STRUCTURAL TECHNICAL REPORT 3
LATERAL SYSTEM ANALYSIS AND CONFIRMATION DESIGN

EXECUTIVE SUMMARY

This third technical assignment is an analysis that confirms the design of the existing lateral load resisting system of the new tallest building in Philadelphia, the Comcast Center. Currently under construction, the Comcast Center will be 57 stories tall reaching a height of 1,002 feet. The glass-clad skyscraper will primarily function as office space with a few retail and restaurant spaces. This LEED-certified structure promotes public transportation with a new grand entrance to the Suburban Station providing access to the commuter rail and two subway lines.

The gravity load system of the Comcast Center is composed of a composite metal deck floor supported by steel beams. The steel beams frame into steel columns along the perimeter of the building and a massive concrete core at the center of the building.

Lateral loads from wind and seismic activity are resisted by the massive concrete core walls which range from 1'-6" thick to 4'-6" thick and act as shear walls. The glass façade is supported at every floor with a steel tube which resists the local lateral forces caused by wind.

The total drift of the structure is 17.5 inches. A drift limit of L/600 allows for 20 inches of lateral drift. Due to the cantilevered condition of the building the story drifts experienced at the top of the structure are greater than the story drifts at the base.

Member checks such as shear strength were performed to confirm the design of the shear walls. The total shear strength of the core walls is 22,000 kips. This is much greater than the 6,300 kips the structure experiences. With a shear strength 3 times greater than structurally necessary it poses the question of whether or not all the shear walls are needed. This theory will be explored in my thesis this Spring.

INTRODUCTION AND BACKGROUND

The Comcast Center is a 57 story glass-clad office building located in Philadelphia, Pennsylvania. The top of the roof of the Comcast Center is at a height of 1001 feet and 6 inches. The structural system of the Comcast Center is composed of a concrete core which supports the steel framed building shell. The slab is typically composite metal deck with 3-1/4 inches of concrete on 3 in metal deck. The composite action of the floor slab allows for a minimal floor depth. Figure 1 below is a sketch of the 19th & 20th floor framing plan which is representative of the framing of the other floors.

The gravity loads are resisted by the composite deck slab, steel beams and columns and the concrete core. Lateral loads are resisted through the thick concrete shear walls of the core at the center of the structure. Six shear walls run north and south and range in thickness from 1 feet 6 inches to 2 feet at the base. The number and thickness of the shear walls decreases as higher floors. Figure 2 below is a photograph of the concrete core.

The foundation is designed with an allowable bearing capacity of 20 tons per square foot. Caissons are located under all columns and are socketted a minimum of 6 feet into rock. The caissons range in thickness from 3 to 8 feet. A 10 foot thick mat foundation supports the concrete core. The perimeter dimensions of the concrete core are 130 feet by 48 feet. The mat foundation is 156 feet by 76 feet to distribute the load from the concrete core.

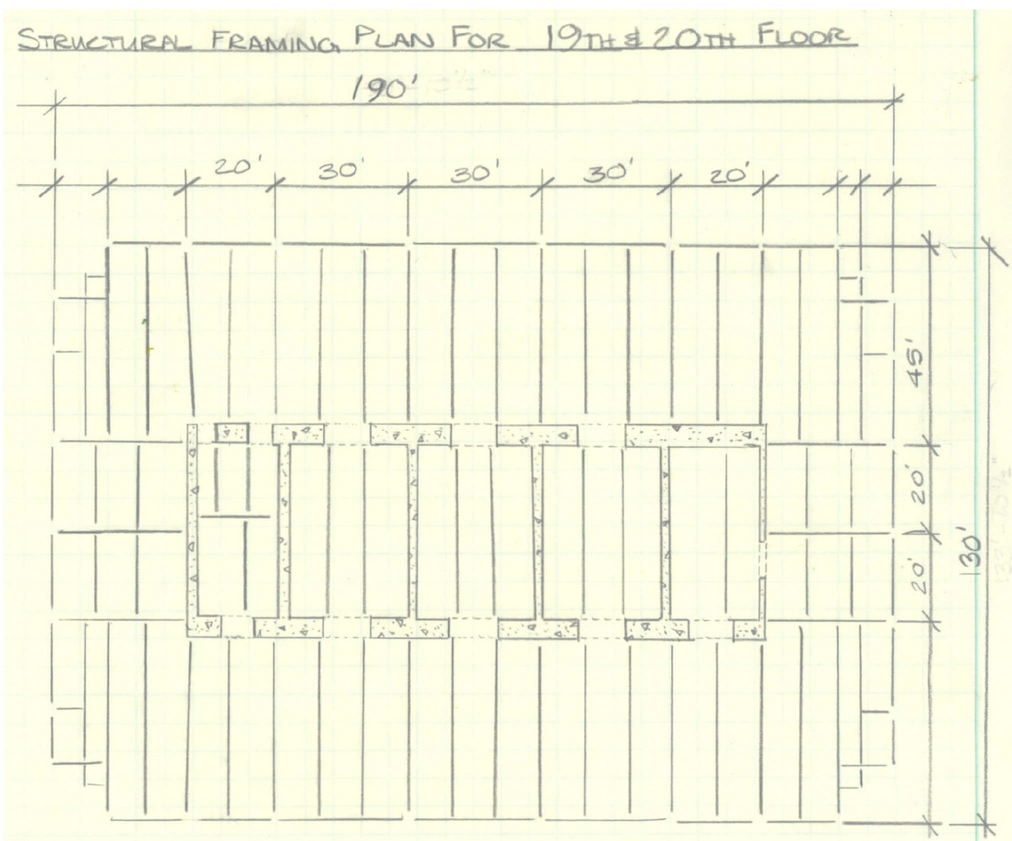


FIGURE 1: TYPICAL 19TH & 20TH LEVEL FLOOR FRAMING PLAN

Photograph Courtesy of R. Bradley Maule

FIGURE 2: CONCRETE CORE CONSTRUCTED FIRST, STEEL ERECTION FOLLOWS

The format of this report is as follows:

- Loads & Load Cases
- Distribution of Loads
- Analysis
- Member Checks
- Conclusion

LOADS & LOAD CASES

WIND LOAD

A wind tunnel test was performed to quantify and assess the wind loads on the Comcast Center. A summary of the wind tunnel test was acquired and used to approximate the wind loads acting on the main wind force resisting system. Only one elevation was provided. It was assumed that a similar loading pattern was experience on all facades in order to complete this assignment. The wind load resisted by the Main Wind Force Resisting System ranges in magnitude from 20psf to 50 psf. Local areas of the façade experience wind loads as high as 70 psf. These areas are rather small compared to the rest of the surface area of the building.

DEAD LOAD

The dead load of the structure were calculated based on the information provided in the design documents. Light weight concrete with a weight of 115 pcf was used for the composite metal deck floor slabs. The concrete core was constructed with reinforced normal weight concrete weighing 150 pcf. Dead Loads used for calculating seismic loads were based on the space planning diagrams in the architectural drawings. A spreadsheet was used to calculate the weight of the structure for seismic loads.

LOAD CASES

The critical load case was 1.2D + 1.6W +0.5L + 0.5Lr. To determine over turning moment the load case 0.9Dead + 1.6Wind was used.

LATERAL SYSTEM

The concrete shear walls of the central core make up the lateral force resisting system. Six shear walls ranging in thickness from 1 foot 6 inches to two feet run north and south along grid lines C, D, E, F, G and H. The length of these walls ranges from 45 to 48 feet depending on the thickness of the flange element. Figure 3 identifies the flange and web elements of the concrete core. Shear walls in the east and west direction are short due to the large walkway opening. These short shear wall segments are connected with a deep beam.

