Technical Report #1

Structural Concepts | Structural Existing Conditions Report

Three PNC Plaza

Pittsburgh, PA

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Architectural Engineering

Structural Option

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EXECUTIVE SUMMARY

The purpose of this technical report is to analyze and assemble the existing structural conditions of the Three PNC Plaza building located in Pittsburgh, Pennsylvania. Analysis of the current structural systems, gravity loads, and lateral loads were studied in detail and provided within this report along side of graphics and tables to maximize understanding of the concepts.

The report begins with a basic introduction for the reader to become familiar with Three PNC Plaza and the functions it needs to accomplish. The brief introduction to the building is followed by a more in-depth introduction of the structural systems specifically the foundation, floor system, columns, and lateral system. The structural system narrative is followed by the design codes, references, and materials used during the design of the building and the design codes that were used for the calculations in this report.

Gravity loads for calculations in this report were assembled using ASCE 7-10. These values were compared to the actual loads used during the design from earlier versions of these documents. Following the gravity load calculations a wind analysis was completed according to ASCE 7-10. Seismic forces were also calculated according to ASCE 7-10.

Spot check calculations were performed at critical portions of the building. The spot checks were performed on a typical 30' by 42.5' bay located on the 17th floor. The decking, fill beams, and girders were all checked for this bay. A column check was also performed spanning the length of the building.

INTRODUCTION

Three PNC Plaza is a 23 story, 780,000 square foot, mixed use high-rise building located in the heart of downtown Pittsburgh, Pennsylvania as seen in figure 2 highlighted in red. The erection of this building was a significant part to revitalizing the downtown area and marked the first new high-rise built in the city in the last 20 years.

Figure 1- Three PNC Occupancy Layout

The building is mixed-use and allows for several different tenants occupy the building as seen in figure 1.Fairmont Hotels and Resorts move into the building in March, 2010 with 185 rooms that are located on floors 14 through 23. Along with the Fairmont Hotels, 28 Residences condominium units will occupy floors 14 through 23 in the fall of 2010. The building has 10 floors of office space located from the 3rd through 13th floor. These office spaces are home to PNC Bank and the REED Smith Law Firm. The lower floors of the building house several different retail stores, restaurant, and wine bar.

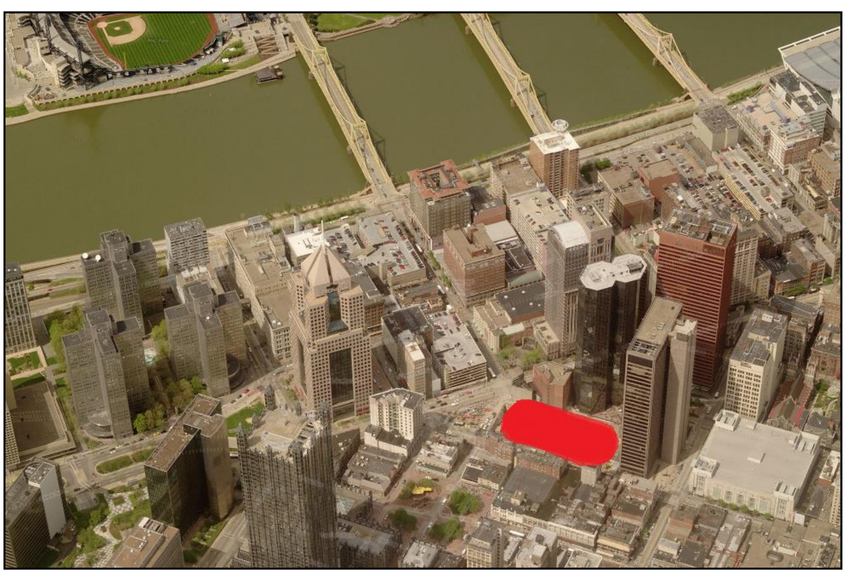
Figure 2- Three PNC Site Location

STRUCTURAL SYSTEMS

Foundation System

Pittsburgh is known for alluvial deposits which mean shallow foundations were not possible and deep foundations were required for Three PNC Plaza. Also, the Pittsburgh area soil overburden is 60' to bedrock. This means that after the 30' of excavation for the buildings parking garage structure, 30' of soil would still remain until the bedrock would be reached. Several different options for the foundation of the building were considered such as; auger cast pile, piles, H-piles, and caissons. Ultimately, the foundation system chosen for Three PNC Plaza were caissons bearing on bedrock to achieve maximum axial capacity. Four different size caissons were chosen for the foundation as seen in the Caisson Schedule in figure 3. The caissons were designed for a typical column reaction of 3500 kips. Brayman Construction Corporation was in charge of the installation of the 121 caissons for the building. A typical caisson detail has been provided in figure 2. The caissons bearing value is 15 tons per square foot and were drilled to auger refusal or socketed into the bedrock. The layout for the caissons can be seen in figures 5 and 6 located on the next page.

CAISSONS $F_{BR.}$ =30K/SQ. FT.

MARK	SIZE Ø	VERT.REINF. length=3 X DIA.	TIES	DOWELS
A.	48"	7–#10	#4@18" O.C.	4–#8 X 8'–0" DEVELOP INTO PEDESTAL
B.	54"	9–#10	#4@18" O.C.	
C.	42"	7–#9	#3@18" O.C.	
D.	60"	9–#11	#3@18" O.C.	

