STRUCTURAL TECHNICAL REPORT 1
STRUCTURAL CONCEPTS & EXISTING CONDITIONS

EXECUTIVE SUMMARY

A detailed description and a preliminary analysis of the existing structural system of the 57 story Comcast Center located in Philadelphia, PA is presented in this report. The building is used primarily as office space with some restaurant and retail spaces. Three floors of parking are located below grade. A blast-resistant concrete core supports the steel framing of the shell of the building. Composite metal deck floors are utilized to minimize the depth of each floor system. Gravity loads are transferred through caissons to solid rock with a bearing capacity of 20 tons per square foot.

The walls of the concrete core function as shear walls in the lateral force resisting system. A wind tunnel test was conducted to determine the wind loading. Wind load controls the design of the lateral force resisting system with a base shear of 6,247 kips. A vierendeel truss is used to transfer the gravity loads in certain areas where columns are not continuous to the foundation.

The latest edition of the City of Philadelphia Building Code and the 1996 Boca Building code were used to design the Comcast Center. A preliminary analysis was done using the same codes to verify the existing design. Spot checks were performed for a typical steel beam with composite metal deck slab, a steel girder, a steel column, a concrete shear wall, and a steel braced frame were calculated for this report.

INTRODUCTION

Formerly named One Pennsylvania Plaza, the Comcast Center was renamed after Comcast Corporation signed a 15.5 year lease for 24 of the 57 floors. Comcast will occupy 534,000 SF of the 1.2 million SF of office space making up 44% of the building. The Comcast Center has a grand total of 1.6 million SF. With a 90,000 SF footprint the Comcast Center takes up most of the city block sectioned off by JFK Boulevard, Arch Street, and 17th Street.

Construction on the Comcast Center began in mid January of 2005 and is expected to be completed by Fall of 2007. The construction of the Comcast Center has been divided into two phases; the 57 story tower will be constructed in the first phase and a 16 story office building in the second. Upon completion the Comcast Center will surpass the recently completed Cira Center as the tallest building in Philadelphia and take its place in the growing skyline as Philadelphia begins a period of urban renewal.

When planning the new tallest building between New York City and Chicago concerns arose that the Comcast Center would become a target for terrorists. In response to these concerns the Comcast Center has a blast resistant concrete core with steel framing and composite metal deck floors. Three below grade levels offer 120 private parking spaces. Bollards are used on parking levels around all columns to prevent any vehicular damage to the structural system.

Built above the 17th Street railway station, the Comcast Center will provide convenient access to the suburban station for commuters. The exterior landscape of Pennsylvania Plaza will serve as a public square for dining and outdoor meetings.

Originally the architects of Robert A. M. Stern had designed the Comcast Center to compliment the Philadelphia Museum of Art but the architecture has since changed to resemble a European style tower. The Comcast Center will be clad with Low-E high performance glass curtain wall. Occupants of the Comcast Center will be greeted with a 110 foot tall winter garden. Crowning the Comcast is a glass clad steel framed box.

At a cost of $435 million, Liberty Property Trust, the owner, is taking on the largest private and commercial development project in the state. Governor Ed Rendell contributed $30 million in aid to help the project take off. The Comcast Center is spending more than $40 Million in public improvements.

With dimmers, occupancy sensors, timers, and other energy efficient electronic products, the Comcast Center will be LEED rated. Another energy conscious concept utilized by the Comcast Center is the large scale use of daylighting. Floor heights of 15-17 feet allow for larger windows which permit greater amounts of natural light into the space, decreasing the need for artificial light.

CODES AND CODE REQUIREMENTS

The structural system of the Comcast Center complies with the City of Philadelphia Building Code, latest edition and the Boca Building Code, 1996 (BOCA 96). When designing the model of the Comcast Center for wind tunnel testing both the ASCE 7 requirements and the BOCA 96 requirements were met. ACI was used for concrete design and the National Electric Code was used for electrical design.

The same building codes were used for this technical assignment in order to limit the variance in design values and thereby determine the accuracy of my assumptions and calculations.

GRAVITY AND LATERAL LOADS

GRAVITY LOADS

Live Loads:	PSF	Comments
Office Live	50	
w/ Partitions	20	
Elevator Lobby & Corridors above first floor	80	
w/ Partitions	20	
Corridors above first floor	75	
All other Lobbies and Corridors	100	
Exit Facilities	100	
Retail Areas	100	
Kitchen	150	
Cafeteria	100	
Winter Garden and Atrium	100	
Light Storage Area	125	
Loading Dock	250	or AASTO HS20-44
Mechanical Floors	150	or actual weight, whichever is greater
Mechanical / Fan Rooms	75	or actual weight, whichever is greater
Sidewalks	250	
Parking Ramp Live	50	

Dead Loads:	PSF
Office Superimposed Dead	15
Lobby Superimposed Dead	45
Mechanical Superimposed Dead	45
Storage Superimposed Dead	15
Parking Ramp Superimposed Dead	20

WIND LOADS

For economy a wind tunnel test was performed by Alan G. Davenport Wind Engineering Group on a model of the Comcast Center. Several wind tests were done during the design process as the design changed geometry and materials. The results of the most recent wind tunnel test were obtained from Thornton Tomasetti for this report. Please refer to the Summary of Main Findings in Appendix C for the full summary.

The base shear was calculated by simplifying a pressure diagram from the wind tunnel testing reports. The building was assumed to act as a cantilever and the shear was summed at the base of the tower. With a base shear of 6,247 kips, wind loading controls the lateral load resistance system.

Some of the main findings from the wind tunnel test pertain to the wind climate, overall building response, local differential pressures, and the pedestrian wind environment. For strong winds, westerly directions are the most important. A 97mph extreme mean hourly wind speed was determined for a 50 year return period.

Various values of total damping ratios were used to calculate predicted accelerations and base moments for 10 year and 50 year return periods. A 10 year return yields a building acceleration of 38.4 milli-g with a 1.5% of critical damping ratio. With structural damping values of 3.0% and 4.0% the accelerations reduce to 27.1 milli-g and 23.5 milli-g repectively. These structural damping values may be obtained by adding an auxiliary damper system. The largest predicted base bending moments of 3,500,000,000 lb-ft and 3,250,000,000 lb-ft for 2.0% and 2.5% and a 50 year return occur in the Y-direction. Below is the Loads and Responses for the Comcast Center for a 10-Year Return Period table. For notes and more information refer to the summary in Appendix C.

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

VARIABLE	Damping Ratio				
	$\xi = 1.5\%$	$\xi = 2.0\%$	$\xi = 2.5\%$	$\xi = 3.0\%$	$\xi = 4.0\%$
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

A nominally-sealed building condition was assumed to calculate the internal pressure coefficients. Internal pressures will increase when operable windows are opened. In areas where a double wall system is present a study was performed to obtain more accurate differential pressures and suctions across the inner and outer wall.

A differential pressure of 46.0 psf and a suction of 84.6 psf were obtained from a study evaluating the exposure of cladding elements to internal pressure. The largest differential pressure occurs on the south elevation at the 49th level and the largest differential suction occurs on the north elevation at the 54th story.

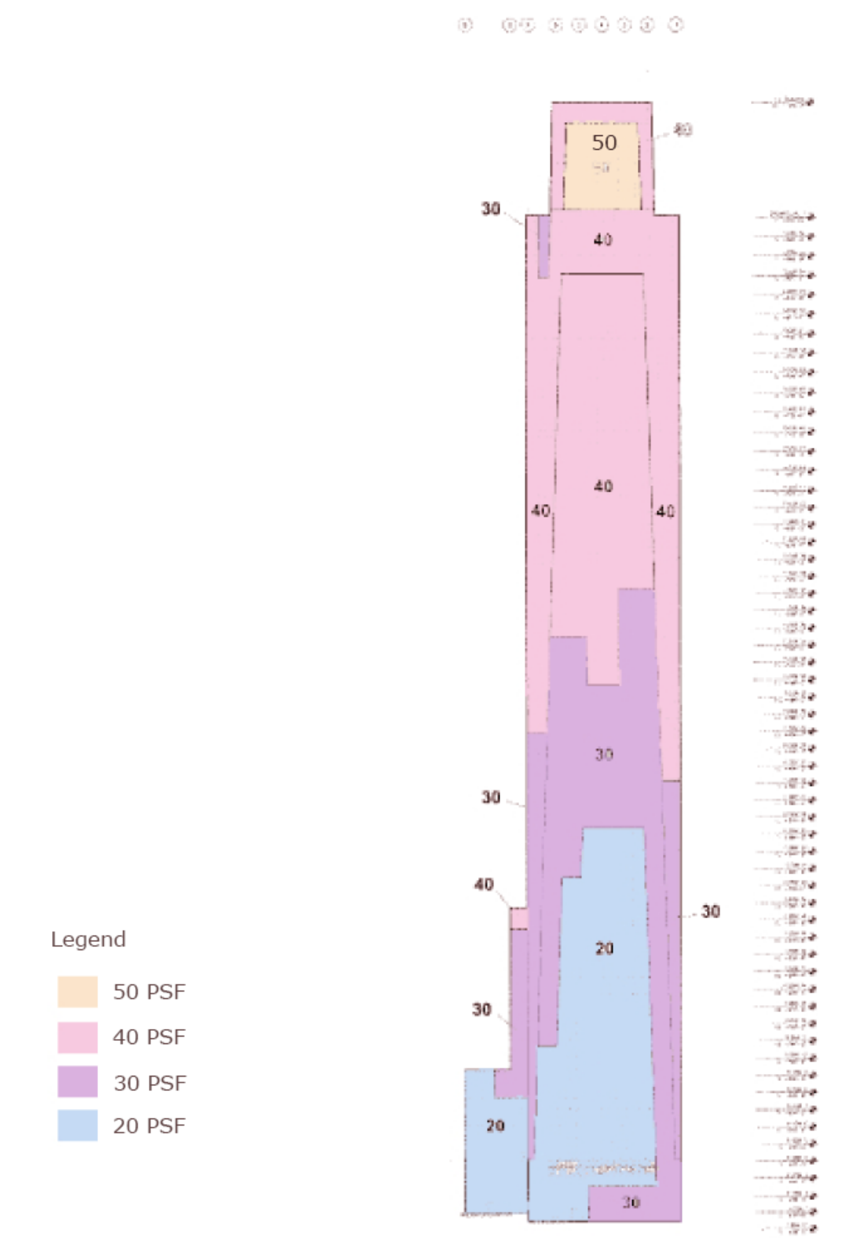


DIAGRAM OF WIND PRESSURES ON BUILDING

All of the wind load data in this report is from the wind tunnel testing report. I plan to compare the difference between the analytic method results and the wind tunnel testing results in a future report.

