Fall 2009

Hunter College school of Social Work

Lateral System Analysis and Confirmation Design

The Lateral System Analysis and Conditions Report consists of a check on strength, drift, story drift, overturning and impact on foundations. The existence of a logical load path for the distribution of the calculated loads in the real structure was confirmed and the controlling load combination per ASCE7-05 was determined.

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Technical Report 3 – Lateral System Analysis and Confirmation Design

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Executive Summary

Hunter College School of Social Work is located on Third Avenue between 118th and 119th street. It is designed to be both a college and university space. The structure is comprised of a composite steel floor system that utilizes steel braced and moment frames to resist lateral forces. Drilled caissons and spread footings make up the foundation system. The cellar floor is a reinforced slab on a mat foundation. The total height is 133ft above ground level.

The third consists of a check on strength, drift, story drift, overturning and impact on foundations. The existence of a logical load path for the distribution of the calculated loads in the real structure was confirmed and the controlling load combination and wind load case per ASCE7-05 were determined. ETABS was used for the lateral analysis of Hunter College School of Social Work, and hand calculations were performed to verify results from the program output. Members of the braced frame and moment frame were checked for strength and drift requirements.

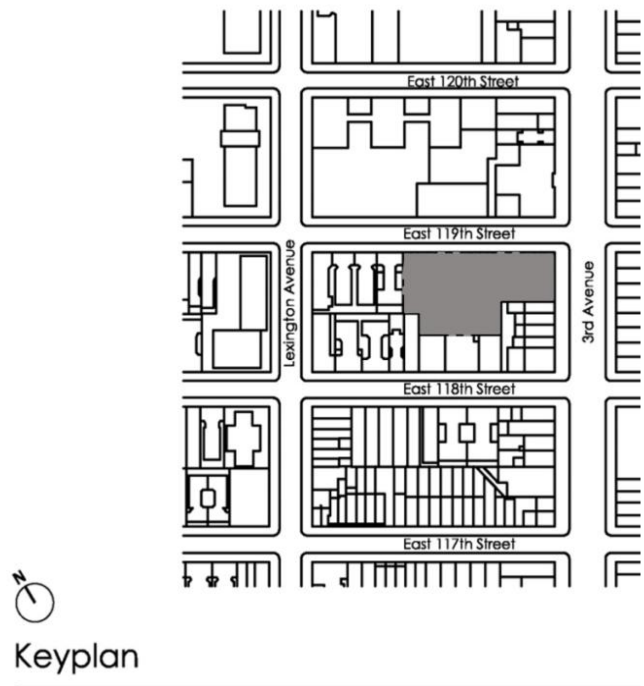
The controlling wind case was found to be case 1 and the controlling load combination was
1.2 (Dead) +1.6 (Wind) +1.0(Live) +0.5(Roof Live).

Drift limits were found to be well above the actual drift calculated through ETABS for both wind and seismic loads. Drift values for wind were serviceability requirements taken to be a max of H/400. Seismic drift limitation was taken to be $\Delta_{seismic}=0.015h_{sx}$ (in.) based on ASCE 7-05. Strength spot checks were also found to be satisfactory.

Overturning was found to be resisted by all frames except the five-story braced frame at grid 1. This indicates an impact on the foundation. However, since seismic forces used were those determined using ASCE 7-05, they do not accurately represent the values used by the structural engineer. It is very possible that a "no impact on foundation" conclusion was found by the structural engineer.

Introduction

The building's design responds to the School of Social Work's mission by providing an open and engaging face to the neighborhood and opportunities for community use of parts of the facility. The entrance lobby, conceived as an interior street, is glazed from floor to ceiling along 119th Street to provide a transparent and welcoming appearance from the exterior and to link the interior of the building to its neighborhood surroundings. Classrooms and lecture halls occupy the lower levels with academic departments and offices on upper floors. An auditorium on the second floor is expressed on the facade, with a glazed wall allowing views of activity in and outside the building. A rear landscaped terrace will link the School to a planned CUNY Residential building adjacent to the site on 118th Street. The School of Social Work building will be LEED certified. -Cooper Robertson & Associates



The structure of Hunter College School of Social Work is comprised of a composite steel floor system that utilizes steel braced and moment frames to resist lateral forces. Drilled caissons and spread footings make up the foundation system. The cellar floor is a reinforced slab on a mat foundation. The total height is 133ft above ground level.

Technical Report III consists of a check on strength, drift, story drift, overturning and impact on foundations. The existence of a logical load path for the distribution of the calculated loads in the real structure was confirmed and the controlling load combination per ASCE7-05 was found.

Code and Design Requirements

Applied to original Design

The Building Coded of the City of New York (most current) - Amended seismic design

AISC-LRFD, LRFD Specification for Structural Steel Buildings (applied except on the lateral force resisting frame)

AISC- ASD 1989, Specifications for Structural Steel Buildings- ASD and Plastic Design (for the design and construction of steel framing in lateral force resisting system)

ACI 318-89, Building Code Requirements for Structural Concrete

Substituted for thesis analysis

2006 International Building Code

ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Steel Construction Manual 13th edition, American Institute of Steel Construction

ACI 318-05, Building Code Requirements for Structural Concrete, American Concrete Institute

Material strength requirement summary

Structural Steel:

- All W Beams and Columns: ASTM A992, Fy=50ksi
- HSS Steel, Fy=46ksi
- Connection Material:Fy=36 ksi
- Base plates: ASTM 572 GR50, Fy=50ksi

Metal Decking:

- Units shall be 3” galvanized composite deck of 18 gage formed with integral locking lugs to provide a mechanical bond between concrete and deck

-Strength: Fy=40ksi

-Deflection of form due to dead load of concrete and deck does not exceed L/180 , but not more than ¾”

-Deflection of composite deck cannot exceed L/360 of deck span under superimposed live load.

Concrete:

-Caissons and Piers: 4000psi normal weight concrete

-Slabs on ground and footings: 4000psi normal weight concrete

-Retaining Walls: 4000 psi normal weight concrete

-Slab on deck: 3500psi lightweight concrete

- Foundations: 4000psi, air entrained, normal weight

-Walls, curbs, and parapets: 4000 psi

Reinforcement:

-Strength: 60ksi

Building Load Summary

Gravity Loads

Total building weight was found to be approximately 15,388kips. Detailed charts in Appendix A tabulate the columns and beams used in finding the total weight. Curtain wall weight was approximated to be 15 psf although curtain wall type varies as you go up in elevation. Glass curtain wall is used on the upper and lower sections of the building façade and precast masonry and stucco panels are used on the middle section of the building façade. Calculation of the building weight was tedious due to the varying bay sizes, column and beam sizes, and varying lengths of these members. In erection of the structure, careful coordination must be taken in order to correctly identify and place these frame elements.

Level	Floor Height (ft)	Slab Weight (lbs)	Column Weight (lbs)	Beam Weight (lbs)	Curtainwall Weight (lbs)	Total Level Weight (lbs)
Penthouse	134	80750	0	38245	0	118995
Roof	120	492300	3440	50726	70560	617026
8	104	403570	15938	37130	61740	518378
7	91	374170	24463	42135	57330	498098
6	78	1108370	24463	116396	127335	1376564
5	64	1201959	16940	169389	144690	1532978
4	50	1201959	86174	90008.7	144690	1522831.7
3	36	1201959	76816.5	140824.5	144690	1564290
2	19	3223770.5	76816.5	220889.5	178755	3700231.5
1	0	3356119.75	236557.1637	177844	168240	3938760.916
					Total Building Weight:	15388153.12

Figure 1. Building Dead Load Summary

ID	location	Live Loads (psf)			Dead Loads (psf)
		Design Live Loads	ASCE 705-05	NYC BLDG CODE 08	Design Dead Loads
1	loading dock	600	-	-	150
2	1st floor	100	100	100	130
3	podium	100	100	-	200
4	archive	350	-	-	75
5	offices	50	50	50	71
6	roof with garden	100	100	100	365
7	library stacks	100	100	100	71
8	classrooms	40	40	60	71
9	corridor	100	100	100	71
10	auditorium	60	60	100	85
11	roof with pavers on 2	100	-	-	150
12	roof	45	20	30	90
13	roof with drift	60	45	-	85
14	mechanical	100	125	100	120

Figure 2. Loading Schedule

Wind Load Summary

Since the Hunter College School of Social Work is located in New York City, the NYC Building Code governed the structural design. For this analysis, however, ASCE-7-05 was used along with Fanella Wind Analysis flowcharts. For detailed calculations please refer to Appendix A. In the north/south direction the base shear due to lateral wind loads was found to be 559 kips, much larger than in the East/West direction; 162 kips. This difference in base shear is due to building's rectangular shape as opposed to a square footprint. Wind forces were found to be much higher than seismic forces (figure 14). Seismic base shear was found to be 154 kips, less than wind-caused shear in either direction; north/south or east/west.

