Washington Park Condominiums

Mt. Lebanon, Pennsylvania

Technical Report #3
Lateral System Analysis and Confirmation Design

Architectural Engineering
2008-2009 Senior Thesis

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Executive Summary:

Washington Park Condominiums is an 8-story multi-use retail and residential building located in Mt. Lebanon, Pennsylvania. The lateral force resisting system of the building which will be studied in this report is a steel moment frame system. There are four frames that run in the north – south direction of the building which also is the longest length of the building. Furthermore, there are nine, 3 bay moment frames that run in the east – west direction of the building. These frames are connected to brace frames in the basement and sub-basement levels which help to carry the soil pressure and transfer the lateral load to the foundations.

The purpose of Technical Report #3 was to investigate the lateral system of the building and to prepare an analysis and design verification summary of the structure. The loads found in Technical Report #1 were used as a base and were eventually modified based on the stiffness's of the frames that was determined during the design and analysis process. The investigation used hand methods along with computer analysis to determine and discuss the following:

- Direct and Torsional Shear Forces
- Controlling Load Combinations
- Logical Load Path
- Story and Overall Structure Drift
- Beam, Column and Brace Strength
- Overturning Moments and Uplift

The analysis shows how the moment frames, running in both directions through the building, resist the lateral forces that are applied by both wind and seismic. Once the loads were applied to the structure a frame analysis was performed by both hand calculation and computer modeling. This allowed the strength of certain critical members to be accessed. After reviewing these members it was determined that the member sizes were chosen based on the governing drift. Upon completion of the analysis it was determined that in most cases the wind controls the lateral design. However, this is not apparent in the drift calculations which show the wind drifts meeting the acceptable code limits while the seismic drifts fails the limits deemed acceptable by code. This issue will be studied at length in the proposal to see if there is really an issue in terms of seismic drift and if so how can the drift be minimized without compromising the architectural aspects of the floor system and therefore vastly increasing building costs.

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 contain 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.

Existing Composite Joist and Precast Concrete Plank System:

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 with 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".

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 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 and f'_c = 4000 psi concrete is used for the entire system.

Floor Systems

Two separate floors systems 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.

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. 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.

Lateral System

The lateral resisting system within the building is mainly moment resisting steel frames made up of wide flange beams. 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. Figure 1 below shows the four different types of moment frames that exist within the building. These four frames 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 and the foundations. 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 shown in Figure 2 below. This plan detail and the detail of the brace frames can be found in Figures 2 and 3.

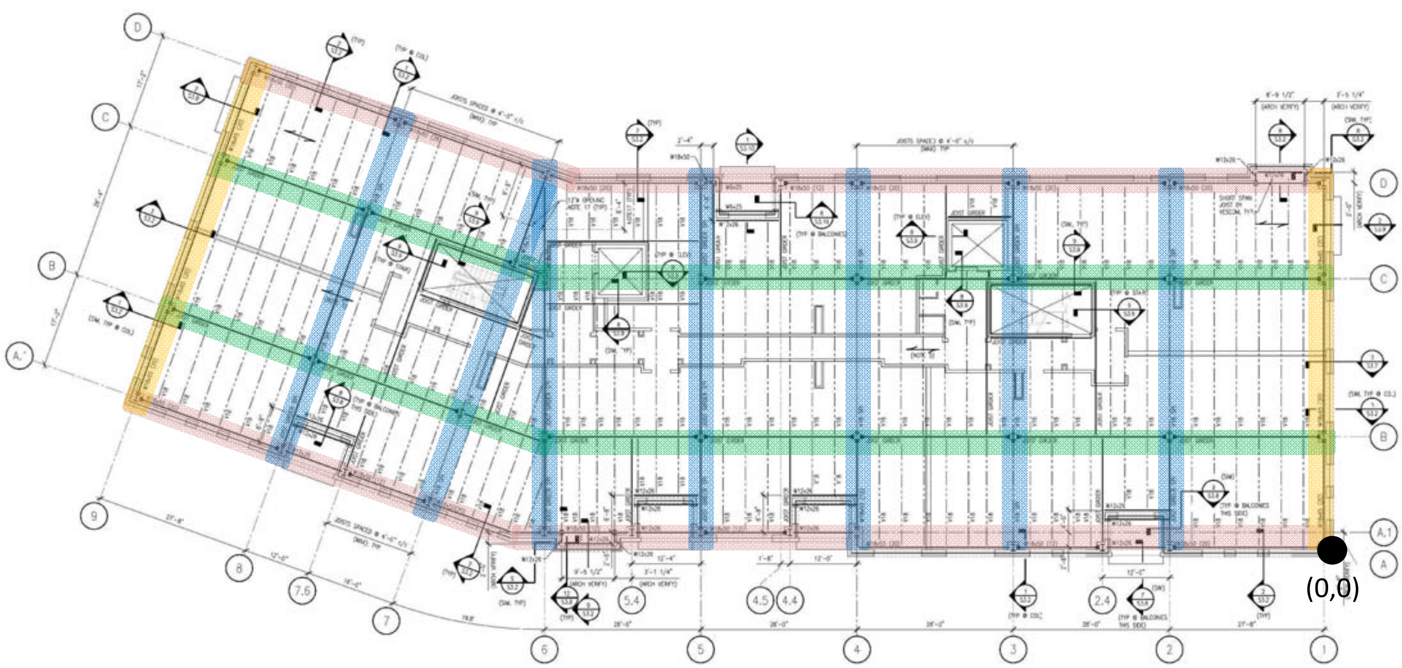


Figure 1: Moment Frame Diagram

North – South Frames (A & D)

East – West Frames (F thru L)

North – South Frames (B & C)

East – West Frames (E & M)

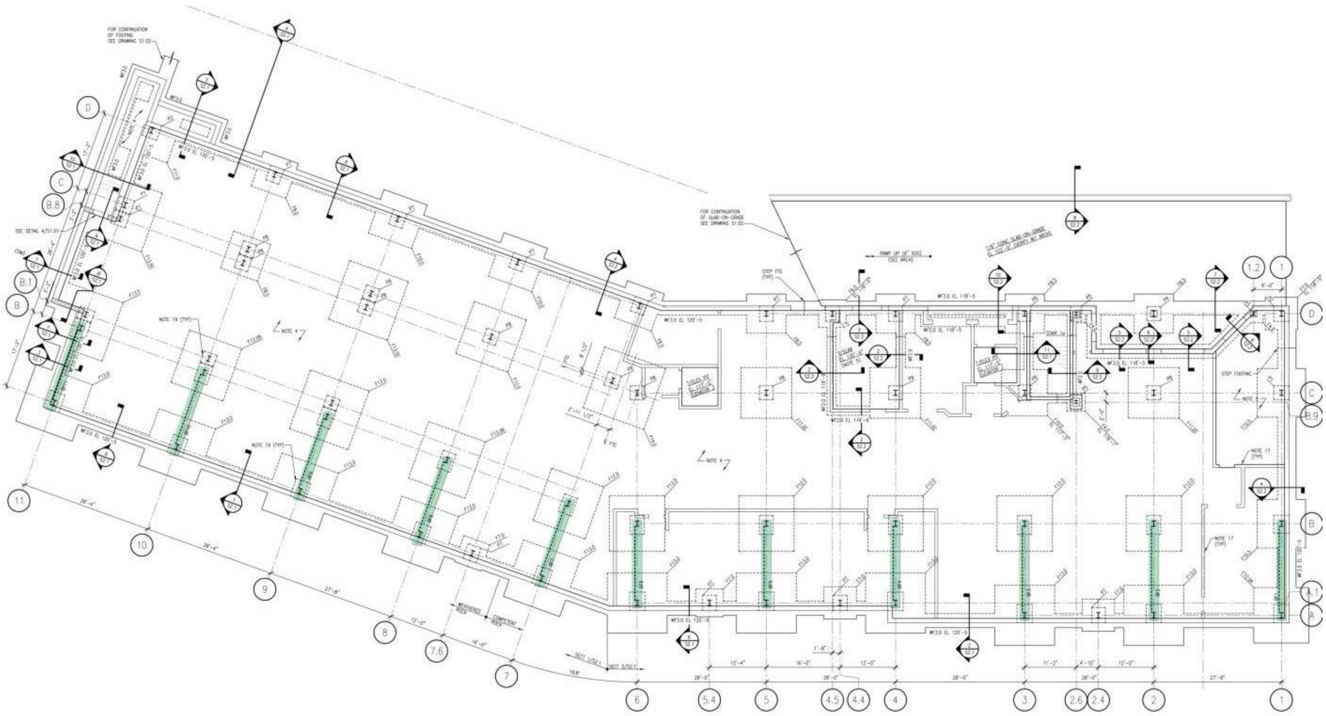


Figure 2: Braced Frame Location Diagram

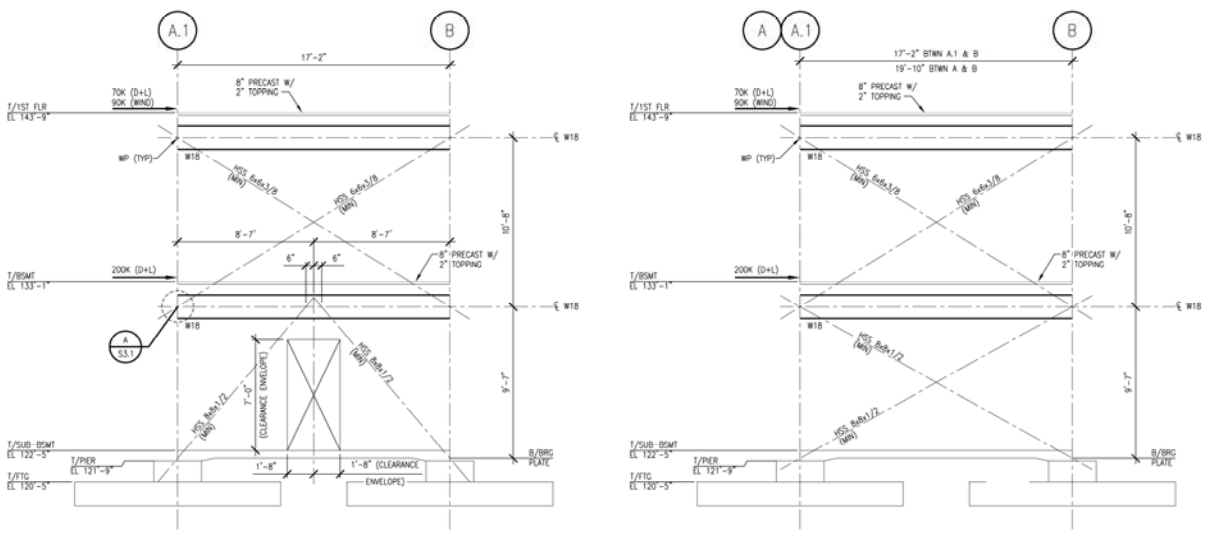


Figure 3: Braced Frames Elevation

Applicable Codes, Design Criteria and Load Cases

Since an in depth lateral analysis was performed there were many reference manuals and materials that were used to complete the design, these are listed as follows:

- IBC 2003 with Amendments for Mt. Lebanon, Pennsylvania
- ASCE 7-05 : Minimum Design Loads for Buildings and Other Structures
- ACI 318-08 Building Code and Commentary
- Design of Concrete Structures Textbook (AE 431)
- AISC Specification for Structural Steel Buildings, 13^{th} Edition
- RAM Structural System (Gravity Loads)
- STAAD Pro 2006 (Lateral Load Analysis)

Design Deflection Criterion

The following design criterion was also used and can be found in IBC 2006 and ASCE 7-05:

- $\Delta = H/400$ for Allowable Story and Building Drift due to Wind Loading
- $\Delta = 0.015h_{sx}$ for Allowable Story and Building Drift due to Seismic Loading

Design Load Combinations

The following Load and Resistance Factor Design load combinations were considered for analysis, as noted in ASCE 7-05 Chapter 2:

- 1.4(Dead)
- 1.2(Dead) + 1.6(Live) + 0.5(Roof Live)
- 1.2(Dead) + 1.6(Roof Live) + 0.8(Wind)
- 1.2(Dead) + 1.6(Snow) + 0.8(Wind)
- 1.2(Dead) + 1.6(Snow) + 1.0(Live)
- 1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Snow)
- 1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Roof Live)
- 1.2(Dead) + 1.0(Earthquake) + 1.0(Live) + 0.2(Snow)

Building Design Loads and Lateral Criteria

In order to complete an analysis of Washington Park Condominiums, that gravity and lateral loads acting on the building need to be identified. The dead and live loads that are used were also used in Technical Report #1 because it was determined that those loads closely matched those used by the design engineer. The lateral loads were recalculated by determining the equivalent stiffness for each moment frame in the building. Finding the stiffness of each frame allowed for the direct shear and torsional shear at each level to be calculated and for the loads to be distributed throughout the building according to stiffness. These loads and the process in which they were determined will be discussed in detail in the next section. The tables below list the dead and live loads for the building as well as the wind and seismic design criteria used in the computation of the lateral loads.

