Technical Report #3

Administration Building

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Structural

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Pennsylvania

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EXECUTIVE SUMMARY

The purpose of this report is to perform a detailed analysis of the lateral system for the Administration Building. This includes explaining the lateral system, determining the combination of lateral loads that govern the lateral design, finding how the lateral loads will be distributed to the braced frames, lateral strength check, drift, overturning moment and their impact on the foundations, and overall building torsion.

RAM Structural System was used to model the Administration Building and its lateral braced frames. A computer model is an easy way to compute a rather complicated calculation like drift. RAM was used to compare the hand calculated loads against RAM's calculated lateral loads; both methods were still used in the analysis.

The hand calculated wind loads were 830 kips in the long direction and 271 kips in the short direction. RAM calculated 660 kips in the long direction and 192 kips in the short direction. The hand calculated seismic load was 566 kips compared to 344 kips calculated by RAM. The hand calculated values were higher than the RAM calculated values due to being too conservative on the hand calculations.

Assuming the composite metal deck and slab act as a rigid diaphragm, the lateral loads will be distributed due to relative stiffness. Due to stiffness, the individual braced frames roughly take 17% of the lateral force in each direction. Refer to page 13 for a more detailed distribution breakdown.

There is a 42' eccentricity in the long direction and a 10' eccentricity in the short direction. With an eccentricity, it creates torsion in the building. Since the eccentricity is larger than the accidental 5% eccentricity that RAM Structural System assumes, torsion should be calculated. However, the torsion is 46 kips on the braced frames in both directions, which will not create a problem. So, torsion can be ignored in this case.

The total drift of the building is limited to H/400 for serviceability issues of the occupants in the building. The actual building height is 87' but the first floor is below grade, making the real building height 67'. This is a conservative approach, which will limit the total building drift of H/400 = 2". The maximum building drift is 0.53" in the long direction and 0.57" in the short direction, making them both under the allowed serviceability criteria.

Foundation design was also considered in this report, as the footings under the braced frames will have to resist the gravity loads in addition to the lateral loads. The overall overturning moments that the administration building must resist is 53,051 K-FT in the long direction and 17, 295 K-FT in the short direction.

Finally, a strength check was performed on a braced frame 11 which goes the height of the building. The controlling load combination was 1.2D + 0.5L + 1.6W, where a majority of the members were stressed below 59% of their total strength. The hand calculations agreed with the analysis results that RAM Frame provided for braced frame 11.

STRUCTURAL SYSTEM OVERVIEW:

BUIDING INFORMATION:

This is an administration building for a confidential client in Pennsylvania that was constructed in July 2003. It offers offices and specialty amenity spaces as the architectural layout of 311,905 S.F. of usable floor area. There are five floors, four of which are above grade with a cost ranging between $70-75 million.

FOUNDATION:

The foundation system will consist of reinforced concrete spread footings that are sized utilizing bearing capacities ranging from 4,000 psf at soil bearing footings and 15,000 psf at rock-bearing footings. Total building settlements will be less than 1" with differential settlements not exceeding ½" or 1/300, based on a 20' bay. Typical perimeter frost walls are supported on continuous reinforced concrete strip footings. Foundation walls at basement or below grade levels are reinforced concrete basement walls designed for soil lateral loads and appropriate surcharge loads and supported by continuous reinforced concrete strip footings. These walls are drained on the soil side to minimize lateral surcharge loads on the walls and buildings. The slab on grade varies between a 5", 6" and 8" thickness, typically with 6x6-W4.0xW4.0 W.W.F.

FLOOR SYSTEM:

The structural floor system is 3¼" concrete slab on a 3", 20 gauge composite metal deck, totaling 6¼". The metal deck utilizes ¾" steel studs, supported by wide-flange beams and wide-flange columns. The typical sizes of the beams range from W18x40 to W30x116. The girders range from W21x50 to W27x146. The columns range from W10x43 to W14x211. The concrete is lightweight weight (115 pcf), cast-in-place concrete and will have a 28 day strength of 4,000 psi. The concrete slab is reinforced with 6x6-W2.9xW2.9 W.W.F. to minimize plastic shrinkage cracking. The thickness of the concrete is established based on the required 2 hour fire rating for the floor construction without spray fireproofing applied to the underside of the metal deck. Structural steel shall comply with ASTM A572, Grade 50. Steel stud shear connectors shall conform to ASTM A108.

To maintain the 5'-0" building module within the typical bay sizes of 20'-0" and 40'-0", the typical beams supporting the composite slab are spaced at 10'-0" on center. These beams supporting the composite slab for the typical bays span to girders oriented across the width of the building. The wide flange steel girders in the long direction or the building support the wide flange steel beams that span the 3 bay width of the building consisting of (1) 20'-0" and (2) 40'-0" bays. Spanning the beams across the width of the building works best in combination with a braced frame lateral load resisting system.

ROOF SYSTEM:

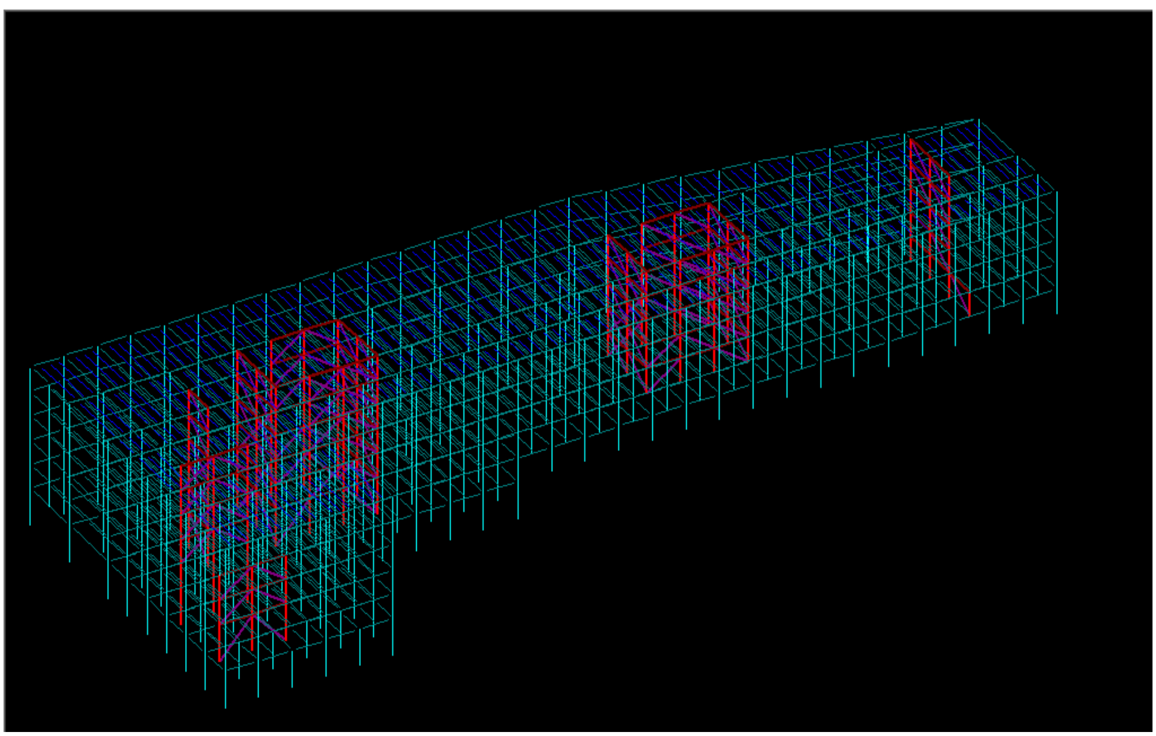
The structural roof system consists of a 1½", 20 gauge, Type B, galvanized metal roof deck with spray fireproofing. Below mechanical equipment a concrete slab on composite metal deck is used instead of the standard roof deck and the concrete slab is reinforced with 6x6-W2.9xW2.9 W.W.F. to minimize shrinkage cracking. The framing members supporting the metal deck are either open-web joists or wide flange steel beams at 4'-0" and 5'-0" centers. The beams supporting the composite slab are wide flange steel beams at 10'-0" centers that span the width of the building.

LATERAL SYSTEM:

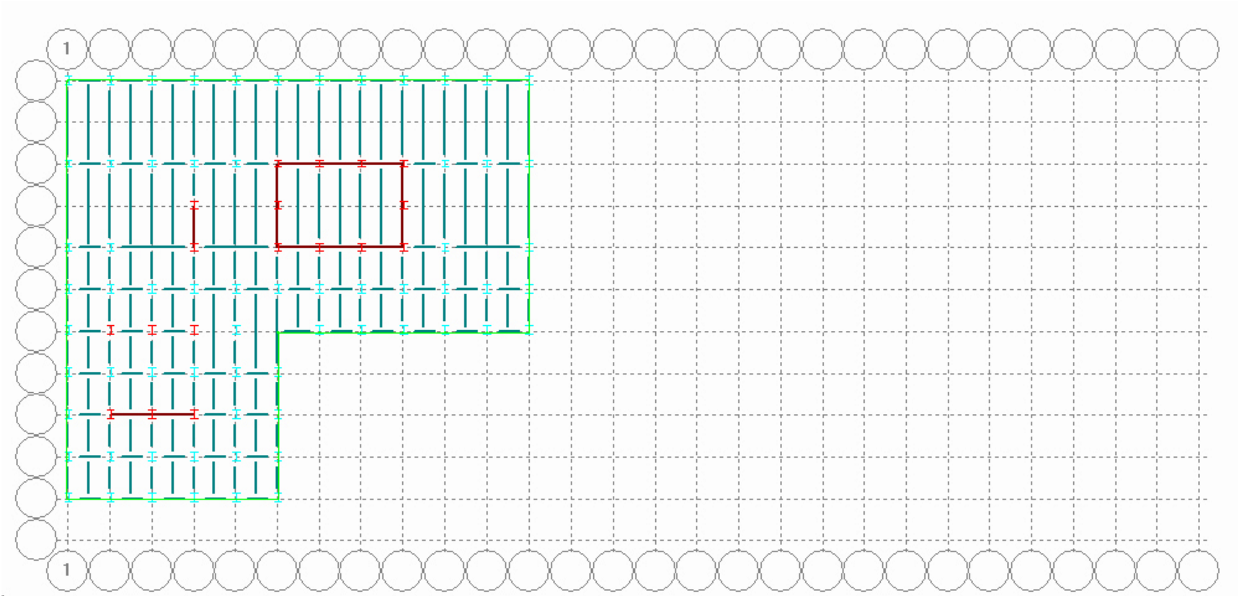
The typical composite steel-framed building utilizes a braced frame lateral load resisting system. The braced frames have been coordinated, located and configured to integrate with the architectural layout and mechanical distribution. These frames consist of wide flange columns, wide flange beams at each story and one HSS (hollow structural section) diagonal braces between each story. Typically the HSS braces will be HSS8x6x1/2.

