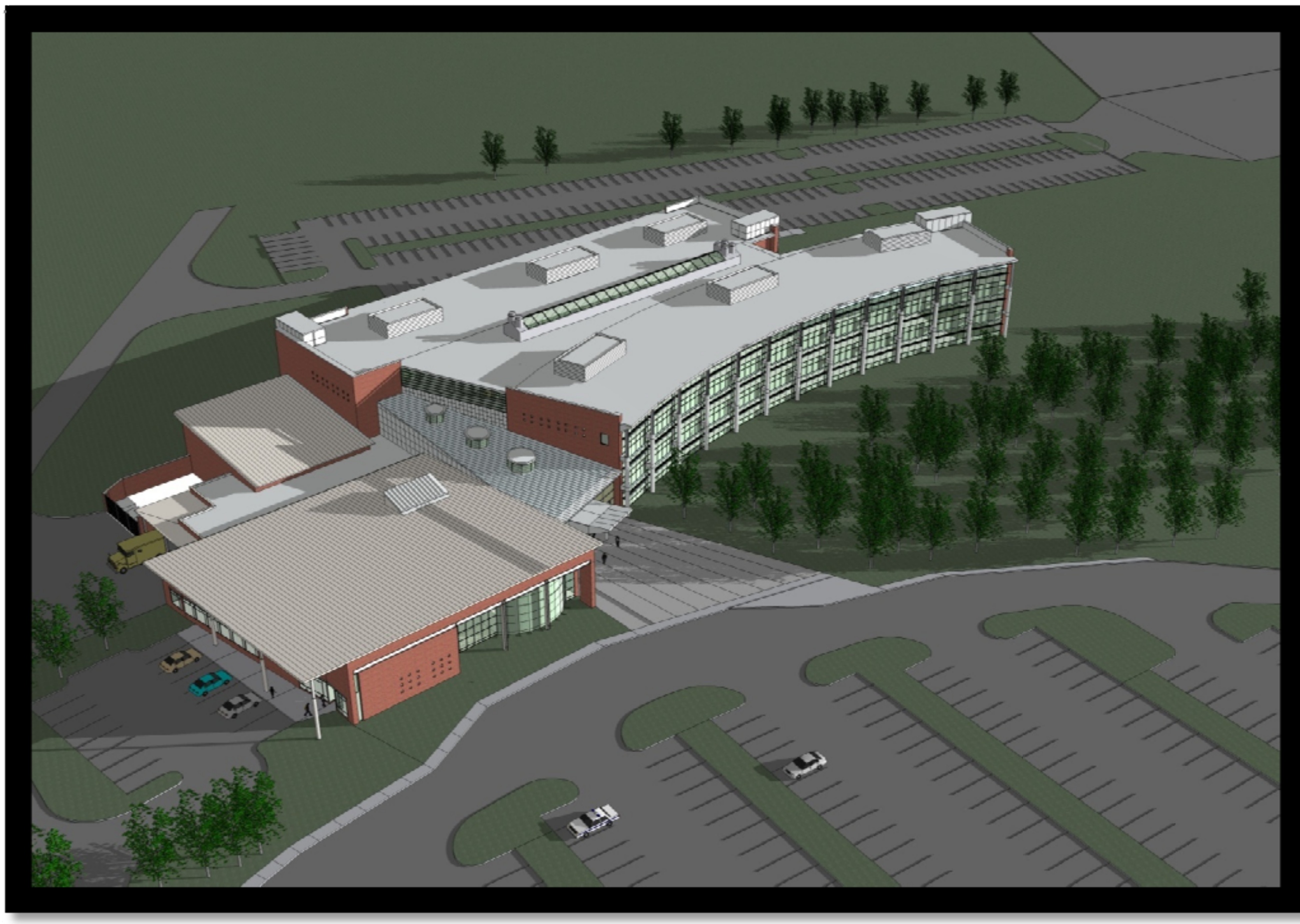
The Edward L Kelly Leadership Center
Prince William County School Administration Center



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Technical Report 3

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Introduction

The Edward L Kelly building is an administrative building for the Prince William County Public Schools. The building is located in the northern Virginian city of Manassas. Currently housed in separate facilities, the architectural goal of the building is to combine the several School Administration functions into one single facility. The facility is light-filled with a 3-story atrium with skylights and a clerestory entrance. The building program contains flexible office space for 500 employees as well as meeting and training rooms for the district.

The building is composed of essentially three distinct sections. The gross square footage of the building is approximately 150,000 square feet. There is a one-story section on the west of the building plan. It is here that the main School Board meeting rooms, meeting rooms, exercise, kitchen, and "public" spaces are located. This section of the building is approximately 25,000 square feet and structurally independent from the rest of the building. On the northern portion of the building is a three-story, rectangular, 17,000 square foot section of the building where offices for district employees are located. The southern share consists of another three-story building that is radial in nature and has a footprint of approximately 19000 square feet. An atrium and walkways separate the two three-story buildings by approximately 36 feet at its midpoint and represent another 20,000 square feet of the building. The two three-story buildings are approximately 60 feet in width and the rectangular and radial buildings are 265 feet and 295 feet, respectively.

Because the two three-story buildings are separated by a relatively large distance and only connected by a few small walkways above the atrium, the rudimentary assumption is made that the two buildings act independently of each other under lateral loading. Figure 1 outlines the area under analysis.

Figure 1. Overall Architectural Floor Plan with Area Under Analysis Outlined

Executive Summary

The intent of this report if to further analyze the lateral structural system of the Edward L Kelly Leadership Center located in Manassas, VA. Code provision (ASCE7, IBC) interpretations as well as initial calculations will be reviewed from Technical Report 1 and incorporated into this technical report, as necessary. Attempts to eliminate errors from Technical Report 1 are made in this report. This primarily includes a more detailed calculation of the seismic weights of the building.

A different approach is made in this report compared to the previous. For instance, this report makes the assumption that there exist three distinctly separate lateral resisting sections in the total building, one for each "individual building." This means that the one-story section, three-story rectangular section, and three-story radial section act separately in a lateral analysis. The basis for this assumption stems from relatively independent nature of the systems. The two three-story buildings are separated by a large atrium and connected at three small points by walkways. The one-story building does not share any structural components with the adjacent building. That is, no column or beam from the one-story building has any connection with any columns of beams from the adjacent building.

A lateral analysis is made in this report on the northern section of the building. This portion of the building is outlined in Figure 1 and will be referenced as the "rectangular building" in this report.

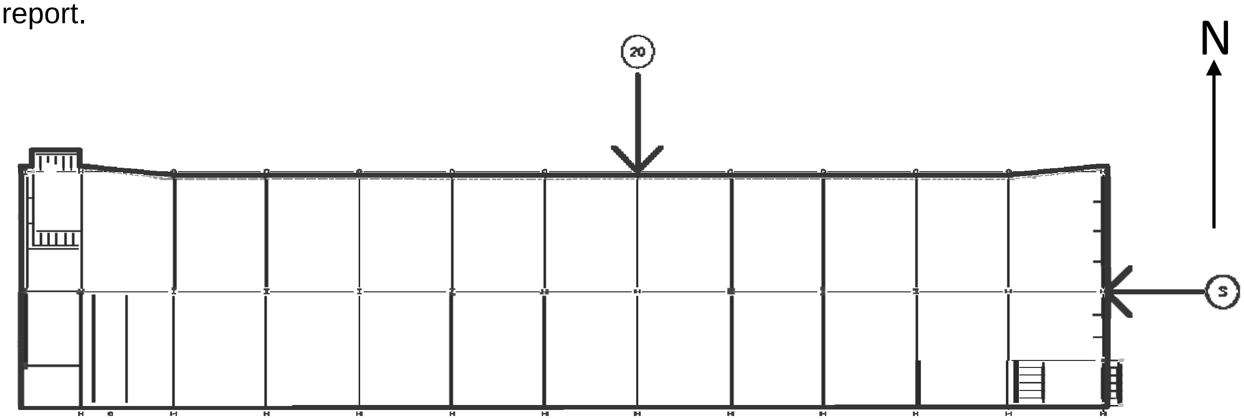


Figure 2. Frames Along Column Lines "20" (N-S) and "S" (E-W) Noted

Figure 2 shows the lateral structural elements of the building as well as the frames under investigation. In the North-South direction, that frame is along column line "20", and in the East-West direction, the frame is along column line "S".

Hand calculations show that in the North-South direction, wind is the controlling lateral force in the system. In the East-West direction, seismic forces controlled. This seems rational despite the low seismicity of the region due to the much larger seismic weight in the East-West direction and much less in the North-South direction.

The building was also modeled using the computer software RAM Structural System. Much of the output data is echoed in hand calculations. Some data cannot be directly compared because of the two distinctly differing methods of analysis. Hand calculations utilize a strictly tributary calculation, where the RAM model analyzes the building much more in depth. However, seismic weights and wind pressures match very closely with slight error most likely due to the more precise nature of the model.

Spot checks of the frames under consideration were compiled using hand calculations and computer models. Hand calculations for each frame were completed utilizing the portal frame method of approximate analysis. In addition, the computer program RISA was used to model the frames for more exact computation. The portal method yields results much more approximate than the computer model because of simplifications and standardizing the steel members (such as stiffness). Thus, the portal method is included in the appendix, but references will be made to the computer model when analyzing the frame.

Story drift in the RISA model were calculated to be 0.117 inches in the East-West frame and 0.156 inches in the North-South direction. Both of these values are well below the prescribed maximum of H/400 = (46)(12)/400 = 1.38 inches.

The centers of mass and centers of rigidity are assumed to exist at nearly the same location. Therefore, torsional effects are not considered in this report. The RAM data confirmed that their relative locations are very close.

Figure 3 shows a 3D structural rendering of the building under consideration. Figures 4 and 5 show the other two parts of the building. The latter, however, not considered for analysis in this report.