Gravity Loads

Dead Load Table

Floor Dead Load	
Material/System	Load
Normal Weight Concrete	145 pcf
Steel	Per Shape
Brick Veneer w/ studs	40 psf
8" P/C Plank w/ 2" Structural Topping	90 psf
MEP	6 psf
Sprinklers	3 psf
Ceiling	5 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

Roof Dead Load	
Material/System	Load
4 11/16" RWC Slab on 1 5/16" FLR Deck	68 psf
MEP	6 psf
Sprinklers	3 psf
Ceiling	8 psf
VESCOM Joists	4 psf
Asphalt Shingles/Felts	4 psf
1/2" Cement Bonded Particle Board	5 psf
Light Gauge Roof Trusses @ 2'-0" O.C.	4 psf

Table 1: Dead Load Tables

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 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			
Corridors Above 1st Floor	80 psf			
Mech/Elec Spaces	150 psf			
Second Floor Terrace	100 psf			
Apartment Balconies	100 psf			

Table 2: Live Load Tables

Wind Criteria

The wind loads for Washington Park Condominiums were calculated using the design criteria found in ASCE 7-05, Chapter 6 and it was determined that it was permitted to use Method 2 – Analytical Procedure for the design. The table below lists the applicable wind design factors.

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
Enclosure Class	Enclosed

Table 3: Wind Design Criteria

Seismic Criteria

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. The table below lists the applicable seismic design factors.

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	Period Coefficient
1	3.5	1.7	1.082	12	0.6333	0.01706	1.291	0.80

Table 4: Seismic Design Parameters

Load Distribution and Analysis

The distribution of lateral forces throughout the structure can be determined using varying design methods. For this technical report the lateral loads were ascertained based on the relative stiffness of each frame in conjunction with the rest of the frames in the building. The composite joist floor system is used as a shear diaphragm and it distributes load to the various frames based on their respective relative stiffness. This process allows for a more accurate distribution of loads along with the ability to study how the frames work together to transfer the loads from the moment frames to the foundations. For the analysis, hand calculations, Microsoft Excel and STAAD.Pro 2006 were utilized as a way to obtain a more thorough understanding of the distribution.

Determination of Relative Stiffness (k) and Center of Rigidity

The determination of each frame's relative stiffness was done using hand calculations and STAAD.Pro 2006. To begin the four different frames (interior and exterior in each direction) were constructed in STAAD using the steel sizes and connections given by the structural engineer. Next a one kip load was applied at the top of each moment frame to establish the amount of deflection that occurs at the top of the frame. From there, the inverse of this deflection was taken and these values were used for k. From these k values, the center of rigidity of the building was found. The x and y distance values used for the center of rigidity were taken from a (0,0) point chosen on the bottom right corner of the building as shown in Appendix C. The relative stiffness's for each frame are included in the table for the direct and torsional shear shown below.

Determination of Direct Shear and Torsional Shear

After establishing the stiffness values for each frame, the direct and torsional shears for each frame at each level could be determined. For this analysis, it was decided that four regular frames could be chosen that would be representative of all the frames in the building. Frames A and E are the exterior frames in the north - south and east - west directions respectively. Also, frames C and F are the interior frames in the north - south and east - west directions respectively. The calculation for the moments for each frame at each floor used in the torsional force can be found in Appendix F. The direct and torsional shears are then found by using the following equations and variables. These variables can be found in Table 5 below along with the values for the forces of the four frames that are being studied:

$$F_{Direct} = \frac{K_i}{\sum K}(F_i) \qquad F_{Torsional} = \frac{M_i K_i d_i}{I_p}$$

Where: K = Equivalent Stiffness of Frame

F_i = Lateral Story Force

$M_i = F_i \times (d_{CM} - d_R)$

d_i = Distance from Center of Rigidity to Frame

$$I_P = \sum K_{ty} x_i^2 + \sum K_{tx} y_i^2$$

Torsion Constants							
Floor	Center of Rigidity		Center of Mass		I_X (in^4)	I_y (in^4)	I_P (in^4)
	X_R (ft)	Y_R (ft)	X_R (ft)	Y_R (ft)			
Second	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Third	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Fourth	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Fifth	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Sixth	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Seventh	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Eighth	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Roof	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41

Table 5: Center of Rigidity and Mass along with Calculated Moment of Inertia

Resultant Shears Due to Wind Loading																
Floor	Frames															
	A				C				E				F			
	Frame Stiffness (k/in)	Direct Shear	Torsional Shear	Total Shear	Frame Stiffness (k/in)	Direct Shear	Torsional Shear	Total Shear	Frame Stiffness (k/in)	Direct Shear	Torsional Shear	Total Shear	Frame Stiffness (k/in)	Direct Shear	Torsional Shear	Total Shear
Second	9.71	2.544	-0.012	2.555	19.23	5.039	0.013	5.052	16.95	5.159	-0.115	5.274	22.22	6.764	-0.110	6.875
Third	9.71	2.201	-0.010	2.211	19.23	4.360	0.011	4.371	16.95	4.404	-0.098	4.502	22.22	5.775	-0.094	5.869
Fourth	9.71	2.318	-0.010	2.328	19.23	4.591	0.012	4.602	16.95	4.595	-0.102	4.697	22.22	6.025	-0.098	6.123
Fifth	9.71	2.408	-0.011	2.419	19.23	4.769	0.012	4.781	16.95	4.743	-0.105	4.848	22.22	6.218	-0.101	6.320
Sixth	9.71	2.484	-0.011	2.495	19.23	4.920	0.013	4.932	16.95	4.868	-0.108	4.975	22.22	6.382	-0.104	6.486
Seventh	9.71	3.051	-0.014	3.065	19.23	6.044	0.015	6.059	16.95	5.954	-0.132	6.086	22.22	7.807	-0.127	7.934
Eighth	9.71	3.212	-0.014	3.227	19.23	6.363	0.016	6.379	16.95	6.242	-0.138	6.381	22.22	8.184	-0.133	8.318
Roof	9.71	3.370	-0.015	3.385	19.23	6.674	0.017	6.691	16.95	6.524	-0.145	6.669	22.22	8.554	-0.139	8.693

Table 6: Resultant Shears Due to Wind Loading

Resultant Shears Due to Seismic Loading																
	Frames															
	A				C				E				F			
Floor	Frame Stiffness (k/in)	Direct Shear	Torsional Shear	Total Shear	Frame Stiffness (k/in)	Direct Shear	Torsional Shear	Total Shear	Frame Stiffness (k/in)	Direct Shear	Torsional Shear	Total Shear	Frame Stiffness (k/in)	Direct Shear	Torsional Shear	Total Shear
Second	9.71	1.213	0.003	1.215	19.23	2.401	-0.005	2.406	16.95	0.647	0.008	0.655	22.22	0.848	0.008	0.856
Third	9.71	1.852	0.004	1.857	19.23	3.669	-0.007	3.676	16.95	0.988	0.013	1.001	22.22	1.295	0.012	1.307
Fourth	9.71	2.951	0.007	2.958	19.23	5.844	-0.011	5.856	16.95	1.574	0.020	1.594	22.22	2.063	0.019	2.083
Fifth	9.71	4.116	0.010	4.126	19.23	8.152	-0.016	8.168	16.95	2.195	0.028	2.224	22.22	2.878	0.027	2.905
Sixth	9.71	5.391	0.013	5.404	19.23	10.677	-0.021	10.698	16.95	2.875	0.037	2.912	22.22	3.769	0.035	3.805
Seventh	9.71	6.974	0.016	6.991	19.23	13.812	-0.027	13.839	16.95	3.720	0.048	3.768	22.22	4.876	0.046	4.922
Eighth	9.71	8.753	0.020	8.774	19.23	17.335	-0.034	17.369	16.95	4.669	0.060	4.729	22.22	6.120	0.057	6.177
Roof	9.71	10.408	0.024	10.432	19.23	20.612	-0.040	20.652	16.95	5.551	0.071	5.622	22.22	7.277	0.068	7.345

Table 7: Resultant Shears Due to Seismic Loading

After obtaining the direct and torsional forces for each frame it is obvious that these values are smaller than that of story forces that are determined using tributary area. This could be due to the fact that using relative stiffness allows for the building to be analyzed as an entire structure since the direct shears are determined by using a ratio of frame stiffness to total stiffness. The method of tributary area does not take into account any other frame or how the other frames in the building may be working together to distribute the lateral load throughout the building. This allows for the calculated forces using the relative stiffness to be more accurate and therefore make the subsequent portal frame, strength and drift analyses more reliable.

Frame Analysis

Portal Frame

Following the determination of the direct and torsional forces through the establishment of relative stiffness, the story forces were applied to the structure and were analyzed using Portal Method. In this case, three different portal method analyses were performed. Frames E and F were analyzed using resultant shears due to wind since the value for total shear due to wind were greater than that of seismic. Next, a single floor of frame C was analyzed using the resultant shear due to seismic loading since for frame C the shear for seismic was greater than the shear due to wind. The moments in the beams and columns that were obtained from the portal frame analysis were used in the beam and column strength checks. The results of the portal frame analysis can be found in Appendix D.

STAAD Analysis

In conjunction with the portal frame analysis, a frame analysis using STAAD.Pro 2006 was performed. This analysis was useful in providing information concerning story and overall building drift as well as determining whether or not the steel members chosen in the design were adequate to carry the loads. Figures 4 and 5 below show the two frame constructions that were analyzed using STAAD. For ease of analysis along with a conservative design, frames A and C were analyzed as if they were straight moment frames. This was done because the entire frame works together and a continuous frame would be more conservative in terms of wind loads added to the building. Furthermore, both types of frames were analyzed twice for exterior and interior moment frame conditions. In each case, the lateral forces that were added to the frame were changed based on the values that were determined for total story shear in Tables 2 and 3 on the previous pages. Using these loads that story drifts and overall drift of the building was determined and was used in the comparison with the allowable drift values determined in Tables 4 and 5. STAAD was also used to determine the controlling load combinations in each direction. The following load combinations control in the given direction. Finally, the STAAD model was one of the ways that steel members used in the building's design were verified. These steel shapes were also designed in the member and strength checks beginning on the next page. Overall, using STAAD allowed for a simplified method of member verification and drift analysis, as well as a method of comparison to the hand calculations performed for both member and frame checks.

