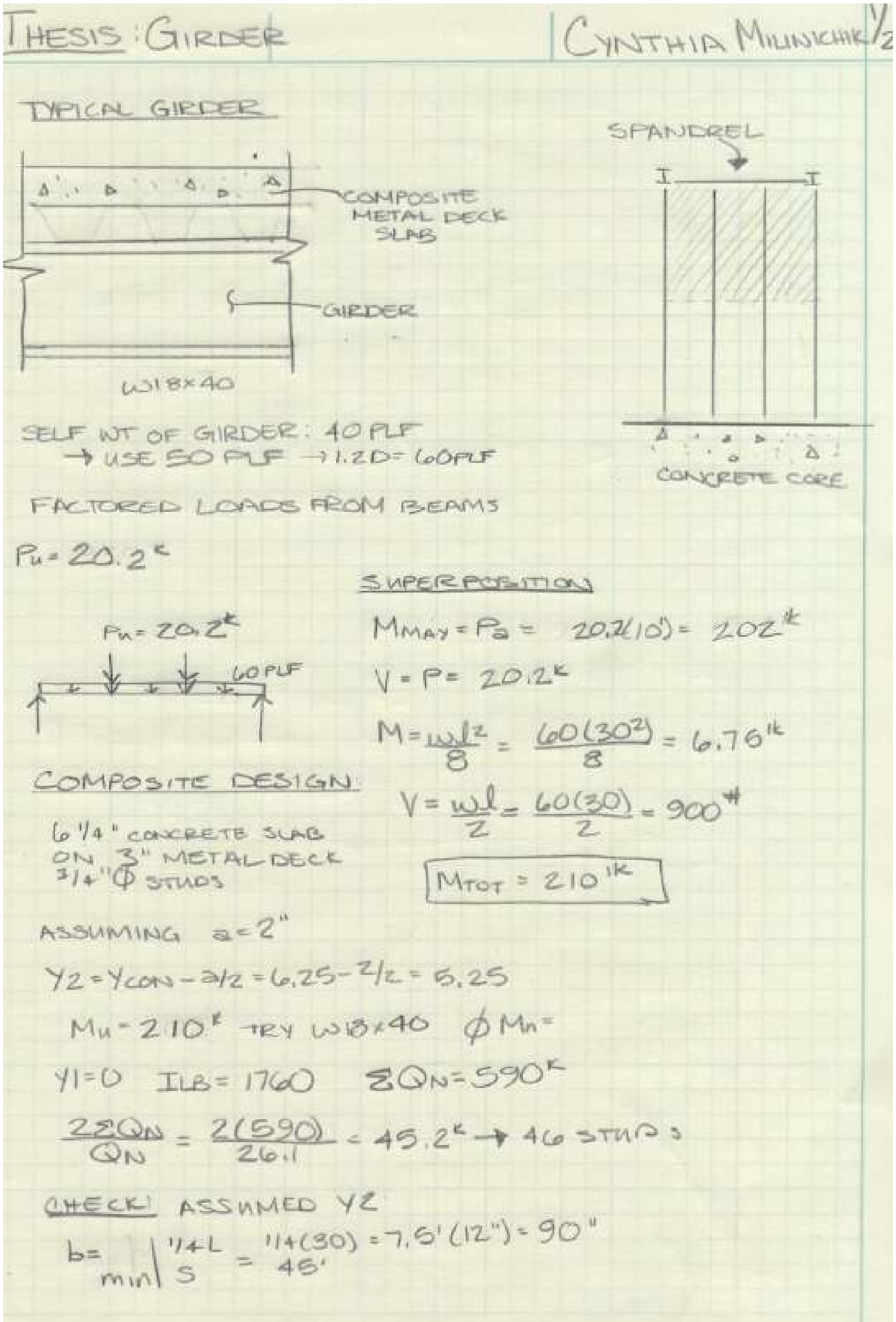