Due to the building's setbacks, it has differing roof levels, creating a potential for snow drifts. The allowable snow drift calculations were found to be 46psf (refer to Appendix A for details). The allowable snow drift values, along with the wind or seismic analysis, were not checked against the values originally found by the structural designers. The information needed was not provided on the construction documents for verification.

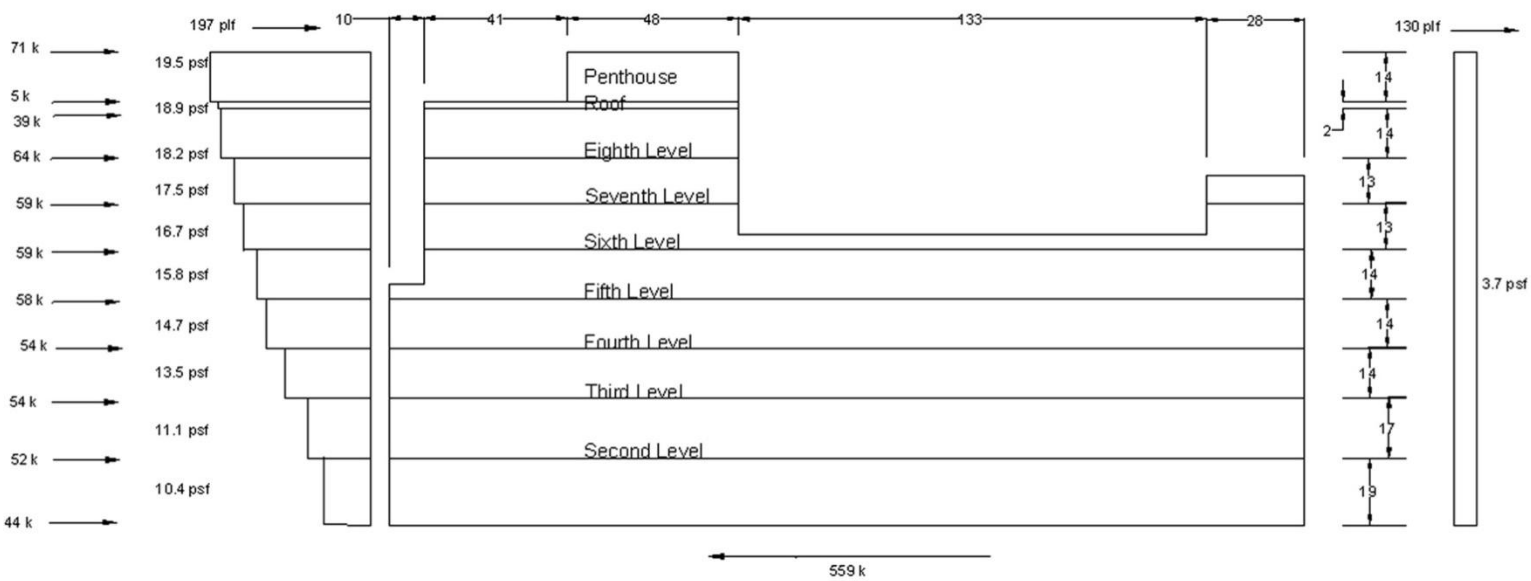


Figure 3. Wind Diagram using ASCE7 – In East/West wind direction

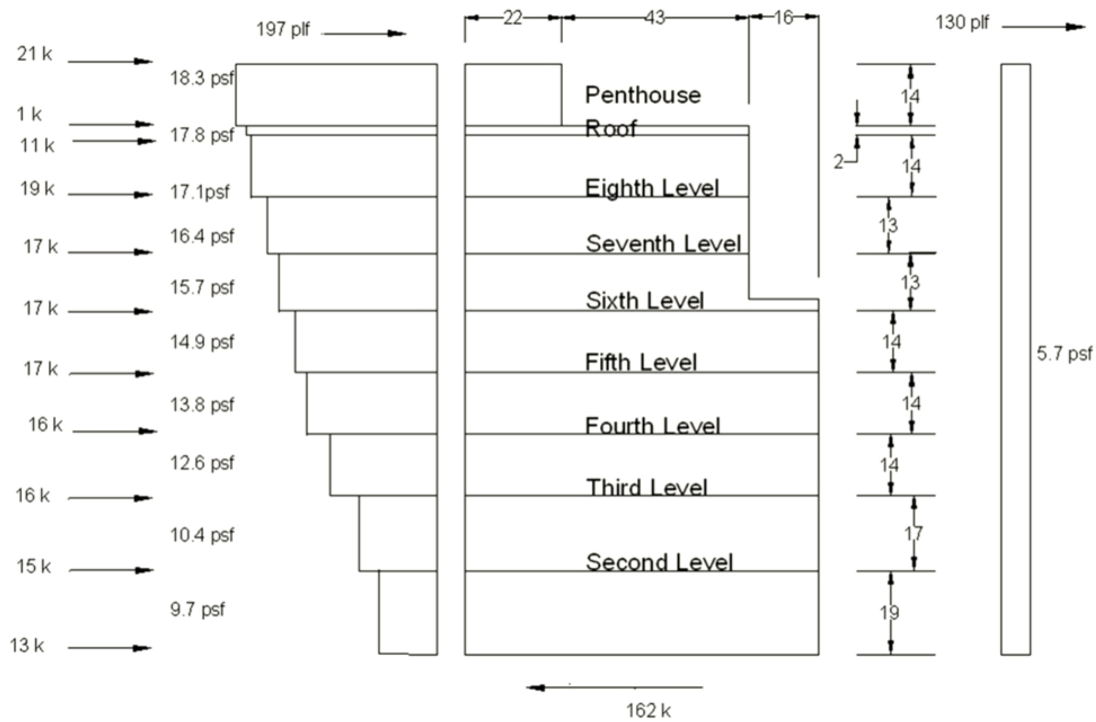


Figure 4. Wind Diagram using ASCE7 – In North/South wind direction

Refer to figures 11 through 13 for design forces, shears, moments, and assumptions for wind using ASCE 7. For detailed calculations, refer to the appendix.

Level	Height Above Ground (ft)	Floor Height (ft)	h/2 above	h/2 below	Wind Forces					
					Load (kips)		Shear (kips)		Moment (ft-kips)	
					N-S	E-W	N-S	E-W	N-S	E-W
Pent house	134	14	14	0.125	71	21	71	21	9580	2783
T.O. Parapet	120	0.25	0.125	0.9	5	1	77	22	605	176
Roof	118	1.7	0.9	7.0	39	11	115	33	4557	1324
8	104	14	7	6.5	64	19	179	52	6641	1930
7	91	13	6.5	6.5	59	17	238	69	5372	1561
6	78	13	6.5	7	59	17	297	86	4583	1331
5	64	14	7	7	58	17	354	103	3687	1071
4	50	14	7	7	54	16	408	119	2682	779
3	36	14	7	8.5	54	16	462	134	1953	568
2	19	17	8.5	9.5	52	15	514	149	987	287
Ground	0	19	9.5	7	44	13	559	162	0	0

Figure 5. Wind Design Forces and Shears

Design Category	III
V (mph)	90
K_d=	0.85
Importance Factor (I)	1.1
Exposure Category	B (urban areas)
K_{zt}=	1
n1=	0.75
Gf	1.173 (N-S)
	1.189 (E-W)
Qp	20.16
GCpn	+1.5 windward
	-1.0 leeward
GCpi	n/a
z_g=	1200 ft
α=	7

Figure 6. Wind Design Criteria

	Level	Height Above Ground (ft)	Floor Height (ft)	K_z	q_z
windward	Penthouse	134	14	1.07	20.75
	T.O. Parapet	120	0.25	1.04	20.16
	Roof	118	1.7	1.04	20.16
	8	104	14	1	19.39
	7	91	13	0.96	18.61
	6	78	13	0.92	17.84
	5	64	14	0.87	16.87
	4	50	14	0.81	15.70
	3	36	14	0.74	14.35
	2	19	17	0.61	11.83
	Ground	0	19	0.57	11.05
Leeward	All	All	All	1.04	20.16

Figure 7. Wind Design qz factors for different story levels

Seismic Summary

Seismic loads were analyzed using chapters 11 and 12 of ASCE 7-05. Please refer to Appendix A for detailed calculations used to obtain building weight as well as base shear and overturning moment distribution for each floor as seen in figure 14 below. According to the construction documents, seismic analysis was not found to control this design. The site was declared not an issue for soil liquefaction.

