Washington Park Condominiums
Mt. Lebanon, Pennsylvania

Technical Report #1
Structural Concepts/Existing Conditions Report

Architectural Engineering
2008-2009 Senior Thesis

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September 29, 2008

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Executive Summary:

The purpose of this report is to examine the structural concepts and existing conditions of Washington Park Condominiums and to use those concepts and conditions to interpret the structural design of the building. In more detail the report discusses the structural conditions of Washington Park Condominiums through a comprehensive description of the foundation, floor, gravity and main lateral systems. This detail is achieved by completing a summary of all relevant building codes and materials used in the building as well as using ASCE 7-05 to identify the dead, live, snow, wind and gravity loads that the structure is subjected to. The gravity loads in the building were determined by the design documents, specifications and drawings given to me by the design professional. Also, a more detailed breakdown of the overall weight of the building was completed for use in both dead load gravity calculation and seismic analysis.

The wind loads in the building were analyzed using Method 2 – Analytical Procedure that can be found in section 6.5 of ASCE 7-05. During the wind analysis, I chose to add a third wind direction since a large section of the building extends at an angle from the main rectangular portion of the structure. After completing the analysis I concluding that the controlling direction for wind was the East – West direction with a base shear of 493.889 k and an overturning moment of 23767.689 k-ft.

The seismic loading was analyzed using the equivalent lateral found in Chapter 12 of ASCE 7-05. I decided to split the floors into separate categories based on dead load similarities. After calculating these loads, I was able to determine the entire seismic weight of the building. Before I could complete the seismic analysis I had to determine which value for R should be used for my structure. Since there are two different types of construction in my building there would be two different values for R that could be used. I decided to use an R value of 3.5 because in doing so it allowed for the worst case Seismic Response Coefficient and therefore resulted in a more conservative base shear. My final calculations gave me a design base shear of 248.325 k. In the end, I determined that wind would control the lateral design, however I believe more detailed research needs to be done since the seismic base shear fell between the calculated base shears for the North – South and East – West directions.

For my spot checks I decided to check my composite floor system, a beam that supports my floors system and one of the columns that transfers the load down to the foundations. From the VESCOM design tables I was able to verify the floor design used by the structural engineers. I was also able to verify the sizes of the composite beam and column using the AISC Steel Construction Manual, 13th edition. However, since I chose an external bay for my spot checks, the size of the members used was higher than what I determined. I gather this is because these members are part of the moment frame system and therefore are designed to carry lateral loads.

Introduction:

Washington Park Condominiums is a multi-use retail and residential building located at the intersection of Bower Hill Road and Washington Road in Mt. Lebanon, Pennsylvania. Site work and excavation has begun at the site and construction should begin sometime before the end of the Fall 2008, with the project lasting until Fall 2010. Washington Park Condominiums is the first of two buildings proposed to be built on the site. Building One is a nine-story, 148,000 ft^2 structure which is owned by Zamagias Properties of Pittsburgh, PA. The building was architecturally designed by Indovina Associates Architects and is being constructed by PJ Dick, Inc. for a price of $23,418,000. The building's primary use is residential and it contains 7 stories of condominiums on the 2nd through 8th floors. The first floor of the building is used for retail space and as a location for extra amenities for the residents of Washington Park. The building also contains two below grade levels of parking. The enclosed parking garage contains 78 parking spaces that can be used by the residents. Two elevators and two stairs serve the parking areas that also contain resident storage, a wine room and trash collection along with mechanical and electrical rooms. The ground floor serves primarily as retail space with four separate areas available for possible tenants. Also contained on the floor are a resident exercise room and a private entrance and lobby for the residents.

As the building moves to the second floor, the function changes from primarily retail to one of solely residential with six upscale condominiums located on the floor. These condominiums each have different floor plans and layouts with overall areas ranging from 1523 ft^2 to 2288 ft^2. Each unit contains two or three bedrooms and bathrooms depending on size, along with a living room, dining room, kitchen, study, laundry, entry and in some cases a balcony. This floor layout continues throughout the next four floors, with a total of 30 units on floors 2 through 6. The 7th and 8th floors of the building are the penthouse level. This floor contains five condominiums that range from 1732 ft^2 to 2453 ft^2. These units contains the same amenities and spaces as the units on the below floors do. All of the condominiums floors are served by two elevators and two stairways that are connected by a hallway that runs through the center of the building in the long direction. Finally, the roof contains mechanical spaces that are accessed by using the northern most stairway or elevator.

The typical exterior wall system of the building consists mainly of 4" brick veneer backed by a 2" airspace and 2" of rigid XPS insulation, then containing another 2" layer of rigid spray-foam insulation that is followed by an airspace and then 5/8" gypsum board. This exterior wall system is typical for the first 6 floors of the building. The 7th and 8th floors of the building consist of a similar wall construction except for the exterior façade which is a 5/16" layer of painted fiber-cement siding.

Structural Systems:

Foundations

The foundation system can be best described as a spread footing system with attached concrete piers. The sizes for the spread footings range from the smallest, a 4'-0" x 4'-0" x 2'-0" footing with #8 @ 12" each way, to a 14'-0" x 14-0" x 3'-6" footing with #8 @ 6" each way. These spread footings have a concrete strength of f'_c = 4000 psi, and the deepest of the footings will be 25'-0" below grade. In addition to the spread footings, interior and exterior wall footings were used and are either 2'-0" or 3'-0" wide by 1'-4" deep. The steel reinforcing in these wall footings are (3) #5 continuous bars and #5 x 1'-8" @ 16".

Although a typical spread footing foundation system was used, there were some interesting design issues that needed to be determined because of the presence of the sub grade parking garage. The garage is two stories deep with soil pushing on one side of the building without the benefit of soil on the other side pushing in the opposite direction. As a result of this, the floor system below grade as well as the slab on grade is used to help tie the various foundations together. This also helped resist sliding and overturning forces caused by the imbalance of soil pressure. Finally, some of the foundations were inserted directly into the rock and utilized rock skin friction to resist up-lift as well as increasing the sliding resistance of the foundation system.

The slab on grade in this system consist of either a 6" or 8" normal weight concrete slab reinforced with 6x6-W2.9xW2.9 welded wire fabric or 6x6-W4xW4 welded wire fabric. The slab on grade is also thickened to a minimum if 1'-0" at non-load bearing walls and (2) #4 bars are added for tensile strength. Connecting the columns to the slab on grade and the spread footings are column piers that range from 16" x 16" with (4) #7 of vertical reinforcement to 40" x 40" w/ (12) #7 of vertical reinforcement. The slab on grade and column piers are both f'_c = 4000 psi concrete.

Floor Systems

There are two separate floors systems that are typical within the structure of Washington Park. The first is a precast concrete plank system that is used in the parking areas as well as the first and second floor framing. The precast concrete plank is 8" thick and also contains a 2" thick structural topping. The reinforcing in the structural topping is 6x6-W1.4xW1.4 welded wire fabric. The precast concrete plank system bears on W shapes which then carry the load to the columns. This system was used in the parking areas because of the systems diaphragm capacity (ability to transfer horizontal loading) and because of its durability and strength. Moreover, the system was utilized on the 1st and 2nd floors due to its weight. It was also useful because of the contractors need to backfill the building early on the meet the owner's schedule.