Figure 3- Caisson Schedule

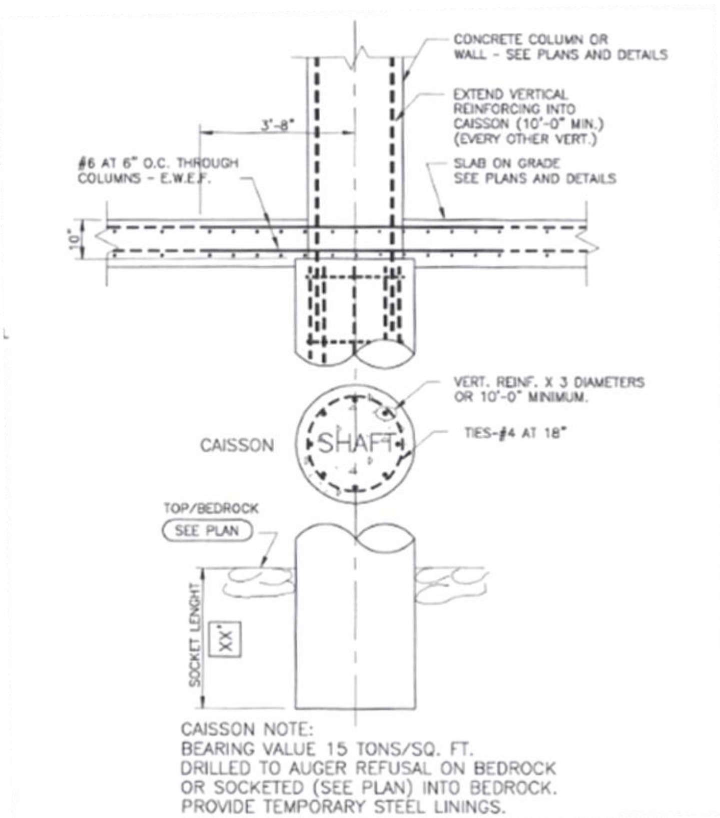

Figure 4- Caisson Detail

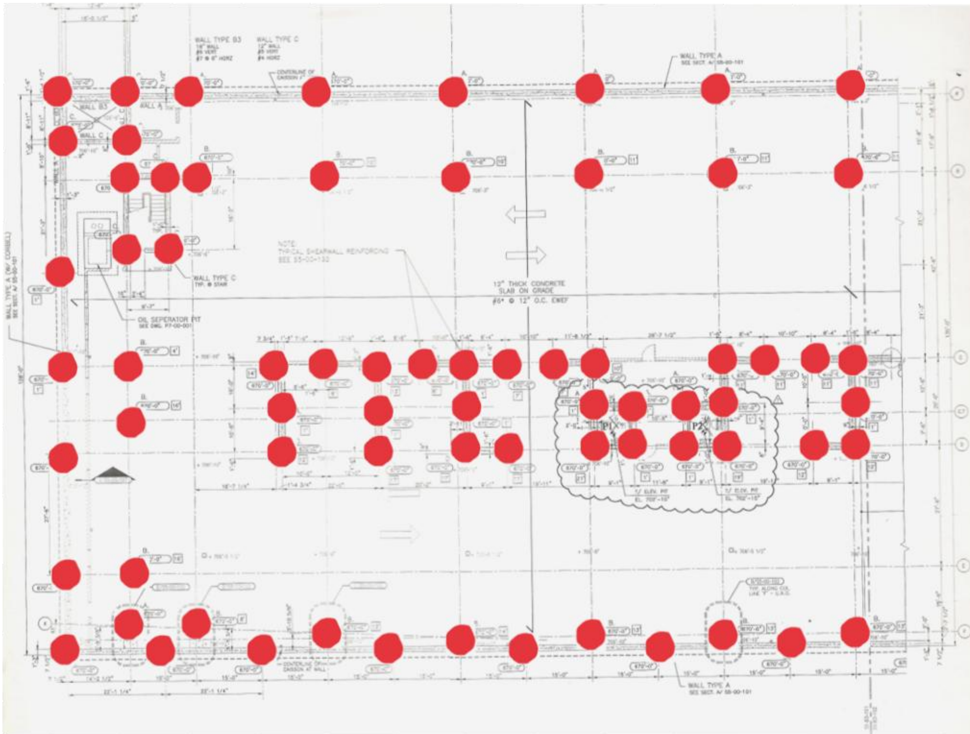

Figure 5- West End Building Caissons

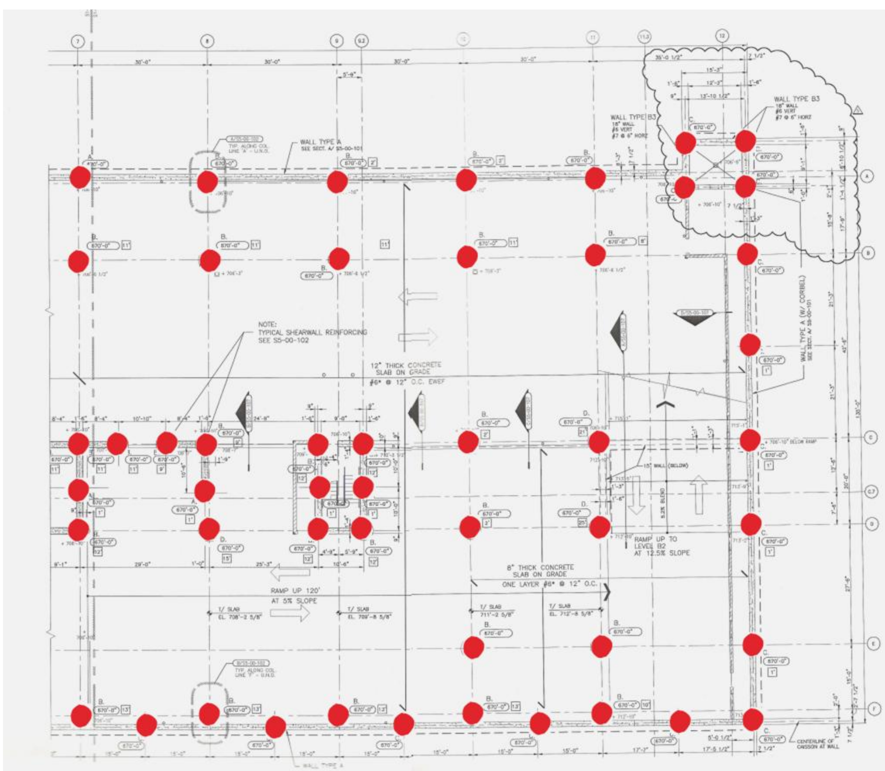

Figure 6- East End Building Caissons 1

Floor System

Three PNC uses a composite steel and concrete floor system with a typical bay size of 30'-0" x 42'-6". The composite slab is composed of 2" 18-gauge metal floor deck with 3-½" light weight concrete, netting a total thickness of 5-½". The concrete is reinforced with one layer of 6x6-W2.1xW2.1 wielded wire fabric. The composite deck transfers its load to fill beams that are placed at 10'-0" on center and primarily W21X44 beams with W24X62 girders. This floor design is used throughout the structure and different sized fill beams are used to deal with higher load areas.

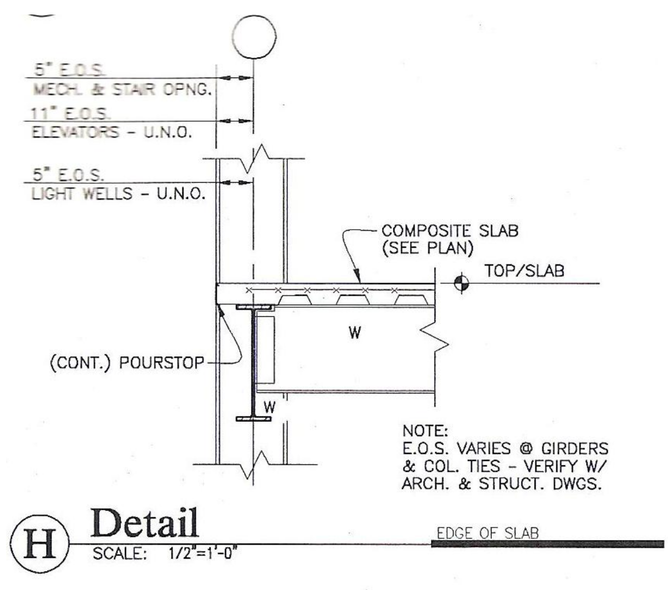

Figure 7- Composite Floor

Columns

Three PNC Plaza uses a variety of steel and concrete to support the gravity load of the building. The size of these columns can range in sizes from W14x68 all the way to a W14x740 in some cases. The core of the building is supported by concrete shear walls up until the 14th floor which they then switch over to steel columns. The remainder of the building is supported by steel columns from the ground floor that attach to concrete columns located in the parking garage. The steel columns in the building are spliced together at a typical distance of 24'-0" as see in figure 8.

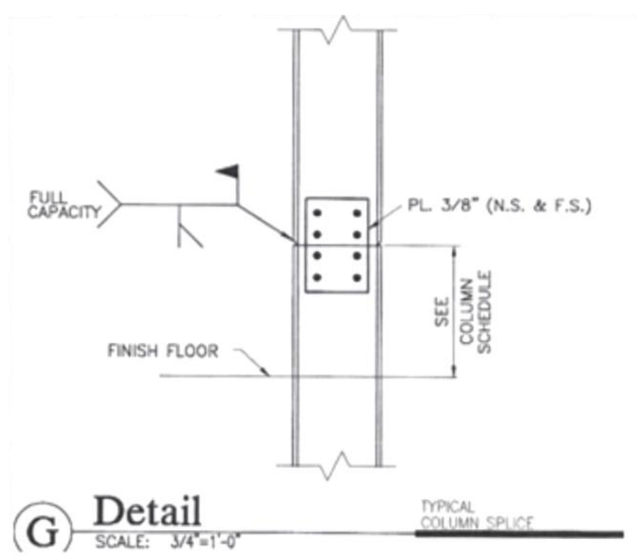

Figure 8- Splice Detail

Lateral System

The main lateral resistant system used in Three PNC Plaza is a combination of several concrete shear walls. These shear walls are located throughout the core of the building and encase the stairwells and elevators as seen in figure 9 highlighted in red. The shear walls start at the lowest level of the parking garage structure and extend up until the 14th floor where they are met with steel columns. All of the shear walls used a concrete with a compressive strength of 5000 ksi. The reinforcement for the shear walls changed depending on the location and can be seen in the shear wall Reinforcement schedule located in Appendix D. A more detailed view of the shear walls at key locations of the wall can be in figures 10-13.

Figure 9- Shear Wall Layout

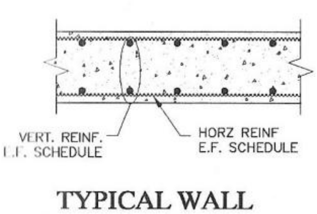

Figure 10- Typical Shear Wall

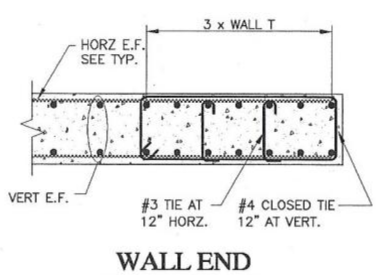

Figure 11- Shear Wall End

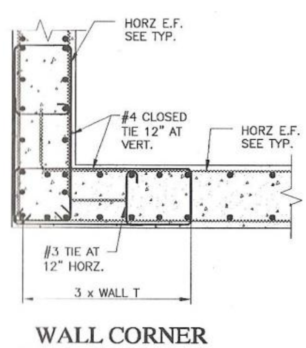

Figure 12- Shear Wall Corner

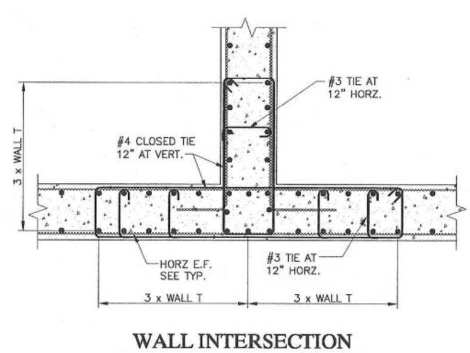

Figure 13- Shear Wall Intersection

Roof System

The roof structural system is very similar to the floor structural system used throughout the building. It utilizes the same composite deck and slab configuration along with same typical bay dimensions. However, the fill beams are spaced closer together, at a typical spacing of 7.5 feet. These fill beams can differ in size from a W21x44 to a W27x129.

CODES AND REFERENCES

Design Codes Used:

1. International Building Code 2003
2. AISC Manual of Steel Construction Ninth Edition (ASD)
3. AISC Manual of Steel Construction Load and Resistance Factor Design Second Edition
4. ACI 318 American Concrete Institute Building Code Requirements for Structural Concrete
5. ASCE 7-98 Minimum Design Loads for Buildings and Other Structures

Thesis Codes Used:

1. International Building Code, IBC 2010
2. American Society of Civil Engineers, ASCE 7-10
3. AISC Manual of Steel Construction Thirteenth Edition (LRFD)

MATERIAL STRENGHTS

Concrete

Location	Strength f'c (ksi)
Columns	10000 psi
Interior Slab on Grade	5000 psi
Caissons and Grade Beams	5000 psi
Retaining Walls	5000 psi
Post Tension Slabs	5000 psi
Beams with PT Slab	5000 psi
Core Walls	5000 psi
Exterior Slab on Grade	4000 psi
Exterior topping Slabs	4000 psi
Composite Slab Fill	3000 psi
Footings and Misc.	3000 psi

Structural Steel

Type	Standard	Grade
W Shapes	ASTM A992	50 ksi
S,M, and HP Shapes	ASTM A36	
Tubes	ASTM A500	Class B
Channels	ASTM A36	
Angles	ASTM A36	
Plates	ASTM A36	

LOADINGS

Location	Design (IBC 2003)	Thesis (ASCE 7-10)
Retail	100 psf	100 psf
Office	50 psf	50 psf
Library	150 psf	150 psf
Hotel	40 psf	40 psf
Condominium	40 psf	40 psf
Ballroom	100 psf	100 psf
Garage	40 psf	40 psf
Mechanical Rooms	200 psf	-
Assembly Areas	100 psf	Depends on Area
Balconies	100 psf	1.5*Live Load
Restaurants	100 psf	-
Roof	30 psf	20 psf
Stairs and Lobby	100 psf	100 psf
Corridors	80 psf	80 psf

Floor Dead Loads	
Composite Decking	44 psf
Superimposed Dead Load	30 psf
Total	**74 psf**

Curtain Wall Dead Load:

Assumed curtain wall was 8” thick and that the material weighted 40psf. This resulted in a load of 60plf.

Snow Loads

Snow loads were designed by section 7.3 of ASCE 7-10. It was found that the Pf would be 17.325 lb/sq.ft. The shape of the building also results in a drift load calculation. The calculations can be seen below.