Seismic Loads

Seismic loads were calculated using BOCA 96. The base shear calculated is 1,492 kips. Below is a chart with weight, height, vertical distribution factor, and the lateral force for each floor.

The 2.25 kip force at Crown level 3 may appear too small however Crown level 3 is just a steel framed box used to create the "crown" aesthetic and is therefore very light compared to the rest of the structure which has concrete floors and a massive concrete core.

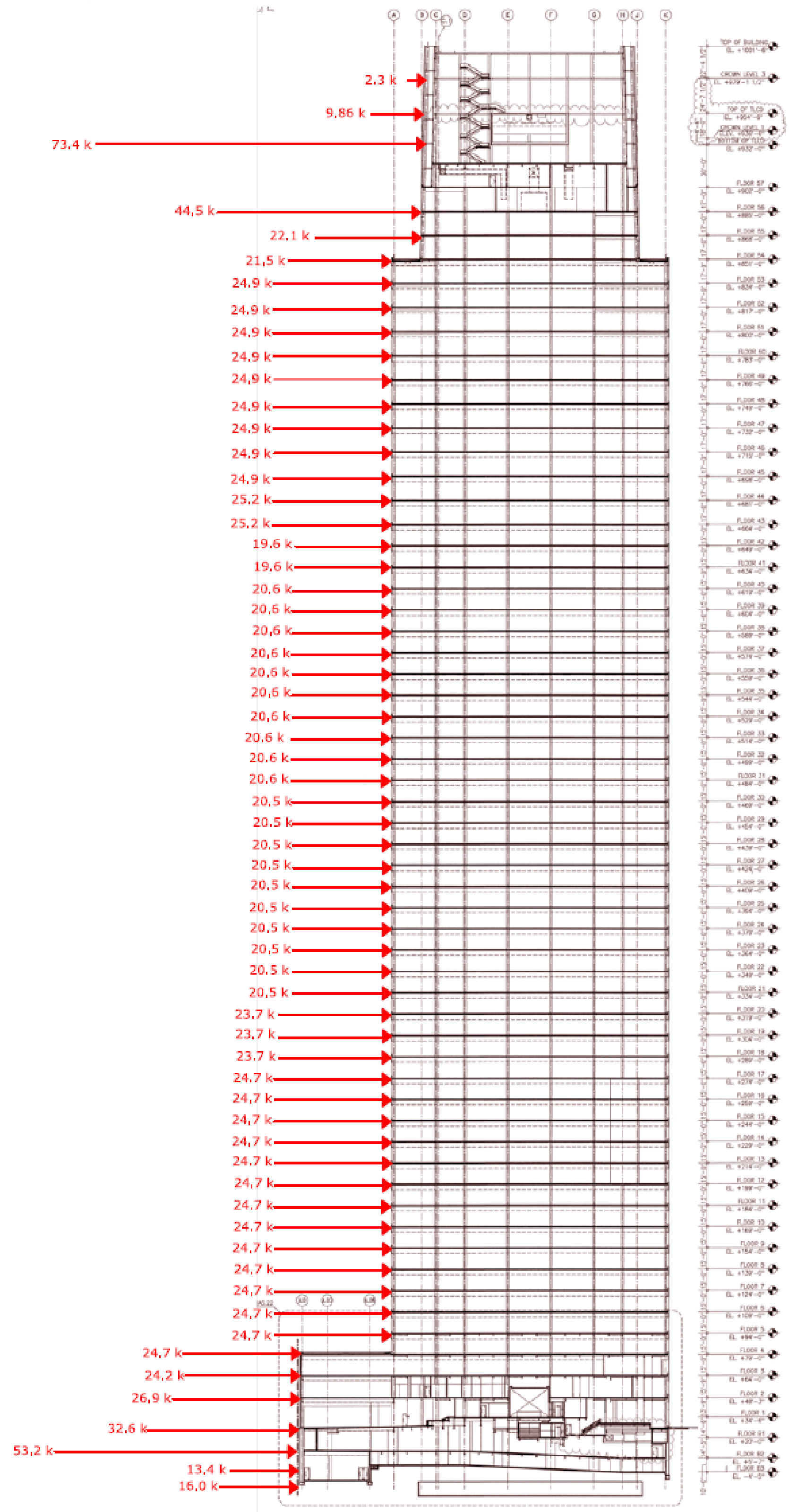
Seismic Loads

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	

An excel spreadsheet was used to sum the floor weight based on the area allotted to specific functions as noted in the architectural plans. Please see appendix D for the spreadsheet and other seismic load calculations.

SEISMIC LOADS ELEVATION



DESCRIPTION OF THE STRUCTURAL SYSTEM

FOUNDATION

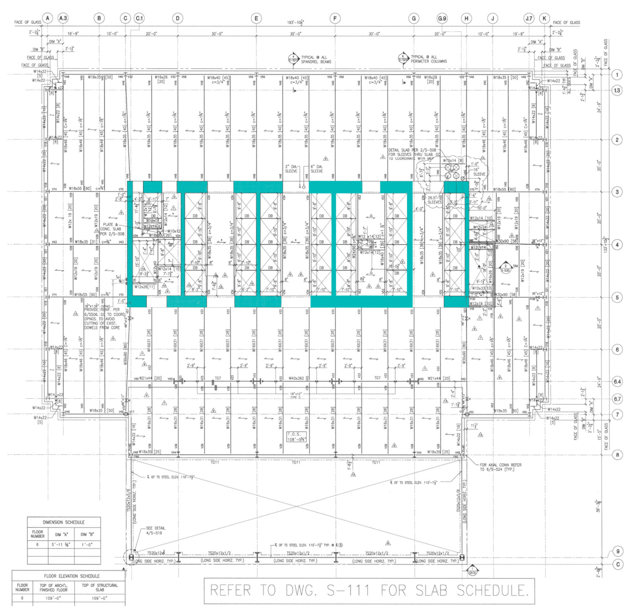
The foundation has a 20 ton per square foot allowable bearing capacity. Caissons are socketed a minimum of 6 feet into solid rock and range in diameter from 3 to 8 feet. Soil retaining walls are protected from deterioration with a waterproof membrane and drainage pad.

CONCRETE CORE

The concrete core serves several functions in the Comcast Building. The core houses the buildings many elevators as well as some heating, cooling, and ventilation ducts. With continuous concrete walls from the base to the 58th floor of the tower, the core provides lateral stability through structural shear walls. The walls range in thickness from 4'-6" to 1'-6" and decrease in thickness as the elevation increases. The penetrations in the concrete core create coupling beams. The coupling beams help to transfer the shear forces across the openings in the wall. The concrete core supports the steel framing that makes up the rest of the building.

Photograph Courtesy of R. Bradley Maule

CONCRETE CORE CONSTRUCTED FIRST,
STEEL ERECTION FOLLOWS



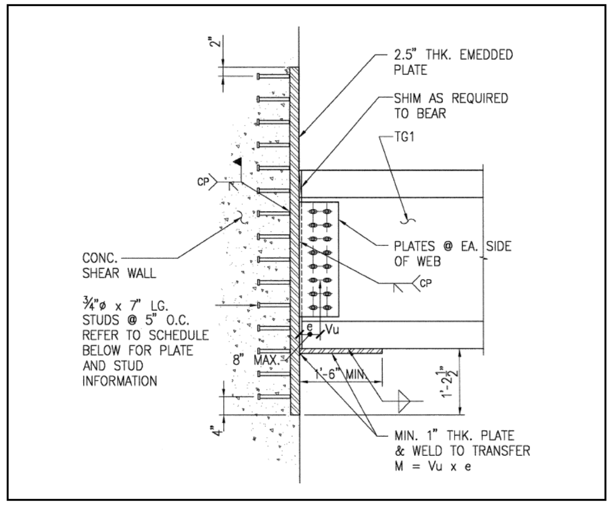
NOTE: CONCRETE CORE IN FLOOR PLAN

STEEL FRAMING

The steel beams are bolted to the steel plates (pictured below) embedded in the concrete core creating a shear connection. Shear studs welded on the embedded plates and embedded in the concrete. Beams spanning 45+ feet have a camber to counter the initial deflection from the dead load of the wet concrete.