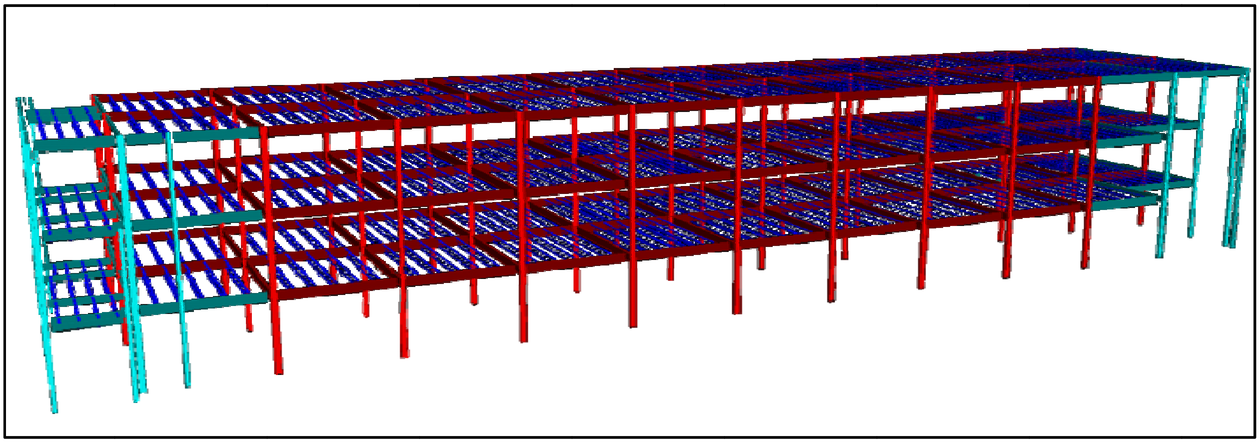


Figure 3. Rectangular Building (Under Analysis)

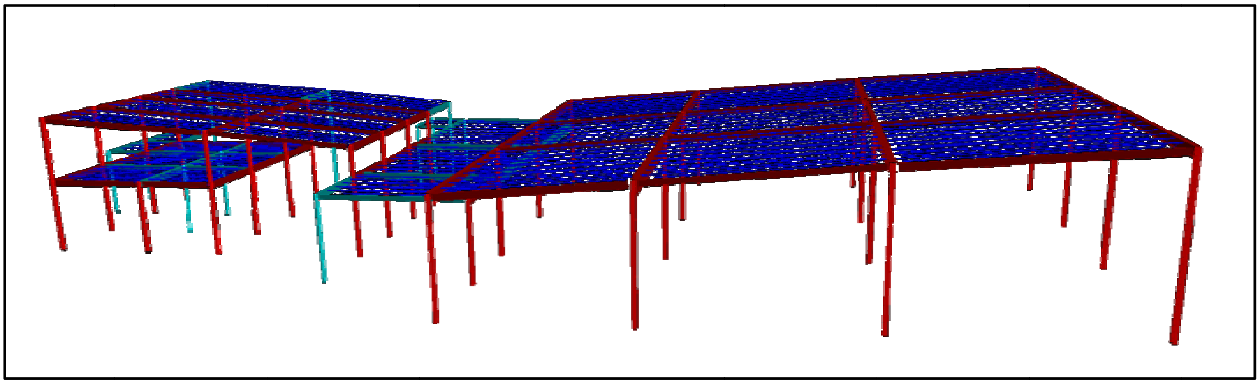


Figure 4. One Story Building

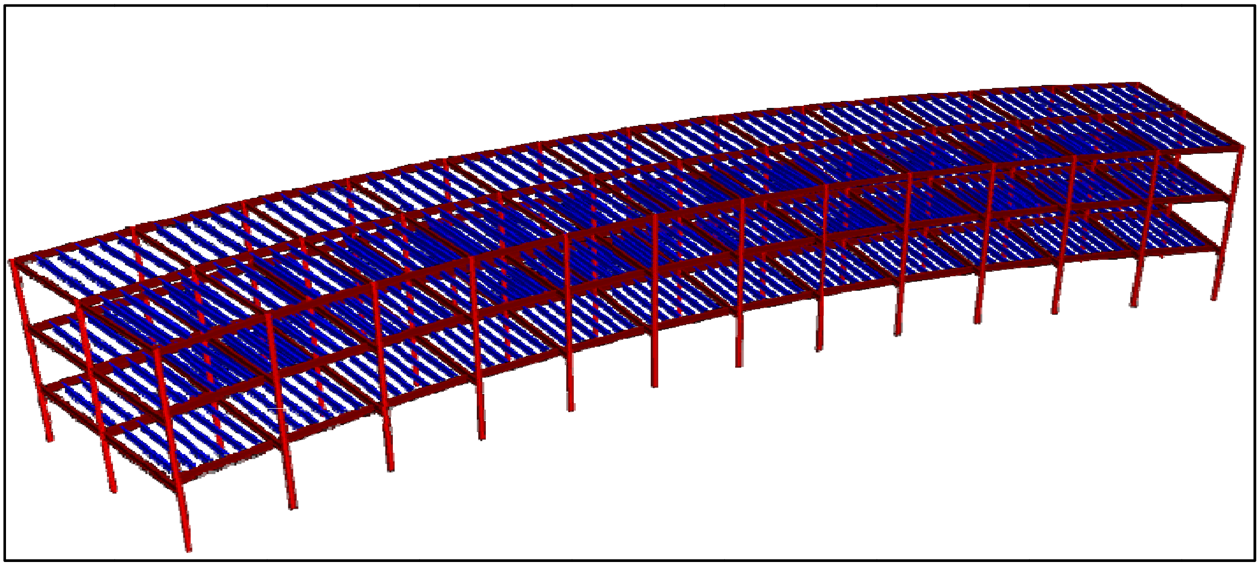


Figure 5. Radial Building

Structural System

LATERAL SYSTEM:

The lateral forces, such as wind and seismic forces, in the building are resisted entirely through moment frames. The engineer chose to implement a moment frame to resist these horizontal forces. The particular frame is a space moment frame, meaning that all of the steel frames are used in the moment frame system. Figures 6 and 7 below show typical details of moment connections used throughout the building. The girder to column flange connections is made through welds of the girder flange to the column flange. A shear plate connects the girder web to the flange. The girder to column web is connection is made with a plate welded to the column web and bolted to the girder flange. A shear plate connects the web of the column and girder.

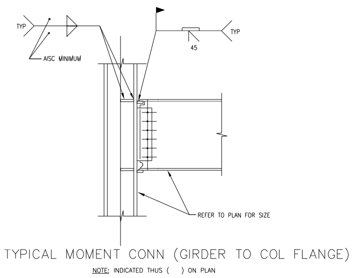


Figure 6. Moment Connection – Girder to Column Flange

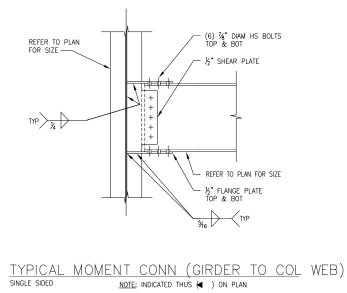


Figure 7. Moment Connection – Girder to Column Web

FLOOR AND ROOF FRAMING:

Three-story portion:
W21 shapes with HSS2½ (TOP) are typically used for beams (Figure 8) while W24 are used for girders. The sizes of the bays are generally 24' wide and span approximately 30'. Steel joists are used to span inside the bays. 28K8 joists are the most common joist in the framing. Typical spacing is approximately 4' on center. Joists also frame the roof, where, to account for the heavy and asymmetric loads of mechanical equipment, KCS joists are commonly found. Roof beams are typically W18x35 and girders W21x44.

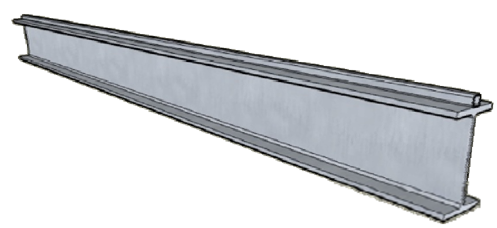


Figure 8

One-story portion:
This part of the building contains an elevated area that serves as an equipment platform. It covers a good portion of the footprint of this section. The "floor joists" are 26K9 spanning 30' in one part of this platform and 24K3/26K4 spanning 16'/19' respectively. Roof joists in the one-story portion are typically slightly larger than the 3-story building (28K10) since they span a much longer distance of around 47'. The structural plans show an area where the joists become increasingly closer to each other. This is due to the higher roof causing snow to drift onto the lower roof in addition to windward drift. A few special joists (KSP) are used in certain areas of the one-story roof framing to account for unique loading. This is generally where there

are folding partitions, causing heavy concentrated loading at points, in meeting rooms such as the School Board Meeting room.

FOUNDATIONS:

A shallow foundation type is used for this building. Foundations consist of spread footings and strip wall footings. The geotechnical engineer for the project indicated that the allowable bearing capacity of the soil is 3000 PSF. The top of the footings are set at (-2'-0") from grade. Reinforcement for spread footings range from (4)#5 BOT bars for the 3'-0"x3'-0" footings to (11)#7 TOP & BOT for the 11'-0"x11'-0" footings. Exterior column spread footings are typically 4'-0"x4'-0" to 6'-0"x6'-0" in the one-story portion and 7'-0"x7'-0" in the three-story portion. Interior column footings in the one-story portion are typically 6'-0"x6'-0" to 8'-0"x8'-0". The three-story interior column footings are 9'-0"x9'-0" to 11'-0"x11'-0". The strip wall footings are typically 2'-0" wide and 1'-0" thick. Reinforcement for strip footings are (3) continuous #5 bars. The strength of the concrete used for foundations is 3000 psi. The concrete strength for the 4" slab on grade is 3500 psi and contains 6x6-W1.4xW1.4 WWF at mid-depth.

