- Robert Whitaker
- Structural ~ Parfitt
- Parkview at Bloomfield Station
- Bloomfield, NJ
- 11-21-05

Structural Technical Report #3

By Robert Whitaker

Executive summary

This report covers a more extensive check of the main lateral force resisting components for Parkview at Bloomfield Station, a six story residential apartment in Bloomfield, New Jersey. Lateral load calculations, including seismic and wind from three directions, are used to compute the loads for the shear walls in the building. The two braced frames at the drive aisles are also analyzed to check their capacity. Finally, a building drift limit is established and compared to code values.

Structural Overview

The structural system for Parkview at Bloomfield Station is a light gage roof composed of trusses spaced 2' on center (oc) spanning front to back, panelized bearing light gage walls 4" and 6" wide continuously capped with a steel tube for load distribution purposes. These walls not only hold the 16" deep D500 Hambro® floor system but also act as the main lateral force resisting system for the building. Thin cross bracing straps attached to the light gage bearing walls give these walls the lateral capacity required. There are a total of 38 shear walls in the building: 17 in the North-South direction, 17 in the East-West direction, and 4 concrete masonry unit (cmu) stair towers that resist load mainly in the East-West direction. The precast garage is structurally separate, and only the 4" building separation will be considered for story drift in the lateral review.

Calculation Overview

All spot checks performed on structural components in the building were calculated using ETABS, a finite element based analysis program, and hand calculations using the shear wall analysis method and the area method. The lowest level of the building was used to check the shear wall assemblies, and the two braced frames.

All spot checks showed that the members were adequately sized for the calculated loads. The light gage shear wall straps, the structural columns and bracing in the drive aisles, and the cmu stair towers were all determined to be adequately sized for the Load and Resistance Factor Design (LRFD) loadings. It was also determined that seismic design controlled over wind in the lateral analysis. The computer output yielded loadings that made the current sizes inadequate, so a more in depth analysis of these lateral loadings will need to be determined. A check of the input and reasonability of loads will be performed to check accuracy of the computer generated data.

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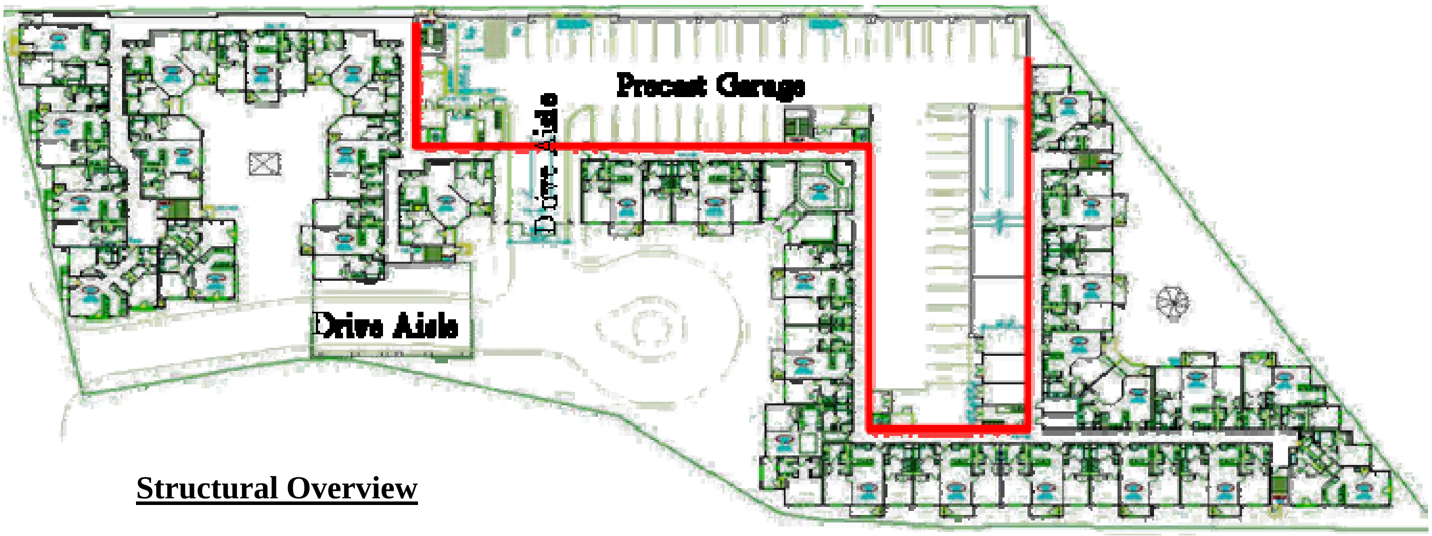
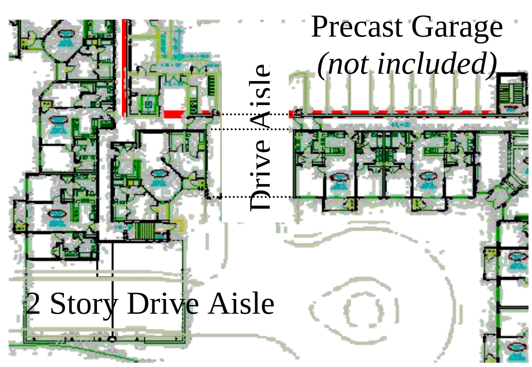


Structural Overview

The structural system for Parkview at Bloomfield Station, from the top down, is a roof composed of light gage roof trusses spaced 2’ on center (oc) spanning front to back with some hip conditions incorporated, bearing on exterior and corridor walls, and girder trusses at hip roof conditions. The bearing walls are panelized bearing light gage steel stud walls 4” and 6” wide continuously capped with a steel tube, HSS 4x4x5/16” and HSS 6x4x5/16” respectively, for load distribution purposes. Beams and transfer beams provide bearing points for the floor system, columns, and roof trusses. A 16” deep Hambro® D500™ floor system makes up the composite rigid floor diaphragm and consists of joists spaced at 4’ oc connected to a 3” concrete floor (3000psi). The 16” joists span the short direction of the living units (typically 30’) and Hambro RTC joists (top cord only joists) span the corridor (typically 6’).



Along with the bearing light gage walls, there are two braced frame systems at the drive aisles that pass under the building. The upper floors in these sections are supported by a series of one or two story columns that are part of this W18 braced frame system. All 6 floors of the building have mainly the same floor plans with the exception of 4 locations: an entry/lobby unit, a 2 story drive aisle, a 1 story drive aisle, and a

1st floor exit route. In these areas, transfer beams are utilized creating much larger beam sizes. A two story braced frame system is utilized in the 2 story drive aisle, consisting of 19 W18 columns placed along bearing lines. There is a similar system at the one story drive aisle consisting of 12 columns. While these braced frames act as the lateral force resisting system in these two unique areas, the main lateral force resisting system for the building is a shear wall system provided by thin steel cross bracing straps attached to the light gage shear walls.

There are a total of 38 shear walls in the building, consisting of 17 in the North-South direction, 17 in the East-West direction, and 4 concrete masonry unit (cmu) stair towers that resist load mainly in the East-West direction. Due to the fact that the shear walls are fairly evenly spaced and similar in thickness and length, a fairly even load distribution is present throughout the entire building.

Finally, continuous 2'-6" wide footings make up most of the building bearing wall support under the 4" slab-on-grade foundation. However, larger spread footings (typically 4'x4') are utilized below leaning column point loads. The spread footings at the drive aisle's braced frames merge together and resemble larger single spread footings. The precast garage's footings are separate from that of the main building and encompass a deep foundation system rather than the buildings shallow footing system.

The precast garage located at the center of the building, consists of precast double-T planks bearing on load bearing elements. The vertical elements in the garage transfer their load to pile caps encompassing 100 ton H piles drilled to bedrock (ranging from 42-53 ft below the slab-on-grade surface). The precast garage is structurally separated from the main building by a 4" air gap and by 4" expansion joints at building connection points. Because of this, the garage will not be considered in this building analysis. Furthermore, due to the overall rigidity of this parking structure, which has an assumed deflection of 1", the main building is allowed to have a building drift of up to 3" in the direction of the garage.

Lateral Check Overview

This report takes a closer look at the loads and load cases used in the design and check of Parkview at Bloomfield Station. A reasonable method of distributing these loads to the shear walls throughout the building is checked using a computer program utilizing finite element analysis and the shear wall analysis method. Finally, story drift and overturning of the building are considered.

Load Design Theory

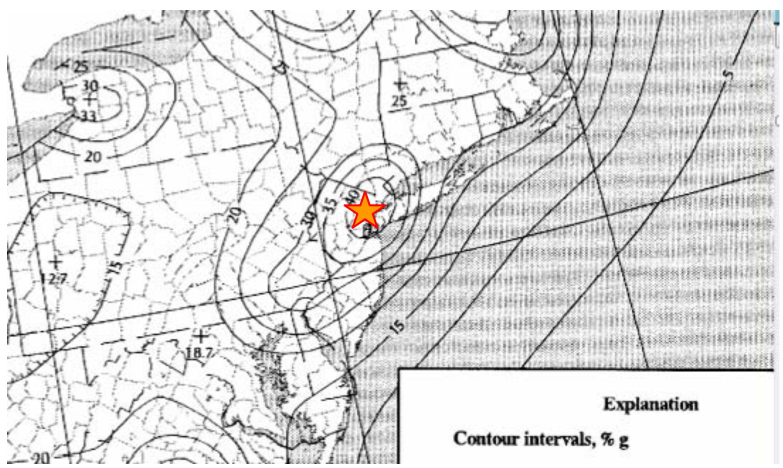
The design theory used in the analysis of Parkview at Bloomfield Station was Allowable Stress Design (ASD). The beam calculations were designed using the American Institute of Steel Construction (AISC) 9th Edition ~ ASD and designed using the Enercalc® program (ASD based). The tube steel leaning columns were also designed based on the column tables in chapter 3 of the AISC 9th Edition ~ ASD.

The lateral spot check of the building using the ETABS program is based on Load and Resistance Factor Design (LRFD), ACI 318-99, and loads from ASCE-7. The LRFD design will yield a change in the overall loads calculated as compared to the ASD design calculations. However, LRFD should yield equal sized or slightly smaller members than those designed due to the effect of the phi factors as compared to ASD factors of safety.

<u>Lateral Code References</u>

The design of the structure was in accordance with the International Building Code (IBC) 2000 with New Jersey (NJ) amendments, the NJ Uniform Construction Code, and local county and township requirements. IBC 2000 used design loads specified in ASCE 7 for both gravity and lateral loadings. Furthermore, the NJ amendments to IBC 2000 did not create any changes to the structural code requirements of IBC 2000, but focused more on non-structural issues throughout the code. In addition, no changes to the structural design requirements were added by the NJ Uniform Construction Code or any of the local requirements.

Since Bloomfield, NJ is located at the center of an east coast seismic epicenter, seismic has a much larger effect on the lateral analysis. Similarly, Bloomfield is located near the coast line, so it also experiences greater wind speeds (basic wind speed of 110 mph). It was determined that the effects of seismic loading, while close to the loading incurred by wind, created larger forces to be resisted in the shear walls.

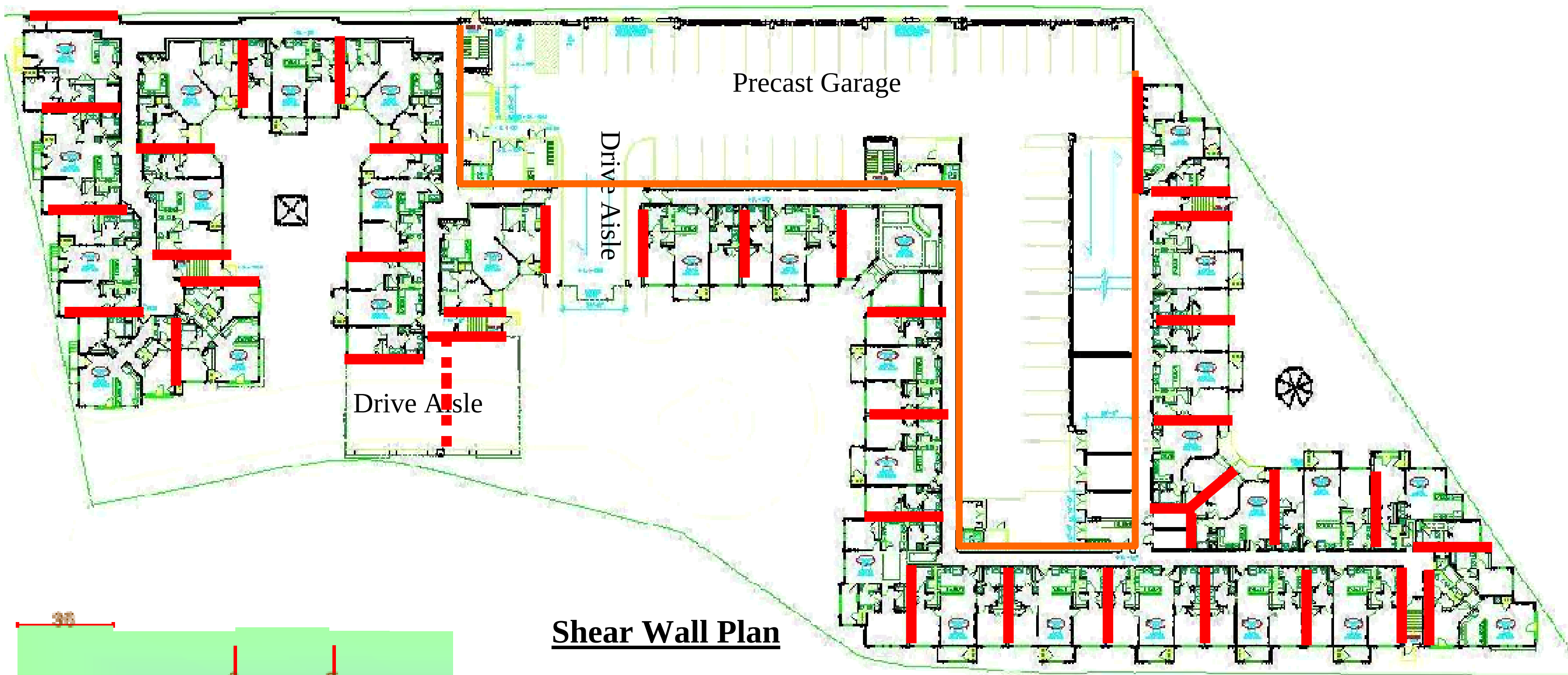


Original Wind and Lateral Load Overview

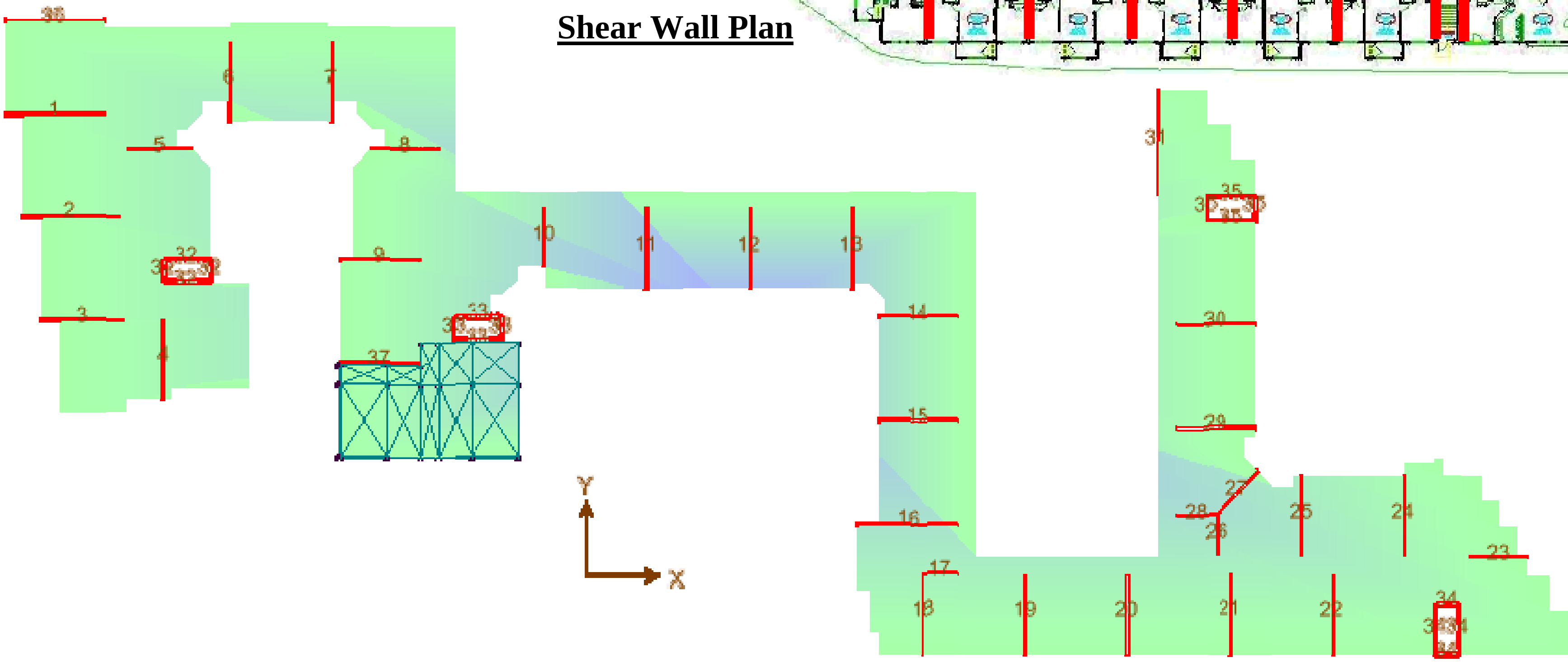
SNOW LOADS: ASCE 7-98
GROUND SNOW LOAD (Pg) = 30 PSF
EXPOSURE FACTOR (Ce) = 0.8 (TABLE 7-2, TERRAIN D)
THERMAL FACTOR (Ct) = 1.1 (TABLE 7-3)
IMPORTANCE FACTOR (I) = 1.00 (TABLE 7-4)
ROOF SLOPE FACTOR (Cs) = 1.0 WITH ROOF PITCH < 8:12 (FIG 7-2)
FLAT ROOF SNOW LOAD (Pf=0.7 Ce Ct I Pg) = 21 PSF
SLOPED ROOF SNOW LOAD (Ps=Cs Pf) =21 PSF