Photograph Courtesy of R. Bradley Maule

EMBEDDED STEEL PLATES FOR BEAM TO CONCRETE CORE CONNECTION



DETAIL OF TYPICAL EMBEDDED PLATE

COMPOSITE METAL DECK SLABS

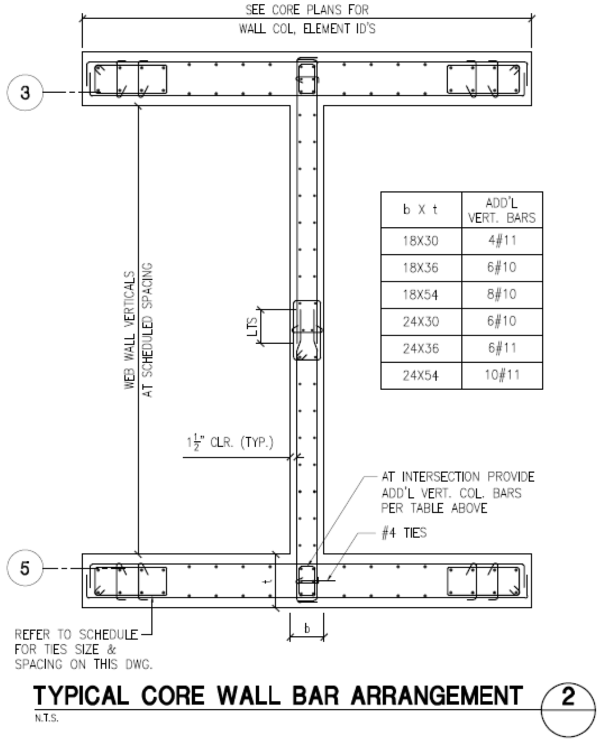
Composite metal deck slabs span 10 feet on average between beams. Most slabs are lightweight concrete on 3" metal deck. The depths of the slabs range from 6" to 11" and the guage of the metal deck ranges from 18 to 20.

Photograph Courtesy of R. Bradley Maule

COMPOSITE SLABS DECREASE FLOOR SYSTEM DEPTH

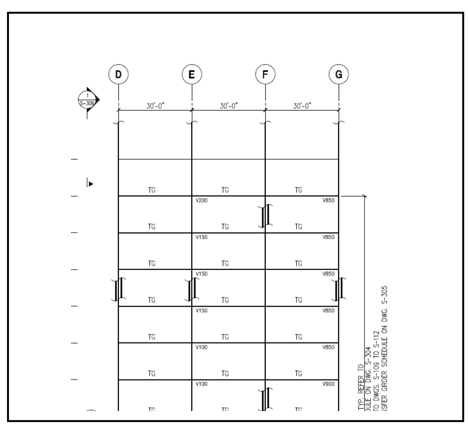
LATERAL LOAD RESISTING SYSTEM

The lateral load resisting system is composed of the concrete shear walls of the concrete core. Wind loads on the glass cladding are transferred to the steel framing which cannot resist lateral loads and therefore transfers the load to the concrete core. At the base of the tower the concrete core walls are 4'-6" thick and gradually step back as the tower rises ending at a thickness of 2'-0".



VIERENDEEL TRUSS

A vierendeel truss is used in several locations to divert column loads from upper levels to other locations in order to leave the space below open and free of columns. One such location is visible from the exterior and is pictured below. The large opening in the façade creates a condition in which the columns above cannot transfer their load directly to the foundation and therefore a vierendeel truss is used to transfer the load to other areas. Unlike a typical truss, a vierendeel truss does not have diagonal members, and therefore relies on moment connections to transfer the loads.

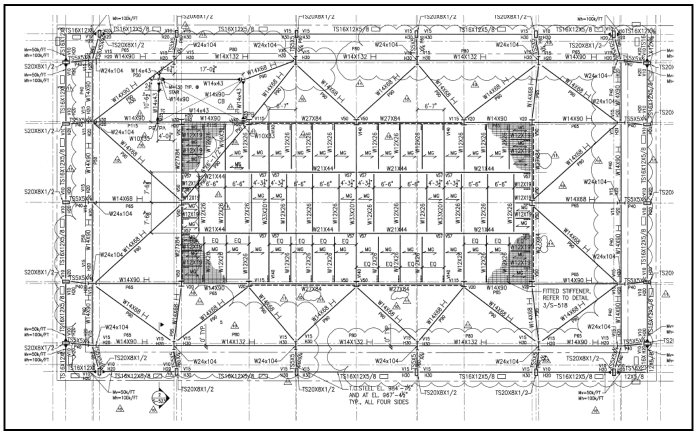


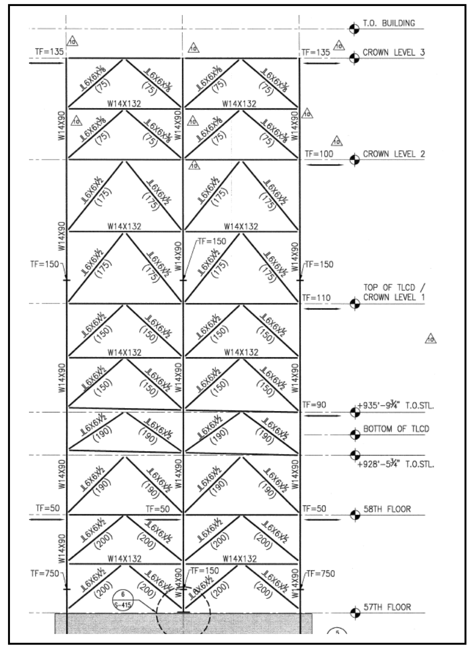
A vierendeel truss is also used in the lower portion of the structure to allow for larger open spaces on the floors below.

BRACED FRAMES IN CROWN

The top of the Comcast Center is crowned with a steel framed box. The steel frame has cross bracing to supports its own lateral loads and to transfer its weight to the concrete core. The cross bracing does not contribute to the lateral resistance of the overall structure.

The cross bracing members are mostly W14x68s and 2L6x6x1/2. This steel framed box allows the Comcast Center to increase in height without a significant increase in weight for the lightly used space.





STRUCTURAL ANALYSIS

Wind load controls the lateral force resisting system design with a base shear of 6,247 kips. Seismic had a base shear of 1,492 kips.

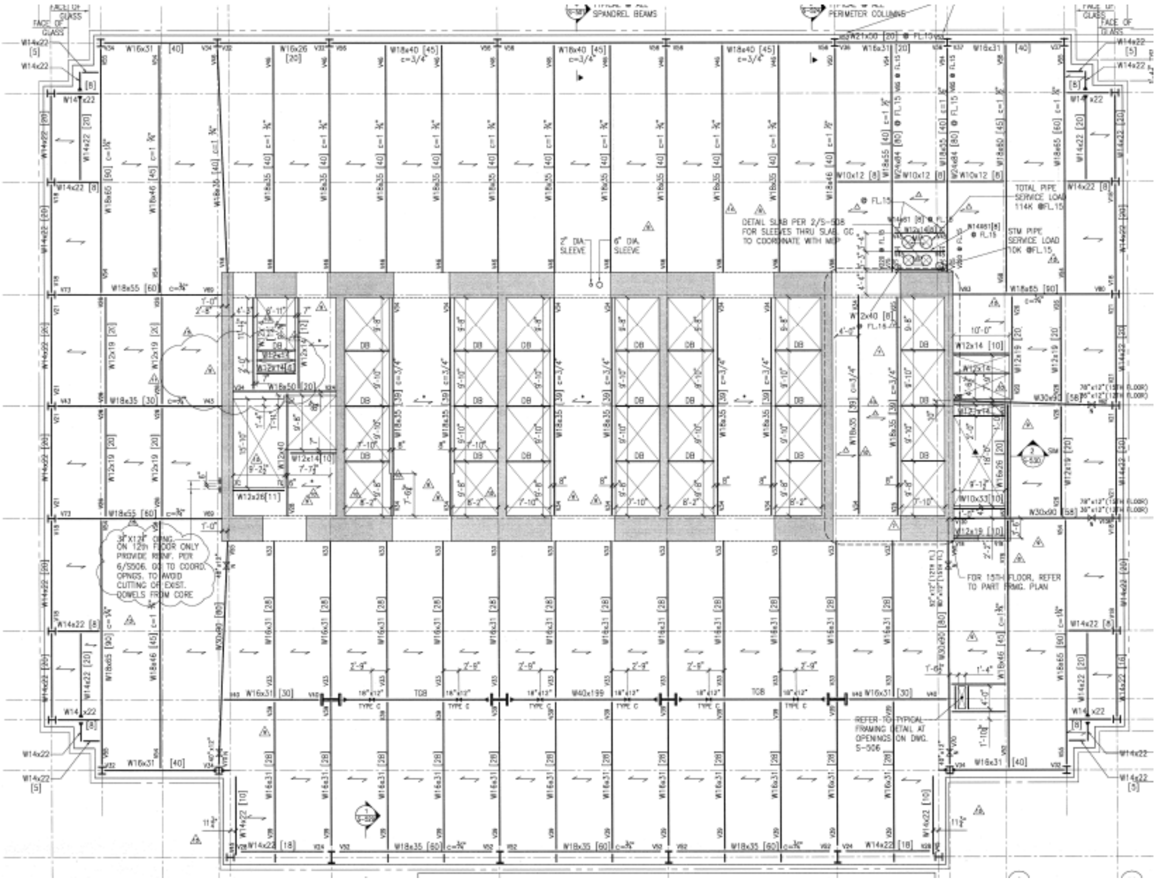
Load Resistance and Factor Design (LRFD) was used to spot check all steel members. Judging by the closeness in the design strength of the members used in the building and the factored loads I calculated I believe that the steel members were originally designed with LRFD. With the scale of this building LRFD would yield significant savings over ASD, so it likely that the engineers chose LRFD for economy reasons.