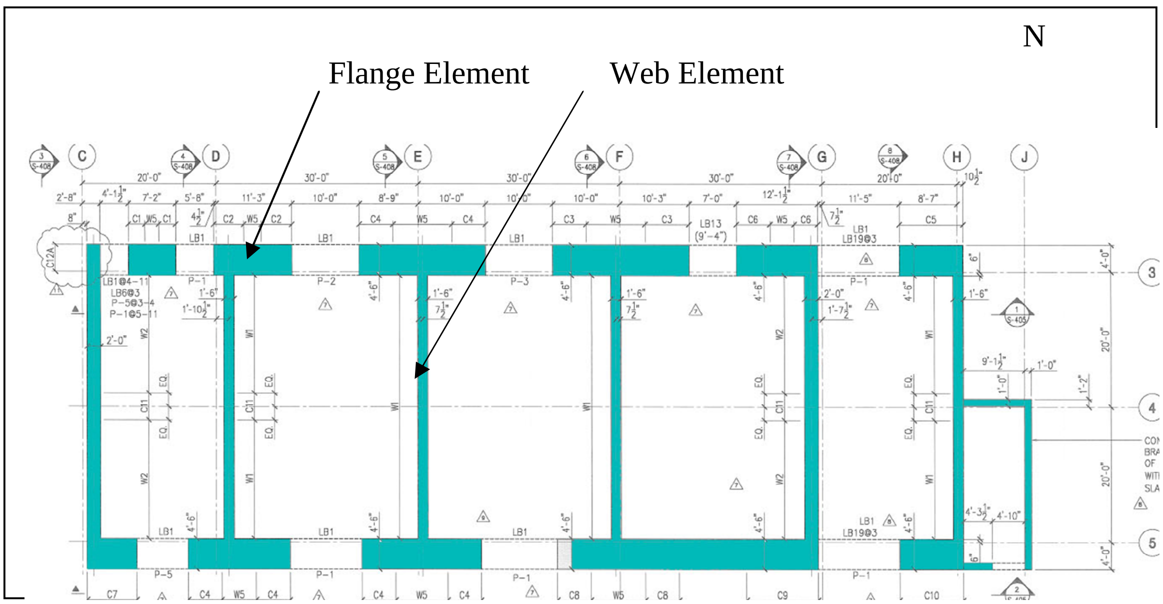


FIGURE 3: CONCRETE CORE SHEAR WALLS

DISTRIBUTION OF LOADS

LOAD PATH OF WIND LOADS

As wind applies pressure and suction on the glass façade of the Comcast Center the load is transferred to the steel framing members. The lateral load transferred to the steel framing members is passed through the rigid floor diaphragm which on the majority of floors is a composite concrete and metal deck floor slab. The diaphragm transfers the load to the massive concrete core located at the center of the building. The web members of the concrete core resist the wind load applied to the north and south façades. The flanges of the concrete core resist the wind load applied to the east and west facades. The flanges of the concrete core resist the overturning moment in the weak direction of the building. Figures 4 and 5 below illustrate the wind load applied to the structure and the load path.

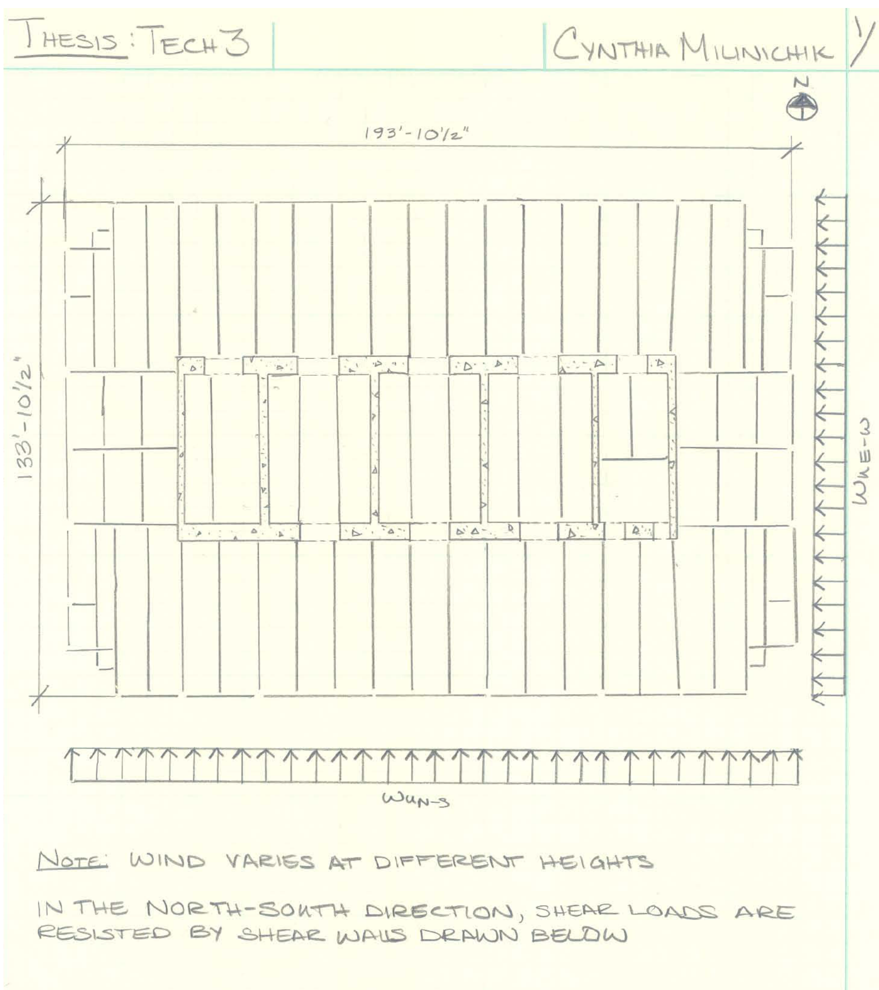


FIGURE 4: TYPICAL FLOOR PLAN WITH WIND LOADING

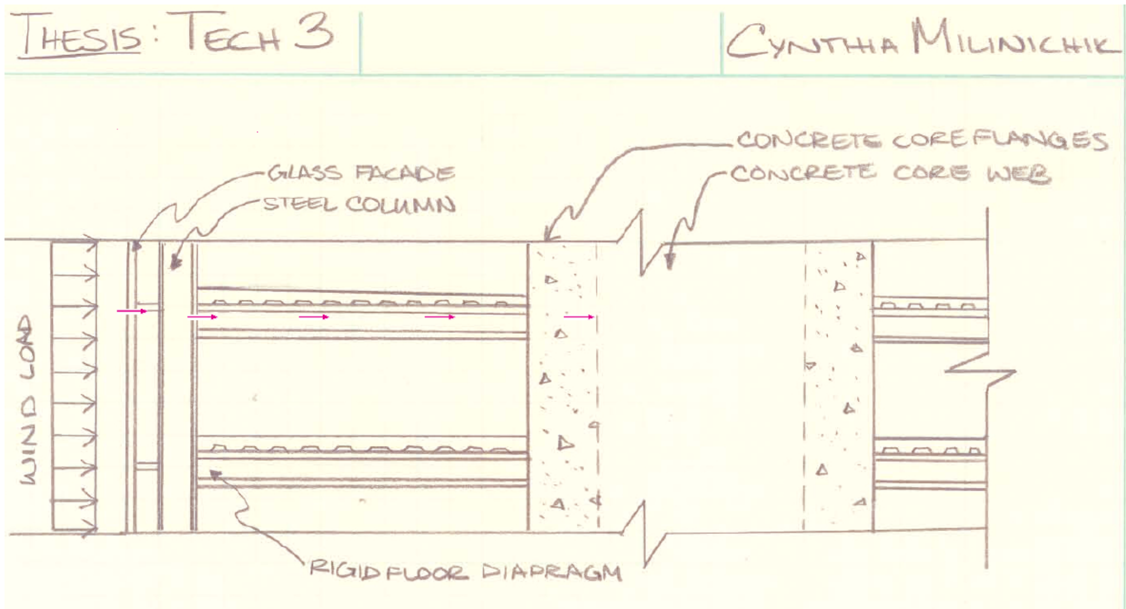


FIGURE 5: LOAD PATH FROM GLASS FAÇADE TO FRAMING TO CONCRETE CORE

The 2 foot thick web elements of the concrete core are stiffer than the 1 foot 6 inch web elements. The load applied to the main force resisting system is distributed to the shear walls in proportion to the stiffness of the wall. In the north and south direction all the shear walls are the same length. This, however, is not the case for the flange elements of the concrete shear wall. The flange elements are shorter and thicker than the web elements. The flange elements transfer shear force through deep beams that connect the short flange shear walls.

ANALYSIS

It was determined in Technical Report one that wind loads controlled the design of the lateral load resisting system. Corrections were made to the seismic calculations. The corrected calculations for seismic loading are included in Appendix (). Seismic loads were calculated using BOCA 96. Figure 6 below shows the distribution of the seismic forces on the Comcast Center. Table 1 was used to calculate the weight, height, vertical distribution factor, and the lateral force at each level due to seismic activity. The base shear calculated is 1,492 kips.