The second primary floor system in the building is the VESCOM composite joist floor system. The composite joist system interlocks the top chord of a joist with the concrete producing less deflection, less vibration and greater stiffness. The floor construction consists of a 2 11/16" reduced weight concrete slab that is poured on top of the 1 5/16", 22 Gage galvanized floor decking. The bottom chord acts as the main tension member, and in the composite stage the embedded top chord serves as a continuous shear connection. The concrete is also reinforced with welded wire fabric and compressive strength of the concrete is f'_c = 3500 psi. These floors are also to act as a diaphragm that is able to resist lateral forces of 250 to 350 PLF. Finally, the system was used as an architectural element since the ceiling could be installed directly to the joist bottom chord and the mechanical systems (HVAC, plumbing, fire protection, electrical and telecommunications) could be installed with the joist system, saving space and allowing for higher ceilings and floor to floor height within the apartments. A section of the VESCOM Composite floor system can be found in Appendix B.

Roof System

The flat roof system of the building is similar to floors 3 through 6 in that it also utilizes the VESCOM composite floor system. The only main difference in the system is that the concrete is 4 11/16" thick. The pitched roof is framed with a typical pre-engineered light gauge roof truss system spaced at 2'-0" on center. This roof is topped with asphalt shingles and ½" cement bonded particle board. Finally, the roof of the stair towers are framed with light gauge purlins, topped with roof insulation and shingles.

Lateral System

The lateral resisting system within the building is mainly moment resisting steel frames. These frames begin on the second floor and continue up through the top of the building. These frames run in the north-south direction and run along column lines A, B, C and D. Rigid connections also occur on these floors along column lines 1 through 9. Since the VESCOM floor system is being used as a diaphragm to transfer shear loading the load path begins at the exterior beams and then continue on through the floor system to joist girders which are to be designed and manufactured by the joist manufacturer. The load is then transferred into the large W14 columns, and finally to the brace frames. There are a total of eleven braced frames located in the basement and sub-basement levels running along column lines 1 through 11 from column lines A.1 to B. The brace frames are 17'-2" in length and they begin at the sub-basement level and connect into the framing for the ground floor. The bracing in the frames consists of HSS 8x8x1/2 up to the basement level, and HSS 6x6x3/8 from the basement level to the ground floor. These frames are used in conjunction with the precast plank system to create a diaphragm that can resist 280 PLF. This plan detail and the detail of the brace frames can be found in Appendix B.

Columns

The columns in Washington Park Condominiums have all been designed using AISC 9th Ed. ASD and are ASTM A992 Grade 50 wide flange columns. The columns are spaced at 27'-8" or 28'-0" in the north-south direction and 17'-2" or 28'-4" in the east-west direction. The columns at the base of the structure that run the entire height of the building range from W12x96 to W14x193 at the bottom to W12x40 and W14x74 at the top of the structure.

Applicable Codes and Design Requirements:

Design Standards

International Building Code 2003 w/ Amendments for Mt. Lebanon

ACI 318-02 (Reinforced Concrete Design)

AISC 9th Edition ASD (Structural Steel Design)

ACI 530-02 (Masonry Design)

ASCE 7-05

Deflection Criteria

Floor Deflection Criteria

L/240 Total Load and L/360 Live Load Deflections

L/360 Gravity Loads on Exterior Stud Walls

L/600 Gravity Loads on Masonry/Veneer Backup Walls

Lateral Deflection Criteria

H/500 Total Permitted Wind Drift

H/400 Story Wind Drift

H/600 Total Allowable Seismic Drift

Materials:

Structural Materials		
Concrete	Column Footings	f'c = 4000
	Wall Footings	f'c = 4000
	Foundation Walls and Column Piers	f'c = 4000
	Pit Walls and Slabs	f'c = 4000
	VESCOM Floor Slab	f'c = 3500
	Roof Slab	f'c = 4000
Reinforcing Steel	Rebar	ASTM A615 Gr. 60
	Welded Wire Reinforcement	ASTM A185
	Rebar to be Welded	ASTM A706
Structural Steel	All Beams (W-shape)	ASTM A992 Gr. 50
	All Girders	ASTM A992 Gr. 50
	All Columns	ASTM A992 Gr. 50
	Base Plates	ASTM A36
	Bolts	ASTM A325
	Electrodes	AWS D1.1 and D1.4
	Anchor Bolts	ASTM F1554 Gr. 36
Metal Deck/Studs	Typical Composite Floor Deck	2"-20 Gage Composite Galvanized Metal Floor Deck
	Composite Shear Studs	3/4" ϕ x 4.5" Headed
	Metal Roof Deck	1½" - 20 GAGE Type B wide rib galvanized metal roof deck.
Masonry	Typical Prism Strength	f'_m = 2000 psi
	Grout Compressive Strength	f'_c = 2000 psi
	Mortar Type	Type S
	Ivany Block	f'_m = 2000 psi, f'c = 3000 psi (grout), Type M Mortar

Gravity Loading:

To begin the process of analyzing Washington Park Condominiums, the gravity loads acting on the structure needed to be determined. The information concerning the dead loads was found in the design documents, specifications and drawings given to me by the design professional. The specific calculation for the dead loads that are used for the total seismic weight of the building can be found in Appendix D. These loads were calculated by using the weight found either in the specifications or the code and were multiplied by the square footage of the floor. The weight for each level was calculated precisely by counting the steel beams and columns and then multiplying them by their length and weight per linear foot. The live loads for the building were determined using the information provided to me by the design professionals and by using the live load tables in ASCE 7-05 Chapter. 4.

Dead Load Table			
Floor Dead Load		Roof Dead Load	
Material/System	Load	Material/System	Load
Normal Weight Concrete	145 pcf	4 11/16" RWC Slab on 1 5/16" FLR Deck	68 psf
Steel	Per Shape	MEP	6 psf
Brick Veneer w/ studs	40 psf	Sprinklers	3 psf
8" P/C Plank w/ 2" Structural Topping	90 psf	Ceiling	8 psf
		VESCOM Joists	4 psf
MEP	6 psf	Asphalt Shingles/Felts	4 psf
Sprinklers	3 psf	1/2" Cement Bonded Particle Board	5 psf
Ceiling	5 psf	Light Gauge Roof Trusses @ 2'-0" O.C.	4 psf
Floor Finishes	5 psf		
Partitions	20 psf		
VESCOM Joists	4 psf		
2 11/16" RWC Slab on 1 5/16" FLR Deck	43 psf		