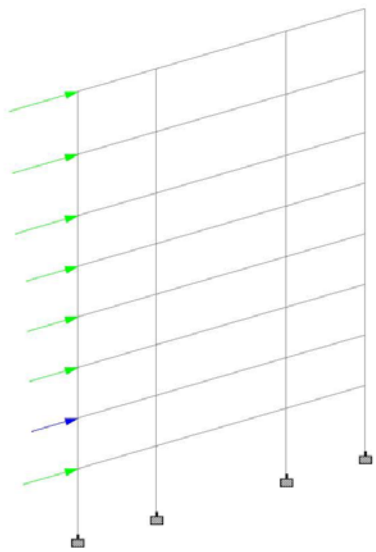


Figure 4: Frames E and F
(East – West Direction)

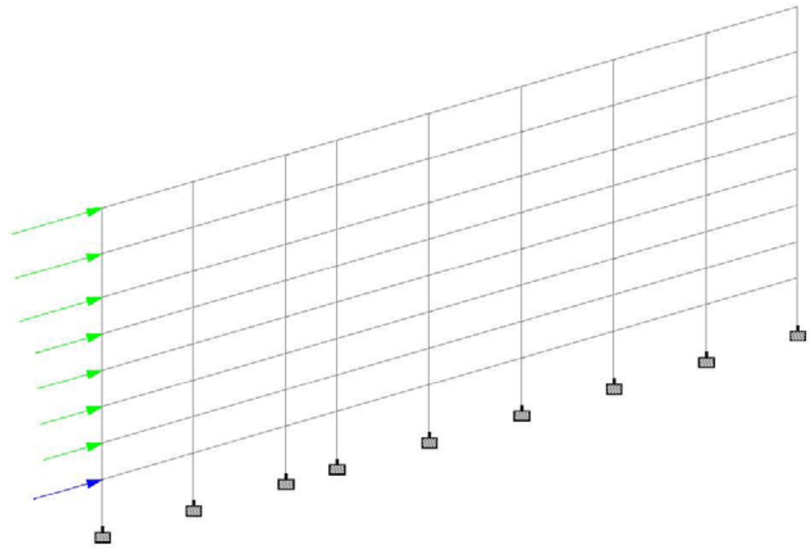


Figure 5: Frames A and C
(North – South Direction)

By the computer analysis it was determined that the controlling load combinations for each direction were as follows:

North – South Direction (Frames A thru D) : 1.2(Dead) + 1.0(Earthquake) + 1.0(Live) + 0.2(Snow)

East – West Direction (Frames E thru M) : 1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Snow)

These combinations makes sense since the shorter of the moments frames would be controlled by the wind loads since the story shears on the frame would be a result of the wind pressures on the long side of the building. These load combinations were used to verify member sizes and strengths in the next section.

Member Verification and Strength Checks

Upon completion of the portal method, the moments from the analysis were used in conjunction with moments and forces found in the STAAD analysis to verify beam, column and brace frame members throughout the building. Members in frames running both north – south and east – west were analyzed using the controlling load combination and are denoted on the calculations. Although the members were designed and checked using STAAD, it is still necessary to check the computer output so that a competent understanding of member strength can be achieved. The members that were chosen are shown in Figure 6. Both the blue and red highlighted members are the longest spanning beams in their respective moment frames and therefore were chosen for analysis. The columns, denoted by circles, were chosen based because in the portal analysis the interior columns carry twice the shear when the lateral loads are distributed throughout the frame. Finally, a brace in each of the braced frames was designed for the axial load caused by dead, wind, live and soil pressure.

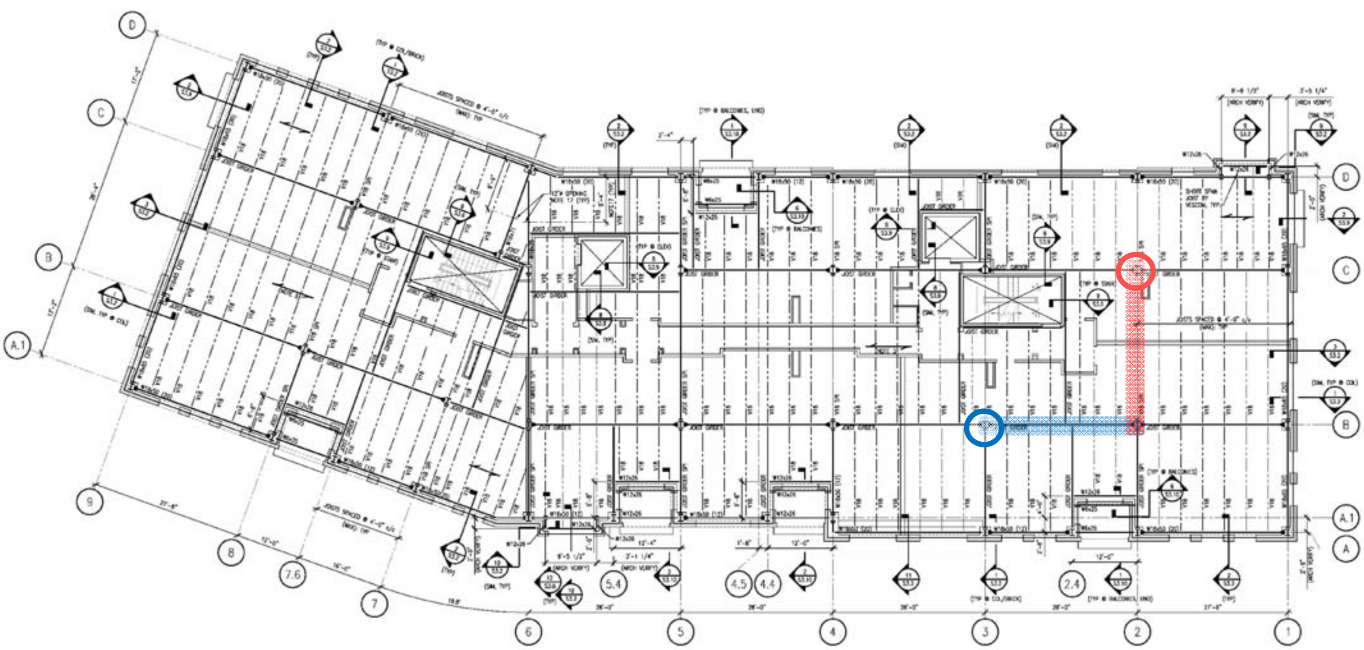


Figure 6: Members used for Verification (Frames C and F)

W18x97 (Frame F)

W18x97 (Frame C)

W14x90 (Frame F)

W14x90 (Frame C)

Frame F: Beam and Column Strength Checks (Red on Plan)

STRENGTH CHECKS:

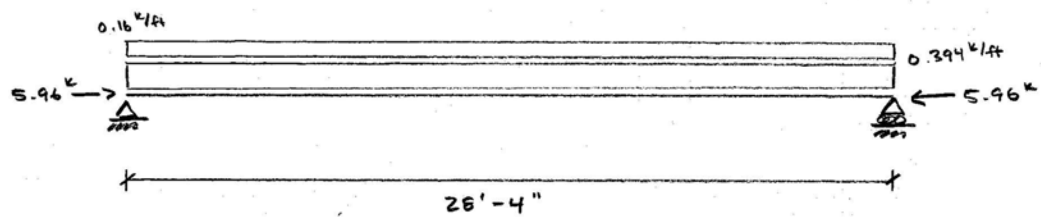
BEAM MEMBERS:

MAX AXIAL = 5.955^k (1.2D + 1.6W + L + 0.2S)

FRAME F: FLOOR S ⇒ W18×97

MAX M_z = $1481.7^{k \cdot in}$

FOR ANALYSIS THE LOAD CASE USED IS: 1.2D + 1.6W + L + 0.2S



$W_D = 0.394\ ^{k}/ft$

$W_L = 0.16\ ^{k}/ft$

$W_W = 0.52\ ^{k}/ft$

$W_S = 0.003\ ^{k}/ft$

$Z_x = 211\ in^3$

$\phi M_p = 791^{k \cdot ft}$

LOAD COMBINATION:

$$1.2(0.394) + 1.6(0.52) + 1.0(0.16) + 0.5(.003)$$

$$0.4728 + 0.832 + 0.16 + .0015 = 1.465\ ^{k}/ft$$

$$\text{MOMENT: } \frac{(1.465\ ^{k}/ft)(28.33^{ft})^2}{8} = 147^{k \cdot ft}$$

AXIAL CHECK:

FROM TABLE 6-1:

$L_b = 28.33^{ft} \Rightarrow p = 2.55 \times 10^{-3}$ ($kips^{-1}$)

$pP_n = (2.55 \times 10^{-3})(5.96^k) = 0.015 < 1.0$ ∴ OK

SHEAR CHECK:

$\phi V_n = \phi\, 0.6\, F_y\, d\, t_w$

$\phi V_n = (1.0)(0.6)(50^{ksi})(18.6\ in)(0.535\ in) = 298.53^k$

$\phi V_n = 298.53^k > 17^k$ ∴ OK

BENDING CHECK:

$L_b = 28.33^{ft}$ => TABLE 6.1

$b_x = (1.78 - 1.69) = .09(.1667) = .015 + 1.78$

$b_x = 1.795 \times 10^{-3}$ (kips^{-1})

$b_y = 4.29 \times 10^{-3}$ (kips^{-1})

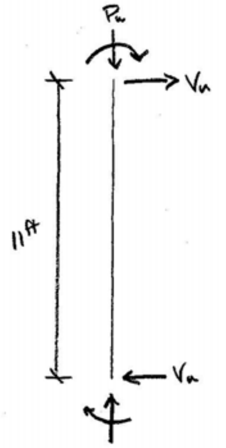
COMBINE LOADING:

$= 0.015 + (1.795 \times 10^{-3})(147) + (4.29 \times 10^{-3})(0)$

$= 0.279 < 1.0$ ∴ OK!

* OVERALL BEAM SEEMS TO BE ADEQUATELY DESIGNED.

LATERAL COLUMN CHECK: W14x90 :



$P_u = 512.4^k$

$V_u = 8.34^k$

$M_{x_1} = 44.75^{k\text{-}ft}$

$M_{x_2} = 46.33^{k\text{-}ft}$

$$\frac{KL_y}{r_y} = \frac{(1.0)(11^{ft})(12^{in}/_{ft})}{(3.70)} = 35.7 < 200 \quad \therefore \underline{\underline{OK!}}$$

SHEAR CHECK:

$\phi V_n = \phi_v \, 0.5 F_y \, d \, t_w$

$\phi V_n = (1.0)(0.6)(50^{ksi})(14\text{ in})(0.440) = 184.8^k$

$\phi V_n = 184.8^k > 8.09^k \quad \therefore \underline{\underline{OK!}}$

COMBINED LOADING:

$$B_1 = \frac{C_m}{1 - \alpha P_r/P_{e_1}} \geq 1$$

$$P_{e_x} = \frac{(\pi^2)(29000\text{ ksi})(999\text{ in}^4)}{[11 \times 12]^2} = 16410.3^k$$

$$C_{m_x} = 0.6 - 0.4\left(\frac{44.75}{46.33}\right) = 0.214$$

$$B_1 = \frac{0.214}{1 - 1.0(512.4/16410.3)} = 0.22$$

$\hookrightarrow \therefore B_1 = 1.0$

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$$\Sigma P_{e2} = R_m \frac{\Sigma HL}{\Delta H} \qquad R_m = 0.85$$

$$= \frac{0.85(12)(11)(8.34)}{(11)(12)(1/400)} = 2835.6^k$$

$$B_{2x} = \frac{1}{1 - 1.0(512.4/2835.6)}$$

$B_{2x} = 1.22$

$P_r = (1.22)(512.4) = 625.13^k \qquad M_{rx} = (1.22)(46.33) = 56.52^{k\text{-}ft}$

$p = 0.998 \times 10^{-3} \qquad b_x = 1.55 \times 10^{-3} \qquad b_y = 3.26 \times 10^{-3}$

$pP_r = (0.998 \times 10^{-3})(625.13) = 0.624 \qquad b_x M_{rx} = (1.55 \times 10^{-3})(56.52) = 0.0876$

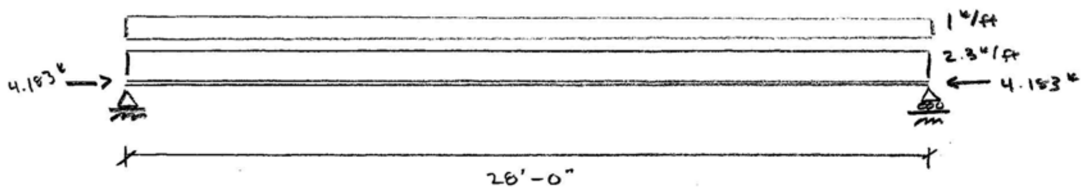
$0.624 + 0.0876 = 0.712 < 1.0 \quad \therefore \underline{\underline{OK!}}$

Frame C: Beam and Column Strength Checks (Blue on Plan)

BEAM MEMBER:

FRAME C: FLOOR 5 ⟹ W18×97

LOAD COMBINATION: 1.2D + 1.0E + 1.0L + 0.2S



$w_D = 2.3$ k/ft
$w_L = 1.0$ k/ft
$w_E = 2.68$ k/ft
$w_S = 0.033$ k/ft

LOAD COMBINATION:

$1.2(2.3) + 1.0(2.68) + 1.0(1.0) + 0.2(0.033) =$
$= 6.45$ k/ft

MOMENT: $\dfrac{(6.45\ \text{k/ft})(28\ \text{ft})^2}{8} = 632.1$ k·ft

AXIAL CHECK:

FROM TABLE 6-1:

$L_b = 4.0$ ft

$p = 0.807 \times 10^{-3}$ (kips^{-1})

$pP_n = (0.807 \times 10^{-3})(4.183^k) = .0034 < 1.0$ ∴ OK!