SEISMIC LOAD CALCULATIONS

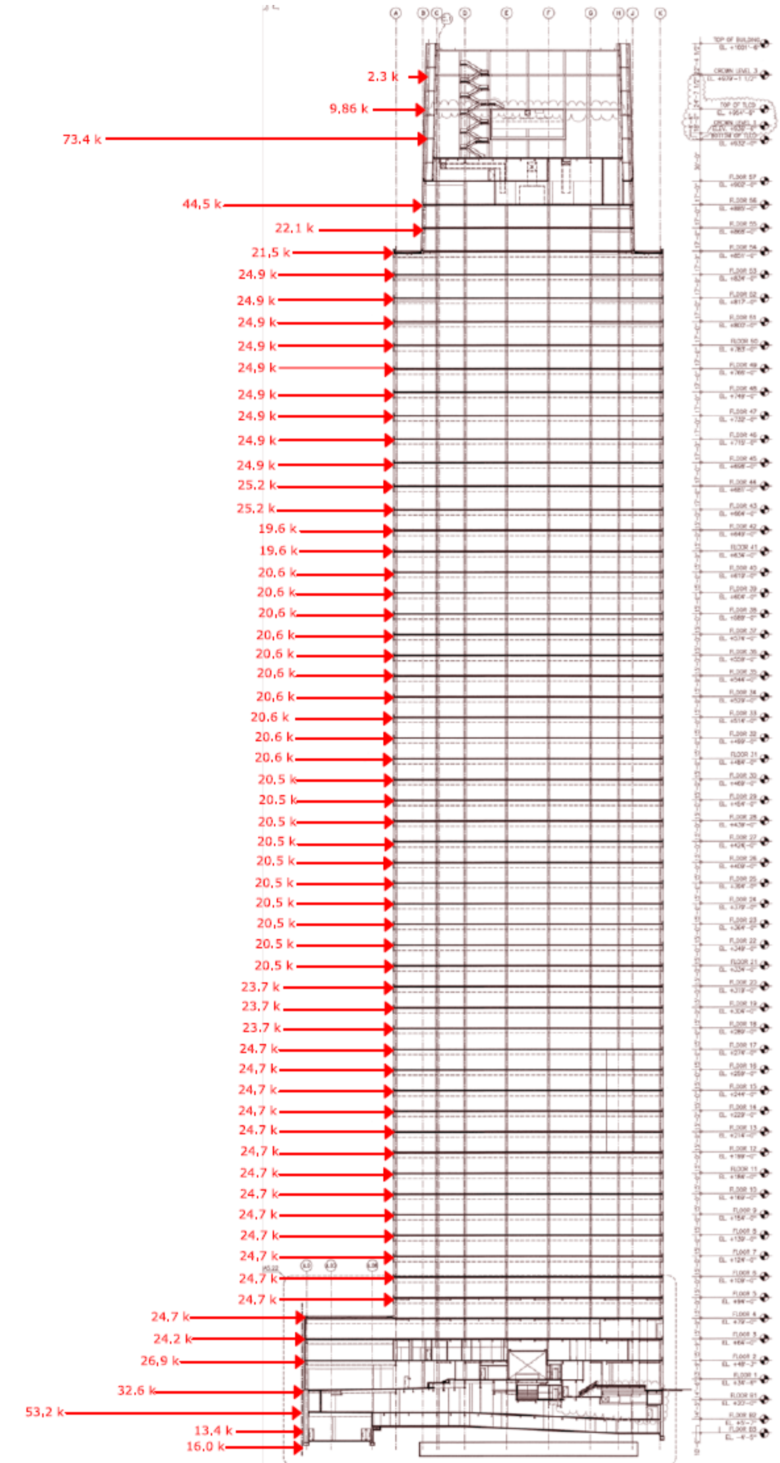


FIGURE 6: LATERAL LOAD PER FLOOR RESULTING FROM SEISMIC LOADING

Table 1: Seismic Loads

k=	Level	w_x (k)	h_x (ft)	$w_x h_x^2$	C_{vx} (k)	F_x(k)
						V=1492
Crown 3	59	180	22	87120	0.001508	2.25
Crown 2	58	788	22	381513	0.006606	9.86
Crown 1	57	1373	45.5	2842971	0.049223	73.44
Office	56	5958	17	1721975	0.029814	44.48
Office	55	2953	17	853397	0.014776	22.05
Office	54	2885	17	833904	0.014438	21.54
Office	53	3337	17	964260	0.016695	24.91
Office	52	3337	17	964260	0.016695	24.91
Office	51	3337	17	964260	0.016695	24.91
Office	50	3337	17	964260	0.016695	24.91
Office	49	3337	17	964260	0.016695	24.91
Office	48	3337	17	964260	0.016695	24.91
Office	47	3337	17	964260	0.016695	24.91
Office	46	3337	17	964260	0.016695	24.91
Office	45	3337	17	964260	0.016695	24.91
Office	44	3372	17	974473	0.016872	25.17
Office	43	3372	17	974473	0.016872	25.17
Office	42	3372	15	758673	0.013136	19.60
Office	41	3372	15	758673	0.013136	19.60
Office	40	3552	15	799173	0.013837	20.64
Office	39	3552	15	799173	0.013837	20.64
Office	38	3552	15	799173	0.013837	20.64
Office	37	3552	15	799173	0.013837	20.64
Office	36	3552	15	799173	0.013837	20.64
Office	35	3552	15	799173	0.013837	20.64
Office	34	3552	15	799173	0.013837	20.64
Office	33	3552	15	799173	0.013837	20.64
Office	32	3552	15	799173	0.013837	20.64
Office	31	3552	15	799173	0.013837	20.64
Office	30	3531	15	794385	0.013754	20.52
Office	29	3531	15	794385	0.013754	20.52
Office	28	3531	15	794385	0.013754	20.52
Office	27	3531	15	794385	0.013754	20.52
Office	26	3531	15	794385	0.013754	20.52
Office	25	3531	15	794385	0.013754	20.52
Office	24	3531	15	794385	0.013754	20.52
Office	23	3531	15	794385	0.013754	20.52
Office	22	3531	15	794385	0.013754	20.52
Office	21	3531	15	794385	0.013754	20.52
Office	20	4071	15	915885	0.015858	23.66
Office	19	4071	15	915885	0.015858	23.66

Office	18	4071	15	915885	0.015858	23.66
Office	17	4243	15	954702	0.016530	24.66
Office	16	4243	15	954702	0.016530	24.66
Office	15	4243	15	954702	0.016530	24.66
Office	14	4243	15	954702	0.016530	24.66
Office	13	4243	15	954702	0.016530	24.66
Office	12	4243	15	954702	0.016530	24.66
Office	11	4243	15	954702	0.016530	24.66
Office	10	4243	15	954702	0.016530	24.66
Office	9	4243	15	954702	0.016530	24.66
Office	8	4247	15	955514	0.016544	24.68
Office	7	4247	15	955514	0.016544	24.68
Office	6	4247	15	955514	0.016544	24.68
Office	5	4247	15	955514	0.016544	24.68
Office	4	4247	15	955514	0.016544	24.68
Office	3	4169	15	937977	0.016240	24.23
Office	2	4193	15.75	1040177	0.018010	26.87
Office	1	6681	13.75	1263076	0.021869	32.63
Parking	B1	9799	14.5	2060272	0.035672	53.22
Parking	B1.5	4012	10	401234	0.006947	10.36
Parking	B2	5168	10	516826	0.008948	13.35
Parking	B3	6184	10	618359	0.010706	15.97
Totals		240319	1001.5	57756557	1	

The wind load condition controls the lateral force resisting system design. Wind loads create a base shear of 6,247 kips on the Comcast Center. Elevations of the Comcast Building are given in Figure 7. The slight contours of the façade contribute to the wind loading experienced by the building. A representative elevation of the concrete core is presented in Figure 8. Figure 9 below results from the wind tunnel test performed and indicates the pressures experienced by the main wind force resisting system. The simplifying assumption used to calculate the base shear on the Comcast Building is evident in Figure 10. The structure was treated as a cantilevered beam for the base shear calculation.