COLUMNS:

All columns in the structural system are steel. In the one-story building, some typical interior columns include W12x79 and W10x68. Exterior columns are often rectangular HSS shapes. Typical shapes include HSS8x6x1/4 in the one-story building. In the three-story building, columns are, again, typically W-shapes for the interior and HSS shapes for the exterior. Typical shapes include W14x68 and W14x82 for the interior and circular HSS12.75x0.375 for the exterior.

Codes and Loading

The Virginia Uniform Statewide Building Code (VUSBC), 2000 edition was used for the design of the Edward L Kelly Leadership Center. This code absorbs much of its code from the International Building Code (IBC). IBC2000 will be used when referencing the original design of this building. In addition to IBC, the following codes and specifications were also implemented into the design.

ASCE 7-98, Minimum Design Loads for Buildings and Other Structures
ACI 530-99, Building Code Requirements for Masonry Structures With Commentary
AISC Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design
AISC Code of Standard Practice for Steel Buildings and Bridges
Steel Deck Institute Design Manual for Composite Desks, Form Decks, and Roof Decks
AISI Specification for the Design of Cold Formed Steel Structural Members

Live Loads	IBC 2006	Snow Load
Meeting Rooms	50 + 20 PSF	
Office Space	50 + 20 PSF	
1st Floor Corridors	100 PSF	
Corridors above 1st Floor	80 PSF	
Stairwell	100 PSF	
Mechanical Rooms	150 PSF	
Storage	125 PSF	
Flat Roof		21 PSF
Sloped Roof		21 PSF

Floor - Superimposed Dead Loads	
Mechanical	4 PSF
Electrical / Lighting	3 PSF
Sprinklers	3 PSF
Drop Ceiling	5 PSF
Total	15 PSF

Roof - Superimposed Dead Loads	
Roofing / Insulation	5 PSF
Mechanical	4 PSF
Electrical / Lighting	3 PSF
Sprinklers	3 PSF
Drop Ceiling	5 PSF
Total	20 PSF

3 story Rectangular Building

Lateral Analysis

An important assumption is made prior to any lateral analysis. That assumption is: lateral force at any story will be distributed to each of the frames equally. Because of the near perfect symmetry of the floor plan with nearly identical bays, this assumption is validated as the relative stiffness of each frame is likely equal to the others. For lateral analysis, one frame in each direction will be analyzed. The tributary area of that frame will be used to calculate seismic and wind forces.

Seismic - North/South Direction

In the North – South direction of the building, an initial inspection indicates that seismic loads are likely not the controlling loading situation. The reasons for this basis are:

- The location of the building has relatively low spectral response accelerations
- The depth of the building is only 61 ft in two bays which typically indicates that the seismic weight will be relatively low
- The height of the building is 46 ft. When compared to the depth, it seems that the wind pressures along the length of the building (265 ft) will contribute lateral loads much more than seismic loads.

This initial inspection is also concluded in an in depth analysis for both seismic and wind forces. The frame does not bear a significant seismic weight because of the shallow depth and therefore less tributary area. Figure 9 is a table of the calculation for the story shear. The story shears at the roof, third, and second floor are 3.23 kips, 2.57 kips, and 0.64 kips, respectively. The total base shear is 6.435 kips. Figure 10 displays the seismic overturning moment in the north south direction Figure 11 shows the distribution of the seismic forces at each story.

SEISMIC FORCE DISTRIBUTION						
3-STORY RECTANGULAR						
STORY HEIGHT	h^k	WEIGHT	Wx*hx^k	Cvx	V	Fx
46	2116	60	126960	0.5012688	6.435	3.225665
30.66	940.0356	107.5	101053.83	0.398985	6.435	2.567468
15.33	235.0089	107.5	25263.457	0.0997462	6.435	0.641867
		275	253277.28	1		6.435

Figure 9

OVERTURNING MOMENT						
3-STORY RECTANGULAR						
	STORY LABEL	STORY HEIGHT	STORY FORCE	OT MOMENT		
	R	46	3.23	148.58		
	3	30.66	2.57	78.7962		
	2	15.33	0.64	9.8112		
			6.44	237.1874		

Figure 10

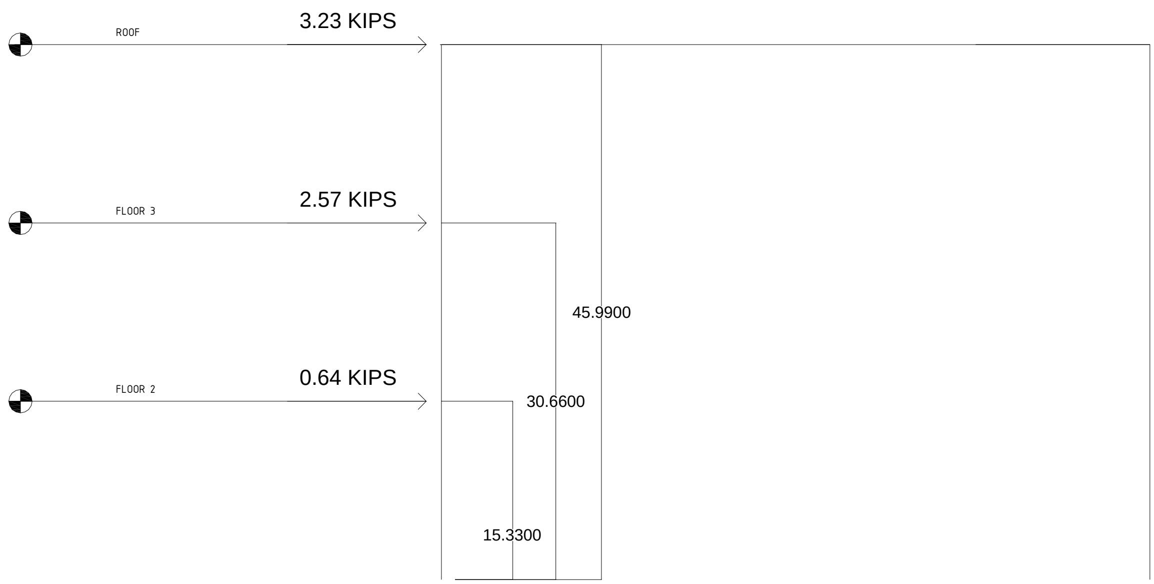


Figure 11

Wind – North/South Direction

As previously mentioned, because of the relatively shallow depth of the building, seismic weights are relatively low. As a result, wind loading is the controlling lateral force in the North-South direction. The most intense case is windward loading on the North façade. Because the southern portion of the section is enclosed within the interior of the building, no leeward wind pressures exist on this area and are set to zero. In reality, leeward pressure exists on the opposing side of the radial building. However, because of the assumption that the forces will not transfer between buildings, for this analysis, they are zero. Figures 12 and 13 show the story shear at each level, the base shear, and the wind overturning moment.

STORY	SHEAR	HEIGHT
2	3.34	15.33
3	4.05	30.66
ROOF	2.19	46

Figure 12

BASE SHEAR		3.34 + 4.05 + 2.19 =		9.58	K		
OVERTURNING		3.34 * 15.33 + 4.05 * 33.66 + 2.19 * 46 + 9 * 38.11 + 1.22 * 7.5					
MOMENT		=	640.4052	FT-K			

Figure 13

The following figures are graphical representations of the wind pressures (Fig. 14) and story shears (Fig. 15)