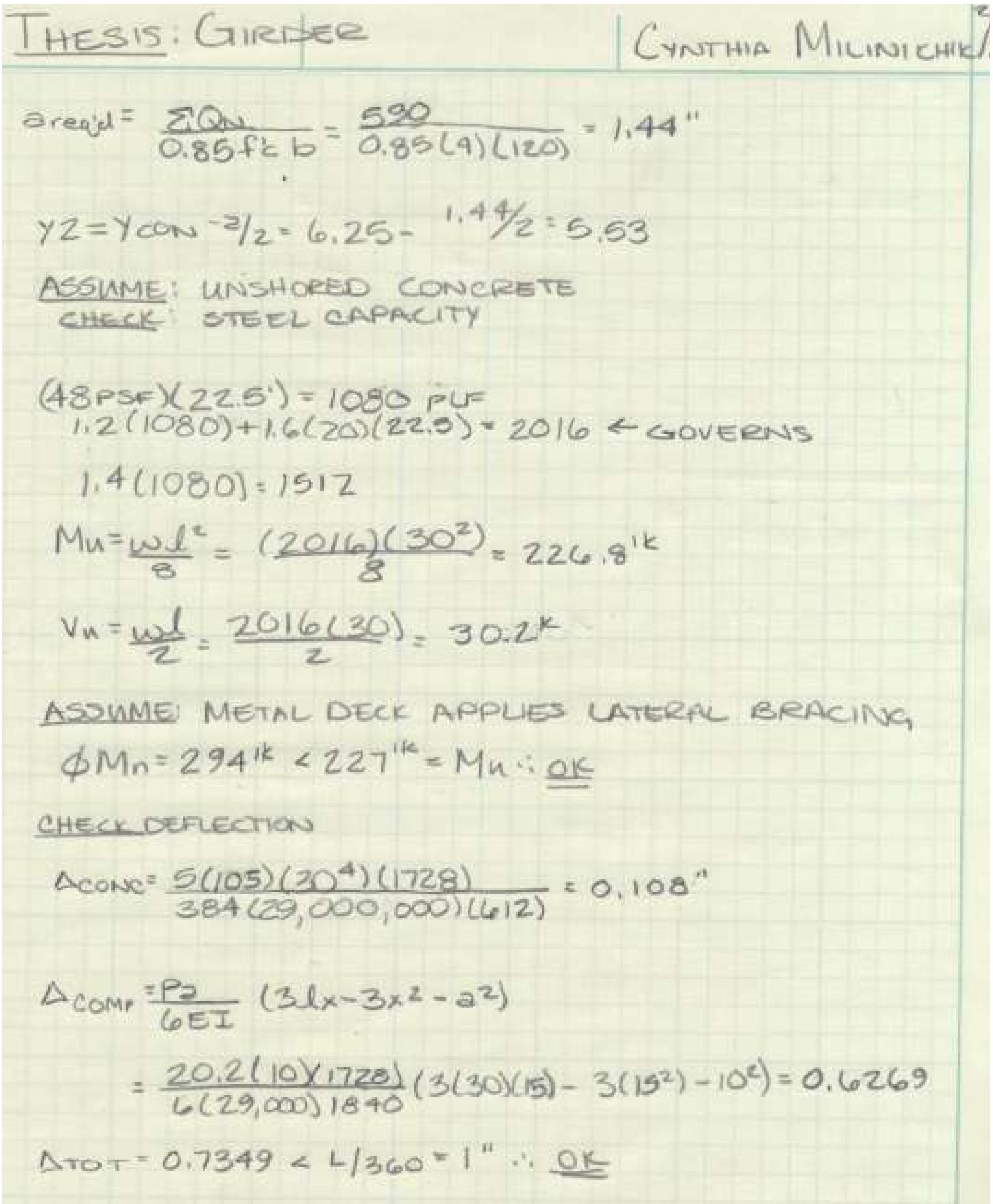