WIND LOAD: WIND LOADS ARE EVALUATED IN ACCORDANCE WITH PROVISIONS OF SECTION 1609 OF THE 2000 INTERNATIONAL BUILDING CODE BASED ON THE FOLLOWING PARAMETERS:
- BASIC WIND SPEED 110 MPH (3-SECOND GUST WIND SPEED)
- WIND LOAD IMPORTANCE FACTOR: 1.0
- WIND EXPOSURE CATEGORY: EXPOSURE D

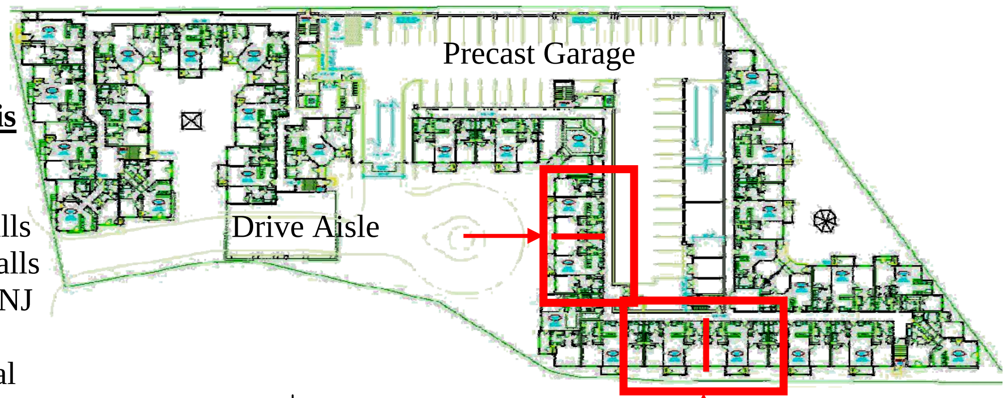
EARTHQUAKE LOADS: EARTHQUAKE LOADS ARE EVALUATED IN ACCORDANCE WITH PROVISIONS OF SECTION 1615 OF THE 2000 INTERNATIONAL BUILDING CODE BASED ON THE FOLLOWING PARAMETERS:
- MAX. EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS, Ss = 0.43g
- MAX. EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT 1 SECOND, S_1 = 0.095g
- SITE CLASS = F
- EARTHQUAKE LOAD IMPORTANCE FACTOR: 1.00
- MAX. CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS, Sms = 0.77g
- MAX. CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT 1 SECOND, S_{M1}= 0.20g



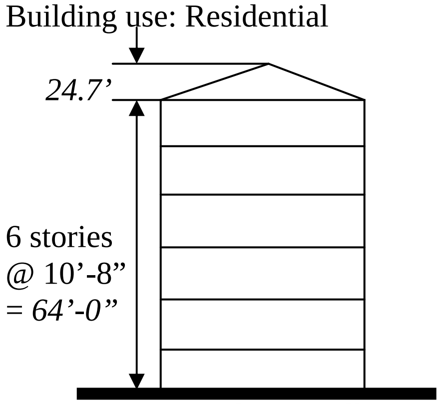
Shear Wall Plan



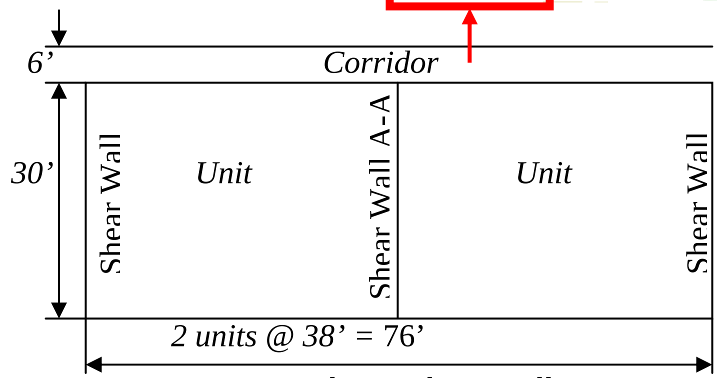
Original Wind Analysis



Building information
N-S direction: Shear walls
E-W direction: Shear walls
Locations: Bloomfield, NJ
Exposure: B
Building use: Residential



Section @ Shear Wall A-A



Plan at Shear Wall

WIND DIRECTION: NORTH-SOUTH (Y-DIR) # Stories: 6

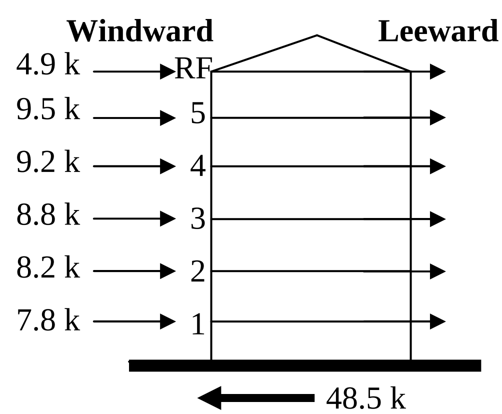
WIND SPEED:	110.00 MPH	L :	36.000 ft	FREQ n_1:	1.9300 Hz	Ground to	
EXP. CAT:	B	B :	38.000 ft	ALPHA=	7	Base h:	0.000 ft
IMPORT. FACTOR:	1.00	Mean Roof h :	76.40 ft	Zg (ft)=	1200 ft		
DIREC. FACT. Kd:	0.85	Kh =	0.915	G=	0.8	Wind Load	
TOPOG. FACT Kzt:	1.00 (see sht. Kzt)	L/B =	0.947	Gf=	0.800	to be applied	
$0.00256\ K_z\ K_{zt}\ V^2\ I$ =	26.33 psf	Cp (wind)	0.8			at Yo:	0.000 ft
		Cp (leew)	-0.50				

WIND FORCE CALCULATION PER ASCE7-02 - MAIN WIND FORCE RESISTING SYSTEM

(Same results on the E-W shear wall)[1]

FLOOR I.D.	FL TO FL HEIGHT (ft)	TRIB. WIDTH (ft)	Kz	Wind Press Wind	Wind Press Lee	WIND PRESS. total (psf)	WIND FORCE (Kips)	FLOOR SHEAR (Kips)	FLOOR MOMENT (Kip-ft)	Case 1 Mz (Kip-ft)	Case 2 Mz (Kip-ft)	
6	10.667	38.000	0.870	14.659	9.637	24.30	4.9	4.9	52.5	0.0	5.8	-5.8
5	10.667	38.000	0.826	13.915	9.637	23.55	9.5	14.5	206.9	0.0	11.3	-11.3
4	10.667	38.000	0.775	13.056	9.637	22.69	9.2	23.7	459.4	0.0	10.9	-10.9
3	10.667	38.000	0.714	12.025	9.637	21.66	8.8	32.5	805.5	0.0	10.4	-10.4
2	10.667	38.000	0.636	10.710	9.637	20.35	8.2	40.7	1,239.7	0.0	9.8	-9.8
1	10.667	38.000	0.575	9.685	9.637	19.32	7.8	48.5	1,757.3	0.0	9.3	-9.3
	0.000	38.000				0.00	0.0					

The loading on the diagram (right) represents the loading on a single shear wall in the building. These loads are compared with those calculated in the following seismic section, and the overall larger of the 2 loading sets will be used in the design of the shear wall system. This was initially how the design of the system was carried out. The design specified exposure class D which is conservative, does not correspond with the recommendations of the Geo-tech report for exposure B. This makes the design values more conservative and is the only area of difference.

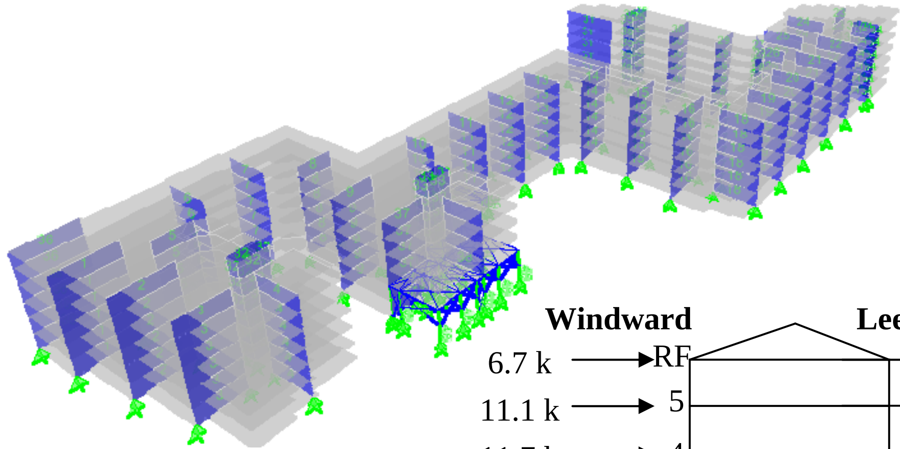


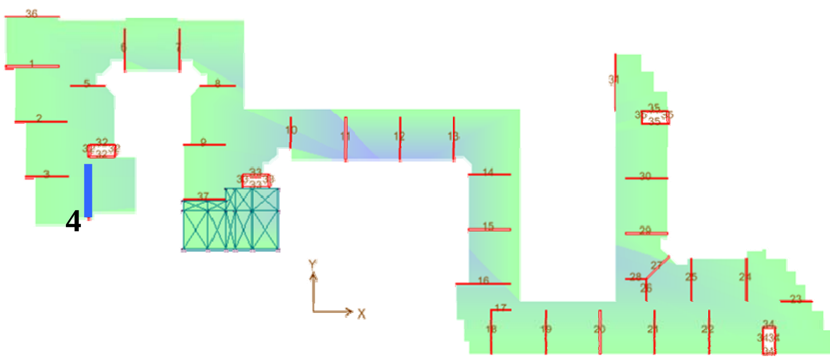
Section @ Shear Wall A-A

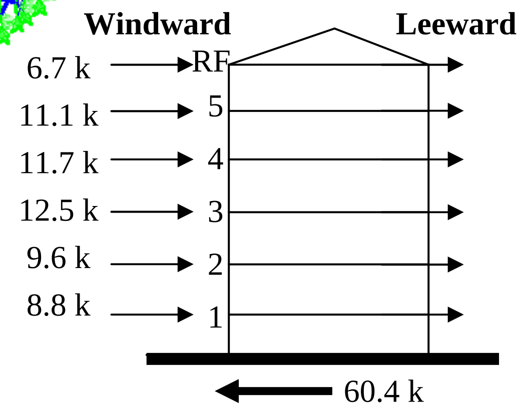
[1] I have not yet learned all the aspects of this code; see Tech 3 for a more complete analysis of the building's wind and dynamic seismic loading.

Computer Wind Analysis

Building information
N-S direction: Shear walls
E-W direction: Shear walls
Locations: Bloomfield, NJ
Exposure: B
Building use: Residential







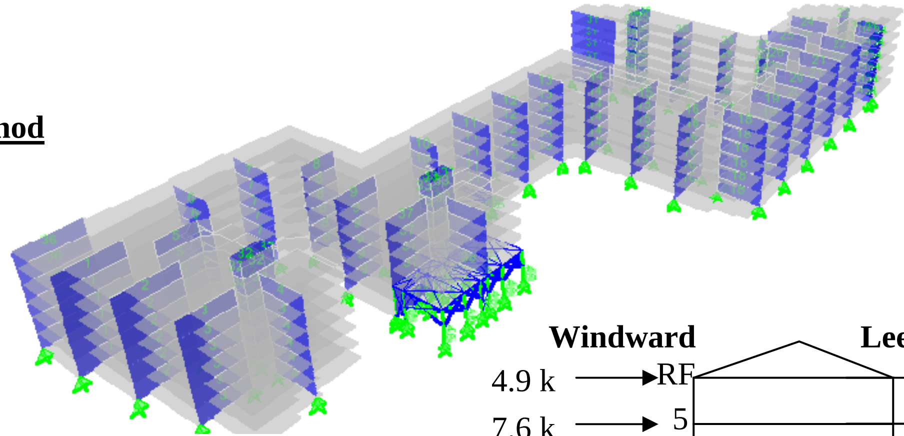
Section @ Shear Wall 4

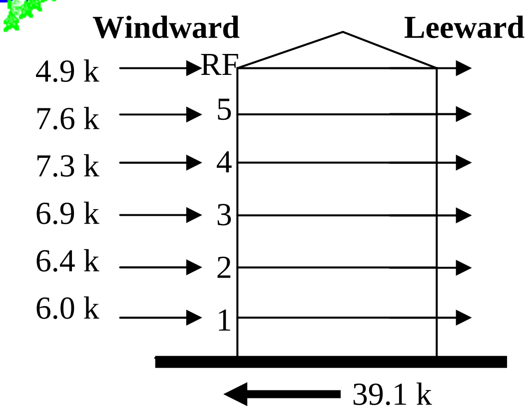
The loading on the diagram represents the loading on shear wall number four (see above). This is the shear wall that is required to resist the largest wind load in the building. The loads on this wall are nearly ¼ times larger than the load that was originally calculated. The load in this wall appears to be greater than those around it because it is the only wall in the vicinity that is oriented in the North-South direction, making it solely responsible for resisting the forces in that direction. Additionally, it is located away from the center of rigidity of the system, requiring it to contribute greatly towards resisting torsion in the building.

Of the four wind load approaches utilized in this report, this analysis yielded the largest base shear. This is due to the ability of the finite element analysis program to distribute the forces more accurately to each member in the system. Another possibility for the increased force as compared to all the other results could be a computer input error, which after reviewing the seismic loadings, is a distinct possibility. While this loading of the shear wall (60.4k) is the largest created from the four wind loading approaches, the seismic loading still controls the overall design based on the building location.

Shear Wall Analysis Method

Building information
N-S direction: Shear walls
E-W direction: Shear walls
Locations: Bloomfield, NJ
Exposure: B
Building use: Residential

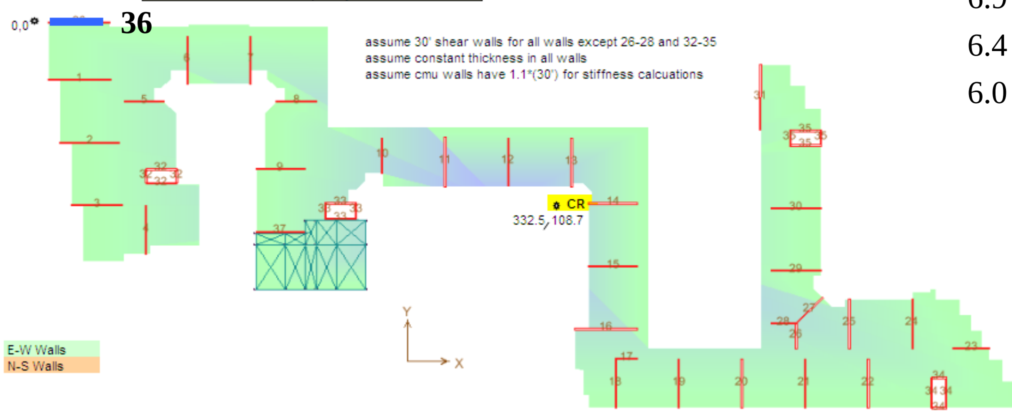




Section @ Shear Wall 36

story # =	1	Story h	120	in	bldg Lx	583	ft ==>	ecc. x=	-41.0	Tx=	-1444.4 ft-K
wind psf	15 psf	Wall L	360	in	bldg Ly	235	ft ==>	ecc. y=	8.8	Ty=	770.3 ft-K
Px=	35.25 Kip	Wall t	4	in						T total=	-674.1 ft-K
Py=	87.45 Kip	Aw =	17280	in^2	CR x=	332.5	ft				
		I =	15552000	in^4	CR y=	108.7	ft				

Robert Whitaker



(For all results of shear walls on this floor see page A 14)

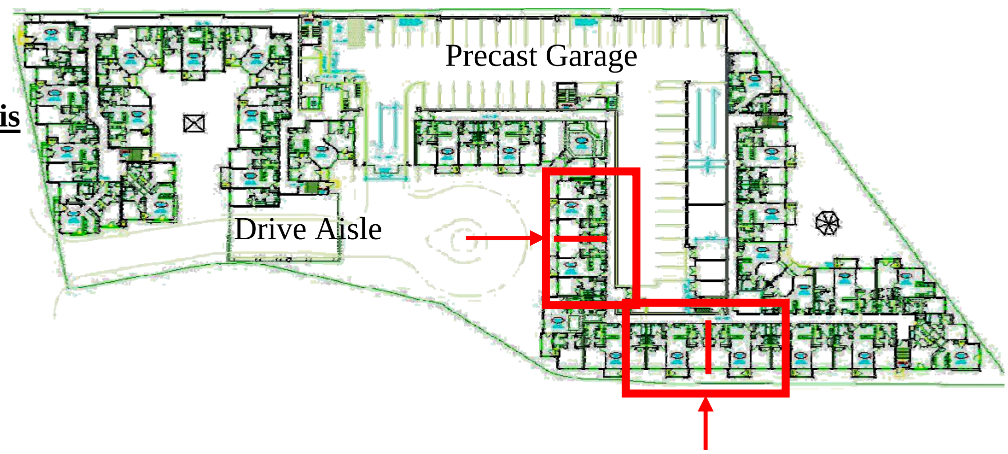
	Dist from 0,0				stiffness															
Wall #	y	x	(h/L)	Wall type	k=	k*y	k*x	y from C	x from C	K*y^2	K*x^2	ky/Σky^2	kx/Σkx^2	Direct V	T shear	Direct V	T shear	Total (K)	wall
1	34.5		0.33	Intermediate	3.722	128.4	0.0	74.19	0.00	20485	0	0.00127	0	0.00	0.00	4517.97	977.47	5.50	1
34		525.5	0.30	Intermediate	4.194	0.0	2204.0	0.00	-193.03	0	156265	0	-0.00053	2194.51	758.44	0.00	0.00	2.95	34
35	68.5		0.30	Intermediate	4.194	287.3	0.0	40.19	0.00	6775.1	0	0.00077	0	0.00	0.00	5091.56	596.76	5.69	35
36	0.1		0.33	Intermediate	3.722	0.4	0.0	108.59	0.00	43886	0	0.00186	0	0.00	0.00	4517.97	1430.69	5.95	36
37	125.5		0.33	Intermediate	3.722	467.1	0.0	-16.81	0.00	1051.4	0	-0.0003	0	0.00	0.00	4517.97	-221.44	4.30	37
38		162	0.33	Intermediate	3.722	0.0	602.9	0.00	170.47	0	108155	0	0.00041	1947.28	-594.37	0.00	0.00	1.35	38
				sum Ky	72.03	7829.6	22398.3		sum Rx^2	217576	1541717					Σ forces x :	35.25	kip	
				sum Kx	67.37	sum K*dist										Σ forces y :	87.45	kip	

The Shear wall analysis method is based off of the combination of direct shear and torsional shear effects on individual shear walls. With this in mind, it is evident why shear wall number 36 has the largest load. This wall is located the furthest away from the center of rigidity (CR) and is located so that the torsional effects are purely additive with the direct shear. Since it is located away from the CR it will be responsible for a larger portion of the torsion in the building.