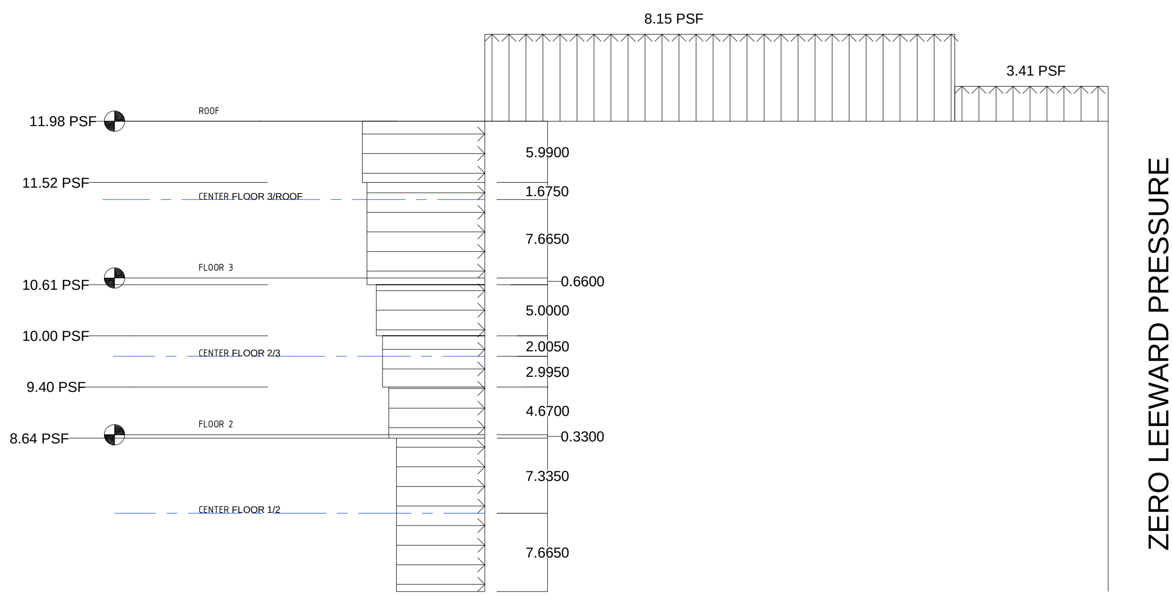


Figure 14

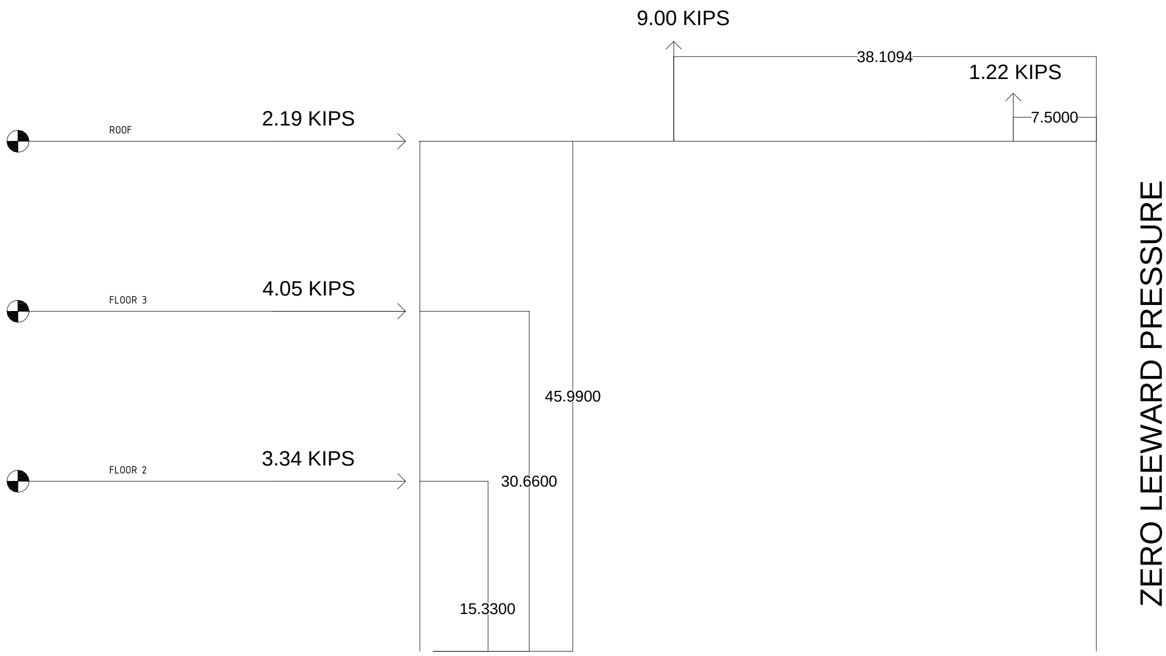


Figure 15

Seismic – East/West Direction

Figure 16 shows the seismic story shear at each level and the total seismic base shear of 36.98 kips. Figure 17 shows the overturning moment of 1486.03 ft-kips.

SEISMIC FORCE DISTRIBUTION E/W						
3-STORY RECTANGULAR						
STORY HEIGHT	h^k	WEIGHT	Wx*hx^k	Cvx	V	Fx
46	2116	614	1299224	0.6631641	36.98	24.52381
30.66	940.0356	614	577181.86	0.2946115	36.98	10.89473
15.33	235.0089	352	82723.133	0.0422244	36.98	1.56146
		1580	1959129	1		36.98

Figure 16

OVERTURNING MOMENT						
3-STORY RECTANGULAR						
	STORY LABEL	STORY HEIGHT	STORY FORCE	OT MOMENT		
	R	46	24.52	1127.92		
	3	30.66	10.9	334.194		
	2	15.33	1.56	23.9148		
			36.98	1486.0288		

Figure 17

Figure 18 shows a graphical representation of the seismic shear at each level.

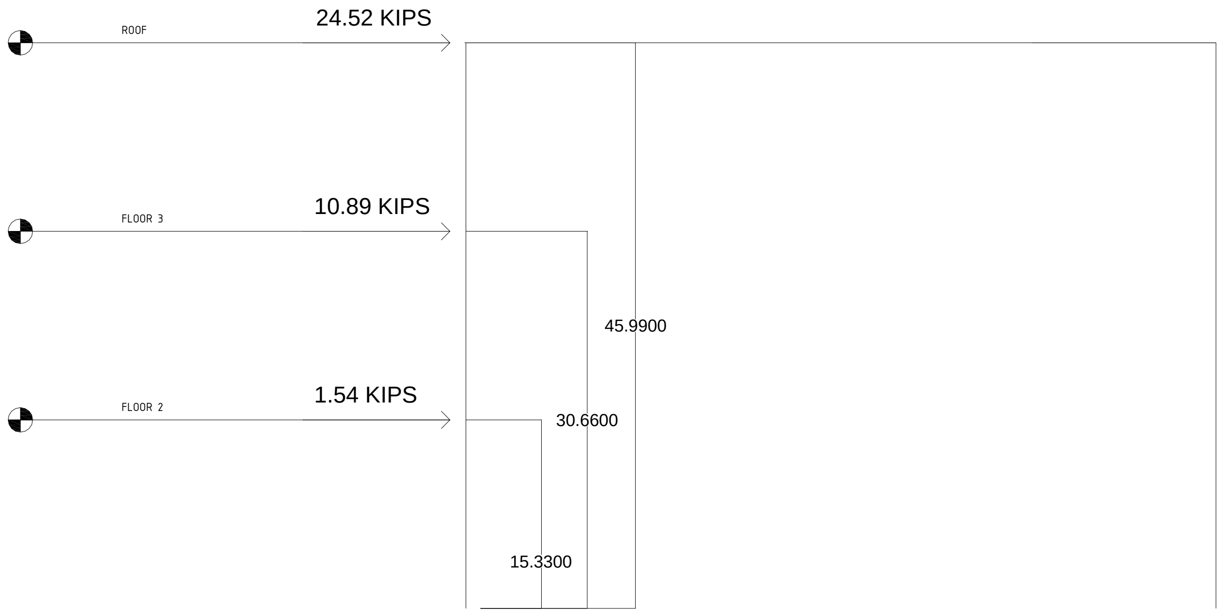


Figure 18

Wind – East/West Direction

In the East – West direction, there is a dramatic change is the floor plan. The depth of the frame now extends 265 ft. The tributary width of the frame is approximately 30.5 ft. In reference to windward force on the building, the only change compared to the North – South frame is the slight increase in tributary width. The width changes from 24 ft to 30.5 ft. However, the depth dramatically increases to 265 ft. Therefore, the seismic weight will vastly increase.

The story shear at each level due to wind is shown in Figure 19. The total wind base shear is found to be 12.17 kips. The wind overturning moment is shown in Figure 20.

STORY	SHEAR	HEIGHT
2	4.24	15.33
3	5.15	30.66
ROOF	2.78	46

Figure 19

BASE SHEAR		4.24 + 5.15 + 2.78		12.17	K			
OVERTURNING		4.24 * 15.33 + 5.15 * 33.66 + 2.78 * 46 + 11.43 * 242 + 4.78 * 196 + 5.49 * 86.5						
MOMENT		=	4544.053	FT-K				

Figure 20

Figures 21 and 22 on the following page are graphical representations of the wind pressures and story shears. Again, there is zero leeward pressure due to the obstruction of the building beyond.

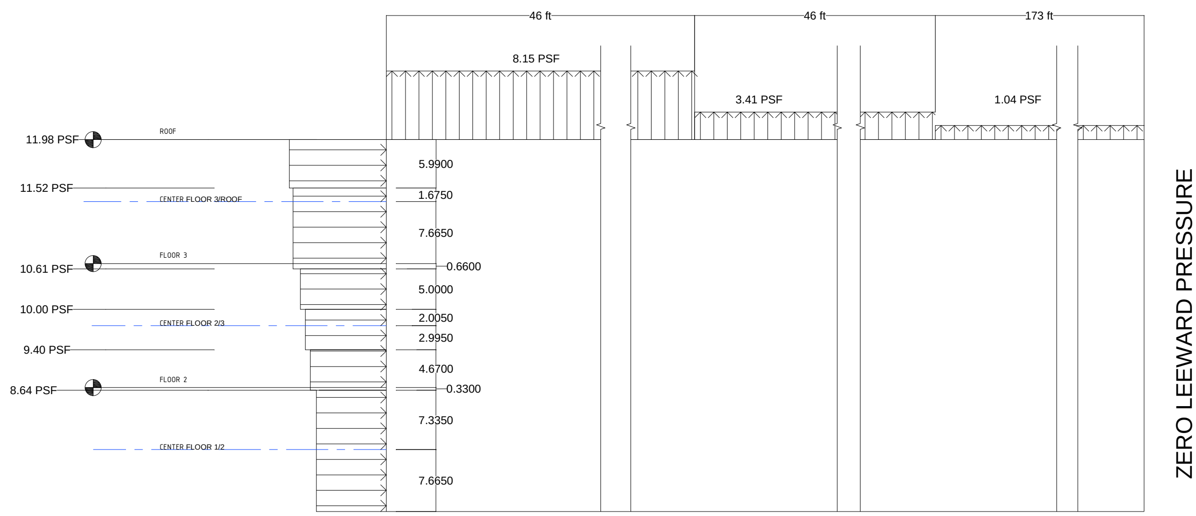


Figure 21

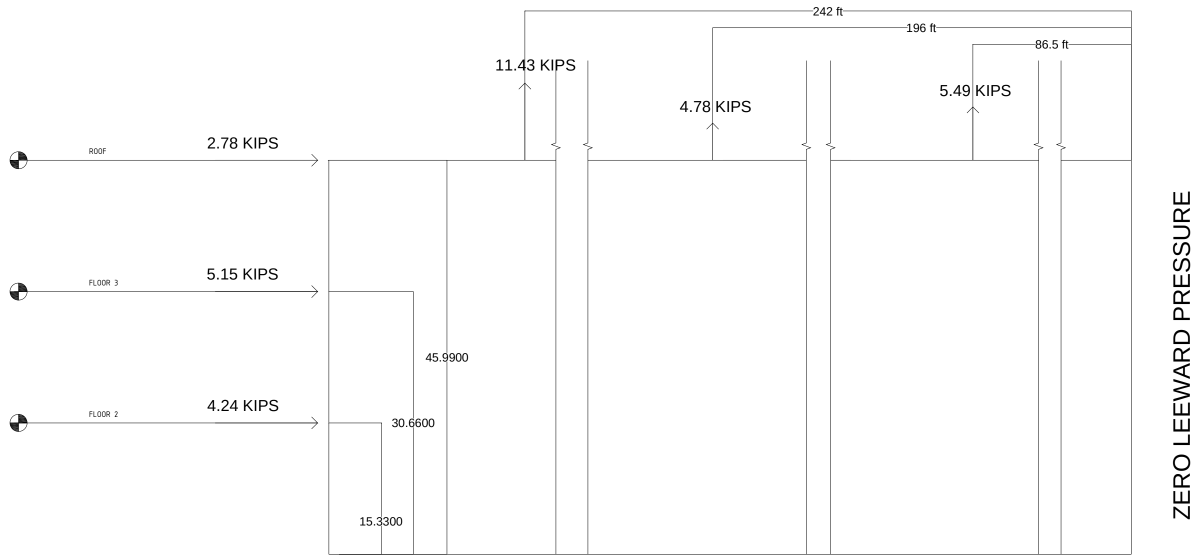


Figure 22

Torsion

This section of the building has a very regular framing pattern. So much so that it is very much symmetrical. Therefore, by inspection, it seems that the centers of mass and rigidity will be centrally located in the rectangular footprint. Figure 23 shows the centers of mass and rigidity for each level.