It was noted that in many cases steel beams frame into steel girders of the same depth. Although this is a standard design condition, using a slightly deeper girder could save money on beam coping. Based on the number of coped beams in a building of this scale such a savings could save a substantial amount. One reason for which the engineers may not have selected a deeper girder may have been to keep the floor system depth at a minimum. All the steel girders are spandrels and a major design concept of the building is the use of a large volume of daylight.

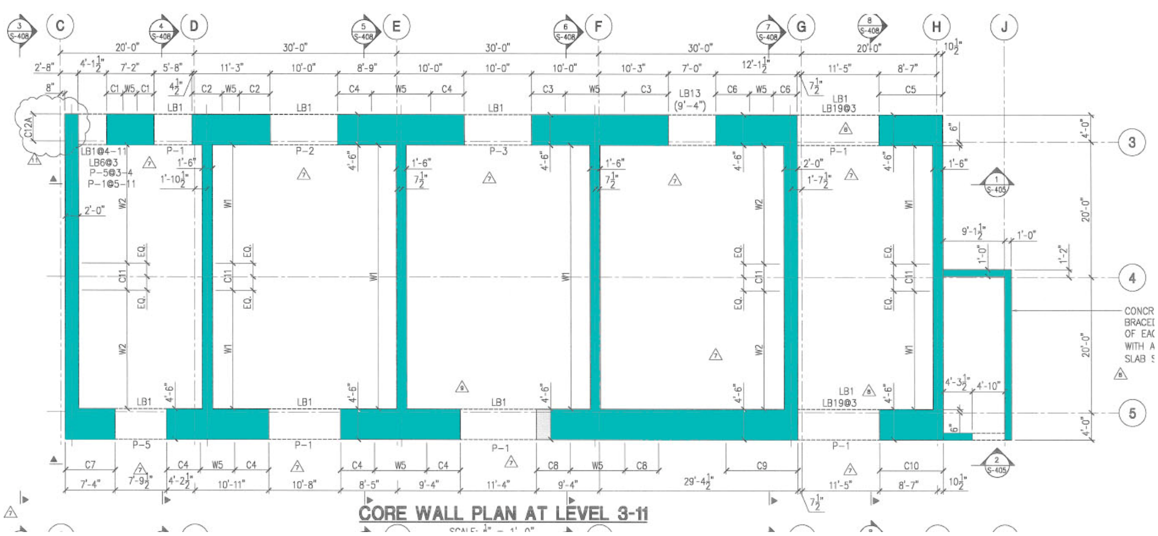
Spot checks were performed to verify typical members in the structure. All members checked were adequate for the given loading conditions. Some spot checks revealed some interesting conditions. For example, the column that was checked only had one level of live load, dead load and snow load on it and only needs to be a W14x43. However the column schedule calls for a W14x605. The steel framing cannot transfer moments and therefore a combined loading condition does not exist. One possibility which needs to be explored is that a beam framing into the column might be a transfer girder, adding much heavier point load in addition to the assumed distributed load. Please see Appendix B at the end of the report for all calculations done on structure.

A spot check of the shear wall was performed and the analysis yielded that the nominal strength of the shear was greater than the factored shear. This indicates that the shear was designed correctly. The simplifying assumption that the shear wall could be analyzed for just one story was made and yielded interesting results. The factored shear was much less than the design strength of the wall and that boundary steel was unnecessary. However in the actual shear walls, boundary steel is used. Since the shear wall was designed to support much greater loads than those that were accounted for in the spot check a great difference in values existed. A more thorough evaluation will be performed in another report.

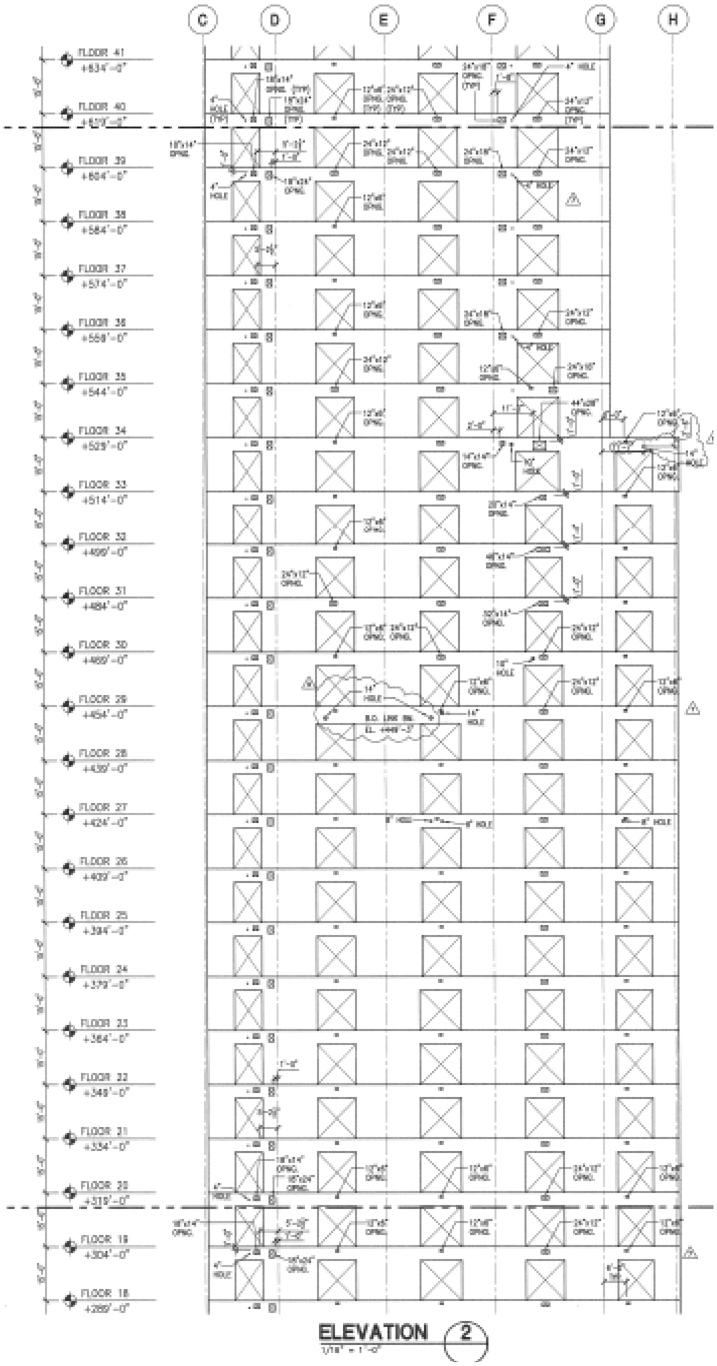
APPENDIX A: TYPICAL FLOOR PLAN & SECTIONS



