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Pennsylvania Judicial Center

Harrisburg, PA

Technical Report #3
Lateral System Analysis

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Executive Summary

The purpose of this report is to perform a detailed analysis of the lateral system for the Pennsylvania Judicial Center. This is a nine-story, 425,000 square foot building project currently under construction in Harrisburg, PA. This $95 million building will house the Pennsylvania Unified Judicial System, and features courtrooms, conference rooms, and offices.

The Pennsylvania Judicial Center has a steel frame with composite floor slabs. The building resists lateral loads using concentrically braced frames between the floor slabs, which act as rigid diaphragms. The frames use stiffness in the plane of the lateral load and act similar to a truss to transfer the loads to the columns, which then transfer the loads to the foundation below.

A three-dimensional computer model created using RAM Structural System was a significant tool that aided the both the determination and the distribution of the lateral loads. As expected for Harrisburg, not an area of high seismicity, wind was the controlling force for the design. The base shear was calculated to be 634k, which is within 1% error of the base shear calculated by the design professional (640k). RAM also calculated the distribution of the shear to the frames. These results were compared to a distribution done based on relative stiffness. The relative stiffnesses were found using RAM Advanse by putting a unit load at the top of one-floor representative frame for each building frame and comparing the deflections. Even though this was intended to provide a rough estimate, most of the values this method yielded were actually quite close to the RAM Structural System analysis, giving confidence that the distribution was reasonably accurate.

A detailed analysis on the torsional forces was performed. It was found that torsional shear could be as high as one-third of the direct shear on a frame story but was typically on the order of 5-10%. Therefore, torsion caused by accidental eccentricity of the wind force should be included in frame design.

A check on building drift under design loading was performed using RAM Structural System. The frame was found to easily meet the H/400 drift requirement, as the maximum drift the building will experience is 2.06" compared to a drift limit of 4.2".

Foundation design was also a consideration in this report, as the footings beneath the frames had to be able to resist moments in addition to gravity loads. The overall overturning moments that the building must resist are 41,900 ft-lbs in the E-W direction and 33,200 ft-lbs in the N-S direction. However, most of the stress transferred to the foundations will be axial rather than flexural since the braces transfer most of the lateral load into the column axis.

Finally, a strength check was performed on a typical nine story braced frame under design wind loading. Performed using the RAM Advanse computer software, all of the members passed the code check.

Structural System Overview

Floor system:

The typical floor is supported by a composite steel and concrete system. The concrete is lightweight (110 pcf dry unit weight) and has a minimum 28-day strength of 4000 psi. There is 3½" of concrete above a 3" 18-gage galvanized composite cellular metal deck, for a total slab depth of 6½". Typical reinforcement is welded wire fabric, 6x6-W2.9xW2.9. The slab is supported by steel beams with typical sizes ranging from W16x36 to W24x68. Typical spans run as long as 42 feet, and the widest spacing between beams is ten feet. The typical spacing between beams is also approximately ten feet. Composite action is enforced by ¾" diameter shear studs with 5½" length.

Roof system:

The flat roof system is identical to the typical 6½" concrete slab floor system. The sloped monitor roof on the ninth-floor tower has a 3" 20-gage galvanized metal deck. The roof is supported by sloped beams ranging from W8x10 to W12x19, with spans no longer than 25 feet and a 9' maximum spacing. The monitor above the main atrium features the same deck, but it is supported by bent W30x90 beams spanning 56' and spaced at ten feet o.c.

Lateral system:

The structure is laterally supported by concentrically braced steel frames in both the N-S and E-W directions. These frames consist of the wide flange columns, wide flange beams at each story and two HSS (hollow structural section) diagonal braces between each story. The geometry of the diagonal members varies, and this has an impact on their relative stiffnesses. This lateral system features no moment connections, and relies on concrete floor and roof slabs to act as rigid diaphragms and to distribute the lateral loads accordingly.

Foundation:

The slab on grade concrete is normal-weight (145 pcf dry unit weight) and has minimum 28-day strength of 5000 psi. The slab on grade is fiber-reinforced at not less than 1.5 lb/yd^3 in some areas and is reinforced with #3 bars @ 18" c/c in the rest of the slab. Typical slab thicknesses are 5" with 6" drainage fill and 8" with 8" drainage fill. Column loads of up to 1,000 kips are supported using concrete piers with diameter of up to eight feet end bearing on rock. Larger column loads are supported by socketed caissons with diameters up to 4.5 feet with up to 18' depth. The piers will bear on grey limey shale bedrock with an allowable bearing stress of 30 ksf. The median core depth to reach bedrock was 9.5 feet, and bedrock depth is relatively uniform throughout the site. The concrete basement foundation walls will be supported by continuous wall footings.

Columns:

The columns are ASTM A992 Grade 50 wide flange steel shapes laid out in a mostly rectangular grid. In this system the columns are acting as the primary gravity resistance members. The columns that are attached as braced frames are also the main lateral force resisting members. The braces between columns are ASTM A500 Grade B HSS shapes ranging in size from 8×8×1/2" to 12×12×5/8". The largest column is a W14x550, though most of the columns are on the order of 300 lb/ft at the ground floor.

Loads

Floor Live Loads:

Load Area	Building Design Load	Minimum Load, ASCE 7-05
Corridors	125 psf	100 psf, first floor 80 psf, all other floors
Offices	125 psf	50 psf
Courtrooms	60 psf + 20 psf partition	60 psf, if seats are fixed
Lobbies and Stairs	125 psf	100 psf
Storage Rooms	125 psf	125 psf for light storage (warehouse)
Archive Storage Room	250 psf	250 psf for heavy storage (warehouse)
Conference Center	125 psf	100 psf (assembly area)
Library (Stacks)	150 psf	150 psf
Cafeteria	100 psf	100 psf (assembly area)
Mechanical Rooms (fans only)	125 psf	n/a
Mechanical Penthouse	250 psf	n/a
Exterior Plaza	100 psf	100 psf (assembly area)
fire vehicle access area	300 psf	n/a
Parking Garage	100 psf	40 psf
Loading Dock	250 psf	n/a

Roof Live Loads:

Item	Design Value	Code Basis
Roof Live Load	20 psf min	ASCE 7-05
Ground Snow Load (Pg)	30 psf	IBC Figure 1608.2
Flat-roof Snow Load (Pf)	21 psf + drift	IBC Section 1608.3
Snow Exposure Factor (Ce)	1.0	IBC Table 1608.3.1
Snow Importance Factor (I)	1.0	IBC Table 1604.5
Thermal Factor (Cf)	1.0	IBC Table 1608.3.2
Rainwater Ponding Load	30 psf (avg. of 6")	n/a

Dead Loads:

Item	Design Value
Concrete Slab, Typical Floor	50 psf
Superimposed Dead Loads	
Mechanical, Electrical, Sprinkler	20 psf
Ceiling Finishes	5 psf
Floor Finishes	5 psf
Steel Structure	Varies
Other Dead Loads	Where applicable

Seismic Analysis

Harrisburg, Pennsylvania is not considered a high-risk area for seismic activity. However, due to an increased emphasis on seismic design in the new codes, seismic loads must be considered for almost every new structure constructed in the United States. For the hand calculations, the equivalent lateral force method was deemed appropriate and sufficient for a seismic analysis for this area. The seismic coefficients used in the design were provided in the construction documents, and were shown to be in line with the ASCE 7-05. The only discrepancy was that the new code suggests a value of 3.25 for R and C_d, rather than 3, but the coefficients in the design of this report were made to match those used in the building design an attempt to keep my analysis in line with the design professional's as much as possible.

Seismic weight typically includes dead load only, but there are code provisions to include percentages of certain live loads. This was accounted for with a relatively conservative uniform dead load, 100 psf. I also added an exterior wall load of 45 pounds per square foot of wall area. Index force analysis, the simplest possible seismic analysis, was performed to determine if my assumptions were reasonable. The result was a base shear of 407k, which is at least on the same order of magnitude as the design base shear of 640k. The spreadsheet can be found on page 18 of this report.

When attempting to find seismic forces using the equivalent lateral force method, the result was a base shear of 1100k, which was too far away from the design base shear. Since all of the same coefficients were used as those of the design professional, the two possibilities for the discrepancy were weight and building period. I ruled out weight since the difference in base shears was so great, it would require a radically different weight to approach the same value. The approximate period equation in the code was used to obtain a period T of 0.89s in the initial calculation, but a provision was found enabling the period to be increased to up to 1.51s. At this longer period, I calculated the base shear to be 655k, almost exactly equal to the design value.

The values obtained by the RAM Structural System analysis are different still. The base shear was found to be 453k, with periods ranging from 3.12s for the first mode to 0.47s for the ninth mode. Clearly, the key to finding the correct base shear is to get an accurate representation of the period, which is not easy to do. Which base shear value should be the one to trust? If one has faith in the computer model, when earthquake loads are compared to wind loads, wind clearly controls. This is what was expected for this building in an area of low seismicity. Even if the designer was to be conservative and use the hand calculation, that base shear is almost identical to that of wind; therefore, it is a moot point from design in this case.

Note: In each method, the base shear was calculated without deduction mass for slab penetrations, which is slightly conservative.

Seismic Coefficients:

Item	Design Value	Code Basis
Hazard Exposure Group	I	IBC Section 1616.2
Performance Category	B	IBC Table 1616.3
Importance Factor (I)	1.0	IBC Table 1604.5
Spectral Acceleration for Short Periods (Ss)	0.21g	IBC Figure 1615 (1)
Spectral Acceleration for a One Second Period (S_1)	0.064g	IBC Figure 1615 (2)
Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	0.168g	IBC Section 1615.1.3
Damped Design Spectral Response Acceleration at Short Periods (S_{D1})	0.073g	IBC Section 1615.1.3
Seismic Response Coeff. (Cs)	0.013	IBC Section 1617.4
Site Class	C (very dense soil)	IBC Table 1615.1.1
Basic Structural System	Building Frames	IBC Table 1617.6.2
Seismic Resisting System	Concentric Braced Frames	IBC Table 1617.6.2
Response Modification Factor (R)	3.0	IBC Table 1617.6.2
Deflection Modification Factor (Cd)	3.0	IBC Table 1617.6.2
Analysis Procedure Utilized	Equivalent Lateral Force	
Design Base Shear	640k	

Wind Analysis

Since seismic is usually not a driving factor in this building's region, it will probably be the wind force that controls the design of the lateral resistance system. Therefore, a relatively rigorous wind calculation would be an essential endeavor. For this report, Method 2 will be used to calculate wind pressures on the main wind-force resisting system. To perform a detailed wind design for a building, a components and cladding analysis is necessary. However, for the purpose of getting a wind load on the overall building for this report, a MWFRS analysis is sufficient.