Centers of Rigidity			Centers of Mass		
Level	X Coordinate	Y Coordinate	Level	X Coordinate	Y Coordinate
Roof	136.13	43.86	Roof	141.28	43.47
Third	135.8	43.22	Third	144.69	39.87
Second	135.61	46.52	Second	144.69	39.87

Figure 23

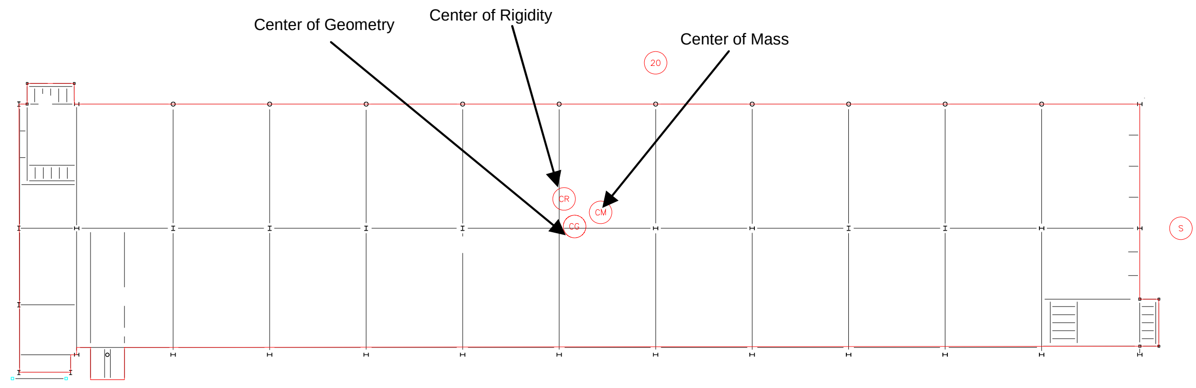


Figure 24

As Figure 24 shows, the centers of mass and rigidity are, in fact, centrally located. Both the center of mass and rigidity are located just north of the geometric center of the building. The eccentricities in the x- and y-direction are 8.89 ft and 3.35 ft, respectively. Torsion is not much of a concern in this building, but should be studied in depth when looking at the building as a whole.

Story Drift

The story drift was analyzed as an individual frame and the building as a whole. The following figures 25 and 26 display the results from RISA frame analysis. It is clear that the story drifts are not of concern. The values are much less than the required H/400 = (46)(12)/400 = 1.38 in.

Story Drift		
2	0.156	in
3	0.151	in
Roof	0.08	in

Figure 25 NS

Story Drift		
2	0.117	in
3	0.137	in
Roof	0.093	in

Figure 26 EW

Conclusions

Some data from the RAM model matches hand calculations. Seismic weight and wind pressures generally match up well with only slight error. A greater analysis using RAM Structural System needs to be completed to fine tune hand and computer calculations. Some RAM output does not make intuitive sense. Such is the case where the wind forces are lower higher on the building with no change in profile. Some of the error can be attributed to the computer model having the capability to quickly calculate lengthy and complex calculations, such as for flexibility, gust factors, and true period.

Appendix

A

Wind Data A1

Seismic Data A5

Seismic Weight A8

Lateral Frame Analysis Data....................................... A12

Portal Method A13

Centers of Mass and Rigidity A14

RAM Output Data.. A15

Wind Data

WINDWARD

HEIGHT	KZ	KZT	KD	V	I	QZ	GF	CP	qGCP	(+)GCPI	(-)GCPI	Qi(+)GCPI	Qi(-)GCPI	p=qGCP-qi(+)GCPI	p=qGCP-qi(-)GCPI
0-15	0.57	1	0.85	90	1	10.05	0.85	0.8	6.83	0.18	-0.18	1.80838656	-1.80838656	5.02	8.64
20	0.62	1	0.85	90	1	10.93	0.85	0.8	7.43	0.18	-0.18	1.96701696	-1.96701696	5.46	9.40
25	0.66	1	0.85	90	1	11.63	0.85	0.8	7.91	0.18	-0.18	2.09392128	-2.09392128	5.82	10.00
30	0.7	1	0.85	90	1	12.34	0.85	0.8	8.39	0.18	-0.18	2.2208256	-2.2208256	6.17	10.61
40	0.76	1	0.85	90	1	13.40	0.85	0.8	9.11	0.18	-0.18	2.41118208	-2.41118208	6.70	11.52
46	0.79	1	0.85	90	1	13.92	0.85	0.8	9.47	0.18	-0.18	2.50636032	-2.50636032	6.96	11.97
50	0.81	1	0.85	90	1	14.28	0.85	0.8	9.71	0.18	-0.18	2.56981248	-2.56981248	7.14	12.28
60	0.85	1	0.85	90	1	14.98	0.85	0.8	10.19	0.18	-0.18	2.6967168	-2.6967168	7.49	12.88
70	0.89	1	0.85	90	1	15.69	0.85	0.8	10.67	0.18	-0.18	2.82362112	-2.82362112	7.84	13.49

ROOF

HEIGHT	KZ	KZT	KD	V	I	QH	GF	CP	qGCP	(+)GCPI	(-)GCPI	Qi(+)GCPI	Qi(-)GCPI	p=qGCP-qi(+)GCPI	p=qGCP-qi(-)GCPI
0 to h/2	0.79	1	0.85	90	1	13.92	0.85	-0.9	-10.65	0.18	-0.18	2.50636032	-2.50636032	-13.16	-8.15
h/2 to h	0.79	1	0.85	90	1	13.92	0.85	-0.9	-10.65	0.18	-0.18	2.50636032	-2.50636032	-13.16	-8.15
h to 2h	0.79	1	0.85	90	1	13.92	0.85	-0.5	-5.92	0.18	-0.18	2.50636032	-2.50636032	-8.42	-3.41
>2h	0.79	1	0.85	90	1	13.92	0.85	-0.3	-3.55	0.18	-0.18	2.50636032	-2.50636032	-6.06	-1.04

LEEWARD

HEIGHT	KZ	KZT	KD	V	I	QZ	GF	CP	qGCP	(+)GCPI	(-)GCPI	Qi(+)GCPI	Qi(-)GCPI	p=qGCP-qi(+)GCPI	p=qGCP-qi(-)GCPI
N/S															
46	0.79	1	0.85	90	1	13.92	0.85	-0.5	-5.92	0.18	-0.18	2.50636032	-2.50636032	-8.42	-3.41
E/W															
46	0.79	1	0.85	90	1	13.92	0.85	-0.3	-3.55	0.18	-0.18	2.50636032	-2.50636032	-6.06	-1.04

Rectangular North - South Windward Direction Calculations

STORY 2 SHEAR N/S			
HEIGHT	PRESSURE	W (PLF)	TOTAL
7.335	8.46	62.0541	
0.33	9.4	3.102	
4.67	9.4	43.898	
3	10	30	
			139.0541
139.05 PLF FOR TRIB WIDTH = 24'			
139.0541 * 24		3337.2	POUNDS
		3.34	KIPS

STORY 3 SHEAR N/S			
HEIGHT	PRESSURE	W (PLF)	TOTAL
2	10	20	
5	10.61	53.05	
0.66	11.52	7.6032	
7.665	11.52	88.3008	
			168.954
168.95 PLF FOR TRIB WIDTH = 24'			
168.95 * 24		4054.896	POUNDS
		4.054896	KIPS

ROOF SHEAR N/S			
HEIGHT	PRESSURE	W (PLF)	TOTAL
1.675	11.52	19.296	
6	11.98	71.88	
			91.176
91.176	PLF FOR TRIB WIDTH = 24'		
91.176 * 24		2188.224	POUNDS
		2.188224	KIPS

ROOF UPLIFT (0 TO H) N/S			
HEIGHT	PRESSURE	W (PLF)	
23	8.15	187.45	
23	8.15	187.45	
			374.9
375 PLF FOR TRIB WIDTH = 24'			
375 * 24		9000	POUNDS
		9	KIPS

ROOF UPLIFT (0 TO H) N/S			
HEIGHT	PRESSURE	W (PLF)	
15	3.41	51.15	
			51.15
51 PLF FOR TRIB WIDTH = 24'			
51 * 24		1224	POUNDS
		1.224	KIPS

Rectangular East - West Windward Direction Calculations

STORY 2 SHEAR			
HEIGHT	PRESSURE	W (PLF)	TOTAL
7.335	8.46	62.0541	
0.33	9.4	3.102	
4.67	9.4	43.898	
3	10	30	
			139.0541
139.0541	PLF FOR TRIB WIDTH =		30.5
		4241.1501	POUNDS
		4.2411501	KIPS