FIGURE H-9.5: GRAVITY LOAD CALC



THESIS: GIRDER | CYNTHIA MILINICHIK

$a_{req'd} = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{590}{0.85(4)(120)} = 1.44"$

$Y2 = Y_{con} - a/2 = 6.25 - 1.44/2 = 5.53$

ASSUME: UNSHORED CONCRETE
CHECK: STEEL CAPACITY

$(48 PSF)(22.5') = 1080$ PLF
$1.2(1080) + 1.6(20)(22.5) = 2016$ ← GOVERNS
$1.4(1080) = 1512$

$M_u = \frac{wl^2}{8} = \frac{(2016)(30^2)}{8} = 226.8'^k$

$V_u = \frac{wl}{2} = \frac{2016(30)}{2} = 30.2^k$

ASSUME: METAL DECK APPLIES LATERAL BRACING
$\phi M_n = 294^{'k} < 227^{'k} = M_u \therefore$ OK

CHECK DEFLECTION

$\Delta_{CONC} = \frac{5(105)(30^4)(1728)}{384(29,000,000)(612)} = 0.108"$

$\Delta_{COMP} = \frac{Pa}{6EI}(3lx - 3x^2 - a^2)$

$= \frac{20.2(10)(1728)}{6(29,000)1840}(3(30)(15) - 3(15^2) - 10^2) = 0.6269$

$\Delta_{TOT} = 0.7349 < L/360 = 1" \therefore$ OK

Figure H-9.6: Gravity Load Calc

H.10 LATERAL LOAD CALCS

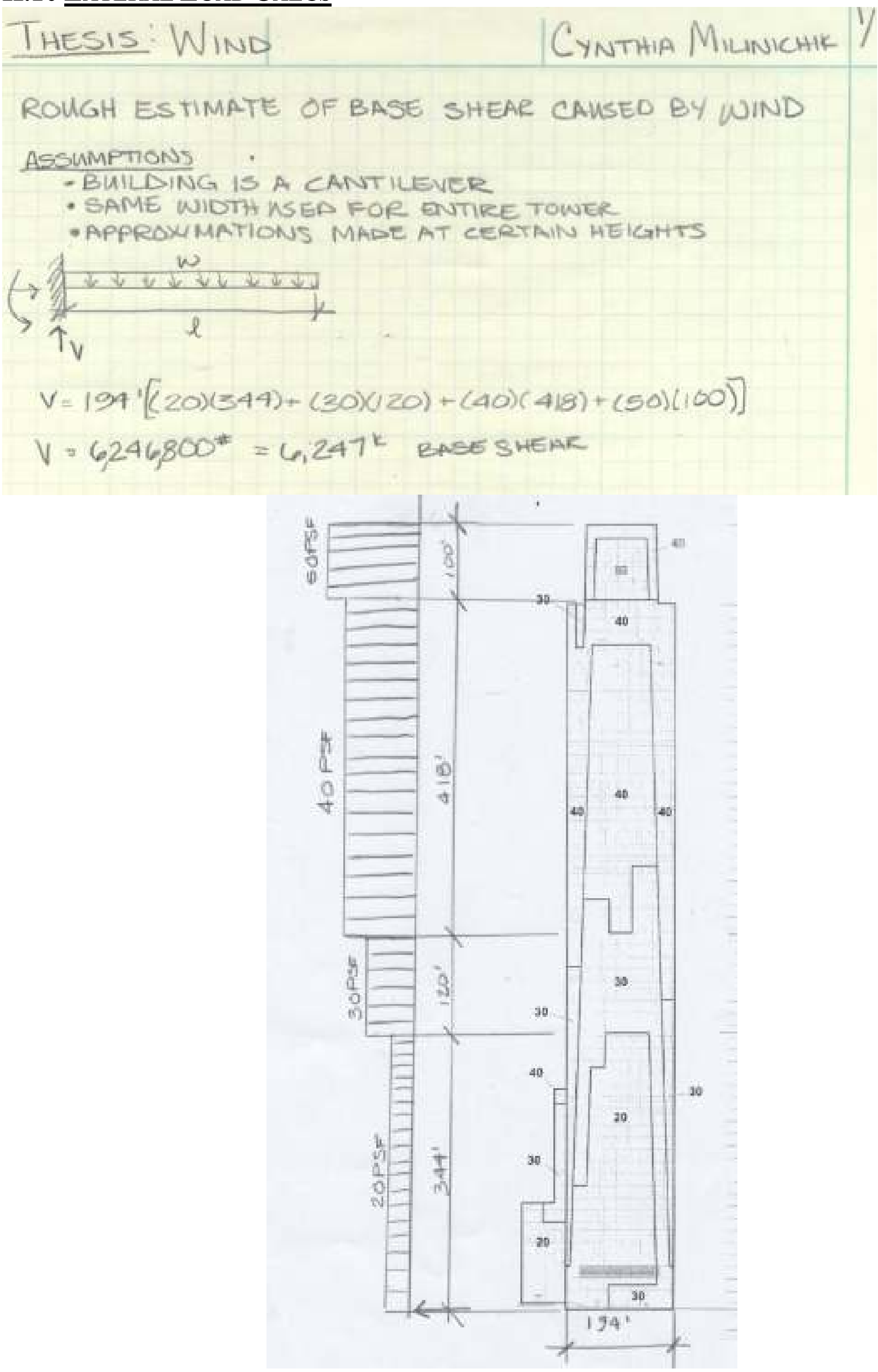


FIGURE H-10.1: BASE SHEAR CALC

H.11 WIND REPORT SUMMARY

SUMMARY AND MAIN FINDINGS

The One Pennsylvania Plaza (Liberty Tower) Tower was previously tested in our laboratory in 2001 and 2003. This latest study in 2005 was necessary due to changes in the building geometry near the top of the tower. The pressure model constructed for the 2003 study was modified to reflect the 2005 geometry and tested in order to provide new overall structural loads and cladding pressure results.

This report on the study of wind action on One Pennsylvania Plaza provides the following information:

1. Overall wind loads from integration of local pressures suitable for use in the design of the structural system;
2. local peak pressures acting on the external surfaces of the project;
3. local peak pressure differences (external pressure less internal pressure and net pressures across parapets, canopies, etc. open to the wind on both sides) suitable for use in the design of the windows, cladding and free-standing elements; and,
4. predictions of the wind environment in pedestrian areas around the site (from 2003 study).

The updated pressure model was instrumented for pressure measurements at 703 locations. It was tested in turbulent boundary layer flow conditions for 36 wind angles. Figure 3 shows close-up views of the pressure model.