The first step in the wind calculations was to determine all of the wind coefficients; this work is shown on pages 22-23. An analysis was conducted in each of the two principal directions. The windward and leeward pressures are the essential values for the overall building system. Roof pressure is relatively unimportant for this building, since the uplift will be easily resisted by the heavy, primarily flat roof slab. Side wall pressures may be important to component design or deflection criteria, but for overall system design, they will not control and can be ignored.

A positive pressure on the windward building face and a negative pressure on the leeward face will both occur in the same direction; therefore, their effects can be considered cumulative when discussing overall building criteria such as base shear. For 90 MPH wind acting on the north or south face, the building experiences a 665k windward force and a 600k leeward force. The east and west faces, which have a smaller surface area normal to the wind, would experience a 577k windward force and a 448k leeward force.

These hand calculations provide an excellent ground for comparison to the values given by the RAM Structural System model of the building. In the north-south direction, RAM calculated a total base shear for the building of 634k, which differs from the hand-calculated value for windward force by just 3%. The east-west RAM base shear was 569k, a difference of less than 2% from the hand calculations. A possible small source of error in the RAM model is that the sloped roofs were not modeled; however, the surface area neglected is fairly small and so is the effect on total building shear. The agreement between the computer model and hand calculations gives great confidence in the validity of all of the calculations.

Lateral Force Distribution

The primary lateral force resistance is achieved using concentric braced frames. All of the frames in this system safely transfer the forces using the same concepts; however, minor differences in geometry can have a large impact on the frame's stiffness and, therefore, its contribution to lateral force resistance.

Using RAM Structural System, a very detailed distribution analysis was performed. All of the frames in the computer model feature the same geometry, member sections, and location as the frames designed in the building. The columns and beams that were not part of the frame were not necessarily all the same as those in the construction documents; they were optimized in the RAM model to work. However, since we are assuming that the frames take 100% of the lateral resistance; this is a moot point for this analysis.

RAM Structural System calculated all of the wind and seismic load cases, along with their combinations, and determined their effects on the building. The IBC 2003 load cases were used. It was determined that wind controlled in both orthogonal directions; again, this is a result that was expected. RAM calculated the shear at each floor; the base shears in the N-S direction and E-W direction are 634k and 569k, respectively. RAM also distributed the forces to each frame; see the spreadsheet on page 28 for load distribution. Since all floors are assumed to act as rigid diaphragms, the forces are assumed to be distributed according to relative stiffness.

My analysis of the lateral system also included constructing models of typical frame bays in RAM Advanse and subjecting the frames to a unit force to find the relative deflections. The inverses of these deflections will then provide the relative stiffnesses of the frames. This method is, of course, an approximation, but it is a reasonably accurate analysis to test that the data that is produced in RAM is logical. For the RAM Advanse analysis, five typical types of frames were considered. For most frames, the patterns usually repeat from the ground floor to the roof.

The spreadsheets on pages 30-33 show how the lateral loads are divided by the frames. If one compares that distribution to the one from RAM, one can see that the load percentages in each analysis are quite similar. This gives confidence that both analyses have a reasonable degree of accuracy, with the RAM Structural System data being the numbers that would be used because RAM more closely models the actual system. Due to a relatively symmetrical geometry, one would not expect torsion to be a critical design issue. However, since some frames are located on the exterior of a relatively large building footprint, torsion will create some force on the lateral system. Therefore, torsional factors were not considered in either distribution analysis, but are calculated in the Torsion section of this report.

Torsion

In addition to the lateral forces that the eccentrically braced frames are designed to withstand, if the lateral force is eccentric it can torque the frames and put extra burden on them. Torsional effects should be always calculated for lateral systems, and their contribution to the load can range from negligible to substantial. The symmetry of the Pennsylvania Judicial Center is a favorable case to limit torsion. For wind load, torsion increases as the eccentricity between the center of mass and the geometrical center increases; this eccentricity is very small for this structure. As a matter of fact, on every floor the distance between the mass and geometric centers was less than 5% of the building length; therefore, an “accidental” eccentricity of 5% was assumed on each floor. However, since some of the building frames are located far from center, torsion could still have some impact on the system.

The analysis used the relative stiffnesses calculated based on the deflections of representative frames under unit loads. This was proven in the Lateral Distribution section to be a reasonably accurate assumption. The shear forces came from the RAM data since that is the most accurate data available. Torsional shear was calculated using the following equation:

$$\text{Torsion} = \frac{H_S\, e\, K_{SN}\, C_N}{\sum(K_{SN}\, C_N^2)}$$

where H_S = story shear, K_{SN} = relative stiffness, C_N = distance to frame

After calculating the numbers, it became evident that torsion has a relatively strong impact on the frame design. Torsional shear was as much as a third of the direct shear on some frames, but for most frames it was between 5-10%. The frames that ran along the short dimension of the building had approximately three times the torsional shear as those running parallel to the long dimension; since some of the E-W frames are on the exterior wall farthest from the center, this makes sense. The absolute value of the torsional shear of each frame should be added to the direct shear of each frame, and this force is what the frame needs to be able to resist.

Torsional Effects on Braced Frames

East-West Direction

Frame	1/Defl	2	2T	3	3T	4	4T	5	5T	6	6T	7	7T	8	8T	9	9T	PH	PHT	Total
17/J-N	3.76	64	18.2	57	12.0	51	10.5	44	8.6	37	7.1	80	0.8	58	0.4	35	0.2	12	0.0	57.8
16/P-T	7.75	131	29.5	118	18.8	105	16.3	91	13.3	76	11.0	166	0.8	120	0.2	72	0.0	25	0.0	90.0
10/E-F	7.58	128	2.4	116	4.5	103	4.4	89	4.6	75	4.0	0	2.6	0	1.9	0	1.1	0	0.4	26.0
10/W-X	7.58	128	2.4	116	4.5	103	4.4	89	4.6	75	4.0	0	2.6	0	1.9	0	1.1	0	0.4	26.0
6/E-G	2.29	39	12.6	35	10.1	31	9.1	27	8.1	23	6.8	0	2.1	0	1.4	0	0.8	0	0.3	51.2
6/V-X	2.29	39	12.6	35	10.1	31	9.1	27	8.1	23	6.8	0	2.1	0	1.4	0	0.8	0	0.3	51.2
	31.25	529	77.5	477	60.0	425	53.9	367	47.3	308	39.7	246	11.0	178	7.2	107	4.1	37	1.4	302.1

North-South Direction

Frame	1/Defl	2	2T	3	3T	4	4T	5	5T	6	6T	7	7T	8	8T	9	9T	PH	PHT	Total
J/16-17	7.58	208	7.6	174	5.9	139	4.9	105	3.6	70	2.2	60	0.5	43	0.3	26	0.2	9	0.1	25.3
T/16-17	7.58	208	13.2	174	8.3	139	6.4	105	5.0	70	3.5	60	0.6	43	0.3	26	0.2	9	0.1	37.5
E/9-10	1.14	31	3.3	26	2.4	21	1.9	16	1.4	11	0.9	0	0.2	0	0.1	0	0.1	0	0.0	10.4
X/9-10	1.14	31	4.2	26	2.7	21	2.2	16	1.7	11	1.1	0	0.2	0	0.1	0	0.1	0	0.0	12.3
E/6-7	1.14	31	3.3	26	2.4	21	1.9	16	1.4	11	0.9	0	0.2	0	0.1	0	0.1	0	0.0	10.4
X/6-7	1.14	31	4.2	26	2.7	21	2.2	16	1.7	11	1.1	0	0.2	0	0.1	0	0.1	0	0.0	12.3
	19.72	541	35.8	453	24.4	362	19.5	274	14.7	183	9.8	120	1.9	86	1.2	52	0.6	18	0.2	108.2

Blue- Torsion Force (kips)
White- Shear Force (kips)

		2		3		4		5		6		7		8		9		PH	
Frame	e =	17	K*Cn^2	13.5	K*Cn^2	13.5	K*Cn^2	13.5	K*Cn^2	13.5	K*Cn^2	5.5	K*Cn^2	5.5	K*Cn^2	5.5	K*Cn^2	5.5	K*Cn^2
17/J-N	Cn	108	43857	97	35378	95	33934	91	31137	90	30456	44	7279	33	4095	26	2542	22	1820
16/P-T		85	55994	74	42439	72	40176	68	35836	67	34790	21	3418	10	775	3	70	1	8
10/E-F		7	371	18	2456	20	3032	24	4366	25	4738	71	38211	82	50968	89	60041	93	65559
10/W-X		7	371	18	2456	20	3032	24	4366	25	4738	71	38211	82	50968	89	60041	93	65559
6/E-G		123	34645	134	41119	136	42356	140	44884	141	45527	187	80079	198	89777	205	96237	209	100029
6/V-X		123	34645	134	41119	136	42356	140	44884	141	45527	187	80079	198	89777	205	96237	209	100029
J/16-17		22	3669	25	4738	26	5124	25	4738	23	4010	29	6375	30	6822	31	7284	31	7284
T/16-17		38	10946	35	9286	34	8762	35	9286	37	10377	31	7284	30	6822	29	6375	29	6375
E/9-10		64	4669	67	5117	68	5271	67	5117	65	4817	71	5747	72	5910	73	6075	73	6075
X/9-10		80	7296	77	6759	76	6585	77	6759	79	7115	73	6075	72	5910	71	5747	71	5747
E/6-7		64	4669	67	5117	68	5271	67	5117	65	4817	71	5747	72	5910	73	6075	73	6075
X/6-7		80	7296	77	6759	76	6585	77	6759	79	7115	73	6075	72	5910	71	5747	71	5747
sum=		721	201133	746	195984	751	195900	758	196490	758	196910	856	278504	879	317733	894	346724	904	364561

Drift

For serviceability considerations and building inhabitant comfort, building deflections should be limited as much as possible. As the total stiffness of the lateral system increases, building drift will decrease. Unless there are special considerations for a building, the industry standard is to design the building so that the maximum deflection is equal to $1/400^{th}$ of the building height. Since the RAM analysis only considers to the top of the roof slab, we will compare the drift at that level to the height at the top of the roof slab. Deflection will be limited to Δ_{max} = (140' × 12"/ft) / 400 = 4.2".