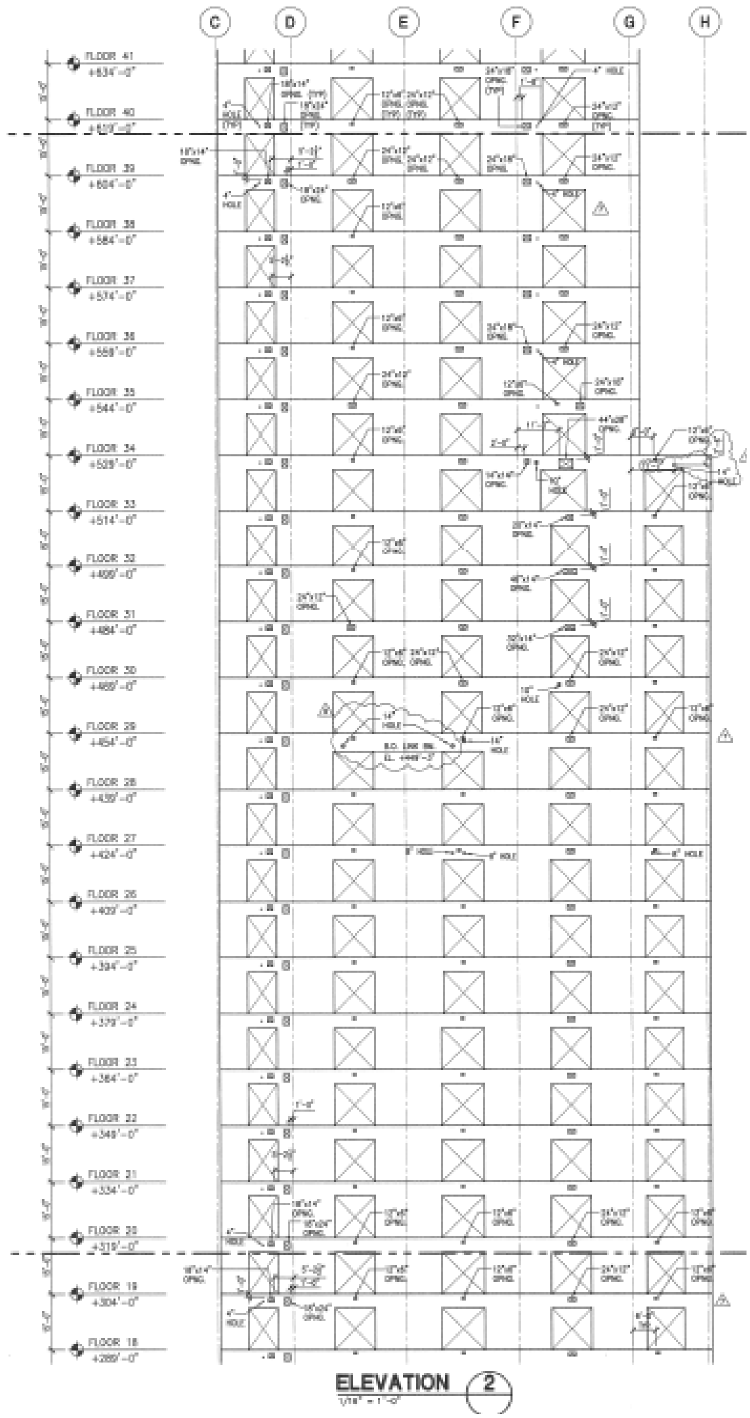


Figure 8: Typical Elevation of Concrete Core

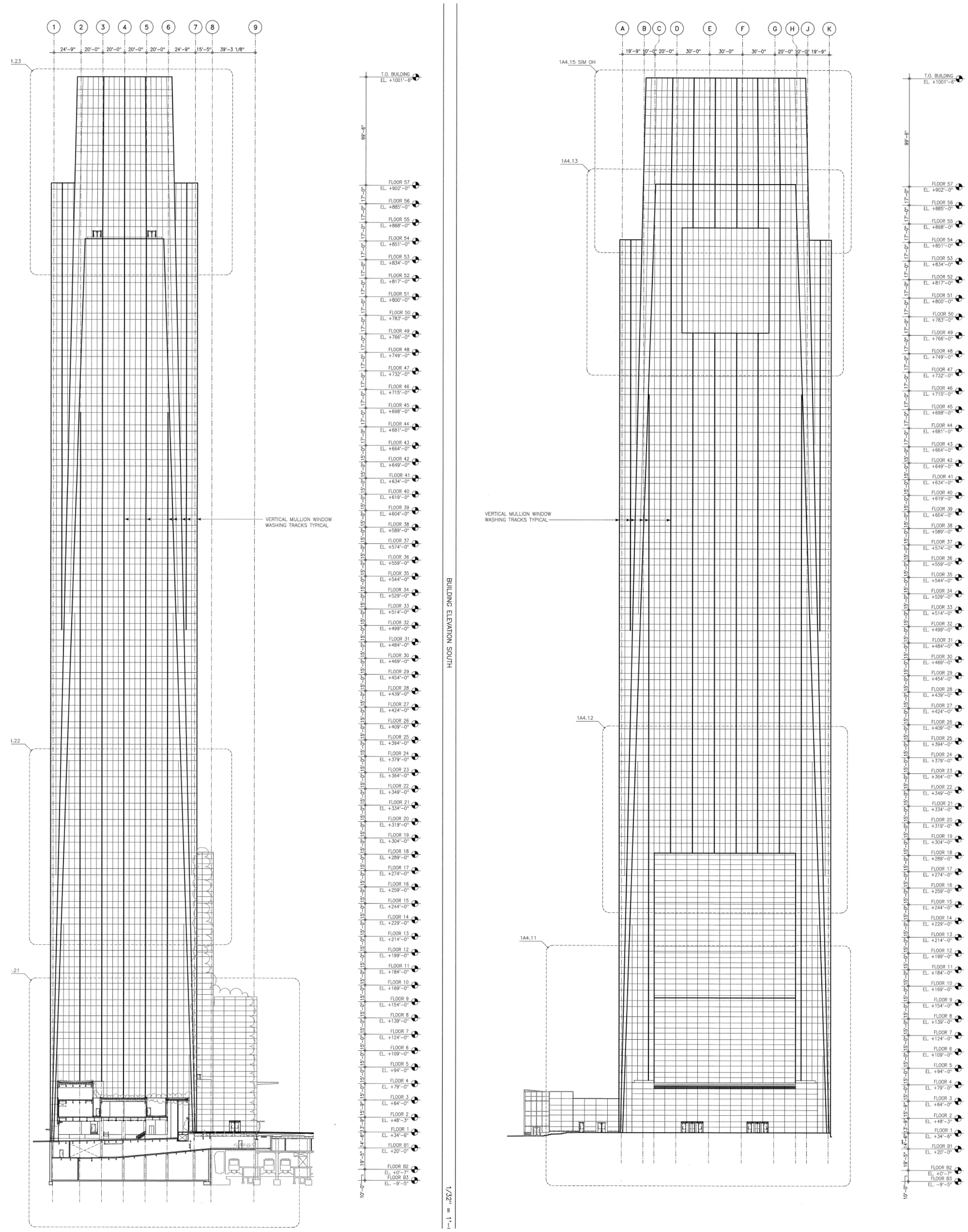


FIGURE 7: ELEVATIONS OF THE COMCAST CENTER

WIND LOAD CALCULATIONS

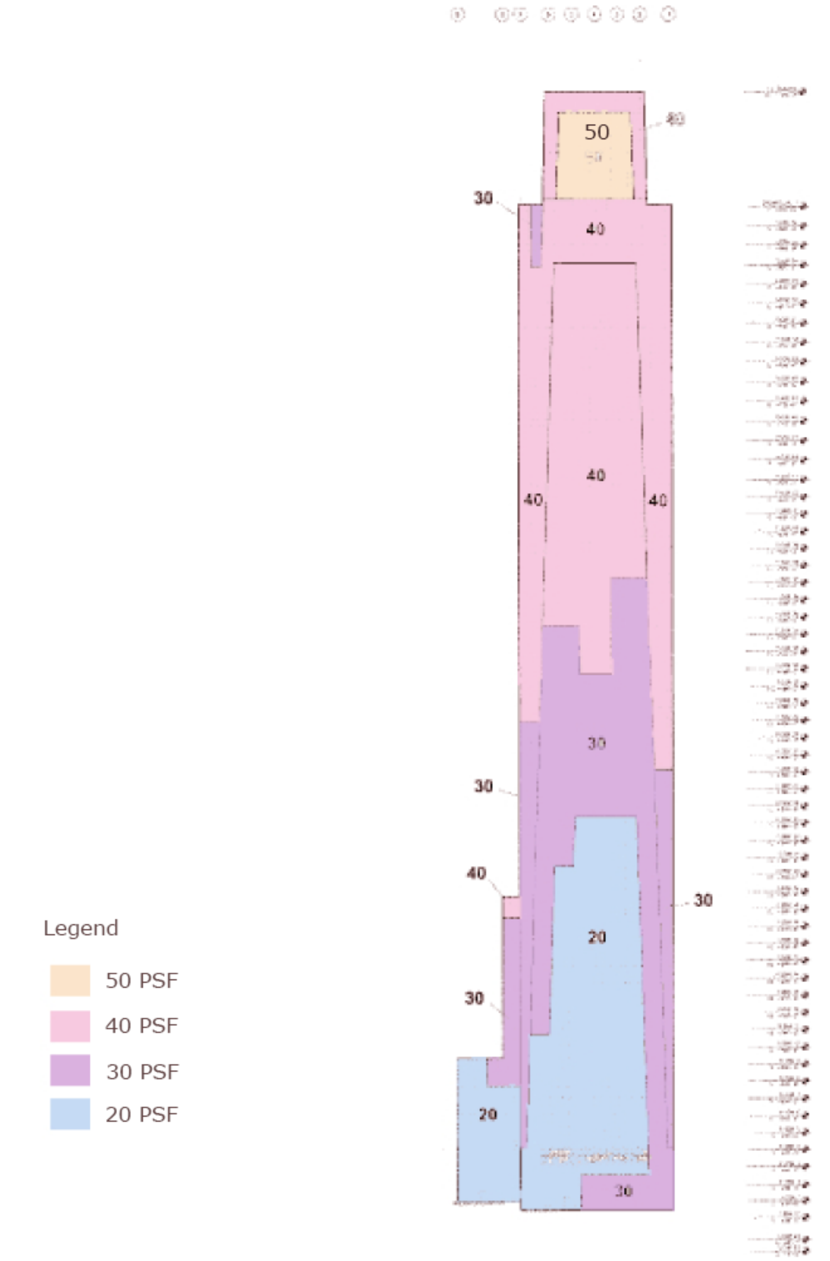


FIGURE 9: PRESSURES ON THE MAIN FORCE RESISTING SYSTEM OF THE COMCAST CENTER DETERMINED THROUGH WIND TUNNEL TESTING

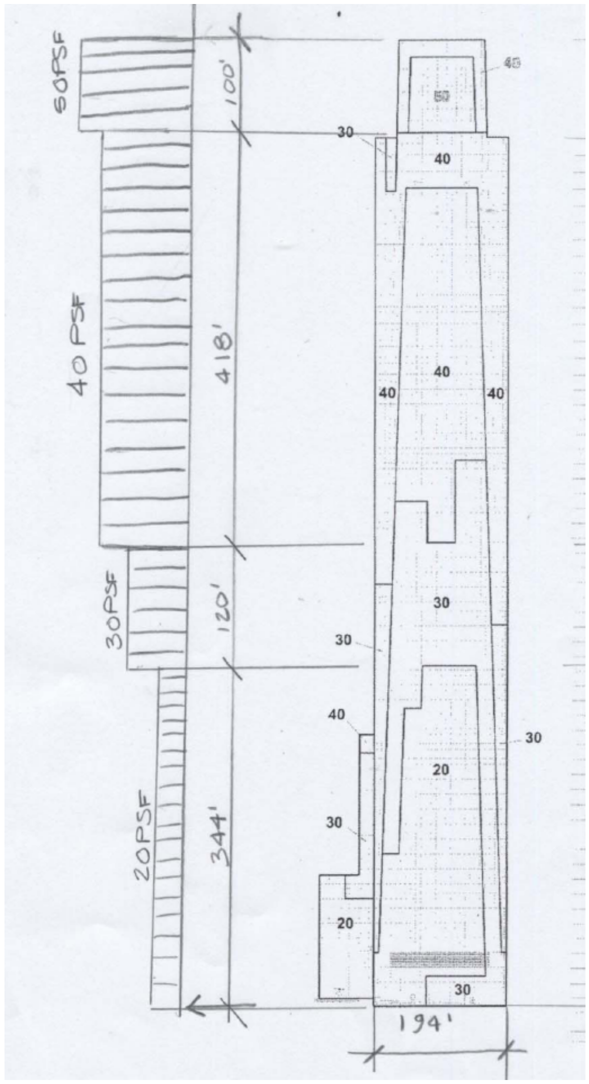


FIGURE 10: WIND LOADING ANALYZED AS A CANTILEVERED BEAM

Detailed calculations for the story drift and the drift of the over all structure can be found in Appendix A. The story drift is 5.83x10^(-5) inches for the levels with a 15 foot story height. The drift of the entire structure was calculated by dividing the building up in segments and using superposition. The angle of rotation at the start of each segment needed to be calculated in order to use superposition. The overall drift of the structure is approximately 17.5 inches. Using a limit of L/600 the Comcast Center would be able to move 20 inches. The 17.5 inches is an acceptable drift value for the 1001.5 foot tall office building.

MEMBER CHECKS

The shear strength of each type of shear wall was calculated. For wall type W1 the factored shear strength is 3,255 kips. Wall W1 is 1 foot 6 inches thick. Wall typeW2 is 2 feet thick and has a factored shear strength of 4505 kips. The total shear strength of the six walls is 22,034 kips. The base shear caused by the wind load is 6,247 kips which is significantly less the shear strength of the walls. This suggests that the size and or number of shear walls is not structurally necessary. This theory will be explored in greater depth in the thesis to come next semester. Figure 11 displays the type of shear walls.

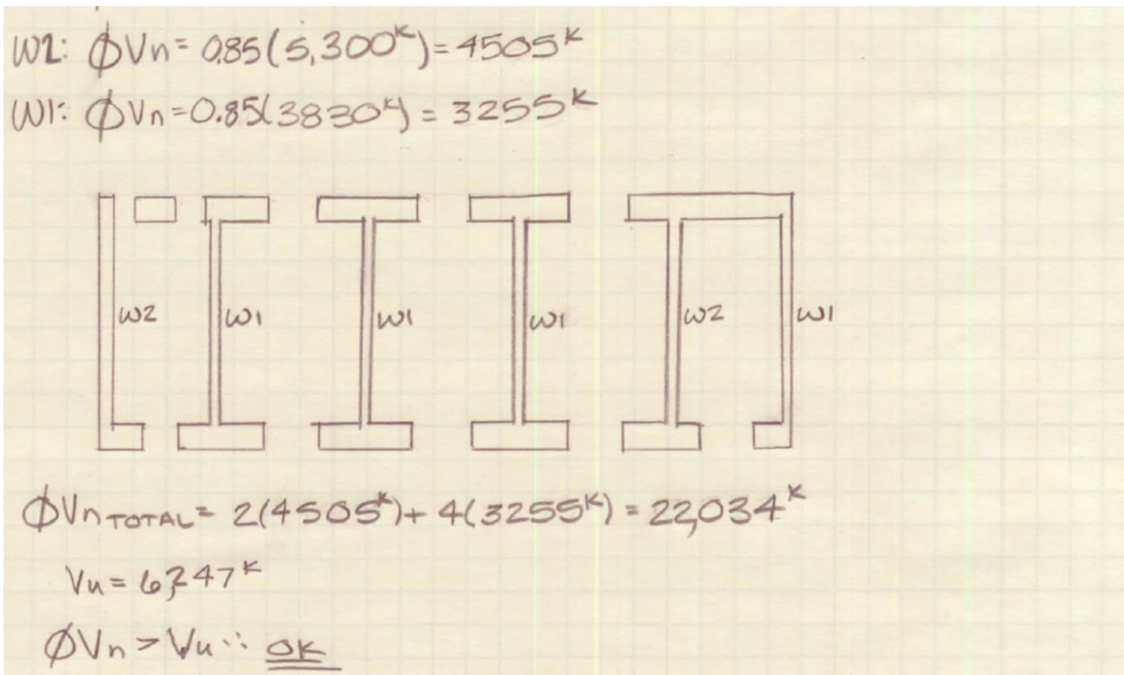