Due to low approximations on the building weight the base shear may in actuality be higher than what is reported in figure 14. However it would not control because the shear cause by lateral wind loads is more than 3 times in magnitude.

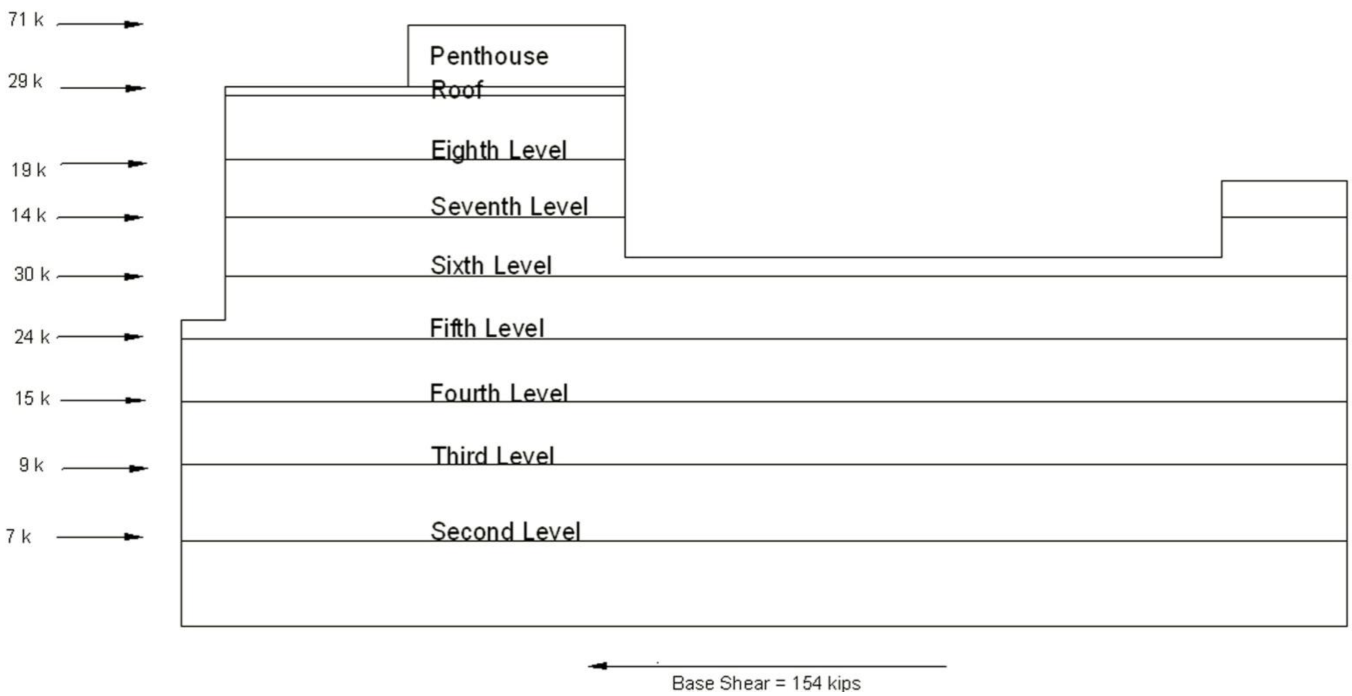


Figure 8. Seismic Force Diagram

Structural Systems

Foundation System

There is one below-grade level in the Hunter College School of Social Work. This level known as the cellar contains a parking garage for the residential building adjacent, a library, computer labs, large kitchen areas, and mechanical rooms.

Slab thickness varies throughout the cellar level. It can be 30”, 33”, or 40”. Steel reinforcement varies according to the slab thickness. For a 30” slab #7@11 are required top and bottom (T&B) each way, for a 33” slab #8@13 top and bottom, and for a 40” slab #9@13 top and bottom each way. The mat foundation will have a 2” mud slab above 12” of ¾ crushed stone to facilitate installation of waterproofing membrane. The subgrade is composed of undisturbed soil or compacted back fill with a required bearing capacity of 1.5 tons.

The soil is not considered susceptible to liquefaction for a Magnitude 6 earthquake and a peak ground acceleration of 0.16g. It is expected to encounter ground water during erection of the cellar level. Excavation depths are anticipated to vary from about 12ft to 20ft below existing ground surface grades. Footings shall bear on sound rock with a bearing capacity of 20 ton per square foot or on decomposed rock with a bearing capacity of 8 ton per square foot or on sand with a bearing capacity of 3 ton per square foot.

Foundation walls are designed to resist lateral pressures resulting from static earth, groundwater, adjacent foundations, and sidewalk surcharge loads. These walls will extend 14ft below existing ground surface grades. Concrete for foundations and site work shall be air-entrained normal weight stone concrete with a minimum compressive strength of 4000psi at 28 days and a maximum water to cement ratio of 0.45 by weight.

In the western portion of the six story faculty housing building footprint, it is recommended to excavate rock 12” below bottom of foundation in order to limit differential settlement between sections of the mat foundation bearing on rock and that bearing on soil.

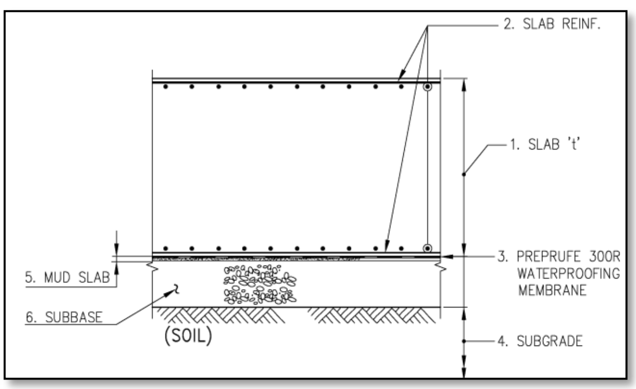


Figure 9: Mat Foudation Detail

Gravity System

Columns in the basement are 4000psi air-entrained concrete and vary in size from 32x48 to 36x60. The bay sizes vary from 30'x28', 30'x 28'2", 30'x31'5" and 30'x36' from north to south respectively.

All columns in the superstructure are W14s. Due to setbacks and varying story footprint, service loads carried by the columns at the ground level vary ranging from 137 to 1154kips. Because the service loads vary greatly throughout the floor, the column sizes vary as well; for example, on the ground floor column sizes range from w14x68 to w14x730. In the levels above the cellar, the bay sizes do not change.

There are non-composite beams as well as composite beams (with studs). Non-composite beams are found where beam to beam, and beam to column connections are designed to transfer the reaction for a simply supported, uniformly loaded beam. For composite beams, connections are designed to have 160% capacity of the reaction for a simply supported, uniformly loaded beam of the same size, span, fy, and allowable unit stress. For framed beam connections, including single plate connections, the minimum number of horizontal bolt rows should be provided based on 3" center-to-center.

Roof System

The roof is typically composed of 3 1/2 "light weight concrete over 3"-18 gage metal deck reinforced with 6x6-2.9x2.9 WWF. In a 200 square foot section the slab is 8" lightweight concrete slab reinforced with #4@12 top and bottom E.W. Columns are placed where needed and don't necessarily follow a typical framing layout. To provide additional vibration control, 4" concrete pads are located below mechanical equipment. Curbs on the roof are of CMU and concrete.

Floor System*- Composite steel beam and deck floor system*

The slab thickness for all floors is 3 ¼" thick 3500psi lightweight concrete placed over 3" deep 18 gage composite galvanized metal deck reinforced with 6x6- W2.9xW2.9 welded-wire-fabric. Exceptions on the ground floor are on the outdoor court, entry vestibules, and loading area; here 3" lightweight concrete is placed over 16 gage metal deck is used and instead of WWF, reinforcement is #4@12" o.c. top bars each way and 1-#5 bottom bars each rib. The exception for the second floor is the roof terrace where there is 5" of lightweight concrete over 3"-16 gage metal deck. On the roof level, the floor slab for the electrical control room is 8" lightweight concrete formed slab reinforced with to#4@12"o.c. top and bottom each way.