A design probability distribution of gradient wind speed and direction had been previously developed for the area on the basis of full scale meteorological records from the Philadelphia area. Peak wind-induced overall loads and responses measured in the wind tunnel were combined with this design probability distribution to predict extreme values for various return periods. Similarly, predictions were made for external and differential pressures.

The highlights and main findings of this study are as follows:

Wind Climate

- The directional characteristics associated with the wind climate model are shown in Figure 1 for various return periods. It can be seen that for strong winds, westerly directions are the most important.
- A surface (10m) wind climate model was developed based on the surface meteorological records for Philadelphia. For strength requirements, the wind climate model was scaled to conform to ASCE-7. The design 3-second gust wind speed from ASCE for the project site was found to be approximately 90mph. From BOCA 96, a design fastest-mile wind speed at 10 metres for the same location was estimated to be approximately 76mph. These are equivalent values with different gust durations. Thus the requirements of both ASCE-7 and BOCA 96 for the design for wind loads have been met.
- Predictions of extreme mean hourly wind speeds for various return periods are shown in Figure 2. The 50 year return period mean hourly wind speed at gradient is 97 mph (43 m/s).

Overall Building Response

- The predicted accelerations and base moments were calculated for both 10 year and 50 year return periods for various values of total damping ratios. The results are summarized in Tables 2 and 3. Figure 7 shows the sign convention and centre of coordinates used.
- The largest building acceleration for a 10 year return period is 38.4 milli-g and with a damping ratio of 1.5% of critical. The BLWTL criterion for acceptable building motions recommends that a 10-year acceleration not exceed 20-25 milli-g for an office building. The accelerations reduce to

27.1 milli-g and 23.5 milli-g with structural damping values of 3.0% and 4.0% respectively which may be attainable with the introduction of an auxiliary damper system.