FIGURE 11: SHEAR STRENGTH CALCULATION. THE TYPE OF SHEAR WALL IS INDICATED IN THE SKETCH.

CONCLUSION

The existing lateral system appears to be more than adequate for the wind loads experienced. The shear strength of the shear wall was more that 3 times load experience by the wind. The drift of the tower was only 17.5 inches. The fact that the shear strength was so higher than the need strength suggests that the concrete core is larger than it needs to be. This theory will be tested in the thesis next semester by removing the outer two web members.

Note: A computer model in Etabs was being used to analyze the forces and the reactions of the lateral system. Due to time restraints and complications with the software, the model was not completed in time for this report. The model will be used for the thesis next semester.

APPENDIX A: DRIFT CALCS

THESIS : TECH 3 | CYNTHIA MILINICHIK | 1/4

NORTH SOUTH DIRECTION

$V_B = (194')(344')(20PSF) + (194')(120')(30 PSF) + (194')(418')(40PSF) + (152')(100')(50PSF)$ 6036.8^K

$V_B = 6036.8^K$

NOTE: WEBS C & G ARE 2' THICK, WEBS D, E, F, H ARE 1.5' THK
$C = 1\tfrac{1}{3} D$

$V = V_B / \text{SHEAR WALLS}$

$6036.8 / (4 + 1\tfrac{1}{3}(2)) = 907^K$ TO WEBS D, E, F, H
1204^K TO WEBS C & G

NOTE: SOME OF THE WEB MEMBERS OF THE CORE DROP OUT AS ELEVATION INCREASE. TO ACCOUNT FOR THIS THE CALCULATIONS WILL BE SEGMENTED AT LEVELS WHERE THESE CHANGES OCCUR

LEVEL 1-32: ELEVATION @ TOP OF 32ND FLOOR = 479.5'

$$\Delta_{@TOP} = \frac{1.5V}{bE}\left(\frac{H}{d}\right) + \frac{1.5V}{bE}\left(\frac{H}{d}\right)^3$$

$$\Delta_{SHEAR} = \frac{1.5(431^K)}{24(\cancel{4000}\ 5700)}\left(\frac{479.5}{48'}\right) = \cancel{0.067} = 0.0472''$$

NOTE: ASSUME $E = 4{,}000^{KSI}$ FOR CONCRETE

$$\Delta_{BENDING} = \frac{1.5(431^K)}{24(\cancel{4000}\ 5700)}\left(\frac{479.5}{48}\right)^3 = \cancel{6.71''}\ 4.711$$