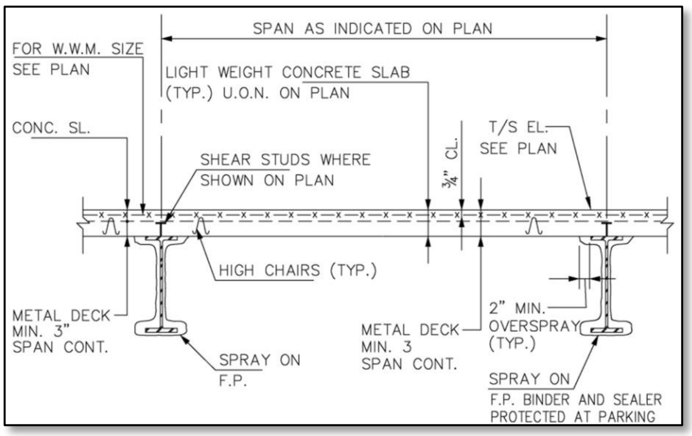


Figure 10. Typical Floor Construction, Metal Deck Perpendicular to Floor Beams on Girders

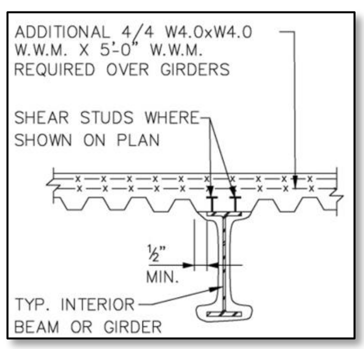


Figure 11. Typical Floor Construction, Metal Deck Parallel to Beams or Girders

Lateral System

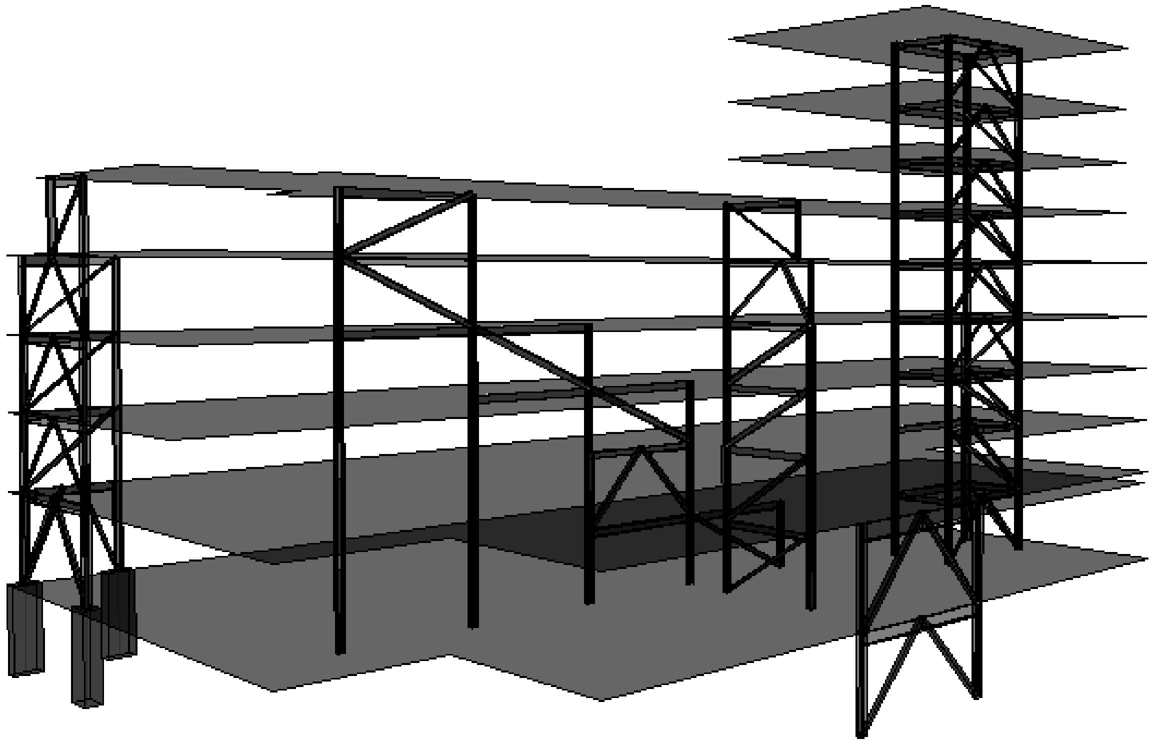


Figure 12. ETABS model of the Lateral Force Resisting System

The lateral system is made up of braced frames and moment frames. Braced frames with column splices at four feet above floor level with vertical members attached using moment connections make up the lateral system. Locations of these frames are represented on figure 2 in red; they run all the way up to the top of the building. The only exception to this is the braced frame represented on figure 2 as blue since it changes as you go up in elevation. An elevation view of this truss is shown as figure 3. Braced frames were chosen to resist lateral forces because they are more efficient than moment frames in both cost and erection time. The exceptions are the two moment frames used to surround the storm water detention tank. Moment frames provide unobstructed access to the tank that would not be possible if it was a braced frame. The other two frames surrounding the tank are in fact braced frames.

The remainder of this report further analyses the existing lateral force system. ETABS was used for the lateral analysis of Hunter College School of Social Work, and hand calculations were performed to verify results from the program output. Members of the braced frame and moment frame were checked for strength and drift requirements. Throughout this report, frames will be referred to in reference to their location as shown in figure 2.

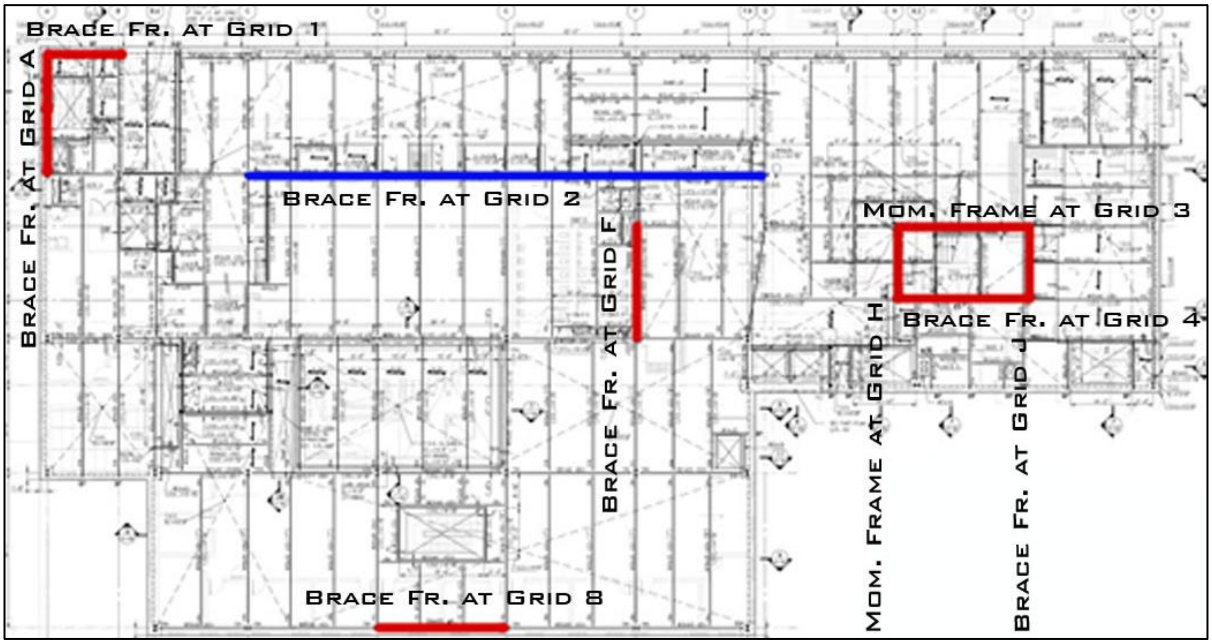


Figure 13. Location of Lateral Force Resisting Systems (Braced Frames)