SHEAR CHECK:

$\phi V_n = \phi\, 0.6\, F_y\, d\, t_w$

$\phi V_n = (1.0)(0.6)(50\ \text{ksi})(18.6\ \text{in})(0.535\ \text{in}) = 298.53^k$

$\phi V_n = 298.53^k > 62.65^k$

BENDING:

$L_b = 4.0$ ft

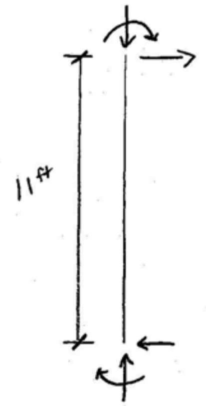
$b_x = 1.12 \times 10^{-3}$ (kips^{-1}) $b_y = 4.29 \times 10^{-3}$ (kips^{-1})

$.0034 + (1.12 \times 10^{-3})(632.1) + (4.29 \times 10^{-3})(0) = 0.711$

$0.711 < 1.0$ ∴ OK!

* BEAM SEEMS TO BE ADEQUATELY DESIGNED.

LATERAL COLUMN CHECK: W14x90



$P_u = 431^k$
$V_u = 22.4^k$
$M_x = 126.33^{k\cdot ft}$
$M_y = 0$

$M_x = 120.07^{k\cdot ft}$

$$\frac{KL_y}{r_y} = \frac{(1.0)(11ft)(12\ in/ft)}{(3.70)} = 35.7 < 200 \quad \therefore \underline{\underline{OK!}}$$

SHEAR CHECK:

$$\phi V_n = \phi_v\, 0.6\, F_y\, d\, t_w$$

$$\phi V_n = (1.0)(0.6)(50^{ksi})(14\ in)(0.440) = 184.8^k$$

$$\phi V_n = 184.8^k > 22.4^k \quad \therefore \underline{\underline{OK!}}$$

<u>COMBINED</u> <u>LOADING</u>:

AXIAL:

$$P_{ex} = \frac{(\pi^2)(29000\ ksi)(999\ in^4)}{[11ft\,(12\ in/ft)]^2} = 16410.3^k$$

$$P_{ey} = \frac{(\pi^2)(29000\ ksi)(362\ in^4)}{[11ft\,(12\ in/ft)]^2} = 5946.5^k$$

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \geq 1.0 \qquad C_{mx} = 0.6 - 0.4\left(\frac{120.07}{126.33}\right) = 0.22$$

$$B_1 = \frac{0.22}{1 - 1.0\left(\frac{431}{16410.3}\right)} = 0.23 \Rightarrow \therefore B_1 = 1.0$$

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$$\Sigma P_{e2} = R_m \frac{\Sigma HL}{\Delta H} \qquad R_m = 0.85$$

$$\Sigma P_{e2} = \frac{(0.85)(12)(11)(27.41)}{(11)(12)/400}$$

$$\Sigma P_{e2} = 7616^k$$

$$B_{2x} = \frac{1}{1 - 1.0(431/7616)}$$

$$B_{2x} = 1.06$$

$$P_r = (1.06)(431^k) = 456.86^k$$

$$M_{rx} = (1.06)(126.33) = 133.91^{k \cdot ft}$$

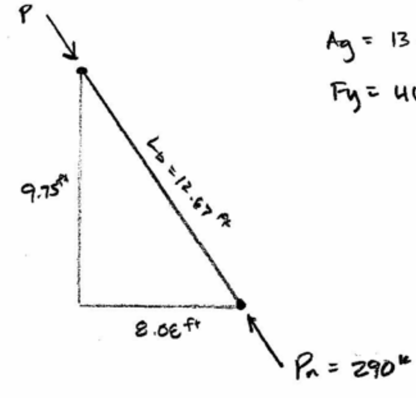
$$p = 0.998 \times 10^{-3} \qquad b_x = 1.55 \times 10^{-3} \qquad b_y = 3.26 \times 10^{-3}$$

$$pP_r = (0.998 \times 10^{-3})(456.86^k) = 0.456 > 0.2 \quad \therefore \text{H1-1a}$$

$$0.456 + (1.55 \times 10^{-3})(123.91) = 0.664 < 1.0 \quad \therefore \underline{\text{ok!}}$$

Braced Frame Strength Checks

BRACED FRAME CHECK: HSS 8 × 8 × 1/2 : FRAME A



$A_g = 13.5\ in^2$

$F_y = 46\ ksi$

$$\frac{KL_x}{r} = \frac{(1.0)(12.67)(12)}{3.04} = 50.01 < 200 \quad \therefore \underline{OK!}$$

$$\text{LIMIT: } (4.71)\sqrt{\frac{E}{F_y}} = (4.71)\sqrt{\frac{(29000)}{46}} = 118.26 > 50.01 \quad \therefore \text{INELASTIC!}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL_x}{r}\right)^2} = \frac{(\pi^2)(29000)}{(50.01)^2} = 114.44\ ksi$$

$$F_{cr} = \left[0.658^{(F_y/F_e)}\right](F_y) = \left[0.658^{(46/114.44)}\right](46) = 38.88\ ksi$$

$$\phi P_n = 0.9 A_g F_{cr} = (0.9)(13.5\ in^2)(38.88) = 472.39^k$$

$$\phi P_n = 472.4^k$$

CHECK STEEL MANUAL:

$KL = 12.67^{ft} \quad \phi P_n = 472.4^k$

BY TABLE, $\phi P_n = 479^k \quad \therefore \underline{OK!}$

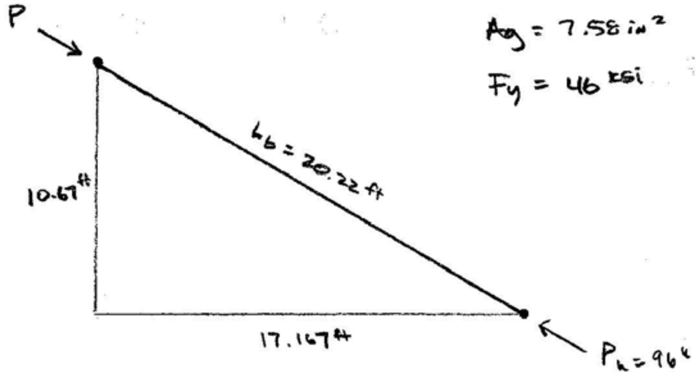
STRESS RATIO:

$$\frac{P_n}{\phi P_n} = \frac{290^k}{472.4^k} = 0.614 < 1.0 \quad \therefore \underline{OK!}$$

BRACED FRAME CHECK: HSS 6x6x3/8 : FRAME B

$A_g = 7.58\ in^2$

$F_y = 46\ ksi$



$$\frac{KL_x}{r} = \frac{(1.0)(20.22)(12)}{(2.28)} = 106.42 < 200 \quad \therefore \underline{OK!}$$

$$\text{LIMIT: } (4.71)\sqrt{\frac{E}{F_y}} = (4.71)\sqrt{\frac{(29000)}{46}} = 118.26 > 106.42 \quad \therefore \underline{\text{INELASTIC.}}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{(\pi^2)(29000)}{(106.42)^2} = 25.27\ ksi$$

$$F_{cr} = \left[0.658^{(F_y/F_e)}\right]F_y = \left[0.658^{(46/25.27)}\right](46^{ksi}) = 21.47\ ksi$$

$$\phi P_n = 0.9 A_g F_{cr} = (0.9)(7.58\ in^2)(21.47) = 146.5^k$$

$$\phi P_n = 146.5^k > 96^k$$

CHECK STEEL MANUAL: TABLE 4-4

$KL = 20.22^{ft} \quad \phi P_n = 146.5^k$

BY TABLE $\phi P_n = 149^k$ $\therefore$ <u>OK!</u>

STRESS RATIO:

$$\frac{P_u}{\phi P_n} = \frac{96^k}{146.5^k} = 0.66 < 1.0 \quad \therefore \underline{\underline{OK!}}$$

Serviceability Check (Drift Analysis)

The drift of a structure is extremely important to the overall performance of the building. Too much story drift or total drift in a structure could result in damage to the building exterior and other systems causing repairs and obviously added costs due to maintenance. Because of this the IBC and ASCE 7-05 have laid out parameters for drift control based on occupancy category, building height and structure type. In the case of Washington Park Condominiums the wind drift determined in STAAD was compared to Δ_w = H/400 for the entire building. The seismic drift, also determined in STAAD, was compared to $\Delta_S = 0.015h_{sx}$ where h_{sx} is the story height of the building at a certain level. The allowable wind drift can be found on Table 1604.3 of the IBC and the allowable drift due to seismic is found on Table 12.12-1 in ASCE 7-05. The tables below display the comparison between actual and allowable drift for both wind and seismic.

Controlling Wind Drift								
Floor	Story Height (ft)	Total Height (ft)	Story Drift (in)	Allowable Story Drift (in) Δ_{wind} = H/400	Acceptable?	Total Drift (in)	Allowable Story Drift (in) Δ_{wind} = H/400	Acceptable?
Second	14.333	14.33	0.325	< 0.43	Yes	0.325	< 0.43	Yes
Third	11.000	25.33	0.297	< 0.33	Yes	0.622	< 0.76	Yes
Fourth	11.000	36.33	0.312	< 0.33	Yes	0.934	< 1.09	Yes
Fifth	11.000	47.33	0.284	< 0.33	Yes	1.218	< 1.42	Yes
Sixth	11.000	58.33	0.28	< 0.33	Yes	1.498	< 1.75	Yes
Seventh	13.167	69.33	0.248	< 0.33	Yes	1.746	< 2.08	Yes
Eighth	13.500	82.67	0.274	< 0.40	Yes	2.020	< 2.48	Yes
Roof	13.833	96.33	0.18	< 0.41	Yes	2.200	< 2.89	Yes

Table 8: Actual Wind Drift Compared to Allowable Drift

Controlling Seismic Drift								
Floor	Story Height (ft)	Total Height (ft)	Story Drift (in)	Allowable Story Drift (in) $\Delta_{seismic} = 0.020h_{sx}$	Acceptable?	Total Drift (in)	Allowable Story Drift (in) $\Delta_{seismic} = 0.020h_{sx}$	Acceptable?
Second	14.333	14.33	0.232	< 0.287	Yes	0.232	< 0.287	Yes
Third	11.000	25.33	0.314	< 0.22	No	0.546	< 0.506	No
Fourth	11.000	36.33	0.336	< 0.22	No	0.882	< 0.727	No
Fifth	11.000	47.33	0.332	< 0.22	No	1.214	< 0.947	No
Sixth	11.000	58.33	0.311	< 0.22	No	1.525	< 1.17	No
Seventh	13.167	69.33	0.308	< 0.22	No	1.833	< 1.39	No
Eighth	13.500	82.67	0.467	< 0.267	No	2.300	< 1.65	No
Roof	13.833	96.33	0.349	< 0.273	No	2.649	< 1.93	No

Table 9: Actual Seismic Drift Compared to Allowable Drift

As displayed in the above tables the controlling wind drift in the building is acceptable at both individual floors and for the overall height of the building when compared to the code allowable drift of Δ_w = H/400. However, the controlling seismic drift in the building is greater than the code allowable value of $\Delta_S = 0.020h_{sx}$. A possible reason for this could be the fact that the individual moment frames were analyzed individually in STAAD. Another reason could be the fact that within the analysis that braced frames located on the basement and sub-basement levels were not included in the calculation since they are considered to be below grade. These braced frames primarily serve to resist the soil pressure which can be seen in the braced frame strength checks. Another possibility is the fact that the stiffness of the composite joist floor system was not taken into account. Since, the floor system serves as a diaphragm, it can also be assumed that it also has some level of stiffness and therefore would help to resist and transfer some amount of lateral loading to the columns and ultimately the foundations. Many solutions to the problem could be used including, increase column and beam sizes and possibly added some braced frames throughout the building. Regardless of the solution that is used, the issue of more than allowable story and overall drift in the structure could be studied in more depth in the proposal and research later.