EXTERIOR WALL SYSTEM:

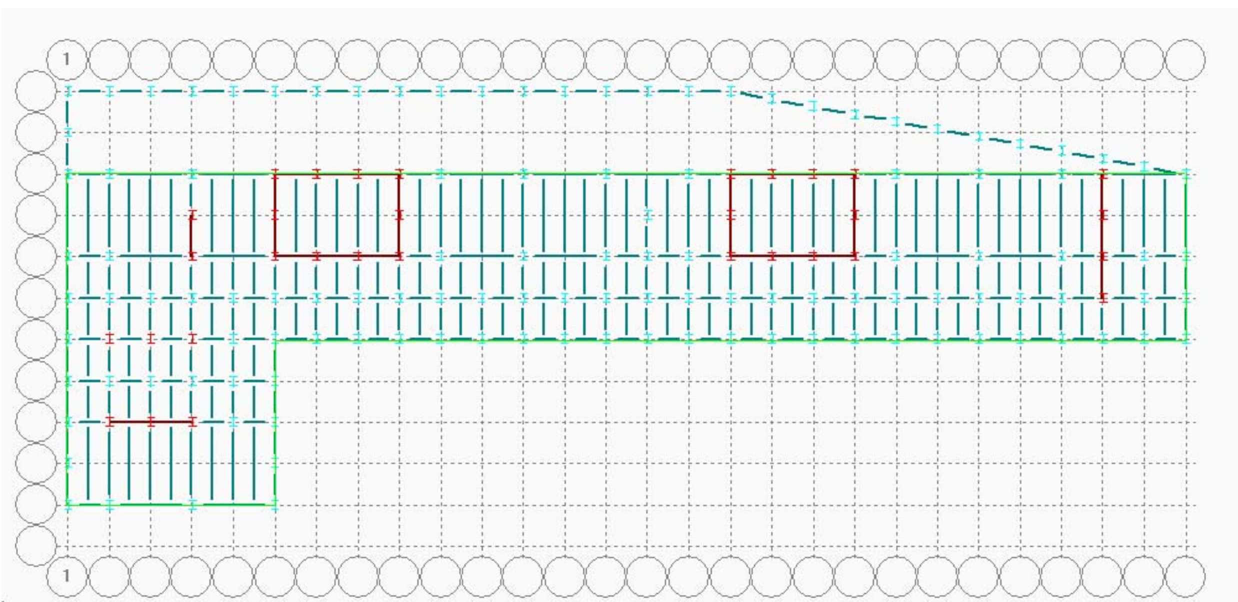
Pre-fabricated brick truss panel assemblies comprised of structural tube and stud infill, steel relieving lintels, and shop-applied exterior brick face. There was a nine-month lead-time for brick materials. This system is independent of the floor and roof framing thus enabling smaller spandrel beam sizes.



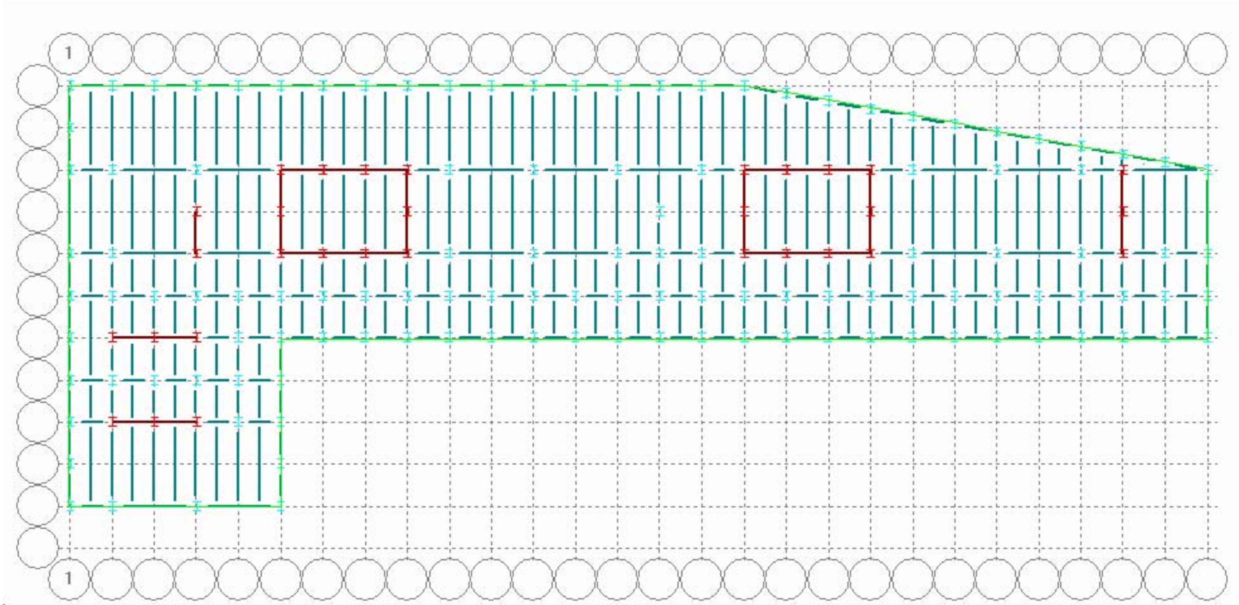
FIRST FLOOR FRAMING PLAN:



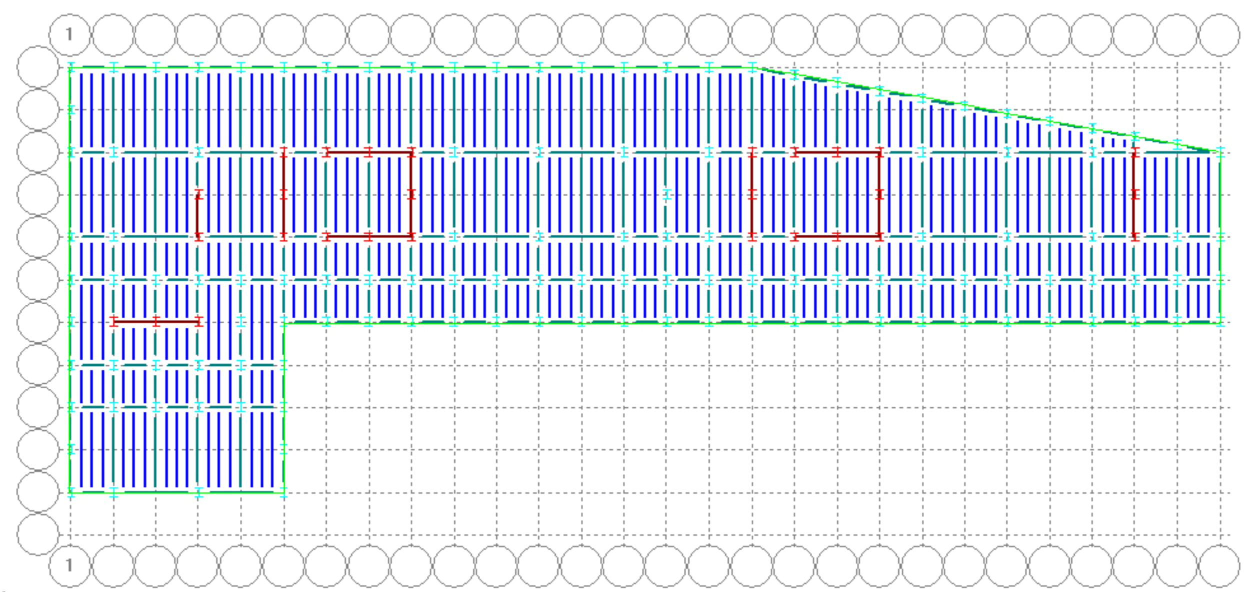
SECOND FLOOR FRAMING PLAN:



THIRD-FIFTH FLOOR FRAMING PLAN:



ROOF FRAMING PLAN:



Red indicates braced frame
Blue indicates open-web joists
Dark green indicates composite beams
Light green indicates columns

LOADS

The administration building's gravity loads are shown below based on live load, dead load and snow load. The live load lists all the applicable areas inside the building and using 100 PSF as the standard floor live load. The floor dead load is found by the concrete slab, superimposed dead load, steel structure/deck and the façade which only applies to the edge beams. The design snow loads are given for easy reference. All these loads were used to design the building.

FLOOR LIVE LOAD:

ROOM	MIN DESIGN LOAD (PSF) ASCE7-05	DESIGN LOAD
Fitness Center	100	100
Lobbies	100	100
Stairs and Exits	100	100
Offices	50	100
Dining Room	100	100
Mechanical Rooms	N/A	150
Corridors	100-FIRST FLOOR 80-ALL OTHER FLOORS	100
Roof	20	30

FLOOR DEAD LOAD:

ITEM	DESIGN LOAD
CONCRETE SLAB	35 PSF
SUPERIMPOSED DEAD LOAD	30 PSF
STEEL STRUCTURE + DECK	15 PSF
EXTERIOR BRICK TRUSS PANEL	40 PSF

ROOF SNOW LOAD:

ITEM	DESIGN VALUE	CODE BASIS
ROOF LIVE LOAD	30 PSF	ASCE7-05
GROUND SNOW LOAD (Pg)	30 PSF	ASCE7-05
FLAT ROOF SNOW LOAD (Pf)	24 PSF	ASCE7-05
SNOW EXPOSURE FACTOR (Ce)	0.9	ASCE7-05
SNOW IMPORTANCE FACTOR (I)	1.2	ASCE7-05

WIND ANALYSIS

The Administration Building is located in Pennsylvania, where wind is the controlling factor in the lateral system. Since wind is the controlling factor, a very detailed wind analysis should be preformed. To perform the wind analysis, a Main Wind Force Resisting System analysis was the prescribed method.