Even with the additional torsion load acting on the wall, this shear wall has the least amount of load as compared to the other three wind loaded shear walls being compared. This lack of load is attributed to the procedure of the shear wall analysis method. In this method, walls oriented perpendicular to the direct load still carry portions of the torsional shear. This creates less loading in the parallel oriented walls, and helps to make the system work together. When a wind load is applied in both the x and y direction at the same time, the loading will increase, unlike what happens in the area method analysis.

Area Method Wind Analysis

Building information
N-S direction: Shear walls
E-W direction: Shear walls
Locations: Bloomfield, NJ
Exposure: B
Building use: Residential



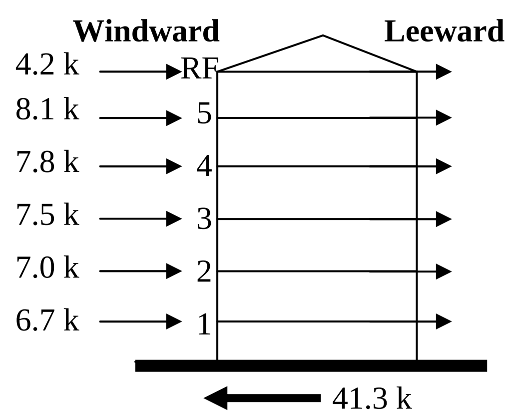
WIND SPEED:	110.00	MPH	L :	235.000	ft	FREQ n_1:	1.9300	Hz	Ground to Base h:	0.000 ft
EXP. CAT:	B		B :	583.000	ft	ALPHA=	7			
IMPORT. FACTOR:	1.00		Mean Roof h :	76.40	ft	Zg (ft)=	1200	ft		
DIREC. FACT. Kd:	0.85		Kh =	0.915		G=	0.8		Wind Load to be applied at Yo:	0.000 ft
TOPOG. FACT Kzt:	1.00	(see sht. Kzt)	L/B =	0.403		Gf=	0.800			
$0.00256\ K_z\ K_{zt}\ V^2\ I$ =	26.33	psf	Cp (wind)	0.8						
			Cp (leew)	-0.50						

WIND FORCE CALCULATION PER ASCE7-02 - MAIN WIND FORCE RESISTING SYSTEM

FLOOR I.D.	FL TO FL HEIGHT (ft)	TRIB. WIDTH (ft)	Kz	Wind Press Wind	Wind Press Lee	WIND PRESS. total (psf)	WIND FORCE (Kips)	FLOOR SHEAR (Kips)	FLOOR MOMENT (Kip-ft)	Case 1 Mz (Kip-ft)	Case 2 Mz (Kip-ft)	
6	10.667	583.000	0.870	14.659	9.637	24.30	75.5	75.5	805.9	0.0	1,376.2	-1,376.2
5	10.667	583.000	0.826	13.915	9.637	23.55	146.5	222.0	3,174.1	0.0	2,668.2	-2,668.2
4	10.667	583.000	0.775	13.056	9.637	22.69	141.1	363.1	7,047.8	0.0	2,570.8	-2,570.8
3	10.667	583.000	0.714	12.025	9.637	21.66	134.7	497.9	12,358.5	0.0	2,454.1	-2,454.1
2	10.667	583.000	0.636	10.710	9.637	20.35	126.5	624.4	19,019.0	0.0	2,305.1	-2,305.1
1	10.667	583.000	0.575	9.685	9.637	19.32	120.2	744.6	26,961.2	0.0	2,188.9	-2,188.9
	0.000	583.000				0.00	0.0					

(Results shown are higher than those on the E-W shear walls)

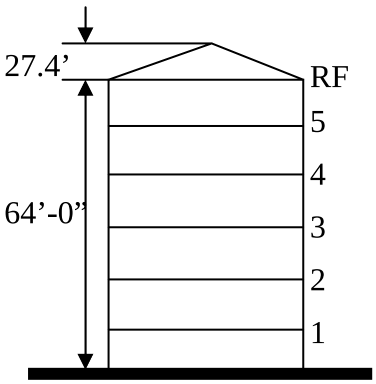
The loading on the diagram (right) represents the loading on a single shear wall in the building. These loads were calculated differently than the original loadings from Tech #1 in that they are based off of the tributary area of the entire building. The wind force loads in the table above were divided by the number of shear walls oriented in the East-West direction (18) in order to obtain the force on any one of the shear walls. This approach yielded a base shear that was 17% lower than the area method of a single bay. This approach is quick to analyze but does not appear to have the sophistication or real life effects that are taken into account in either the computer analysis or the shear wall analysis method. It does not take into account any out-of-plane loadings, or any effects from out-of-plane shear walls.



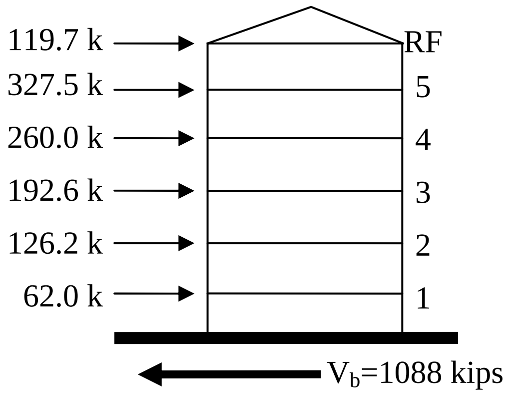
Section @ Shear Wall A-A

The computer model and the shear wall analysis method appear to be the most accurate of the wind loading analysis. Therefore, the larger of the computer model and shear wall analysis (60.4k) will be compared with the results of the seismic analysis. All tabulated values for the wind loading section can be found in the appendix.

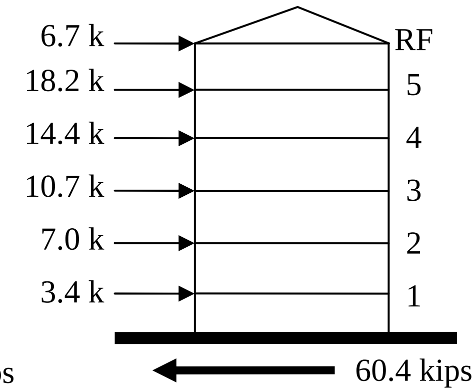
Original Seismic Analysis



Building Dimensions



Total Shear on Building Section

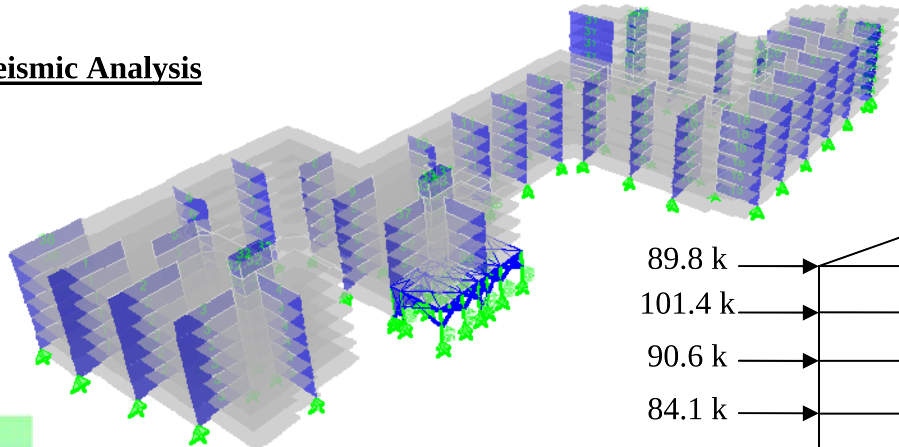


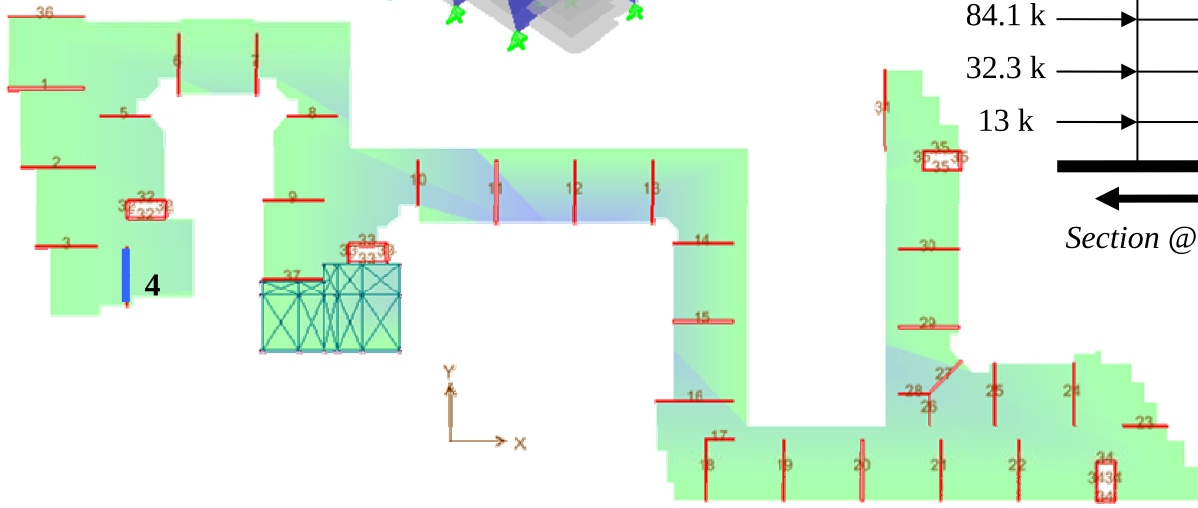
Section @ Shear Wall A-A

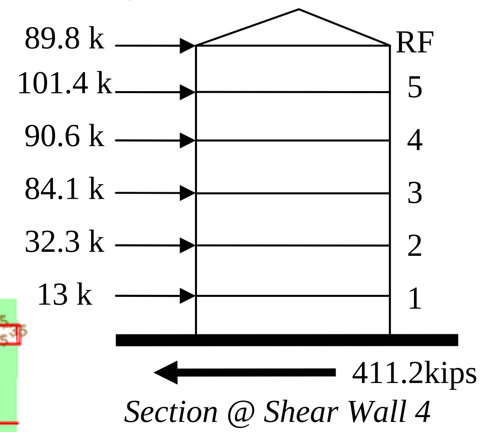
Floor	**$W_x h_x$^1.039**	**Cvx**	**Fx=Cvx*Vb**	**Fx/18**
Roof	63,339.6	0.110	119.7 k	6.7 k
5	172,697.4	0.301	327.5 k	18.2 k
4	136,960.8	0.239	260.0 k	14.4 k
3	101,574.6	0.177	192.6 k	10.7 k
2	66,654.0	0.116	126.2 k	7.0 k
1	32,438.1	0.057	62.0 k	3.4 k
sum	*573,664.5*	*1.000*	*1088.0 k*	*60.4k*

There are 18 shear walls in the North-South direction and 20 shear walls in the East-West direction (see next page for shear wall locations). Therefore, the critical direction is the North-South direction, and an individual shear wall needs to have 1/18th the capacity of the total floor shear, resulting in the loading shown on the diagram of section A-A. In reality there may be some walls that carry more than 1/18th of the total floor shear due to distribution, but this was ignored until Tech 3. With the individual shear wall loads computed, it is evident that seismic loading will control the design of the shear walls (60.4 k seismic > 48.5 k wind).

Computer Seismic Analysis





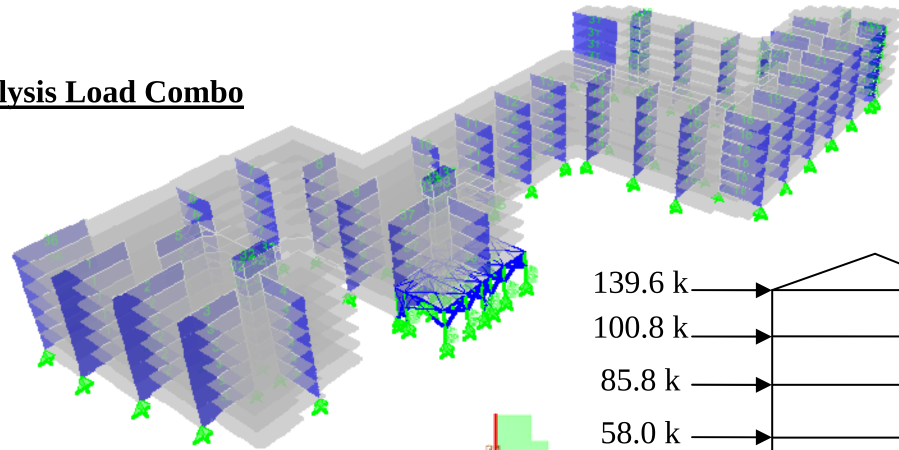


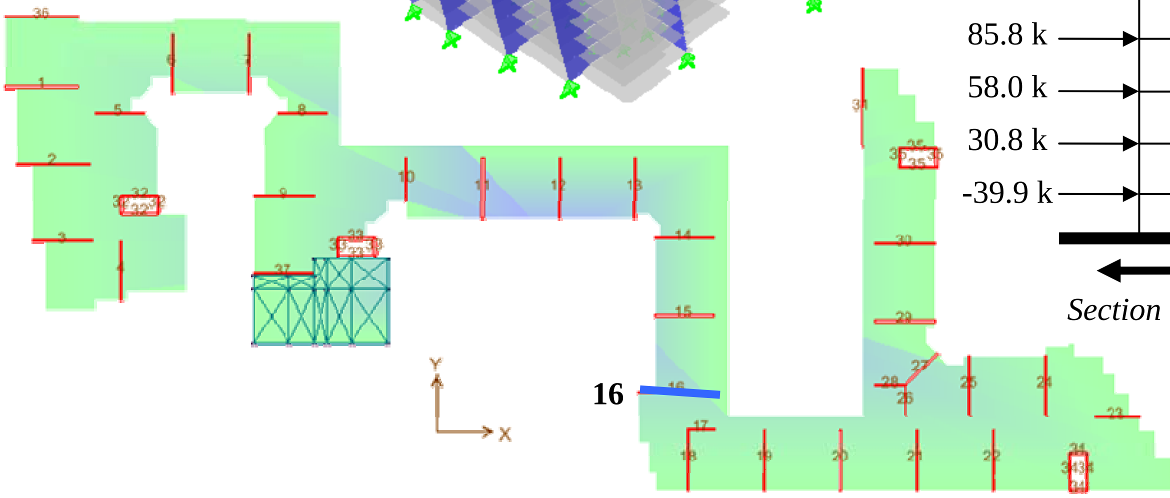
Based on the computer analysis using a six mode seismic analysis, it is determined that shear wall number four controls not only wind but also the seismic design. This is the shear wall that is required to resist the largest seismic load in the building. The loads on this wall are nearly seven times larger than the load that was originally calculated.

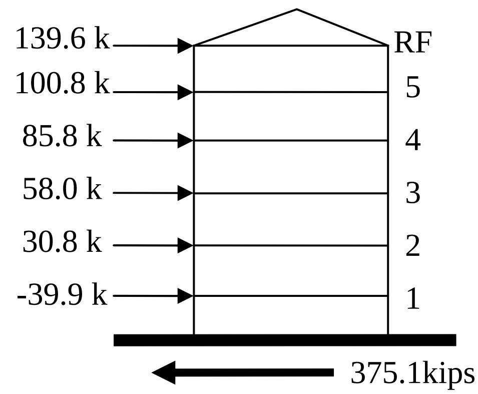
This extreme change in loading on the wall means that either the modal analysis, as compared to the simplified seismic approach, made this much of a difference or there is a problem with the computer input. While the load in this wall appears to be greater than those around it because it is the only wall in the vicinity, it does not account for the extreme jump in values. Furthermore, the finite element modal seismic analysis will create some increases in loading and distribution to certain members, but the value obtained appears to be in error. A more extensive search into the cause of this situation will need to be carried out at a later point. To be overly conservative, the shear walls will be designed based on these seismic loadings, to account for the possibility of the values being correct.