Live Load Table				
Floor Live Load Table			Roof Live and Snow Load Table	
Occupancy	Load		Material/System	Load
Typ. Condominium Floor	40 psf		Roof Live Load	20 psf
Stairs	100 psf		Roof Live Load (Mechanical)	150 psf
First Level (Plaza and Traffic/Parking Areas	250 psf		Ground Snow Load (P_g)	25 psf
First Level (Non-Plaza Areas)	100 psf		Flat Roof Snow Load (P_f)	23.1 psf
Basement Level Parking Areas/Ramps	50 psf		Exposure Factor (C_e)	1.2
Slabs-on-Grade	150 psf		Thermal Factor (C_t)	1
Exercise Area at Ground Floor	150 psf		Importance Factor (I)	1.1
Corridors On 1st Floor	100 psf		Terrain Category	B
Corridors Above 1st Floor	80 psf			
Mech/Elec Spaces	150 psf			
Second Floor Terrace	100 psf			
Apartment Balconies	100 psf			

Flat Roof Snow Load Equation:

$$P_f = 0.7C_eC_tIp_g$$

Lateral Loading:

Wind

The wind loads for Washington Park Condominiums were calculated using the design criteria and data found in ASCE 7-05, Chapter 6. I determined that I was permitted to use Method 2 – Analytical Procedure to determine my wind loading in the North-South, East-West and North West-South East direction. I chose to add a third wind analysis since a large section of the building extends at an angle from the main rectangular portion of the structure. This can be better seen in the plans located in Appendix C. I also realized during my calculations that the fundamental frequency of my building is less than 1 and therefore is classified as a flexible structure. All of this was taken into account when calculating the lateral loads. Listed below are the parameters that are used for the Analytical Method, which can be found in ASCE 7-05 Chapter 6. Other parameters needed for the calculation of the design wind pressures can be found in Appendix C.

Basic Wind Speed (V)	90 mph
Wind Direction Factor (K_d)	0.85
Importance Factor (I)	1
Exposure Category	C
Velocity Pressure Coefficient (K_z)	Case 2
Topographic Factor (K_{zt})	1
Gust Factor (G)	See Table in Appendix C
Enclosure Class	Enclosed

Using the design parameters above and the data shown in Appendix #, the design wind pressure was determined and is listed for the three wind directions below. Only the Windward and Leeward pressures on the wall of the structure were determined. Once these were determined, a total design wind pressure was calculated and then used to determine the story force at each level. This story force is found by the following equation:

$$Story\ Force = L * H * P_t$$

L = Length of Building Perpendicular to Wind

H = Tributary Height of Floor

P_t = Total Design Wind Pressure (Windward + Leeward)

Finally, the story shear and the overturning moments are calculated which allows for the determination of the base shear in each direction and the overall overturning moment for that direction. The tables below display the design wind pressures for each direction along with the story force, story shear and overturning moment. The wind load/base shear diagrams for these wind load directions are located in Appendix C.

Wind (North - South Direction) B = 68'-8" & L = 216'-5 1/2"										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.00	0.00	0.849	0.00	0.00	0.00	0.00	0.00	133.686	6487.847
Second	14.33	18.165	0.849	14.964	10.403	-5.758	16.161	20.158	133.686	6487.847
Third	25.33	11.000	0.948	16.709	11.616	-5.758	17.374	13.123	113.528	4905.356
Fourth	36.33	11.000	1.023	18.031	12.535	-5.758	18.293	13.817	100.405	3728.723
Fifth	47.33	11.000	1.081	19.053	13.246	-5.758	19.003	14.354	86.588	2076.624
Sixth	58.33	11.000	1.130	19.917	13.846	-5.758	19.604	14.807	72.235	1826.709
Seventh	69.33	13.167	1.172	20.657	14.361	-5.758	20.118	18.190	57.427	1113.563
Eighth	82.67	13.500	1.216	21.433	14.900	-5.758	20.658	19.150	39.237	536.123
Roof	96.33	13.833	1.256	22.138	15.390	-5.758	21.148	20.088	20.088	277.877

Wind (East - West Direction) B = 216'-5 1/2" & L = 68-8"										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.00	0.00	0.849	0.00	0.00	0.00	0.00	0.000	493.889	23767.869
Second	14.33	18.165	0.849	14.964	10.140	-9.353	19.493	76.644	493.889	23767.869
Third	25.33	11.000	0.948	16.709	11.322	-9.353	20.675	49.228	417.245	17927.222
Fourth	36.33	11.000	1.023	18.031	12.218	-9.353	21.571	51.361	368.017	13608.280
Fifth	47.33	11.000	1.081	19.053	12.910	-9.353	22.263	53.010	316.656	9889.078
Sixth	58.33	11.000	1.130	19.917	13.496	-9.353	22.849	54.404	263.646	6650.892
Seventh	69.33	13.167	1.172	20.657	13.997	-9.353	23.350	66.551	209.242	4050.000
Eighth	82.67	13.500	1.216	21.433	14.523	-9.353	23.876	69.770	142.692	1947.879
Roof	96.33	13.833	1.256	22.138	15.001	-9.353	24.354	72.922	72.922	1008.730

Wind (Southeast - Northwest Direction) B = 73'-5 1/4" & L = 66-0"										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.00	0.00	0.849	0.00	0.00	0.00	0.00	0.000	171.518	8251.916
Second	14.33	18.165	0.849	14.964	10.379	-9.574	19.953	26.617	171.518	8251.916
Third	25.33	11.000	0.948	16.709	11.589	-9.574	21.163	17.096	144.901	6225.748
Fourth	36.33	11.000	1.023	18.031	12.506	-9.574	22.080	17.837	127.805	4725.865
Fifth	47.33	11.000	1.081	19.053	13.215	-9.574	22.789	18.409	109.968	3418.113
Sixth	58.33	11.000	1.130	19.917	13.814	-9.574	23.388	18.893	91.559	2309.715
Seventh	69.33	13.167	1.172	20.657	14.328	-9.574	23.902	23.112	72.666	1414.554
Eighth	82.67	13.500	1.216	21.433	14.866	-9.574	24.440	24.230	49.554	676.454
Roof	96.33	13.833	1.256	22.138	15.355	-9.574	24.929	25.324	25.324	350.307

The results of the wind analysis are as follows:

Base Shear in North - South Direction = 131.686 k

Base Shear in East – West Direction = 493.889 k (Controlling)

Base Shear in Southeast – Northwest Direction = 171.518 k

Overturning Moment in North – South Direction = 6487.847 ft-k

Overturning Moment in East – West Direction = 23767.869 ft-k (Controlling)

Overturning Moment in Southeast – Northwest Direction = 8251.916 ft-k

Note on lateral forces/design:

After completing my wind and seismic analysis I believe that the wind will govern in the design. However, since I am not entirely sure I plan on further researching the lateral forces in the subsequent technical assignments using a computer model to determine which lateral force will control. Also, in looking at the base shears for seismic determined by the structural engineer I have determined that my overall building weight may be slightly lower. This could have been caused by my lack of weights and sizes for the joist girders on all of the floors and because of my underestimation on the roof due to the fact that I did not know the weight of the roof top mechanical units.