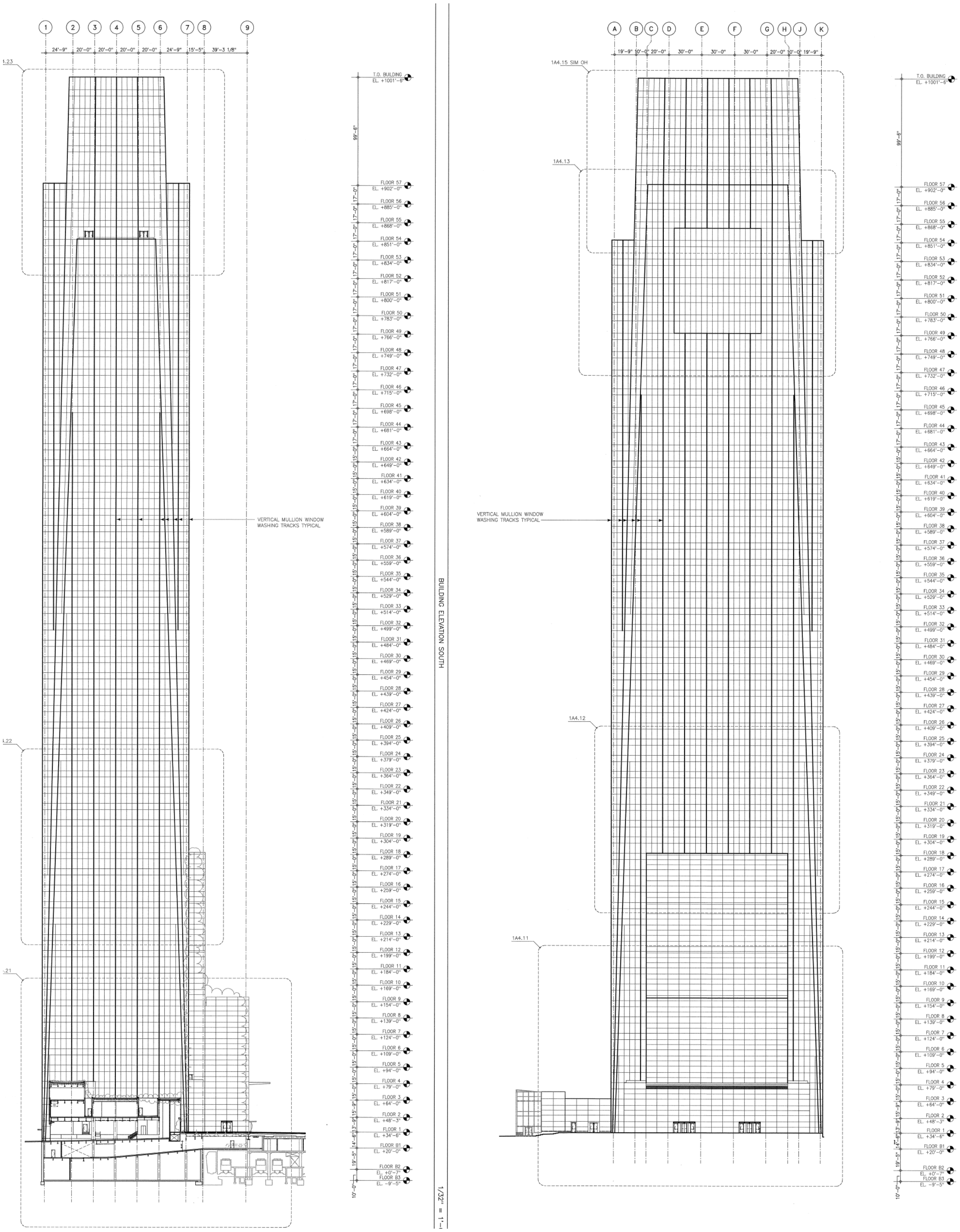
TYPICAL FLOOR PLAN AT LOWER LEVEL



TYPICAL CONCRETE CORE PLAN DETAIL FOR FLOORS 3-11



ELEVATION OF CONCRETE CORE



ELEVATIONS OF THE COMCAST CENTER

APPENDIX B: GRAVITY LOAD CHECKS

THESIS : SNOW LOAD | CYNTHIA MILINICHIK 1/1

BOCA 96

GROUND SNOW LOAD: FIG 1608.3(1) 25PSF

FLAT ROOF

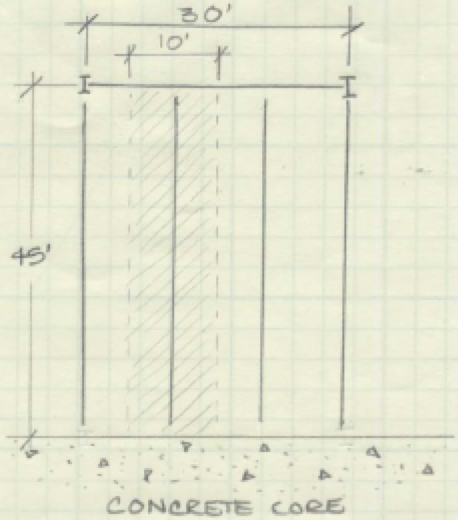
$P_f = C_e I P_g = 0.7(1.0)(25PSF) = \underline{17.5PSF}$

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

$I = 1.0$ - T1609.5

THESIS: GRAVITY LOAD ANALYSIS CYNTHIA 1/

TYPICAL FLOOR FRAMING
- LOWER LEVEL
- NORTH SIDE



OFFICE LOADING

LL: 50 PSF
20 PSF PARTITION
70 PSF

DL: 15 PSF SUPERIMPOSED
55 PSF FLOOR ASSEMBLY
70 PSF

1.2D + 1.6L LOAD CASE GOVS.

$1.2(70) + 1.6(70) = 196\text{ PSF}$

$(196\text{ PSF})(10') = 1.96\text{ KLF}$

$$M = \frac{wl^2}{8} = \frac{(1.96)(45^2)}{8}$$

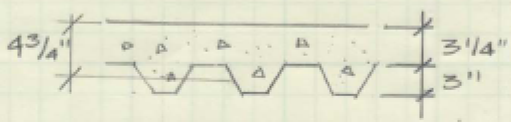
$M = 497^{1k}$

$$V = \frac{wl}{2} = \frac{(2.0)(45)}{2} = 45^k$$

FLOOR ASSEMBLY ESTIMATE

LT WT CONCRETE 115 PCF
REINFORCING 5 PCF
120 PCF

6 1/4 ON 3" METAL DECK



$3\,1/4 + 1\,1/2 = 4\,3/4$

$(120\text{ PCF})(4.75''/12''/') = 47.5\text{ PSF}$

ASSUME: 40 PLF FOR BEAM

40 PLF / 10' TRIB WIDTH = 4 PSF

47.5 + 4 = 51.5 PSF → USE 52 PSF

3 PSF COLLATERAL → 55 PSF

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 $W18\times35 \rightarrow \phi M_n = 505^{'k}$

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

CHECK: ASSUMED Y2

$b_{min} = \begin{vmatrix} 1/4 L \\ s \end{vmatrix} = \begin{matrix} 1/4(45') = 11.25' \\ 10' \leftarrow \text{USE} \end{matrix}$ 120"

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

$Y2 = Y_{CON} - \dfrac{a}{2} = 6.25 - \dfrac{1.26}{2} = 5.62$ 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

$(48\,PSF)(10') = 480\,PLF$
$1.4(480) = 672\,PLF$

$1.2(480) + 1.6(20\,PSF)(10) = 896\,PLF \leftarrow$ GOVERNS

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

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

THESIS: GRAVITY LOAD ANALYSIS | CYNTHIA MILINICHIK 3/3

ASSUME: METAL DECK PROVIDES LATERAL STABILITY FOR BEAMS

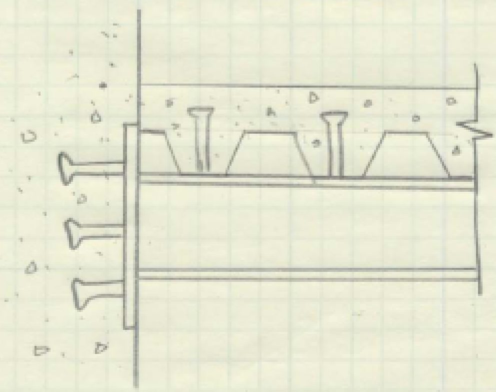
TABLE 5-3

$\phi M_n = 249'^k < 226.8'^k \therefore$ 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 $\quad I = 510 \text{ in}^4$

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

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

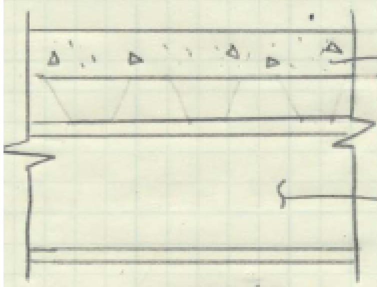
$$\Delta_{comp} = \frac{5wl^4}{384EI} = \frac{5(500)(45^4)(1728)}{384(29{,}000{,}000)} = 1.061''$$

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

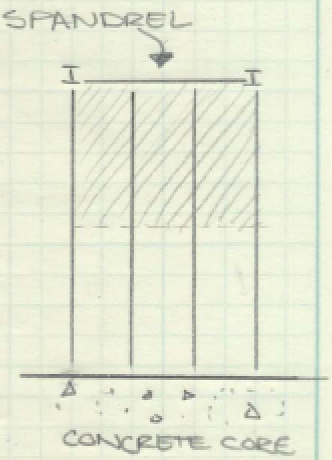
THESIS: GIRDER

CYNTHIA MILINICHIK 1/2

TYPICAL GIRDER



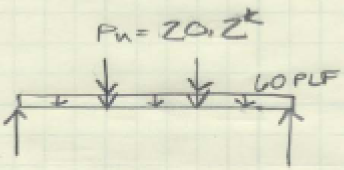
W18x40



SELF WT OF GIRDER: 40 PLF
→ USE 50 PLF → 1.2D = 60 PLF

FACTORED LOADS FROM BEAMS

$P_u = 20.2^k$



SUPERPOSITION

$M_{MAX} = Pa = 20.2(10') = 202^{1k}$

$V = P = 20.2^k$

$M = \frac{wl^2}{8} = \frac{60(30^2)}{8} = 6.76^{1k}$

$V = \frac{wl}{2} = \frac{60(30)}{2} = 900^{\#}$

$\boxed{M_{TOT} = 210^{1k}}$

COMPOSITE DESIGN:

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

ASSUMING $a = 2''$

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

$M_u = 210^k$ TRY W18x40 $\phi M_n =$

$Y1 = 0$ $I_{LB} = 1760$ $\Sigma Q_N = 590^K$

$\frac{2\Sigma Q_N}{Q_N} = \frac{2(590)}{26.1} = 45.2^k \rightarrow 46$ STUDS

CHECK! ASSUMED Y2

$b = \min \begin{cases} 1/4 L = 1/4(30) = 7.5'(12'') = 90'' \\ S = 45' \end{cases}$

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

$(48\,\text{PSF})(22.5') = 1080\ \text{PLF}$
$1.2(1080) + 1.6(20)(22.5) = 2016 \leftarrow$ 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

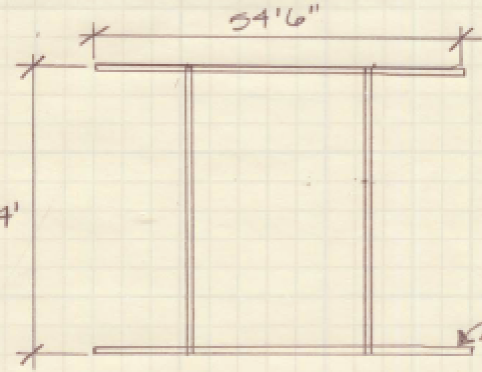
THESIS: SHEAR WALL | CYNTHIA MILINICHIK | 1/3

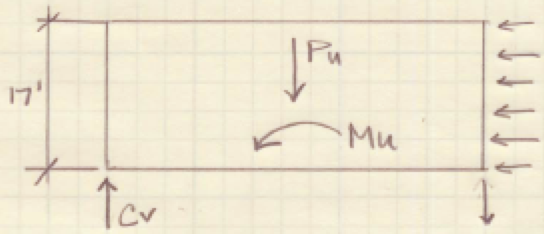
SPOT CHECK: LEVEL 58 CORE WALL

SHEAR WALL: CONCRETE

NOTE: FOR SIMPLIFICATION PURPOSES, SHEAR WALL ONLY ANALYZED FOR ONE FLOOR

NOTE: USING ACI





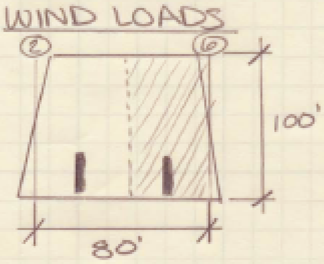
$$C_v = \frac{P_u}{2} + \frac{M_u}{d} = \frac{278^k}{2} + \frac{65^{'k}}{54.5'} = 140.2^k$$

GRAVITY LOADS ON SHEAR WALL
- SINCE THIS WALL IS @ TOP OF STRUCTURE ONLY GRAVITY LOAD IS SELF WT.