Computer Analysis Load Combo

LL+DL+ELx







Section @ Shear Wall 16

The live, dead and seismic loading combination creates a large load on shear wall number 16. The interesting thing about this load combination is that it created a negative force on the lowest level, caused by the oscillation of the upper floors during an earthquake. This causes the base shear to be less than the purely seismic loading.

The maximum loading (415k) actually occurs at the interface at the 2nd floor, and is greater than the seismic case just evaluated. This leads to the need to brace the lowest level to the same amount as the 2nd floor. This can be determined from the following page, and will require the lower two floors to be braced with (8) DTN5 12” 12 gage straps.

This wall also appears to be overly loaded which further confirms the assumption that there is an error somewhere in the computer analysis. However, the reverse in loading is correct because of the location of the wall at a building corner intersection. This could be accounted for by the motion of the right half of the building with respect to the left. The design of the shear walls will be determined using shear wall number four and the seismic loading on the previous page.

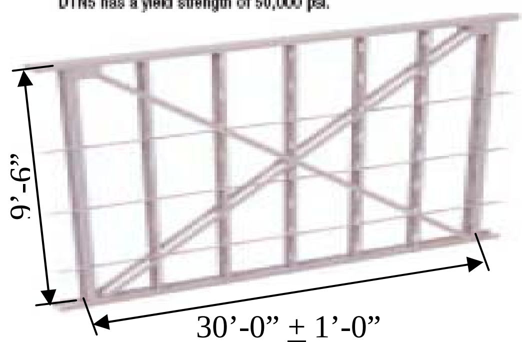
Shear Wall Design[2]

Strapping (DT Series™)

DMF Product Code	Thickness				Size Available				Length		Packaging
	Gauge	Mils	Design Thickness		Min. Width		Max. Width				Pcs./ Skid
			Inches	(mm)	Inches	(mm)	Inches	(mm)	ft.	(m)	
DTN3	20	33	0.0346	0.879	2	50.8	12	305	10	3.05	250
	18	43	0.0451	1.146	2	50.8	12	305	10	3.05	250
	16	54	0.0566	1.438	2	50.8	12	305	10	3.05	250
	14	68	0.0713	1.811	2	50.8	12	305	10	3.05	250
	12	97	0.1017	2.583	2	50.8	12	305	10	3.05	250
DTN5	16	54	0.0566	1.438	2	50.8	12	305	10	3.05	250
	14	68	0.0713	1.811	2	50.8	12	305	10	3.05	250
	12	97	0.1017	2.583	2	50.8	12	305	10	3.05	250

DTN3 has a yield strength of 33,000 psi.
DTN5 has a yield strength of 50,000 psi.

www.dietrichmetalframing.com



Floor	Strap Size		P Allow	P Actual
RF	DTN5 4"	12 gage	9.8 k	6.7 k
5	DTN5 6"	12 gage	29.2 k	24.9 k
4	DTN5 10"	12 gage	48.6 k	39.3 k
3	DTN5 12"	12 gage	50.8 k	50.0 k
2	DTN5 12"	12 gage	58.3 k	57.0 k
1	DTN5 12" DTN5 4"	12 gage 12 gage	68.1 k	60.4 k

A converted tensile capacity based on the allowable axial tension of a 12 Gage 12" wide shear cross bracing strap raised at a 17° angle from the floor is used to check the shear straps at the lowest level. The strap allows a tensile load of $P_{allow} = A_s*F_y*\cos\theta$ = (12"*0.1017")*50ksi*cos 17° = 58.3 k $\leq$ 411.2 k and is therefore not acceptable for the lowest floor. An additional 7 straps will need to be added at the lowest floor making P_{allow} = 7*[(12")*0.1017"]*50ksi*cos 17° = 466.4 k $\geq$ 411.2 k which is now acceptable.

Using the same approach, the shear cross bracing strap sizes for the other floors is determined and is listed on the chart below. These sizes are larger than the original design sizes for the shear walls, and require more straps. The difference may be accounted for in the fact that a modal seismic analysis was used in the computer program rather than the simplified procedure that was originally used.

Floor	Strap Size		P Allow	P comp.
RF	(2) DTN5 12"	12 gage	116.6 k	89.8 k
5	(4) DTN5 12"	12 gage	233.2 k	191.2 k
4	(5) DTN5 12"	12 gage	291.5 k	281.8 k
3	(7) DTN5 12"	12 gage	408.1 k	365.9 k
2	(7) DTN5 12"	12 gage	408.1 k	398.2 k
1	(8) DTN5 12"	12 gage	466.4 k	411.2 k

[2] I have not ruled out a computer error for this unusually large scale loading of this particular shear wall. If an error is found, the results will be updated.

Story and Building Drift

(Deflection at the base floor):

h/L=9.5'/30'= 0.316

K = (Et)/[4(h/L)^3+2.78(h/L)] = 129890 k/in

I = t*L^3/12 = [4.5" * (30'*12)^3]/12 = 17496000 in^4

$$\Delta = \frac{Ph^3}{3EI} + \frac{2.78Ph}{A_wE} = \frac{60.4k * (9.5'*12)^3}{3*29000ksi*I} + \frac{2.78 * 60.4k*9.5'*12}{4.5"*30'*12*29000ksi} = 0.00046"$$

H/400 = 9.5'*12/400 = 0.285" > 0.00046" therefore ok

Total Building H/400 = 64'*12/400 = 1.92"

Computer Drift Analysis

The calculated maximum drift from the computer method (in feet) was nearly 2.5 times the value produced by hand, and may be due to the higher seismic loads that were calculated during the computer analysis. The deflection is still well below the allowable H/400 = 0.285 > 0.013178"*12=0.15814" and also below the total building limit of 1.92"≥ .059008*12 = 0.158136". This also shows that the 4" air gap between the garage and main building is an adequate separation.

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY	x=	0.013178	max drift
STORY2	Max Drift X	LDWXY	1404	-738.36	521.208	240	0.000177				
STORY2	Max Drift Y	LDWXY	1410	-657.36	845.208	240		0.000097			
STORY1	Max Drift X	DEAD	1404	-738.36	521.208	120	0.000148				
STORY1	Max Drift Y	DEAD	1410	-657.36	845.208	120		0.000082			
STORY1	Max Drift X	WINDX	1407	-300.36	521.208	120	0.000002				
STORY1	Max Drift Y	WINDX	1411	-738.36	845.208	120		0			
STORY1	Max Drift X	WINDY	1406	-513.36	521.208	120	0.000004				
STORY1	Max Drift Y	WINDY	1402	-1095.36	521.208	120		0.000008			
STORY1	Max Drift X	SEISMICY	1406	-513.36	521.208	120	0.000033				
STORY1	Max Drift Y	SEISMICY	1402	-1095.36	521.208	120		0.000062	x=	0.013178	max drift
STORY1	Max Drift X	SEISMICX	69	2548.64	1680.21	120	0.013178		y=	0.000138	max drift
STORY1	Max Drift Y	SEISMICX	69	2548.64	1680.21	120		0.000028			
STORY1	Max Drift X	WINDXY	1409	-513.36	845.208	120	0.000002				
STORY1	Max Drift Y	WINDXY	1402	-1095.36	521.208	120		0.000002			
STORY1	Max Drift X	LDW	1404	-738.36	521.208	120	0.000176				
STORY1	Max Drift Y	LDW	1410	-657.36	845.208	120		0.000098			
STORY1	Max Drift X	LEDX	69	2548.64	1680.21	120	0.013178				
STORY1	Max Drift Y	LEDX	1405	-657.36	521.208	120		0.000097			
STORY1	Max Drift X	LEDY	1411	-738.36	845.208	120	0.000194				
STORY1	Max Drift Y	LEDY	1418	-513.36	1025.208	120		0.000138			
STORY1	Max Drift X	LDWY	1411	-738.36	845.208	120	0.000174				
STORY1	Max Drift Y	LDWY	1410	-657.36	845.208	120		0.000103			
STORY1	Max Drift X	LDWXY	1404	-738.36	521.208	120	0.000175				
STORY1	Max Drift Y	LDWXY	1410	-657.36	845.208	120		0.0001			

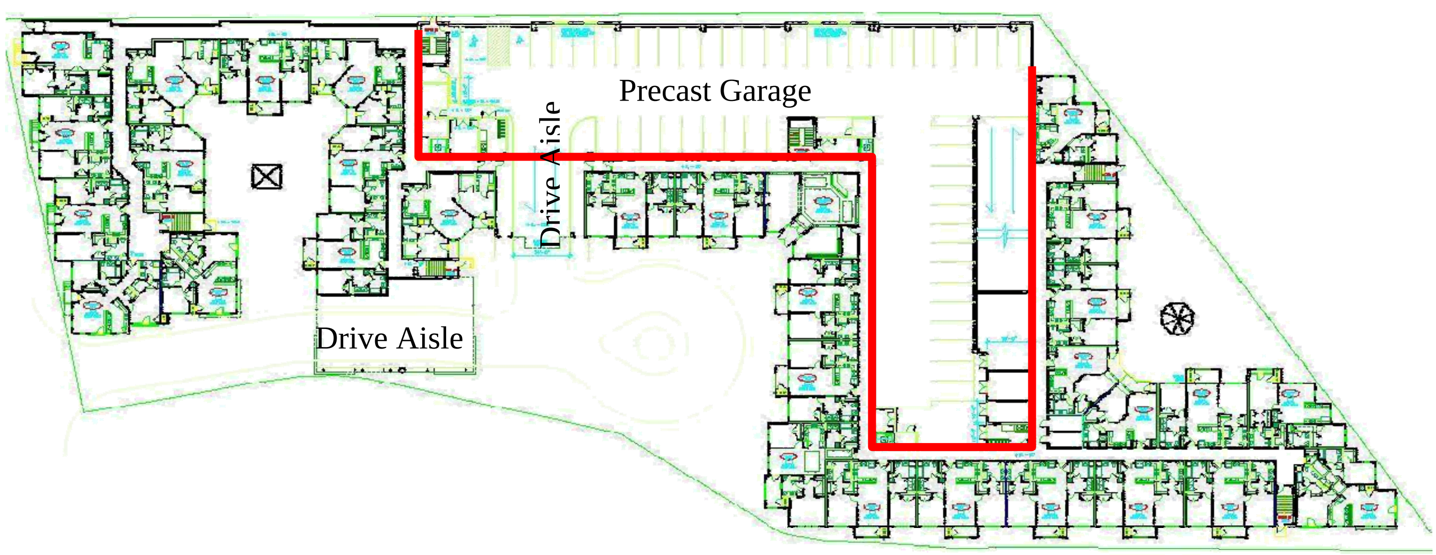
Building Overturning Moment

The overturning moment for the building is found to be 2567.2 ft-kips. In order to resist overturning, the moment created by the dead load of the structure about the edge of the building must be greater than the overturning moment. Since the resistive moment is found to be 308,543.4 ft-kips >> 2,567.2 ft-kips there will be no building overturning. See page A 16 for complete calculations.

Spot Check Overview

By utilizing the hand calculations of the wind and seismic analysis it is shown that all of the spot checks of the members are adequate. Furthermore, the initial investigation from Tech #1 into the effects of lateral loading appears to be approximately accurate. The maximum loading was confirmed to be seismic loading and adequate bracing was selected at the time of the design. The W18x60 braced frames were shown to be correct in the computer model and are adequately braced to be capable of carrying both the lateral and gravity loadings. However, assuming that the computer calculations are in fact correct, then the shear wall bracing is under sized and needs to be increased.

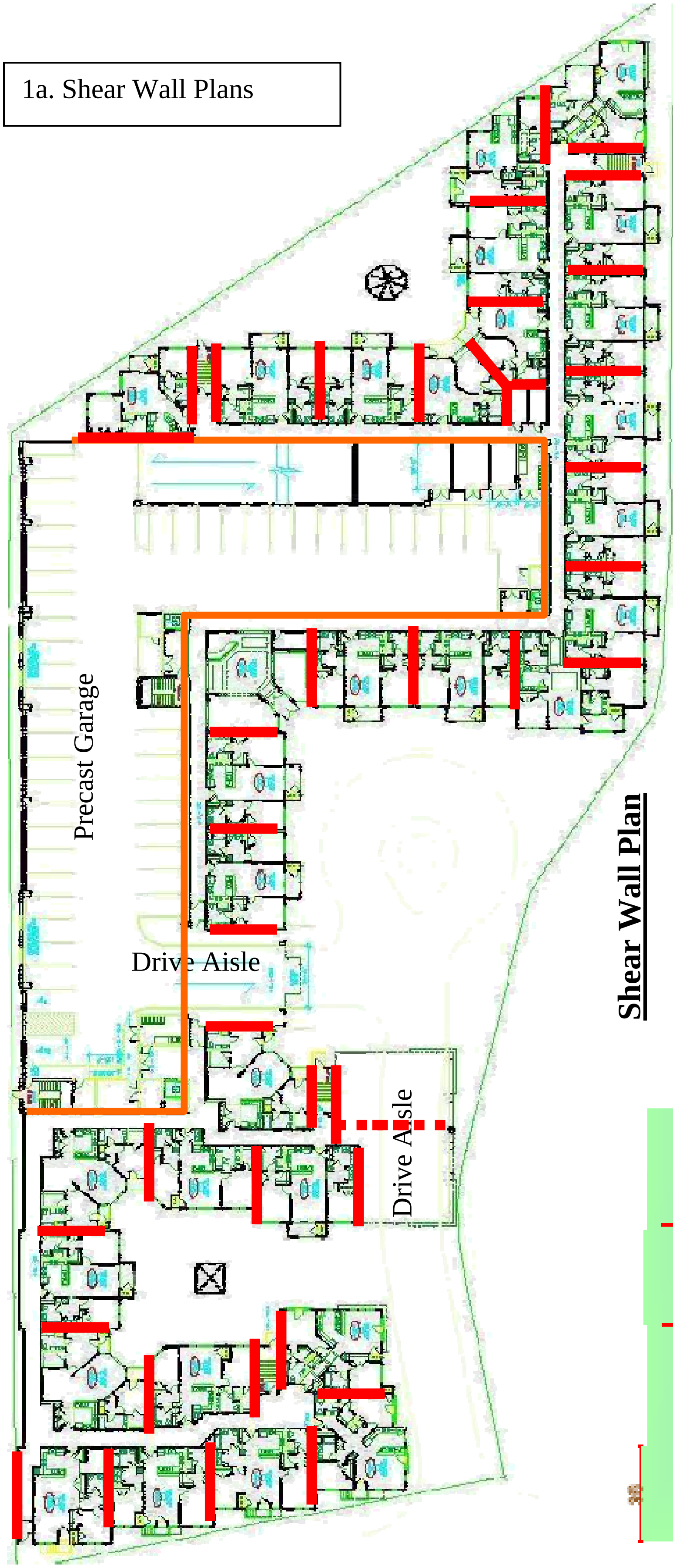
There appears to be load errors from part of the computer model, and a more extensive look into ETABS will be required. To accommodate for this conflict in values, the most conservative values should be used, or further consultations with design professionals should be utilized to determine the best approach and typical sizes used in practice. Summaries of the load calculations are included in the following appendix.