$\Delta_{@TOP} = \cancel{6.777''} = \underline{\underline{4.758''}}$

$V_{32ND}(479.5') = (194)(344)(20) + (194)(120)(30) + (15.5)(194)(40)$
$= 2153.4^K$

$2153.4 / (4 + 1\tfrac{1}{3}(2)) = 323^K$ TO WEBS D, E, F, H
431^K TO WEBS C & G

$\Delta_{@TOP} < L/400 = (479.5)(12)/400 = 14.385'' > \cancel{6.77''}\ 4.758''$ ∴ OK

THESIS: TECH 3 | CYNTHIA MILINICHIK 2/

LEVELS 33 - 55

ELEVATION @ TOP OF 55TH LEVEL = 885'

DIFFERENCE: ~~885' - 479.5' = 405.5'~~

418 - 15.5 = 402.5'

$V_B = (194)(402.5')(40\ PSF) + (152)(3')(50\ PSF) = 3146.2$

NOTE: WEB H NO LONGER EXISTS
5 WEBS

$V = V_B / \#\ WALLS$

$V = 3146.2 / (3 + 1.33(2)) = 556^K$ TO WEBS D, E, F,
739^K TO WEBS C & G

$\Delta_{TOP} = \frac{1.5V}{bE}\left(\frac{H}{d}\right) + \frac{1.5V}{bE}\left(\frac{H}{d}\right)^3$

$\Delta_{SHEAR} = \frac{1.5(739)}{(24)(\cancel{4000}\ 5700)}\left(\frac{405.5'}{45'}\right) = \cancel{0.104} = 0.073''$

$\Delta_{BENDING} = \frac{1.5(739)}{(24)(\cancel{4000}\ 5700)}\left(\frac{405.5'}{45'}\right)^3 = \cancel{8.449''} = 5.929''$

$\Delta_{TOP} = \cancel{8.5531''} = 6.00''$

$\Delta @ TOP < H/400 = (405.5)(12)/400 = 12.165'' > \cancel{8.449''}\ 6.00'' \therefore OK$

LEVELS 56 - CROWN 3

NOTES & ASSUMPTIONS

- LEVEL 56 IS CONSERVATIVELY ANALYZED AS ONLY HAVING 2 WEBS TO SIMPLIFY CALCULATIONS
- CONCRETE CORE WALL ENDS @ 58TH FLOOR, HOWEVER THE FACADE OF THE CROWN IS LOADED W/ WIND & TRANSFERS FORCES TO CONCRETE SHEAR WALLS.

ELEVATION @ TOP OF BUILDING = 1001.5'

DIFFERENCE: 1001.5' - 885' = 116.5

$V_B = (152)(116.5)(50\ PSF) = 885.4$

$V = 885.4 / 2 = 442.7^K$

$\Delta_{SHEAR} = \frac{1.5(442.7)}{(18)(\cancel{4000}\ 5700)}\left(\frac{116.5}{44}\right) = \cancel{0.0244''} = 0.01713''$

$\Delta_{BENDING} = \frac{1.5(442.7)}{(18)(\cancel{4000}\ 5700)}\left(\frac{116.5}{44}\right)^3 = \cancel{0.1712''} = 0.1201''$

$\Delta_{TOP} = \cancel{0.3434''} = 0.1372''$

THESIS: TECH 3 | CYNTHIA MILINICHIK | 3/4

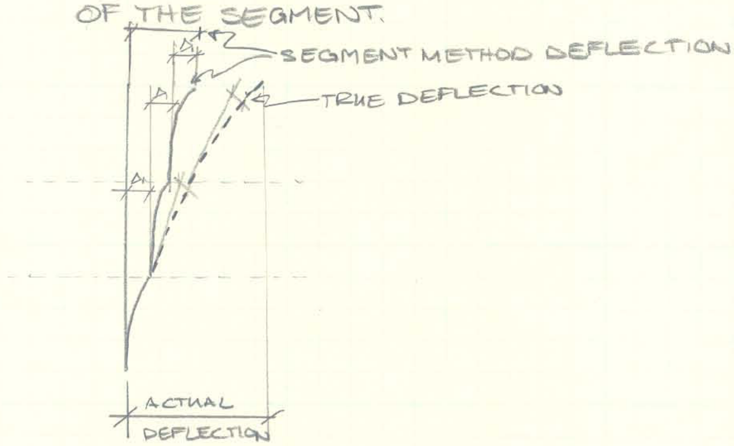
SUM: DEFLECTIONS OF ALL SEGMENTS

$$\Delta_{TOT\text{-}TOP} = 6.71'' + 8.5531'' + 0.3434'' = 15.61'' = L/769$$

$$\Delta_{TOT\text{-}TOP} \leq L/400 = (1001.5)(12)/400 = 30'' > 15.61'' \therefore \text{OK}$$

COMMENTS: WITH A DEFLECTION OF L/769, IT IS POSSIBLE THAT THE CONCRETE CORE WHICH IS MORE THAN ADEQUATE TO RESIST THE WIND LOAD CAN BE REDUCE IN SIZE. THIS IDEA WILL BE EXPLORE IN DEPTH IN A FUTURE REPORT.

ERROR CLAUSE: AFTER REVIEW OF THIS METHOD IT HAS BECOME EVIDENT THAT AN UNCONSERVATIVE ERROR OCCURS WHEN THE DEFLECTIONS ARE SEGMENTED & SUMMED. BY SEGMENTING THE DEFLECTION THE ROTATION AT WHICH THE SECOND SEGMENT'S DEFLECTION STARTS IS NEGLECTED. TO CORRECT THIS THE ANGLE OF ROTATION NEEDS TO BE CALCULATED & ACCOUNTED FOR. TO DO THIS TAKE THE DERIVATIVE OF THE BENDING DEFLECTION WITH RESPECT TO THE HEIGHT OF THE SEGMENT.



$$f = \frac{1.5V}{be}\left(\frac{H}{d}\right)^3 \longrightarrow f' = \frac{3(1.5)V}{be}\left(\frac{H}{d}\right)^2 = \frac{4.5V}{be}\left(\frac{H}{d}\right)^2 = \theta$$

NOTE: V, b, e, d are CONSTANTS

θ = ROTATION

THESIS: TECH 3 | CYNTHIA MILINICHIK | 4/4

ROTATION @ LEVEL 33-55

$$\theta = \frac{4.5(431)}{(24)(4000)}\left(\frac{479.5'}{45'}\right)^2 = 2.29 \text{ radians}$$

$$\Delta_{ROTATION} = H\tan\theta = (405.5')(12''/')\tan(2.81) =$$

a = Height

$\tan\theta = \frac{o}{a}$

$o = H\tan\theta$

$o = \Delta\text{rotation}$

COMMENT: 2.29 RADIANS IS A SIGNIFICANTLY LARGE ANGLE. AN ERROR IN THE CALCULATION OR A FLAW IN THE DERIVATION OF THIS EQN MUST EXIST, BUT CANNOT BE DETERMINE AT THIS POINT

DERIVATION ERROR FOUND

d term is STILL CUBED

$$f = \frac{1.5V}{be}\left(\frac{H}{d}\right)^3 \longrightarrow f' = \frac{3(1.5)V}{be}\left(\frac{H^2}{d^3}\right) = \frac{4.5V}{be}\left(\frac{H^2}{d^3}\right) = \theta$$

MODULUS OF ELASTICITY CHANGE

- ORIGINAL ASSUMPTION OF $E_c = 4000$ INCORRECT

$f'_c = 10\text{ksi}$ SHEAR WALLS

$$E_c = 57\sqrt{10{,}000} = 5700 \text{ ksi}$$

ROTATION @ LEVEL 33-55

$$\theta = \frac{4.5(431)}{(24)(5700)}\left(\frac{479.5^2}{45^3}\right) = 0.0295 \text{ rad}$$

$$\Delta_{ROTATION} = (479.5)(12)\tan(0.0295) = 2.96$$

ROTATION @ LEVEL 56 - CROWN

$$\theta = \frac{4.5(139)}{(24)(5700)}\left(\frac{405.5^2}{45^3}\right) = 0.043 \text{ rad}$$

$$\Delta_{ROTATION} = (405.5)(12)\tan(0.043) = 3.65''$$

TOTAL DEFLECTION w/ ROTATION CORRECTION

$$\Delta_{TOT} = 4.758'' + 6'' + 2.96'' + 0.1372'' + 3.65'' = 17.5'' = L/686$$

$$\Delta_{TOT} \le L/400 = (1001.5)(12)/400 = 30'' > \boxed{17.5'' \therefore \underline{OK}}$$

NOTE: $\Delta \le L/400$ FOR THIS TALL OF A STRUCTURE IS NOT ADEQUATE BECAUSE 30" OF MOVEMENT AT THE TOP OF THE STRUCTURE IS PERCEIVABLE BY THE BUILDING OCCUPANTS

$\Delta \le L/600$ WOULD BE MORE ACCEPTIBLE

THESIS: TECH3 | CYNTHIA MILINICHIK

STORY DRIFT

IN THE SECTION CONTAINING 6 SHEAR WALL WEBS. (LEVELS 1–32)

LOAD ON ONE STORY

$V_B = (192')(15')(30PSF) = 86.4^K$

15' STORY HEIGHT

$\Delta_{SHEAR} = \frac{1.5V}{be}\left(\frac{H}{d}\right) = \frac{1.5(17.0)}{(24)(5700)}\left(\frac{15'}{48'}\right) = 0.0000583"$
5.83×10^{-5} IN

$\Delta_{BENDING} = \frac{1.5(17.0)}{(24)(5700)}\left(\frac{15}{48}\right)^3 = 0.00000569"$
5.69×10^{-6} IN.