A benefit of using computer-modeling software, such as the RAM Structural System program used for this report, is that the computer is able to take what is a relatively complicated calculation for drift, and perform it for all possible load cases almost instantly. See spreadsheet in Appendix B for maximum drift in each direction for each load case. Thanks to this detailed analysis that a computer can effortlessly perform, one can easily see that in the E-W direction, deflection is controlled by wind (Δ_y = 2.06") while in the N-S direction, deflection is controlled by seismic forces (Δ_x = 1.99"). This second result is a bit surprising, since all other analyses conducted so far have been controlled by wind design. It is likely that, without the computer software, a designer would not feel that it would be time-effective to do a drift analysis for seismic if wind was already found to control. However, the worst-case deflections in each direction are less than half of the maximum, so the building easily meets the drift criterion as designed.

The table below shows the drift, by story, of the controlling load case. In both the in-plane and out-of-plane drift, one can see by the increasing change in deflection that a gradient is forming. In the E-W direction, this pattern is disrupted only at the top of the building, where the building's area significantly decreases. The roof tops off on half the building at level 6, which is why there is a large jump in change in drift at floor level 7. Rotation in the Z-direction increases relatively steadily from the bottom to the top.

Story Drift (Wind in E-W direction):

Story	**E-W Drift (in)**	**N-S Drift (in)**	**Θ_z (radians)**
PH	2.09 (+0.24)	0.57 (+0.10)	13×10^{-5}
9	1.85 (+0.26)	0.47 (+0.10)	11
8	1.59 (+0.29)	0.37 (+0.09)	10
7	1.30 (+0.25)	0.28 (+0.16)	9
6	1.05 (+0.22)	0.12 (+0.04)	8
5	0.83 (+0.21)	0.08 (+0.03)	6
4	0.62 (+0.20)	0.05 (+0.02)	4
3	0.42 (+0.17)	0.03 (+0.01)	3
2	0.25	0.02	2

Overturning Moment

The overturning moment was calculated as simply the sums of the concentrated forces on each story multiplied by the height above ground level for each story. The forces using were found using the change in shear between floors from the output from RAM Structural System. Obviously, the shear is additive as the loads carry down to the ground floor, so the amount that you add on each floor is the force on that floor.

The overall overturning moment was calculated to be 41,900 ft-lbs in the east-west direction and 33,200 ft-lbs in the north-south direction. This was a slightly surprising result since the surface area for the wind to act on is much greater on the north and south face. However, when the front part of the building is capped off by a roof on the sixth floor, there is more surface area for the east-west faces than the north-south faces. Since these floors are the highest, they will have the greatest impact on the moment at the base, so this result is logical. The spreadsheet on page 14 shows exactly this; even though there is more total force going N-S, more of the force is higher above ground going E-W.

The overturning moments of the individual frames in the spreadsheet are misleading. The spreadsheet calculations would be applicable only to a shear wall that acts autonomously from the rest of the building system. Therefore, the foundations will not be expected to resist the entire OTMs for the frames calculated in the spreadsheet, which is a good thing because the moments range from 2600 to over 14,000 ft-lbs. Instead, when the loads are transferred to the concentrically braced frames by the rigid diaphragm, each braced frame acts like a truss. The loads are converted into axial load by the intermediate members and transferred into the columns. The columns can handle axial compression load much better than bending load, and this is certainly true for the foundations as well. The concrete foundations perform best under compressive forces, and for this project, the piles and caissons bear on rock with allowable bearing stress of 30 ksf.

Overturning Moment due to Wind

East-West Direction

Story	Height (ft)	Force on Building (k)	0	1	2	3	4	5	6
					Force on Frame #				
PH	140	37		19	18				
9	124	70		35	36				
8	109	71		35	35				
7	93	68		34	34				
6	78	62		-53	-35	29	52	32	38
5	63	59		36	36	1	4	-8	-10
4	48	58		21	13	2	-1	12	12
3	33	52		36	37	-4	-6	-4	-6
2	18	52		-64	-60	156	13	2	4
1	0	40		5	-14	-91	11	61	69
	OTM (ft-k):	41912	0	13155	14264	5097	4296	2472	2784

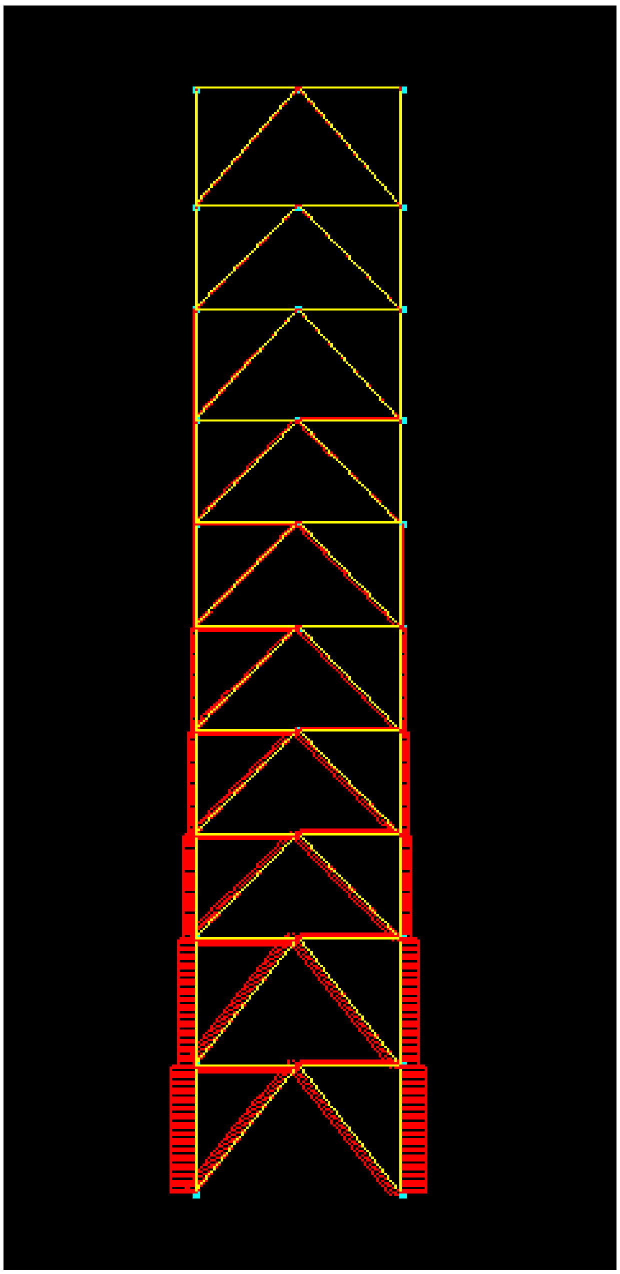
North-South Direction

Story	Height (ft)	Force on Building (k)	0	1	2	3	4	5	6
					Force on Frame #				
PH	140	18		20	-2				
9	124	34		18	16				
8	109	35		0	34				
7	93	34		18	16				
6	78	63		7	16	12	6	12	11
5	63	91		-3	32	22	26	7	7
4	48	88		12	23	12	23	9	9
3	33	92		-2	12	30	44	4	4
2	18	88	11	79	-3	-38	-43	42	41
1	0	93	28	39	-52	62	15	0	1
	OTM (ft-k):	33204	198	8995	11608	3204	3888	2697	2601

Strength Check

A strength check was performed on a nine story braced frame located on grid coordinates 16/P-T. This analysis was performed using RAM Advanse, using all of the section shapes designed by the professional. In this analysis, all of the columns, beams, and diagonal braces passed the code check.

One benefit of using computer modeling is that stresses on all of the members can be easily found. The maximum stress ratio for a member under design wind loading is approximately 0.30. The stresses on the frame are primarily axial, which is typical for a truss-like system. The axial stress distribution of the frame is shown below.



Conclusions

The following conclusions can be made based on calculations performed on the lateral system of the Pennsylvania Judicial Center:

- Wind force controls over seismic force in the design of lateral loads. For a nine-story building in an area of low seismicity, this is not a surprising result.
- Since the concrete slabs are assumed to act as rigid diaphragms, lateral loads will be distributed due to relative stiffness. Even a simple calculation to estimate the relative stiffness of frames can provide a reasonably accurate distribution.
- Based on the building's large footprint and frames located at the corners of the building, torsion will be a relatively significant factor. It should not be ignored; it should be calculated and added to the force analysis.
- Drift under maximum design loads is approximately half of the design goal of $1/400^{th}$ of the height. Even though wind loads control base shear, seismic forces control the building's maximum deflection in the north-south direction. Therefore, all load cases and combinations should be considered for drift when it is practical to do so.
- Based on an analysis that considers only the wind loads acting on the building, an overturning moment of 41,900 ft-lbs. However, not all of this moment will be transferred to the footing; based on how the concentric braced frame system works, a majority of the lateral load is transferred to the columns in the form of compressive axial force.
- A typical frame was checked for strength and was found to meet requirements.

Seismic Calculations

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Seismic Weight:
- Total Dead Load
- 25% of Live Load for Storage
- Include Partition Loads (20psf)
- Equipment Operating Weight
- 20% of flat roof snow load if PF > 30psf

Dead Load for Typical Floor:
50psf Concrete Slab
30psf Superimposed DL (Mech/Elec/Sprinkler/Finishes)
15psf Steel Structure + Metal Deck
5psf Collateral Loads
$\boxed{100\text{psf DL}}$

Storage Areas:
Lightstorage: use $0.25 \times 125\text{psf} \approx \boxed{35\text{psf}}$
Archive storage: use $0.25 \times 250\text{psf} \approx \boxed{65\text{psf}}$

Partition Loads:
$\boxed{+20\text{psf}}$ in courtrooms

Equipment Operating Weights:
As noted on plan.

Snow Load:
Pf = 21psf + drift, which will occur. The flat roof portions feature the same DL as typical floors, and sloped roofs have a DL of just 30 psf. Therefore, I will make the assumption that the whole roof is flat and has 100 psf DL, which should be a conservative assumption based on comparative areas.