Overturning Moment and Uplift

One of the issues that are rarely considered when completing lateral analysis on a structure is how the lateral loads and ensuing transfer of those lateral loads to the foundations will impact their size. The impact of the foundations that needs to be considered is caused by wind and seismic forces producing an overturning moment for the building. In turn, the overturning moment, may cause uplift within the exterior columns and foundations of the building because there is not enough dead weight on the columns and foundations to resist the overturning moment. Tables 6 and 7 list the uplift found due to wind and seismic loading in both directions.

Uplift Due to Wind Loading				
	Moment (k-ft)	Uplift Force (kips)	Dead Load Resistance (kips)	Uplift Problem?
East - West Direction	24400.21	389.36	271.24	Yes
North- South Direction	6606.64	32.67	463.79	No

Table 10: Uplift due to Wind Loading

Uplift Due to Seismic Loading				
	Moment (k-ft)	Uplift Force (kips)	Dead Load Resistance (kips)	Uplift Problem?
East - West Direction	15850.95	252.94	271.24	No
North- South Direction	15850.95	78.38	463.79	No

Table 11: Uplift due to Seismic Loading

The results of the uplift calculation show that there would be net uplift forces in the east – west direction due to wind loads. Because of this the foundations would need to be designed to resist a net uplift of 118.12 kips. Currently the foundations on the exterior of the building range from 8.5’ x 8.5’ to 13’ x 13’ and are placed 25’ below grade. This gives the foundations the ability to resist the net uplift forces applied by the overturning moment. As a result of the 2 story basement and net uplift 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 as well as the uplift. 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. Overall, it seems that the present foundations are adequate in resisting the net uplift.

Conclusion

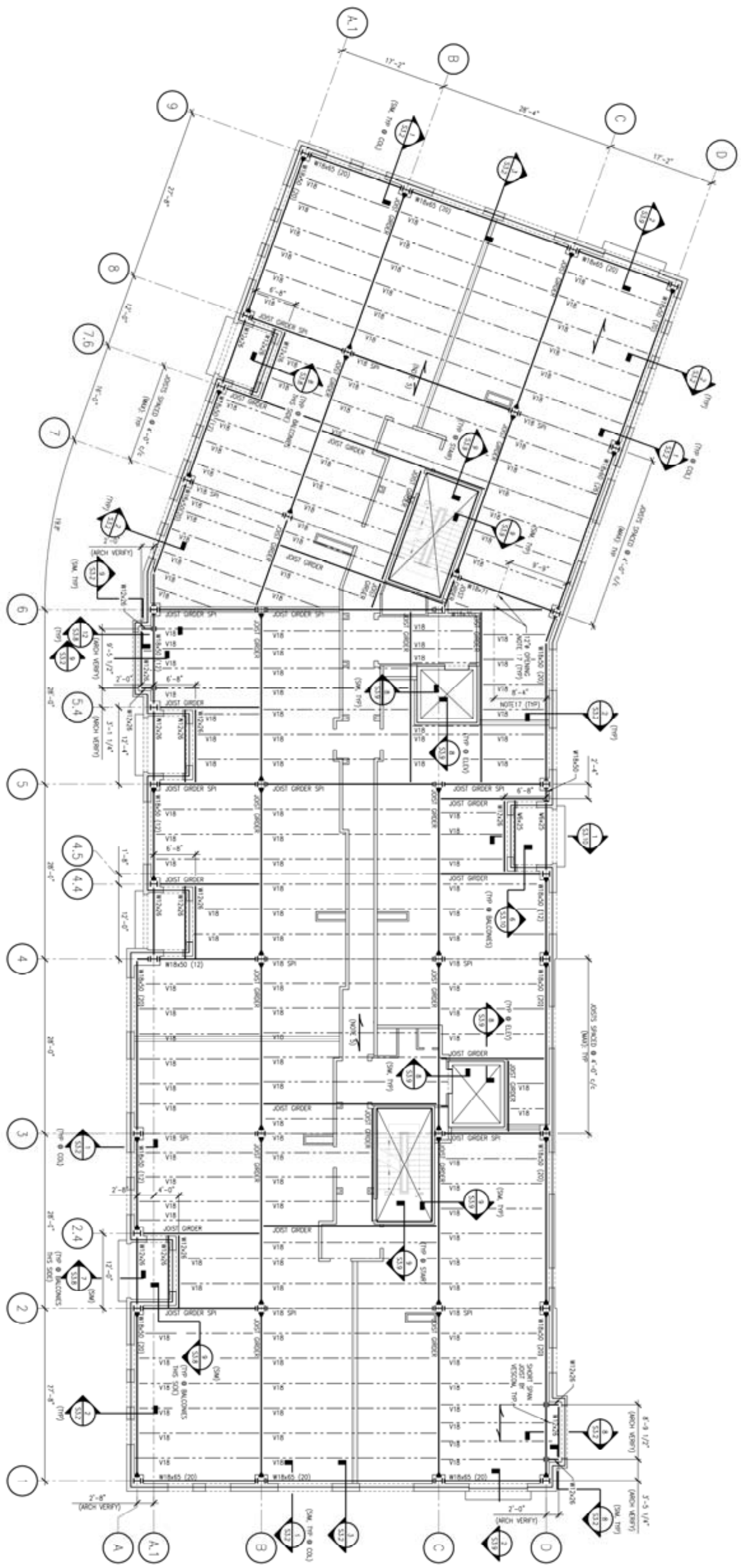
The lateral analysis and design of the moment frame structure for Washington Park Condominiums was discussed in detailed throughout this technical report. The report examines the process which was taken to arrive at design of lateral members within the frames through both hand calculations and computer modeling. Both hand calculations and computer modeling were used to determine the relative stiffness factors for each frame. From there, the direct and torsional shear forces on each level of each frame were calculated so that they could applied to the frame and compared to the results found in Technical Report #1. After determining the forces to be applied to the separate frames, it was necessary to complete a portal frame analysis along with a computer analysis. The portal frame analysis rendered shear and moments in the beams and columns of the frames. The computer model analyzed in STAAD allowed for a comparison and check of the portal frame analysis. The STAAD model was also used to determine the controlling load cases for each frame and determine the drift of both in the individual stories and the entire frame. After obtaining the values for drift from STAAD they were compared to the allowable drift limits for both wind and seismic given in the International Building Code and ASCE 7-05. The controlling wind drift was deemed acceptable by code, however the story drift and overall structure drift due to seismic loads were found to be above the acceptable code limits. Although the values did not meet the acceptable limits found in the code, it is believed that the reason for this could be contributed to the fact that the stiffness of the composite joist floor system was not taken into account. Since, the floor system serves as a diaphragm, it can also be assumed that it also has some level of stiffness and therefore would help to resist and transfer some amount of lateral loading to the columns and ultimately the foundations.

Along with the drift calculations, the design strength of several critical members of the different frames were analyzed based on the loads and moments found in the STAAD analysis. All members that were looked at were well within their ultimate capacity given by the AISC Specification for Structural Steel Buildings, 13th Edition. Finally, an overturning moment and uplift check was done to see if this would be an issue for the foundations. It was determined that in the east – west direction there may be an issue with uplift. This will be further examined in the proposal and could lead to further discussion of the foundation system used on the project.

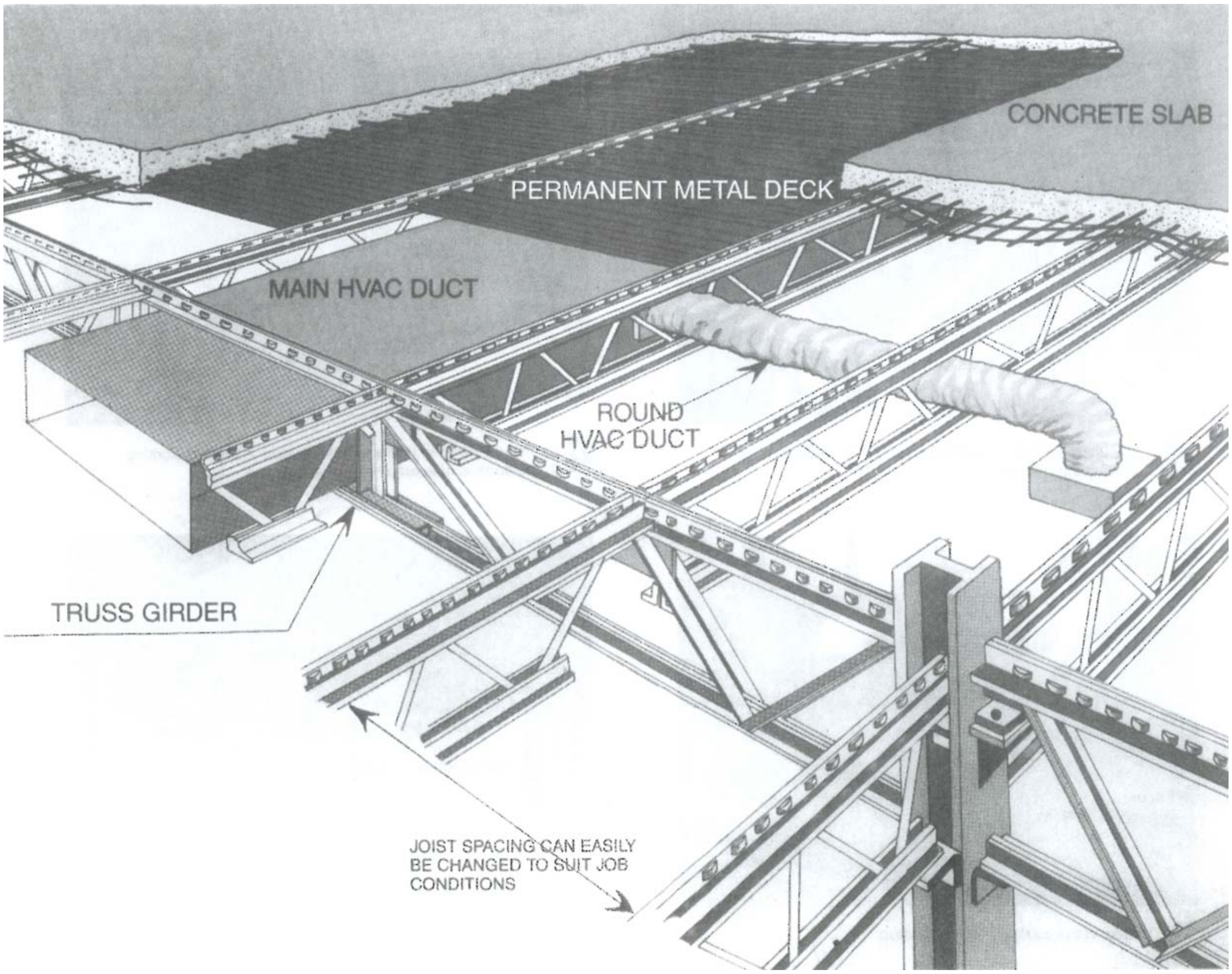
Ultimately, it was concluded that the lateral system is sufficiently designed to carry the lateral loading of the building. The drift issues will be discussed and examined in greater detail through the proposal and following research.