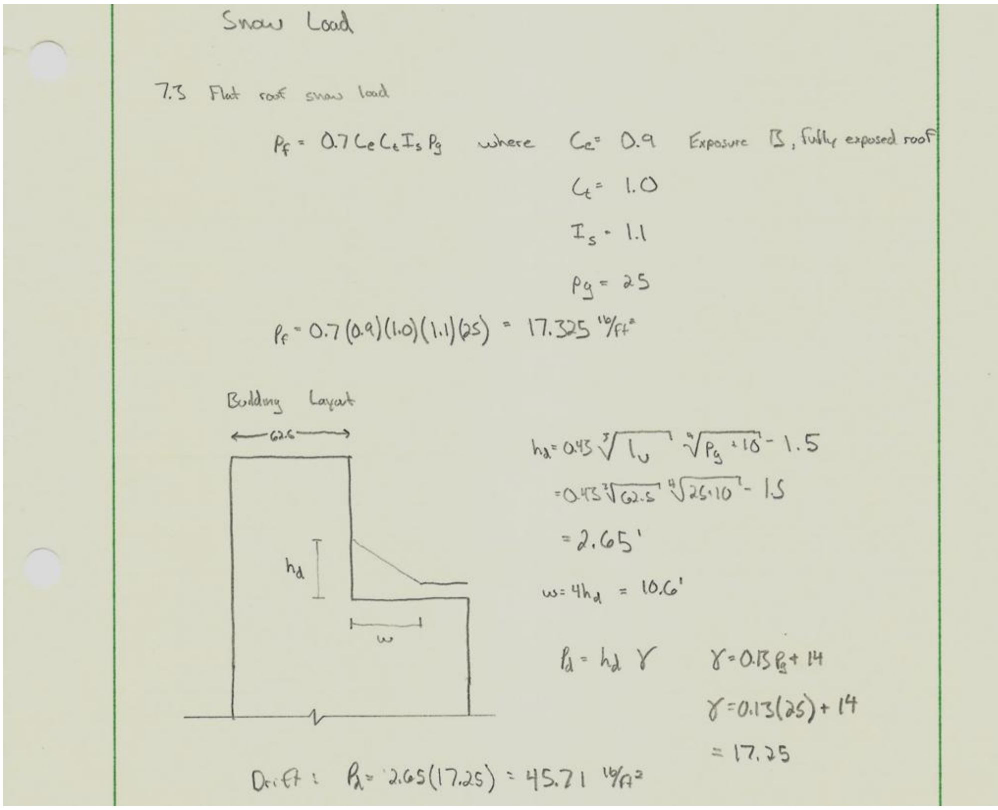

Figure 14- Snow Load Calculations

Wind Load Analysis

Wind load analysis for the Main Wind-Force Resisting System in Three PNC Plaza was determined using ASCE 7-10. The analysis was calculated for both North-South, and East-West directions using the enclosed and partially enclosed section in Chapter 27 of ASCE 7-10.

The first step in the wind calculations was determining if Three PNC Plaza should be calculated under the assumption of a rigid structure or a flexible structure. This was determined by finding the natural frequency of the building according to ASCE 7-10 Chapter 12 section 8. It was found that the natural frequency from the calculations was less than 1 resulting in the building being defined as a flexible structure.

When running the calculations for the wind load the building was simplified into two distinct portions to account for the buildings shape as seen in figure 15. It was also assumed that the small cutout of the building along the front portion 14^{th} floor would be modeled as if it was filled. From the calculations that were performed in an excel spread sheet you can see that the North-South direction produces the strongest wind forces due to the larger surface area.

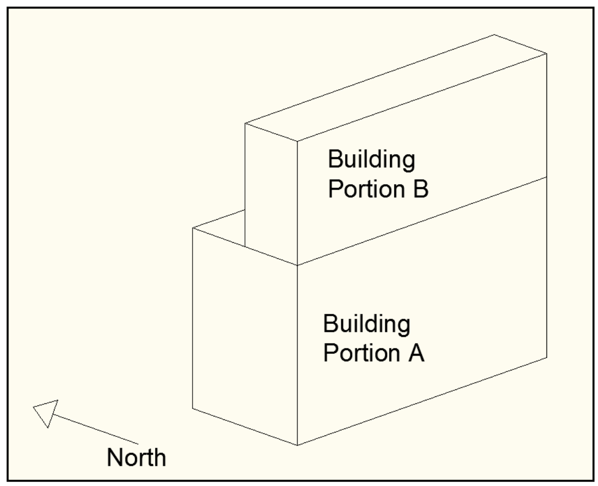

Figure 15- Building Portions

The last part of the analysis was the base shear and overturning moment calculations. Tables for these calculations have been provided along with a typical hand worked solution in appendix A to see the math involved. These values can be compared to the results from the seismic calculations to determine the governing lateral force to use during design.

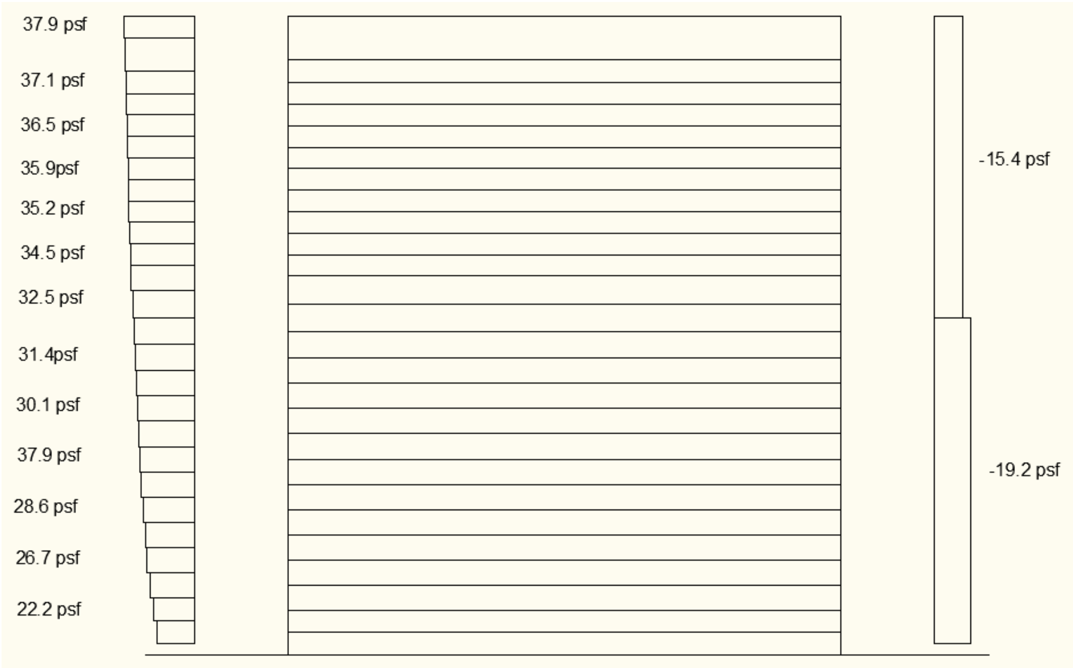

Figure 16- East/West Wind Distribution

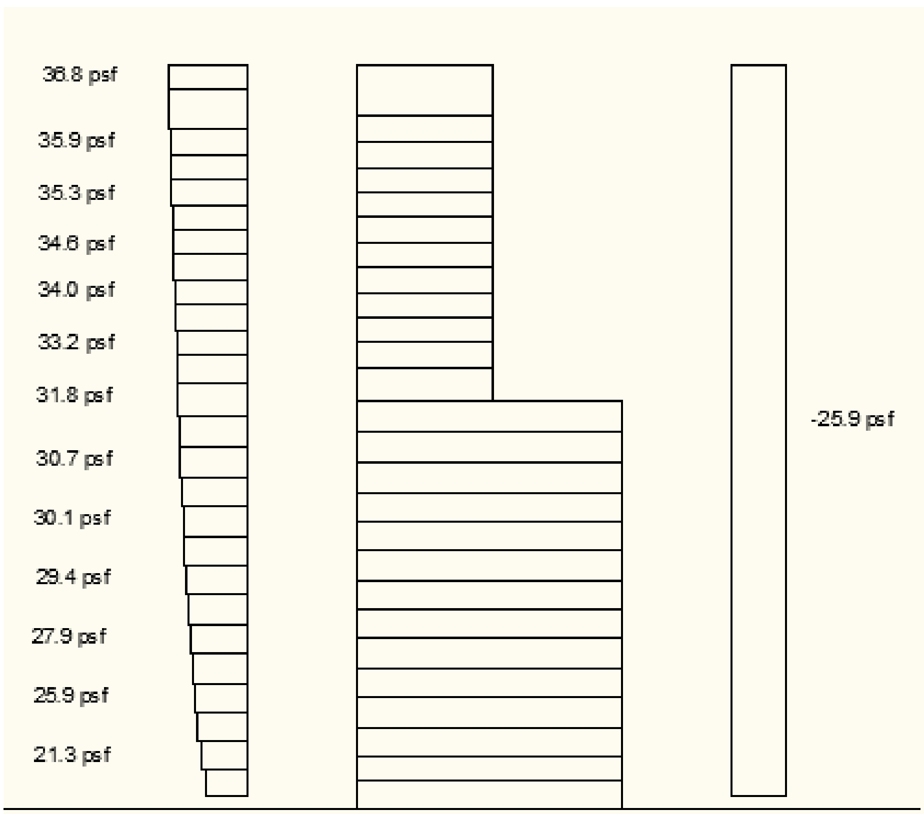

Figure 17- North/South Wind Distribution 1

*Some values not shown to allow for easy reading.