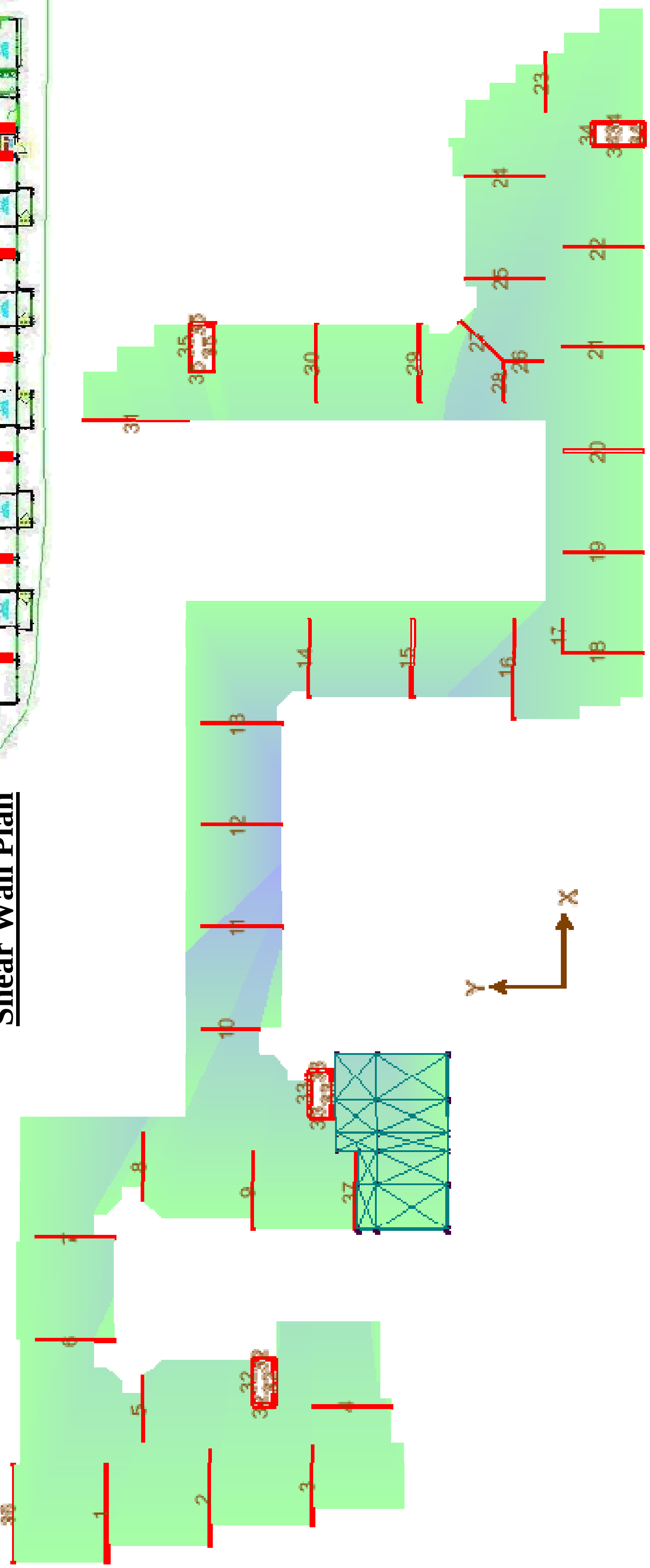
Appendix Tech Report 3

Table of Contents:

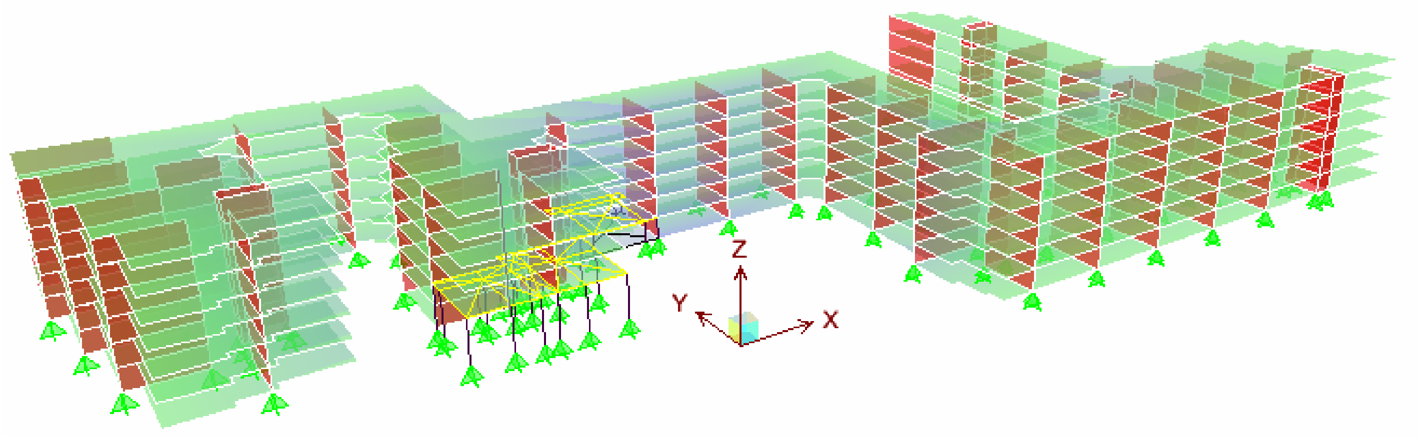
1a. Shear Wall Plans



Shear Wall Plan



1b. Frame and Existing Load Cases



Snow and lateral load overview

SNOW LOADS:
ASCE 7-98

GROUND SNOW LOAD (Pg) = 30 PSF
EXPOSURE FACTOR (Ce) = 0.8 (TABLE 7-2, TERRAIN D)
THERMAL FACTOR (Ct) = 1.1 (TABLE 7-3)
IMPORTANCE FACTOR (I) = 1.00 (TABLE 7-4)
ROOF SLOPE FACTOR (Cs) = 1.0 WITH ROOF PITCH < 8:12 (FIG 7-2)
FLAT ROOF SNOW LOAD (Pf=0.7 Ce Ct I Pg) = 21 PSF
SLOPED ROOF SNOW LOAD (Ps=Cs Pf) =21 PSF

WIND LOAD: WIND LOADS ARE EVALUATED IN ACCORDANCE WITH PROVISIONS OF SECTION 1609 OF THE 2000 INTERNATIONAL BUILDING CODE BASED ON THE FOLLOWING PARAMETERS:

- BASIC WIND SPEED 110 MPH (3-SECOND GUST WIND SPEED)
- WIND LOAD IMPORTANCE FACTOR: 1.0
- WIND EXPOSURE CATEGORY: EXPOSURE D

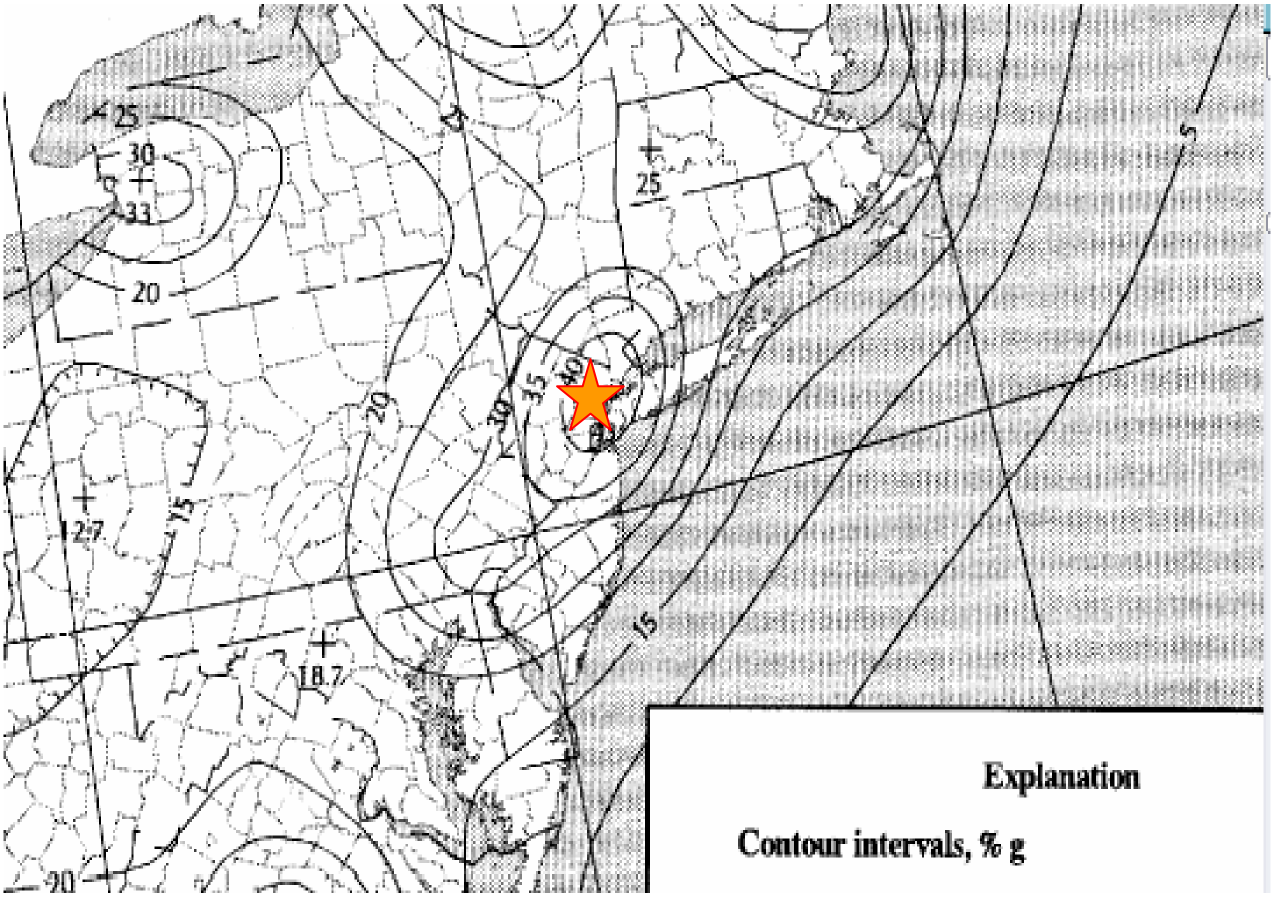
EARTHQUAKE LOADS: EARTHQUAKE LOADS ARE EVALUATED IN ACCORDANCE WITH PROVISIONS OF SECTION 1615 OF THE 2000 INTERNATIONAL BUILDING CODE BASED ON THE FOLLOWING PARAMETERS:

- MAX. EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS, Ss = 0.43g
- MAX. EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT 1 SECOND, S_1 = 0.095g
- SITE CLASS = F
- EARTHQUAKE LOAD IMPORTANCE FACTOR: 1.00
- MAX. CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS, Sms = 0.77g
- MAX. CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT 1 SECOND, S_{M1}= 0.20g

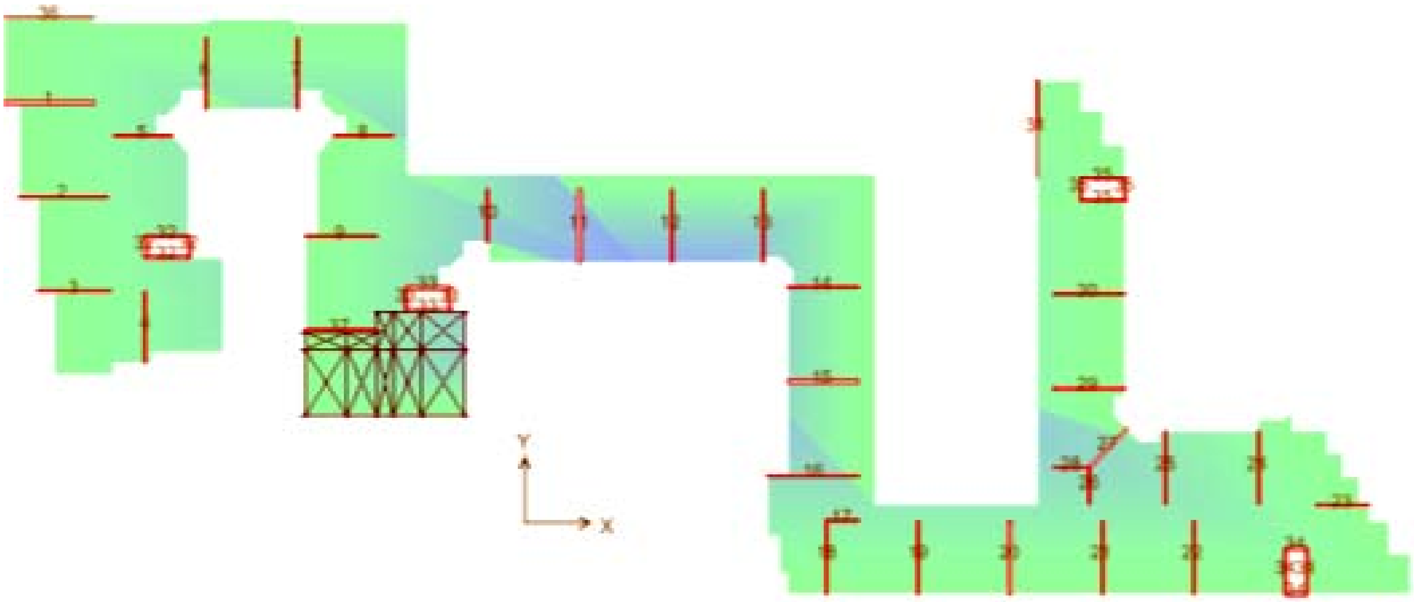
Gravity loads IBC 2000 NJ ~ ASCE 7

Location	Live Load	Dead Load	Total Load	Wall Type	Live Load	Dead Load	Wall Height	Total Load
Roof	40psf	17psf	57psf	***Single Light Gage Wall***	-	11psf	9’-6”	105plf
Unit/Balcony	40psf	57psf	97psf	***Double Light Gage Wall***	-	15psf	9’-6”	143plf
Corridor	100psf	57psf	157psf	***8” CMU Wall***	-	60psf	9’-6”	570plf
Storage	125psf	57psf	182psf					

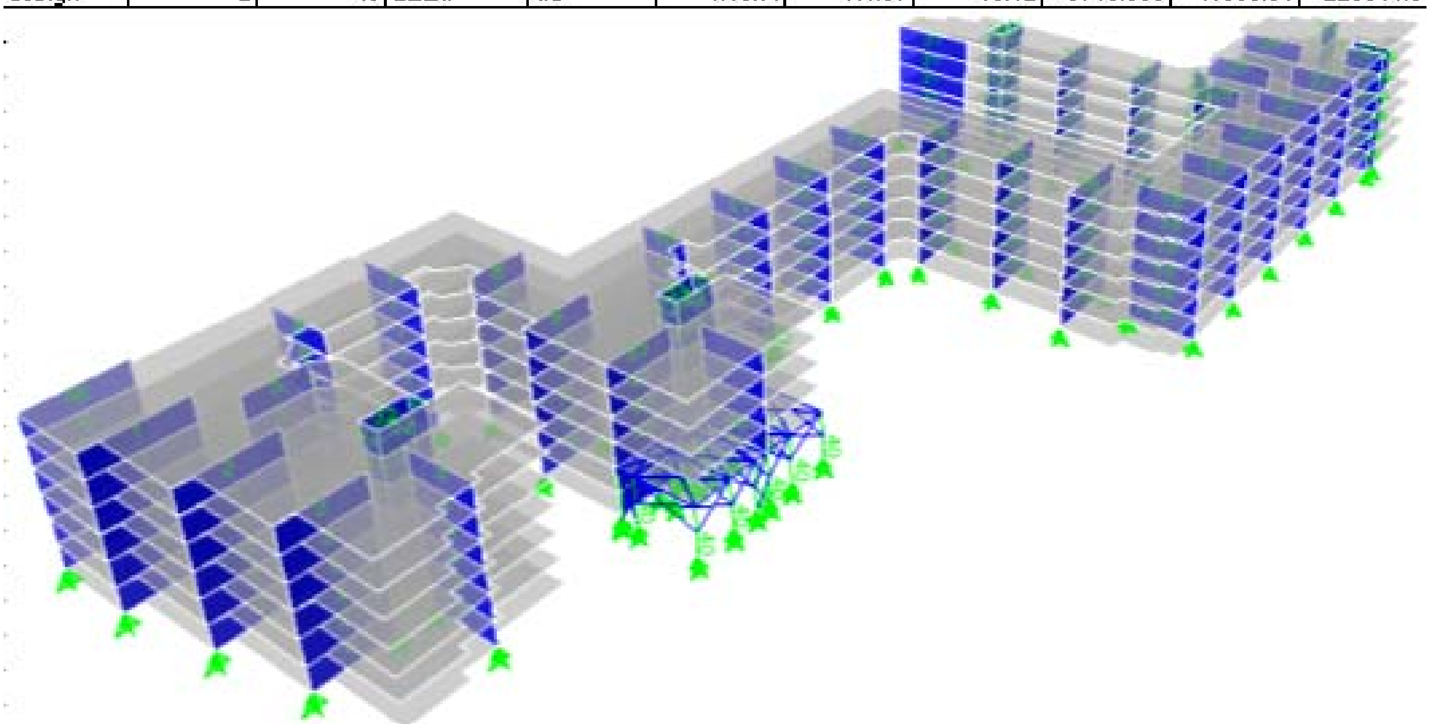
1c. Parkview at Bloomfield
Seismic Location



2a. Max Shear Wall Load Table



	Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
max	2	16	LEDx	t/b	519.81	414.97	40.42	3743.588	14559.94	225644.6
min	3	16	SEISMICY	t/b	-1140.71	-89.82	-27.76	-951.75	-9545.819	-51274.33
design	2	16	LEDx	t/b	1140.71	414.97	40.42	3743.588	14559.94	225644.6



2b. Shear Wall Wind Load in X & Y Direction

(K, ft units for these tables)

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
STORY5	15	WINDX	Top	0	4.49	0	0.007	0.001	138.72
STORY5	15	WINDX	Bottom	0	4.49	0	0.007	-0.027	677.706
STORY4	15	WINDX	Top	0	7.71	0	0.014	-0.027	677.706
STORY4	15	WINDX	Bottom	0	7.71	0	0.014	-0.065	1602.646
STORY3	15	WINDX	Top	0	10.83	0	0.004	-0.065	1602.646
STORY3	15	WINDX	Bottom	0	10.83	0	0.004	-0.116	2901.963
STORY2	15	WINDX	Top	0	13.77	0	0.017	-0.116	2901.963
STORY2	15	WINDX	Bottom	0	13.77	0	0.017	-0.257	4553.766
STORY1	15	WINDX	Top	0	16.73	0	-0.002	-0.257	4553.766
STORY1	15	WINDX	Bottom	0	16.73	0	-0.002	0	6561.039
STORY6	16	WINDX	Top	0	5.04	0	0.012	0	0
STORY6	16	WINDX	Bottom	0	5.04	0	0.012	0.001	604.446
STORY5	16	WINDX	Top	0	9.88	0	0.008	0.001	604.446
STORY5	16	WINDX	Bottom	0	9.88	0	0.008	-0.034	1789.491
STORY4	16	WINDX	Top	0	14.84	0	0.02	-0.034	1789.491
STORY4	16	WINDX	Bottom	0	14.84	0	0.02	-0.082	3570.223
STORY3	16	WINDX	Top	0	19.13	0	0.003	-0.082	3570.223
STORY3	16	WINDX	Bottom	0	19.13	0	0.003	-0.147	5865.623
STORY2	16	WINDX	Top	0	22.41	0	0.027	-0.147	5865.623
STORY2	16	WINDX	Bottom	0	22.41	0	0.027	-0.328	8555.357
STORY1	16	WINDX	Top	0	23.21	0	-0.007	-0.328	8555.357
STORY1	16	WINDX	Bottom	0	23.21	0	-0.007	0	11340.97