Seismic

The seismic loads for Washington Park Condominiums were calculated using ASCE 7-05, Chapter 12, as well as using the information provided by the structural engineer and the geotechnical engineer. From the geotechnical report, it was determined that the Site Class for construction would be Site Class C. The remainder of the information needed to calculate seismic loading and base shear was found in Chapter 12 of ASCE 7-05. Moreover, to determine the total weight of the building, I split the building into five separate sections because of the similarities in the construction of the floors. Floors 1 and 2, floors 3 and 4, floors 5 and 6, floors 7 and 8, and the roof are all separate dead load groupings. Each group is shown in a table that can be found in Appendix D. One of the other decisions that I had to make when doing the seismic design was determine which value for R should be used. Since there are two different types of construction in my building there would be two different values for R that could be used. I decided to use an R value of 3.5 because in doing so it allowed for the worst case Seismic Response Coefficient and therefore resulted in a more conservative base shear. All of the seismic design parameters are located in Appendix D. Below is a table showing the summation of total dead weights in the building and in turn the calculations of the story lateral force, story shear force and overturning moments.

Base Shear Calculation								
Floor	Height (ft)	Tributary Height (ft)	Dead Load (kips)	$w_x h_x^k$	C_{vx}	Lateral Force (F_x)	Story Shear (V_x)	Overturning Moment (ft-kips)
Ground	0.00	7.167	1939.190	0	0	248.325	248.325	15451.152
2nd	14.33	18.165	2050.530	63764.65	0.0291058	7.228	248.325	15451.152
3rd	25.33	11.000	1501.591	97419.23	0.0444676	11.042	241.097	12345.445
4th	36.33	11.000	1501.591	155186.6	0.0708359	17.590	230.055	9754.098
5th	47.33	11.000	1488.721	216478.4	0.0988129	24.538	212.464	7320.232
6th	58.33	11.000	1488.721	283517.5	0.1294133	32.137	187.927	5118.067
7th	69.33	13.167	1540.881	366773.9	0.1674162	41.574	155.790	3227.610
8th	82.67	13.500	1540.881	460325.8	0.2101186	52.178	114.217	1609.312
Roof	96.33	13.833	1503.864	547324.3	0.2498296	62.039	62.039	858.184
Total		W=	14555.970	2190790				

The results of the seismic design analysis are as follows:

Effective Seismic Weight = 14,555.97 k

Calculated Base Shear = 248.325 k

Gravity Loading Spot Checks:

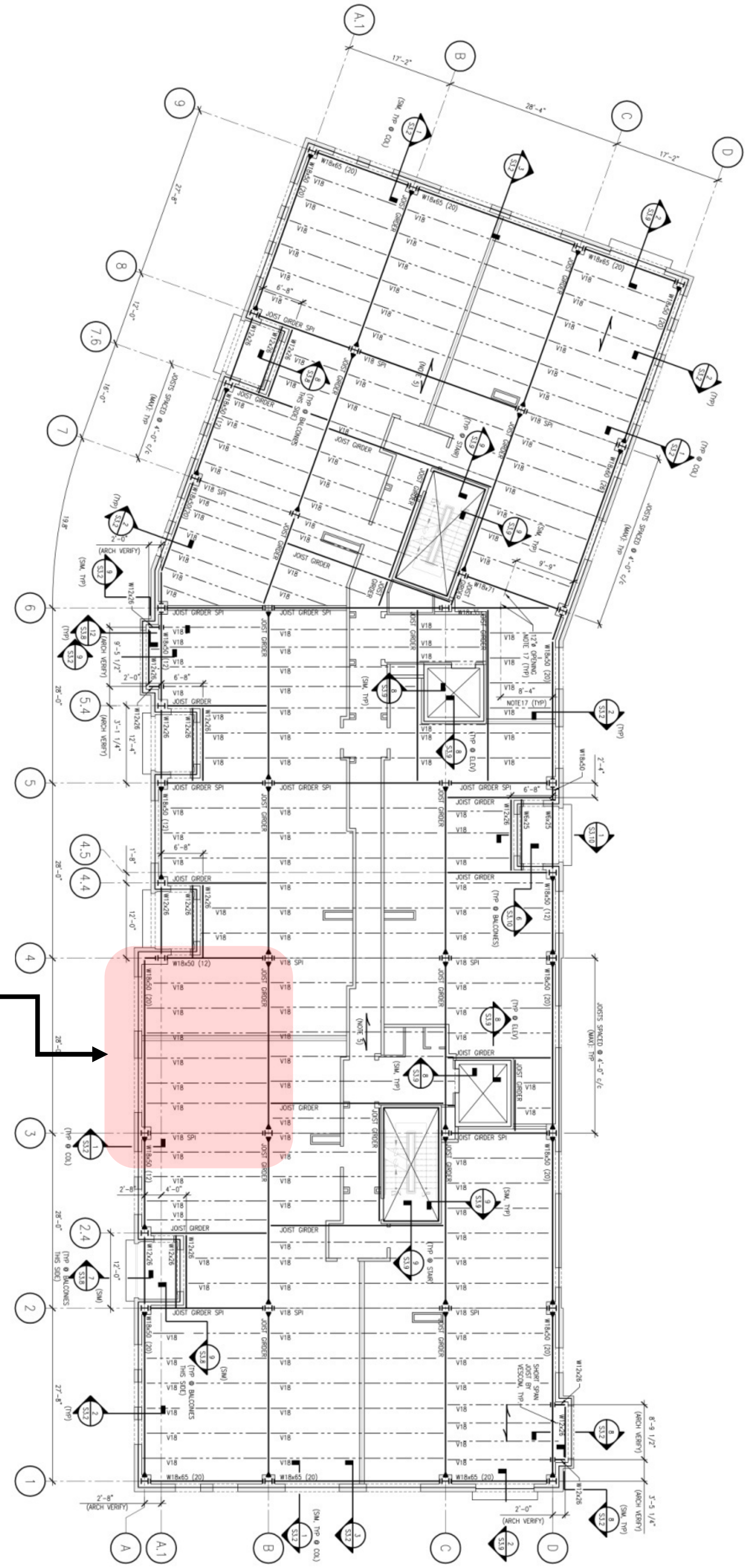
For my gravity system spot checks, I decide to check my composite floor system, a beam that supports my floors system and one of the columns that transfers the load down to the foundations. When choosing a typical bay to analyzed I realized that I do not know the size of the joist girders or how much that they weigh. These joist girders support the composite floor system on the interior of the building and because I do not know their design I cannot analyze them. This led me to analyze an exterior bay which can be found between column lines 3 to 4 and A to B. The size of this bay is 28'-0" x 20'-0" and a detailed plan is shown in Appendix E. Finally, upon completion of my spot check, which can be found in Appendix E, I have determined that the composite steel beam and the steel column are significantly oversized. I believe this is due to the fact that both are part of the lateral resisting system. The beam is part of the moment frame and is designed to resist the lateral forces and therefore would be over designed for the gravity loading. The same can be said for the column since the column is on the exterior of the building.