North/South

Story	Height (ft)	kz or kh	qz	Windward (psf)	Windward (plf)	Windward (kips)	Leeward (psf)	Leeward (plf)	Leeward (kips)	Story Force (kips)	Moment (k-ft)
Building Portion A											
1mezz	12.5	0.5700	17.8606	19.5169	5796.5228	36.3386	-25.9635	-7711.1691	-46.2670	82.6056	1032.5698
2	24.0	0.6520	20.4300	21.3432	6338.9335	38.0589	-25.9635	-7711.1691	-48.1948	86.2537	2070.0891
2mezz	37.5	0.7450	23.3441	23.3311	6929.3223	44.7804	-25.9635	-7711.1691	-52.0504	96.8308	3631.1533
3	51.0	0.8140	25.5062	24.7911	7362.9656	48.2365	-25.9635	-7711.1691	-52.0504	100.2869	5114.6300
4	64.5	0.8680	27.1983	25.9259	7699.9949	50.8375	-25.9635	-7711.1691	-52.0504	102.8879	6636.2685
5	78.0	0.9220	28.8903	27.0530	8034.7497	53.1048	-25.9635	-7711.1691	-52.0504	105.1552	8202.1021
6	91.5	0.9645	30.2220	27.9364	8297.1251	55.1201	-25.9635	-7711.1691	-52.0504	107.1705	9806.0979
7	105.0	1.0025	31.4127	28.7231	8530.7504	56.7941	-25.9635	-7711.1691	-52.0504	108.8445	11428.6695
8	118.5	1.0363	32.4703	29.4190	8737.4353	58.2801	-25.9635	-7711.1691	-52.0504	110.3305	13074.1664
9	132.0	1.0700	33.5278	30.1130	8943.5628	59.6734	-25.9635	-7711.1691	-52.0504	111.7238	14747.5363
10	145.5	1.1010	34.4992	30.7484	9132.2840	61.0060	-25.9635	-7711.1691	-52.0504	113.0564	16449.7024
11	159.0	1.1280	35.3452	31.2996	9295.9857	64.5194	-25.9635	-7711.1691	-53.9782	118.4976	18841.1168
12	173.5	1.1570	36.2539	31.8905	9471.4772	68.0321	-25.9635	-7711.1691	-55.9060	123.9380	21503.2480
13	188.0	1.1820	37.0373	32.3975	9622.0429	70.4168	-25.9635	-7711.1691	-56.8699	127.2866	23929.8879
Building Portion B											
14	203.0	1.2048	37.7517	32.8887	9767.9585	64.1655	-25.9635	-7711.1691	-51.0865	115.2520	23396.1635
15	214.5	1.2232	38.3282	33.2600	9878.2203	56.4828	-25.9635	-7711.1691	-44.3392	100.8220	21626.3161
16	226.0	1.2416	38.9048	33.6309	9988.3703	57.1164	-25.9635	-7711.1691	-44.3392	101.4557	22928.9815
17	237.5	1.2600	39.4813	34.0014	10098.4172	57.7495	-25.9635	-7711.1691	-44.3392	102.0887	24246.0749
18	249.0	1.2784	40.0579	34.3716	10208.3683	58.3820	-25.9635	-7711.1691	-44.3392	102.7212	25577.5864
19	260.5	1.2947	40.5686	34.6980	10305.3196	58.9769	-25.9635	-7711.1691	-44.3392	103.3161	26913.8376
20	272.0	1.3108	41.0731	35.0201	10400.9685	59.5306	-25.9635	-7711.1691	-44.3392	103.8698	28252.5858
21	283.5	1.3269	41.5776	35.3419	10496.5495	60.0804	-25.9635	-7711.1691	-44.3392	104.4196	29602.9528
22	295.0	1.3430	42.0821	35.6635	10592.0666	60.6298	-25.9635	-7711.1691	-44.3392	104.9690	30965.8531
23	306.5	1.3578	42.5458	35.9581	10679.5446	63.8258	-25.9635	-7711.1691	-46.2670	110.0928	33743.4380
Roof Main	319.0	1.3728	43.0159	36.2556	10767.9035	95.2890	-25.9635	-7711.1691	-68.4366	163.7256	52228.4816
Roof High	342.0	1.4004	43.8807	36.8025	10930.3534	61.9154	-25.9635	-7711.1691	-44.3392	106.2547	36339.0962
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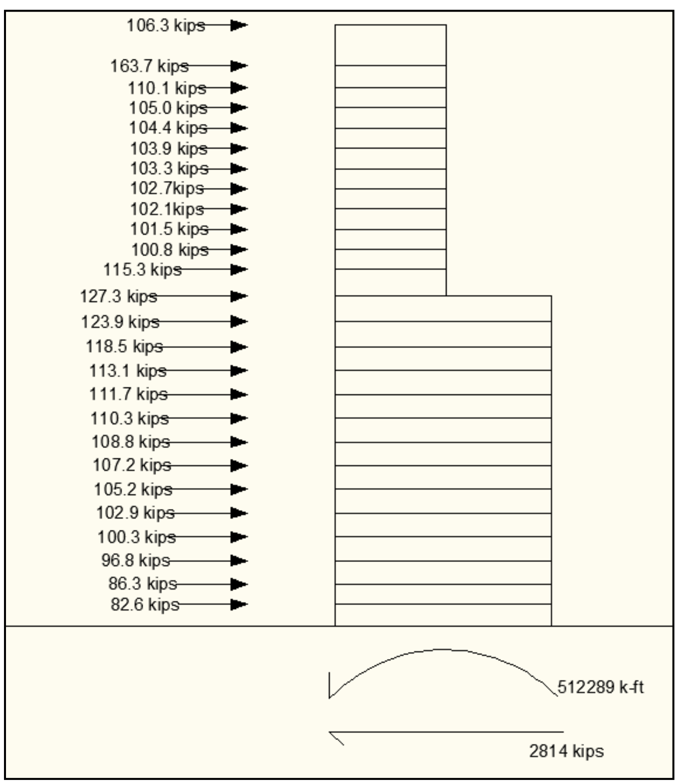


Figure 18- North/South Wind Story Forces 1

East/West

Story	Height (ft)	kz or kh	qz	Windward (psf)	Windward (plf)	Windward (kips)	Leeward (psf)	Leeward (plf)	Leeward (kips)	Story Force (kips)	Moment (k-ft)
Building Portion A											
1mezz	12.5	0.5700	17.8606	20.4289	2507.6452	15.6829	-19.1592	-2351.7925	-14.1108	29.7937	372.4207
2	24.0	0.6520	20.4300	22.2340	2729.2197	16.4206	-19.1592	-2351.7925	-14.6987	31.1193	746.8632
2mezz	37.5	0.7450	23.3441	24.2069	2971.3970	19.2396	-19.1592	-2351.7925	-15.8746	35.1142	1316.7818
3	51.0	0.8140	25.5062	25.6370	3146.9396	20.6494	-19.1592	-2351.7925	-15.8746	36.5240	1862.7233
4	64.5	0.8680	27.1983	26.7385	3282.1476	21.6982	-19.1592	-2351.7925	-15.8746	37.5728	2423.4436
5	78.0	0.9220	28.8903	27.8417	3417.5639	22.6115	-19.1592	-2351.7925	-15.8746	38.4861	3001.9178
6	91.5	0.9645	30.2220	28.6985	3522.7371	23.4235	-19.1592	-2351.7925	-15.8746	39.2981	3595.7775
7	105.0	1.0025	31.4127	29.4610	3616.3335	24.0944	-19.1592	-2351.7925	-15.8746	39.9690	4196.7410
8	118.5	1.0363	32.4703	30.1345	3699.0127	24.6893	-19.1592	-2351.7925	-15.8746	40.5639	4806.8212
9	132.0	1.0700	33.5278	30.8101	3781.9423	25.2482	-19.1592	-2351.7925	-15.8746	41.1228	5428.2125
10	145.5	1.1010	34.4992	31.4285	3857.8446	25.7843	-19.1592	-2351.7925	-15.8746	41.6589	6061.3670
11	159.0	1.1280	35.3452	31.9628	3923.4360	27.2427	-19.1592	-2351.7925	-16.4625	43.7052	6949.1313
12	173.5	1.1570	36.2539	32.5382	3994.0612	28.7009	-19.1592	-2351.7925	-17.0505	45.7514	7937.8718
13	188.0	1.1820	37.0373	33.0296	4054.3872	29.6824	-19.1592	-2351.7925	-17.3445	47.0269	8841.0560
Building Portion B											
14	203.0	1.2048	37.7517	34.2302	2139.3860	21.3547	-15.4056	-962.8528	-11.5874	32.9421	6687.2484
15	214.5	1.2232	38.3282	34.5769	2161.0537	12.3638	-15.4056	-962.8528	-5.5364	17.9002	3839.5860
16	226.0	1.2416	38.9048	34.9253	2182.8327	12.4887	-15.4056	-962.8528	-5.5364	18.0251	4073.6674
17	237.5	1.2600	39.4813	35.2753	2204.7086	12.6142	-15.4056	-962.8528	-5.5364	18.1506	4310.7639
18	249.0	1.2784	40.0579	35.6267	2226.6690	12.7402	-15.4056	-962.8528	-5.5364	18.2766	4550.8770
19	260.5	1.2947	40.5686	35.9338	2245.8615	12.8585	-15.4056	-962.8528	-5.5364	18.3949	4791.8790
20	272.0	1.3108	41.0731	36.2377	2264.8579	12.9683	-15.4056	-962.8528	-5.5364	18.5047	5033.2844
21	283.5	1.3269	41.5776	36.5427	2283.9211	13.0777	-15.4056	-962.8528	-5.5364	18.6141	5277.1096
22	295.0	1.3430	42.0821	36.8487	2303.0437	13.1875	-15.4056	-962.8528	-5.5364	18.7239	5523.5586
23	306.5	1.3578	42.5458	37.1275	2320.4706	13.8727	-15.4056	-962.8528	-5.7771	19.6498	6022.6754
Roof Main	319.0	1.3728	43.0159	37.4082	2338.0117	20.6950	-15.4056	-962.8528	-8.5453	29.2404	9327.6737
Roof High	342.0	1.4004	43.8807	37.9270	2370.4379	13.4436	-15.4056	-962.8528	-5.5364	18.9800	6491.1500
									Sum=	795.11	123470.60

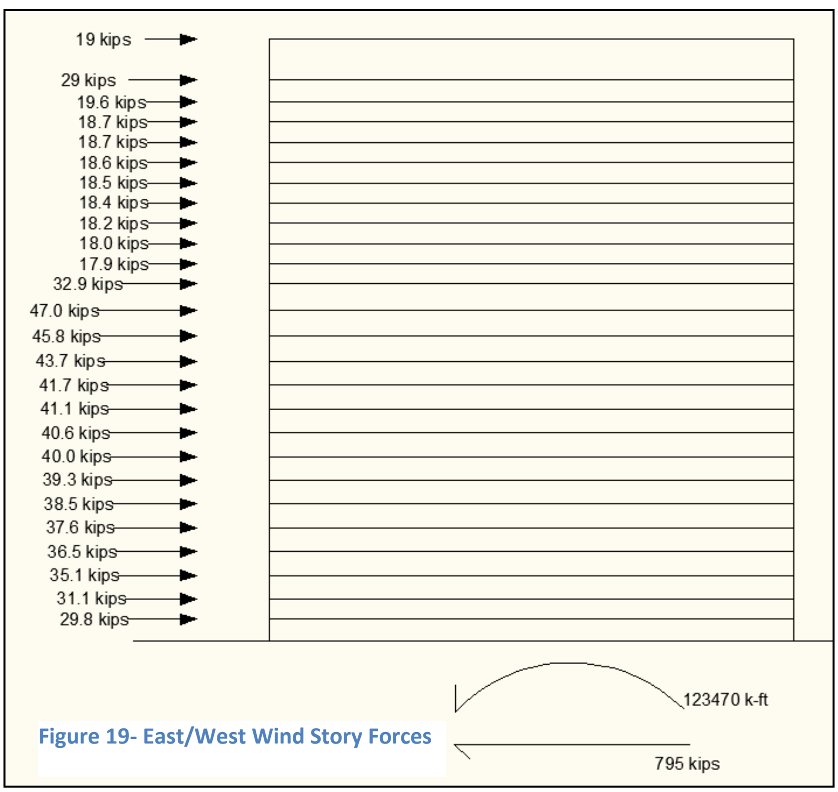


Figure 19- East/West Wind Story Forces

Seismic Load

The seismic loads were determined using the Equivalent Lateral Force Procedure according to ASCE 7-10. To aid in these calculations some of the seismic design parameters for Pittsburgh, PA were found from the USGS website using the Ground Motion Parameter Application. The configuration of Three PNC Plaza led me to use Special Reinforced Concrete Shear Walls as my seismic force-resisting system. The walls classify as special due to their cast-in-place construction.