- Note that the corner accelerations in Table 2 are the worst that would be expected in the tower since they are calculated at the maximum distance from the centre of coordinates at the top occupied floor (approximately 95 feet). All accelerations decrease at lower elevations. Furthermore, the torsion-induced acceleration reduces as the centroid is approached at any floor.
- The largest predicted base bending moments occur in the Y-direction and are 3.50E+09 lb-ft and 3.25E+09 lb-ft for a 50-year return period and 2.0% and 2.5% damping respectively. These moments were calculated at Level B3 (EL -4'-5").
- The equivalent floor-by-floor static wind loads are given in Table 3 for a 10 year return period and in Table 4 for a 50 year return period and for damping values of 1.5%, 2.0%, 2.5%, 3.0% and 4.0% of critical. These are to be applied at the centre of coordinates given in Figure 7 at every floor level. Diagrams of the distributed equivalent static forces, corresponding to the predicted base moments, for a damping ratio of 2.0% of critical, are shown in Figures 10.
- Combined load cases should be considered in order to ensure that the combined action of various wind forces is allowed for properly. Table 5 contains the relevant load combination factors to be used in conjunction with the above equivalent static wind loads.

Local Differential Pressures

- Unless otherwise noted, the results contained in this report are based on the as tested building geometry. The additional details of the double wall systems at the top of the building (parapet) and the winter garden areas are given special attention and is discussed further in Section 5.
- Internal pressure coefficients were determined assuming a nominally-sealed building and were subtracted from the external pressure coefficients. The internal pressures could be larger if operable windows were open or the building envelope was to be breached during a storm event. For the case of the double wall system present for the first four levels of the Winter Garden area, an additional study was conducted to better estimate the differential pressures and suctions across the inner and outer walls. See Section 5.2.1.1 for further details.
- The resulting differential pressure coefficients were combined with the design probability distribution of wind speed and direction to form predictions of differential suctions and pressures for various return periods. The results are summarized in block zone format in Figures 11 and 12.
- When considering cladding elements exposed to the internal pressure of the building, the largest predicted differential pressure and suction for a return period of 50 years are 46.0 psf and 84.6 psf, respectively. The largest differential pressure occurs at tap location 412 (south elevation near level 49) and the largest differential suction occurs at tap location 109 (north elevation near level 54).
- The largest predicted net differential pressure and suction for the locations indicated in Appendix E for a return period of 50 years are 50.7 psf and 68.8 psf, respectively. The largest differential pressure occurs at tap location 1068 (west elevation parapet wall) and the largest differential suction occurs at tap location 1019 (north elevation parapet wall). The differential pressures for the double wall parapet at the top of the tower are discussed in Section 5.2.2.1.
- Table 7 summarizes the 20 largest predicted differential pressures and suctions and their corresponding tap location for each of the above cases. Table 8 contains the estimates of the differential pressures across the inner and outer walls of the Winter Garden region.
- None of the local pressures include any allowance for stack (thermal) effects or the direct effects of mechanical systems. The Canadian building code recommends an allowance for stack effects of 0.2 kPa per 100m (equivalent to about 4psf per 330 ft.) of building height and an allowance of 0.1 kPa (2 psf) for mechanical system effects. These allowances would be added to both the differential suctions and the differential pressures.

Pedestrian Wind Environment

- Figure 16 shows the locations where pedestrian level wind speeds were measured.
- Experimental results have been combined with the extratropical wind climate to provide predictions of the wind speeds expected to be exceeded for 5% of the time and those expected to be exceeded once per year. These predictions can be compared directly with acceptance criteria for pedestrian comfort and safety respectively.
- Figure 17 shows that all of the measured locations are acceptable based on the safety criteria for all-weather areas. When compared to the comfort criteria, all locations are acceptable for common activities with exposures of long duration. Near the main entrances: wind speeds are moderate - suitable for prolonged stays such as short or long sitting. Location 16 is located in the plaza area, not far from the southwest building entrance. Based on our comfort criteria, this location exhibits wind speeds which are slightly higher than the other locations and would be suitable for longer duration activities such as leisurely walking. A number of the locations produced predicted pedestrian level wind speeds which exceed those typically experienced in a suburban terrain. Some of these locations approach wind speeds typically encountered in open country terrain. Under these circumstances, particularly those approaching the open country benchmark, pedestrians may experience wind conditions to which they may be unaccustomed to in an urban setting.
- Figures 18 and 19 provide colour coded diagrams which summarize the suitability of each measurement location with respect to pedestrian level comfort and safety respectively. The comfort and safety categories used correspond to those summarized in Section 6.5.
- Compared to the annual wind speeds presented here, wind speeds in spring and winter are on average about 9% higher and in summer they are about 22% lower. Autumn does not differ much from the annual wind speeds reported.