Areas:
East Wing "A": $60' \times 90' = 5400 \text{ft}^2$
Tower "B": $(110' \times 200') + 2(5' \times 60') + (5' \times 110') + (9' \times 90') = 24000 \text{ft}^2$
West Wing "C": $(110' \times 40') + (155' \times 120') + (87' \times 30') - 2(\frac{1}{2})(43' \times 12') = 25\,000 \text{ft}^2$
* Separations between parts are match lines on drawings

Total Area $\approx 54400 \text{ft}^2$

# of Floors:
"A": 2 (accommodates future expansion to 5 floors)
"B": 9
"C": 6

Walls:
Curtainwalls weigh 15 lb/sq ft wall area and comprise approx. 1/3 of total wall area.
Limestone panels weigh 60 lb/sq ft and comprise approx. 2/3 of wall area.
Therefore, I will use an allowance of $(15 \times \frac{1}{3}) + (60 \times \frac{2}{3}) = 45\text{psf}$ for walls.

Index Force Analysis
- Very simplified method will be conducted in order to get a general order of magnitude for the calculations.
- For this method, I will just use area x typical DL to get a rough estimate of the weights.
- Building is treated like a cantilever

Floor	Area(ft²)	Weight(k)	$F_x=0.01W_x$(k)	T/slab	Height Above Ground
ground	54400	5440	54.4	19.0'	0'
2	54400	5440	54.4	37.0'	18'
3	49000	4900	49.0	52.0'	33'
4	49000	4900	49.0	67.0'	48'
5	49000	4900	49.0	82.0'	63'
6	49000	4900	49.0	97.0'	78'
7	24000	2400	24.0	112.0'	93'
8	24000	2400	24.0	128.0'	109'
9	24000	2400	24.0	143.0'	124'
penthouse/roof	24000	2400	24.0	158.25'	139.25'
$\sum_{2}^{roof}$	346 400	34640	346.4k		

Wall weights were not included on the above spreadsheet. Typically, I will consider that a floor slab will support 1/2 the wall height of the floor below it and 1/2 of the floor above it,

Perimeter "A" = 210'
Perimeter "B" = 465' @ floor 1, 555' above floor 2, 620' above floor 7
Perimeter "C" = 555'

Floor	Wall Area (ft²)	Weight (k)	$F_x=0.01W_x$ (k)	Height
ground	11685	526	5.3	0
2	19395	873	8.7	18'
3	16650	750	7.5	33'
4	16650	750	7.5	48'
5	16650	750	7.5	63'
6	16650	750	7.5	78'
7	13285	598	6.0	93'
8	9610	433	4.3	109'
9	9300	419	4.2	124'
penthouse/roof	4650	210	2.1	139.25'
$\sum_{2}^{roof}$	134 525	6059	60.6k	

Index Force Analysis Results

Calculated Base Shear = 346.4k + 60.6k = 407k
Design Base Shear = 640k

-The base shear calculated is about 2/3 of the design base shear. A lot of simplifications and assumptions were made in this method, some of them unconservative, so this is a reasonable result.

Overturning Moment = $\Sigma F_x h_x$ = 26700 ftk

Equivalent Lateral Force Method:

- Since lateral loads are not resisted by moment frames, calculate period by: $T_a = C_t h_n^x$, where C_t and x are found in ASCE Table 9.5.5.3.2

$T_a = 0.02(158')^{0.75} = 0.89 < 1 \therefore$ rigid system

$C_s = \frac{S_{DS}}{R/I} \leq \frac{S_{D1}}{T(R/I)}$

$C_s = \frac{0.168}{3/1} \leq \frac{0.073}{(0.89 \times 3)}$

$C_s = 0.056 \leq \boxed{0.027}$

$C_s \geq 0.044 S_{DS} I = 0.007$ ✓ ok

$V = C_s W = 0.027 \times 40700^k = 1100^k$ base shear $> 640^k$ base shear

The value that I calculated for base shear is considerably higher than the design base shear used. Since I used the same seismic coefficients in my analysis as the design professional, I believe that there are two possibilities for the discrepancy. One source of difference could be dead loads that I used were more conservative. However, since the difference is almost a factor of 2, it is doubtful that this had a large impact.

More likely, the period that the designer used was higher than mine, which could have a dramatic effect on base shear. The code has a provision listing that $T \leq C_u T_a$, where C_u ranges from $1.4 \rightarrow 1.7$. Bumping T to $1.7T_a$ changes analysis to a flexible building.

This yields a Cs of 0.016 for a base shear of 655k, which is almost exactly the value the design professional calculated.

Distribution of Loads:

$C_{vx} = \frac{w_x h_x^k}{\sum w_i h^k}$

$F_x = C_{vx} V$

For $T = T_a = 0.89s$, $k = 1.20$ by interpolation ($V = 1100^k$)
$T = 1.7T_a = 1.51s$, $k = 1.50$ by interpolation ($V = 655^k$)

See spreadsheet for load distribution.

Seismic Load Distribution – Equivalent Lateral Frame

Period T = Approximate Period Ta
V = 1100k
k = 1.20

Level	Weight	Story Height h	h^k	Wx*hx^k	Cvx	Fx
2	6320	18	32.09	202789	0.03	34.6
3	6190	33	66.41	411062	0.06	70.2
4	5650	48	104.11	588217	0.09	100.4
5	5650	63	144.28	815187	0.13	139.1
6	5650	78	186.43	1053324	0.16	179.8
7	3000	93	230.24	690718	0.11	117.9
8	2840	109	278.56	791098	0.12	135.0
9	2820	124	325.17	916970	0.14	156.5
penthouse/roof	2610	139.25	373.73	975427	0.15	166.5
Sum	**40730**			**6444792**	**1**	**1100**

Period T = Max Cu*Ta
V = 650k
k = 1.50

Level	Weight	Story Height h	h^k	Wx*hx^k	Cvx	Fx
2	6320	18	76.37	482643	0.02	12.9
3	6190	33	189.57	1173442	0.05	31.3
4	5650	48	332.55	1878929	0.08	50.1
5	5650	63	500.05	2825266	0.12	75.4
6	5650	78	688.88	3892157	0.16	103.9
7	3000	93	896.86	2690579	0.11	71.8
8	2840	109	1137.99	3231901	0.13	86.2
9	2820	124	1380.81	3893872	0.16	103.9
penthouse/roof	2610	139.25	1643.21	4288775	0.18	114.4
Sum	**40730**			**24357563**	**1**	**650**

Wind Calculations

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Design Wind Speed V=90 MPH — IBC 2003 Figure 1609
Wind Importance Factor I=1.0 — Table 1604.5
Wind Exposure Category B — Table 1609.4

Building is Partially Enclosed
→ Internal Pressure Coefficient $GC_{pi} = \pm 0.55$

Exposure Category (K_h/K_z) — ASCE 7-05 Table 6-3

Height Above Ground	C+C	MWFRS
0-15'	0.70	0.57
20'	0.70	0.62
25'	0.70	0.66
30'	0.70	0.70
40	0.76	0.76
50	0.81	0.81
60	0.85	0.85
70	0.89	0.89
80	0.93	0.93
90	0.96	0.96
100	0.99	0.99
120	1.04	1.04
140	1.09	1.09
160	1.13	1.13
180	1.17	1.17

Topographic Factor $K_{zt} = 1.0$ — ASCE 7-05 Section 6.5.7
Wind Directionality Factor $K_d = 0.85$ — Table 6-4
T = 0.1 × # stories = 0.9
$n_1 = \frac{1}{T} = 1.11 \Rightarrow$ assume rigid structure

Gust Factor G = 0.85 or $G = 0.925\left(\frac{1+1.7g_Q I_{\bar{z}} Q}{1+1.7g_v I_{\bar{z}}}\right)$ — Section 6.5.8.1

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{0.6(177)}\right)^{1/6} = 0.25$$

$g_Q = g_v = 3.4$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = 0.835 \perp \text{short dir}, \ 0.805 \perp \text{long dir}$$

B = 213' in short direction, 380' in long direction
h = 177'

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{10}\right)^{\bar{\epsilon}} = 320\left(\frac{0.6(177)}{10}\right)^{1/3} = 703.4$$

G = 0.835 for short dir, G = 0.818 for long dir

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From Fig. 6-6:

C_p: Windward Wall → $C_p = 0.8$ (Use q_z)
Leeward Wall → $C_p = -0.5$ for $L/B \leq 1$
-0.3 $= 2$ (Use q_h)
-0.2 ≥ 4
Side Wall → $C_p = -0.7$ (use q_h)

Roof (see spread sheet)

$P = qGC_p - q_i(GC_{pi})$
$q = q_z$ for Windward Wall (height dependent)
$q = q_h$ for Leeward wall, side walls, and roofs
$q_i = q_h$ for negative internal pressure evaluation
$q_i = q_z$ for positive internal pressure evaluation
z is highest opening that will effect enclosure status (Assume 90')

$q = 0.00256\, K_z \underbrace{K_{zt}}_{1.0} \underbrace{K_d}_{0.85} \underbrace{V^2}_{90} \underbrace{I}_{1.0}$

$q = 17.625\, K_z$

$q_h = 17.625(1.17) = 20.62$ psf

Long dimension ~ 310'
Short dimension ~ 200'
Tower extends ~ 150' in the long dimension

Tall Roof 140' above ground
Wing Roof 78' above ground

Surface Area in N-S direction:
$(310' \times 78') + (150' \times 62') = 33500\,ft^2$
Surface Area in E-W direction:
$(200' \times 140') = 28000\,ft^2$

When wind acts on N-S direction, $L/B < 1$ ∴ LW pressure = -17.9 psf
Total LW Force = $17.9 \times 33500 = 600$k

When wind acts on E-W direction, $L/B = 1.55$ ∴ LW pressure ≈ -16.0 psf
Total LW Force = $16.0 \times 28000 = 448$k