The building weight was calculated to find the base shear force from the equation V=Cs(W). The weight was tabulated by finding the weight of each floor from all structural components such as beams, slabs, and columns as show in appendix B. These weights were then summed to get the total weight of the building. After the base shear force was calculated the vertical distribution of the seismic forces could be calculated as according to ASCE 7-10 section 12.8.3. These calculations resulted in a base shear of 2301 kips and a overturning moment of 492067 k-ft. The calculations relied heavily on Microsoft excel and can be seen below in the tables provided.

Seism Calculation Table

Floor Level	Floor Height (ft)	Total Height (ft)	Weight (kips)	$w*h^k$	C_{VX}	f_i (kips)	V_i (kips)	M_z (k-ft)
Main Roof	23	319.0	1500	1919975	0.068	157	157	50172
23	12.5	306.5	1542	1878212	0.067	154	311	47157
22	11.5	295.0	1541	1789995	0.064	147	458	43256
21	11.5	283.5	1541	1703809	0.061	140	597	39568
20	11.5	272.0	1547	1624764	0.058	133	730	36202
19	11.5	260.5	1547	1539954	0.055	126	857	32862
18	11.5	249.0	1549	1457924	0.052	119	976	29738
17	11.5	237.5	1549	1374833	0.049	113	1089	26748
16	11.5	226.0	1557	1299383	0.046	106	1195	24056
15	11.5	214.5	1557	1217839	0.043	100	1295	21399
14	11.5	203.0	1842	1345526	0.048	110	1405	22375
13	15	188.0	2207	1465657	0.052	120	1525	22572
12	14.5	173.5	2529	1520266	0.054	125	1650	21607
11	14.5	159.0	2483	1339401	0.048	110	1759	17445
10	13.5	145.5	2500	1207961	0.043	99	1858	14398
9	13.5	132.0	2500	1070464	0.038	88	1946	11575
8	13.5	118.5	2510	940066	0.033	77	2023	9125
7	13.5	105.0	2723	877695	0.031	72	2095	7549
6	13.5	91.5	2729	741527	0.026	61	2156	5558
5	13.5	78.0	2729	608265	0.022	50	2205	3887
4	13.5	64.5	2739	482232	0.017	40	2245	2548
3	13.5	51.0	2726	358608	0.013	29	2274	1498
2mezz	13.5	37.5	1665	149550	0.005	12	2287	459
2	13.5	24.0	2754	142169	0.005	12	2298	280
1mezz	11.5	12.5	1452	33360	0.001	3	2301	34
Ground	12.5	0.0	1452	0	0.000	0	2301	0
		Σ	51518	28089435	1	2301		492067

T=	1.510	s
k=	1.241	
V_b=	2301	kips

SPOT CHECKS

Spot checks were performed to determine the result of the gravity loads on the structure. The spot checks were performed at a typical bay located on the 7th floor as show in figure 20. The detailed hand calculations can be found in Appendix C. The 1st spot check was analyzing the composite slab used in construction. The slab used throughout all floors of the building is a Composite 2"x18 G.A Steel Deck with 3 ½" of light weight 3000psi concrete fill netting a total thickness of 5 ½". The decking was check in accordance to the Vulcraft Decking Catalog. The results of the first spot check concluded that the composite decking used throughout the building met the standards required.

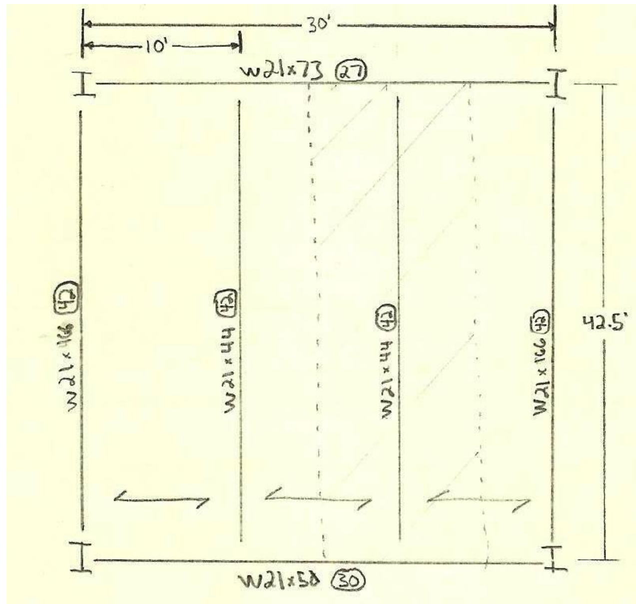


Figure 20- Typical Bay

The 2nd spot check was performed on the typical W21x44 fill beam in the typical 30' by 42.5' bay also located on the 7th floor. The calculations on theses fill beams showed that they are more then adequate to withstand the loadings being used. They also met the deflection checks for both live and wet concrete. They were finally check for shear stud requirements and resulted in a much lower value then used in the plans. The plans noted however that the shear stud requirements should be adjusted according to the deck manufacture. The 3rd spot check was very similar to the fill beam and analyzed the W21x50 girder. The girder also passed all checks that were performed. The conclusion of these two calculations was that the beams used in the design meet and exceed the loading.

A final check of an interior column was performed on at location D-10 in the plans as seen the figure XXX. The loading on the beam was assumed to be the dead load of the slab at 44psf and a superimposed load of 30psf. The live load changes throughout different floor levels and was taken into account in the spreadsheet. The final loading was the self weight which could become substantial at the lower portions of the column when large members were used. The typical splice length of the columns used in the building was 24'. Table XXX provides the calculations and the final force from the column.

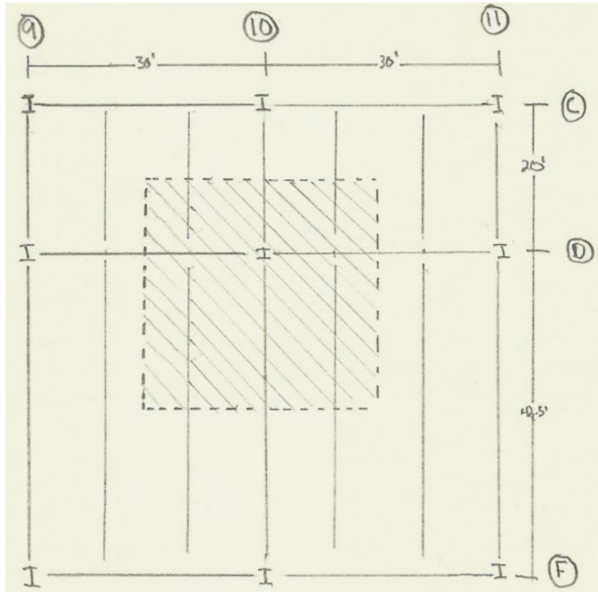


Figure 21- Column Tributary Area

Column Check						
Floor	Area	DL	LL	column size	spliced length	Pu
3	937.5	74	80	426	24	215518.8
4	937.5	74	80			203250
5	937.5	74	80	398	24	214712.4
6	937.5	74	80			203250
7	937.5	74	80	370	24	213906
8	937.5	74	100			233250
9	937.5	74	100	311	24	242206.8
10	937.5	74	100			233250
11	937.5	74	100	283	24	241400.4
12	937.5	74	100			233250
13	937.5	74	100	257	24	240651.6
14	937.5	74	80			203250
15	937.5	74	80	145	24	207426
16	937.5	74	80			203250
17	937.5	74	80	120	24	206706
18	937.5	74	80			203250
19	937.5	74	80	120	24	206706
20	937.5	74	80			203250
21	937.5	74	80	120	24	206706
22	937.5	74	80			203250
23	937.5	74	80	120	24	206706
roof	937.5	74	20			113250
					kips=	4638.396

CONCLUSION

From the analysis of Three PNC Plazas existing structural systems it can be concluded that it can adequately withstand the applied loads. The gravity system of the building was tested by spot checks of the composite slab, beams, and girders that showed they could all care they loads being applied. Also, the beams and girders were under the required construction and serviceability deflections.

The lateral forces due to wind and seismic were analyzed using ASCE 7-10. The North-South wind load was found to produce a base shear of 2814 kips and a overturning moment of 512289 k-ft. The East-West load was found to produce a base shear of 795 kips and overturning moment of 123470 k-ft. The seismic analysis produced results of a base shear of 2301kips and an overturning moment of 49067 k-ft. From these results it can be concluded that the North-South wind induces the largest base shear and overturning moment.

The loads used during the design of the building do not vary greatly for the design loads found in ASCE 7-10. However, the spot checks produced results that made the beams and girders seem over designed. This could be due to specific loading that was not specified on the floor plans of the structure documents. One of these loads could be due to the Reed Smith law firm occupying the building and would need to account for large filing cabinet systems that would produce large live loads.