150 PCF - REINFORCED CONCRETE

$$(150\text{PCF})(2')(54.5')(17') = 277950^{\#} \rightarrow 278^k = P$$

WIND LOADS



$$(40')(45\text{PSF}) = 1800\text{PLF}$$

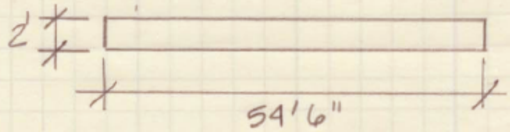
$$M_u = \frac{1800\text{PLF}(17')^2}{8} = 65^{'k}$$

$$V_u = \frac{wl}{2} = \frac{(1800)(17')}{2} = 15.3^k$$

THESIS: SHEAR WALL | CYNTHIA MILINICHIK 2/3

CHECK: NEED FOR BOUNDARY ELEMENTS
$f_c > 0.2 f'_c$

WALL SECTION



$A_g = (2')(54.5') = 109 \text{ SF}$

$I_g = \frac{(2')(54.5')^3}{12} = 27{,}000 \text{ FT}^4$

$f_c = \frac{P_u}{A_g} + \frac{M_u \frac{h_w}{2}}{I_g} = \frac{278}{109} + \frac{\frac{65(54.5)}{2}}{27{,}000} = 2.62 \text{ ksf}$
$= 0.01817 \text{ ksi}$

$0.2 f'_c = 0.2(4 \text{ ksi}) = 0.8 \text{ ksi}$

$f_c = 0.01817 < 0.8$

∴ DO NOT NEED BOUNDARY ELEMENTS

DETERMINE: LONGITUDINAL & TRAVERSE REINFORCEMENT

$V_u \geq 2 A_{cv} \sqrt{f'_c} \rightarrow$ 2 CURTAINS OF REINFORCING

$2((2')(54.5')(144))\sqrt{4000}/1000 = 1985^k$

$V_u = 15.3^k < 1985$ ∴ DO NOT NEED 2 CURTAINS OF REINFORCING

REQ'D ρ_l & ρ_t based on MIN REQ'T:

$\rho_l, \rho_t = \frac{A_{sv}}{A_{cv}} \geq 0.0025$

$A_{cv} = (12")(24") = 288 \text{ IN}^2/\text{ft}$

$A_{sl_{req'd}} = (0.0025)(288) = 0.72 \text{ IN}^2/\text{FT}$

USE EITHER (1) #8 BAR = 0.79 IN^2
(2) #6 BARS = 0.88 IN^2

$\frac{0.72}{12"} = \frac{0.88}{S_{req'd}} \rightarrow S_{req'd} = 14.67 < 18"$ ∴ OK

THESIS : SHEAR WALL | CYNTHIA MILINICHIK 3/3

TRY: (2) NO. 6 BARS @ 15"

CHECK: SHEAR CAPACITY FOR THE TRIAL VERTICAL REINFORCEMENT

NOMINAL SHEAR CAPACITY $V_u = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_t f_y)$

$\frac{h_w}{l_w} = \frac{54.5}{17} = 3.2 > 2 \therefore \alpha_c = 2.0$

$A_{cv} = (24)(54.5 \times 12) = 15{,}696 \text{ IN}^2$

$\rho_t = \frac{2(0.44)}{(12)(24)} = 0.00305$

$V_n = 15696(2\sqrt{4000} + 0.00305(60{,}000))/1000$
$= 4858^k$

$\phi V_n = 0.6(4858^k) = 2915^k$

$\phi V_n = 2915^k > 16.3^k$

COMMENTS: THE V_u IS MUCH GREATER THEN THE ϕV_n LIKELY BECAUSE THIS ANALYSIS IS ONLY OF ONE FLOOR AND THE SHEAR WALL IS DESIGNED FOR MANY FLOORS

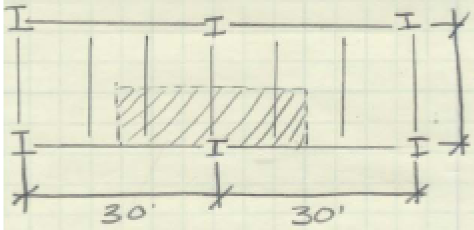
SINCE $\phi V_n > V_u$ (2) #6 BARS @ 15" C.C. FOR BOTH HORIZONTAL & VERTICAL DIRECTIONS

THESIS: COLUMN | CYNTHIA MILINICHIK 1/1

SPOT CHECK:

TYPICAL 49TH FLOOR COLUMN ALONG SOUTH FACE OF BUILDING PERIMETER

W14×605
h=17'



TRIB AREA = 30'×22.5'
= 675 SF

LOADING — OFFICE
LL = 50 PSF
PART = 20 PSF
70 PSF

DL = 55 PSF
SI = 15 PSF
70 PSF

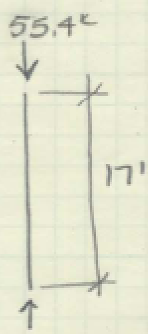
SNOW: 17.5 PSF

LOAD CASES
1.4D = 1.4(70) = 98 PSF

1.2D + 1.6L + 0.5S
1.2(70) + 1.6(70) + 0.5(17.5) = 205 PSF
GOVS.

1.2D + 1.6S + 0.5L
1.2(70) + 1.6(17.5) + 0.5(70) = 147 PSF

(675 SF)(205 PSF) = 138.4K



W14×43 ϕP = 228k

NOTE: THIS COLUMN IS MUCH SMALLER THEN THE W14×605 IN THE ACTUAL BUILDING.

I AM NOT SURE WHY THE USE SUCH A LARGE COLUMN BECAUSE THE STEEL MEMBERS CANNOT TAKE MOMENT & THEREFORE DONT HAVE COMBINE LOADING FROM LATERAL LOADS.

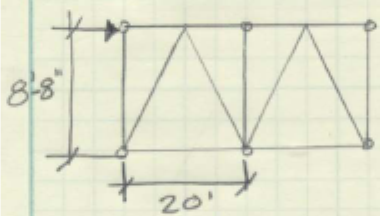
THESIS: BRACED FRAME | CYNTHIA MILINICHIK 1/1

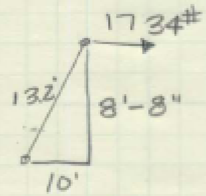
BRACED FRAME SPOT CHECK

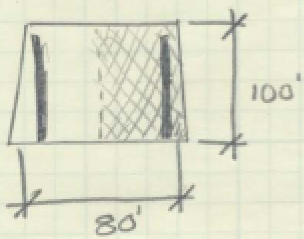
CROWN LEVEL 3
EAST ELEVATION

$\sqrt{10^2+(8'-8")^2} = 13.2'$

50 PSF = WIND







$(40')(50\text{ PSF}) = 200\text{ PLF}$

$200\text{ PLF}(8.67') = 1734\text{ lbs}$

$A_g = \frac{1734^{\#}}{50\text{ KSI}} = 0.03468\text{ IN}^2$

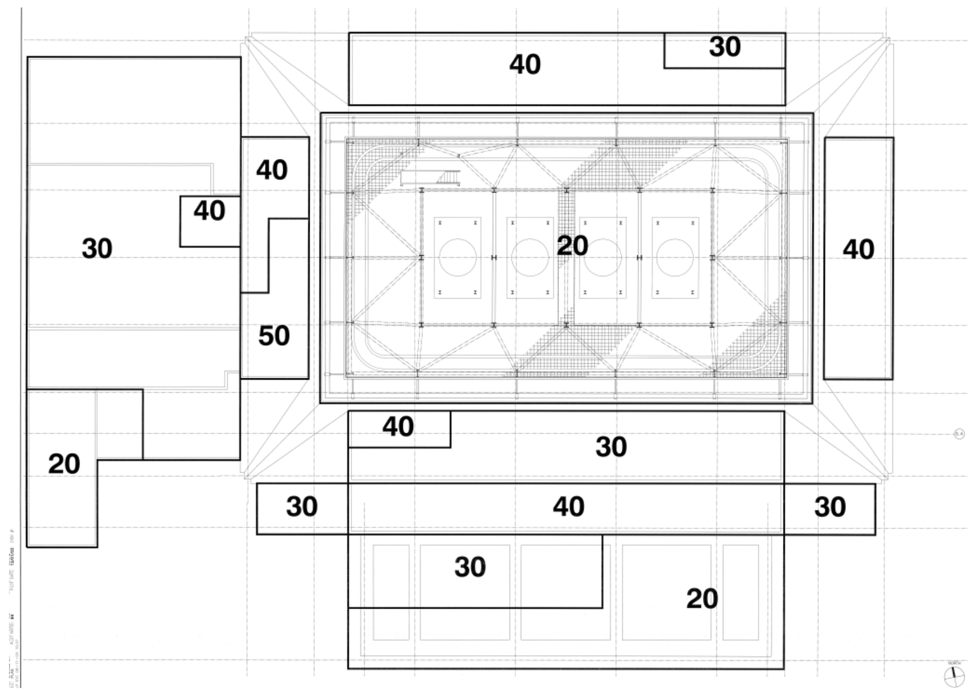
COMMENT: THIS VALUE SEEMS VERY SMALL LIKELY BECAUSE YIELDING DOESNT CONTROL, INSTEAD BUCKLING