$V = 86.4/(4+2(1.3)) =$ 13.1^K ON 1.5' WEB
17.0^K ON 2' WEB

$\Delta_{TOT} = 6.394E-6$ IN

APPENDIX B: SHEAR STRENGTH CALCS

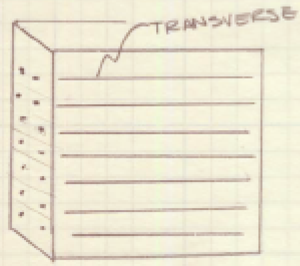
THESIS: TECH 3 | CYNTHIA MILINICHIK

USING: ACI 318-05 : 21.7.4.1

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

$\frac{h_w}{l_w} = \frac{514'}{48'} = 10.7 > 2 \therefore \alpha_c = 2$

$\rho_t = \frac{A_s}{A_{conc}} =$



FOR WEB ELEMENTS

W2: 2' THK WALL – HOR. 6@12" $A_s(\#6) = 0.44\,in^2 \rightarrow 2: 0.88\,in^2$

W1: 1' THK WALL – HOR. 5@12" $A_s(\#5) = 0.31\,in^2 \rightarrow 2: 0.61\,in^2$

W2: $\rho_t = \frac{0.88\,in^2}{(24")(12")} = 0.003056$ $A_{cv} = (48')(12)(24) = 13824\,in^2$

W1: $\rho_t = \frac{0.61\,in^2}{(18")(12")} = 0.002824$ $A_{cv} = (48')(12"/')(18) = 10368\,in^2$

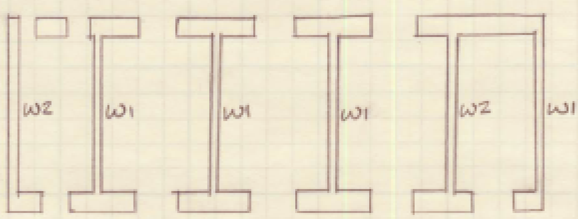
W2: $V_n = (13824\,in^2)(2(\sqrt{10{,}000}) + (0.003056)(60{,}000)) = 5{,}299{,}600^{\#} \rightarrow 5{,}300^k$

W1: $V_n = (10368\,in^2)(2\sqrt{10{,}000} + (0.002824)(60{,}000)) = 3{,}830{,}000^{\#} \rightarrow 3830^k$

$\phi = 0.85$ FOR WIND

W2: $\phi V_n = 0.85(5{,}300^k) = 4505^k$

W1: $\phi V_n = 0.85(3830^k) = 3255^k$



$\phi V_{n\,TOTAL} = 2(4505^k) + 4(3255^k) = 22{,}034^k$

$V_u = 6747^k$

$\phi V_n > V_u \therefore$ OK

COMMENT: THE SHEAR STRENGTH IS MUCH GREATER THAN THE SHEAR FORCE ON THE STRUCTURE. THIS SUGGEST THAT THE CORE MAY BE REDUCED IN SIZE.

APPENDIX C: WIND DESIGN CALCS

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.

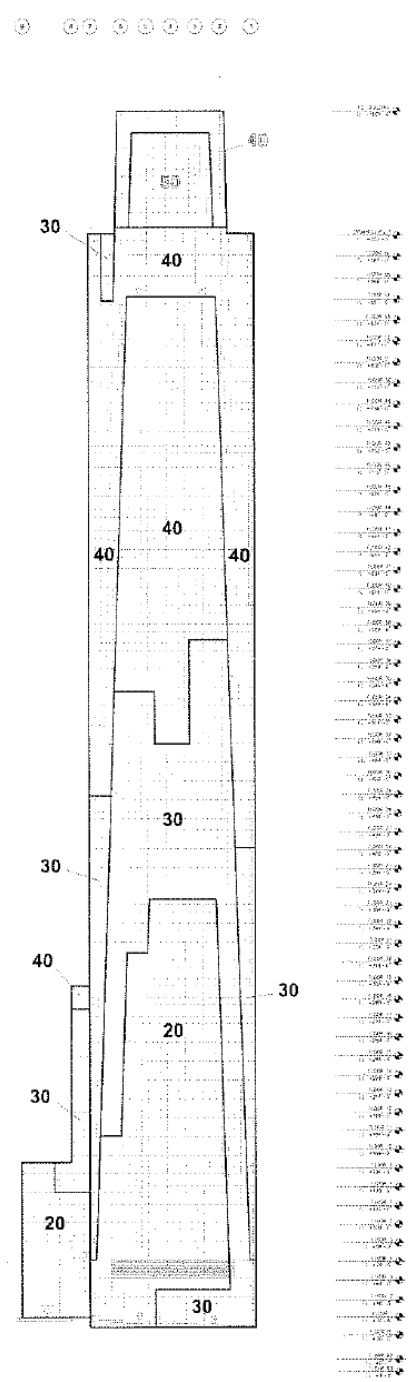
TABLE 2a LOADS AND RESPONSES FOR ONE PENNSYLVANIA PLAZA FOR A 10-YEAR RETURN PERIOD

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.

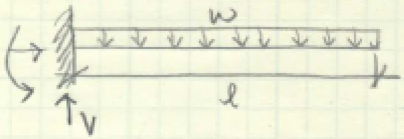
40 Leader indicates zoned surface is perpendicular to drawing plane.

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

THESIS: WIND | CYNTHIA MILINICHIK

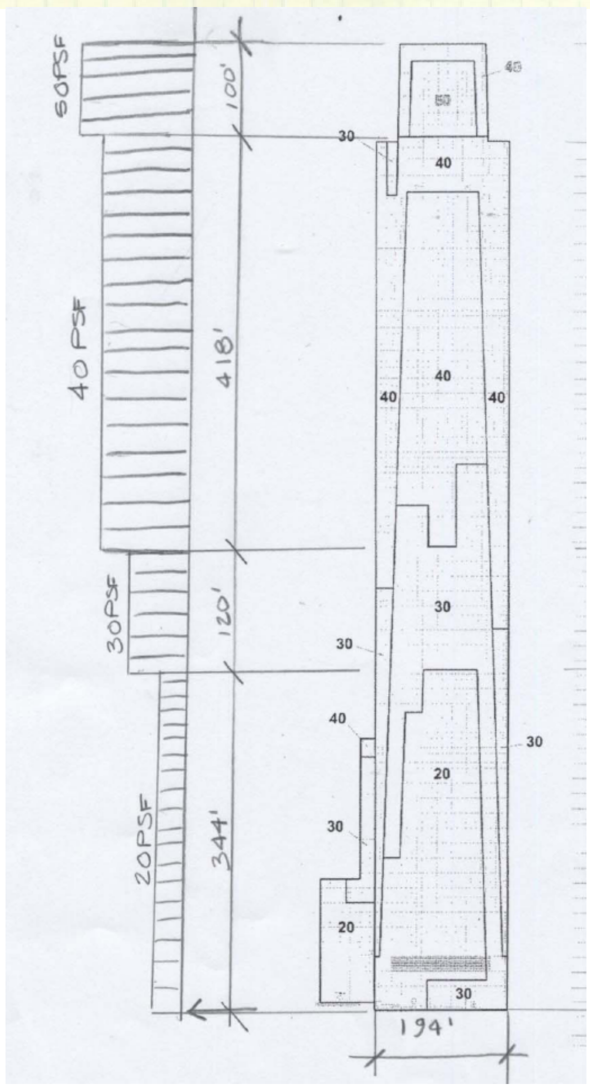
ROUGH ESTIMATE OF BASE SHEAR CAUSED BY WIND