Appendix A: Project Delivery Team

Role	Firm	Website
Owner	Zamagias Properties 336 4th Ave., 8th Floor Pittsburgh, PA 15222	www.zamagias.com
Architect	Indovina Associates Architects 5880 Ellsworth Ave., Pittsburgh, PA 15232	www.indovina.net
General Contractor	PJ Dick, Inc. 1020 Lebanon Road West Mifflin, PA 15122	www.pjdick.com
Structural Engineer	WBCM, LLC 600 Bursca Dr., Suite 609 Pittsburgh, PA 15017	www.wbcm.com
Civil Engineer	Phillips & Associates, Inc. 1122 Mosside Blvd., Wall, PA 15148	www.paiservices.net
MEP Engineers	CJL Engineering 1550 Coraopolis Heights Rd., Suite 340 Moon Township, PA 15108	www.cjlengineering.com
Geotechnical Engineer	Professional Service Industries, Inc. 850 Poplar Street Pittsburgh PA 15220	www.psiusa.com
Landscape Architect	Laquatra Bonci Associates 95 South Tenth Street Pittsburgh, PA 15203	www.laquatrabonci.com

Appendix B: Building Layout Plans and Details

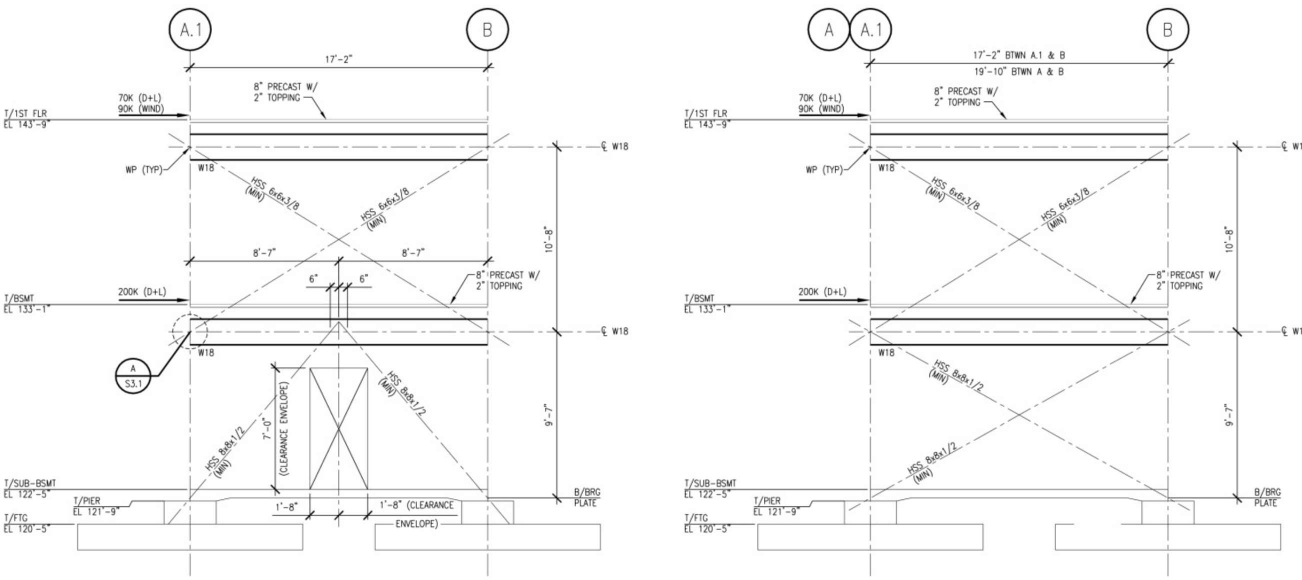
Typical Floor Layout



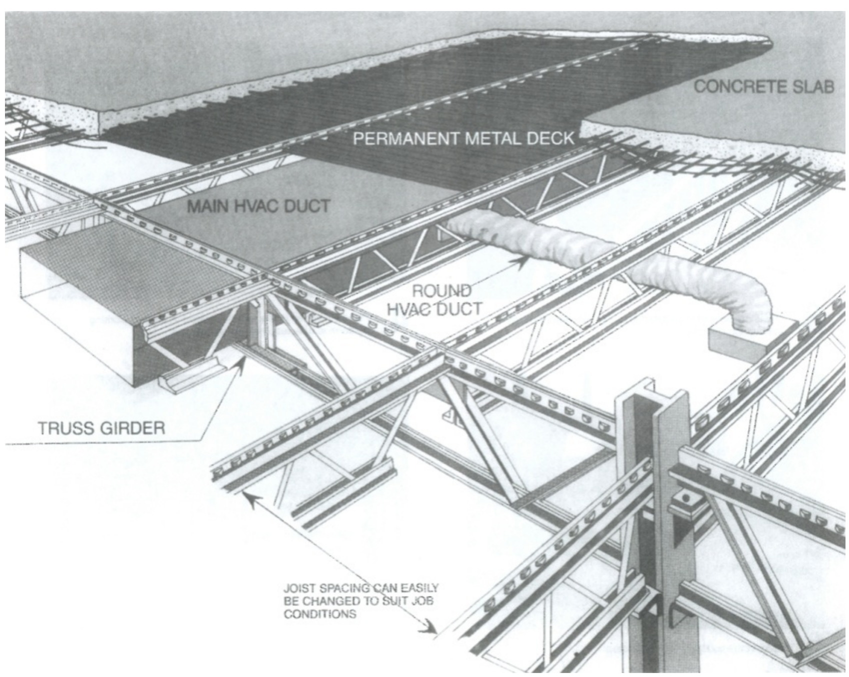
Bay used for Spot Check, for detail see Appendix E.

Location of Braced Frames (Basement Plan)

Typical Brace Frame Elevations



Perspective Section of VESCOM Composite Joist System



Appendix C: Wind Analysis

Floor	Height Above Ground Level (z)	Exposure Category	K_z
2nd	14.33	C	0.849
3rd	25.33	C	0.948
4th	36.33	C	1.023
5th	47.33	C	1.081
6th	58.33	C	1.130
7th	69.33	C	1.172
8th	82.67	C	1.216
Roof	96.33	C	1.256

Figure C-1: Velocity Pressure Exposure Coefficients

Floor	K_z	K_{zt}	K_d	K_h	V (mph)	I	q_h (lb/ft^2)	q_z (lb/ft^2)
2nd	0.849	1.0	0.85	1.253	90.0	1.0	22.085	14.964
3rd	0.948	1.0	0.85	1.253	90.0	1.0	22.085	16.709
4th	1.023	1.0	0.85	1.253	90.0	1.0	22.085	18.031
5th	1.081	1.0	0.85	1.253	90.0	1.0	22.085	19.053
6th	1.13	1.0	0.85	1.253	90.0	1.0	22.085	19.917
7th	1.172	1.0	0.85	1.253	90.0	1.0	22.085	20.657
8th	1.216	1.0	0.85	1.253	90.0	1.0	22.085	21.433
Roof	1.256	1.0	0.85	1.253	90.0	1.0	22.085	22.138

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (lb/ft^2) $q_h = 0.00256 K_h K_{zt} K_d V^2 I$ (lb/ft^2)

Figure C-2: Velocity Pressures, q_z and q_h

Gust Factor Variables					
I_z	c	$\bar{z}$	g_Q	g_V	h
0.182	0.20	57.8	3.4	3.4	96.33
$L_{\bar{z}}$	l	g_r	n_1	V_z	$\bar{b}$
559.31	500	4.055	0.575	93.527	0.65
$\bar{\alpha}$	V (mph)	N_1	R_n	R_h	β
0.1538	90	3.44	0.0642	0.1843	1.0