STORY 3 SHEAR			
HEIGHT	PRESSURE	W (PLF)	TOTAL
2	10	20	
5	10.61	53.05	
0.66	11.52	7.6032	
7.665	11.52	88.3008	
			168.954
168.954	PLF FOR TRIB WIDTH =		30.5
		5153.097	POUNDS
		5.153097	KIPS

ROOF SHEAR			
HEIGHT	PRESSURE	W (PLF)	TOTAL
1.675	11.52	19.296	
6	11.98	71.88	
			91.176
91.176	PLF FOR TRIB WIDTH =		30.5
		2780.868	POUNDS
		2.780868	KIPS

ROOF UPLIFT (0 TO H=46)			
HEIGHT	PRESSURE	W (PLF)	
23	8.15	187.45	
23	8.15	187.45	
			374.9
374.9	PLF FOR TRIB WIDTH =		30.5
		11434.45	POUNDS
		11.43445	KIPS

ROOF UPLIFT (H=46 to 2H=92)			
HEIGHT	PRESSURE	W (PLF)	
46	3.41	156.86	
			156.86
156.86	PLF FOR TRIB WIDTH =		30.5
		4784.23	POUNDS
		4.78423	KIPS

ROOF UPLIFT (>2H=92)			
HEIGHT	PRESSURE	W (PLF)	
173	1.04	179.92	
			179.92
179.92	PLF FOR TRIB WIDTH =		30.5
		5487.56	POUNDS
		5.48756	KIPS

Seismic Data

The following calculations are based upon ASCE7-05 Chapter 11, "Seismic Design Criteria." The total seismic weight is calculated following the code summary.

11.4.1
0.2 Second Spectral Response Acceleration [5% of Critical Damping]
$S_s = 0.162$ [Figure 21-1]
1.0 Second Spectral Response Acceleration [5% of Critical Damping]
$S_1 = 0.052$ [Figure 21-3]

11.4.2
Site Classification: D

11.4.3
Site Coefficients and Adjusted Maximum Considered Earthquake Spectral Response Acceleration
$F_a = 1.6$ [Table 11.4-1]
$S_{MS} = F_a S_s = (1.6)(0.162) = 0.2592$ [Equation 11.4-1]
$F_v = 2.4$ [Table 11.4-2]
$S_{M1} = F_v S_1 = (2.4)(0.052) = 0.1248$ [Equation 11.4-2]

11.4.4
Design Spectral Acceleration
$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.2592) = 0.1728$ [Equation 11.4-3]
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.1428) = 0.0832$ [Equation 11.4-4]

12.8.2
Period Determination
12.8.2.1
Approximate Fundamental Period
$C_t = 0.028$
$x = 0.8$ [Table 12.8-2]
$h_n = 46$
$T_a = C_t h_n^x = (0.028)(46)^{0.8} = 0.5989$
$C_u = 1.7$ [Table 12.8-1]
$T = C_u T_a = (1.7)(0.5989) = 1.018$

11.4.5
Design Response Spectrum

$T_0 = 0.2\dfrac{S_{D1}}{S_{DS}} = 0.2\dfrac{0.0832}{0.1728} = 0.0930$

$T_S = \dfrac{S_{D1}}{S_{DS}} = \dfrac{0.0832}{0.1728} = 0.4815$

$T_L = 8$ [Figure 22-15]

3. For $T > T_S$ and $T \le T_L$

$S_a = \dfrac{S_{D1}}{T} = \dfrac{0.0832}{1.018} = 0.0817$

11.5.1
Occupancy Category: II [Table 1-1]
Importance Factor: $I = 1.0$ [Table 11.5-1]

11.6 Seismic Design Category
Seismic Design Category Based on 1-s Period Response Acceleration: B [Table 11.6-2]
$0.067 \le S_{D1} < 0.133$

12.8 Equivalent Lateral Force Procedure
12.8.1 Seismic Base Shear
$V = C_s W$ [Equation 12.8-2]

12.2 Structural System Selection
Response Modification Coefficient: $R = 3.5$
System Overstrength Factor: $\omega_0 = 3.0$
Deflection Amplification Factor: $C_d = 3.0$
Structural System Limitations and Building Height Limit: NL for SDC B, C, D, E, F
12.8.1.1 Calculation of Seismic Response Coefficient
For $T \le T_L$

$$C_s = \min\begin{cases} \dfrac{S_{DS}}{\left(\dfrac{R}{I}\right)} = \dfrac{0.1728}{\left(\dfrac{3.5}{1.0}\right)} = 0.0494 \\ \dfrac{S_{D1}}{T\left(\dfrac{R}{I}\right)} = \dfrac{0.0832}{1.018\left(\dfrac{3.5}{1.0}\right)} = 0.0234 \end{cases} \ge 0.01$$

12.8.3 Vertical Distribution of Seismic Forces

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$ [Equation 12.8-12]

Base Shear

$$V = C_s W = (0.0234)(275) = 6.435\,\text{kips}$$

3-Story Portion

Shear at Roof Level

$$C_{v1} = 0.501$$

$$C_{v1}V = (0.501)(6.435) = 3.22\,\text{kips}$$

Shear at Third Floor

$$C_{v1} = 0.399$$

$$C_{v1}V = (0.399)(6.435) = 2.57\,\text{ips}$$

Shear at Second Floor

$$C_{v1} = 0.0997$$

$$C_{v1}V = (0.0997)(6.435) = 0.642\,\text{kips}$$

Notes: All frames have approximately the same relative stiffness; lateral load will be transferred based upon tributary area of the frame at each level

There are 11 Frames to carry the lateral force. Each frame will carry 1/11th the load.

The following data is a detailed takeoff of the structural components that are considered in the seismic weight of the building. It contains a breakdown of all the components of the structural dead loads for Floor 2, Floor 3, and the Roof level.

The North-South frame in consideration is along column line 20 of the 3-story rectangular portion of the building. The East-West frame under analysis is along column line S. Basic tributary areas and widths are taken into account with the important assumption that the lateral story loads are distributed equally to all frames. This is a valid assumption due to the fact of equal frame spacing throughout the floor plan creating a perfectly symmetrical structural layout in the building under consideration.

FLOOR 2									
Descript	W / HSS	PLF	Length	#	Weight (#)	kips	Full/Half		
M	14	82	15.33		1257.06	1.25706	1	1.25706	
S	14	68	15.33		1042.44	1.04244	1	1.04244	
X	12.75x.375	49.6	15.33		760.368	0.760368	1	0.760368	
M19-20	21	68	24		1632	1.632	0.5	0.816	
M20-21	21	68	24		1632	1.632	0.5	0.816	
S19-20	24	55	24		1320	1.32	0.5	0.66	
S20-21	24	55	24		1320	1.32	0.5	0.66	
X19-20	21	55	24		1320	1.32	0.5	0.66	
X20-21	21	55	24		1320	1.32	0.5	0.66	
20M-S	21	68	31.5		2142	2.142	1	2.142	
20S-X	21	57	31		1767	1.767	1	1.767	
MS19-20	28K8	12.7	29.66	5	1883.41	1.88341	0.5	0.941705	
MS20-21	28K8	12.7	29.66	5	1883.41	1.88341	0.5	0.941705	
SX19-20	28K9	13	31	5	2015	2.015	0.5	1.0075	
SX20-21	28K9	13	31	5	2015	2.015	0.5	1.0075	
		PSF	SF						
MS19-20	1.0C22,t=4	43	742		31906	31.906	0.5	15.953	
MS20-21	1.0C22,t=4	43	742		31906	31.906	0.5	15.953	
SX19-20	1.0C22,t=4	43	756		32508	32.508	0.5	16.254	
SX20-21	1.0C22,t=4	43	756		32508	32.508	0.5	16.254	
MS19-20	Super DL	15	742		11130	11.13	0.5	5.565	
MS20-21	Super DL	15	742		11130	11.13	0.5	5.565	
SX19-20	Super DL	15	756		11340	11.34	0.5	5.67	
SX20-21	Super DL	15	756		11340	11.34	0.5	5.67	
		PSF	Length	Height					
Wall Wt		15	24	15.33	5518.8	5.5188	1	5.5188	
								TOTAL	107.542078

FLOOR 3									
Descript	W / HSS	PLF	Length	#	Weight (#)	kips	Full/Half		
M	14	82	15.33		1257.06	1.25706	1	1.25706	
S	14	68	15.33		1042.44	1.04244	1	1.04244	
X	12.75x.375	49.6	15.33		760.368	0.760368	1	0.760368	
M19-20	21	68	24		1632	1.632	0.5	0.816	
M20-21	21	68	24		1632	1.632	0.5	0.816	
S19-20	24	55	24		1320	1.32	0.5	0.66	
S20-21	24	55	24		1320	1.32	0.5	0.66	
X19-20	21	55	24		1320	1.32	0.5	0.66	
X20-21	21	55	24		1320	1.32	0.5	0.66	
20M-S	21	68	31.5		2142	2.142	1	2.142	
20S-X	21	57	31		1767	1.767	1	1.767	
MS19-20	28K8	12.7	29.66	5	1883.41	1.88341	0.5	0.941705	
MS20-21	28K8	12.7	29.66	5	1883.41	1.88341	0.5	0.941705	
SX19-20	28K9	13	31	5	2015	2.015	0.5	1.0075	
SX20-21	28K9	13	31	5	2015	2.015	0.5	1.0075	
		PSF	SF						
MS19-20	1.0C22,t=4	43	742		31906	31.906	0.5	15.953	
MS20-21	1.0C22,t=4	43	742		31906	31.906	0.5	15.953	
SX19-20	1.0C22,t=4	43	756		32508	32.508	0.5	16.254	
SX20-21	1.0C22,t=4	43	756		32508	32.508	0.5	16.254	
MS19-20	Super DL	15	742		11130	11.13	0.5	5.565	
MS20-21	Super DL	15	742		11130	11.13	0.5	5.565	
SX19-20	Super DL	15	756		11340	11.34	0.5	5.67	
SX20-21	Super DL	15	756		11340	11.34	0.5	5.67	
		PSF	Length	Height					
Wall Wt		15	24	15.33	5518.8	5.5188	1	5.5188	
								TOTAL	107.542078