To start the analysis, the building was simplified to make for easier calculations. The next step is to determine the wind coefficients, which can be found on page 21. Following the designer's assumptions, an importance factor of 1.15 was chosen.

After all the coefficients were determined, the windward and leeward wall pressures can be found. The roof uplift pressure is not going to be an issue being the administration building is a flat roof with mechanical equipment on it, so it is not going to be moving anytime soon. The side-wall pressures do not control and are very small, so they can be ignored. Also the side-wall pressures only really matter in components and cladding analysis, using a MWFRS, it can be ignored.

The windward and leeward building pressures occur in the same direction and can be added together when discussing base shear. Using a wind speed of 90 mph, the base shear in the long direction is 830 kips. The building's base shear in the short direction is 271 kips. The huge difference in base shear between short and long direction is due to the long direction being 300' longer than the short direction. The long direction has a significantly bigger area to resist the wind. Refer to page 25 for the wind loading diagrams.

Hand calculations are a great tool to compare to computer calculated values in RAM Structural System. In the long direction, RAM calculated a base shear for the building of 660 kips and 192 kips in the short direction. There is a significant difference between the hand calculated base shear and the RAM calculated base shear. This is accounted for a too conservative hand calculated value which will make the base shear higher. Refer to page 34 for the RAM Structural System base shear calculated values.

WIND PRESSURES:

WINDWARD WALL PRESSURE – M.W.F.R.S.

HEIGHT(FT)	Kz	qz	P(SHORT DIRECTION,PSF)	P(LONG DIRECTION,PSF)
0-15	0.85	17.255	18.1	19.3
15-20	0.9	18.27	18.9	20.1
20-25	0.94	19.082	19.6	20.9
25-30	0.98	19.894	20.3	21.6
30-40	1.04	21.112	21.2	22.6
40-50	1.09	22.127	22.1	23.5
50-60	1.13	22.939	22.7	24.2
60-70	1.17	23.751	23.4	24.9

LEEWARD WALL PRESSURE – M.W.F.R.S.

DIRECTION	PRESSURE (PSF)
LONG	-9.4
SHORT	-15

SIDEWALL PRESSURE – M.W.F.R.S.

DIRECTION	PRESSURE (PSF)
LONG	-19.3
SHORT	-18.1

SEISMIC ANALYSIS

In Pennsylvania, wind is the controlling factor and seismic is not too big of an issue. However, there is a stricter take on seismic in the new codes and seismic has to be considered for almost every new building in the United States. For the seismic analysis, the equivalent lateral force method was used in the hand calculations.

The seismic coefficients were determined by the design professional in the construction documents. Following the design professional's assumptions, the analysis will be easier to compare to theirs. The design professional chose a response modification coefficient (R) of 5, over strength factor of 2, deflection amplification factor of 4.5, an importance factor of 1.25 which leads to an occupancy category of 3, and seismic design category B. The other seismic coefficients can be found below.

Seismic analysis deals primarily with the weight of the building, meaning dead load only. However, there are code provisions to include a portion of the live load. Using a conservative dead load of 100 PSF, this includes the exterior brick truss panel of 40 PSF. After performing the seismic analysis, a base shear of 566 kips was determined. The seismic load distribution can be found on 12.

Using RAM Structural System calculated seismic loads as a comparison to the hand calculated loads. RAM calculated a base shear of 344 kips, which is much lower than the hand calculated values. However, seismic does not control, so it is not that big of an issue. RAM calculated base shear can be found on page 37.

ITEM	DESIGN VALUE
SITE CLASS	C
SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS (Ss)	0.328
SPECTRAL RESPONSE ACCELERATION AT PERIOD OF 1s (S1)	0.008
SHORT PERIOD SITE COEFFICIENT (Fa)	1.2
LONG PERIOD SITE COEFFICIENT (Fv)	1.7
DAMPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS (Sds)	0.26
DAMPED SPECTRAL RESPONSE ACCELERATION AT PERIOD OF 1s (Sd1)	0.0091
SEISMIC RESISTING SYSTEM	CONCETRICALLY BRACED FRAMES
RESPONSE MODIFICATION COEFFICIENT, (R)	5
OVERSTRENGTH FACTOR	2
DEFLECTION AMPLICATION FACTOR	4.5
IMPORTANCE FACTOR	1.25
OCCUPANCY CATEGORY	3
SEISMIC DESIGN CATEGORY	B
BASE SHEAR	566 (K)

BASE SHEAR:

FLOOR	DEAD LOAD (PSF)	WALL DEAD LOAD (PSF)	FLOOR AREA (SF)	WALL AREA	W(k)	Cs	V=CsW
1	100	40	50000	0	5000	0	50
2	100	40	113680	10507	11788.28	0	117.88
3	100	40	113680	21014	12208.56	0	122.09
4	100	40	113680	21014	12208.56	0	122.09
5	100	40	113680	21014	12208.56	0	122.09
ROOF	24	40	113680	10507	3148.6	0	31.486
TOTAL					**56562.56**		**565.63**

SEISMIC LOAD DISTRIBUTION:

FLOOR	W(k)	hx(FT)	Hx^k(Wx)	Cvx	Fx=CvxV	Mx=hx*Fx (K-FT)
1	5000	20	2000000	0.01142	6.463919	129.2783873
2	11788	33.33	13095158.4	0.074776	42.32302	1410.626385
3	12209	46.67	26592287.4	0.151847	85.9452	4011.062517
4	12209	60	43952400	0.250976	142.0524	8523.143088
5	12209	73.33	65651320.2	0.374881	212.1824	15559.33686
ROOF	3149	87	23834781	0.136101	77.03305	6701.87546
TOTAL	**56563**		**175125947**	**1**	**566**	**36335.3227**

LATERAL FORCE DISTRIBUTION

The typical composite steel-framed building utilizes a braced frame lateral load resisting system. The braced frames have been coordinated, located and configured to integrate with the architectural layout and mechanical distribution. These frames consist of wide flange columns, wide flange beams at each story and one HSS (hollow structural section) diagonal braces between each story.

The lateral system was analyzed using RAM Structural System. The frames in the RAM model represent the exact locations and sizes of the frames designed in the building. For simplification, we are assuming that the lateral braced frames take the entire lateral load, so an analysis of the frames is very important.

Using the calculated wind loads on page 25, RAM Structural System was able to determine their effects on the building. With the calculated wind base shear of 830 kips in the long direction and 271 kips in the short direction being higher than the RAM calculated wind loads, they are going to control.

Using the assumption that all floors act as a rigid diaphragm and the forces are assumed to be distributed by stiffness. To find stiffness, you take the inverse of the deflection of the braced frames. Having found the stiffness, you can make an accurate assumption as to how the braced frames take the lateral load. Refer to the chart below to see how the loads are distributed to the braced frames. The braced frames in the long direction, all take the same amount of load, which is 17% of the total lateral load. The long direction is where the wind is the highest, so this makes sense as to have uniform stiffness along the long direction. The braced frames in the short direction do not act as uniformly as the braced frames in the long direction. This is probably due to the building being longer in the front causing a larger surface area for the wind and shorter in the back of the building.

Long Direction			
Frame	**Deflection (")**	**1/Deflection (1/")**	**Distribution (%)**
BF-11	0.53	1.90	16.67
BF-12	0.53	1.90	16.67
BF-12	0.53	1.90	16.67
BF-14	0.53	1.90	16.67
BF-14	0.53	1.90	16.67
BF-16	0.53	1.90	16.67
Total	3.15	11.41	100.01
Short Direction			
Frame	**Deflection (")**	**1/Deflection (1/")**	**Distribution (%)**
BF-12	0.57	1.76	15.51
BF-13	0.48	2.07	18.27
BF-13	0.53	1.90	16.76
BF-15	0.48	2.07	18.27
BF-15	0.53	1.90	16.76
BF-17	0.61	1.63	14.41
Total	3.20	11.34	99.98

TORSION

Building torsion occurs when the center of mass and center of rigidity do not coincide at the same point. In the administration building, the center of mass and center of rigidity are not located at the same point, which means there is torsion. There is a 42' eccentricity in the X-direction and a 10' eccentricity in the Y-direction causing a torsional force into the rigid diaphragm at each story. Refer to the Center of Mass and Center of Rigidity spreadsheets below for the exact location at each story. The Center of Mass and Center of Rigidity were calculated by RAM Structural System.

Center of Mass					
Story	**Weight (K)**	**Mass (K-S^2/FT)**	**Inertia (FT-F-S^2)**	**Xm (FT)**	**Ym (FT)**
Roof	3955.4	122.84	3295846	231.74	145.36
5	4709.9	146.27	3922095	231.57	145.58
4	4716.1	146.46	3926596	231.52	145.59
3	4728.8	146.86	3938151	231.48	145.58
2	3562.5	110.64	3158541	235.54	127.87
1	2424.5	75.29	514490	96.37	137.25

Center of Rigidity				
Story	**Xr (FT)**	**Yr (FT)**	**Eccentricity X (FT)**	**Eccentricity Y (FT)**
Roof	273.32	151.31	27.05	10.05
5	271.27	150.36	27.05	10.05
4	271.42	148.45	27.05	10.05
3	282.05	144.35	27.05	10.05
2	303.02	148.04	27.05	10.05
1	119.75	121.63	11.05	10.05

The actual eccentricity which is measured from the geometrical center of the building is somewhat higher than the 5% accidental eccentricity that RAM Structural System assumes. The eccentricity used is 5% of the total building dimension. This is a conservative measure, but the actual eccentricity is 42' in the X-direction and 10' in the Y-direction. Since the eccentricity is larger than 5% of the total building dimension, torsion should be calculated.

As stated above, the difference in the location of the center of mass and center of rigidity will introduce torsion into the structure. After calculating the torsion, it became clear that torsion is relatively small in comparison to the wind. Even though it was relatively small, it was still a good idea to calculate the torsion. The frames in the long direction and short direction took the torsion relatively equal to each other. This is probably due to the fact that there are just about the same number of frames in both directions. Also they are almost located in equal length from the center of mass and center of rigidity. 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. The torsion calculations frame by frame and story by story can be found below.