Figure C-3: Gust Factor Variables

North - South Direction					
B	Q	R_b	R_L	R	G_f
68.667	0.879	0.385	0.0476	0.05016	0.869

East - West Direction					
B	Q	R_b	R_L	R	G_f
216.44	0.834	0.150	0.1419	0.032543	0.847

Southeast - Northwest Direction					
B	Q	R_b	R_L	R	G_f
73.44	0.878	0.367	0.1472	0.002602	0.867

Figure C-4: Directional Gust Factors, G_f

Wall Coefficients					
North - South Direction		East - West Direction		Southeast - Northwest Direction	
	C_p		C_p		C_p
Windward	0.8	Windward	0.8	Windward	0.8
Leeward	-0.3	Leeward	-0.5	Leeward	-0.5
Side Wall	-0.7	Side Wall	-0.7	Side Wall	-0.7
Roof Coefficients					
North - South Direction		East - West Direction		SouthEast - NorthWest Direction	
	C_p		C_p		C_p
Windward	0.36	Windward	0.3	Windward	0.3
Leeward	-0.6	Leeward	-0.6	Leeward	-0.6

Figure C-5: External Pressure Coefficients, C_p

Wind (North - South Direction) B = 68'-8" & L = 216'-5 1/2"										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.00	0.00	0.849	0.00	0.00	0.00	0.00	0.00	133.686	6487.847
Second	14.33	18.165	0.849	14.964	10.403	-5.758	16.161	20.158	133.686	6487.847
Third	25.33	11.000	0.948	16.709	11.616	-5.758	17.374	13.123	113.528	4905.356
Fourth	36.33	11.000	1.023	18.031	12.535	-5.758	18.293	13.817	100.405	3728.723
Fifth	47.33	11.000	1.081	19.053	13.246	-5.758	19.003	14.354	86.588	2076.624
Sixth	58.33	11.000	1.130	19.917	13.846	-5.758	19.604	14.807	72.235	1826.709
Seventh	69.33	13.167	1.172	20.657	14.361	-5.758	20.118	18.190	57.427	1113.563
Eighth	82.67	13.500	1.216	21.433	14.900	-5.758	20.658	19.150	39.237	536.123
Roof	96.33	13.833	1.256	22.138	15.390	-5.758	21.148	20.088	20.088	277.877

Wind (East - West Direction) B = 216'-5 1/2" & L = 68-8"										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.00	0.00	0.849	0.00	0.00	0.00	0.00	0.000	493.889	23767.869
Second	14.33	18.165	0.849	14.964	10.140	-9.353	19.493	76.644	493.889	23767.869
Third	25.33	11.000	0.948	16.709	11.322	-9.353	20.675	49.228	417.245	17927.222
Fourth	36.33	11.000	1.023	18.031	12.218	-9.353	21.571	51.361	368.017	13608.280
Fifth	47.33	11.000	1.081	19.053	12.910	-9.353	22.263	53.010	316.656	9889.078
Sixth	58.33	11.000	1.130	19.917	13.496	-9.353	22.849	54.404	263.646	6650.892
Seventh	69.33	13.167	1.172	20.657	13.997	-9.353	23.350	66.551	209.242	4050.000
Eighth	82.67	13.500	1.216	21.433	14.523	-9.353	23.876	69.770	142.692	1947.879
Roof	96.33	13.833	1.256	22.138	15.001	-9.353	24.354	72.922	72.922	1008.730

Wind (Southeast - Northwest Direction) B = 73'-5 1/4" & L = 66-0"										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.00	0.00	0.849	0.00	0.00	0.00	0.00	0.000	171.518	8251.916
Second	14.33	18.165	0.849	14.964	10.379	-9.574	19.953	26.617	171.518	8251.916
Third	25.33	11.000	0.948	16.709	11.589	-9.574	21.163	17.096	144.901	6225.748
Fourth	36.33	11.000	1.023	18.031	12.506	-9.574	22.080	17.837	127.805	4725.865
Fifth	47.33	11.000	1.081	19.053	13.215	-9.574	22.789	18.409	109.968	3418.113
Sixth	58.33	11.000	1.130	19.917	13.814	-9.574	23.388	18.893	91.559	2309.715
Seventh	69.33	13.167	1.172	20.657	14.328	-9.574	23.902	23.112	72.666	1414.554
Eighth	82.67	13.500	1.216	21.433	14.866	-9.574	24.440	24.230	49.554	676.454
Roof	96.33	13.833	1.256	22.138	15.355	-9.574	24.929	25.324	25.324	350.307