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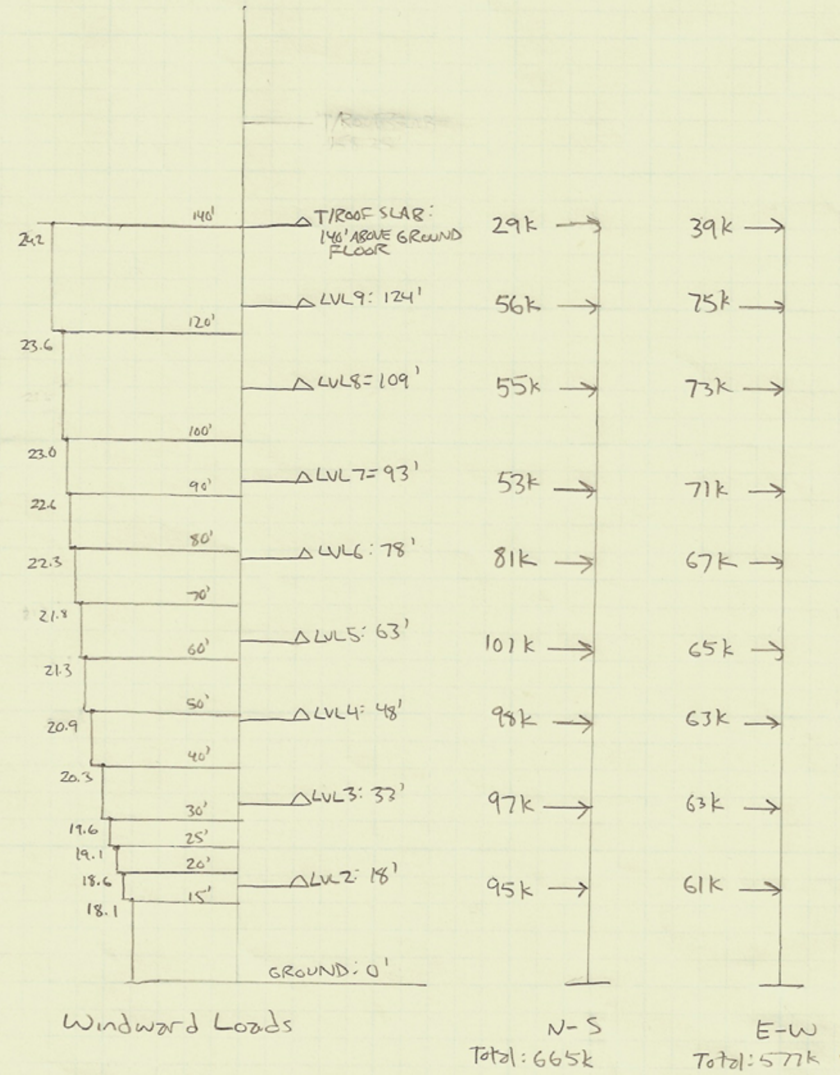


Wind Pressures

Windward Wall Pressures (MWFRS)

Height	Kd	qz	P (short dir)	P (long dir)
0-15'	0.57	10.05	18.1	17.9
20	0.62	10.93	18.6	18.5
25	0.66	11.63	19.1	19.0
30	0.7	12.34	19.6	19.4
40	0.76	13.40	20.3	20.1
50	0.81	14.28	20.9	20.7
60	0.85	14.98	21.3	21.1
70	0.89	15.69	21.8	21.6
80	0.93	16.39	22.3	22.1
90	0.96	16.92	22.6	22.4
100	0.99	17.45	23.0	22.8
120	1.04	18.33	23.6	23.3
140	1.09	19.21	24.2	23.9
160	1.13	19.92	24.6	24.4
180	1.17	20.62	25.1	24.8

Leeward Wall Pressures (MWFRS)

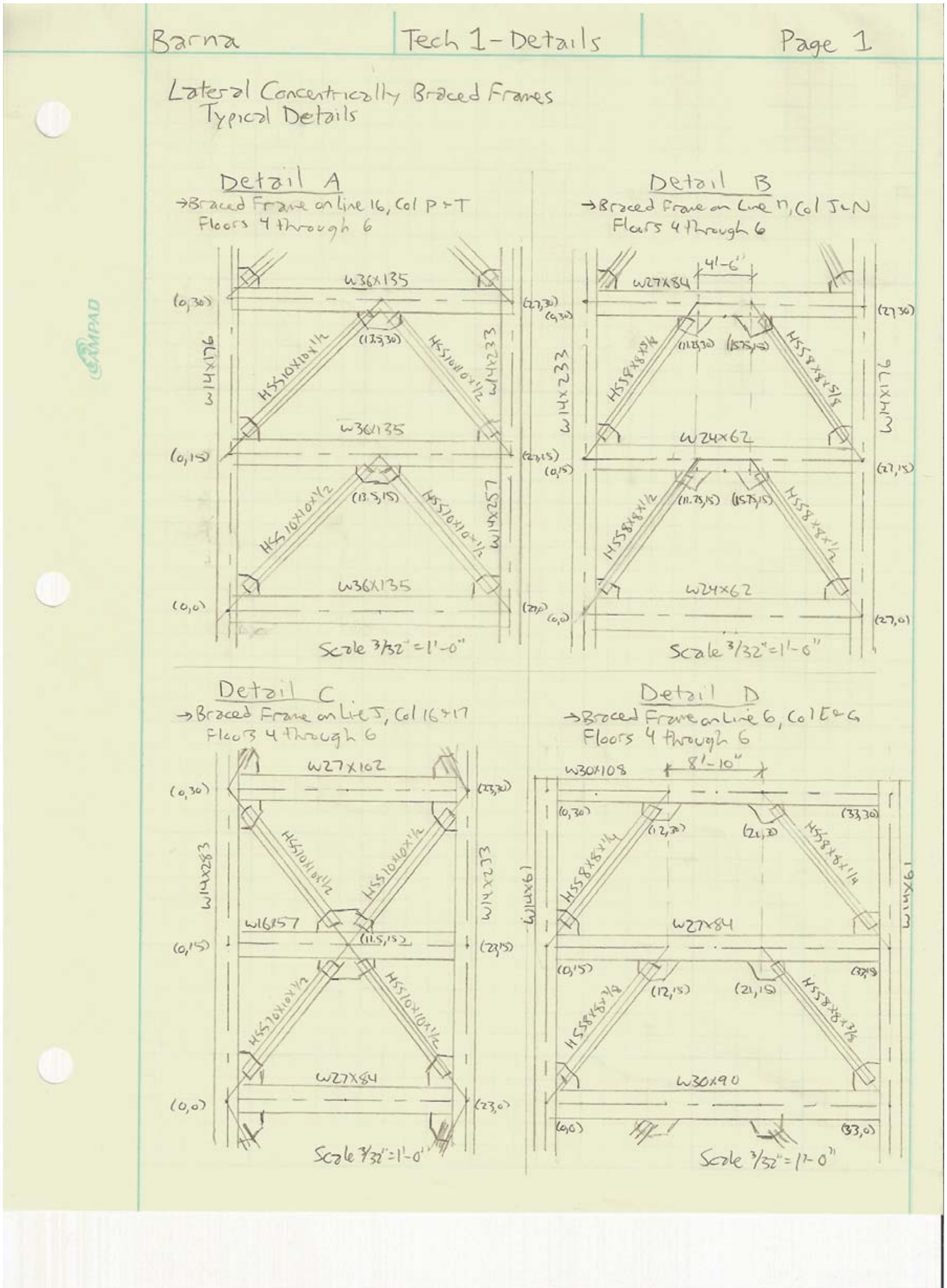
L/B<1	-17.9
L/B=2	-14.5
L/B>4	-12.7

Side Wall Pressure (MWFRS)

P= -21.4

Long direction: 665k windward + 600k leeward = 1265k
Short direction: 577k windward + 448k leeward = 1025k

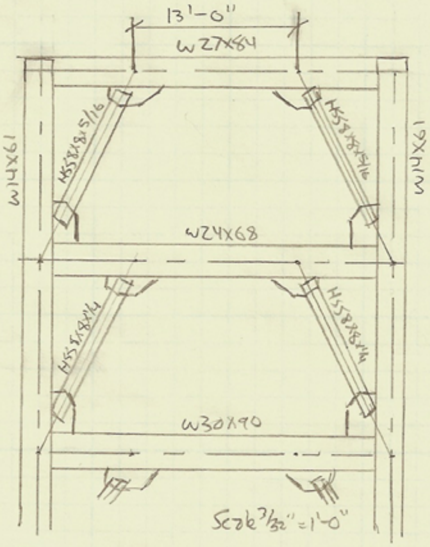
Lateral System Details



Lateral Concentrically Braced Frames
Typical Details

Detail E

→ Braced Frame on Line E, Col 6 + 7
Floors 4 through 6



Lateral Force Distribution

Distribution of Lateral Loads - RAM Model

Level	Shear (Wind X)	Shear (Wind Y)
Penthouse	18	37
9	52	107
8	86	178
7	120	246
6	183	308
5	274	367
4	361	425
3	453	477
2	541	529
1	634	569

X-Direction (N-S)		Shear (kips)									
Frame	Frame #	1	2	3	4	5	6	7	8	9	Penthouse
22/N-R	0	39	11	0	0	0	0	0	0	0	0
17/J-N	1	186	148	69	71	59	62	56	38	37	20
16/P-T	2	92	144	148	136	112	80	65	49	14	-2
10/E-F	3	100	38	76	45	33	12	0	0	0	0
10/W-X	4	71	56	99	55	32	6	0	0	0	0
6/E-G	5	73	73	31	28	19	12	0	0	0	0
6/V-X	6	73	72	31	27	18	11	0	0	0	0
		634	542	454	362	273	183	121	87	51	18

Y-Direction (E-W)		Shear (kips)									
Frame	Frame #	1	2	3	4	5	6	7	8	9	Penthouse
n/a	0	0	0	0	0	0	0	0	0	0	0
J/16-17	1	104	99	162	126	106	69	123	89	53	19
T/16-17	2	100	114	174	137	124	88	123	89	54	18
E/9-10	3	93	184	28	32	30	29	0	0	0	0
X/9-10	4	73	62	49	56	57	52	0	0	0	0
E/6-7	5	94	33	31	35	23	32	0	0	0	0
X/6-7	6	106	37	33	39	27	38	0	0	0	0
		570	529	477	425	367	308	246	178	107	37