Long Direction											
Frame	**1/Deflection**	**2**	**2T**	**3**	**3T**	**4**	**4T**	**5**	**5T**	**ROOF**	**ROOF T**
BF-11	1.90	101.00	2.38	69.00	1.63	50.00	1.18	36.00	0.85	16.00	0.38
BF-12	1.90	115.00	2.20	94.00	1.80	126.00	2.41	0.00	0.00	0.00	0.00
BF-12	1.90	115.00	1.44	94.00	1.17	126.00	1.57	0.00	0.00	0.00	0.00
BF-14	1.90	468.00	2.46	286.00	1.50	215.00	1.13	104.00	0.55	28.00	0.15
BF-14	1.90	468.00	5.59	286.00	3.42	215.00	2.57	104.00	1.24	28.00	0.33
BF-16	1.90	149.00	3.76	98.00	2.47	65.00	1.64	61.00	1.54	26.00	0.66
Total	11.41	1416.00	17.84	927.00	12.00	797.00	10.51	305.00	4.18	98.00	46.04

Short Direction											
Frame	**1/Deflection**	**2**	**2T**	**3**	**3T**	**4**	**4T**	**5**	**5T**	**ROOF**	**ROOF T**
BF-12	1.76	115.00	2.50	94.00	2.05	126.00	2.74	0.00	0.00	0.00	0.00
BF-13	2.07	118.00	1.61	220.00	3.00	247.00	3.36	139.00	1.89	40.00	0.54
BF-13	1.90	118.00	1.47	220.00	2.75	247.00	3.09	139.00	1.74	40.00	0.50
BF-15	2.07	556.00	4.54	239.00	1.95	168.00	1.37	100.00	0.82	43.00	0.35
BF-15	1.90	556.00	4.17	239.00	1.79	168.00	1.26	100.00	0.75	43.00	0.32
BF-17	1.63	0.00	0.00	31.00	0.42	45.00	0.61	42.00	0.57	13.00	0.18
Total	11.34	1463.00	14.29	1043.00	11.95	1001.00	12.44	520.00	5.77	179.00	46.34

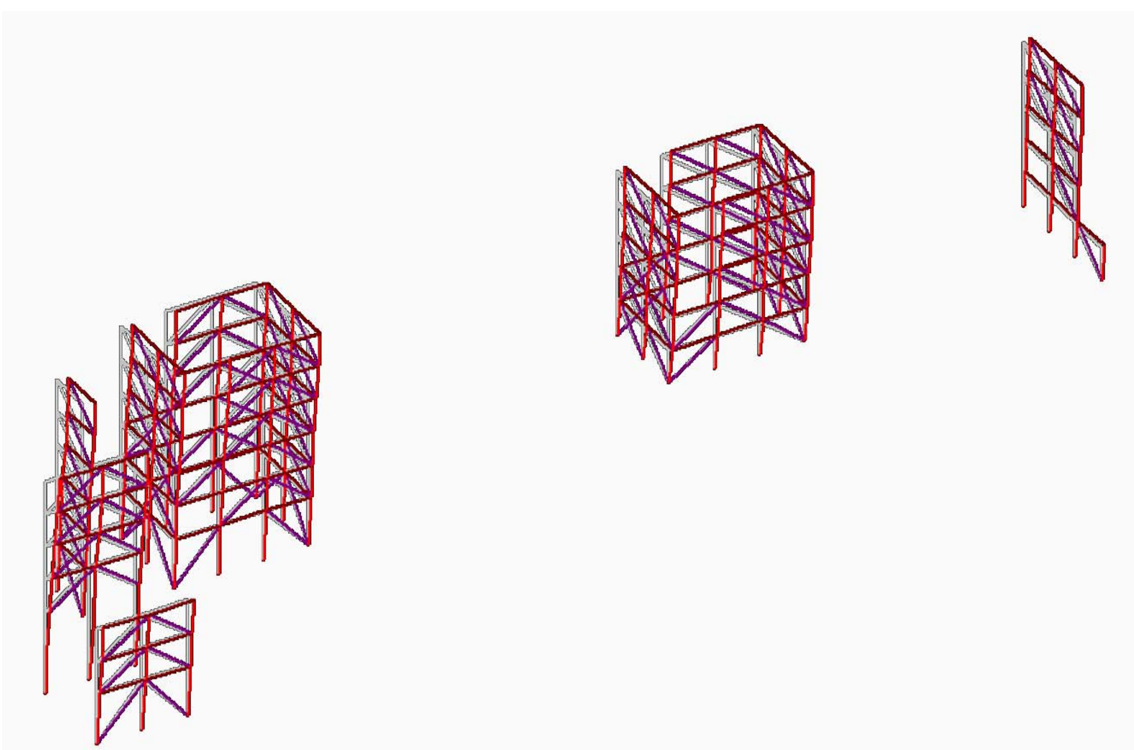
DRIFT

Based on serviceability and comfort levels, the industry accepted standard for the amount of drift a building is allowed to experience is H/400. The administration building is 87' from basement to top of roof, which makes the industry accepted standard of H/400 = 2.61". However, the 1st floor is below grade, making the height of the building above grade of 67'. That would make the allowed drift of H/400 = 2". The more conservative allowable drift of 2" is going to be used.

The drift limitation is solely based on serviceability and comfort levels of the occupants inside the building. Most of the time, serviceability levels are what controls the design. Strength is usually more than enough, but it might make the occupants feel unsafe and that is where the serviceability constraints come into play. For the administration building being limited to 2" drift at the roof, the occupants would never feel the building being moved by lateral loads.

Refer to the chart below, which lists the drift values at each floor. The maximum drift that occurs is 0.57", which is significantly under the serviceability limit of H/400 = 2". The occupants in the administration building will be happy and feel safe.

Level	Long Direction Drift	Short Direction Drift
Roof	0.53"	0.57"
5	0.46"	0.50"
4	0.36"	0.39"
3	0.25"	0.27"
2	0.17"	0.18"



OVERTURNING MOMENT

The overall overturning moment was determined by the hand calculated wind loads. Using the wind point loads on each story, this in turn is multiplied by the height above ground level for each story. The overall overturning moment in the long direction was found to be 53,051 K-FT and the overall overturning moment in the short direction was determined to be 17,295 K-FT. Refer to the overturning moment chart below for the overturning moment at each floor and each direction.

Floor	Long Direction OM (K-FT)	Short Direction OM (K-FT)
1	190	62
2	2060	672
3	5856	1909
4	12444	4057
5	22716	7405
Roof	9785	3190
Total	**53051**	**17295**

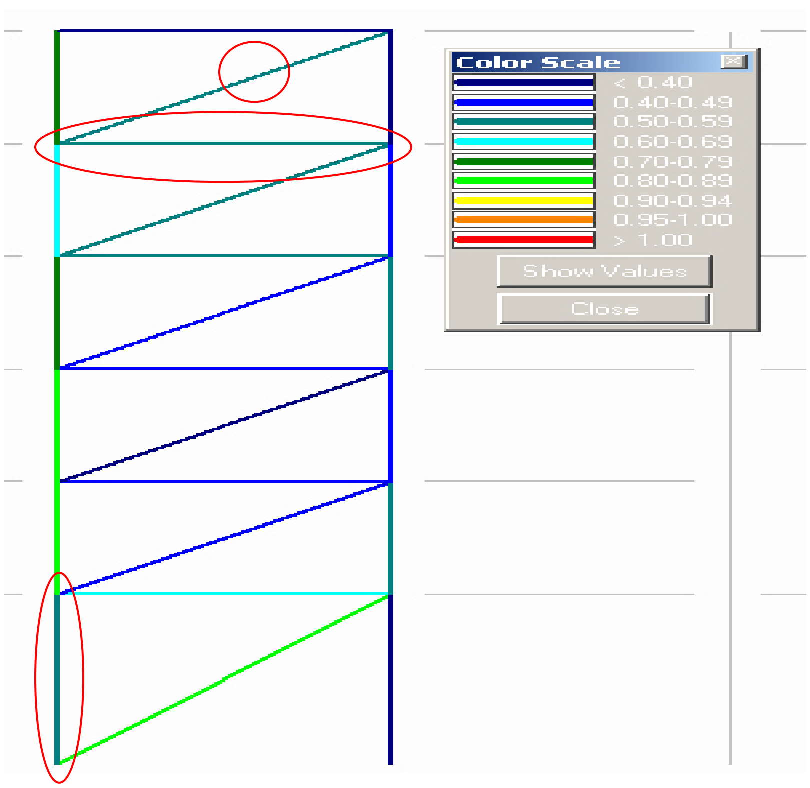
The foundation consists of reinforced concrete spread footings utilizing bearing capacities of 15,000 PSF at rock-bearing footings. The footings are significantly increased under the lateral columns to resist the higher moments, larger combined axial, and overturning moments onto the spread footings. The loads are converted into axial load by the intermediate members and transferred into the columns. The columns are designed to handle axial compression load much better than bending and the same applies for the foundations.

LATERAL STRENGTH CHECK

A strength check was performed on braced frame 11 located on grid coordinates 48/E-F. This analysis was performed by RAM Frame and double checked by a hand calculation which can be found on page 30. A hand calculation was performed for the members circled in red in the frame below, which is a HSS 6x6x14 brace, W16x26 beam, and a W14x193 column at the base of the frame. All hand checked values agreed with RAM's calculated values which were ample size. Using a computer model allows for easy assessment of the stresses on all the members in the matter of seconds. The code used for the standard provisions check is AISC's LRFD and ASCE 7-02. The load cases included in the check were a combination of dead, live, wind, and earthquake loading. The following load cases were used:

- 1.4D
- 1.2D + 1.6L
- **1.2D + 0.5L + 1.6W**
- 1.2D + 0.5L + 1.6E
- 1.2D + 1.0E

The controlling case was 1.2D + 0.5L + 1.6W, which was used to generate the member forces on each frame. Refer to frame below as an elevation view of braced frame 11. The color scale refers to the percentage of the framing member being stressed. A majority of the framing members are dark green or below which is stressed at a maximum of 59%, which is adequate for this frame.