Figure C-6 thru 8: Building Design Wind Pressures

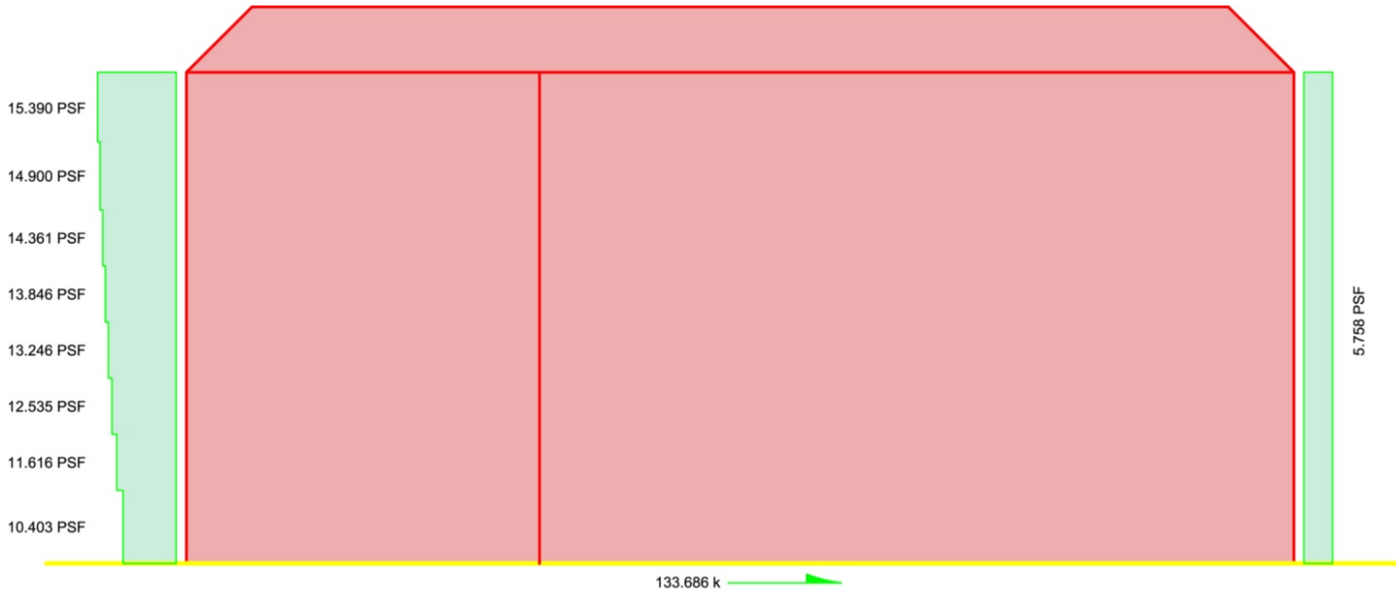


Figure C-9: Wind Load Summary Distribution for North-South Direction

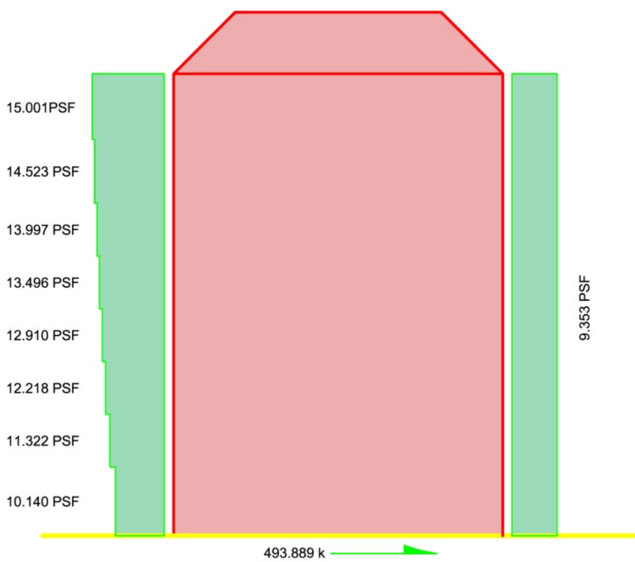


Figure C-10: Wind Load Summary Distribution for East-West Direction

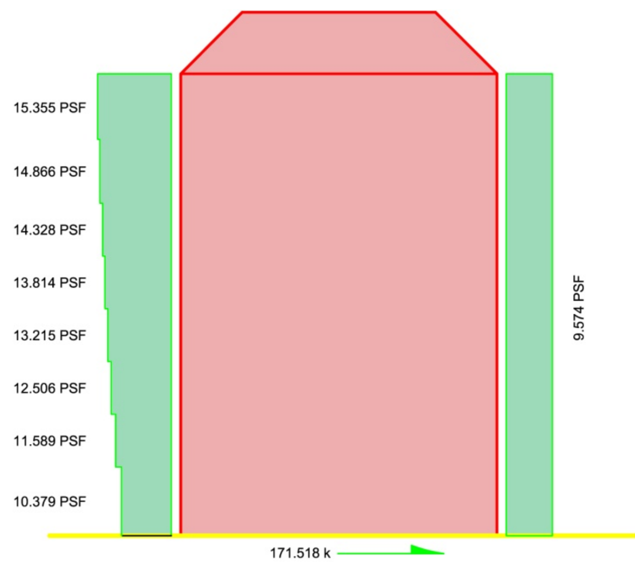


Figure C-11: Wind Load Summary Distribution for Southeast-Northwest Direction

Appendix D: Seismic Analysis

Seismic Parameters for Washington Park Condominiums								
S_s	S_1	Site Class	F_a	F_v	S_{ds}	S_{d1}	Seismic Design Category	Seismic Use Group
0.128	0.058	C	1.6	2.4	0.137	0.093	B	I
I	R	C_u	T_a	T_L	T_s	C_s	k	V (kips)
1	3.5/5	1.7	1.082	12	0.6333	0.01706	1.291	248.3248

Figure D-1: Seismic Design Parameters

1st and 2nd Floor Dead Weights	
Item	Weight (kips)
Steel Beams	199.501
8" P/C Plank w/ 2" Structural Topping	1256.13
Steel Columns	45.731
Brick Wall Façade	158.688
MEP	83.742
Sprinklers	41.871
Ceiling	69.785
Flooring	69.785
Misc	13.957
Total	1939.190

5th and 6th Floor Dead Weights	
Item	Weight (kips)
Steel Beams	31.479
2 11/16" RWC Slab on 1 5/16" FLR Deck	587.065
Steel Columns	39.545
Brick Wall Façade	243.567
MEP	81.916
Sprinklers	40.958
Ceiling	68.263
Partitions	273.053
Flooring	68.263
VESCOM Composite Joists	54.611
Total	1488.720

3rd and 4th Floor Dead Weights	
Item	Weight (kips)
Steel Beams	31.479
2 11/16" RWC Slab on 1 5/16" FLR Deck	587.065
Steel Columns	52.415
Brick Wall Façade	243.567
MEP	81.916
Sprinklers	40.958
Ceiling	68.263
Partitions	273.053
Flooring	68.263
VESCOM Composite Joists	54.611
Total	1501.590

Figure D-2 thru D-4: Total Floor Weights

7th and 8th Floor Dead Weights	
Item	Weight (kips)
Steel Beams	31.479
2 11/16" RWC Slab on 1 5/16" FLR Deck	587.065
Steel Columns	40.04
Brick Wall Façade	295.233
MEP	81.916
Sprinklers	40.958
Ceiling	68.263
Partitions	273.053
Flooring	68.263
VESCOM Composite Joists	54.611
Total	1540.881

Figure D-5: Total Floor Weights

Roof Dead Weights	
Item	Weight (kips)
Steel Beams	28.986
4 11/16" RWC Slab on 1 5/16" FLR Deck	928.381
Steel Columns	40.04
Brick Wall Façade	151.307
MEP	81.916
Sprinklers	40.958
Ceiling	109.221
VESCOM Composite Joists	54.611
Flooring	68.263
Total	1503.683

Figure D-6: Total Floor Weights

Base Shear Calculation								
Floor	Height (ft)	Tributary Height (ft)	Dead Load (kips)	$w_x h_x^k$	C_{vx}	Lateral Force (F_x)	Story Shear (V_x)	Overturning Moment (ft-kips)
Ground	0.00	7.167	1939.190	0	0	248.325	248.325	15451.152
2nd	14.33	18.165	2050.530	63764.65	0.0291058	7.228	248.325	15451.152
3rd	25.33	11.000	1501.591	97419.23	0.0444676	11.042	241.097	12345.445
4th	36.33	11.000	1501.591	155186.6	0.0708359	17.590	230.055	9754.098
5th	47.33	11.000	1488.721	216478.4	0.0988129	24.538	212.464	7320.232
6th	58.33	11.000	1488.721	283517.5	0.1294133	32.137	187.927	5118.067
7th	69.33	13.167	1540.881	366773.9	0.1674162	41.574	155.790	3227.610
8th	82.67	13.500	1540.881	460325.8	0.2101186	52.178	114.217	1609.312
Roof	96.33	13.833	1503.864	547324.3	0.2498296	62.039	62.039	858.184
Total		W=	14555.970	2190790				

Figure D-7: Base Shear and Overturning Moment

Appendix E: Spot Check Calculations

COMPOSITE FLOOR SPOT CHECK:

FOR TYPICAL BAY SIZE SEE ABOVE DRAWING.