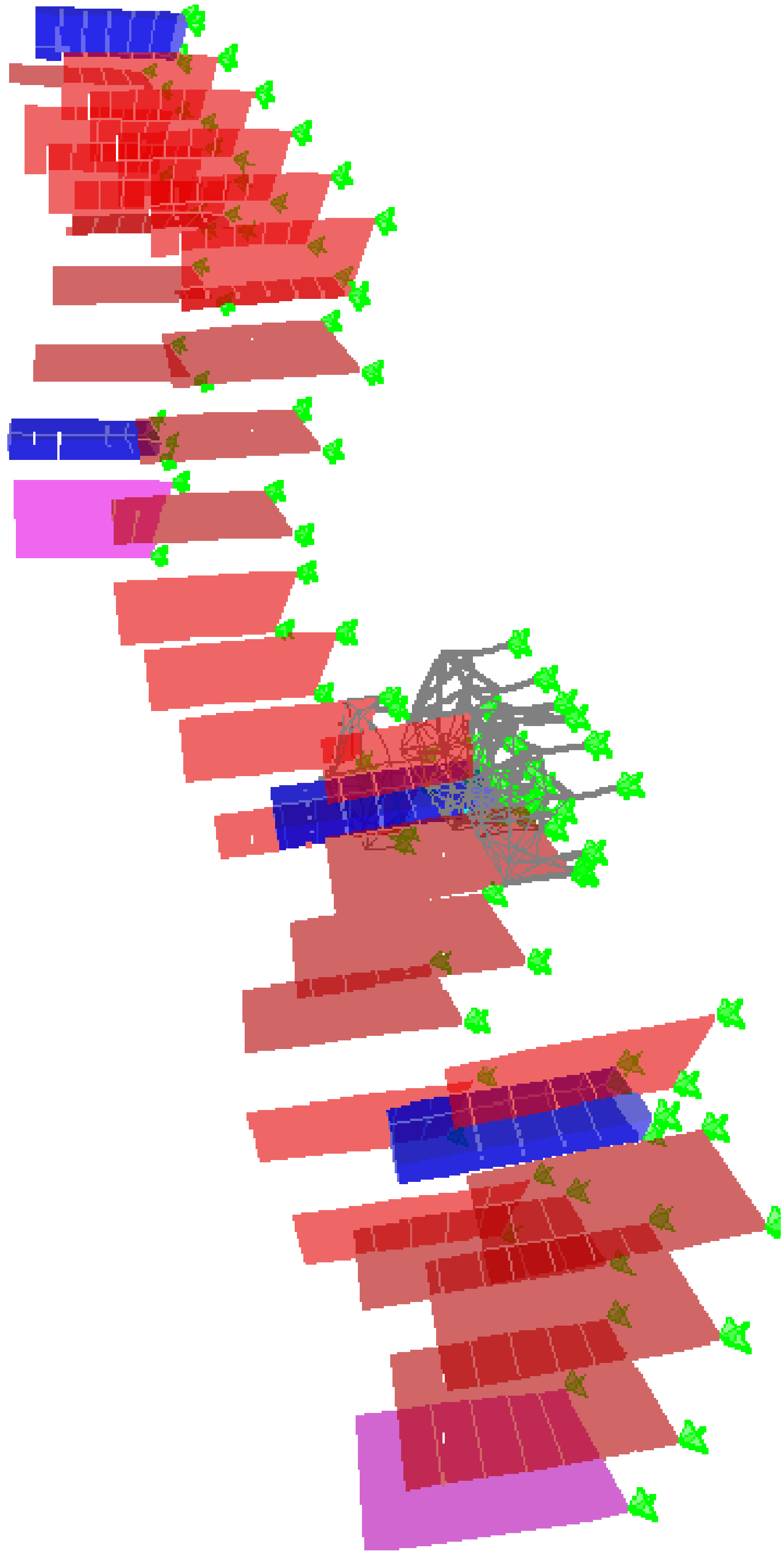
Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
STORY2	3	WINDY	Top	0	-1.52	0.07	-0.703	6.362	-484.215
STORY2	3	WINDY	Bottom	0	-1.52	0.07	-0.703	14.543	-666.61
STORY1	3	WINDY	Top	0	-1.06	-0.12	-0.06	14.543	-666.61
STORY1	3	WINDY	Bottom	0	-1.06	-0.12	-0.06	0	-793.363
STORY6	4	WINDY	Top	0	6.7	0	-0.642	0	0
STORY6	4	WINDY	Bottom	0	6.7	0	-0.642	-0.021	803.641
STORY5	4	WINDY	Top	0	17.82	0	-0.668	-0.021	803.641
STORY5	4	WINDY	Bottom	0	17.82	0	-0.668	0.069	2941.558
STORY4	4	WINDY	Top	0	29.47	0	-0.702	0.069	2941.558
STORY4	4	WINDY	Bottom	0	29.47	0	-0.702	0.203	6478.244
STORY3	4	WINDY	Top	0	41.97	0	-0.57	0.203	6478.244
STORY3	4	WINDY	Bottom	0	41.97	0	-0.57	0.349	11514.86
STORY2	4	WINDY	Top	0	51.57	0	-0.66	0.349	11514.86
STORY2	4	WINDY	Bottom	0	51.57	0	-0.66	0.853	17703.36
STORY1	4	WINDY	Top	0	60.37	-0.01	-0.074	0.853	17703.36
STORY1	4	WINDY	Bottom	0	60.37	-0.01	-0.074	0	24947.26
STORY6	5	WINDY	Top	0	0.01	-0.01	-0.534	0	0
STORY6	5	WINDY	Bottom	0	0.01	-0.01	-0.534	-1.275	1.345

2b. Shear Wall Wind Load in XY Direction

(K, ft units for these tables)

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
STORY3	9	WINDXY	Top	0	8.08	0.01	-0.17	0.564	1175.343
STORY3	9	WINDXY	Bottom	0	8.08	0.01	-0.17	1.482	2144.466
STORY2	9	WINDXY	Top	0	10.29	0.02	-0.189	1.482	2144.466
STORY2	9	WINDXY	Bottom	0	10.29	0.02	-0.189	3.381	3379.206
STORY1	9	WINDXY	Top	0	12.7	-0.03	-0.023	3.381	3379.206
STORY1	9	WINDXY	Bottom	0	12.7	-0.03	-0.023	0	4903.257
STORY6	1	WINDXY	Top	0	4.02	0	-0.235	0	0
STORY6	1	WINDXY	Bottom	0	4.02	0	-0.235	-0.593	482.121
STORY5	1	WINDXY	Top	0	7.87	0.01	-0.253	-0.593	482.121
STORY5	1	WINDXY	Bottom	0	7.87	0.01	-0.253	0.148	1426.489
STORY4	1	WINDXY	Top	0	11.84	0.01	-0.263	0.148	1426.489
STORY4	1	WINDXY	Bottom	0	11.84	0.01	-0.263	0.849	2847.676
STORY3	1	WINDXY	Top	0	15.39	0.01	-0.213	0.849	2847.676
STORY3	1	WINDXY	Bottom	0	15.39	0.01	-0.213	2.256	4694.099
STORY2	1	WINDXY	Top	0	17.98	0.02	-0.269	2.256	4694.099
STORY2	1	WINDXY	Bottom	0	17.98	0.02	-0.269	5.035	6851.638
STORY1	1	WINDXY	Top	0	18.86	-0.04	0.009	5.035	6851.638
STORY1	1	WINDXY	Bottom	0	18.86	-0.04	0.009	0	9114.299
STORY6	2	WINDXY	Top	0	2.97	0	-0.226	0	0
STORY6	2	WINDXY	Bottom	0	2.97	0	-0.226	-0.567	356.085
STORY5	2	WINDXY	Top	0	6.43	0.01	-0.242	-0.567	356.085
STORY5	2	WINDXY	Bottom	0	6.43	0.01	-0.242	0.14	1127.252

2b. Deflected Shape due to Wind in XY Direction



2c. Shear Wall Seismic Load

(K, ft units for these tables)

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
STORY6	15	SEISMICX	Top	0	43.17	0	0.995	0	0
STORY6	15	SEISMICX	Bottom	0	43.17	0	0.995	-0.116	5180.594
STORY5	15	SEISMICX	Top	0	113.58	-0.01	0.987	-0.116	5180.594
STORY5	15	SEISMICX	Bottom	0	113.58	-0.01	0.987	-0.834	18810.78
STORY4	15	SEISMICX	Top	0	169.59	-0.01	1.222	-0.834	18810.78
STORY4	15	SEISMICX	Bottom	0	169.59	-0.01	1.222	-1.588	39161.53
STORY3	15	SEISMICX	Top	0	213.14	-0.01	0.752	-1.588	39161.53
STORY3	15	SEISMICX	Bottom	0	213.14	-0.01	0.752	-2.815	64738.7
STORY2	15	SEISMICX	Top	0	242.62	-0.02	1.157	-2.815	64738.7
STORY2	15	SEISMICX	Bottom	0	242.62	-0.02	1.157	-5.4	93852.98
STORY1	15	SEISMICX	Top	0	259.94	0.04	-0.001	-5.4	93852.98
STORY1	15	SEISMICX	Bottom	0	259.94	0.04	-0.001	0	125045.3
STORY6	16	SEISMICX	Top	0	138.91	0	1.255	0	0
STORY6	16	SEISMICX	Bottom	0	138.91	0	1.255	-0.131	16668.75
STORY5	16	SEISMICX	Top	0	239.39	-0.01	1.24	-0.131	16668.75
STORY5	16	SEISMICX	Bottom	0	239.39	-0.01	1.24	-1.064	45396.05
STORY4	16	SEISMICX	Top	0	324.73	-0.01	1.667	-1.064	45396.05
STORY4	16	SEISMICX	Bottom	0	324.73	-0.01	1.667	-2.053	84363.99
STORY3	16	SEISMICX	Top	0	381.08	-0.01	0.865	-2.053	84363.99
STORY3	16	SEISMICX	Bottom	0	381.08	-0.01	0.865	-3.646	130093.5
STORY2	16	SEISMICX	Top	0	404.83	-0.03	1.739	-3.646	130093.5
STORY2	16	SEISMICX	Bottom	0	404.83	-0.03	1.739	-7.019	178673.7
STORY1	16	SEISMICX	Top	0	369.39	0.06	-0.264	-7.019	178673.7
STORY1	16	SEISMICX	Bottom	0	369.39	0.06	-0.264	0	223000.2
STORY6	17	SEISMICX	Top	-3.87	4.16	0.1	-6.869	-5.691	-305.601

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
STORY1	3	SEISMICY	Top	0	-5.98	-1.19	-0.174	142.858	-5215.35
STORY1	3	SEISMICY	Bottom	0	-5.98	-1.19	-0.174	0	-5932.67
STORY6	4	SEISMICY	Top	0	89.81	0	-6.94	0	0
STORY6	4	SEISMICY	Bottom	0	89.81	0	-6.94	-0.119	10776.98
STORY5	4	SEISMICY	Top	0	191.16	0.01	-7.382	-0.119	10776.98
STORY5	4	SEISMICY	Bottom	0	191.16	0.01	-7.382	0.894	33715.62
STORY4	4	SEISMICY	Top	0	281.78	0.01	-7.41	0.894	33715.62
STORY4	4	SEISMICY	Bottom	0	281.78	0.01	-7.41	2.169	67529.55
STORY3	4	SEISMICY	Top	0	365.93	0.01	-6.089	2.169	67529.55
STORY3	4	SEISMICY	Bottom	0	365.93	0.01	-6.089	3.557	111441.3
STORY2	4	SEISMICY	Top	0	398.21	0.03	-6.526	3.557	111441.3
STORY2	4	SEISMICY	Bottom	0	398.21	0.03	-6.526	7.621	159225.9
STORY1	4	SEISMICY	Top	0	411.21	-0.06	-0.338	7.621	159225.9
STORY1	4	SEISMICY	Bottom	0	411.21	-0.06	-0.338	0	208571.2
STORY6	5	SEISMICY	Top	0	1.28	-0.09	-5.782	0	0

2d. Max Shear Wall Loads

(K, ft units for these tables)

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
STORY3	15	LEDX	Bottom	-523.74	214.25	-0.01	0.769	-2.897	64945.8
STORY2	15	LEDX	Top	-523.74	248.55	-0.03	1.442	-2.897	64945.8
STORY2	15	LEDX	Bottom	-654.68	248.55	-0.03	1.442	-6.707	94772.38
STORY1	15	LEDX	Top	-654.68	262.91	0.06	0.011	-6.707	94772.38
STORY1	15	LEDX	Bottom	-785.62	262.91	0.06	0.011	0	126321.1
STORY6	16	LEDX	Top	0	139.56	0	1.418	0	0
STORY6	16	LEDX	Bottom	-166.51	139.56	0	1.418	-0.061	16747.41
STORY5	16	LEDX	Top	-166.51	240.35	-0.01	1.312	-0.061	16747.41
STORY5	16	LEDX	Bottom	-333.03	240.35	-0.01	1.312	-1.129	45589.4
STORY4	16	LEDX	Top	-333.03	326.16	-0.01	1.913	-1.129	45589.4
STORY4	16	LEDX	Bottom	-499.54	326.16	-0.01	1.913	-2.088	84728.61
STORY3	16	LEDX	Top	-499.54	384.24	-0.01	0.826	-2.088	84728.61
STORY3	16	LEDX	Bottom	-666.05	384.24	-0.01	0.826	-3.741	130837
STORY2	16	LEDX	Top	-666.05	414.97	-0.04	2.195	-3.741	130837
STORY2	16	LEDX	Bottom	-832.56	414.97	-0.04	2.195	-8.703	180633.5
STORY1	16	LEDX	Top	-832.56	375.09	0.07	-0.301	-8.703	180633.5
STORY1	16	LEDX	Bottom	-999.08	375.09	0.07	-0.301	0	225644.6
STORY6	17	LEDX	Top	-3.73	3.94	0.1	-6.588	-5.522	-294.387
STORY6	17	LEDX	Bottom	-62.05	3.94	0.1	-6.588	6.145	178.494
STORY5	17	LEDX	Top	79.35	27.35	0.11	10.083	6.894	1161.16

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
STORY3	15	SEISMICY	Top	0	-30.15	0.16	-6.148	18.259	-6360.7
STORY3	15	SEISMICY	Bottom	0	-30.15	0.16	-6.148	37.151	-9978.88
STORY2	15	SEISMICY	Top	0	-31.09	0.33	-6.621	37.151	-9978.88
STORY2	15	SEISMICY	Bottom	0	-31.09	0.33	-6.621	76.411	-13709.3
STORY1	15	SEISMICY	Top	0	-26.75	-0.64	-0.303	76.411	-13709.3
STORY1	15	SEISMICY	Bottom	0	-26.75	-0.64	-0.303	0	-16919
STORY6	16	SEISMICY	Top	0	-43.25	-0.09	-8.803	0	0
STORY6	16	SEISMICY	Bottom	0	-43.25	-0.09	-8.803	-10.946	-5190.22
STORY5	16	SEISMICY	Top	0	-60.12	0.15	-9.673	-10.946	-5190.22
STORY5	16	SEISMICY	Bottom	0	-60.12	0.15	-9.673	7.024	-12404.6
STORY4	16	SEISMICY	Top	0	-76.99	0.14	-9.807	7.024	-12404.6
STORY4	16	SEISMICY	Bottom	0	-76.99	0.14	-9.807	23.44	-21643.9
STORY3	16	SEISMICY	Top	0	-89.82	0.2	-7.808	23.44	-21643.9
STORY3	16	SEISMICY	Bottom	0	-89.82	0.2	-7.808	47.846	-32421.8
STORY2	16	SEISMICY	Top	0	-88.04	0.42	-9.61	47.846	-32421.8
STORY2	16	SEISMICY	Bottom	0	-88.04	0.42	-9.61	98.054	-42986.2
STORY1	16	SEISMICY	Top	0	-69.07	-0.82	1.151	98.054	-42986.2
STORY1	16	SEISMICY	Bottom	0	-69.07	-0.82	1.151	0	-51274.3
STORY6	17	SEISMICY	Top	-25.5	36.02	0.5	-45.628	-31.413	-1988.21
STORY6	17	SEISMICY	Bottom	-25.5	36.02	0.5	-45.628	28.72	2333.683
STORY5	17	SEISMICY	Top	-82.72	38.77	0.79	-55.365	-44.288	-2135.78

2e. Story Shear and Drift

(K, ft units for these tables)

Story	Load	Loc	P	VX	VY	T	MX	MY		max story shear
STORY1	SEISMICX	Top	0	-4153.16	0	4022137	0	-1572931		
STORY1	SEISMICX	Bottom	0	-4153.16	0	4022137	0	-2071310		
STORY1	LEDX	Top	22377.71	-4153.16	0	4022137	21023525	-1.9E+07		
STORY1	LEDX	Bottom	26770.38	-4153.16	0	4022137	25182339	-2.3E+07		

Story	Load	Loc	P	VX	VY	T	MX	MY		max story shear
STORY1	SEISMICY	Top	0	0	-4153.16	-3087416	1572931	0		
STORY1	SEISMICY	Bottom	0	0	-4153.16	-3087416	2071310	0		
STORY1	LEDY	Top	22377.71	0	-4153.16	-3087416	22596455	-1.7E+07		
STORY1	LEDY	Bottom	26770.38	0	-4153.16	-3087416	27253648	-2.1E+07		