DIFFERENCES

As mentioned above in the wind and seismic analysis section, the hand calculated values for wind and seismic were much higher than the RAM calculated values. The calculated wind was 830 kips in the long direction and 271 kips in the short direction. Compared to the RAM calculated value of 660 kips in the long direction and 192 kips in the short direction. The calculated seismic was 566 kips compared to RAM calculated values of 344 kips. After analyzing the lateral loads distributed to the braced frames, portions of certain braced frames did not pass the member code check by 10%.

Due to the fact that some structural members of the braced frames failed under lateral load, leads one to believe the calculated values are higher than the designer's calculated lateral loads. Being that a few frame members failed by 10%, one might suggest the hand calculated loads are 10% higher than the designer's calculated load, which is most likely already over sized. This can be fixed by decreasing the wind loads by 10%, and then the lateral system should work fine.

CONCLUSIONS

The following conclusions can be made based on the calculations performed on the lateral system of the Administration Building in Pennsylvania:

● Wind load controls over seismic load in the braced frames. Being that the Administration Building is located in Pennsylvania comes to no surprise that wind load controls over seismic load.

● The braced frames uniformly take 17% of the lateral load which is distributed to each braced frame in both directions of the building which is distributed by the concrete slab acting as a rigid diaphragm.

● The center of mass and center of rigidity are not located at the same location which will induce torsion. However, a torsional force of 46 kips in both directions is too small to make a difference and can be ignored.

● The total drift of the building is limited to H/400 for serviceability issues of the occupants of the building. The actual building height is 87' but the first floor is below grade, making the real building height 67'. This is a conservative approach, which will limit the total building drift of H/400 = 2". The maximum building drift is 0.53" in the long direction and 0.57" in the short direction, making them both under the allowed serviceability criteria.

● The overturning moment in the long direction was found to be 53,051 K-FT and 17, 295 in the short direction. With the bearing capacity of the spread footings being 15,000 PSF at rock, the footings are adequate to carry the overturning moment.

● 1.2D + 0.5L + 1.6W was the controlling load case and a strength check was performed on braced frame 11. The majority of the frame elements were stressed at 59% or below, which is sufficient to carry the lateral loads.

● Some of the frames failed which is due to the hand calculated wind loads being 10% too big.

WIND CALCULATIONS

WIND 1

WIND

- DESIGN LOADS

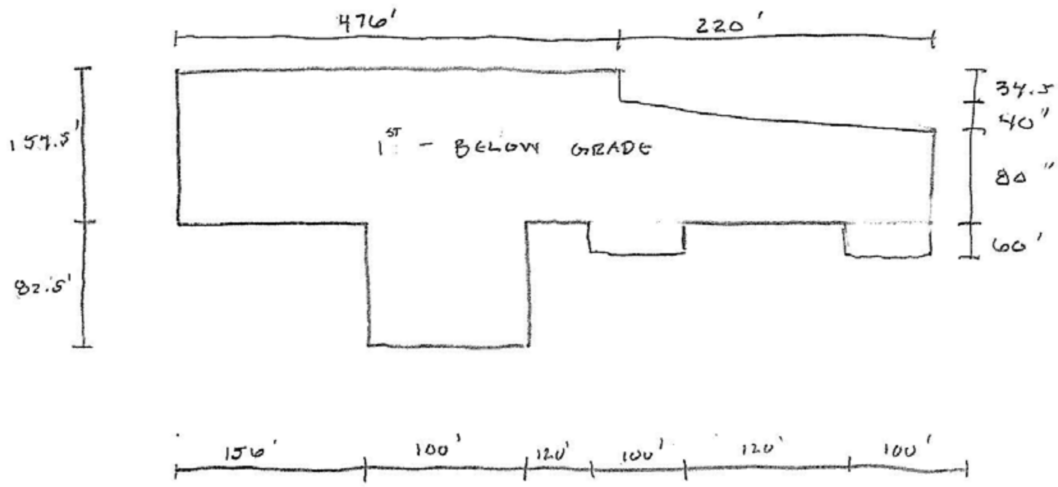
$V = 90$ MPH

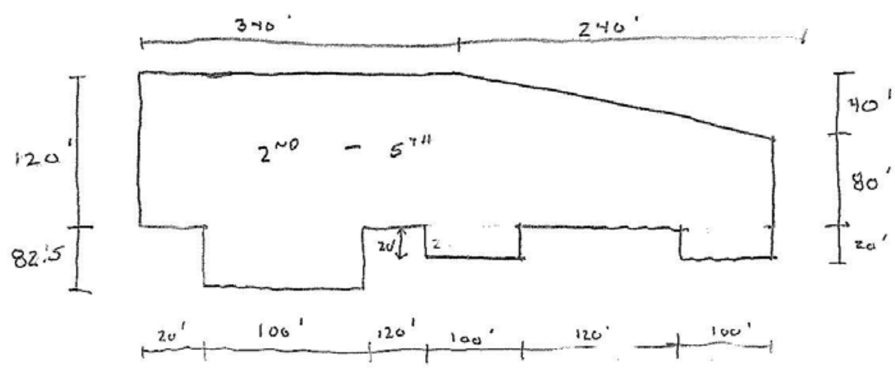
$I_w = 1.15$

WIND EXPOSURE C

MAIN WINDFORCE-RESISTING SYSTEM

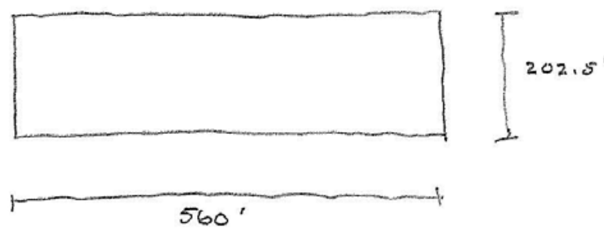
MEAN ROOF HEIGHT = 67' (COMPLETLY ABOVE GRADE)





WIND 2

1.



- $K_{zt} = 1.0 \rightarrow 6.5.7$
- GUST EFFECT FACTOR $G \rightarrow 6.5.8$

- RIGID = FUND. FREQ ≥ 1 Hz

$T_a = C_t h_n^x$

$= 0.028(67')^{0.8} = 0.81\,s \rightarrow 1/T_a = 1.24$ Hz

$\rightarrow$ RIGID

$$G = 0.925\left(\frac{(1 + 1.7 g_Q I_{\bar{z}} Q)}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} \rightarrow \bar{z} = 0.6h \geq z_{min}$

$= 0.6(67') = 40.2 \geq 15'$ ✓

$= 0.2\left(\frac{33}{40.2}\right)^{1/6}$

$= 0.194$

$g_Q = g_v = 3.4$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)}} =$$

$B = 202.5'$

$h = 67'$

$L_{\bar{z}} = L\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500'\left(\frac{40.2'}{33}\right)^{1/5} = 520.13'$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{202.5' + 67'}{520.13'}\right)}} = 0.87$$

$Q = 0.75$ (SHORT DIRECTION)

WIND 3

G = 0.93 (SHORT DIRECTION)

$$G = 0.925\left(\frac{(1 + 1.7(3.4)(0.194)(0.87))}{1 + 1.7(3.4)(0.194)}\right) = 0.86$$

- ENCLOSURE CLASSIFICATION → 6.5.9
  → ENCLOSED

- INTERNAL PRESSURE COEFFICIENT, GC_{pi} → 6.5.11.1

  $GC_{pi} = +0.18$
  $\qquad\;\; -0.18$

- EXT. PRESSURE COEFFICIENTS C_p OR GC_{pf} → 6.5.11.2/3

  - WALL PRESSURE, C_p

    WINDWARD WALL $= 0.8 \rightarrow q_z$
    LEEWARD WALL $= -0.3 \rightarrow q_h$
    SIDE WALL $= -0.7 \rightarrow q_h$

  - ROOF PRESSURES, C_p

    $0 - h/2 = -0.9/-0.18$
    $h/2 - h = -0.9/-0.18$
    $h - 2h = -0.5/-0.18$
    $> 2h = -0.3/-0.18$