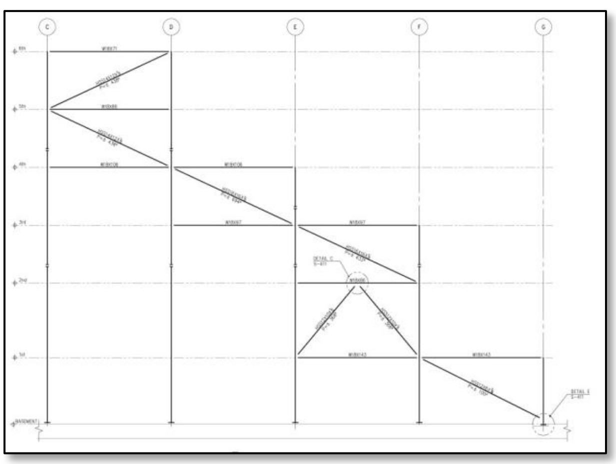


Figure 14. Truss Elevation at Grid 2

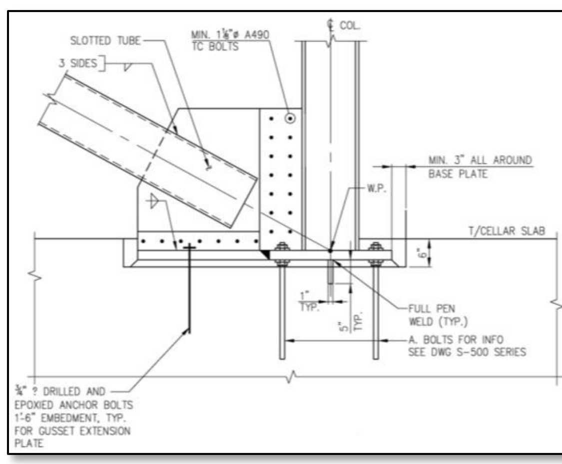


Figure 15. Lateral Load Resisting Detail

Lateral System in-depth Analysis

Relative Stiffness (Refer to relative stiffness tables located on the following page)

An ETABS model of the Hunter College School of Social Work lateral system was created for the determination of the relative stiffness of the frames in the lateral system. A 1000 kip force was placed at the center of mass of the roof diaphragm, in both the X and Y directions. The connections at the base were modeled as fixed connections because on average the mat foundation is three feet deep with an area of approximately 28, 130 square feet. Moments were released on the bracing members in the m_{33} direction. For the moment frames a reduced beam section was used in accordance with the program default because the moment frame design assumes 75% moment capacity. Rigid diaphragm mass definitions were assigned to every level in reference to the loading diagrams. The diaphragm definitions are presented in figure 5; for loading diagrams please see appendix. Section cuts were then taken at every story for every frame designed to resist the specified load, either X1000 or Y1000. Relative stiffness was determined based on how much of the 1000 kip load a frame member took with respect to the overall 1000 kip force. Gravity members were neglected for this analysis but were later accounted for in the building's weight for seismic analysis.

Story	Average weight per unit area	
	(psf)	(Kip-in)
Cellar	164	2.9474E-06
1	100	1.7972E-06
2	164	2.9474E-06
3	71	1.2760E-06
4	71	1.2760E-06
5	71	1.2760E-06
6	105	1.8871E-06
7	71	1.2760E-06
8	71	1.2760E-06
Roof	90	1.6175E-06

Figure 16. Diaphragm additional mass assignments on ETABS model

Load distribution for lateral loads in the X direction based on stiffness

At the three upper stories; stories eight, seven, and six, approximately eighty percent of the lateral force is resisted by the braced frame at grid 4 and the remaining force if resisted by the moment frame at grid 3. At the fifth story, forces go to braced frames at grids 2 and 4 while only a small portion goes to the moment frame at grid 3. Once the fourth story is reached, the braced frame at grid 1 is engaged. Half of the force goes to the braced frame at grid 2 at this story and the rest is split between braced frames at grids 1 and 4 with very little going to the moment frame at grid 3 (only 4 ½ %). At the third and second stories, forces primarily go to braced frame at grid 2 while the remaining force is distributed among braced frames at grids 1 and 4. Lastly, at the first story half the forces are taken by frame 2 while the rest is split up between frames 1, 4, and 8. Frame 8; which rises only to the top of story one, is now engaged and takes 30% of the load at this story.

Load distribution for lateral loads in the Y direction based on stiffness

The upper three stories distribute approximately sixty-six percent of the lateral force to the moment frame at grid H and the remaining force to the braced frame at grid J. At the fifth story shear reversal is encountered due to the vertical projection of only a 5290 square foot tower; generating stories six, seven, and eight. For stories one through five, story forces go primarily to the braced frame at grid J and the remaining goes to the frame at grid F.

Figure 17. Relative Stiffness for frames resisting X1000 and Y1000 lateral force

Eighth story					
Grid	X Force	% X	Grid	Y Force	% Y
1	0	0	A	0	0
2	0	0	H	-676	68
3	-175	17	F	0	0
4	-824	82	J	-322	32
8	0	0			0
total=	-999			-998	

Fourth story					
Grid	X Force	% X	Grid	Y Force	% Y
1	-178	18	A	45	-4
2	-572	57	H	-34	3
3	-45	5	F	-463	46
4	-203	20	J	-549	55
8	0	0			
total=	-999			-1000	

Seventh story					
Grid	X Force	% X	Grid	Y Force	% Y
1	0	0	A	0	0
2	0	0	H	-660	66
3	-210	21	F	0	0
4	-790	79	J	-338	34
8	0	0			
total=	-1000			-998	

Third story					
Grid	X Force	% X	Grid	Y Force	% Y
1	-87	9	A	45	-5
2	-832	83	H	-24	2
3	6	-1	F	-456	46
4	-88	9	J	-563	56
8	0	0			
total=	-1000			-1000	

Sixth story					
Grid	X Force	% X	Grid	Y Force	% Y
1	0	0	A	0	0
2	0	0	H	-660	66
3	-226	23	F	0	0
4	-774	77	J	-337	34
8	0	0			
total=	-1000			-997	

Second story					
Grid	X Force	% X	Grid	Y Force	% Y
1	-143	14	A	32	-3
2	-653	65	H	-2	0
3	-32	3	F	-397	40
4	-171	17	J	-636	64
8	0	0			
total=	-1000			-1000	

Fifth story					
Grid	X Force	% X	Grid	Y Force	% Y
1	0	0	A	-6	1
2	-770	77	H	150	-15
3	80	-8	F	-354	35
4	-311	31	J	-788	79
8	0	0			
total=	-1001			-999	

First story					
Grid	X Force	% X	Grid	Y Force	% Y
1	-95	9	A	-103	10
2	-479	48	H	-50	5
3	-22	2	F	-347	35
4	-105	10	J	-488	49
8	-297	30			
total=	-999			-998	

Expected load path for the computed lateral loads to the lateral resisting elements

By visual inspection, it is expected that the lateral load would be funneled towards the eight story tower in the north-east quadrant. Because three stories of the eight story tower are unshielded form the wind, deflection is expected to be an area of concern at higher levels. Since the 8 story tower will likely be the most rigid, the other trusses will be funneling the lateral forces towards it as seen by the braced frame at grid 2; which is a step-down truss towards the tower.

The building's south end is up against another building structure and is for the most part shielded from the wind. Therefore, not much lateral resistance is needed in the south end; this explains the scarcity of braced frames in this area. Only a two-story braced frame is provided for the south end; braced frame at grid 8.

In the northwest corner of the building, there are two braced frames forming an L-shaped lateral system, this corner where the L-shaped system is located, likely sees high wind forces head-on. The building's north face is on a third avenue; a wide avenue which can potentially create a wind tunnel effect due to the surrounding high-rises. In 2004 the worst case of wind in terms of wind speed for New York City was found to be in the month January. Figure 7 shows the wind rose for this month (courtesy of the Natural Resources Conservation Service). The wind rose shows the highest winds coming from the north-west quadrant.