ROOF									
Descript	W / HSS	PLF	Length	#	Weight (#)	kips	Full/Half		
Roof									
M	14	82	15.33		1257.06	1.25706	1	1.25706	
S	14	68	15.33		1042.44	1.04244	1	1.04244	
X	12.75x.375	49.6	15.33		760.368	0.760368	1	0.760368	
M19-20	21	62	24		1488	1.488	0.5	0.744	
M20-21	21	62	24		1488	1.488	0.5	0.744	
S19-20	21	44	24		1056	1.056	0.5	0.528	
S20-21	21	44	24		1056	1.056	0.5	0.528	
X19-20	18	50	24		1200	1.2	0.5	0.6	
X20-21	18	50	24		1200	1.2	0.5	0.6	
20M-S	18	35	31.5		1102.5	1.1025	1	1.1025	
20S-X	18	35	31		1085	1.085	1	1.085	
MS19-20	20KCS2	9.5	29.66	5	1408.85	1.40885	0.5	0.704425	
MS20-21	20KCS2	9.5	29.66	5	1408.85	1.40885	0.5	0.704425	
SX19-20	22KCS3	12.5	31	5	1937.5	1.9375	0.5	0.96875	
SX20-21	22KCS3	12.5	31	5	1937.5	1.9375	0.5	0.96875	
		PSF	SF						
MS19-20	1.5B22 DECK	1.78	742		1320.76	1.32076	0.5	0.66038	
MS20-21	1.5B22 DECK	1.78	742		1320.76	1.32076	0.5	0.66038	
SX19-20	1.5B22 DECK	1.78	848		1509.44	1.50944	0.5	0.75472	
SX20-21	1.5B22 DECK	1.78	848		1509.44	1.50944	0.5	0.75472	
MS19-20	Super DL	20	742		14840	14.84	0.5	7.42	
MS20-21	Super DL	20	742		14840	14.84	0.5	7.42	
SX19-20	Super DL	20	756		15120	15.12	0.5	7.56	
SX20-21	Super DL	20	756		15120	15.12	0.5	7.56	
RTU-2					10588	10.588	1	10.588	
		PSF	Length	Height					
Wall Wt		15	24	15.33	5518.8	5.5188	1	5.5188	
								TOTAL	59.977658

TOTAL 275.0618 kips

Calculate Average Weight Density				
Floor	Weight from full calc (K)	Area of full calc (SF)	Average KSF	Average PSF
2	108	1452	0.07438017	74.3801653
3	108	1452	0.07438017	74.3801653
R	60	1604	0.03740648	37.4064838

The average weight (in psf) will be as follows

2nd floor: 74.4 PSF

3rd floor: 74.4 PSF

Roof: 37.4 PSF

Estimate Weight in East - West Direction				
Floor	Average PSF	Area (E/W Direction)	Average Weight (#)	Average Weight (kips)
2	74.4	8254	614097.6	614.0976
3	74.4	8254	614097.6	614.0976
R	37.4	9422	352382.8	352.3828