Notes

- Predictions for an R-year return period (mean recurrence interval of R years) represent levels which are expected to occur *on average* once in R years. For reference, the risk of exceeding an R-year return period load in a design life of L years is $1-(1-1/R)^L$. Thus, for example, the risk of exceeding a 50 year load in a design lifetime of 50 years is about 64%, whereas the risk of exceeding a 1000 year load in a 50 year design life is about 5%.
- The predictions in this report are best estimates and **do not** include any load or safefy factors such as those typically required by building codes.

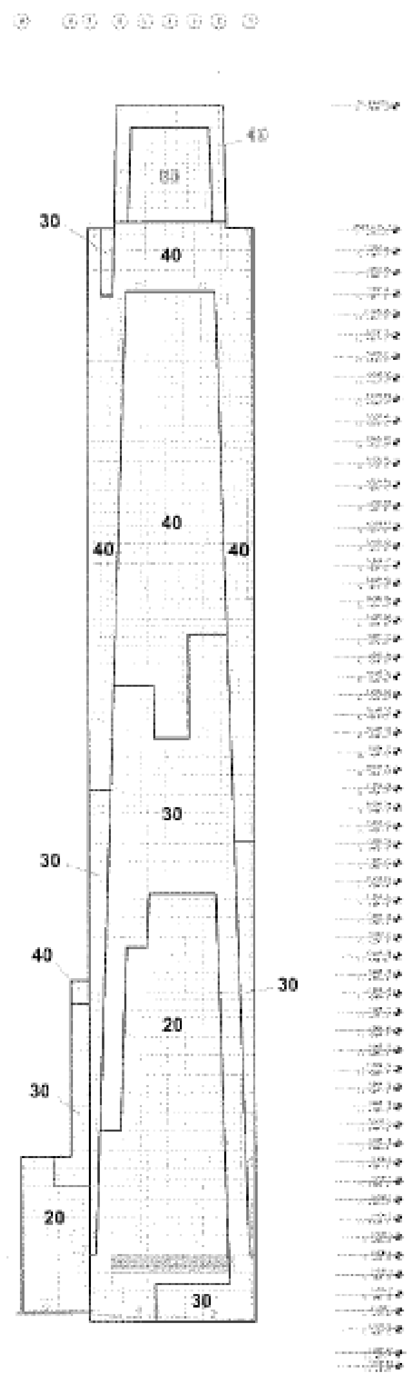
TABLE 2a LOADS AND RESPONSES FOR ONE PENNSYLVANIA PLAZA FOR A 10-YEAR RETURN PERIOD

VARIABLE	Damping Ratio				
	ξ = 1.5%	ξ = 2.0%	ξ = 2.5%	ξ = 3.0%	ξ = 4.0%
X Acceleration (milli-g)	9.0	7.8	7.0	6.4	5.5
Y Acceleration (milli-g)	37.9	32.8	29.4	26.8	23.2
Torsional Acceleration (milli-g)	10.0	8.6	7.7	7.1	6.1
Centroidal Acceleration (milli-g)	38.1	33.0	29.5	26.9	23.3
Corner Acceleration (milli-g)	38.4	33.2	29.7	27.1	23.5
Torsion Velocity (milli-rads/sec)	1.1	1.0	0.9	0.8	0.7
X Moment (lb-ft)	1.46E+09	1.42E+09	1.39E+09	1.36E+09	1.34E+09
Y Moment (lb-ft)	2.95E+09	2.69E+09	2.51E+09	2.39E+09	2.23E+09
Torsional Moment (lb-ft)	6.21E+07	6.04E+07	5.94E+07	5.87E+07	5.79E+07

Notes:

1. All loads and responses above are for a 10-year return period.
2. Moments are calculated about basement level B3 (EL -4'-5").
3. Accelerations are calculated at a height 872.4ft. above level B3, corresponding to the top occupied floor (floor 55).
4. Torsional accelerations are expressed as linear accelerations at a distance of 95.0ft. from the report centre of coordinates (the farthest distance from the centre a person could stand).
5. Centroidal accelerations are the combination of X and Y accelerations with an appropriate coincident action factor.
6. Corner accelerations are the combination of X, Y and T accelerations with an appropriate coincident action factor.
7. Damping: As Shown
8. Periods:

MODE	MODAL MASS FACTORS			UPPER BOUND PERIOD (seconds)
	X	Y	T	
1	0.000	1.000	0.000	7.39
2	0.998	0.000	0.002	3.73
3	0.002	0.006	0.992	2.01



NOTES:

1. Differential pressures and suctions are defined as net loads per unit area respectively acting inward and outward from the surface shown in the drawing. Like all pressures, loads act normal to the actual local building surface.