THE DESIGN CALLS FOR A ⫫ L 6x6x3/8 WHICH HAS AN AREA OF 8.76 IN²

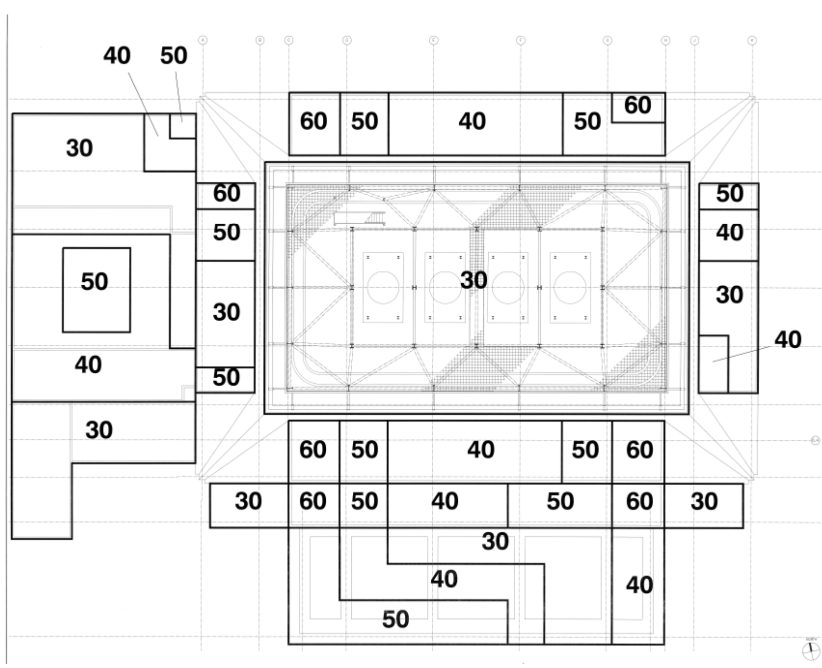
$KL = 14' \rightarrow \phi P_n = 166^K > P_u = 1.734^K$

∴ ANGLE SATISFY DESIGN.