A	B	C	D	E	F	G	H	I	J	K	L
Story	Item	Load	Point	X	Y	Z	DriftX	DriftY	x=	0.013178	max drift
STORY2	Max Drift X	LDWXY	1404	-738.36	521.208	240	0.000177				
STORY2	Max Drift Y	LDWXY	1410	-657.36	845.208	240		0.000097			
STORY1	Max Drift X	DEAD	1404	-738.36	521.208	120	0.000148				
STORY1	Max Drift Y	DEAD	1410	-657.36	845.208	120		0.000082			
STORY1	Max Drift X	WINDX	1407	-300.36	521.208	120	0.000002				
STORY1	Max Drift Y	WINDX	1411	-738.36	845.208	120		0			
STORY1	Max Drift X	WINDY	1406	-513.36	521.208	120	0.000004				
STORY1	Max Drift Y	WINDY	1402	-1095.36	521.208	120		0.000008			
STORY1	Max Drift X	SEISMICY	1406	-513.36	521.208	120	0.000033				
STORY1	Max Drift Y	SEISMICY	1402	-1095.36	521.208	120		0.000062	x=	0.013178	max drift
STORY1	Max Drift X	SEISMICX	69	2548.64	1680.21	120	0.013178		y=	0.000138	max drift
STORY1	Max Drift Y	SEISMICX	69	2548.64	1680.21	120		0.000028			
STORY1	Max Drift X	WINDXY	1409	-513.36	845.208	120	0.000002				
STORY1	Max Drift Y	WINDXY	1402	-1095.36	521.208	120		0.000002			
STORY1	Max Drift X	LDW	1404	-738.36	521.208	120	0.000176				
STORY1	Max Drift Y	LDW	1410	-657.36	845.208	120		0.000098			
STORY1	Max Drift X	LEDX	69	2548.64	1680.21	120	0.013178				
STORY1	Max Drift Y	LEDX	1405	-657.36	521.208	120		0.000097			
STORY1	Max Drift X	LEDY	1411	-738.36	845.208	120	0.000194				
STORY1	Max Drift Y	LEDY	1418	-513.36	1025.208	120		0.000138			
STORY1	Max Drift X	LDWY	1411	-738.36	845.208	120	0.000174				
STORY1	Max Drift Y	LDWY	1410	-657.36	845.208	120		0.000103			
STORY1	Max Drift X	LDWXY	1404	-738.36	521.208	120	0.000175				
STORY1	Max Drift Y	LDWXY	1410	-657.36	845.208	120		0.0001			

2f. Brace and Column Loads
Center of Mass

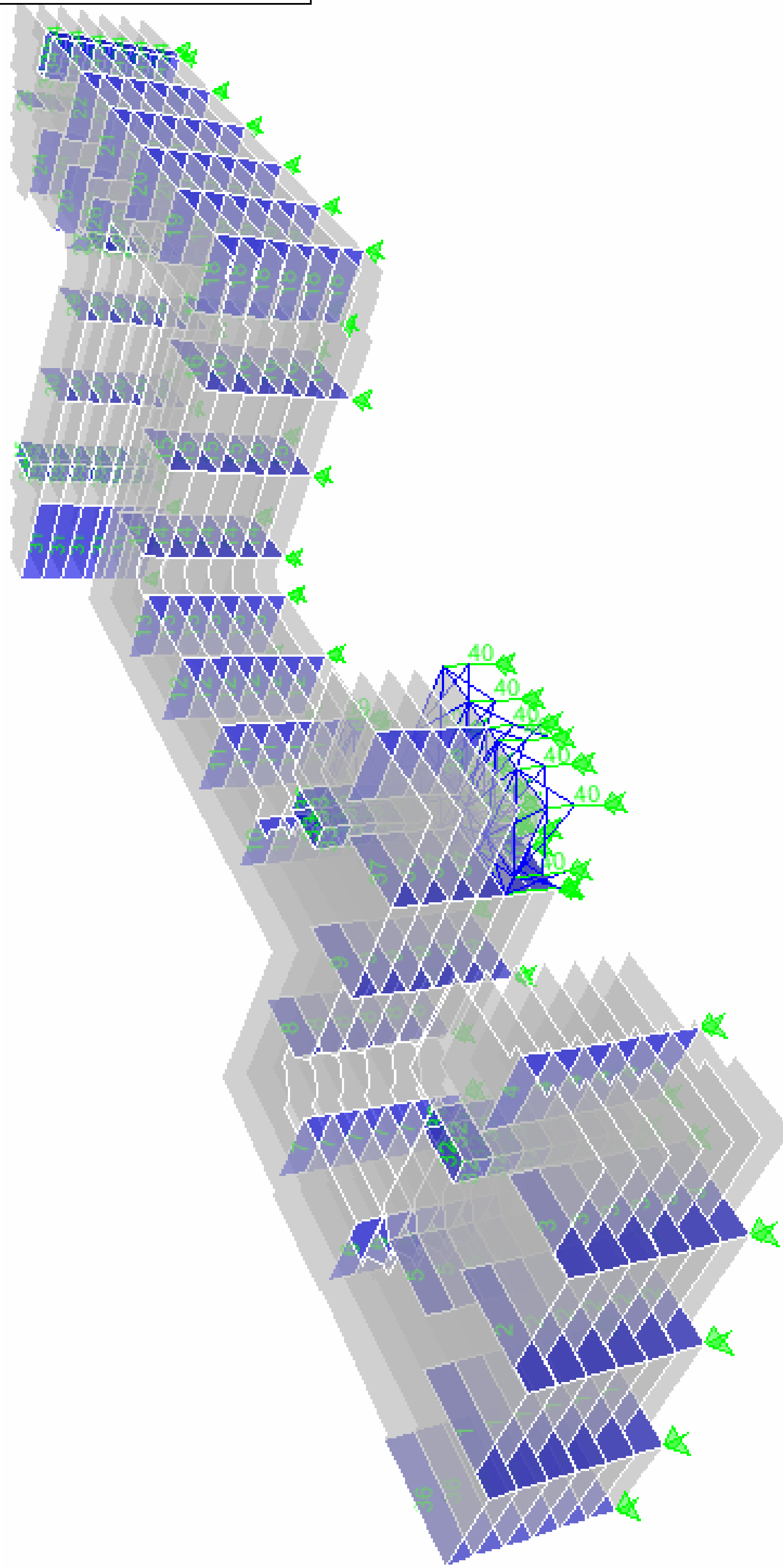
(K, ft units for these tables)

Story	Brace	Load	Loc	P	V2	V3	T	M2	M3
STORY2	D58	LEDY	72.39	-99.58	-10.22	0.05	-0.008	1.77	-312.609

Story	Column	Load	Loc	P	V2	V3	T	M2	M3
STORY2	C10	LEDY	0	-152.84	-1.35	0.04	-0.028	5.756	-83.308

Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM	XCR	YCR
STORY6	D1	9.2674	9.2674	734.213	974.295	9.2674	9.2674	734.213	974.295	1496.289	1081.024
STORY5	D1	13.9838	13.9838	747.769	958.868	23.2512	23.2512	742.366	965.017	1492.335	1081.439
STORY4	D1	13.9838	13.9838	747.769	958.868	37.235	37.235	744.395	962.708	1479.239	1081.899
STORY3	D1	13.9838	13.9838	747.769	958.868	51.2188	51.2188	745.316	961.659	1454.056	1081.91
STORY2	D1	14.0822	14.0822	737.444	958.347	65.301	65.301	743.619	960.945	1424.772	1081.227
STORY1	D1	13.5258	13.5258	799.72	971.484	78.8268	78.8268	753.245	962.753	1376.748	1075.538

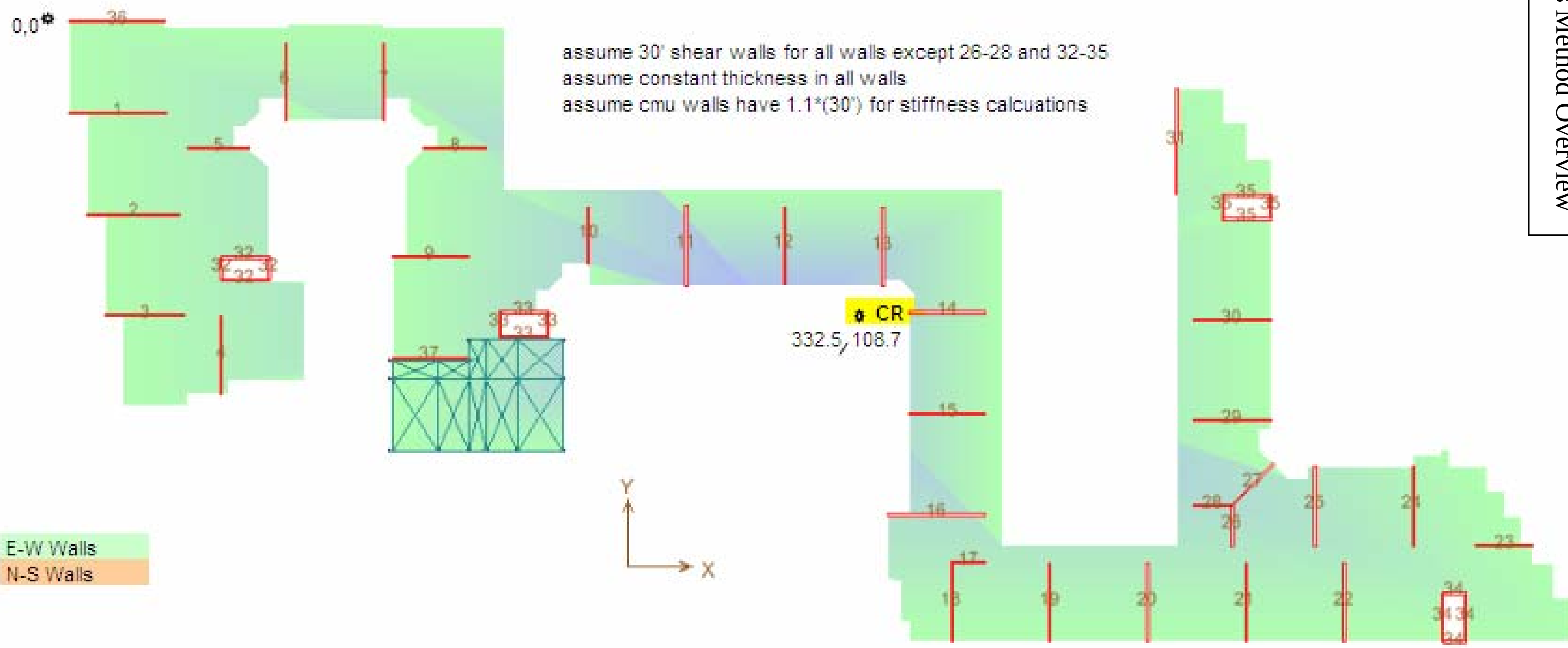
3a. Shear Wall Analysis Method Diagram



3b. Shear Wall Analysis Method Overview

Robert Whitaker

story # =	1	Story h	120 in	bldg Lx	583 ft ==>	ecc. x=	-41.0	Tx=	-1444.4 ft-K
wind psf	15 psf	Wall L	360 in	bldg Ly	235 ft ==>	ecc. y=	8.8	Ty=	770.3 ft-K
Px=	35.25 Kip	Wall t	4 in					T total=	-674.1 ft-K
Py=	87.45 Kip	Aw =	17280 in^2	CR x=	332.5 ft				
		I =	15552000 in^4	CR y=	108.7 ft				



3c. Shear Wall Analysis Method

	Dist from 0,0				stiffness														
Wall #	y	x	(h/L)	Wall type	k=	k*y	k*x	y from C	x from C	K*y^2	K*x^2	ky/Σky^2	kx/Σkx^2	Direct V	T shear	Direct V	T shear	xTotal (K)	wall
1	34.5		0.33	Intermediate	3.722	128.4	0.0	74.19	0.00	20485	0	0.00127	0	0.00	0.00	4517.97	977.47	5.50	1
2	72		0.33	Intermediate	3.722	268.0	0.0	36.69	0.00	5010.4	0	0.00063	0	0.00	0.00	4517.97	483.42	5.00	2
3	109		0.33	Intermediate	3.722	405.7	0.0	-0.31	0.00	0.3528	0	-5E-06	0	0.00	0.00	4517.97	-4.06	4.51	3
4		58.5	0.33	Intermediate	3.722	0.0	217.7	0.00	273.97	0	279350	0	0.00066	1947.28	-955.23	0.00	0.00	0.99	4
5	48		0.33	Intermediate	3.722	178.6	0.0	60.69	0.00	13709	0	0.00104	0	0.00	0.00	4517.97	799.61	5.32	5
6		84	0.33	Intermediate	3.722	0.0	312.6	0.00	248.47	0	229769	0	0.0006	1947.28	-866.33	0.00	0.00	1.08	6
7		122	0.33	Intermediate	3.722	0.0	454.0	0.00	210.47	0	164864	0	0.00051	1947.28	-733.84	0.00	0.00	1.21	7
8	48		0.33	Intermediate	3.722	178.6	0.0	60.69	0.00	13709	0	0.00104	0	0.00	0.00	4517.97	799.61	5.32	8
9	88		0.33	Intermediate	3.722	327.5	0.0	20.69	0.00	1593.4	0	0.00035	0	0.00	0.00	4517.97	272.62	4.79	9
10		201.5	0.33	Intermediate	3.722	0.0	749.9	0.00	130.97	0	63841.4	0	0.00032	1947.28	-456.65	0.00	0.00	1.49	10
11		239	0.33	Intermediate	3.722	0.0	889.5	0.00	93.47	0	32517.4	0	0.00023	1947.28	-325.91	0.00	0.00	1.62	11
12		278	0.33	Intermediate	3.722	0.0	1034.6	0.00	54.47	0	11043.8	0	0.00013	1947.28	-189.93	0.00	0.00	1.76	12
13		315.5	0.33	Intermediate	3.722	0.0	1174.2	0.00	16.97	0	1072.36	0	4.1E-05	1947.28	-59.18	0.00	0.00	1.89	13
14	108.5		0.33	Intermediate	3.722	403.8	0.0	0.19	0.00	0.1373	0	3.3E-06	0	0.00	0.00	4517.97	2.53	4.52	14
15	146		0.33	Intermediate	3.722	543.3	0.0	-37.31	0.00	5180	0	-0.0006	0	0.00	0.00	4517.97	-491.53	4.03	15
16	184.5		0.33	Intermediate	3.722	686.6	0.0	-75.81	0.00	21387	0	-0.0013	0	0.00	0.00	4517.97	-998.76	3.52	16
17	202.5		0.33	Intermediate	3.722	753.6	0.0	-93.81	0.00	32750	0	-0.0016	0	0.00	0.00	4517.97	-1235.91	3.28	17
18		342	0.33	Intermediate	3.722	0.0	1272.8	0.00	-9.53	0	337.648	0	-2.3E-05	1947.28	33.21	0.00	0.00	1.98	18
19		379.5	0.33	Intermediate	3.722	0.0	1412.3	0.00	-47.03	0	8229.73	0	-0.00011	1947.28	163.96	0.00	0.00	2.11	19
20		417.5	0.33	Intermediate	3.722	0.0	1553.8	0.00	-85.03	0	26904.2	0	-0.00021	1947.28	296.45	0.00	0.00	2.24	20
21		455.5	0.33	Intermediate	3.722	0.0	1695.2	0.00	-123.03	0	56326.6	0	-0.0003	1947.28	428.94	0.00	0.00	2.38	21
22		494	0.33	Intermediate	3.722	0.0	1838.5	0.00	-161.53	0	97097.1	0	-0.00039	1947.28	563.17	0.00	0.00	2.51	22
23	196		0.33	Intermediate	3.722	729.4	0.0	-87.31	0.00	28368	0	-0.0015	0	0.00	0.00	4517.97	-1150.27	3.37	23
24		521	0.33	Intermediate	3.722	0.0	1938.9	0.00	-188.53	0	132271	0	-0.00046	1947.28	657.31	0.00	0.00	2.60	24
25		482	0.33	Intermediate	3.722	0.0	1793.8	0.00	-149.53	0	83206	0	-0.00036	1947.28	521.33	0.00	0.00	2.47	25
26		450.5	0.67	Intermediate	1.316	0.0	593.1	0.00	-118.03	0	18337.8	0	-0.0001	688.81	145.56	0.00	0.00	0.83	26
27y	173		0.47	Intermediate	2.313	400.1	0.0	-64.31	0.00	9564.5	0	-0.0007	0	0.00	0.00	2807.68	-526.52	2.28	27y
27x		458.5	0.47	Intermediate	2.313	0.0	1060.4	0.00	-126.03	0	36732.1	0	-0.00019	1210.14	273.06	0.00	0.00	1.48	27x
28	180		0.67	Intermediate	1.316	237.0	0.0	-71.31	0.00	6693.8	0	-0.0004	0	0.00	0.00	1598.14	-332.32	1.27	28
29	149.5		0.33	Intermediate	3.722	556.4	0.0	-40.81	0.00	6197.5	0	-0.0007	0	0.00	0.00	4517.97	-537.64	3.98	29
30	111.5		0.33	Intermediate	3.722	415.0	0.0	-2.81	0.00	29.342	0	-5E-05	0	0.00	0.00	4517.97	-36.99	4.48	30
31		430	0.33	Intermediate	3.722	0.0	1600.3	0.00	-97.53	0	35396.4	0	-0.00024	1947.28	340.03	0.00	0.00	2.29	31
32	92.5		0.30	Intermediate	4.194	388.0	0.0	16.19	0.00	1099.6	0	0.00031	0	0.00	0.00	5091.56	240.41	5.33	32
33	113.3		0.30	Intermediate	4.194	475.0	0.0	-4.56	0.00	87.129	0	-9E-05	0	0.00	0.00	5091.56	-67.67	5.02	33
34		525.5	0.30	Intermediate	4.194	0.0	2204.0	0.00	-193.03	0	156265	0	-0.00053	2194.51	758.44	0.00	0.00	2.95	34
35	68.5		0.30	Intermediate	4.194	287.3	0.0	40.19	0.00	6775.1	0	0.00077	0	0.00	0.00	5091.56	596.76	5.69	35
36	0.1		0.33	Intermediate	3.722	0.4	0.0	108.59	0.00	43886	0	0.00186	0	0.00	0.00	4517.97	1430.69	5.95	36
37	125.5		0.33	Intermediate	3.722	467.1	0.0	-16.81	0.00	1051.4	0	-0.0003	0	0.00	0.00	4517.97	-221.44	4.30	37
38		162	0.33	Intermediate	3.722	0.0	602.9	0.00	170.47	0	108155	0	0.00041	1947.28	-594.37	0.00	0.00	1.35	38
				sum Ky	72.03	7829.6	22398.3		sum Rx^2	217576	1541717						Σ forces x =	35.25	kip
				sum Kx	67.37	sum K*dist											Σ forces y =	87.45	kip