2. Where applicable, the data include the effects of both the external and internal wind-induced pressures. The internal building pressures were determined assuming a nominally sealed building in which leakage is through many small, well-distributed holes. Larger pressures and suctions could develop in the presence of a larger opening such as operable windows or due to accidental window breakage. No allowances have been made for stack (thermal) and mechanical system effects.

3. For free standing elements such as parapets and canopies, the data include the net effect of the external wind-induced pressures acting on the front and back or top and bottom surfaces.

4. No allowances have been made for any possible increase in the wind-induced pressures due to resonant vibrations of the cladding components.

5. These wind-induced pressures do not include any load or safety factors.

6. Where the external load is greater than the differential load, the external load is reported.

7. A minimum or baseline value of 20 psf was used with increments of 10 psf.

UNITS: psf

LEGEND:

40 Solid numbers represent pressures or suctions based on the internal pressures of the building.

40 Open-faced numbers represent pressures or suctions across parapets or canopies.

(40) Leader indicates coned surface is perpendicular to drawing plane.

FIGURE 11b BLOCK DIAGRAMS OF PREDICTED PEAK DIFFERENTIAL PRESSURES (i.e. inward-acting loads) FOR A 50-YEAR RETURN PERIOD - East Elevation.

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H.12.1 SUSTAINABILITY BREADTH CALCULATION

AE 482: WIND TURBINE BREADTH — CYNTHIA MILINICHIK

FIND: ENERGY USED BY BUILDING
TO CALCULATE WIND TURBINE REQ'TS

SOLN: USE BUILDING AREA METHOD

(NOTE: DISTRIBUTION PANELS ≤ 5125 A ∴ DOESN'T SEEM RIGHT)

NOTE: THIS IS AN ESTIMATION APPROACH SINCE THE BUILDING IS NOT COMPLETE YET THE ACTUAL ENERGY USAGE IS NOT KNOWN. THIS APPROACH SHOULD YIELD A MORE CONSERVATIVE ANSWER

TOTAL BUILDING AREA: 1.2 M SF — 1,200,000 SF

ELECTRICAL LOAD - MECHANICAL SYS. — 8.0 VA / SF

LIGHTING LOAD
NEC 2005 TABLE 220.12
"OFFICE BUILDING — 3.5 VA/SF

RECEPTACLE LOAD — 0.5 VA/SF

TOTAL:
$1{,}200{,}000\ SF\,(8+3.5+0.5)\ VA/SF = 11{,}700{,}000\ VA$
$= 11{,}700\ KVA$

AEROTURBINES: 1 KW IN 30 MPH

PHILADELPHIA = 0.33 KW IN 9.5 MPH (MEAN WIND SPEED) @ 30' STANDARD

AT 1000' HAVE A MUCH HIGHER WIND SPEED

WIND GRADIENT

$$U_{1000'} = \frac{1}{K}\log_e \frac{1000'}{30'}(U_{30'}) = \frac{1}{0.4}\ln\left(\frac{1000'}{30'}\right)(9.5\ MPH)$$

K = 0.4 KARMAN'S CONSTANT

WIND SPEED @ 1000'
$U_{1000'} = 83\ MPH$

$$\frac{83\ MPH}{30\ MPH} = \frac{x}{1\ KW} \qquad 28\ KW = x$$

TURBINE SIZE: 5'×5'×10' CAGE
COST $ 20,000 - $15,000
ROOF DIMENSIONS 70'×120'

#	COST	KW	%
5	$100,000	14	0.12%
10	$200,000	28	0.24%
15	$300,000	35	0.30%

FIGURE H-12.1: BUILDING ENERGY AND WIND TURBINE ENERGY CALCULATION