Distribution of Lateral Loads - RAM Model

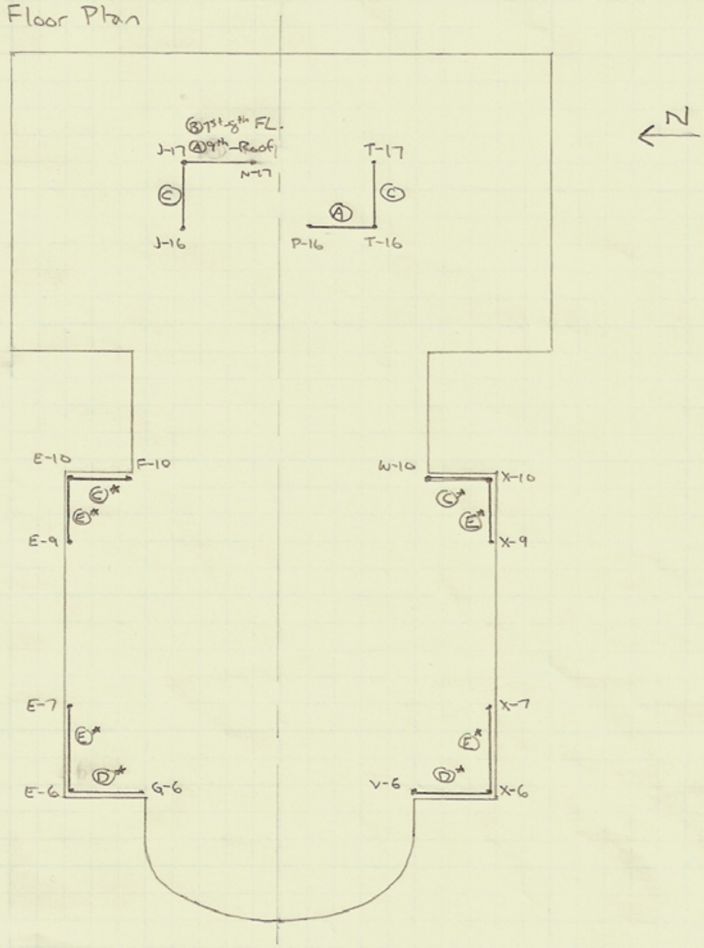
X-Direction (N-S)		Percentage of Story Shear Distributed to Frame									
Frame	Frame #	1	2	3	4	5	6	7	8	9	Penthouse
22/N-R	0	6%	2%	0%	0%	0%	0%	0%	0%	0%	0%
17/J-N	1	29%	27%	15%	20%	22%	34%	47%	44%	71%	111%
16/P-T	2	15%	27%	33%	38%	41%	44%	54%	57%	27%	-11%
10/E-F	3	16%	7%	17%	12%	12%	7%	0%	0%	0%	0%
10/W-X	4	11%	10%	22%	15%	12%	3%	0%	0%	0%	0%
6/E-G	5	12%	13%	7%	8%	7%	7%	0%	0%	0%	0%
6/V-X	6	12%	13%	7%	7%	7%	6%	0%	0%	0%	0%
		100%	100%	100%	100%	100%	100%	101%	101%	98%	100%

Y-Direction (E-W)		Percentage of Story Shear Distributed to Frame									
Frame	Frame #	1	2	3	4	5	6	7	8	9	Penthouse
n/a	0	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%
J/16-17	1	18%	19%	34%	30%	29%	22%	50%	50%	50%	51%
T/16-17	2	18%	22%	36%	32%	34%	29%	50%	50%	50%	49%
E/9-10	3	16%	35%	6%	8%	8%	9%	0%	0%	0%	0%
X/9-10	4	13%	12%	10%	13%	16%	17%	0%	0%	0%	0%
E/6-7	5	17%	6%	6%	8%	6%	10%	0%	0%	0%	0%
X/6-7	6	19%	7%	7%	9%	7%	12%	0%	0%	0%	0%
		100%	100%	100%	100%	100%	100%	100%	100%	100%	100%

Relative Stiffness
- to be performed using a unit load on 2-stories of each of 4 typical braced framed details. (see details)
- RAM Advanse models using 100k unit loads

	Δ @ Top Right Corner	1/Δ
Detail A	0.129	7.75
Detail B	0.266	3.76
Detail C	0.132	7.58
Detail D	0.436	2.29
Detail E	0.881	1.14

Typical Floor Plan



* Represents that frame stops at floor 6

Note: A braced frame in, the east wing only between floors 1-2 to accommodate future expansion was ignored.

Lateral Distribution of Loads

East-West Direction

			Percent of Load Distributed to Frame, by floor								
Frame	**Detail**	**1/Defl**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**penthouse/roof**
J/16-17	C	7.58	38.4%	38.4%	38.4%	38.4%	38.4%	50.0%	50.0%	50.0%	50.0%
T/16-17	C	7.58	38.4%	38.4%	38.4%	38.4%	38.4%	50.0%	50.0%	50.0%	50.0%
E/9-10	E	1.14	5.8%	5.8%	5.8%	5.8%	5.8%	0.0%	0.0%	0.0%	0.0%
X/9-10	E	1.14	5.8%	5.8%	5.8%	5.8%	5.8%	0.0%	0.0%	0.0%	0.0%
E/6-7	E	1.14	5.8%	5.8%	5.8%	5.8%	5.8%	0.0%	0.0%	0.0%	0.0%
X/6-7	E	1.14	5.8%	5.8%	5.8%	5.8%	5.8%	0.0%	0.0%	0.0%	0.0%
		19.72	100%	100%	100%	100%	100%	100%	100%	100%	100%

North-South Direction

			Percent of Load Distributed to Frame, by floor								
Frame	**Detail**	**1/Defl**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**penthouse/roof**
17/J-N	B*	3.76	12.0%	12.0%	12.0%	12.0%	12.0%	32.7%	32.7%	50.0%	50.0%
16/P-T	A	7.75	24.8%	24.8%	24.8%	24.8%	24.8%	67.3%	67.3%	50.0%	50.0%
10/E-F	C	7.58	24.3%	24.3%	24.3%	24.3%	24.3%	0.0%	0.0%	0.0%	0.0%
10/W-X	C	7.58	24.3%	24.3%	24.3%	24.3%	24.3%	0.0%	0.0%	0.0%	0.0%
6/E-G	D	2.29	7.3%	7.3%	7.3%	7.3%	7.3%	0.0%	0.0%	0.0%	0.0%
6/V-X	D	2.29	7.3%	7.3%	7.3%	7.3%	7.3%	0.0%	0.0%	0.0%	0.0%
		31.25	100%	100%	100%	100%	100%	100%	100%	100%	100%

*Detail B for 1st-8th floor, then Detail A up to roof

Seismic Load Distribution on Braced Frames

Period T = Approximate Period
Ta
V = 1100k
k = 1.20

Approximate Load on Each Frame Story, kips

Frame	**Detail**	**1/Defl**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**penthouse/roof**	**Total Load**
J/16-17	C	7.58	13	27	39	53	69	59	68	78	83	489
T/16-17	C	7.58	13	27	39	53	69	59	68	78	83	489
E/9-10	E	1.14	2	4	6	8	10	0	0	0	0	30
X/9-10	E	1.14	2	4	6	8	10	0	0	0	0	30
E/6-7	E	1.14	2	4	6	8	10	0	0	0	0	30
X/6-7	E	1.14	2	4	6	8	10	0	0	0	0	30
		19.72	35	70	100	139	180	118	135	157	166	1100

North-South Direction

Approximate Load on Each Frame Story, kips

Frame	**Detail**	**1/Defl**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**penthouse/roof**	**Total Load**
17/J-N	B*	3.76	4	8	12	17	22	39	44	78	83	307
16/P-T	A	7.75	9	17	25	35	45	79	91	78	83	462
10/E-F	C	7.58	8	17	24	34	44	0	0	0	0	127
10/W-X	C	7.58	8	17	24	34	44	0	0	0	0	127
6/E-G	D	2.29	3	5	7	10	13	0	0	0	0	38
6/V-X	D	2.29	3	5	7	10	13	0	0	0	0	38
		31.25	35	70	100	139	180	118	135	157	166	1100

*Detail B for 1st-8th floor, then Detail A up to roof

Seismic Load Distribution on Braced Frames

Period T = Max Cu*Ta
V = 650k
k = 1.50

Approximate Load on Each Frame Story, kips

Frame	Detail	1/Defl	2	3	4	5	6	7	8	9	penthouse/roof	Total Load
J/16-17	C	7.58	5	12	19	29	40	36	43	52	57	293
T/16-17	C	7.58	5	12	19	29	40	36	43	52	57	293
E/9-10	E	1.14	1	2	3	4	6	0	0	0	0	16
X/9-10	E	1.14	1	2	3	4	6	0	0	0	0	16
E/6-7	E	1.14	1	2	3	4	6	0	0	0	0	16
X/6-7	E	1.14	1	2	3	4	6	0	0	0	0	16
		19.72	13	31	50	75	104	72	86	104	114	650

North-South Direction

Approximate Load on Each Frame Story, kips

Frame	Detail	1/Defl	2	3	4	5	6	7	8	9	penthouse/roof	Total Load
17/J-N	B*	3.76	2	4	6	9	12	23	28	52	57	194
16/P-T	A	7.75	3	8	12	19	26	48	58	52	57	283
10/E-F	C	7.58	3	8	12	18	25	0	0	0	0	66
10/W-X	C	7.58	3	8	12	18	25	0	0	0	0	66
6/E-G	D	2.29	1	2	4	6	8	0	0	0	0	20
6/V-X	D	2.29	1	2	4	6	8	0	0	0	0	20
		31.25	13	31	50	75	104	72	86	104	114	650

*Detail B for 1st-8th floor, then Detail A up to roof

Wind Load Distribution on Braced Frames

Windward load only

East-West Direction - Total Load: 577k

Approximate Load on Each Frame Story, kips

Frame	Detail	1/Defl	2	3	4	5	6	7	8	9	penthouse/roof	Total Load
J/16-17	C	7.58	23	24	24	25	26	36	37	38	20	252
T/16-17	C	7.58	23	24	24	25	26	36	37	38	20	252
E/9-10	E	1.14	4	4	4	4	4	0	0	0	0	18
X/9-10	E	1.14	4	4	4	4	4	0	0	0	0	18
E/6-7	E	1.14	4	4	4	4	4	0	0	0	0	18
X/6-7	E	1.14	4	4	4	4	4	0	0	0	0	18
		19.72	61	63	63	65	67	71	73	75	39	577

North-South Direction - Total Load: 665k

Approximate Load on Each Frame Story, kips

Frame	Detail	1/Defl	2	3	4	5	6	7	8	9	penthouse/roof	Total Load
17/J-N	B*	3.76	11	12	12	12	10	17	18	28	15	135
16/P-T	A	7.75	24	24	24	25	20	36	37	28	15	232
10/E-F	C	7.58	23	24	24	24	20	0	0	0	0	114
10/W-X	C	7.58	23	24	24	24	20	0	0	0	0	114
6/E-G	D	2.29	7	7	7	7	6	0	0	0	0	35
6/V-X	D	2.29	7	7	7	7	6	0	0	0	0	35
		31.25	95	97	98	101	81	53	55	56	29	665