· VELOCITY PRESSURE q_z OR q_h → 6.5.10

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$K_d = 0.85$ $I = 1.15$

$K_{zt} = 1.0$

K_z = REFER TO CHART

$V = 90$

$q_z = 20.3 K_z$ -

$q_h = 20.3(1.17) = 23.75$

· DESIGN WIND LOAD p → 6.5.12/13/14/15

$p = qGC_p - q_i(GC_{pi})$ → PSF

$q = q_z$ FOR WINDWARD WALLS

$q = q_h$ FOR LEEWARD WALLS

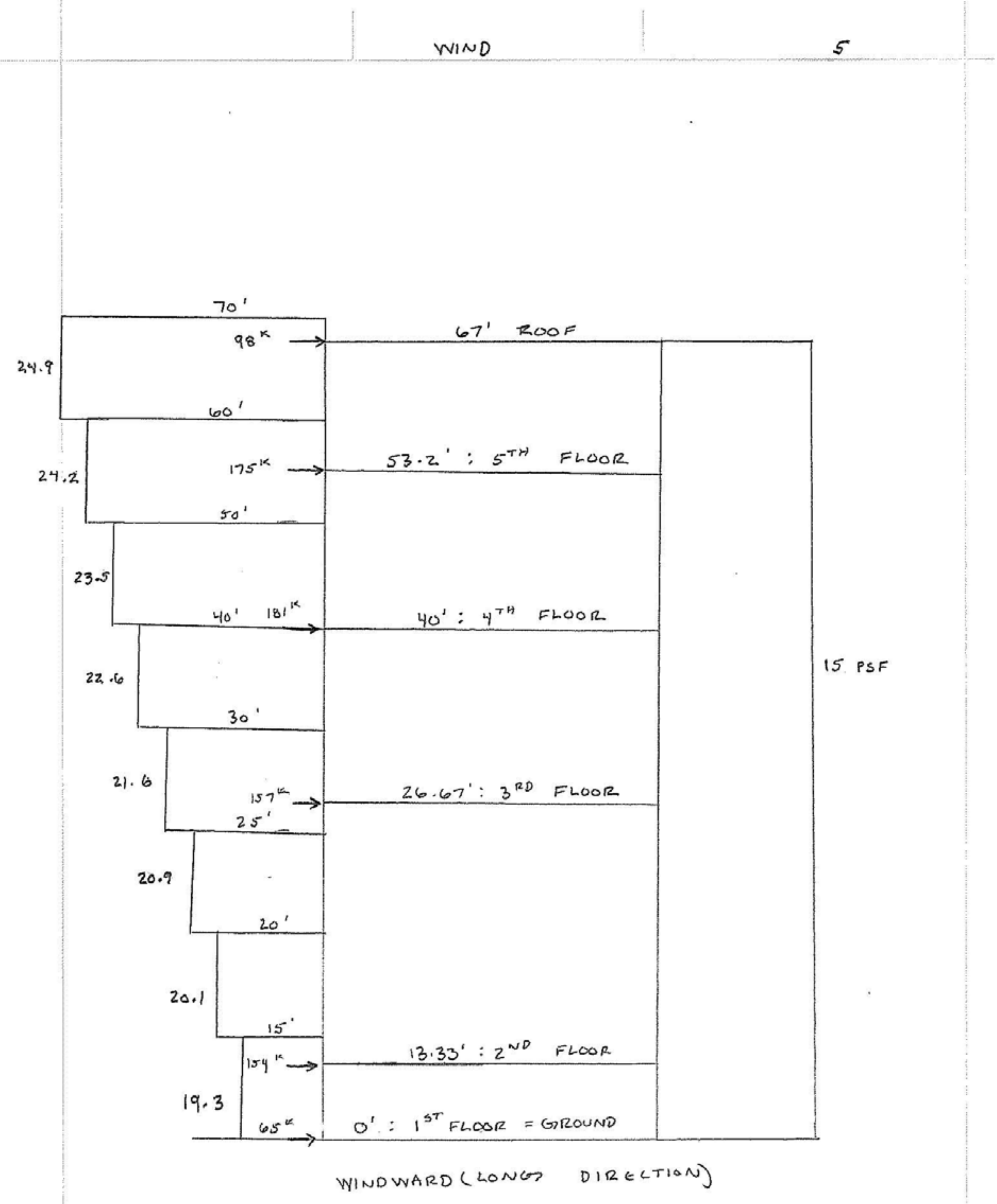
$q_i = q_h$ FOR (−) INT. PRESSURE

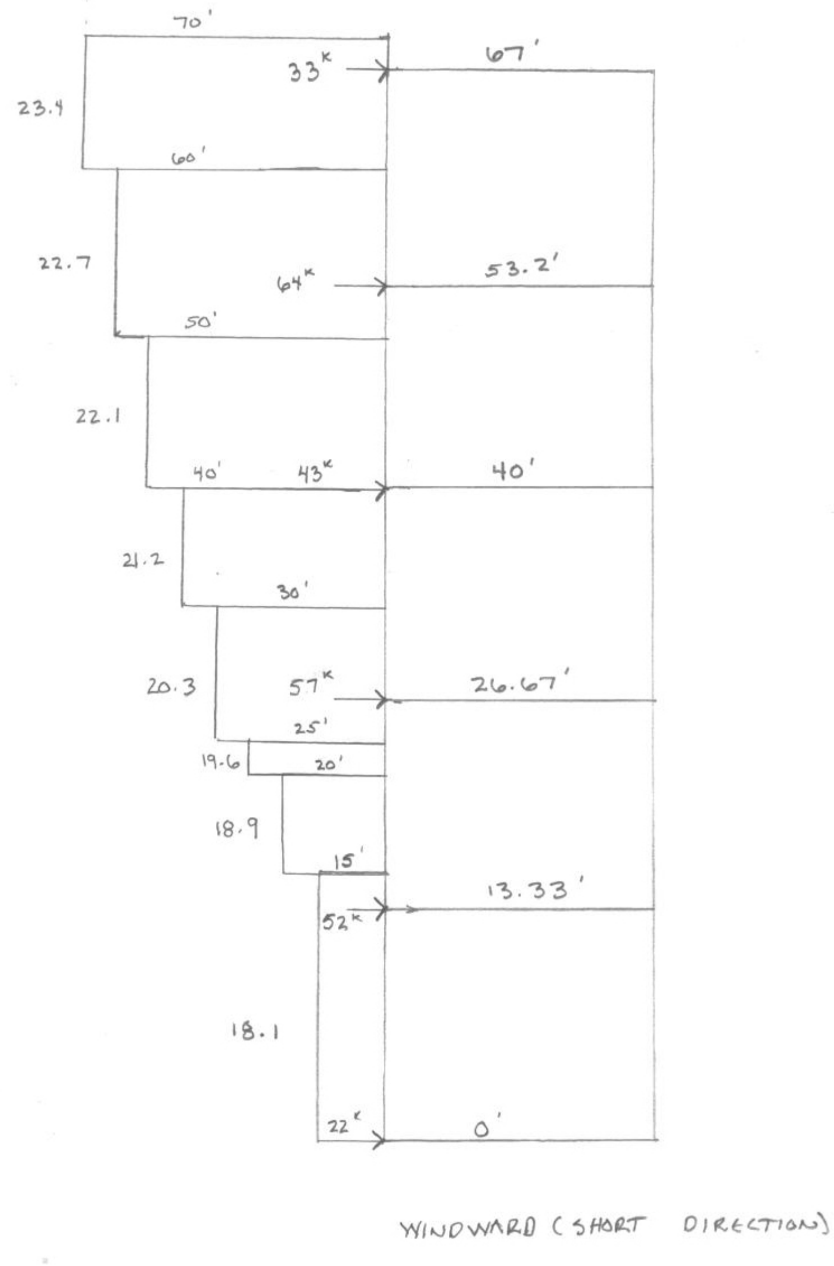
$q_i = q_z$ FOR (+) INT. PRESSURE

$G = 0.86$

$C_p = 0.8$

$GC_{pi} = +0.18 / -0.18$


WIND
5
70'
98K
67' ROOF
24.9
60'
24.2
175K
53.2' : 5TH FLOOR
50'
23.5
40' 181K
40' : 4TH FLOOR
22.6
15 PSF
30'
21.6
157K
26.67' : 3RD FLOOR
25'
20.9
20'
20.1
15'
154K
13.33' : 2ND FLOOR
19.3
65K
0' : 1ST FLOOR = GROUND
WINDWARD (LONG DIRECTION)


70'
33k
67'
23.4
60'
22.7
64k
53.2'
50'
22.1
40'
43k
40'
21.2
30'
20.3
57k
26.67'
25'
19.6
20'
18.9
15'
13.33'
52k
18.1
22k
0'
WINDWARD (SHORT DIRECTION)

SEISMIC CALCULATION

SEISMIC 1

SEISMIC

- SITE CLASS C → 67' TALL ABOVE GRADE
- $S_S = 0.328g$

  $S_1 = 0.008g$
- $F_a = 1.2$

  $F_V = 1.7$
- $S_{MS} = F_a \cdot S_S = 1.2(0.328) = 0.3936$

  $S_{M1} = F_V \cdot S_1 = 1.7(0.008) = 0.0136$
- $S_{DS} = (2/3) S_{MS} = 0.26$

  $S_{D1} = (2/3) S_{M1} = 0.0091$
- COMPOSITE STEEL + CONCRETE CONCENTRICALLY BRACED FRAMES

  $R = 5$

  $\Omega_o = 2$

  $C_d = 4.5$
- $I = 1.25$ → OCCUPANCY CAT. III
- SEISMIC DESIGN CATEGORY B

· DESIGN BASE SHEAR

$V = C_s \cdot W$

- $T_a = C_t h_n^x$

$= 0.028(67')^{0.8} = 0.81\,s$

- $C_u T_a = 1.7(0.81) = 1.38$

- $T_L = 6\,s$

- $C_s \geq$

$S_{DS}/(R/I) = 0.26/(5/1.25) = 0.065$

$S_{D1}/[T(R/I)] = 0.0091/(0.81(5/1.25)) = 0.0028$

$\dfrac{S_{D1} \cdot T_L}{T^2(R/I)} = \dfrac{0.0091(6)}{(0.81)^2(5/1.25)} = 0.021$

$C_s = 0.0028 < 0.01 \rightarrow \underline{0.01}$

- W:
  - FLAT SNOW ROOF LOAD = 24 PSF
  - TYP. FLOOR:
    - 42 PSF CONC. SLAB
    - 30 PSF SDL
    - 15 PSF STEEL STRUCTURE + METAL DECK
    - 13 PSF PARTION LOAD

    100 PSF

- WALLS
  - BRICK TRUSS PANEL = 40 PSF

- AREAS:
  - TYPICAL FLOOR: 560' × 203' = 113,680 FT²
  - TYPICAL WALL: 560'(13.3')2 + 203(13.3')2 = 21,014 FT²
  - BASEMENT: 50,000 FT²

SEISMIC 3

- $V = C_s \cdot W$

  $V = 566^k$ → REFER TO TABLE

- VERTICAL DISTRIBUTION OF FORCES

  - $F_x = C_{vx} V$

$$C_{vx} = \frac{w_x h_x^{\,k}}{\Sigma w_i h_i^{\,k}}$$

$k = 2$

$h_i = 87'$

$W_i = 56{,}563^k$

- OVERTURNING MOMENT $= 36{,}336^{k\text{-}ft}$

LATERAL STRENGTH CHECK

LATERAL CHECK

COL : W14 × 193

$L = 20'$

$k = 1.0$

$P_u = 897^k$

$M_u = 8.62^{k-FT}$

$V_u = 5.21^k$

$F_y = 50$ KSI

$F_u = 65$ KSI

$A_g = 56.8 \ in^2$

$I_x = 2400 \ in^4$

$I_y = 931 \ in^4$

$r_x = 6.5''$

$r_y = 4.05''$

$Z_x = 355 \ in^3$

$Z_y = 180 \ in^3$

$$\frac{kL_x}{r_x} = \frac{1.0(20' \times 12)}{6.5} = 36.92$$

$$\frac{kL_y}{r_y} = \frac{1.0(20 \times 12)}{4.05} = 59.26 \ *$$

$$\lambda_c = \frac{kL}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{59.26}{\pi}\sqrt{\frac{50}{29{,}000}} = 0.78 < 1.5$$

$$\phi_c P_n = \phi_c F_{cr} A_g = \phi_c (0.658^{\lambda_c^2}) A_g \cdot F_y$$

$$= 0.85(0.658^{(0.78)^2}) 56.8 \ in^2 \cdot 50$$

$$= 1871^k > P_u = 897 \ \checkmark$$

$$\phi_v V_n = \phi_v(0.6) F_y (d) t_w = 0.9(0.6)50(15.5)0.89 = 373^k > 5.21^k \ \checkmark$$