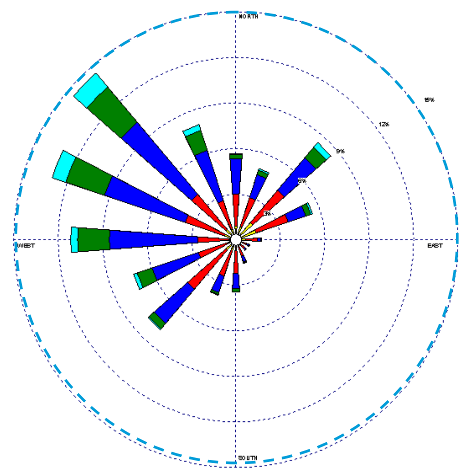


Figure 18. Wind rose for NYC Jan 2003

Z=0.15
I=1.25
R_w = (N-S Plan direction-braced frames) =8
R_w = (E-W Plan direction-braced frames) =8
Site Coefficient=1.0
Soil Profile=S_1

According to the geotechnical report, the soil has no potential for liquefaction on the building site. For the purpose of this report, ASCE7-05 was used in determining seismic forces although for the actual design of the lateral system seismic forces were calculated in accordance with the NYC building code using the factors shown above. For seismic design forces please refer to appendix x.

Shown on the following page is a schematic diagram of the possible load paths for the computed loads to the lateral resisting elements. These schematic was derived from the relative stiffness of the elements calculated from the ETABS output (see previous page). The load path determined through ETABS was consistent with what was initially expected. However, at lower stories, the x-direction forces tend to travel down the frame at grid 1 instead of going down the step-down braced frame at grid 2.

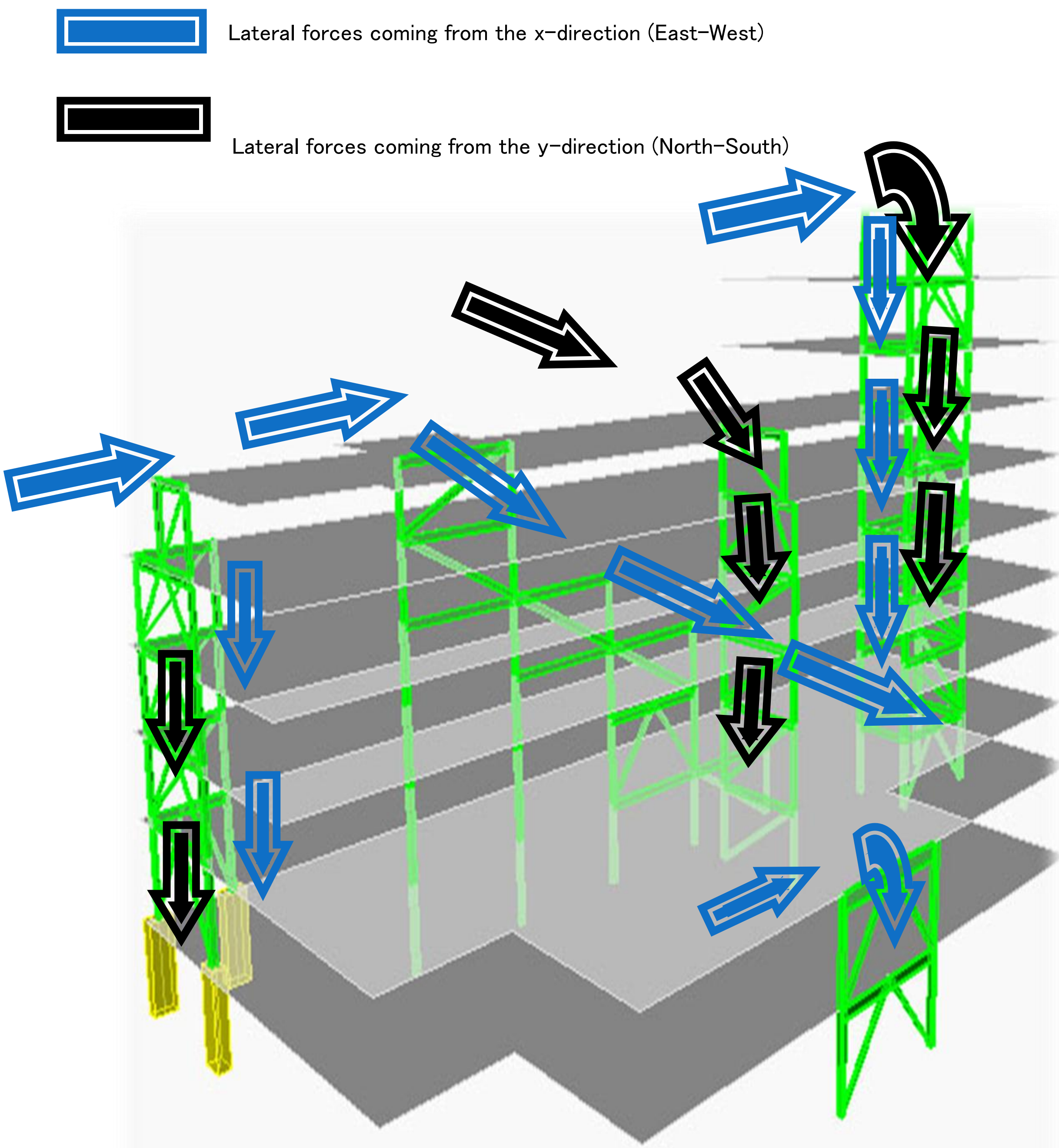


Figure 19. Schematic diagram of the load path for the lateral system

Lateral forces in the East- West direction are resisted by four braced frames and one moment frame. The frames are tied to the rigid floor diaphragms which are composed of a 6 ¼" composite steel deck. The columns then carry the load down the building until it reaches the foundation at the cellar level. Forces in the foundation are then absorbed by the soil. Lateral forces in the North- South direction are resisted by three braced frames and one moment frame. Forces in this direction undergo a similar path to forces in the east-west direction. Refer to figure 8 for a schematic diagram of the load path for the lateral forces.

Determination of the Center of Rigidity

Two methods were used to check against the center of rigidity coordinates determined by ETABS. The first method used SAP2000 for stiffness values while the second used ETABS for stiffness values. With the use of SAP2000, stiffness values were determined for each lateral system element by applying a one kip lateral load at the fourth story and taking the inverse of the resulting displacement at that level. The corresponding x and y coordinates of the center of rigidity were calculated using the following equations.

$$\overline{x} = \frac{\sum k_{iy}\, x_i}{\sum k_{iy}} \quad ; \quad \overline{y} = \frac{\sum k_{ix}\, y_i}{\sum k_{ix}}$$

For this first method, the center of rigidity was found to be at coordinates (79.2, 98.0) feet. Comparing this set of coordinates with the ETABS output, it is evident that there is a large gap of error. This error may be due to the neglecting of the center of rigidity effects of floors above and below story four.

Story four- Approximate COR Check using SAP2000 relative stiffness values				
Frame (dir)	Load Applied in Diaphragm (kips)	Displacement (in.)	Stiffness	Distance to Origin (ft)
1 (E-W)	1	0.01	105.26	132.5
2 (E-W)	1	0.00	227.27	104.5
3 (E-W)	1	0.00	238.10	92.5
4 (E-W)	1	0.00	625.00	75.4
8 (E-W)	1	0.00	0.00	0
A (N-S)	1	0.00	277.78	0
F (N-S)	1	0.01	142.86	136.5
H (N-S)	1	0.10	10.03	196.5
J (N-S)	1	0.01	161.29	226.5
Center of Rigidity in the x-direction:			79.2 ft compare to	113 ft
Center of Rigidity in the y-direction:			89 ft compare to	88 ft

Figure 20. Center of Rigidity values calculated using SAP2000

In ETABS; used for second method, wind forces calculated in accordance with ASCE 7-05 were applied in both directions at the center of pressure for each story. Section cuts were then taken at the fourth story on every lateral frame. Relative stiffness was determined based on the percentage of the total lateral load taken by the individual frames. The above equations for the center of rigidity was applied once again to obtain the values of (169.5, 83.5) feet. Although it was expected that this method would provide more accurate results, it did not, due to an unknown error. This same procedure was repeated was levels two and five, resulting in discrepancies between the calculated center of rigidity and the expected value.