Appendix A: Building Layout

Typical Floor Layout



Composite Joist System Isometric Section



Appendix B: Wind and Seismic Data

Wind Loading Coefficients and Story Results

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

$q_z = 0.00256K_zK_{zt}K_dV^2I$ (lb/ft^2) $q_h = 0.00256K_hK_{zt}K_dV^2I$ (lb/ft^2)

Wind (North - South Direction)										
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-k)
Ground	0.00	0.00	0.849	0.00	0.00	0.00	0.00	0.00	128.694	6487.847
Second	14.33	13.667	0.849	14.964	10.403	-5.758	16.161	15.166	128.694	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

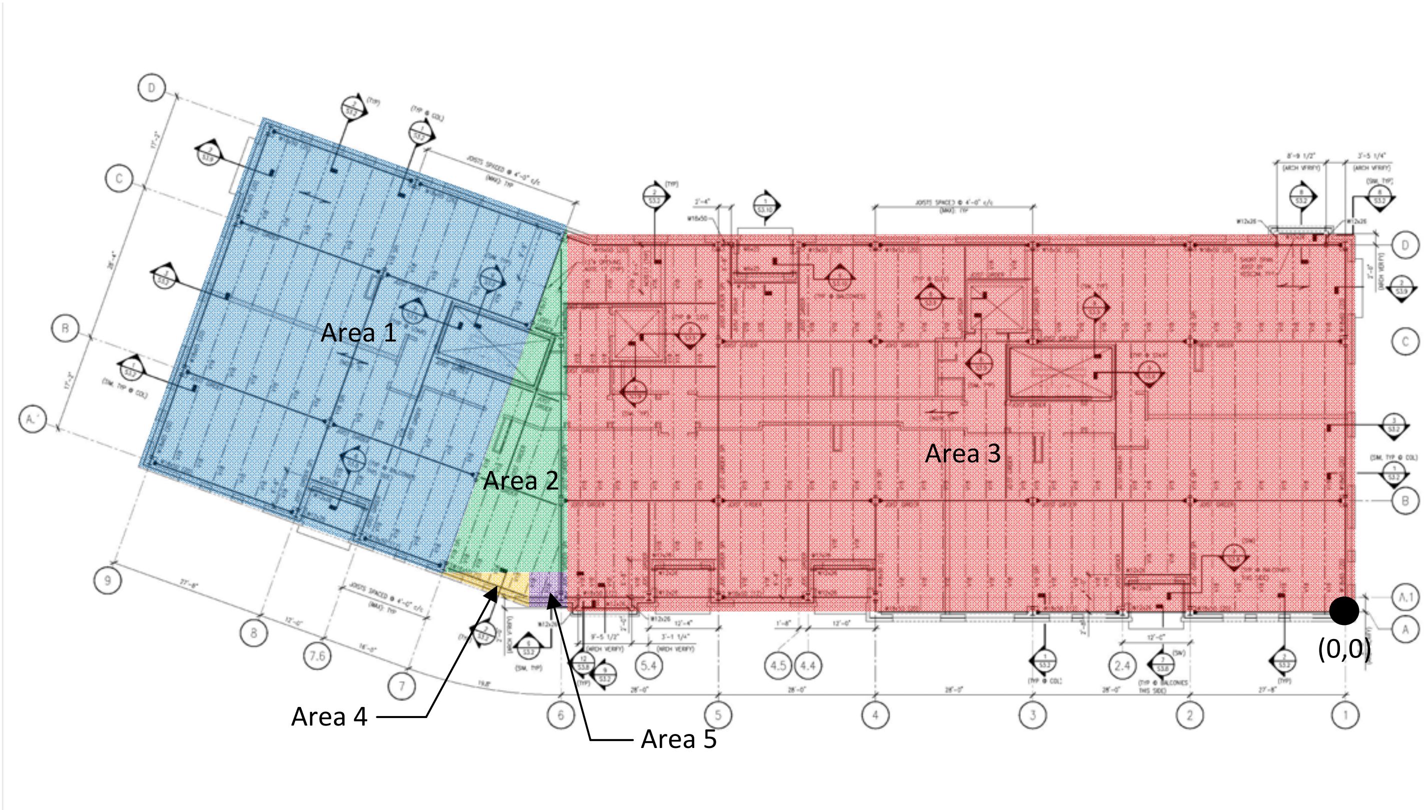
Wind (East - West Direction)										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-k)
Ground	0.00	0.00	0.849	0.00	0.00	0.00	0.00	0.000	474.909	23767.869
Second	14.33	13.667	0.849	14.964	10.140	-9.353	19.493	57.664	474.909	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

Seismic Loading Coefficients and Story Results

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

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	13.667	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				

Appendix C: Center of Mass and Rigidity



Center of mass:

Area 1: $(62.667)(55.667) = 3488.44\ ft^2$

Area 2: $\frac{1}{2}(18.09)(60) = 542.59\ ft^2$

Area 3: $(62.667)(139.667) = 8752.44\ ft^2$

Area 4: $\frac{1}{2}(5)(15) = 37.5\ ft^2$

Area 5: $(5)(6) = 30\ ft^2$

Total Area: $12850.97\ ft^2$

Center of masses:

	x (ft)	y (ft)
Area 1:	176.16	45.28
Area 2:	145.70	25.0
Area 3:	69.833	31.33
Area 4:	149.667	3.33
Area 5:	142.167	2.5

$$X_{cm} = \frac{(3488.44)(176.16) + (542.59)(145.70) + (8752.44)(69.833) + (37.5)(149.667) + (30)(142.167)}{3488.44 + 542.59 + 8752.44 + 37.5 + 30}$$

$X_{cm} = 102.30\ ft$

$$Y_{cm} = \frac{(3488.44)(45.28) + (542.59)(25) + 8752.44(31.33) + (37.5)(3.33) + (30)(2.5)}{12850.97}$$

$Y_{cm} = 34.7\ ft$

Torsion Constants							
Floor	Center of Rigidity		Center of Mass		I_X (in^4)	I_y (in^4)	I_P (in^4)
	X_R (ft)	Y_R (ft)	X_R (ft)	Y_R (ft)			
Second	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Third	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Fourth	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Fifth	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Sixth	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Seventh	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Eighth	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41
Roof	104.92	39.89	102.3	34.7	2837888.75	353494.66	3191383.41

RELATIVE STIFFNESS & CENTER OF RIGIDITY

K FACTORS

INTERIOR NORTH SOUTH FRAME: $K = 1/\Delta = 1/0.052$

$K = 19.23$

EXTERIOR NORTH SOUTH FRAME: $K = 1/\Delta = 1/0.103$

$K = 9.71$

INTERIOR 3 BAY FRAME: $K = 1/\Delta = 1/0.045$

$K = 22.22$

EXTERIOR 3 BAY FRAME: $K = 1/\Delta = 1/0.059$

$K = 16.95$

$$d_x = \frac{k_1 d_1 + k_2 d_2 + k_3 d_3}{\Sigma K_x}$$

$$\frac{(0.059)(0) + (0.045)(27.667') + (0.045)(55.667') + (0.045)(83.667') + (0.045)(111.667') + (0.045)(139.667') + (0.045)(149.9167') + (0.045)(176.22') + (0.059)(202.22)}{(0.059)(2) + (0.045)(7)}$$

16.95

$$d_x = \frac{45.432}{0.433} = 104.92'$$

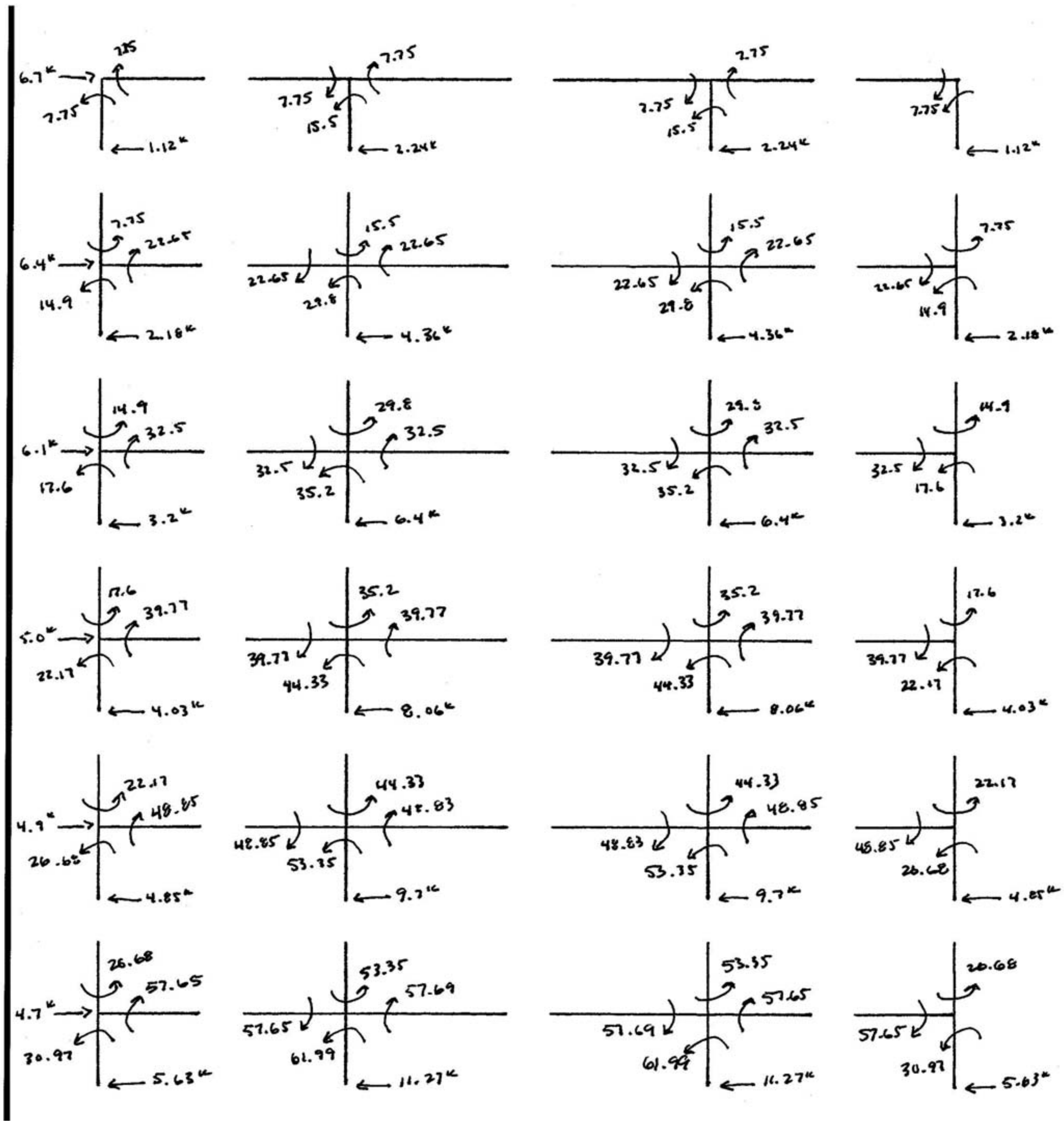
$$d_y = \frac{k_1 d_1 + k_2 d_2 + k_3 d_3}{\Sigma K_y}$$