*Detail B for 1st-8th floor, then Detail A up to roof

Building Story Shears

RAM Frame v10.0
DataBase: whole building 11/21/06 08:51:53

CRITERIA:

Rigid End Zones:		Ignore Effects	
Member Force Output:		At Face of Joint	
P-Delta:	Yes	Scale Factor:	1.00
Diaphragm:	Rigid		
Ground Level:	Base		

Wall Mesh Criteria :
Max. Allowed Distance between Nodes (ft) : 8.00

Load Case: E1 E EQ_IBC03_X_+E_F

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
penthouse	79.96	79.96	1.14	1.14
9th floor	153.39	73.44	2.37	1.23
8th floor	215.07	61.68	3.60	1.23
7th floor	262.83	47.76	4.00	0.40
6th floor	335.90	73.07	4.95	0.95
5th floor	390.86	54.96	4.90	-0.05
4th floor	428.72	37.86	5.60	0.70
3rd floor	449.60	20.87	4.27	-1.33
2nd floor	459.63	10.03	4.45	0.18
1st floor	452.16	-7.47	0.33	-4.12

Load Case: E2 E EQ_IBC03_X_-E_F

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
penthouse	80.07	80.07	1.20	1.20
9th floor	153.65	73.58	2.47	1.27
8th floor	215.65	62.00	3.75	1.28
7th floor	263.54	47.89	4.20	0.45
6th floor	336.86	73.33	5.21	1.01
5th floor	391.88	55.01	5.20	-0.00
4th floor	429.70	37.83	5.95	0.74
3rd floor	450.62	20.92	4.57	-1.37
2nd floor	460.69	10.07	5.01	0.44
1st floor	452.53	-8.16	0.37	-4.63

Load Case: E3 E EQ_IBC03_Y_+E_F

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
penthouse	1.49	1.49	79.61	79.61
9th floor	3.07	1.58	152.49	72.88
8th floor	4.37	1.30	212.64	60.14
7th floor	5.19	0.83	258.28	45.64
6th floor	5.30	0.11	331.07	72.79

5th floor	5.14	-0.16	384.18	53.11
4th floor	4.79	-0.35	422.61	38.44
3rd floor	3.78	-1.01	442.31	19.70
2nd floor	3.51	-0.27	452.69	10.37
1st floor	-0.26	-3.76	440.70	-11.98

Load Case: E4 E EQ_IBC03_Y_-E_F

Level	**Shear-X kips**	**Change-X kips**	**Shear-Y kips**	**Change-Y kips**
penthouse	1.34	1.34	79.54	79.54
9th floor	2.72	1.38	152.37	72.83
8th floor	3.52	0.80	212.45	60.08
7th floor	4.20	0.68	258.03	45.58
6th floor	4.03	-0.17	330.75	72.72
5th floor	3.93	-0.10	383.80	53.05
4th floor	3.71	-0.22	422.19	38.38
3rd floor	2.69	-1.03	441.95	19.76
2nd floor	2.39	-0.29	452.08	10.13
1st floor	-0.63	-3.03	440.66	-11.42

Periods and Modes

RAM Frame v10.0
DataBase: whole building 11/21/06 08:51:53

CRITERIA:

Rigid End Zones:		Ignore Effects	
P-Delta:	Yes	Scale Factor:	1.00
Diaphragm:	Rigid		
Ground Level:	Base		

Wall Mesh Criteria :
Max. Allowed Distance between Nodes (ft) : 8.00

FREQUENCIES AND PERIODS:

Mode	Period sec	Frequency Hz	Frequency rad/sec
1	3.1223	0.3203	2.0123
2	2.6308	0.3801	2.3883
3	2.2058	0.4534	2.8485
4	1.3255	0.7545	4.7404
5	1.0272	0.9736	6.1171
6	0.8502	1.1761	7.3899
7	0.6377	1.5681	9.8529
8	0.5002	1.9993	12.5619
9	0.4731	2.1137	13.2810

MODAL PARTICIPATION FACTORS:

Mode	X-Dir	Y-Dir	Rotation
1	51.9838	52.4786	357.0028
2	0.3654	-70.1115	345.6549
3	-73.1840	43.3139	86.5708
4	-45.9812	1.3747	571.7614
5	28.0722	52.3396	192.6486
6	-40.3720	26.0931	-356.6051
7	0.7727	-1.6006	-159.5505
8	-12.2401	-25.3984	-140.8801
9	1.8898	15.1248	-207.2883

MODAL DIRECTION FACTORS:

Mode	X-Dir	Y-Dir	Rotation
1	39.21	30.23	30.56
2	0.53	51.68	47.79
3	55.98	17.98	26.04
4	27.69	0.07	72.24
5	21.78	73.49	4.73
6	57.66	26.31	16.02
7	1.10	0.31	98.59
8	19.09	62.00	18.91
9	4.36	23.22	72.42

MODAL EFFECTIVE MASS FACTORS:

Mode	X-Dir		Y-Dir		Rotation	
	%Mass	%SumM	%Mass	%SumM	%Mass	%SumM

Building Story Shears

RAM Frame v10.0
DataBase: whole building 11/19/06 12:32:21

CRITERIA:

Rigid End Zones:		Ignore Effects	
Member Force Output:		At Face of Joint	
P-Delta:	Yes	Scale Factor:	1.00
Diaphragm:	Rigid		
Ground Level:	Base		

Wall Mesh Criteria :
Max. Allowed Distance between Nodes (ft) : 8.00

Load Case: W1 W Wind_IBC03_1_X

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
penthouse	17.73	17.73	0.49	0.49
9th floor	51.61	33.88	0.98	0.49
8th floor	86.15	34.54	1.47	0.49
7th floor	120.14	33.99	1.72	0.25
6th floor	183.34	63.20	2.14	0.42
5th floor	274.01	90.67	2.28	0.13
4th floor	361.54	87.53	2.65	0.38
3rd floor	453.14	91.61	2.18	-0.47
2nd floor	541.18	88.03	2.40	0.22
1st floor	633.90	92.72	0.18	-2.23

Load Case: W2 W Wind_IBC03_1_Y

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
penthouse	1.10	1.10	36.97	36.97
9th floor	2.17	1.07	107.34	70.37
8th floor	2.97	0.80	178.19	70.85
7th floor	3.59	0.62	245.88	67.69
6th floor	3.63	0.04	307.96	62.08
5th floor	3.60	-0.03	367.15	59.19
4th floor	3.41	-0.19	425.19	58.04
3rd floor	2.66	-0.75	477.04	51.85
2nd floor	2.53	-0.13	528.83	51.79
1st floor	-0.36	-2.90	568.96	40.13

Frame Story Shears

RAM Frame v10.0
DataBase: whole building 11/19/06 12:32:21

CRITERIA:

Rigid End Zones:		Ignore Effects	
Member Force Output:		At Face of Joint	
P-Delta:	Yes	Scale Factor:	1.00
Diaphragm:	Rigid		
Ground Level:	Base		

Wall Mesh Criteria :

Max. Allowed Distance between Nodes (ft) : 8.00

Frame #0

Load Case: W1 W Wind_IBC03_1_X

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd floor	11.02	11.02	0.10	0.10
1st floor	38.81	27.79	-0.14	-0.24

Load Case: W2 W Wind_IBC03_1_Y

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd floor	4.35	4.35	0.40	0.40
1st floor	-3.65	-8.00	0.18	-0.22

Frame #1

Load Case: W1 W Wind_IBC03_1_X

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
penthouse	19.59	19.59	3.92	3.92
9th floor	37.32	17.73	4.28	0.36
8th floor	37.58	0.26	-1.75	-6.03
7th floor	55.60	18.02	-1.66	0.09
6th floor	62.29	6.69	-19.50	-17.84
5th floor	58.96	-3.33	-15.32	4.18
4th floor	71.41	12.45	-10.33	5.00
3rd floor	69.10	-2.30	-6.76	3.57
2nd floor	147.65	78.55	-7.64	-0.88
1st floor	186.34	38.69	-4.87	2.77

Load Case: W2 W Wind_IBC03_1_Y

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
penthouse	1.50	1.50	18.71	18.71
9th floor	1.09	-0.41	53.48	34.77

8th floor	1.72	0.64	88.95	35.46
7th floor	1.86	0.13	122.64	33.70
6th floor	-3.35	-5.20	69.36	-53.28
5th floor	-5.32	-1.97	105.84	36.48
4th floor	-2.67	2.65	126.50	20.66
3rd floor	-5.09	-2.42	162.50	36.01
2nd floor	17.74	22.83	98.59	-63.91
1st floor	17.03	-0.71	103.53	4.94

Frame #2

Load Case: W1 W Wind_IBC03_1_X

Level	**Shear-X**	**Change-X**	**Shear-Y**	**Change-Y**
	kips	**kips**	**kips**	**kips**
penthouse	-1.86	-1.86	-3.43	-3.43
9th floor	14.29	16.15	-3.30	0.13
8th floor	48.57	34.28	3.22	6.52
7th floor	64.54	15.97	3.38	0.16
6th floor	80.14	15.59	-5.40	-8.78
5th floor	112.37	32.23	-5.94	-0.54
4th floor	135.69	23.32	-10.72	-4.78
3rd floor	147.84	12.15	-6.49	4.23
2nd floor	144.22	-3.61	3.19	9.69
1st floor	92.09	-52.13	2.01	-1.19

Load Case: W2 W Wind_IBC03_1_Y

Level	**Shear-X**	**Change-X**	**Shear-Y**	**Change-Y**
	kips	**kips**	**kips**	**kips**
penthouse	-0.40	-0.40	18.26	18.26
9th floor	1.09	1.48	53.85	35.59
8th floor	1.25	0.17	89.24	35.39
7th floor	1.73	0.48	123.24	34.00
6th floor	-18.39	-20.13	87.98	-35.26
5th floor	-20.49	-2.09	123.89	35.91
4th floor	-15.30	5.18	136.88	13.00
3rd floor	-14.13	1.17	173.89	37.01
2nd floor	15.34	29.47	114.23	-59.66
1st floor	-6.86	-22.20	99.99	-14.24

Frame #3

Load Case: W1 W Wind_IBC03_1_X

Level	**Shear-X**	**Change-X**	**Shear-Y**	**Change-Y**
	kips	**kips**	**kips**	**kips**