Story four- Approximate COR Check using ETABS relative stiffness values				
Frame (dir)	Load Applied in Diaphragm (kips)	Distribution (kips)	Percentage	Distance to Origin (ft)
1 (E-W)	321	41.00	0.13	132.5
2 (E-W)	321	165.31	0.51	104.5
3 (E-W)	321	10.54	0.03	92.5
4 (E-W)	321	103.01	0.32	75.4
8 (E-W)	321	0.00	0.00	0
A (N-S)	94	9.95	0.11	0
F (N-S)	94	33.63	0.36	136.5
H (N-S)	94	2.75	0.03	196.5
J (N-S)	94	47.84	0.51	226.5
Center of Rigidity in the x-direction:			169.54 ft compare to	113 ft
Center of Rigidity in the y-direction:			83.45 ft compare to	88 ft

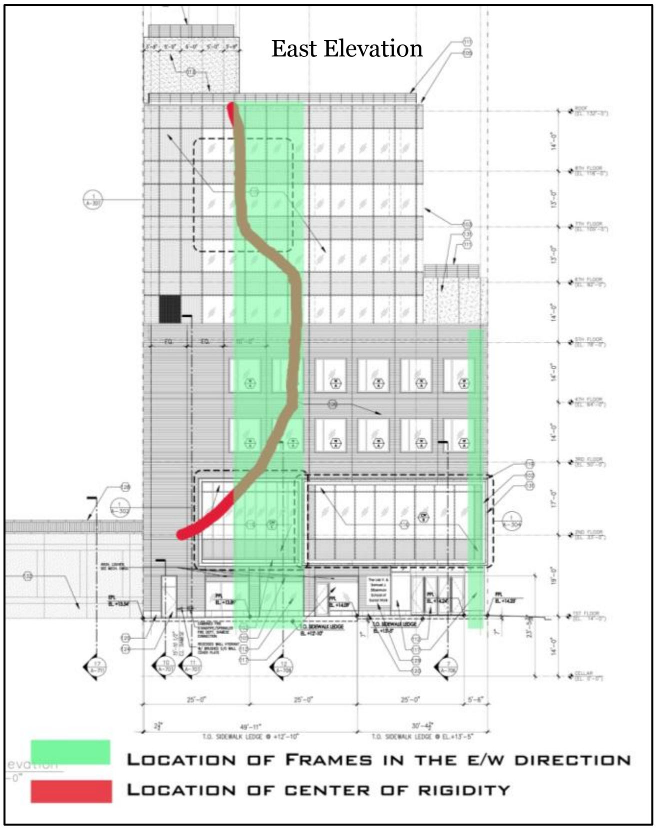
Figure 21. Center of Rigidity values calculated using ETABS

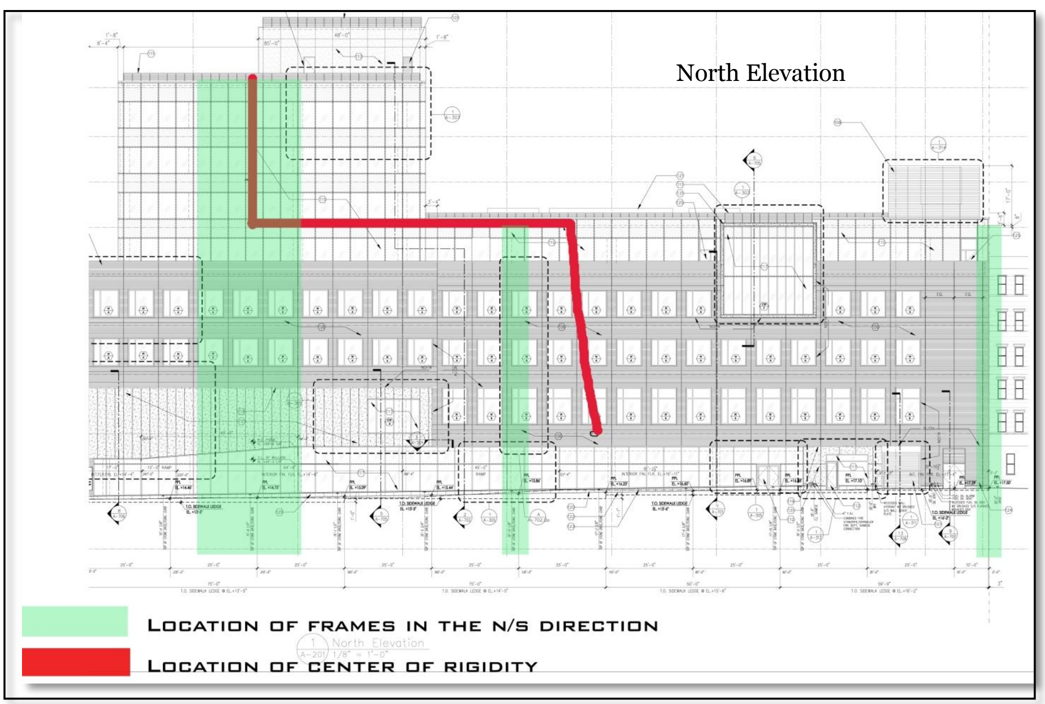
ETABS output for center of rigidity; shown in figure 22, takes into account the center of rigidities of levels above and below. As is shown in the table, there is a lot of changes in the y direction due to the various setbacks in the north south direction of the building. The x coordinates do not change as often as you go up in elevation because the only setback in the east-west direction occurs at the sixth story to seventh story transition where the building only a 5,290 square foot section (out of a total 28,130 square feet) of the building continues up the next three stories. A schematic diagram of the location of the center of rigidity for various buildings levels is shown as figure 23. The locations of the center of rigidities for the diagram were taken from the table presented below.

Center of Rigidity Calculated by ETABS		
Story	XCR	YCR
ROOF	216.733	74.103
STORY8	215.114	74.69
STORY7	210.446	75.703
STORY6	123.542	87.87
STORY5	112.238	89.533
STORY4	112.872	88.042
STORY3	114.427	81.942
STORY2	115.889	67.32
STORY1	n/a	n/a

Figure 22. Center of Rigidity output from ETABS

Figure 23. Schematic diagram of the location of the center of rigidity due to the lateral system





Load Combinations applied to ETABS model

The following load combinations were taken from chapter two of ASCE 7-05. These combinations were all used in the ETABS model. For wind, the four cases in figure 24 were considered. For a more detailed description of cases two and three, please refer to pages 25 and 26. Wind loads are applied at the center of pressure of each level and seismic loads were placed at the center of mass of each level. Seismic load values were placed at an eccentricity of 0.05 as determined by code. Live and dead loads used can be found on the loading diagrams in the appendix. They were added as a uniform load on the diaphragms.

1. 1.4 (Dead)
2. 1.2 (Dead) +1.6 (Live) + 0.5 (Roof Live)
3. 1.2 (Dead) +1.6(Roof Live) + (1.0(Live) or 0.8(Wind))
4. 1.2 (Dead) +1.6 (Wind) +1.0(Live) +0.5(Roof Live)
5. 1.2 (Dead) +1.0 (Seismic) + 1.6(Wind)
6. 0.9 (Dead) + 1.6(Wind)
7. 0.9(Dead) + 1.0 (Seismic)

Wind Load Cases Applied

Case 1: 100 percent of the wind forces in the east-west direction or

100 percent of the wind forces in the north-south direction

Case 2: 75 percent of the east-west or north-south wind forces applied with torsion

Case 3: 75 percent of the east-west and north-south wind forces applied simultaneously

Case 4: 56.3 percent of the east-west and north-south wind forces applied simultaneously with torsion

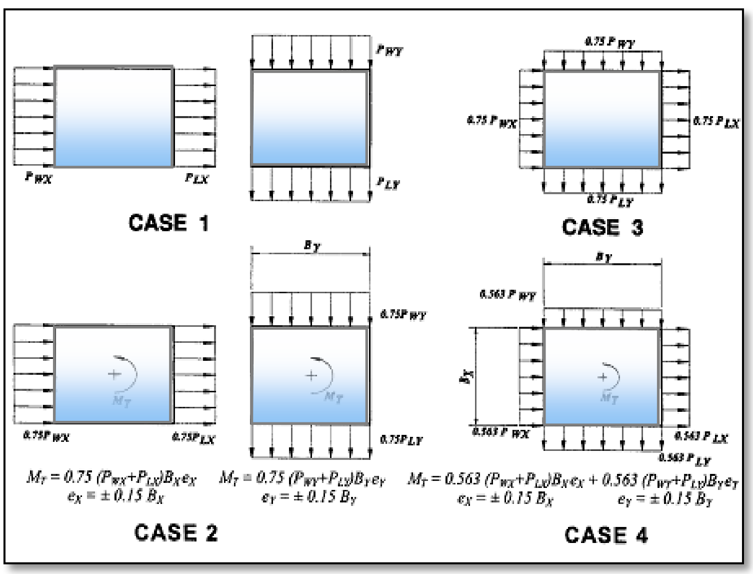
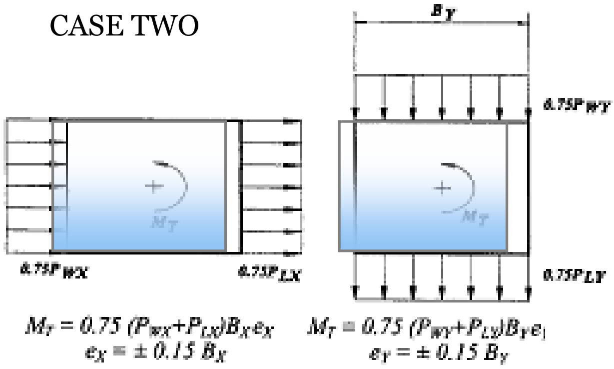