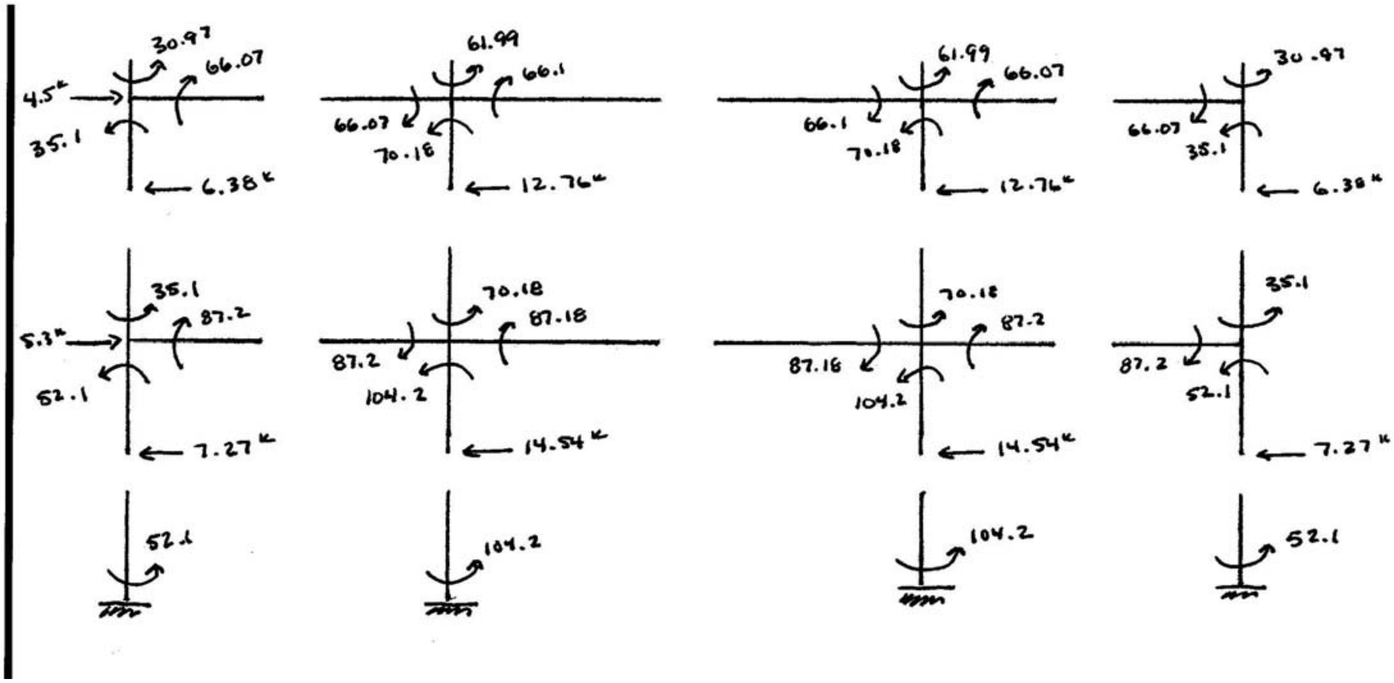
$$\frac{(0.103)(0) + (0.052)(17.1667') + (0.052)(45.5') + (0.103)(62.667') + (0.103)(13.37') + (0.052)(29.422') + (0.052)(56.01') + (.103)(81.71') + (0.059)(53.75') + (0.045)(44.54') + (0.045)(34.71')}{(4)(.103) + 4(0.052) + 2(0.045) + 0.059}$$

$$d_y = \frac{30.682}{0.769} = 39.89'$$

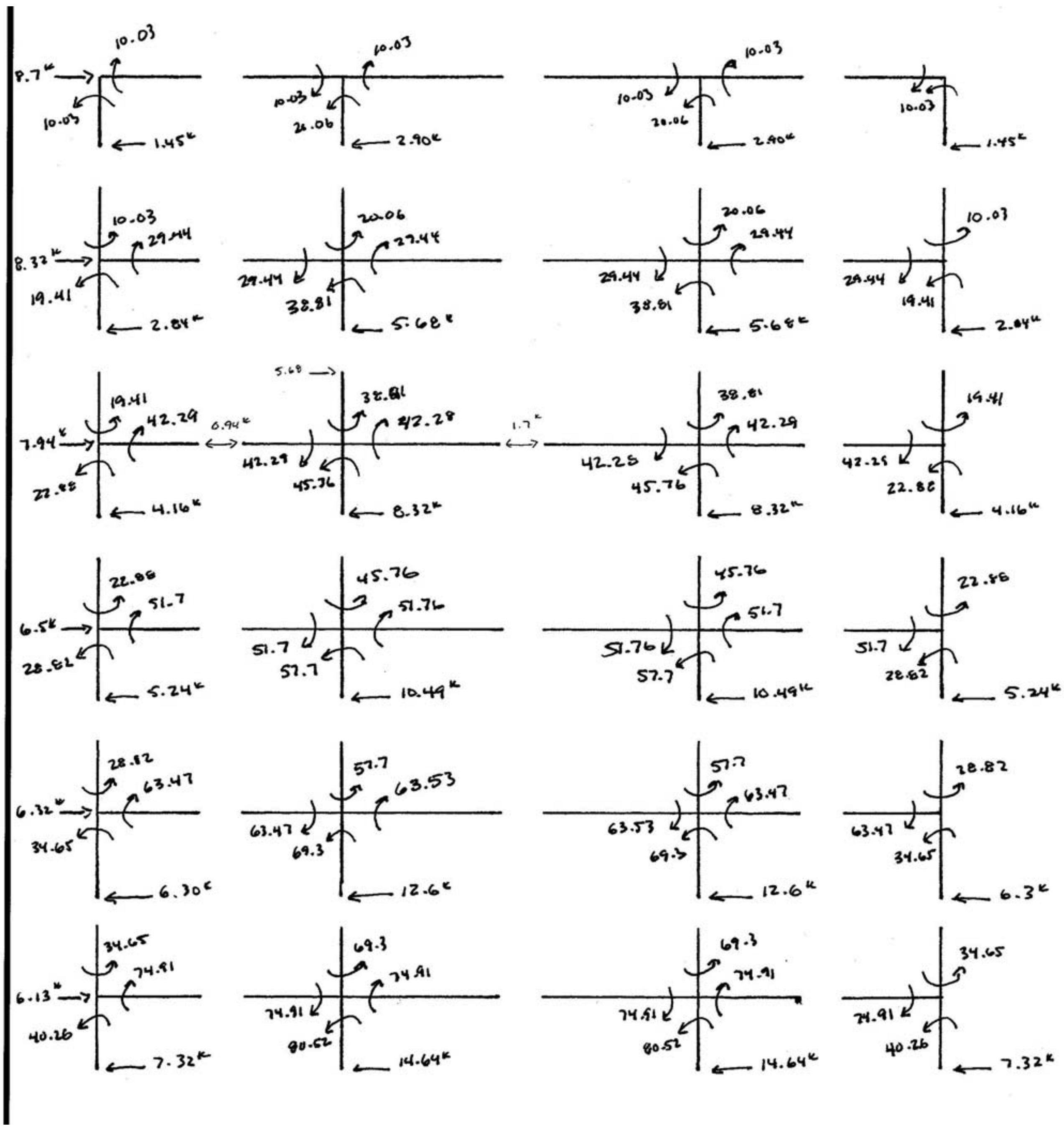
Appendix D: Portal Frame Analysis Calculations

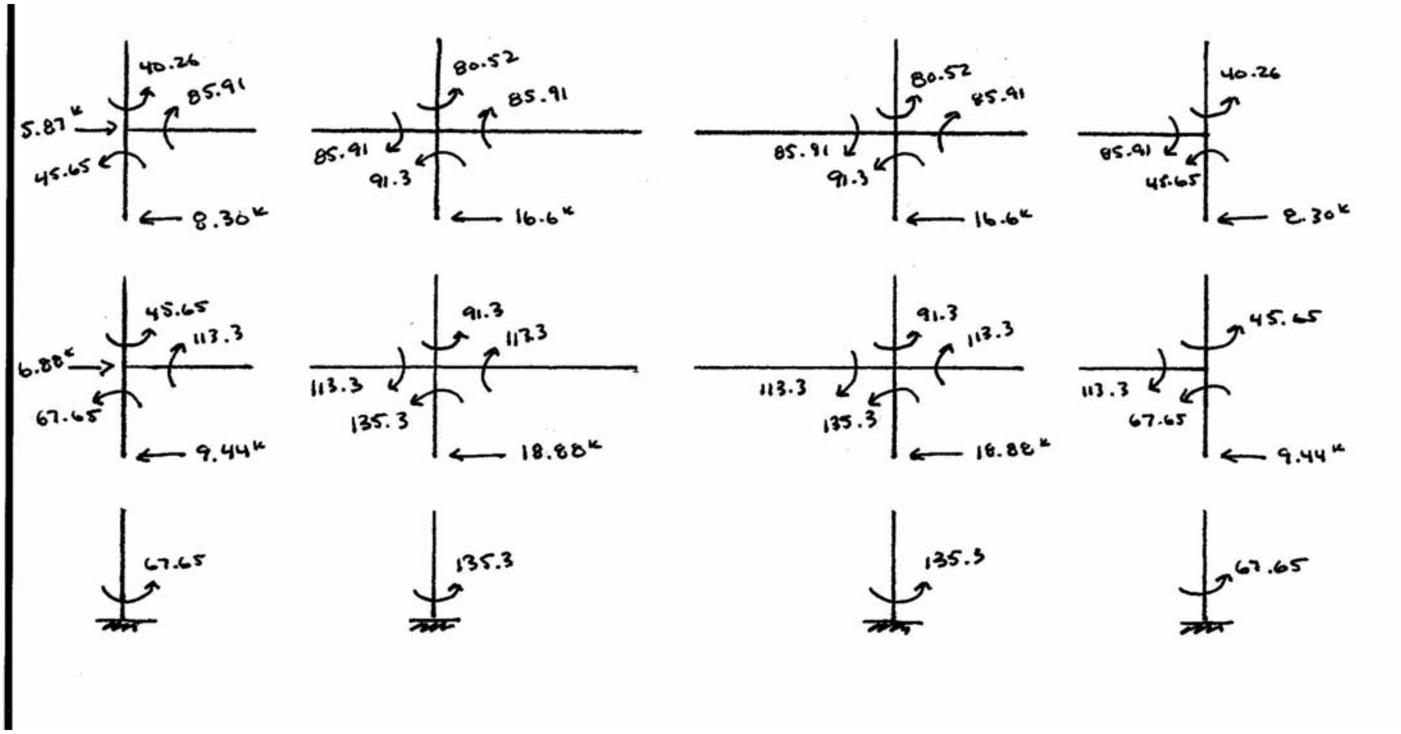
Portal Frame "E"




30.97
66.07
4.5k
35.1
6.38k
61.99
66.1
66.07
70.18
12.76k
61.99
66.07
66.1
70.18
12.76k
30.97
66.07
35.1
6.38k
35.1
87.2
5.3k
52.1
7.27k
70.18
87.18
87.2
104.2
14.54k
70.18
87.2
87.18
104.2
14.54k
35.1
87.2
52.1
7.27k
52.1
104.2
104.2
52.1

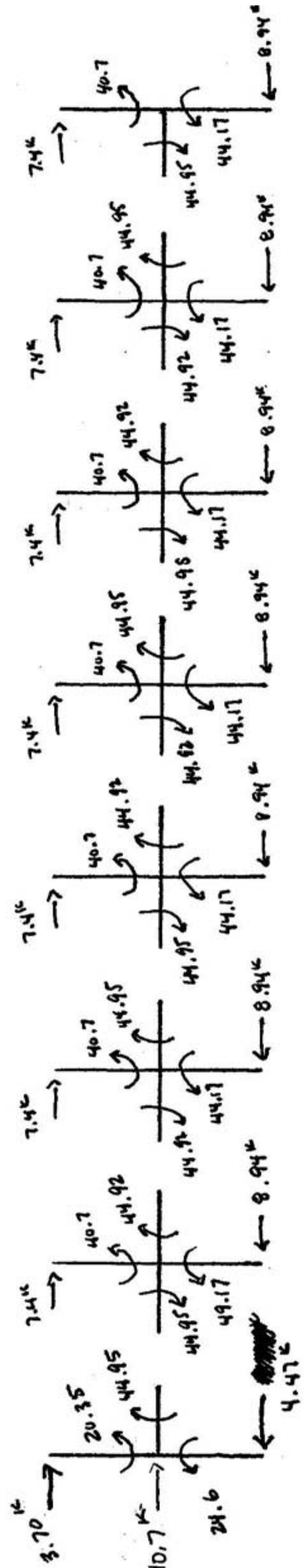
Portal Frame "F"




40.26
85.91
5.87k
45.65
8.30k
80.52
85.91
85.91
91.3
16.6k
80.52
85.91
85.91
91.3
16.6k
40.26
85.91
45.65
8.30k
45.65
113.3
6.88k
67.65
9.44k
91.3
113.3
113.3
135.3
18.88k
91.3
113.3
113.3
135.3
18.88k
45.65
113.3
67.65
9.44k
67.65
135.3
135.3
67.65

Partial Portal Frame "A"

NORTH - SOUTH CONTINUOUS EXTERIOR FRAME (LEVEL 6)