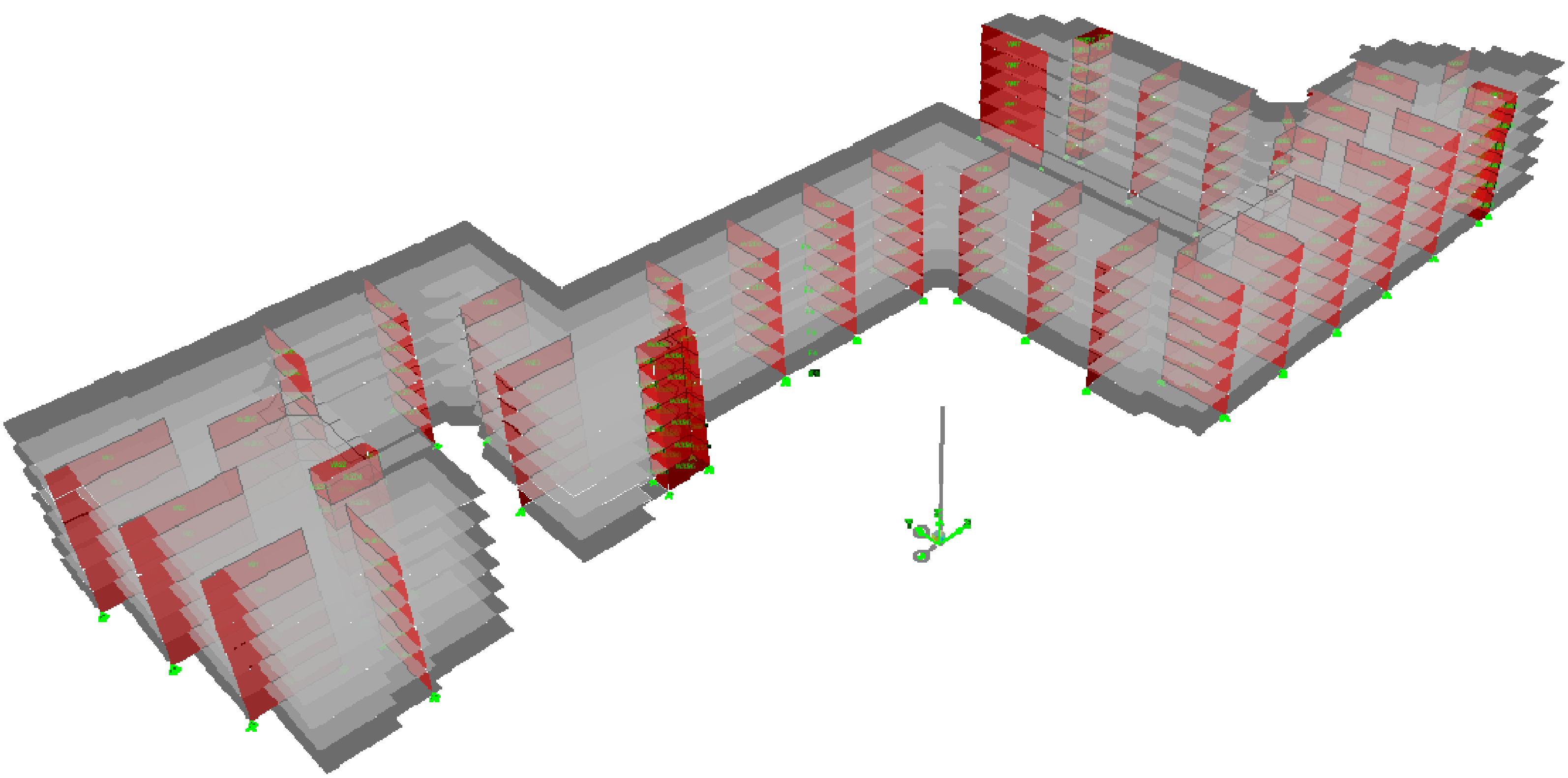
4a. Area Method ASCE-7 Calculation

WIND DIRECTION: NORTH-SOUTH (Y-DIR)   # Stories: 6

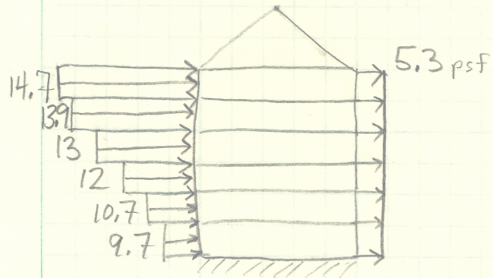
WIND SPEED:	110.00 MPH	L :	583.000 ft	FREQ n_1:	1.9300 Hz	Ground to	
EXP. CAT:	B	B :	235.000 ft	ALPHA=	7	Base h:	0.000 ft
IMPORT. FACTOR:	1.00	Mean Roof h :	76.40 ft	Zg (ft)=	1200 ft		
DIREC. FACT. Kd:	0.85	Kh =	0.915	G=	0.8	Wind Load	
TOPOG. FACT Kzt:	1.00 (see sht. Kzt)	L/B =	2.481	Gf=	0.800	to be applied	
$0.00256\ K_d\ K_{zt}\ V^2\ I$ =	26.33 psf	Cp (wind)	0.8			at Yo:	0.000 ft
		Cp (leew)	-0.28				

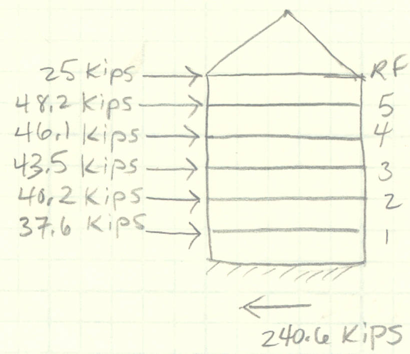
WIND FORCE CALCULATION PER ASCE7-02 - MAIN WIND FORCE RESISTING SYSTEM

FLOOR I.D.	FL TO FL HEIGHT (ft)	TRIB. WIDTH (ft)	Wind Press Wind	Wind Press Lee	WIND PRESS. total (psf)	WIND FORCE (Kips)	FLOOR SHEAR (Kips)	FLOOR MOMENT (Kip-ft)	Case 1 Mz (Kip-ft)	Case 2 Mz (Kip-ft)	
6	10.667	235.000	14.659	5.319	19.98	25.0	25.0	267.1	0.0	183.9	-183.9
5	10.667	235.000	13.915	5.319	19.23	48.2	73.3	1,048.5	0.0	354.0	-354.0
4	10.667	235.000	13.056	5.319	18.37	46.1	119.3	2,321.3	0.0	338.2	-338.2
3	10.667	235.000	12.025	5.319	17.34	43.5	162.8	4,057.8	0.0	319.3	-319.3
2	10.667	235.000	10.710	5.319	16.03	40.2	203.0	6,222.9	0.0	295.0	-295.0
1	10.667	235.000	9.685	5.319	15.00	37.6	240.6	8,789.2	0.0	276.2	-276.2



4b. Overturning Hand Calculation





*All loads are in "psf"

$M_{OT} = \sum V \cdot (\text{trib height})$ tributary height = 10.67 feet

$M_{OT} = 10.67\,ft(25 + 48.2 + 46.1 + 43.5 + 40.2 + 37.6)\,kips = \boxed{2567.2\ kip \cdot ft}$

self weight = $A_{bldg} \cdot 57\,psf = (300725\,ft^2)(57\,psf)$
$= \boxed{17,141.3\ kips}$

Building depth = 30' + 6' corridor = 36'

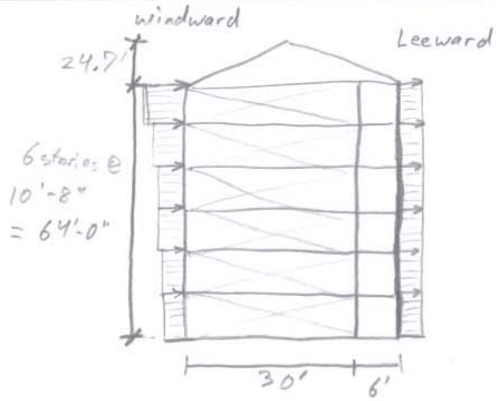
Resisting over-turning moment = ½(36')(17,141.3 kips)
$= \boxed{308543.4\ kip \cdot ft}$

Since Resisting over-turning moment minus over-turning moment = 308543.4 kip·ft − 2567.2 kip·ft = 305976.2 kip·ft, the building will not over-turn.

5a. Hand Calculation ~ Wind Coefficients

Client: ______ Job#: ______
Project: ______
Description: Wind analysis
Design By: ______ Date: ______
Page No.: 1 of 2

Wind analysis (ASCE 7-98)



N-S + E-W direction: Shear walls
Location: Bloomfield, NJ
Exposure: B
Building use: Residential

$P = q\ G\ C_p$

$q = .00256\ K_{zt}\ K_z\ K_d\ V^2\ I$

$K_{zt} = 1.0$, area assumed flat (Fig. 6.2)

$K_d = 0.85$ Table 6-6

$V = 110$ mph Fig 6-1

use group = II

$I = 1.0$ Table 6-1

mean roof $h = 64' + \frac{24.7'}{2} = 76.4'$

z (ft)	K_z (table 6-5) - exposure B case 2
0-15	.57
20	.62
25	.66
30	.70
40	.76
50	.81
60	.85
70	.89
80	.93
90	.96

spread sheet

$q = 26.33 (K_z)$

$G_F = .85$

spread sheet

Leeward + Windward

Floor	wind press	wind force (k)	Floor Shear (k)	Floor moment
6	24.30	4.9	4.9	52.5
5	23.59	9.5	14.5	206.9
4	22.69	9.2	23.7	459.4
3	21.66	8.8	32.5	805.5
2	20.35	8.2	40.7	1239.7
1	19.32	7.8	48.5	1757.3

5b. Excel Spreadsheet

WIND DIRECTION:	NORTH-SOUTH (Y-DIR)	# Stories:	6				
WIND SPEED:	110.00 MPH	L :	36.000 ft	FREQ n_1:	1.9300 Hz	Ground to	
EXP. CAT:	B	B :	38.000 ft	ALPHA=	7	Base h:	0.000 ft
IMPORT. FACTOR:	1.00	Mean Roof h :	76.40 ft	Zg (ft)=	1200 ft		
DIREC. FACT. Kd:	0.85	Kh =	0.915	G=	0.8	Wind Load	
TOPOG. FACT Kzt:	1.00 (see sht. Kzt)	L/B =	0.947	Gf=	0.800	to be applied	
$0.00256\ K_d\ K_{zt}\ V^2\ I$ =	26.33 psf	Cp (wind)	0.8			at Yo:	0.000 ft
		Cp (leew)	-0.50				

WIND FORCE CALCULATION PER ASCE7-02 - MAIN WIND FORCE RESISTING SYSTEM

FLOOR I.D.	FL TO FL HEIGHT (ft)	TRIB. WIDTH (ft)	Kz	Wind Press Wind	Wind Press Lee	WIND PRESS. total (psf)	WIND FORCE (Kips)	FLOOR SHEAR (Kips)	FLOOR MOMENT (Kip-ft)	Case 1 Mz (Kip-ft)	Case 2 Mz (Kip-ft)	
6	10.667	38.000	0.870	14.659	9.637	24.30	4.9	4.9	52.5	0.0	5.8	-5.8
5	10.667	38.000	0.826	13.915	9.637	23.55	9.5	14.5	206.9	0.0	11.3	-11.3
4	10.667	38.000	0.775	13.056	9.637	22.69	9.2	23.7	459.4	0.0	10.9	-10.9
3	10.667	38.000	0.714	12.025	9.637	21.66	8.8	32.5	805.5	0.0	10.4	-10.4
2	10.667	38.000	0.636	10.710	9.637	20.35	8.2	40.7	1,239.7	0.0	9.8	-9.8
1	10.667	38.000	0.575	9.685	9.637	19.32	7.8	48.5	1,757.3	0.0	9.3	-9.3
	0.000	38.000				0.00	0.0					

6a. Hand Calculation ~ Seismic Coefficients

Client: Job#:
Project:
Description: Seismic Design
Design By: Date:
Page No.: 1 of 2

Seismic use group I (Table 9.1.3)

geo tech report Site Class C

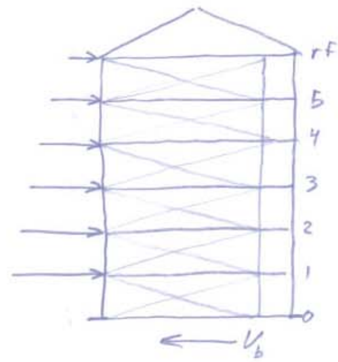
$S_S = 0.420$ $S_{MS} = 50.4\%g$

$S_1 = 0.094$ $S_{M1} = 16.0\%g$

$S_{DS} = \frac{2}{3}S_{MS} = \frac{2}{3} \cdot .504g = .336g$

$S_{D1} = \frac{2}{3}S_{M1} = \frac{2}{3} \cdot .16g = .107g$

$I_E = 1.0$ (Table 9.1.4)



$V_b = C_S W = .074\,(14702^k) = \boxed{1087.95^k}$

9.5.2.2 $C_S = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{.336g}{\left(\frac{2.5}{1}\right)} = .1344g$

9.5.2.2 $R = 2.5$ Building frame system
-light framed walls w/ shear panels of all other materials

9.5.3.3 $T = C_T h_n^{3/4} = .02\,(88.7')^{3/4} = .578$

$C_T = .02$

$h_n = 64' + 24.7' = 88.7'$

$C_S = .1344g > .044\, S_{DS} \cdot I_E = .044(.336g)(1.0) = .01478g$ ∴ ok

$\frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{.107g}{\left(\frac{2.5}{1}\right)(.578)} = .074g$

$.074g \ngeq .1344g \geq .01478g$ ∴ $\underline{C_S = .074g}$

$W_{psf} = DL + \text{Partition load} = 45psf + 11psf = 56psf$

$DL = 45psf$ $DL_{roof} = 17psf$

$\text{Partition} = \frac{105plf \cdot 4}{38'} = 11psf \geq 10psf$

$W = A_{TOT} \cdot \#\text{floors} \cdot W_{psf} = 49500\frac{ft^2}{flr} \cdot 5_{flr} \cdot 56psf + 49500\,(1\,roof)(17psf) = 14702^k$

$A_{TOT} = 49{,}500\ ft^2/flr$

$\#\text{floors} = 5$ (5 above grade)

$F_x = C_{vx} \cdot V_b$ (9.5.3.4)

$C_{v(roof)} = \frac{w_x h_x^k}{\sum^n w_i h_i^k} = \frac{49500(64)^{1.039}(17)\cdot\frac{k}{1000\#}}{573664.5'k} = \frac{63339.6}{573664.5} = .110$

$T = .578$ $k = 1 + 1\left(\frac{.578 - .5}{2.5 - .5}\right) = 1.039$

$\sum w_i h_i^k = 49500\,(56psf)(10.67 + 21.34 + 32.01 + 42.68 + 53.35)^{1.039} + 49500\,(64)^{1.039}(17psf) = 573664.5'k$

6b. Hand Calculation ~ Seismic (cont.) & Story Drift

Client: Job#:

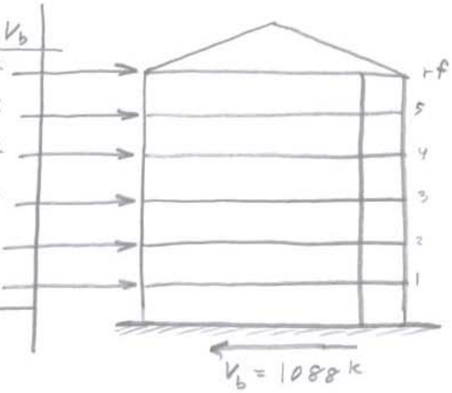
Project:

Description: Seismic

Design By: Date:

$V_b = 1087.95^k$

Location	$w_x h_x^{1.039}$	C_{vx}	$F_x = C_{vx} \cdot V_b$
roof	$63339.6^{1.039} = 63339.6$	.110	119.7^k
5	$\frac{49500(56)(53.35)}{1000}^{1.039} = 172,697.4$	.301	327.5^k
4	$42.68^{1.039} = 136,960.8$	.239	260.0^k
3	$32.01^{1.039} = 101,574.6$	.177	192.6^k
2	$21.34^{1.039} = 66,654.0$	.116	126.2^k
1	$10.67^{1.039} = 32,438.1$	.057	62.0^k
Σ	573664.5	1.000	1088^k



story deflection

$$k = \frac{Et}{4\left(\frac{h}{L}\right)^3 + 2.78\left(\frac{h}{L}\right)} = \frac{29000\,ksi \cdot 4.5''}{4(.316'')^3 + 2.78(.316)} = 129890\ \frac{k}{in}$$

$$I = \frac{tL^3}{12} = \frac{4.5'' \cdot (30' \cdot 12''/ft)^3}{12} = 17496000\ in^4$$

$$\Delta = \frac{Ph^3}{3EI} + \frac{2.78Ph}{A_w E} = \frac{60.4^k (9.5 \cdot 12)^3}{3(29000)(17496000)} + \frac{2.78(60.4)(9.5 \cdot 12)}{4.5''(30' \cdot 12)(29000)} = .00046''$$

$$\Delta_{unit} = \frac{1}{k} = .0000077699$$

$$\Delta = 60.4\left(\frac{1}{k}\right) = .00046''$$

$$\frac{h}{400} = \frac{9.5' \cdot 12''/ft}{400} = .285''$$