Lateral Frame Analysis

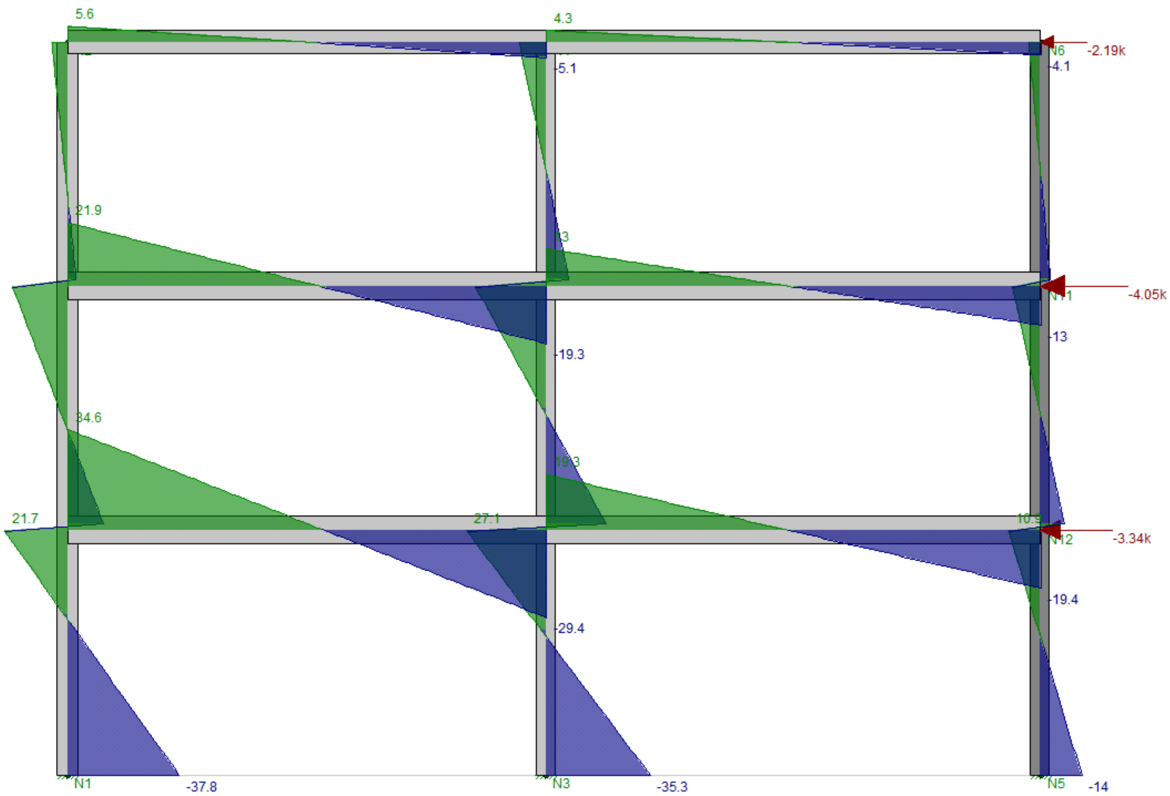


Figure 1

Story Drift		
2	0.156	in
3	0.151	in
Roof	0.08	in

Figure 2

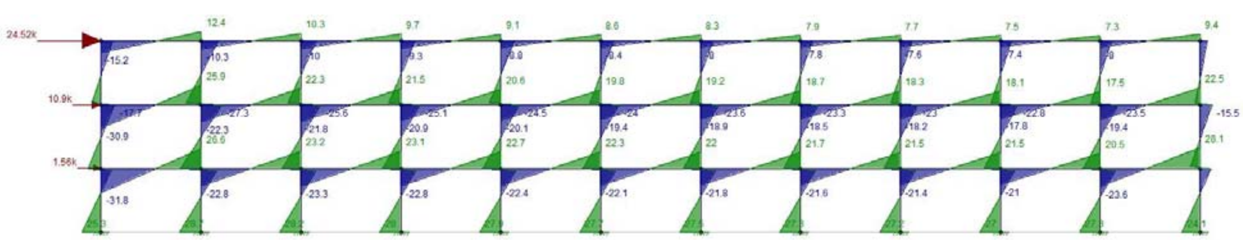
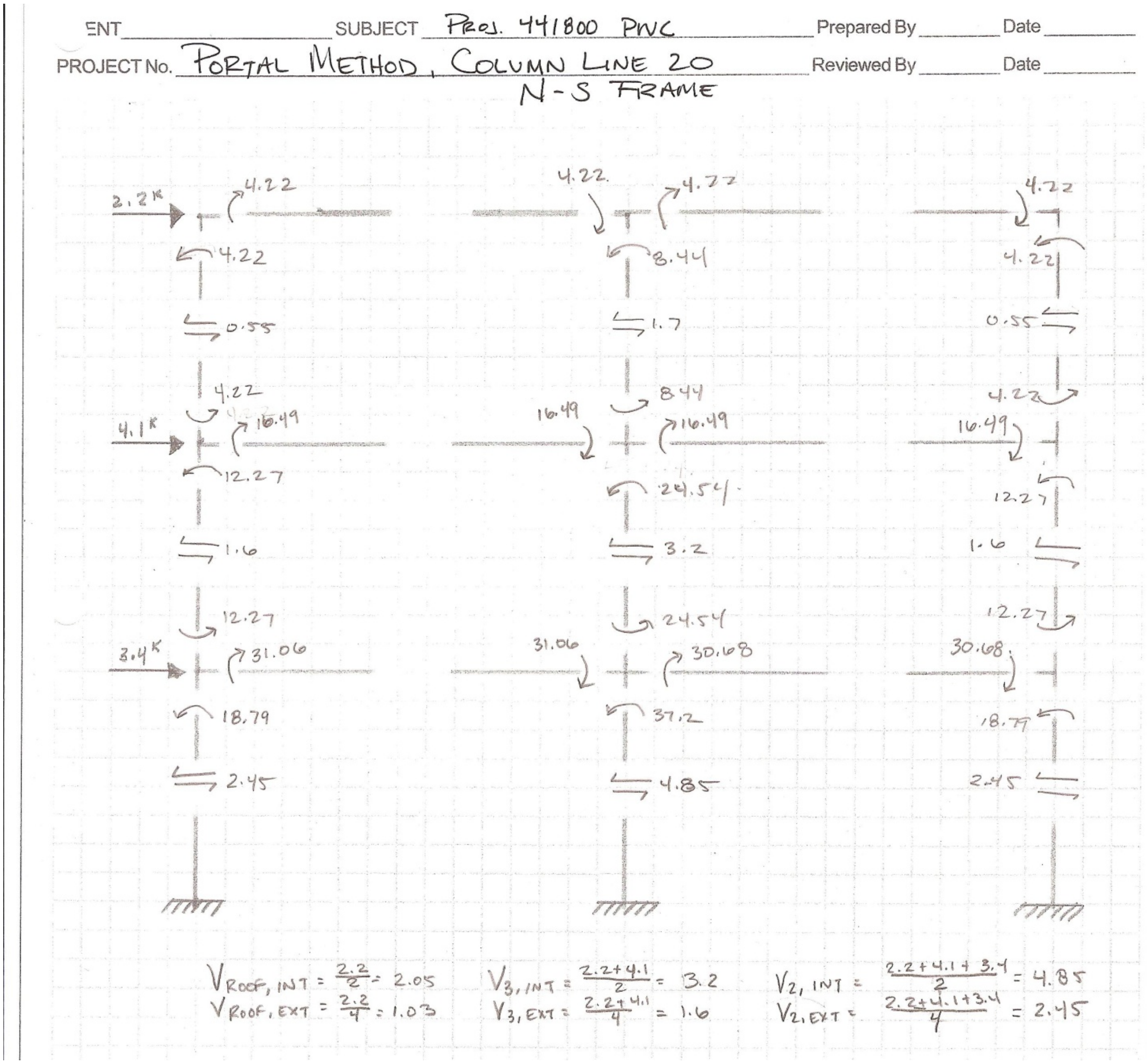
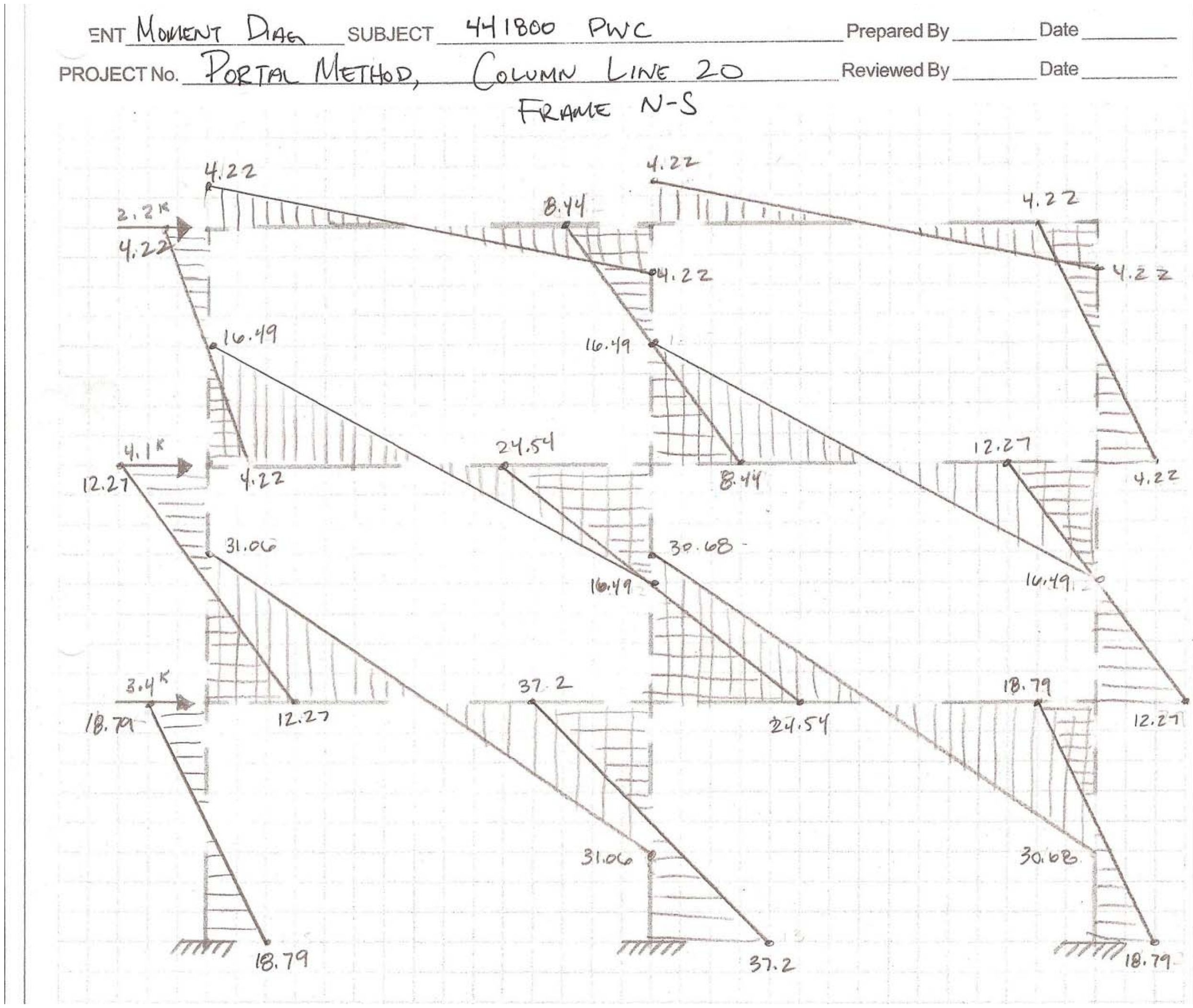


Figure 3

Story Drift		
2	0.117	in
3	0.137	in
Roof	0.093	in

Figure 4





Centers of Rigidity and Centers of Mass

Level	Diaph. #	Centers of Rigidity Xr ft	Centers of Rigidity Yr ft	Centers of Mass Xm ft	Centers of Mass Ym ft
Roof	1	136.13	43.86	141.28	43.47
Story 3	1	135.80	43.22	144.69	39.87
Story 2	1	135.61	36.52	144.69	39.87

Figure 5

LOAD CASE: seismic2

Seismic ASCE 7-02 / IBC 2003 Equivalent Lateral Force

Site Class: A	Importance Factor: 1.00	Ss: 0.050 g	S1: 0.050 g
Fa: 0.800	Fv: 0.800	SDs: 0.027 g	SD1: 0.027 g

Seismic Use Group: I Seismic Design Category: A
Provisions for: Force
Ground Level: Base

Dir	Eccent	R	Ta Equation	Building Period-T
X	+ And -	6.0	Std,Ct=0.030,x=0.75	Calculated
Y	+ And -	6.0	Std,Ct=0.030,x=0.75	Calculated

Dir	Ta	Cu	T	T-used	Eq95521-1	Eq95521-2	Eq95521-3	k
X	0.530	1.700	1.079	0.901	0.004	0.005	0.0012	1.200
Y	0.530	1.700	1.188	0.901	0.004	0.005	0.0012	1.200

Total Building Weight (kips) = 2771.27

APPLIED DIAPHRAGM FORCES

Type: EQ_IBC03_X_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	46.00	4.58	0.00	141.28	46.80
Story 3	1	30.67	5.39	0.00	144.69	43.29
Story 2	1	15.33	2.35	0.00	144.69	43.29

APPLIED STORY FORCES

Type: EQ_IBC03_X_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	46.00	4.58	0.00
Story 3	30.67	5.39	0.00
Story 2	15.33	2.35	0.00
		12.32	0.00

APPLIED DIAPHRAGM FORCES

Type: EQ_IBC03_Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	46.00	0.00	4.58	155.24	43.47
Story 3	1	30.67	0.00	5.39	158.70	39.87
Story 2	1	15.33	0.00	2.35	158.70	39.87

APPLIED STORY FORCES

Type: EQ_IBC03_Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	46.00	0.00	4.58
Story 3	30.67	0.00	5.39
Story 2	15.33	0.00	2.35
		0.00	12.32

LOAD CASE: wind

Wind ASCE 7-02/IBC2003
Exposure: B
Basic Wind Speed (mph): 90.0 Importance Factor: 1.000
Apply Directionality Factor, Kd = 0.85
Use Topography Factor, Kzt: 1.00
Use Calculated Frequency for X-Dir.
Use Calculated Frequency for Y-Dir.
Gust Factor for Flexible Structures, G: Use Calculated G for X-Dir.
Gust Factor for Flexible Structures, G: Use Calculated G for Y-Dir.
Damping Ratio for Flexible Structures= 0.01
Mean Roof Height (ft): Top Story Height + Parapet = 46.00
Ground Level: Base

WIND PRESSURES:

X-Direction: Natural Frequency = 0.927 Structure is Flexible
Y-Direction: Natural Frequency = 0.842 Structure is Flexible
CpWindward = 0.80 qLeeward (qh) = 13.95 psf
GCpn (Parapet): Windward = 1.80 Leeward = -1.10

Height	Kz	Kzt	qz	Gust Factor G		CpLeeward		Pressure (psf)	
ft			psf	X	Y	X	Y	X	Y
46.00	0.792	1.000	13.952	0.909	0.817	-0.200	-0.500	12.656	14.840
30.67	0.705	1.000	12.426	0.908	0.817	-0.200	-0.500	11.528	13.838
15.33	0.578	1.000	10.194	0.908	0.817	-0.200	-0.500	9.910	12.377
0.00	0.575	1.000	10.130	0.908	0.817	-0.200	-0.500	9.864	12.335

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC03_1_X

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	46.00	6.81	0.00	139.19	39.76
Story 3	1	30.67	11.57	0.00	139.40	34.66
Story 2	1	15.33	10.65	0.00	139.40	33.52

APPLIED STORY FORCES

Type: Wind_IBC03_1_X

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	46.00	6.81	0.00
Story 3	30.67	11.57	0.00
Story 2	15.33	10.65	0.00
		29.04	0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC03_1_Y

Level	Diaph.#	Ht	Fx	Fy	X	Y

		ft	kips	kips	ft	ft
Roof	1	46.00	0.00	31.07	139.19	37.29
Story 3	1	30.67	0.00	59.02	139.29	33.52
Story 2	1	15.33	0.00	54.16	139.40	33.52

APPLIED STORY FORCES

Type: Wind_IBC03_1_Y

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	46.00	0.00	31.07
Story 3	30.67	0.00	59.02
Story 2	15.33	0.00	54.16
		0.00	144.25