BEAM MOMENTS: 44.95 ft·k & 44.92 ft·k

COLUMN MOMENTS:

$6^{th} - 7^{th} = 20.35$ ft·k & 40.7 ft·k

$5^{th} - 6^{th} = 24.6$ ft·k & 49.17 ft·k

SHEAR AT LEVEL 7: OUTSIDE

$51.86\left(\frac{1}{14}\right) = 3.70^k$

SHEAR AT LEVEL 6: OUTSIDE

$62.56\left(\frac{1}{14}\right) = 4.47^k$

SHEAR AT LEVEL 7: INSIDE

$51.86\left(\frac{1}{7}\right) = 7.4^k$

SHEAR AT LEVEL 6: INSIDE

$62.56\left(\frac{1}{7}\right) = 8.94^k$

Appendix E: Torsional Force Tables

Wind Torsional Forces

Wind (East - West Direction)										
Floor	Moment (k -ft)	Frames								
		A & D			B & C			E		
		k (k/in)	X_i (ft)	Torsional Shear (kips)	k (k/in)	X_i (ft)	Torsional Shear (kips)	k (k/in)	X_i (ft)	Torsional Shear (kips)
Second	-219.700	9.71	3.81	-0.003	19.23	3.81	-0.005	16.95	104.92	-0.122
Third	-187.559	9.71	3.81	-0.002	19.23	3.81	-0.004	16.95	104.92	-0.105
Fourth	-195.685	9.71	3.81	-0.002	19.23	3.81	-0.004	16.95	104.92	-0.109
Fifth	-201.968	9.71	3.81	-0.002	19.23	3.81	-0.005	16.95	104.92	-0.113
Sixth	-207.279	9.71	3.81	-0.002	19.23	3.81	-0.005	16.95	104.92	-0.116
Seventh	-253.559	9.71	3.81	-0.003	19.23	3.81	-0.006	16.95	104.92	-0.141
Eighth	-265.824	9.71	3.81	-0.003	19.23	3.81	-0.006	16.95	104.92	-0.148
Roof	-277.833	9.71	3.81	-0.003	19.23	3.81	-0.006	16.95	104.92	-0.155

Wind (East - West Direction)										
Floor	Moment (k -ft)	Frames								
		F			G			H		
		k (k/in)	X_i (ft)	Torsional Shear (kips)	k (k/in)	X_i (ft)	Torsional Shear (kips)	k (k/in)	X_i (ft)	Torsional Shear (kips)
Second	-219.700	22.22	77.253	-0.118	22.22	49.253	-0.075	22.22	21.253	-0.033
Third	-187.559	22.22	77.253	-0.101	22.22	49.253	-0.064	22.22	21.253	-0.028
Fourth	-195.685	22.22	77.253	-0.105	22.22	49.253	-0.067	22.22	21.253	-0.029
Fifth	-201.968	22.22	77.253	-0.109	22.22	49.253	-0.069	22.22	21.253	-0.030
Sixth	-207.279	22.22	77.253	-0.111	22.22	49.253	-0.071	22.22	21.253	-0.031
Seventh	-253.559	22.22	77.253	-0.136	22.22	49.253	-0.087	22.22	21.253	-0.038
Eighth	-265.824	22.22	77.253	-0.143	22.22	49.253	-0.091	22.22	21.253	-0.039
Roof	-277.833	22.22	77.253	-0.149	22.22	49.253	-0.095	22.22	21.253	-0.041

Wind (East - West Direction)										
Floor	Moment (k -ft)	Frames								
		I			J			K		
		k (k/in)	X_i (ft)	Torsional Shear (kips)	k (k/in)	X_i (ft)	Torsional Shear (kips)	k (k/in)	X_i (ft)	Torsional Shear (kips)
Second	-219.700	22.22	-6.75	0.010	22.22	-34.75	0.053	22.22	-45	0.069
Third	-187.559	22.22	-6.75	0.009	22.22	-34.75	0.045	22.22	-45	0.059
Fourth	-195.685	22.22	-6.75	0.009	22.22	-34.75	0.047	22.22	-45	0.061
Fifth	-201.968	22.22	-6.75	0.009	22.22	-34.75	0.049	22.22	-45	0.063
Sixth	-207.279	22.22	-6.75	0.010	22.22	-34.75	0.050	22.22	-45	0.065
Seventh	-253.559	22.22	-6.75	0.012	22.22	-34.75	0.061	22.22	-45	0.079
Eighth	-265.824	22.22	-6.75	0.012	22.22	-34.75	0.064	22.22	-45	0.083
Roof	-277.833	22.22	-6.75	0.013	22.22	-34.75	0.067	22.22	-45	0.087

Wind (East - West Direction)							
Floor	Moment (k -ft)	Frames					
		L			M		
		k (k/in)	X_i (ft)	Torsional Shear (kips)	k (k/in)	X_i (ft)	Torsional Shear (kips)
Second	-219.700	22.22	-71.3	0.109	16.95	-97.3	0.114
Third	-187.559	22.22	-71.3	0.093	16.95	-97.3	0.097
Fourth	-195.685	22.22	-71.3	0.097	16.95	-97.3	0.101
Fifth	-201.968	22.22	-71.3	0.100	16.95	-97.3	0.104
Sixth	-207.279	22.22	-71.3	0.103	16.95	-97.3	0.107
Seventh	-253.559	22.22	-71.3	0.126	16.95	-97.3	0.131
Eighth	-265.824	22.22	-71.3	0.132	16.95	-97.3	0.137
Roof	-277.833	22.22	-71.3	0.138	16.95	-97.3	0.144

Wind (North - South Direction)										
		Frames								
		A			B			C		
Floor	Moment (k -ft)	k (k/in)	Y_i (ft)	Torsional Shear (kips)	k (k/in)	Y_i (ft)	Torsional Shear (kips)	k (k/in)	Y_i (ft)	Torsional Shear (kips)
Second	-129.775	9.71	22.77	-0.009	19.23	5.603	-0.004	19.23	-22.723	0.018
Third	-112.294	9.71	22.77	-0.008	19.23	5.603	-0.004	19.23	-22.723	0.015
Fourth	-118.232	9.71	22.77	-0.008	19.23	5.603	-0.004	19.23	-22.723	0.016
Fifth	-122.827	9.71	22.77	-0.009	19.23	5.603	-0.004	19.23	-22.723	0.017
Sixth	-126.703	9.71	22.77	-0.009	19.23	5.603	-0.004	19.23	-22.723	0.017
Seventh	-155.652	9.71	22.77	-0.011	19.23	5.603	-0.005	19.23	-22.723	0.021
Eighth	-163.867	9.71	22.77	-0.011	19.23	5.603	-0.006	19.23	-22.723	0.022
Roof	-171.893	9.71	22.77	-0.012	19.23	5.603	-0.006	19.23	-22.723	0.024

Wind (North - South Direction)										
		Frames								
		D			E, F, G, H, I & J			K		
Floor	Moment (k -ft)	k (k/in)	Y_i (ft)	Torsional Shear (kips)	k (k/in)	Y_i (ft)	Torsional Shear (kips)	k (k/in)	Y_i (ft)	Torsional Shear (kips)
Second	-129.775	9.71	-39.89	0.016	22.22	-8.556	0.008	22.22	-5.18	0.005
Third	-112.294	9.71	-39.89	0.014	22.22	-8.556	0.007	22.22	-5.18	0.004
Fourth	-118.232	9.71	-39.89	0.014	22.22	-8.556	0.007	22.22	-5.18	0.004
Fifth	-122.827	9.71	-39.89	0.015	22.22	-8.556	0.007	22.22	-5.18	0.004
Sixth	-126.703	9.71	-39.89	0.015	22.22	-8.556	0.008	22.22	-5.18	0.005
Seventh	-155.652	9.71	-39.89	0.019	22.22	-8.556	0.009	22.22	-5.18	0.006
Eighth	-163.867	9.71	-39.89	0.020	22.22	-8.556	0.010	22.22	-5.18	0.006
Roof	-171.893	9.71	-39.89	0.021	22.22	-8.556	0.010	22.22	-5.18	0.006

Wind (North - South Direction)							
		Frames					
		L			M		
Floor	Moment (k -ft)	k (k/in)	Y_i (ft)	Torsional Shear (kips)	Y_i (ft)	X_i (ft)	Torsional Shear (kips)
Second	-129.775	22.22	4.403	-0.004	16.95	13.865	-0.010
Third	-112.294	22.22	4.403	-0.003	16.95	13.865	-0.008
Fourth	-118.232	22.22	4.403	-0.004	16.95	13.865	-0.009
Fifth	-122.827	22.22	4.403	-0.004	16.95	13.865	-0.009
Sixth	-126.703	22.22	4.403	-0.004	16.95	13.865	-0.009
Seventh	-155.652	22.22	4.403	-0.005	16.95	13.865	-0.011
Eighth	-163.867	22.22	4.403	-0.005	16.95	13.865	-0.012
Roof	-171.893	22.22	4.403	-0.005	16.95	13.865	-0.013

Seismic Torsional Forces

Seismic (North- South Direction)													
Floor	Moment (k-ft)	Frames											
		A & D			B & C			E			F		
		k (k/in)	X_i (ft)	Torsional Shear (kips)	k (k/in)	X_i (ft)	Torsional Shear (kips)	k (k/in)	X_i (ft)	Torsional Shear (kips)	k (k/in)	X_i (ft)	Torsional Shear (kips)
Second	18.937	9.71	3.81	0.000	19.23	3.81	0.000	16.95	104.92	0.011	22.22	77.253	0.010
Third	28.931	9.71	3.81	0.000	19.23	3.81	0.001	16.95	104.92	0.016	22.22	77.253	0.016
Fourth	46.087	9.71	3.81	0.001	19.23	3.81	0.001	16.95	104.92	0.026	22.22	77.253	0.025
Fifth	64.289	9.71	3.81	0.001	19.23	3.81	0.001	16.95	104.92	0.036	22.22	77.253	0.035
Sixth	84.198	9.71	3.81	0.001	19.23	3.81	0.002	16.95	104.92	0.047	22.22	77.253	0.045
Seventh	108.923	9.71	3.81	0.001	19.23	3.81	0.003	16.95	104.92	0.061	22.22	77.253	0.059
Eighth	136.705	9.71	3.81	0.002	19.23	3.81	0.003	16.95	104.92	0.076	22.22	77.253	0.074
Roof	162.542	9.71	3.81	0.002	19.23	3.81	0.004	16.95	104.92	0.091	22.22	77.253	0.087

Seismic (East - West Direction)													
Floor	Moment (k-ft)	Frames											
		A			C			E, F, G, H, I & J			D		
		k (k/in)	Y_i (ft)	Torsional Shear (kips)	k (k/in)	Y_i (ft)	Torsional Shear (kips)	k (k/in)	Y_i (ft)	Torsional Shear (kips)	k (k/in)	Y_i (ft)	Torsional Shear (kips)
Second	37.512	9.71	22.77	0.003	19.23	-22.77	-0.005	22.22	-8.556	-0.002	9.71	-39.89	-0.005
Third	57.310	9.71	22.77	0.004	19.23	-22.77	-0.008	22.22	-8.556	-0.003	9.71	-39.89	-0.007
Fourth	91.294	9.71	22.77	0.006	19.23	-22.77	-0.013	22.22	-8.556	-0.005	9.71	-39.89	-0.011
Fifth	127.351	9.71	22.77	0.009	19.23	-22.77	-0.017	22.22	-8.556	-0.008	9.71	-39.89	-0.015
Sixth	166.789	9.71	22.77	0.012	19.23	-22.77	-0.023	22.22	-8.556	-0.010	9.71	-39.89	-0.020
Seventh	215.767	9.71	22.77	0.015	19.23	-22.77	-0.030	22.22	-8.556	-0.013	9.71	-39.89	-0.026
Eighth	270.802	9.71	22.77	0.019	19.23	-22.77	-0.037	22.22	-8.556	-0.016	9.71	-39.89	-0.033
Roof	321.982	9.71	22.77	0.022	19.23	-22.77	-0.044	22.22	-8.556	-0.019	9.71	-39.89	-0.039