$L_p = 14.3' < L_b = 20'$

$L_r = 70.1'$

$C_b = 1.0$

$\phi_b M_{px} = 1330^{k-FT}$

$\phi_b M_{rx} = 930^{k-FT}$

$$\phi_b M_n = C_b \left[\phi_b M_p - (\phi_b M_p - \phi_b M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]$$

$$= 1.0 \left[1330 - (1330 - 930)\left(\frac{20 - 14.3}{70.1 - 14.3}\right)\right]$$

$$= 1289^{\text{K-FT}}$$

$$\frac{P_u}{\phi_c P_n} = \frac{897^{K}}{1871^{K}} = 0.48 > 0.2 \quad \text{H1-1A}$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_u}{\phi_b M_n}\right) = 0.48 + \frac{8}{9}\left(\frac{8.62}{1289}\right) = 0.49 < 1.0 \checkmark$$

LATERAL CHECK

BRACE : HSS 6×6×1/4

TENSION CONTROLLED

$P_u = 121^k$

$L = 24'$

$F_y = 46$ KSI $\qquad A_g = 5.24\ in^2$

$F_u = 58$ KSI

$$\phi P_n = \phi F_y A_g$$

$$= 0.9(46\ KSI)\ 5.24\ in^2 = 216.936^k > P_u = 121^k \quad \checkmark$$

$$\frac{P_u}{\phi P_n} = \frac{121}{216.94} = 0.558 > 0.2$$

$$\frac{P_u}{\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad H1-1A$$

$$0.558 + 0 = 0.558 \leq 1.0 \quad \checkmark$$

LATERAL CHECK

BEAM: W16×26

$V_u = 27.33^k$ $f'_c = 4000$ PSI

$P_u = 0$

$M_u = 91.61^{k-FT}$

$b_{eff} = \frac{20'(12)}{4} = 60''$ *

$10'(12) = 120''$

TRY TFL w/ $Y2 = 2''$ ($\phi M_n = 268^{k-FT}$, $\Sigma Q_n = 384^k$)

$T_s = A_s F_y = 7.68 \text{ in}^2 (50 \text{ KSI}) = 384^k$

$C_c = 0.85 f'_c \, b_{eff} (a) = 0.85 (4000 \text{ PSI})(60'') a = 384^k$

$a = \frac{\Sigma Q_n}{0.85 (f'_c) b_{eff}} = \frac{384^k}{0.85(4000 \text{ PSI})(60'')} = 1.88''$

$Y2 = 6.25 - \frac{1.88}{2} = 5.31'' > 2''$ ✓

$M_n = T_s \left(\frac{d}{2}\right) + C_c (t - a/2)$

$= 384^k \left(\frac{15.7''}{2}\right) + 384^k (5.31'') = 203^{k-FT}$

$\phi M_n = 0.9(203) = 182^{k-FT} > 92^{k-FT}$ ✓

WIND STORY SHEAR

Building Story Shears

RAM Frame v11.0
DataBase: Model 12/02/07 18:25:12

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Wall Mesh Criteria :
Wall Element Type : Shell Element with No Out-of-Plane Stiffness
Max. Allowed Distance between Nodes (ft) : 8.00

Load Case: D DeadLoad RAMUSER

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	-0.26	0.23
5	1	-0.71	1.42
4	1	-1.13	2.63
3	1	-0.78	1.80
2	1	-0.69	1.76
1	1	4.10	10.14

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	-0.26	-0.26	0.23	0.23
5	-0.71	-0.45	1.42	1.19
4	-1.13	-0.42	2.63	1.20
3	-0.78	0.35	1.80	-0.82
2	-0.69	0.08	1.76	-0.04
1	4.10	4.80	10.14	8.38

Load Case: Lp PosLiveLoad RAMUSER

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	-0.30	1.20
5	1	-0.79	0.70
4	1	-1.10	0.04
3	1	-1.05	4.77
2	1	-0.33	-2.03
1	1	4.20	10.32

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	-0.30	-0.30	1.20	1.20
5	-0.79	-0.49	0.70	-0.50
4	-1.10	-0.32	0.04	-0.66

Building Story Shears

RAM Frame v11.0
DataBase: Model 12/02/07 18:25:12

4	1	0.24	465.10
3	1	-0.22	642.45
2	1	0.21	879.35
1	1	-16.48	397.51

Summary - Total Story Shears

Level	**Shear-X**	**Change-X**	**Shear-Y**	**Change-Y**
	kips	**kips**	**kips**	**kips**
Roof	0.16	0.16	95.11	95.11
5	0.25	0.09	282.23	187.12
4	0.24	-0.01	465.10	182.87
3	-0.22	-0.46	642.45	177.35
2	0.21	0.43	879.35	236.90
1	-16.48	-16.69	397.51	-481.84

Load Case: W5 Wind3 Wind_IBC03_2_X+E

Level	**Diaph. #**	**Shear-X**	**Shear-Y**
		kips	**kips**
Roof	1	23.16	-0.01
5	1	68.90	-0.01
4	1	113.08	0.01
3	1	159.54	0.02
2	1	190.66	1.48
1	1	94.23	-1.54

Summary - Total Story Shears

Level	**Shear-X**	**Change-X**	**Shear-Y**	**Change-Y**
	kips	**kips**	**kips**	**kips**
Roof	23.16	23.16	-0.01	-0.01
5	68.90	45.74	-0.01	-0.00
4	113.08	44.18	0.01	0.02
3	159.54	46.47	0.02	0.01
2	190.66	31.12	1.48	1.46
1	94.23	-96.43	-1.54	-3.02

Load Case: W6 Wind3 Wind_IBC03_2_X-E

Level	**Diaph. #**	**Shear-X**	**Shear-Y**
		kips	**kips**
Roof	1	23.18	-0.06
5	1	68.94	-0.14
4	1	113.16	-0.14
3	1	159.47	-0.14
2	1	191.25	-1.92
1	1	98.26	-36.54

Building Story Shears

RAM Frame v11.0
DataBase: Model
12/02/07 18:25:12

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	23.18	23.18	-0.06	-0.06
5	68.94	45.76	-0.14	-0.08
4	113.16	44.22	-0.14	-0.00
3	159.47	46.31	-0.14	-0.00
2	191.25	31.77	-1.92	-1.78
1	98.26	-92.98	-36.54	-34.62

Load Case: W7 Wind3 Wind_IBC03_2_Y+E

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.18	71.10
5	1	0.35	211.11
4	1	0.49	348.15
3	1	-0.38	481.08
2	1	0.48	655.90
1	1	10.08	144.42

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	0.18	0.18	71.10	71.10
5	0.35	0.16	211.11	140.01
4	0.49	0.15	348.15	137.04
3	-0.38	-0.88	481.08	132.93
2	0.48	0.87	655.90	174.82
1	10.08	9.60	144.42	-511.48

Load Case: W8 Wind3 Wind_IBC03_2_Y-E

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.05	71.57
5	1	0.02	212.23
4	1	-0.13	349.50
3	1	0.05	482.59
2	1	-0.16	663.12
1	1	-34.81	451.84

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	0.05	0.05	71.57	71.57
5	0.02	-0.03	212.23	140.66

SEISMIC STORY SHEAR

Building Story Shears

RAM Frame v11.0
DataBase: Model 12/02/07 18:25:12

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	98.11	98.11	-0.00	-0.00
5	193.42	95.30	0.02	0.02
4	266.64	73.22	0.10	0.08
3	318.38	51.74	0.12	0.02
2	343.10	24.72	-0.39	-0.51
1	103.24	-239.86	5.33	5.71

Load Case: E3 Seismic2 EQ_IBC00_X_-E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	98.13	-0.06
5	1	193.45	-0.09
4	1	266.69	-0.02
3	1	318.34	-0.00
2	1	343.12	-0.79
1	1	106.19	-14.27

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	98.13	98.13	-0.06	-0.06
5	193.45	95.32	-0.09	-0.04
4	266.69	73.24	-0.02	0.07
3	318.34	51.65	-0.00	0.02
2	343.12	24.78	-0.79	-0.79
1	106.19	-236.93	-14.27	-13.48

Load Case: E4 Seismic2 EQ_IBC00_Y_+E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.07	97.27
5	1	0.10	191.68
4	1	0.07	265.35
3	1	-0.09	319.23
2	1	0.08	343.05
1	1	-8.76	153.61

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	0.07	0.07	97.27	97.27
5	0.10	0.03	191.68	94.41

STORY DISPLACEMENTS

Story Displacements

RAM Frame v11.0
DataBase: Model
Building Code: IBC

12/02/07 15:11:59

CRITICIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Wall Mesh Criteria :
Wall Element Type : Shell Element with No Out-of-Plane Stiffness
Max. Allowed Distance between Nodes (ft) : 8.00

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
W1	Wind	W_User
E1	Seismic	EQ_User
W2	Wind2	W_User
W3	Wind3	Wind_IBC03_1_X
W4	Wind3	Wind_IBC03_1_Y
W5	Wind3	Wind_IBC03_2_X+E
W6	Wind3	Wind_IBC03_2_X-E
W7	Wind3	Wind_IBC03_2_Y+E
W8	Wind3	Wind_IBC03_2_Y-E
W9	Wind3	Wind_IBC03_3_X+Y
W10	Wind3	Wind_IBC03_3_X-Y
W11	Wind3	Wind_IBC03_4_X+Y_CW
W12	Wind3	Wind_IBC03_4_X+Y_CCW
W13	Wind3	Wind_IBC03_4_X-Y_CW
W14	Wind3	Wind_IBC03_4_X-Y_CCW