APPENDIX

APPENDIX A: WIND

Typical Wind Hand Calculation

Risk Category: III
Basic Wind Speed: $V = 120$ mph
Directionality Factor: $K_d = 0.85$
Exposure Category: B, urban
Topographic Factor: $K_{zt} = 1.0$

Gust Factor:

Cannot assume rigid, cannot use Approximate Natural Frequency due to $319 > 300$

Section 26.9.5:

$$g_Q = 3.4 \qquad g_v = 3.4$$

$$g_R = \sqrt{2\ln(3600\, n_1)} + \frac{0.577}{\sqrt{2\ln(3600\, n_1)}}, \text{ where: } n_1 = \frac{1}{T_a} \qquad T_a = \text{natural period}$$

Sec 12.8.2.1: $T_a = C_t h_n^x$ $\qquad C_t = 0.02 \qquad x = 0.75$

$$T_a = 0.02(319)^{0.75} \qquad h_n = 319 \text{ main roof height}$$
$$= 1.5096$$

$$n_1 = \frac{1}{1.5096} = 0.6624$$, less than 1.0, Flexible assumption was correct

Sec 27:
$$g_R = \sqrt{2\ln(3600 \cdot 0.6624)} + \frac{0.577}{\sqrt{2\ln(3600 \cdot 0.6624)}} = \underline{4.09}$$

$$I_z = c\left(\frac{33}{\bar{z}}\right)^{1/6} \text{ where } \bar{z} = 0.6(\text{height}) \qquad 0.6(319) = 191.4$$
$$c = 0.30$$

$$= 0.30\left(\frac{33}{191.4}\right)^{1/6} = \underline{0.2238}$$

$$L_z = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} \text{ where } \ell = 320$$
$$\bar{\epsilon} = 1/3.0$$

$$= 320\left(\frac{191.4}{33}\right)^{1/3} = \underline{574.945}$$

$$Q = \frac{1}{\sqrt{1 + 0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \frac{1}{\sqrt{1 + 0.63\left(\frac{297+319}{574.945}\right)^{0.63}}} = \underline{0.7766}$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} \quad , \quad N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} \quad , \quad \bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} \left(\frac{88}{66}\right) V$$

Solve $\bar{V}_{\bar{z}}$: $\bar{\alpha} = 1/4.0 \quad \bar{b} = 0.45$

$$\bar{V}_{\bar{z}} = 0.45\left(\frac{191.4}{33}\right)^{1/4}\left(\frac{88}{66}\right) \cdot 120 = 111.735$$

Solve N_1 : $\frac{0.6624(574.945)}{111.735} = 3.408$

Solve R_n : $\frac{7.47(3.408)}{(1 + 10.3(3.408))^{5/3}} = 0.06455$

R_L : $\eta = 15.4 n_1 L / \bar{V}_z = 15.4(0.6624)(122.75) / 111.735 = 11.2066$

$$R_L = \frac{1}{11.2066} - \frac{1}{2(11.2066)^2}\left(1 - e^{-2(11.2066)}\right) = \underline{0.08525}$$

R_B : $\eta = 4.6 n_1 B / \bar{V}_{\bar{z}} = 4.6(0.6624)(297) / 111.735 = 8.0993$

$$R_B = \frac{1}{8.0993} - \frac{1}{2(8.0993)^2}\left(1 - e^{-2(8.0993)}\right) = \underline{0.1158}$$

R_h : $\eta = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.6624)(319) / 111.735 = 8.6992$

$$R_h = \frac{1}{8.6992} - \frac{1}{2(8.6992)^2}\left(1 - e^{-2(8.6992)}\right) = \underline{0.1083}$$

* β = 1.5% for steel and concrete buildings

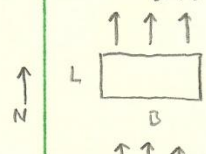
$$R = \sqrt{\frac{1}{0.015}(0.06455)(0.1083)(0.1158)(0.53 + 0.47(0.08525))}$$

$$= \underline{0.1754}$$

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right) = 0.8231$$

Enclosure Classification: Enclosed, $GC_{pi} = \pm 0.18$

$L/B = \frac{122.75}{297} = 0.41$ Windward Wall 0.8
Leeward Wall -0.5



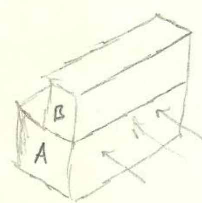
$p = qG_f C_p - q_i(GC_{pi})$ $K_z = 1.3728$

$q = 0.00256(1.3728)(1.0)(.85)(120)^2$
$= 43.0159$

windward $p = 28.325 - 28.325(-.18) = 33.424$ psf

leeward $p = -14.1625 - 5.0985 = -19.261$ psf

* These values are off from spread sheet values due to taking into account shape of buildings



Spread sheet seperates building into two parts to get a more accurate analysis.

Wind Load Design Criteria	
Design Wind Speed, V	120
Directionality Factor, K_d	0.85
Exposure	B
Topographic Factor, k_{zt}	1
Mean Roof Height	319
K_h	1.373
q_h	43.02

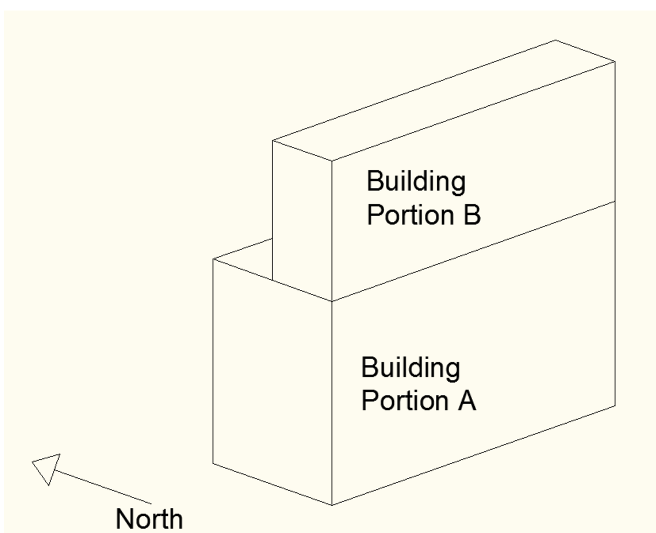
	Building Dimensions			
	Portion A		Portion B	
	N-S Direction	E-W Direction	N-S Direction	E-W Direction
B	297.00	122.75	297.00	62.50
L	122.75	297.00	62.50	297.00
h	188.00	188.00	342.00	342.00

B=Normal to wind direction

L= Parallel to wind direction

h= Height

Velocity Pressure Coefficients			
Story	**Height (ft)**	**k_z**	**q_z**
Ground	0	0.5700	17.8606
1mezz	12.5	0.5700	17.8606
2	24.0	0.6520	20.4300
2mezz	37.5	0.7450	23.3441
3	51.0	0.8140	25.5062
4	64.5	0.8680	27.1983
5	78.0	0.9220	28.8903
6	91.5	0.9645	30.2220
7	105.0	1.0025	31.4127
8	118.5	1.0363	32.4703
9	132.0	1.0700	33.5278
10	145.5	1.1010	34.4992
11	159.0	1.1280	35.3452
12	173.5	1.1570	36.2539
13	188.0	1.1820	37.0373
14	203.0	1.2048	37.7517
15	214.5	1.2232	38.3282
16	226.0	1.2416	38.9048
17	237.5	1.2600	39.4813
18	249.0	1.2784	40.0579
19	260.5	1.2947	40.5686
20	272.0	1.3108	41.0731
21	283.5	1.3269	41.5776
22	295.0	1.3430	42.0821
23	306.5	1.3578	42.5458
Roof Main	319.0	1.3728	43.0159
Roof High	342.0	1.4004	43.8807



North/South

Story	Height (ft)	kz or kh	qz	Windward (psf)	Windward (plf)	Windward (kips)	Leeward (psf)	Leeward (plf)	Leeward (kips)	Story Force (kips)	Moment (k-ft)
Building Portion A											
1mezz	12.5	0.5700	17.8606	19.5169	2395.7009	15.0187	-25.9635	-3187.0236	-19.1221	34.1409	426.7607
2	24.0	0.6520	20.4300	21.3432	2619.8791	15.7297	-25.9635	-3187.0236	-19.9189	35.6486	855.5671
2mezz	37.5	0.7450	23.3441	23.3311	2863.8866	18.5077	-25.9635	-3187.0236	-21.5124	40.0201	1500.7544
3	51.0	0.8140	25.5062	24.7911	3043.1112	19.9361	-25.9635	-3187.0236	-21.5124	41.4485	2113.8749
4	64.5	0.8680	27.1983	25.9259	3182.4053	21.0111	-25.9635	-3187.0236	-21.5124	42.5235	2742.7675
5	78.0	0.9220	28.8903	27.0530	3320.7594	21.9482	-25.9635	-3187.0236	-21.5124	43.4606	3389.9260
6	91.5	0.9645	30.2220	27.9364	3429.1990	22.7811	-25.9635	-3187.0236	-21.5124	44.2935	4052.8570
7	105.0	1.0025	31.4127	28.7231	3525.7563	23.4730	-25.9635	-3187.0236	-21.5124	44.9854	4723.4653
8	118.5	1.0363	32.4703	29.4190	3611.1791	24.0872	-25.9635	-3187.0236	-21.5124	45.5996	5403.5486
9	132.0	1.0700	33.5278	30.1130	3696.3715	24.6630	-25.9635	-3187.0236	-21.5124	46.1754	6095.1518
10	145.5	1.1010	34.4992	30.7484	3774.3699	25.2138	-25.9635	-3187.0236	-21.5124	46.7262	6798.6565
11	159.0	1.1280	35.3452	31.2996	3842.0277	26.6658	-25.9635	-3187.0236	-22.3092	48.9750	7787.0272
12	173.5	1.1570	36.2539	31.8905	3914.5583	28.1176	-25.9635	-3187.0236	-23.1059	51.2235	8887.2852
13	188.0	1.1820	37.0373	32.3975	3976.7871	29.1032	-25.9635	-3187.0236	-23.5043	52.6075	9890.2146
Building Portion B											
14	203.0	1.2048	37.7517	32.8887	2055.5468	20.8226	-25.9635	-1622.7208	-16.6167	37.4393	7600.1798
15	214.5	1.2232	38.3282	33.2600	2078.7501	11.8861	-25.9635	-1622.7208	-9.3306	21.2167	4550.9924
16	226.0	1.2416	38.9048	33.6309	2101.9298	12.0195	-25.9635	-1622.7208	-9.3306	21.3501	4825.1224
17	237.5	1.2600	39.4813	34.0014	2125.0878	12.1527	-25.9635	-1622.7208	-9.3306	21.4833	5102.2885
18	249.0	1.2784	40.0579	34.3716	2148.2256	12.2858	-25.9635	-1622.7208	-9.3306	21.6164	5382.4887
19	260.5	1.2947	40.5686	34.6980	2168.6279	12.4110	-25.9635	-1622.7208	-9.3306	21.7416	5663.6864
20	272.0	1.3108	41.0731	35.0201	2188.7560	12.5275	-25.9635	-1622.7208	-9.3306	21.8581	5945.4095
21	283.5	1.3269	41.5776	35.3419	2208.8698	12.6432	-25.9635	-1622.7208	-9.3306	21.9738	6229.5776
22	295.0	1.3430	42.0821	35.6635	2228.9703	12.7588	-25.9635	-1622.7208	-9.3306	22.0894	6516.3832
23	306.5	1.3578	42.5458	35.9581	2247.3789	13.4313	-25.9635	-1622.7208	-9.7363	23.1677	7100.8918
Roof Main	319.0	1.3728	43.0159	36.2556	2265.9729	20.0524	-25.9635	-1622.7208	-14.4016	34.4541	10990.8421
Roof High	342.0	1.4004	43.8807	36.8025	2300.1586	13.0293	-25.9635	-1622.7208	-9.3306	22.3600	7647.1162
									Sum=	908.58	142222.84