LOADS:

DEAD:

AREA OF BAY = 560.0 ft^2

STEEL BEAMS = 4 PSF

FLOOR FINISHES = 5 PSF

2 11/16" RWC SLAB ON 1 5/16" STEEL DECK = 43 PSF

MEP = 6 PSF

SPRINKLERS = 3 PSF

CEILING = 5 PSF

PARTITIONS = 20 PSF

JOISTS = 13.0 PSF

100 PSF

LIVE:

TYPICAL CONDO FLOOR = 40 PSF

LOAD COMBINATIONS:

$1.4D = 1.4(100.0 \text{ PSF}) = 140.0 \text{ PSF} (5 \text{ ft}) = 700 \text{ PLF}$

JOIST SPACING

$1.2D + 1.6L = 1.2(100.0 \text{ PSF}) + 1.6(40 \text{ PSF}) = 184.0 \text{ PSF} (5 \text{ ft}) = 920.0 \text{ PLF}$

$\therefore$ 1.2D + 1.6L CONTROLS.

* USING THE VESCOM JOIST WEIGHT TABLES:

- CHOOSE 18" COMPOSITE STEEL JOIST (SAME AS ENGINEER DESIGN)
- TOTAL LOAD CAPACITY = 950 PLF
- RESISTANT MOMENT = 570 lb-ft

PARTIALLY COMPOSITE STEEL BEAM:

LOADING:

EXTERIOR WALL = 40 PSF (11 FT) = 440 lb/ft

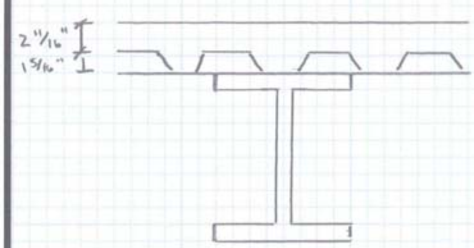
FLOOR SYSTEM
(FROM COMPOSITE FLOOR SPOT CHECK) = 920 PLF (20 FT) = $\frac{18400 \text{ lb}}{2}$ = 9200 lb

$9200 \text{ lb} (6 \text{ JOISTS}) = \frac{55200 \text{ lb}}{28 \text{ ft}} = 1971.43 \text{ lb/ft}$

* I USED THE POINT LOADS TOGETHER AS A UNIFORMLY DISTRIBUTED LOAD SINCE THERE WERE SIX OF THE IN A 28 ft SPAN.

$\therefore W_T = 1971.43 \text{ lb/ft} + 440 \text{ lb/ft} = 2411.43 \text{ lb/ft}$

$$M_u = \frac{w_u l^2}{8} = \frac{(2411.3 \text{ lb/ft})(28 \text{ ft})^2}{8} = \underline{\underline{236.31 \text{ ft}\cdot\text{k}}}$$



$$b_{eff} = \min \begin{cases} \text{SPACING} = 240" \\ \text{SPAN}/4 = 84" \Rightarrow \text{CONTROLS!} \end{cases}$$

ASSUME THAT T=C

$C_c = 0.85 (3.5 \text{ ksi})(2\,{}^{11}/_{16}" \times 84") = 671.61 \text{ k}$

$T_s = (14.7 \text{ in}^2)(50 \text{ ksi}) = 735 \text{ k}$

– ASSUME $a = 1.0"$: $Y2 = 4" - 1/2 = 3.5"$ * P.N.A IS IN STEEL.

@ BFL: $\phi M_n = 566 \text{ ft}\cdot\text{k}$, $\Sigma Q_n = 306 \text{ k}$ $\boxed{\text{USING TABLE 3-19 IN AISC 13.}}$

$$a = \frac{306 \text{ k}}{(0.85)(2\,{}^{11}/_{16}")(84)} = 1.595" > 1.0" \text{ INVALID!}$$

– ASSUME $a = 2.0"$: $Y2 = 4" - 1" = 3.0"$

@ BFL: $\boxed{\phi M_n = 554 \text{ ft}\cdot\text{k}, \quad \Sigma Q_n = 245 \text{ k}}$

$$a = \frac{245 \text{ k}}{(0.85)(2\,{}^{11}/_{16})(84)} = 1.28" < 2.0" \quad \therefore \underline{\underline{\text{OK!}}}$$

$\therefore$ W18x50 IS OK FOR GRAVITY LOADING!

COMPOSITE STEEL BEAM:

DEFLECTION:

$$L/240 = \frac{(28\text{ ft})(12)}{240} = 1.4\text{ in}$$

$$1.4\text{ in} = \frac{5 w_u l^4}{384 EI} = \frac{5(2411.3\text{ lb/ft})(28)^4}{384(29000)(800\text{ in}^4)}$$

$\Delta_{ACT} = .00683\text{ in} < 1.4\text{ in}$ $\underline{\underline{\therefore \text{OK}}}$

STEEL COLUMN SPOT CHECK:

LOADING:

TRIBUTARY AREA =

$$P = \frac{wl}{2} = \frac{(2411.3\text{ lb/ft})(28\text{ ft})}{2} = \underbrace{33.76^k(2)}_{\text{FLOOR LOADING}} + \underbrace{(50)(28)(2)}_{\text{STEEL BEAM WEIGHT}} = 70.32^k$$

* LOADING ON COLUMN PER FLOOR IS 70.32^k, THEREFORE USE:

$$P_u = 70.32^k(7\text{ FLOORS}) = 492.24^k$$

$F_y = 50^{ksi}$
$L = 11\text{ ft}$
$K_x = 1.0$
$r_x/r_y = 1.60$

* COLUMN USED IS W14x176 w/ $\phi P_n = 2150^k$, THE COLUMN IS OVERSIZED SINCE THE COLUMN IS EXTERIOR AND DESIGN PRIMARILY FOR LATERAL FORCES.

$\underline{\underline{\therefore \text{W14x176 IS OK!}}}$

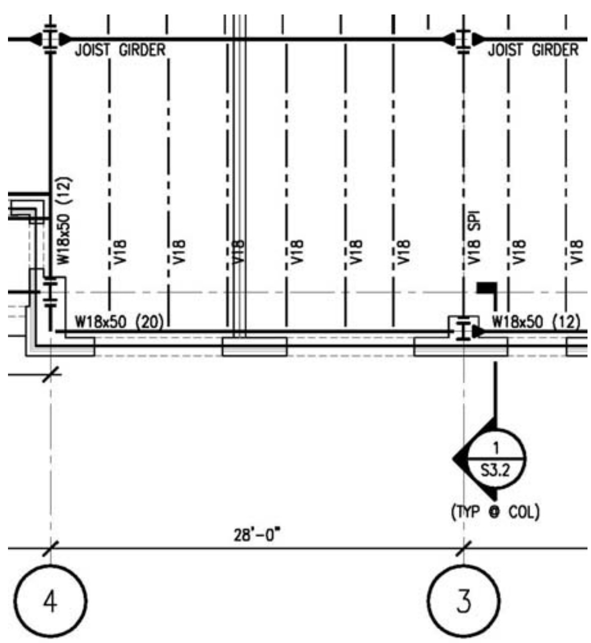


Figure E-1: Plan Detail showing typical bay examined for gravity loading.