6th floor	11.67	11.67	-4.30	-4.30
5th floor	33.33	21.66	-4.89	-0.59
4th floor	45.28	11.96	-1.90	2.98
3rd floor	75.61	30.32	-1.17	0.74
2nd floor	37.97	-37.64	-16.26	-15.09
1st floor	100.20	62.22	-6.21	10.05

Load Case: W2 W Wind_IBC03_1_Y

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6th floor	7.22	7.22	28.83	28.83
5th floor	1.63	-5.60	29.89	1.06
4th floor	-4.23	-5.86	31.94	2.05
3rd floor	-3.05	1.18	27.62	-4.32
2nd floor	-7.18	-4.12	184.04	156.42
1st floor	-3.68	3.50	92.88	-91.15

Frame #4

Load Case: W1 W Wind_IBC03_1_X

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6th floor	5.95	5.95	21.39	21.39
5th floor	32.20	26.24	22.39	1.00
4th floor	54.83	22.63	16.78	-5.61
3rd floor	98.91	44.07	11.50	-5.29
2nd floor	55.76	-43.15	15.47	3.97
1st floor	70.71	14.95	6.37	-9.10

Load Case: W2 W Wind_IBC03_1_Y

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
6th floor	15.17	15.17	52.42	52.42
5th floor	27.47	12.31	56.91	4.48
4th floor	22.36	-5.11	55.53	-1.37
3rd floor	24.86	2.50	49.34	-6.19
2nd floor	9.62	-15.24	62.15	12.81
1st floor	-1.35	-10.96	72.84	10.69

Frame #5

Load Case: W1 W Wind_IBC03_1_X

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips

6th floor	12.36	12.36	2.10	2.10
5th floor	19.08	6.72	1.33	-0.76
4th floor	27.61	8.53	2.37	1.04
3rd floor	31.15	3.55	1.62	-0.75
2nd floor	72.87	41.72	-0.55	-2.17
1st floor	72.96	0.09	-6.91	-6.36

Load Case: W2 W Wind_IBC03_1_Y

Level	**Shear-X kips**	**Change-X kips**	**Shear-Y kips**	**Change-Y kips**
6th floor	6.48	6.48	31.76	31.76
5th floor	3.80	-2.68	23.35	-8.40
4th floor	4.97	1.18	35.06	11.71
3rd floor	2.45	-2.52	30.76	-4.30
2nd floor	-14.93	-17.39	32.81	2.04
1st floor	-0.93	14.01	93.68	60.87

Frame #6

Load Case: W1 W Wind_IBC03_1_X

Level	**Shear-X kips**	**Change-X kips**	**Shear-Y kips**	**Change-Y kips**
6th floor	10.94	10.94	7.86	7.86
5th floor	18.08	7.15	4.71	-3.15
4th floor	26.72	8.64	6.45	1.74
3rd floor	30.53	3.82	3.49	-2.96
2nd floor	71.69	41.15	8.09	4.60
1st floor	72.79	1.11	9.93	1.84

Load Case: W2 W Wind_IBC03_1_Y

Level	**Shear-X kips**	**Change-X kips**	**Shear-Y kips**	**Change-Y kips**
6th floor	-3.50	-3.50	37.61	37.61
5th floor	-3.50	0.00	27.27	-10.33
4th floor	-1.72	1.78	39.28	12.01
3rd floor	-2.37	-0.65	32.92	-6.36
2nd floor	-22.41	-20.03	36.61	3.69
1st floor	-0.93	21.48	105.85	69.24

Story Displacements

RAM Frame v10.0
DataBase: whole building
Building Code: IBC

11/20/06 23:03:46

CRITERIA:

Rigid End Zones:		Ignore Effects	
Member Force Output:		At Face of Joint	
P-Delta:	Yes	Scale Factor:	1.00
Diaphragm:	Rigid		
Ground Level:	Base		

Wall Mesh Criteria :

Max. Allowed Distance between Nodes (ft) : 8.00

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
W1	W	Wind_IBC03_1_X
W2	W	Wind_IBC03_1_Y
W3	W	Wind_IBC03_2_X+E
W4	W	Wind_IBC03_2_X-E
W5	W	Wind_IBC03_2_Y+E
W6	W	Wind_IBC03_2_Y-E
W7	W	Wind_IBC03_3_X+Y
W8	W	Wind_IBC03_3_X-Y
W9	W	Wind_IBC03_4_X+Y_CW
W10	W	Wind_IBC03_4_X+Y_CCW
W11	W	Wind_IBC03_4_X-Y_CW
W12	W	Wind_IBC03_4_X-Y_CCW
E1	E	EQ_IBC03_X_+E_F
E2	E	EQ_IBC03_X_-E_F
E3	E	EQ_IBC03_Y_+E_F
E4	E	EQ_IBC03_Y_-E_F

Level: penthouse

Center of Mass (ft): (127.36, 142.84)

LdC	**Disp X**	**Disp Y**	**Theta Z**
	in	**in**	**rad**
D	0.00811	0.00891	0.00000
Lp	0.02743	0.01860	-0.00001
W1	1.19451	0.28283	0.00019
W2	0.56140	2.06381	0.00012
W3	0.73015	0.18095	-0.00019
W4	1.06161	0.24330	0.00048
W5	0.61746	1.58566	0.00089
W6	0.22464	1.51006	-0.00071
W7	1.31693	1.75998	0.00024
W8	0.47483	-1.33574	0.00005
W9	0.71609	1.26825	-0.00067
W10	1.25930	1.37172	0.00103
W11	0.08451	-1.05354	-0.00081

W12	0.62773	-0.95007	0.00089
E1	1.85247	0.50865	0.00024
E2	1.99131	0.53606	0.00074
E3	0.61090	1.93061	0.00049
E4	0.45501	1.89967	-0.00027

Level: 9th floor

Center of Mass (ft): (127.36, 142.84)

LdC	Disp X in	Disp Y in	Theta Z rad
D	0.00671	0.00851	0.00001
Lp	0.02232	0.01953	0.00000
W1	1.09582	0.23908	0.00019
W2	0.46287	1.82486	0.00011
W3	0.66474	0.15185	-0.00016
W4	0.97898	0.20676	0.00043
W5	0.53152	1.40144	0.00076
W6	0.16279	1.33584	-0.00059
W7	1.16902	1.54795	0.00022
W8	0.47471	-1.18934	0.00006
W9	0.62065	1.11577	-0.00056
W10	1.13288	1.20615	0.00089
W11	0.09992	-0.93719	-0.00068
W12	0.61215	-0.84681	0.00077
E1	1.65098	0.42332	0.00025
E2	1.78106	0.44682	0.00067
E3	0.50704	1.67571	0.00041
E4	0.36169	1.64951	-0.00021

Level: 8th floor

Center of Mass (ft): (127.36, 142.84)

LdC	Disp X in	Disp Y in	Theta Z rad
D	0.00604	0.00700	0.00000
Lp	0.01975	0.01609	0.00000
W1	0.98864	0.19594	0.00017
W2	0.36708	1.57003	0.00010
W3	0.59325	0.12321	-0.00012
W4	0.88971	0.17070	0.00038
W5	0.44655	1.20545	0.00059
W6	0.10407	1.14960	-0.00044
W7	1.01679	1.32448	0.00020
W8	0.46617	-1.03057	0.00006
W9	0.52299	0.95460	-0.00042
W10	1.00219	1.03211	0.00072

Story Displacements

RAM Frame v10.0
DataBase: whole building
Building Code: IBC

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CRITICAL:

Rigid End Zones:		Ignore Effects	
Member Force Output:		At Face of Joint	
P-Delta:	Yes	Scale Factor:	1.00
Diaphragm:	Rigid		
Ground Level:	Base		

Wall Mesh Criteria :

Max. Allowed Distance between Nodes (ft) : 8.00

LOAD CASE DEFINITIONS:

W2	W	Wind_IBC03_1_Y
E2	E	EQ_IBC03_X_-E_F

Level: penthouse

Center of Mass (ft): (127.36, 142.83)

LdC	**Disp X**	**Disp Y**	**Theta Z**
	in	**in**	**rad**
W2	0.57374	2.09343	0.00013
E2	2.15866	0.58583	0.00082

Level: 9th floor

Center of Mass (ft): (127.36, 142.83)

LdC	**Disp X**	**Disp Y**	**Theta Z**
	in	**in**	**rad**
W2	0.47336	1.85094	0.00011
E2	1.93179	0.48863	0.00074

Level: 8th floor

Center of Mass (ft): (127.36, 142.83)

LdC	**Disp X**	**Disp Y**	**Theta Z**
	in	**in**	**rad**
W2	0.37568	1.59235	0.00010
E2	1.68910	0.39427	0.00064

Level: 7th floor

Center of Mass (ft): (127.36, 142.83)

LdC	**Disp X**	**Disp Y**	**Theta Z**
	in	**in**	**rad**
W2	0.28041	1.30464	0.00009
E2	1.38572	0.29380	0.00051

Level: 6th floor

Center of Mass (ft): (127.35, 222.54)

LdC	**Disp X**	**Disp Y**	**Theta Z**
	in	**in**	**rad**

W2	0.12396	1.04873	0.00008
E2	0.74049	0.21528	0.00039

Level: 5th floor

Center of Mass (ft): (127.35, 222.54)

LdC	Disp X in	Disp Y in	Theta Z rad
W2	0.08381	0.82502	0.00006
E2	0.57560	0.15147	0.00030

Level: 4th floor

Center of Mass (ft): (127.35, 222.54)

LdC	Disp X in	Disp Y in	Theta Z rad
W2	0.05274	0.62076	0.00004
E2	0.41859	0.10403	0.00021

Level: 3rd floor

Center of Mass (ft): (127.35, 222.54)

LdC	Disp X in	Disp Y in	Theta Z rad
W2	0.02767	0.42117	0.00003
E2	0.27137	0.06087	0.00014

Level: 2nd floor

Center of Mass (ft): (128.53, 196.07)

LdC	Disp X in	Disp Y in	Theta Z rad
W2	0.01688	0.25059	0.00002
E2	0.17590	0.03427	0.00007

Level: 1st floor

Center of Mass (ft): (128.53, 196.34)

LdC	Disp X in	Disp Y in	Theta Z rad
W2	-0.00235	0.07387	-0.00000
E2	0.04831	0.00221	0.00002