East/West

Story	Height (ft)	kz or kh	qz	Windward (psf)	Windward (plf)	Windward (kips)	Leeward (psf)	Leeward (plf)	Leeward (kips)	Story Force (kips)	Moment (k-ft)
Building Portion A											
1mezz	12.5	0.5700	17.8606	20.4289	6067.3778	37.9456	-19.1592	-5690.2840	-34.1417	72.0873	901.0911
2	24.0	0.6520	20.4300	22.2340	6603.4889	39.7305	-19.1592	-5690.2840	-35.5643	75.2948	1807.0743
2mezz	37.5	0.7450	23.3441	24.2069	7189.4494	46.5512	-19.1592	-5690.2840	-38.4094	84.9606	3186.0219
3	51.0	0.8140	25.5062	25.6370	7614.1839	49.9623	-19.1592	-5690.2840	-38.4094	88.3717	4506.9556
4	64.5	0.8680	27.1983	26.7385	7941.3267	52.4998	-19.1592	-5690.2840	-38.4094	90.9093	5863.6476
5	78.0	0.9220	28.8903	27.8417	8268.9734	54.7098	-19.1592	-5690.2840	-38.4094	93.1192	7263.2960
6	91.5	0.9645	30.2220	28.6985	8523.4453	56.6744	-19.1592	-5690.2840	-38.4094	95.0838	8700.1705
7	105.0	1.0025	31.4127	29.4610	8749.9066	58.2976	-19.1592	-5690.2840	-38.4094	96.7070	10154.2329
8	118.5	1.0363	32.4703	30.1345	8949.9533	59.7370	-19.1592	-5690.2840	-38.4094	98.1464	11630.3537
9	132.0	1.0700	33.5278	30.8101	9150.6058	61.0894	-19.1592	-5690.2840	-38.4094	99.4988	13133.8421
10	145.5	1.1010	34.4992	31.4285	9334.2553	62.3864	-19.1592	-5690.2840	-38.4094	100.7958	14665.7923
11	159.0	1.1280	35.3452	31.9628	9492.9571	65.9151	-19.1592	-5690.2840	-39.8320	105.7471	16813.7840
12	173.5	1.1570	36.2539	32.5382	9663.8386	69.4434	-19.1592	-5690.2840	-41.2546	110.6979	19206.0932
13	188.0	1.1820	37.0373	33.0296	9809.8005	71.8182	-19.1592	-5690.2840	-41.9658	113.7840	21391.3941
Building Portion B											
14	203.0	1.2048	37.7517	34.2302	10166.3621	66.0150	-15.4056	-4575.4766	-34.4931	100.5081	20403.1450
15	214.5	1.2232	38.3282	34.5769	10269.3271	58.7526	-15.4056	-4575.4766	-26.3090	85.0616	18245.7126
16	226.0	1.2416	38.9048	34.9253	10372.8211	59.3462	-15.4056	-4575.4766	-26.3090	85.6552	19358.0677
17	237.5	1.2600	39.4813	35.2753	10476.7752	59.9426	-15.4056	-4575.4766	-26.3090	86.2516	20484.7503
18	249.0	1.2784	40.0579	35.6267	10581.1311	60.5415	-15.4056	-4575.4766	-26.3090	86.8505	21625.7673
19	260.5	1.2947	40.5686	35.9338	10672.3338	61.1037	-15.4056	-4575.4766	-26.3090	87.4127	22771.0089
20	272.0	1.3108	41.0731	36.2377	10762.6048	61.6254	-15.4056	-4575.4766	-26.3090	87.9344	23918.1674
21	283.5	1.3269	41.5776	36.5427	10853.1930	62.1454	-15.4056	-4575.4766	-26.3090	88.4544	25076.8250
22	295.0	1.3430	42.0821	36.8487	10944.0638	62.6671	-15.4056	-4575.4766	-26.3090	88.9761	26247.9507
23	306.5	1.3578	42.5458	37.1275	11026.8762	65.9232	-15.4056	-4575.4766	-27.4529	93.3760	28619.7536
Roof Main	319.0	1.3728	43.0159	37.4082	11110.2314	98.3428	-15.4056	-4575.4766	-40.6074	138.9502	44325.1053
Roof High	342.0	1.4004	43.8807	37.9270	11264.3208	63.8838	-15.4056	-4575.4766	-26.3090	90.1928	30845.9448
									Sum=	2444.83	441145.95

APPENDIX B: Seismic

Hand Calculations

11.4 Seismic Ground Motion Values

Site Class: D (provided)

$S_{MS} = F_a S_s$ $S_{M1} = F_v S_1$

$S_s = 0.201$
$S_1 = 0.118$ } From USGS Web site: earthquake.usgs.gov/designmaps

Table 11.4-1: $F_a = 1.6$
Table 11.4-2: $F_v = 2.328$ (interpolation used)

$S_{MS} = 1.6(0.201) = 0.3216$

$S_{M1} = 2.328(0.118) = 0.2747$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.3216) = 0.2144$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.2747) = 0.1831$

$I_e = 1.25$ due to Risk Category III

Sesmic Design Category: B

Equivalent Lateral Force Procedure:

$V = C_s W$

W = 53477 (calculated using excel)

$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.2144}{\left(\frac{6}{1.25}\right)}$ R = 6, Special reinforced concrete shear walls
↳ Special due to cast in place construction

$= 0.04467$

$V = 0.04467(55481) = 2301^K$

$T_a = 1.5096$ see wind calcs

12.8.3: Vertical Distribution of Seismic Forces: See Spread Sheets

$F_x = C_{vx} V$ $C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ K = 1.2410

Floor Weight Calculations (MEP, Decking, and Curtain Wall)

Floor	Area (sf)	Perimeter (ft)	Composite Deck Weight (psf)	M.E.P Weight (psf)	Curtain Wall Weight (plf)	Total Weight from Decking, MEP, and Curtain Wall (lbs)
1st Floor Mezz.	15165	831.35	44	30	60	1172091
2nd Floor	31420.5	831.35	44	30	60	2374998
2nd Floor Mezz.	17225.5	831.35	44	30	60	1324568
3rd Floor	31338	831.35	44	30	60	2368893
4th Floor	28987.5	780	44	30	60	2191875
5th Floor	28987.5	780	44	30	60	2191875
6th Floor	28987.5	780	44	30	60	2191875
7th Floor	28987.5	780	44	30	60	2191875
8th Floor	28987.5	780	44	30	60	2191875
9th Floor	28987.5	780	44	30	60	2191875
10th Floor	28987.5	780	44	30	60	2191875
11th Floor	28987.5	780	44	30	60	2191875
12th Floor	28987.5	780	44	30	60	2191875
13th Floor	24525	750	44	30	60	1859850
14th Floor	16875	685	44	30	60	1289850
15th Floor	16875	685	44	30	60	1289850
16th Floor	16875	685	44	30	60	1289850
17th Floor	16875	685	44	30	60	1289850
18th Floor	16875	685	44	30	60	1289850
19th Floor	16875	685	44	30	60	1289850
20th Floor	16875	685	44	30	60	1289850
21st floor	16875	685	44	30	60	1289850
22nd Floor	16875	685	44	30	60	1289850
23rd Floor	16875	685	44	30	60	1289850
Main Roof	16875	685	44	30	60	1289850
High Roof	2112.5	613.9	44	30	60	193159
					sum=	43208784.0

Typical Floor Weight Calculation for Floor Beam Weights

4th, 5th, 6th, and 7th Floor Beams				
Beam	# of Beams	Length (ft)	Unit Weight (lbs/ft)	Weight (lbs)
W21x44	50	42.5	44	93500
W21x166	35	42.5	166	246925
W24x55	1	42.5	55	2337.5
W21x73	2	42.5	73	6205
W24x68	1	35	68	2380
W13x35	2	35	35	2450
W21x57	1	20	57	1140
W21x57	1	15	57	855
W21x22	7	10	22	1540
W12x16	15	20	16	4800
W12x22	4	20	22	1760
W18x46	1	20	46	920
W21x73	2	30	73	4380
W21x50	16	30	50	24000
W21x62	1	15	62	930
W21x62	2	30	62	3720
W16x26	1	15	26	390
W16x26	2	30	26	1560
W21x44	2	10	44	880
W21x166	2	20	166	6640
W21x44	5	2.5	44	550
W21x73	1	20	73	1460
W21x73	4	25	73	7300
W8x15	3	5	15	225
			Sum=	416847.5

Typical Floor Weight Calculation for Column Weight

2nd Floor Column Weight				
Column	Number of Columns	Height	Unit Weight	Weight (lbs)
W10x49	4	13.5	49	2646
W12x65	4	13.5	65	3510
W14x455	1	13.5	455	6142.5
W14x550	5	13.5	550	37125
W14x233	7	13.5	233	22018.5
W14x426	2	13.5	426	11502
W14x730	5	13.5	730	49275
W14x455	1	13.5	455	6142.5
W14x605	4	13.5	605	32670
W14x370	2	13.5	370	9990
			sum=	181021.5

Seismic Calculation Table

Floor Level	Floor Height (ft)	Total Height (ft)	Weight (kips)	$w*h^k$	C_{VX}	f_i (kips)	V_i (kips)	M_z (k-ft)
Main Roof	23	319.0	1500	1919975	0.068	157	157	50172
23	12.5	306.5	1542	1878212	0.067	154	311	47157
22	11.5	295.0	1541	1789995	0.064	147	458	43256
21	11.5	283.5	1541	1703809	0.061	140	597	39568
20	11.5	272.0	1547	1624764	0.058	133	730	36202
19	11.5	260.5	1547	1539954	0.055	126	857	32862
18	11.5	249.0	1549	1457924	0.052	119	976	29738
17	11.5	237.5	1549	1374833	0.049	113	1089	26748
16	11.5	226.0	1557	1299383	0.046	106	1195	24056
15	11.5	214.5	1557	1217839	0.043	100	1295	21399
14	11.5	203.0	1842	1345526	0.048	110	1405	22375
13	15	188.0	2207	1465657	0.052	120	1525	22572
12	14.5	173.5	2529	1520266	0.054	125	1650	21607
11	14.5	159.0	2483	1339401	0.048	110	1759	17445
10	13.5	145.5	2500	1207961	0.043	99	1858	14398
9	13.5	132.0	2500	1070464	0.038	88	1946	11575
8	13.5	118.5	2510	940066	0.033	77	2023	9125
7	13.5	105.0	2723	877695	0.031	72	2095	7549
6	13.5	91.5	2729	741527	0.026	61	2156	5558
5	13.5	78.0	2729	608265	0.022	50	2205	3887
4	13.5	64.5	2739	482232	0.017	40	2245	2548
3	13.5	51.0	2726	358608	0.013	29	2274	1498
2mezz	13.5	37.5	1665	149550	0.005	12	2287	459
2	13.5	24.0	2754	142169	0.005	12	2298	280
1mezz	11.5	12.5	1452	33360	0.001	3	2301	34
Ground	11.5	0.0	-	0	0.000	0	2301	0
		Σ	51518	28089435	1	2301		492067