Level: Roof, Diaph: 1

Center of Mass (ft): (231.74, 145.36)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.04485	0.10576	0.00004
Lp	-0.05122	0.12278	0.00005
W1	0.50157	-0.03578	0.00007
E1	0.43390	-0.02609	0.00005
W2	0.50157	-0.03578	0.00007
W3	0.17079	-0.01191	0.00002
W4	0.01074	0.58609	-0.00002
W5	0.12670	-0.00013	0.00000
W6	0.12948	-0.01774	0.00003
W7	0.01738	0.36351	0.00012
W8	-0.00127	0.51563	-0.00015
W9	0.13614	0.43064	0.00000
W10	0.12003	-0.44851	0.00003
W11	0.09407	0.38663	-0.00011

Story Displacements

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W12	0.11015	0.25933	0.00011
W13	0.08199	-0.27273	-0.00009
W14	0.09806	-0.40003	0.00014

Level: 5, Diaph: 1

Center of Mass (ft): (231.57, 145.58)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.03434	0.09633	0.00003
Lp	-0.03932	0.11205	0.00003
W1	0.44158	-0.03009	0.00006
E1	0.38088	-0.02122	0.00005
W2	0.44158	-0.03009	0.00006
W3	0.15204	-0.01007	0.00002
W4	0.00437	0.51474	-0.00002
W5	0.11306	0.00016	0.00000
W6	0.11500	-0.01526	0.00003
W7	0.00965	0.31938	0.00011
W8	-0.00309	0.45274	-0.00013
W9	0.11731	0.37851	0.00000
W10	0.11075	-0.39361	0.00003
W11	0.08248	0.33967	-0.00010
W12	0.09349	0.22809	0.00010
W13	0.07756	-0.23942	-0.00008
W14	0.08857	-0.35100	0.00012

Level: 4, Diaph: 1

Center of Mass (ft): (231.53, 145.59)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.02119	0.07003	0.00002
Lp	-0.02431	0.08161	0.00002
W1	0.34427	-0.02445	0.00005
E1	0.27993	-0.01596	0.00004
W2	0.34427	-0.02445	0.00005
W3	0.12041	-0.00821	0.00002
W4	-0.00019	0.42214	-0.00002
W5	0.08977	0.00034	-0.00000
W6	0.09084	-0.01265	0.00002
W7	0.00326	0.26028	0.00009
W8	-0.00354	0.37293	-0.00011
W9	0.09017	0.31045	-0.00000
W10	0.09045	-0.32276	0.00002
W11	0.06468	0.27995	-0.00008
W12	0.07058	0.18572	0.00008

Story Displacements

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W13	0.06488	-0.19495	-0.00007
W14	0.07079	-0.28919	0.00010

Level: 3, Diaph: 1

Center of Mass (ft): (231.51, 145.58)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.00766	0.03865	0.00002
Lp	-0.00881	0.04507	0.00002
W1	0.24107	-0.02134	0.00003
E1	0.18086	-0.01343	0.00002
W2	0.24107	-0.02134	0.00003
W3	0.08715	-0.00717	0.00001
W4	-0.00305	0.31714	-0.00002
W5	0.06535	0.00018	-0.00000
W6	0.06537	-0.01093	0.00002
W7	-0.00261	0.18958	0.00006
W8	-0.00197	0.28613	-0.00009
W9	0.06307	0.23248	-0.00001
W10	0.06765	-0.24323	0.00002
W11	0.04753	0.21473	-0.00007
W12	0.04707	0.13398	0.00005
W13	0.05097	-0.14205	-0.00004
W14	0.05050	-0.22280	0.00008

Level: 2, Diaph: 1

Center of Mass (ft): (235.60, 127.88)

LdC	Disp X in	Disp Y in	Theta Z rad
D	0.00065	0.02307	0.00001
Lp	0.00076	0.02692	0.00001
W1	0.16306	-0.01807	0.00002
E1	0.11402	-0.01127	0.00001
W2	0.16306	-0.01807	0.00002
W3	0.06043	-0.00607	0.00001
W4	-0.00738	0.20306	-0.00003
W5	0.04385	0.00000	0.00000
W6	0.04679	-0.00911	0.00001
W7	0.00589	0.11301	0.00002
W8	-0.01697	0.19158	-0.00007
W9	0.03979	0.14774	-0.00002
W10	0.05086	-0.15685	0.00003
W11	0.02017	0.14368	-0.00005
W12	0.03952	0.07792	0.00003
W13	0.02847	-0.08475	-0.00002

Story Displacements

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W14	0.04782	-0.15051	0.00006

Level: 1, Diaph: 1

Center of Mass (ft): (96.40, 137.26)

LdC	**Disp X**	**Disp Y**	**Theta Z**
	in	**in**	**rad**
D	0.00459	0.00900	0.00000
Lp	0.00527	0.01025	0.00000
W1	0.09532	-0.03204	0.00002
E1	0.05924	-0.02005	0.00001
W2	0.09532	-0.03204	0.00002
W3	0.04493	-0.01044	0.00000
W4	-0.00441	0.14993	-0.00001
W5	0.03470	0.00068	-0.00001
W6	0.03270	-0.01633	0.00001
W7	-0.00047	0.04791	0.00002
W8	-0.00614	0.17699	-0.00004
W9	0.03039	0.10462	-0.00001
W10	0.03700	-0.12028	0.00001
W11	0.02142	0.13325	-0.00003
W12	0.02417	0.02368	0.00003
W13	0.02638	-0.03542	-0.00002
W14	0.02913	-0.14499	0.00004

LATERAL STRENGTH CHECK

Member Code Check

RAM Frame v11.0
DataBase: Model
Building Code: IBC

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Steel Code: AISC LRFD

BEAM INFORMATION:

Story Level = 5   Frame Number = 11   Beam Number = 383
Fy (ksi) = 50.00
Beam Size = W16X26

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	20.00	0.00
Lu for Bending (ft)	20.00	0.00
K	1.00	1.00
Braced Against Joint Translation	No	No
Top Flange Continuously Braced	Yes	
Bottom Flange Continuously Braced	Yes	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Segment distance (ft) i - end ____ 0.00
j - end ____ 20.00

Load Combination: 1.200 D + 1.600 Lp

SHEAR CHECK:

Vux (kips) = -27.41	0.90*Vnx (kips) = 104.15	Vux/0.90*Vnx = 0.263
Vuy (kips) = -0.00	0.90*Vny (kips) = 102.47	Vuy/0.90*Vny = 0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Segment distance (ft) i - end ____ 0.00
j - end ____ 20.00

Load Combination: 1.200 D + 1.600 Lp

CALCULATED PARAMETERS:

Pu (kips)	= 0.00	0.85*Pn (kips)	= 293.14
Mux (kip-ft)	= -92.39	0.90*Mnx (kip-ft)	= 165.75
Muy (kip-ft)	= -0.00	0.90*Mny (kip-ft)	= 19.63
Cbx	= 1.00		

INTERACTION EQUATION:

Pu/ϕPn = 0.000
Eq H1-1b: 0.000 + 0.557 + 0.000 = 0.557

Member Code Check

RAM Frame v11.0
DataBase: Model
Building Code: IBC

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Steel Code: AISC LRFD

BRACE INFORMATION:

Story Level	= Roof	Frame Number	= 11	Brace Number	= 9
Fy (ksi)	= 46.00				
Brace Size	= HSS6X6X1/4				

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	24.04	24.04
K	1.00	1.00
Braced Against Joint Translation	No	No

CONTROLLING BRACE FORCES - SHEAR

Load Combination: 1.200 D + 0.500 Lp - 1.300 W8

Shear	Top	Vux (kips)	0.00
		Vuy (kips)	0.00
Shear	Bot.	Vux (kips)	0.00
		Vuy (kips)	0.00

SHEAR CHECK:

Vux (kips) =	0.00	0.90*Vnx (kips) =	69.45	Vux/0.90*Vnx =	0.000
Vuy (kips) =	-0.00	0.90*Vny (kips) =	69.45	Vuy/0.90*Vny =	0.000

CONTROLLING BRACE FORCES - FLEXURE

Load Combination: 1.200 D + 1.600 Lp

Axial		Load (kips)	-123.70
Moment	Top	Mux (kip-ft)	0.00
		Muy (kip-ft)	0.00
Moment	Bot.	Mux (kip-ft)	0.00
		Muy (kip-ft)	0.00

CALCULATED PARAMETERS:

Pu (kips) =	-123.70	0.90*Pn (kips) =	216.94
Mux (kip-ft) =	0.00	0.90*Mnx (kip-ft) =	38.64
Muy (kip-ft) =	0.00	0.90*Mny (kip-ft) =	38.64
Cbx =	1.00		

INTERACTION EQUATION:

$Pu/\phi Pn$ = 0.570

Eq H1-1a: 0.570 + 8/9*(0.000 + 0.000) = 0.570

Member Code Check

RAM Frame v11.0
DataBase: Model 12/02/07 15:11:59
Building Code: IBC Steel Code: AISC LRFD

COLUMN INFORMATION:

Story Level = 1 Frame Number = 11 Column Number = 421
Fy (ksi) = 50.00
Column Size = W14X193

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	20.00	20.00
K	1.91	1.00
Braced Against Joint Translation	No	No

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.200 D + 0.500 Lp + 1.300 W8

Shear	Top	Vux (kips)	5.24
		Vuy (kips)	0.07
Shear	Bot.	Vux (kips)	5.24
		Vuy (kips)	0.07

SHEAR CHECK:

Vux (kips) = 5.24 0.90*Vnx (kips) = 372.46 Vux/0.90*Vnx = 0.014
Vuy (kips) = -0.07 0.90*Vny (kips) = 1220.83 Vuy/0.90*Vny = 0.000

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.200 D + 1.600 Lp

Axial		Load (kips)	933.90
Moment	Top	Mux (kip-ft)	-8.43
		Muy (kip-ft)	1.60
Moment	Bot.	Mux (kip-ft)	8.89
		Muy (kip-ft)	-1.82

CALCULATED PARAMETERS:

Pu (kips) = 933.90 0.85*Pn (kips) = 1678.02
Mux (kip-ft) = 8.89 0.90*Mnx (kip-ft) = 1331.25
Muy (kip-ft) = -1.82 0.90*Mny (kip-ft) = 669.38
Cbx = 2.26

INTERACTION EQUATION:

Pu/ϕPn = 0.557
Eq H1-1a: 0.557 + 8/9*(0.007 + 0.003) = 0.565