ASSUMPTIONS
- BUILDING IS A CANTILEVER
- SAME WIDTH USED FOR ENTIRE TOWER
- APPROXIMATIONS MADE AT CERTAIN HEIGHTS



$V = 194'[(20)(344) + (30)(120) + (40)(418) + (50)(100)]$

$V = 6{,}246{,}800^{\#} = 6{,}247^{k}$ BASE SHEAR



APPENDIX D: SEISMIC DESIGN CALCS

SEISMIC DESIGN:
-BOCA NATIONAL BUILDING CODE, 1996

EFFECTIVE PEAK VELOCITY-RELATE ACCELERATION - AV
$A_v = 0.1$

EFFECTIVE PEAK ACCELERATION, Aa
$A_a = 0.05$

SEISMIC HAZARD EXPOSURE GROUP
GROUP II - SUBSTANTIAL PUBLIC HAZARD

NATURE OF OCCUPANCY
GROUP E - >250 OCCUPANT LOAD

SEISMIC PERFORMANCE CATEGORY: C

SITE COEFFICIENT, S = 1.0

RESPONSE MODIFICATION FACTOR, $R = 4\frac{1}{2}$

DEFLECTION APPLICATION FACTOR, $C_d = 4$

HEIGHT-NOT-LIMITED

NO PLAN IRREGULARITIES
NO VERTICAL IRREGULARITIES
∴ USE 1610.4

EQUIVALENT LATERAL FORCE PROCEDURE
• BUILDING ASSUMED TO BE FIXED @ THE BASE

SEISMIC BASE SHEAR, V

$V = C_s W = (0.00621)(240,319) =$

$C_s = \frac{1.2 A_v S}{R T^{2/3}} = \frac{1.2(0.1)(1.0)}{(4.5)(\cancel{8.8983}\ 5.93)^{2/3}} = \cancel{0.00621}\ 0.0081$

$T = < T_a C_a = 1.7(\cancel{5.234}\ 3.49) = \cancel{8.898}\ 5.93\text{s}$

NOTE: MY PLANS INDICATE A T = 7.39s

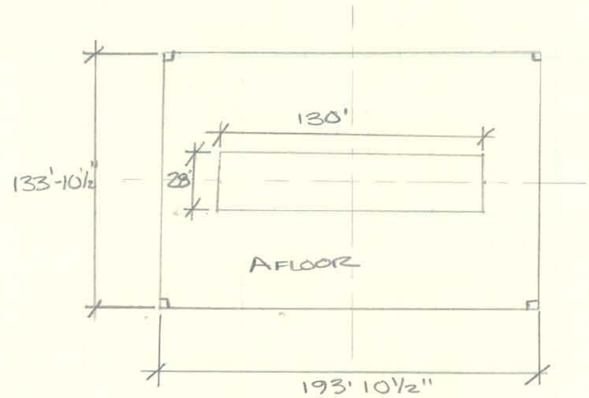
$C_a = 1.7$ UPPER LIMIT

$T_a = C_T h_n^{3/4} = (\cancel{0.03}\ 0.02)(975)^{3/4} = \cancel{5.234}\ 3.49$

$h_n = 975'$

$C_T = \cancel{0.03}\ 0.02$

W= WEIGHT OF BUILDING



TYPICAL LOWER LEVEL FLOOR PLAN

$A_{FLOOR} = [(133.875)(193.875) - (28')(130')] = 18{,}970$ SF

NOTE: SPREADSHEET USED FOR ACCURACY TO DETERMINE FLOOR WT BASED ON ARCHITECTS PLANS - DESIGNATE SQUARE FOOTAGES,

$k = 2$ BECAUSE $T = 8.898s > 2.5s$

$W = 240{,}319.1$ KIPS

$V = C_sW = (\cancel{0.0062T}\ 0.00814)(240{,}319) = \cancel{1492^k}\ \ 1956^k$

			15	45	150	15	45	20	150	100	60	50	65	150
	Level	Total W_{floor} (lbs)	$W_{dead\ Office}$ (lbs)	$W_{dead\ Mechani}$ (lbs)	$W_{live\ Mechanic}$ (lbs)	$W_{dead\ storage}$ (lbs)	$W_{dead\ lobby}$ (lbs)	$W_{office\ partitions}$ (lbs)	$W_{live\ Kitchen}$ (lbs)	$W_{wintergarden}$ (lbs)	$W_{deadfloor}$ (lbs)	$W_{liveparking}$ (lbs)	$W_{deadparking}$ (lbs)	$W_{conc\ core\ slabs}$ (lbs)
Crown 3	59	180000	0	0	0	0	0	0	0	0	180000	0	0	0
Crown 2	58	788250	0	0	0	0	0	0	0	0	180000	0	0	608250
Crown 1	57	1373250	0	135000	450000	0	0	0	0	0	180000	0	0	608250
Office	56	5958390	0	827010	2756700	0	0	0	0	0	1102680	0	0	1272000
Office	55	2952930	265410	0	0	0	0	353880	0	0	1061640	0	0	1272000
Office	54	2885480	254760	0	0	0	0	339680	0	0	1019040	0	0	1272000
Office	53	3336540	325980	0	0	0	0	434640	0	0	1303920	0	0	1272000
Office	52	3336540	325980	0	0	0	0	434640	0	0	1303920	0	0	1272000
Office	51	3336540	325980	0	0	0	0	434640	0	0	1303920	0	0	1272000
Office	50	3336540	325980	0	0	0	0	434640	0	0	1303920	0	0	1272000
Office	49	3336540	325980	0	0	0	0	434640	0	0	1303920	0	0	1272000
Office	48	3336540	325980	0	0	0	0	434640	0	0	1303920	0	0	1272000
Office	47	3336540	325980	0	0	0	0	434640	0	0	1303920	0	0	1272000
Office	46	3336540	325980	0	0	0	0	434640	0	0	1303920	0	0	1272000
Office	45	3336540	325980	0	0	0	0	434640	0	0	1303920	0	0	1272000
Office	44	3371880	331560	0	0	0	0	442080	0	0	1326240	0	0	1272000
Office	43	3371880	331560	0	0	0	0	442080	0	0	1326240	0	0	1272000
Office	42	3371880	331560	0	0	0	0	442080	0	0	1326240	0	0	1272000
Office	41	3371880	331560	0	0	0	0	442080	0	0	1326240	0	0	1272000
Office	40	3551880	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	39	3551880	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	38	3551880	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	37	3551880	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	36	3551880	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	35	3551880	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	34	3551880	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	33	3551880	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	32	3551880	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	31	3551880	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	30	3530600	328200	0	0	0	0	437600	0	0	1312800	0	0	1452000
Office	29	3530600	328200	0	0	0	0	437600	0	0	1312800	0	0	1452000
Office	28	3530600	328200	0	0	0	0	437600	0	0	1312800	0	0	1452000
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Office	26	3530600	328200	0	0	0	0	437600	0	0	1312800	0	0	1452000
Office	25	3530600	328200	0	0	0	0	437600	0	0	1312800	0	0	1452000
Office	24	3530600	328200	0	0	0	0	437600	0	0	1312800	0	0	1452000
Office	23	3530600	328200	0	0	0	0	437600	0	0	1312800	0	0	1452000
Office	22	3530600	328200	0	0	0	0	437600	0	0	1312800	0	0	1452000
Office	21	3530600	328200	0	0	0	0	437600	0	0	1312800	0	0	1452000
Office	20	4070600	328200	0	0	0	0	437600	0	0	1312800	0	0	1992000
Office	19	4070600	328200	0	0	0	0	437600	0	0	1312800	0	0	1992000
Office	18	4070600	328200	0	0	0	0	437600	0	0	1312800	0	0	1992000
Office	17	4243120	355440	0	0	0	0	473920	0	0	1421760	0	0	1992000
Office	16	4243120	355440	0	0	0	0	473920	0	0	1421760	0	0	1992000
Office	15	4243120	355440	0	0	0	0	473920	0	0	1421760	0	0	1992000
Office	14	4243120	355440	0	0	0	0	473920	0	0	1421760	0	0	1992000
Office	13	4243120	355440	0	0	0	0	473920	0	0	1421760	0	0	1992000
Office	12	4243120	355440	0	0	0	0	473920	0	0	1421760	0	0	1992000
Office	11	4243120	355440	0	0	0	0	473920	0	0	1421760	0	0	1992000
Office	10	4243120	355440	0	0	0	0	473920	0	0	1421760	0	0	1992000
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Office	7	4246730	356010	0	0	0	0	474680	0	0	1424040	0	0	1992000
Office	6	4246730	356010	0	0	0	0	474680	0	0	1424040	0	0	1992000
Office	5	4246730	356010	0	0	0	0	474680	0	0	1424040	0	0	1992000
Office	4	4246730	356010	0	0	0	0	474680	0	0	1424040	0	0	1992000
Office	3	4168785	284355	66330	221100	0	0	379140	0	0	1225860	0	0	1992000
Office	2	4193205	47520	291960	973200	9165	0	63360	0	0	726000	0	0	2082000
Office	1	6680735	0	0	0	0	0	0	0	1378900	1999680	634850	825305	1842000
Parking	B1	9799155	0	244575	815250	0	527850	0	2906700	0	2740740	274800	357240	1932000
Parking	B1.5	4012335	0	207765	692550	16815	0	0	0	0	811380	387750	504075	1392000
Parking	B2	5168260	0	0	0	4965	0	0	0	0	1345140	1054850	1371305	1392000
Parking	B3	6183588.75	0	339300	1131000	11389	0	0	0	0	1551780	764400	993720	1392000