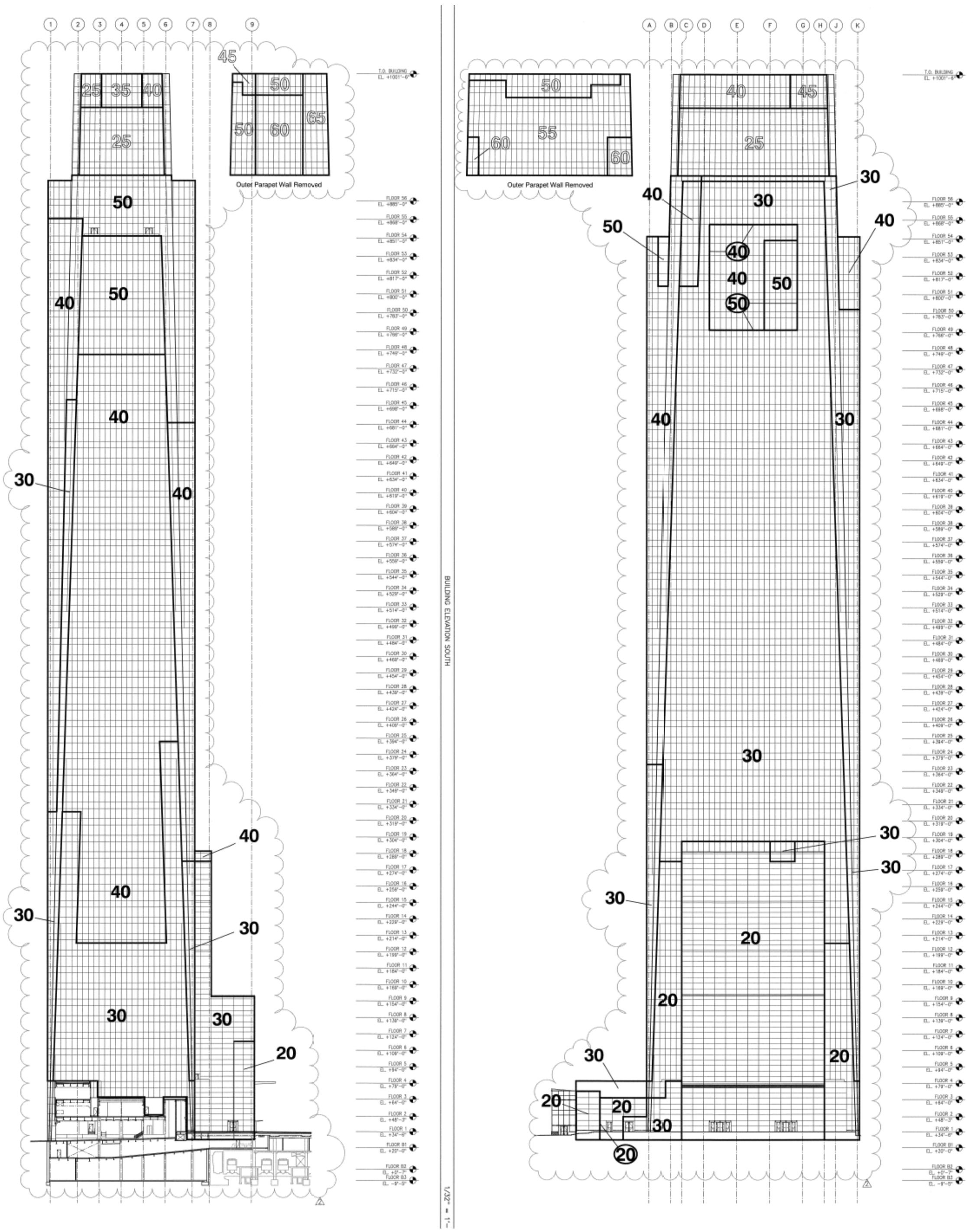
APPENDIX C: WIND LOAD CHECKS & WIND TUNNEL TESTING REPORT



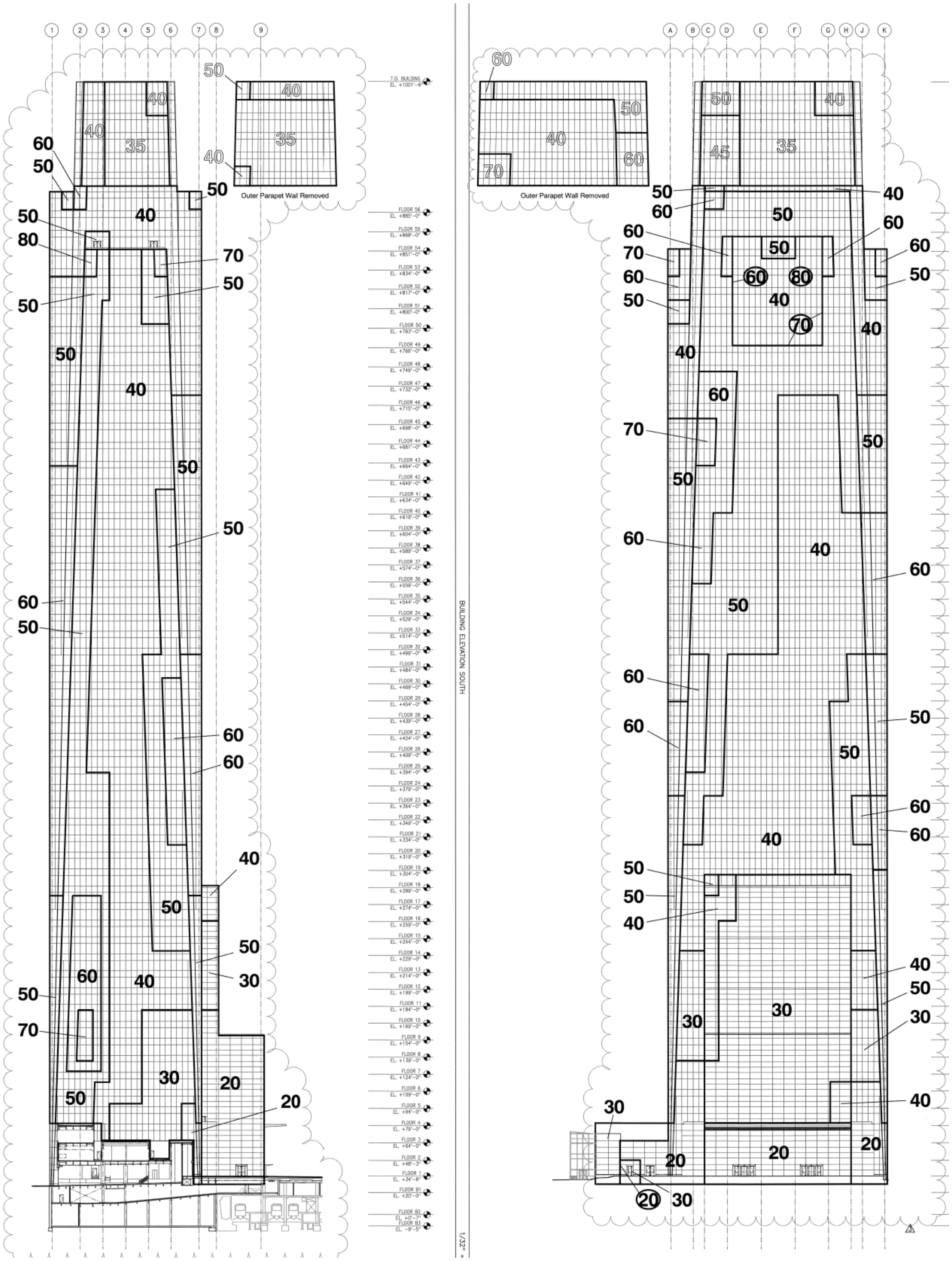
BLOCK DIAGRAMS OF PEAK DIFFERENTIAL PRESSURES FOR A 50 YEAR RETURN PERIOD



BLOCK DIAGRAMS OF PREDICTED PEAK DIFFERENTIAL SUCTIONS FOR A 50 YEAR RETURN PERIOD



BLOCK DIAGRAMS OF PREDICTED PEAK DIFFERENTIAL <u>PRESSURES</u> FOR A 50 YEAR RETURN PERIOD



BLOCK DIAGRAMS OF PREDICTED PEAK DIFFERENTIAL SUCTIONS FOR A 50 YEAR RETURN PERIOD

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.

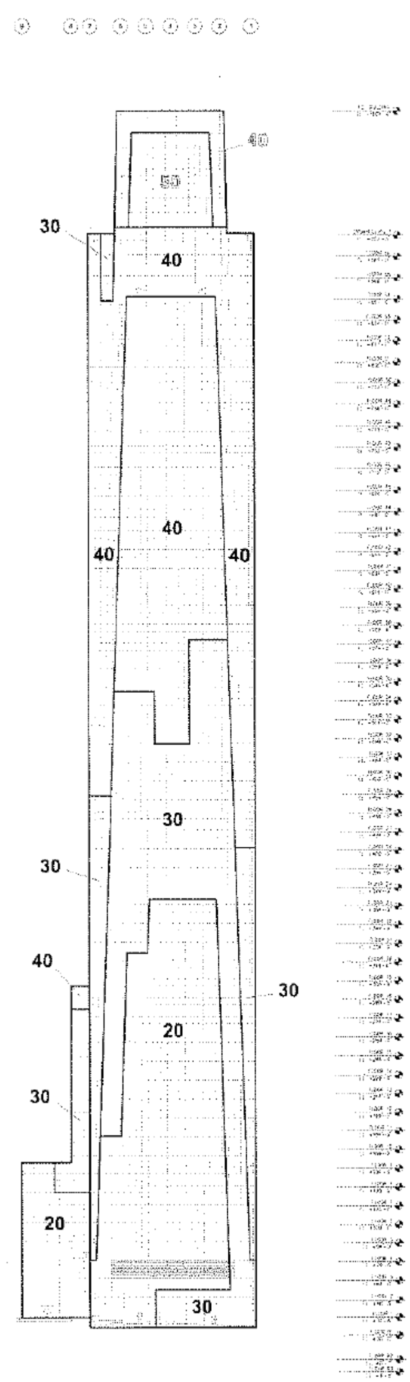
TABLE 2a LOADS AND RESPONSES FOR ONE PENNSYLVANIA PLAZA FOR A 10-YEAR RETURN PERIOD

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.

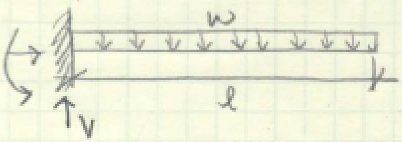
Report: BLWT-SS34-2005 - 61 - Alan G. Davenport Wind Engineering Group

THESIS: WIND

CYNTHIA MILINICHIK

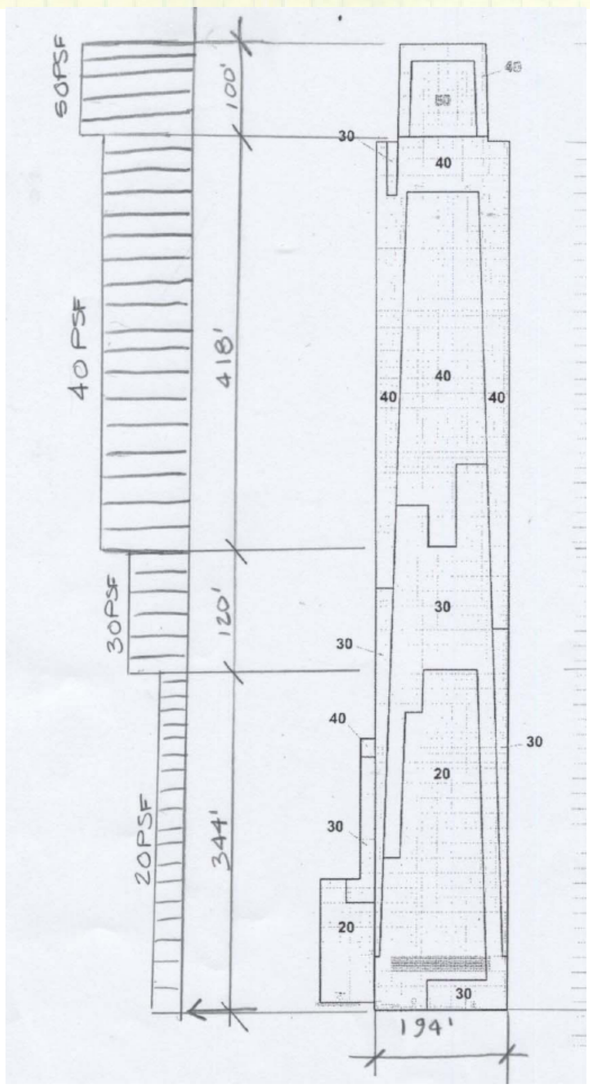
ROUGH ESTIMATE OF BASE SHEAR CAUSED BY WIND

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 LOAD CALCULATIONS

Excel Spreadsheet calculating weight of each floor based on use designated in Architecture plans.

			15	45	150	15	45	20	150	100	60	50	65	150
	Level	Total W_{floor} (lbs)	$W_{dead\ Office}$ (lbs)	$W_{dead\ Mechan}$ (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	3371680	331560	0	0	0	0	442080	0	0	1326240	0	0	1272000
Office	43	3371680	331560	0	0	0	0	442080	0	0	1326240	0	0	1272000
Office	42	3371680	331560	0	0	0	0	442080	0	0	1326240	0	0	1272000
Office	41	3371680	331560	0	0	0	0	442080	0	0	1326240	0	0	1272000
Office	40	3551680	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	39	3551680	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	38	3551680	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	37	3551680	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	36	3551680	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	35	3551680	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	34	3551680	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	33	3551680	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	32	3551680	331560	0	0	0	0	442080	0	0	1326240	0	0	1452000
Office	31	3551680	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
Office	27	3530600	328200	0	0	0	0	437600	0	0	1312800	0	0	1452000
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
Office	9	4243120	355440	0	0	0	0	473920	0	0	1421760	0	0	1992000
Office	8	4246730	356010	0	0	0	0	474680	0	0	1424040	0	0	1992000
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	2062000
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

THESIS: TECH REPORT 1 | CYNTHIA MILINICHIK

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 = 4½

DEFLECTION APPLICATION FACTOR, Cd = 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

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

$C_s = \frac{1.2 A_v S}{R T^{2/3}} = \frac{1.2(0.1)(1.0)}{(4.5)(8.898s)^{2/3}} = 0.00621$

$T = < T_a C_a = 1.7(5.234) = 8.898\,s$

NOTE: MY PLANS INDICATE A T = 7.39s

$C_a = 1.7$ UPPER LIMIT

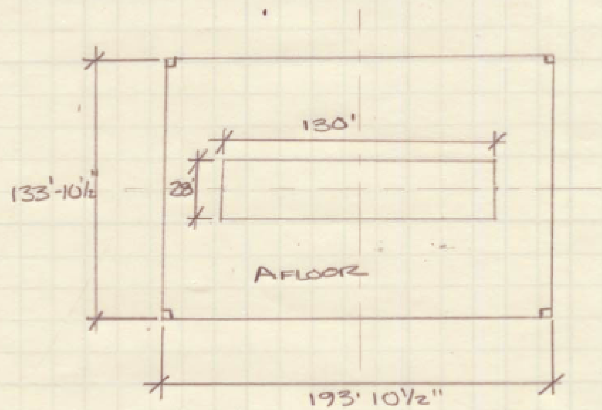
$T_a = C_T h_n^{3/4} = (0.03)(975)^{3/4} = 5.234$

$h_n = 975'$

$C_T = 0.03$

THESIS: TECH REPORT 1 | CYNTHIA MILINICHIK 2/

W= WEIGHT OF BUILDING



TYPICAL LOWER LEVEL FLOOR PLAN

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

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

$k = 2$ BECAUSE $T = 8.898\text{s} > 2.5\text{s}$

$W = 240,319.1$ KIPS

$V = C_s W = (0.00621)(240,319) = 1492^K$