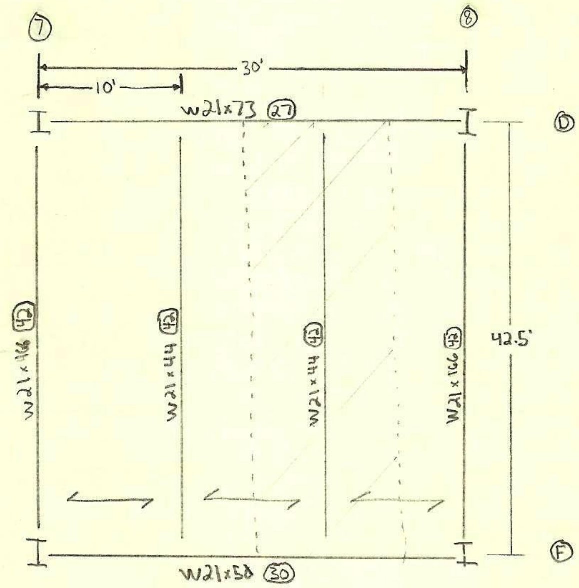
T=	1.510 s
k=	1.241
V_b=	2301 kips

APPENDIX C: Spot Checks

Decking, Fill Beam, Girder

Spot Checks

Floor 7 Typical Bay with Typical Floor



Composite Slab
2" x 18 G.A. Steel Deck } total 5 ½"
3 ½" Lightweight Concrete }
3000psi Fill

• Use Vulcraft Decking Catalog

2VL118: Max unshored clear span, 3 span = 12'-5"

10'-0" < 12'-5" ∴ OK ✓

Superimposed Live Load: 10'-0" span = 217 psf

Loading ASCE 7-10
Office Live Load = 50
Superimposed Dead Load = 30
80 psf

80psf < 217 psf ∴ OK ✓

Decking is ok, over designed for this loading. Not the controlling load factor.

Decking provides a 2 hr fire rating.

Composite Beam W21x44

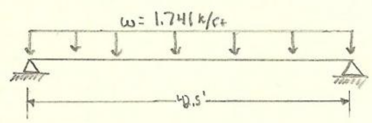
Properties: $A_g = 13.0$
$I_x = 843$
$F_y = 50$ ksi

LL = 50 psf not reducing to be conservative
DL = 44 psf (Decking)
SDL = 30 psf (Superimposed)
SW = 44 plf

$W_u = 1.2D + 1.6L$

Dead = (44 + 30) 10' + 44 = 784 plf
Live = 50 (10) = 500 plf

$W_u = 1.2(784) + 1.6(500) = 1740.8$ plf ≈ 1.741 K/ft



* pin supports assumed

$V_u = 1.741(42.5)\left(\frac{1}{2}\right) = 36.996^k \approx 37^k$

$M_u = 1.741(42.5)^2\left(\frac{1}{8}\right) = 393.085 \approx 393.1^{1k}$

$\phi V_n = 217$ K > 37K ∴ ok ✓

$b_{eff} = \min$ | Span/4 = 10.625'
| Spacing = 10' * controls

PNA = 7 $\Sigma Q_n = 162$

$a = \frac{\Sigma Q_n}{0.85(f'_c)(b_{eff})} = \frac{162}{0.85(3)(10 \cdot 12)} = 0.53$

since a = 0.53 which is < 1.0 treat a as 1.0

$Y_2 = \text{thickness}_{slab} - \frac{a}{2} = 5.5 - \frac{1}{2} = 5$

$\phi M_n = 516^{1k}$ which is larger then $M_u = 393.1$ ∴ ok ✓

$Q_n = \frac{162}{17.2} = 9.4 \Rightarrow 10$ studs required

Deflection

$\Delta_{LL} = \frac{l}{360} = \frac{42.5(12)}{360} = 1.4167$ in Max deflection

$\Delta_{LL} = \frac{5 w_{LL} l^4}{384 EI}$ $I_B = 1450$ $w_{LL} = (50 \cdot 10) = \frac{500}{1000} = 0.5$

$= \frac{5(0.5)(42.5)^4}{384(29000)(1450)}(1728) = 0.8728$ in

0.8728" < 1.4167" ∴ OK ✓

Deflection wet concrete

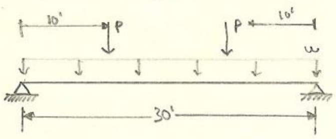
$$\Delta_{Max} = \frac{l}{240} = \frac{42.5(12)}{240} = 2.125 \text{ in}$$

$$I_{req} = \frac{5 w l^4}{384 \Delta_{max} E} \qquad w = \frac{(44 \cdot 10) + 44}{1000} = 0.484$$

$$= \frac{5(0.484)(42.5)^4}{384(2.125)(29000)}(1728) = 576.5 \text{ in}^4$$

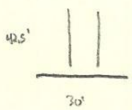
$576.5 < 843 \quad \therefore$ ok ✓

Composite Girder : W21 × 56



* Assumed Pin supports

Assume Load P = 37 k due to 42.5' beam



$w_u = 50/1000 = 0.05$ klf

$V_u = \frac{37}{2} + \frac{0.05(30)}{2} = 19.25^k$

$M_u = \frac{0.05(30)^2}{8} + 37(10) = 375.625^{1k}$

PNA = 7 $\quad \Sigma Q_n = 184 \quad b_{eff} = \begin{cases} 30/4 = 7.5 \text{ * controls} \\ \text{spacing} \end{cases} \quad a = \frac{184}{0.85(3)(7.5)(12)} = 0.8 < 1.0$

use $a = 1$

$Y_2 = 5.5 - 0.5 = 5 \quad \phi M_n = 592^{1k} > 375.625^{1k}$ ∴ ok ✓

$\phi V_n = 237^k > 19.25^k$ ∴ ok ✓

Deflection

$\Delta LL: \quad P_L = \frac{500(42.5)}{100} = 21.25$

$\Delta LL: \quad \frac{l}{360} = \frac{(30)(12)}{360} = 1$ in

$I_{LB} = 1680 \quad \Delta LL = \frac{Pl^3}{28EI} + \frac{5wl^4}{384EI} = \left[\frac{21.25(30)^3}{28(29000)(1680)} + \frac{5(.05)(30)^4}{384(29000)(1680)}\right]1728 = 0.75''$

$0.75'' < 1.0''$ ∴ OK ✓

Wet $\quad \Delta Max = \frac{30(12)}{240} = 1.5'' \quad P = 10.285$

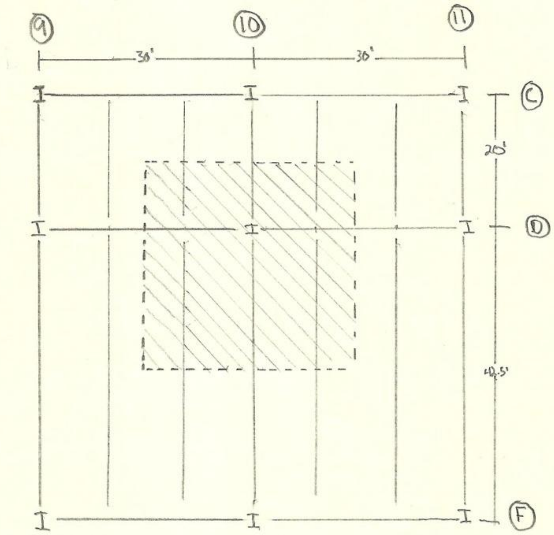
$I_{req} = \frac{(10.285)(30)^3(1728)}{28(29000)(1.5)} + \frac{5(0.05)(30)^4(1728)}{384(29000)(1.5)} = 250.8 \text{ in}^4$

$250.8 \text{ in}^4 < 984 \text{ in}^4$ ∴ OK ✓

Column Tributary Area

Columns

Looking at Column
D 10



Tributary Area
Floor 3 - Roof

Trib Area: $\left(\frac{42.5}{2} + \frac{20}{2}\right)(30) = 937.5$

LL:

Floors 3-7 : 80 psf
8-13 : 100 psf
14-23 : 80 psf

APPENDIX D: Snow Loads

Snow Load

7.3 Flat roof snow load

$p_f = 0.7 C_e C_t I_s p_g$ where $C_e = 0.9$ Exposure B, fully exposed roof

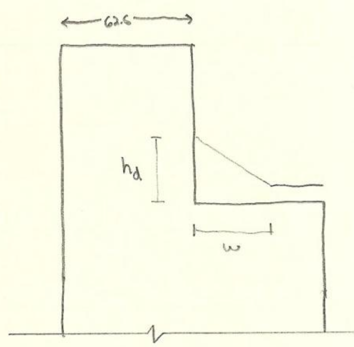
$C_t = 1.0$

$I_s = 1.1$

$p_g = 25$

$p_f = 0.7(0.9)(1.0)(1.1)(25) = 17.325$ lb/ft²

Building Layout



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

$= 0.43 \sqrt[3]{62.5}\,\sqrt[4]{25+10} - 1.5$

$= 2.65'$

$w = 4h_d = 10.6'$

$p_d = h_d \gamma$ $\quad \gamma = 0.13 p_g + 14$

$\gamma = 0.13(25) + 14$

$= 17.25$

Drift: $p_d = 2.65(17.25) = 45.71$ lb/ft²

APPENDIX D: Shear wall reinforcing

SHEARWALL REINFORCING SCHEDULE

LEVEL	ENDWALL/ INTERSECTION	WALL
B3 - OG	20 - #11 VERT.	#11 @ 12" VERT. #6 @ 12" HORIZ.
OG - 3	20 - #10 VERT.	#10 @ 12" VERT. #6 @ 12" HORIZ.
3 - 5	14 - #10 VERT.	#10 @ 12" VERT. #6 @ 12" HORIZ.
5 - 7	14 - #10 VERT.	#9 @ 12" VERT. #6 @ 12" HORIZ.
7 - 9	14 - #10 VERT.	#9 @ 12" VERT. #5 @ 12" HORIZ.
9 - 11	14 - #9 VERT.	#8 @ 12" VERT. #5 @ 12" HORIZ.
11 - 13	14 - #9 VERT.	#8 @ 12" VERT. #5 @ 12" HORIZ.
13 - 14	14 - #9 VERT.	#8 @ 12" VERT. #5 @ 12" HORIZ.