Figure 24. Wind cases for method 2from ASCE7-05 figure 6-9

Figure 25. Effective coordinates for wind case 2



Story	Bx (in.)	ex (in.)	By (in.)	ey (in.)
1	1590	238.5	3078	461.7
2	1590	238.5	3078	461.7
3	924	138.6	3078	461.7
4	924	138.6	3078	461.7
5	924	138.6	3078	461.7
6	924	138.6	2957	443.55
7	745	111.75	1008	151.2
8	745	111.75	1008	151.2
roof	745	111.75	1008	151.2

CASE 2north south __POS e				
STORY	FX	FY	XCCOR	YCCOR
ROOF	0	15.75	2453	1155.75
8	0	9	2453	1155.75
7	0	14.25	2453	1155.75
6	0	12.75	1463.5	1266.6
5	0	12.75	1524	1266.6
4	0	12.75	1524	1266.6
3	0	12	1524	1266.6
2	0	12	1524	1033.5
1	0	11.25	1524	1033.5

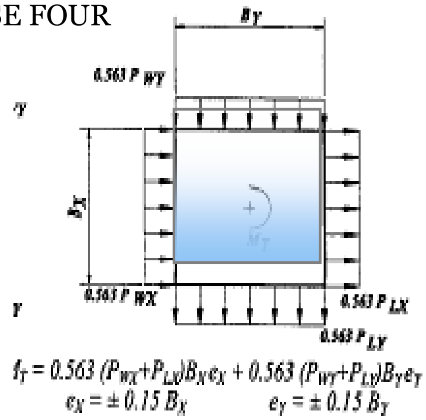
CASE 2north south__NEG e				
STORY	FX	FY	XCCOR	YCCOR
ROOF	0	15.75	2453	932.25
8	0	9	2453	932.25
7	0	14.25	2453	932.25
6	0	12.75	1463.5	989.4
5	0	12.75	1524	989.4
4	0	12.75	1524	989.4
3	0	12	1524	989.4
2	0	12	1524	556.5
1	0	11.25	1524	556.5

CASE 2eastwest__POS e				
STORY	FX	FY	XCCOR	YCCOR
ROOF	53.25	0	2604.2	1044
8	51	0	2604.2	1044
7	48	0	2604.2	1044
6	44.25	0	1907.05	1128
5	44.25	0	1985.7	1128
4	43.5	0	1985.7	1128
3	40.5	0	1985.7	1128
2	40.5	0	1985.7	795
1	39	0	1985.7	795

CASE 2eastwest__NEG e				
STORY	FX	FY	XCCOR	YCCOR
ROOF	53.25	0	2301.8	1044
8	51	0	2301.8	1044
7	48	0	2301.8	1044
6	44.25	0	1019.95	1128
5	44.25	0	1062.3	1128
4	43.5	0	1062.3	1128
3	40.5	0	1062.3	1128
2	40.5	0	1062.3	795
1	39	0	1062.3	795

Figure 26. Effective coordinates for wind case 4

CASE FOUR



Story	Bx (in.)	ex (in.)	By (in.)	ey (in.)
1	1590	238.5	3078	461.7
2	1590	238.5	3078	461.7
3	924	138.6	3078	461.7
4	924	138.6	3078	461.7
5	924	138.6	3078	461.7
6	924	138.6	2957	443.55
7	745	111.75	1008	151.2
8	745	111.75	1008	151.2
roof	745	111.75	1008	151.2

CASE 4NORTHSOUTH__ex pos ey pos				
STORY	FX	FY	XCCOR	YCCOR
ROOF	39.973	11.823	2604.2	1155.75
8	38.284	6.756	2604.2	1155.75
7	36.032	10.697	2604.2	1155.75
6	33.217	9.571	1907.05	1266.6
5	33.217	9.571	1985.7	1266.6
4	32.654	9.571	1985.7	1266.6
3	30.402	9.008	1985.7	1266.6
2	30.402	9.008	1985.7	1033.5
1	29.276	8.445	1985.7	1033.5

CASE 4NORTHSOUTH__ex neg ey neg				
STORY	FX	FY	XCCOR	YCCOR
ROOF	39.973	11.823	2301.8	932.25
8	38.284	6.756	2301.8	932.25
7	36.032	10.697	2301.8	932.25
6	33.217	9.571	1019.95	989.4
5	33.217	9.571	1062.3	989.4
4	32.654	9.571	1062.3	989.4
3	30.402	9.008	1062.3	989.4
2	30.402	9.008	1062.3	556.5
1	29.276	8.445	1062.3	556.5

CASE 4NORTHSOUTH__ex pos ey neg				
STORY	FX	FY	XCCOR	YCCOR
ROOF	39.973	11.823	2301.8	1155.75
8	38.284	6.756	2301.8	1155.75
7	36.032	10.697	2301.8	1155.75
6	33.217	9.571	1019.95	1266.6
5	33.217	9.571	1062.3	1266.6
4	32.654	9.571	1062.3	1266.6
3	30.402	9.008	1062.3	1266.6
2	30.402	9.008	1062.3	1033.5
1	29.276	8.445	1062.3	1033.5

CASE 4NORTHSOUTH__ex neg ey pos				
STORY	FX	FY	XCCOR	YCCOR
ROOF	39.973	11.823	2604.2	932.25
8	38.284	6.756	2604.2	932.25
7	36.032	10.697	2604.2	932.25
6	33.217	9.571	1907.05	989.4
5	33.217	9.571	1985.7	989.4
4	32.654	9.571	1985.7	989.4
3	30.402	9.008	1985.7	989.4
2	30.402	9.008	1985.7	556.5
1	29.276	8.445	1985.7	556.5

The controlling load combination in terms of story drift and story shear for both directions was load combination four:

1.2 (Dead) +1.6 (Wind) +1.0(Live) +0.5(Roof Live)

The wind case that controlled was case 1, where 100 percent of the wind load is applied in the north/south direction. Shown in figure 27 are the story shears due to the controlling load combination. At the first story, the step-down braced frame at grid 2 the largest percentage of the shear force while at the uppermost story, the largest fraction of the shear force is taken by the braced frame at grid 4. This is expected because the step-down truss does not continue onto stories six to eight.

Story	East-West Frames : Shear (kips)					North South Frames : Shear (kips)			
	At Grid 1	At Grid 2	At Grid 3	At Grid 4	At Grid 8	At Grid A	At Grid F	At Grid H	At Grid J
8	0.00	0.00	25.86	87.59	0.00	0.00	0.00	28.84	-28.87
7	0.00	0.00	73.61	262.15	0.00	0.00	0.00	87.02	-87.15
6	0.00	0.00	143.44	517.03	0.00	0.00	0.00	172.03	-172.37
5	0.00	271.65	122.12	686.13	0.00	6.24	-15.93	104.46	-95.31
4	64.75	537.72	138.99	851.69	0.00	1.64	-54.79	108.79	-55.91
3	114.01	955.63	136.92	993.48	0.00	-8.77	-91.62	98.95	1.08
2	213.36	1338.06	155.65	1184.86	0.00	-30.04	-109.46	94.64	37.08
1	277.69	1673.97	175.13	1326.42	216.62	28.79	-120.86	86.93	-4.23

Figure 27. Story Shear for Frames under the controlling load combination

Serviceability check – Wind Drifts

Story drift values were determined by applying Case 1 wind load case according to ASCE 7-05. Case 1 was the controlling case in both East-West and North-South directions. These drift values were computed by ETABS and checked against the allowable drift of H/400. This allowable drift is simply a serviceability criteria meant to keep occupants feeling at ease by preventing excessive sway. Due to the structure's low total building height relative to typical building heights in New York City, it was expected that the actual story drift would be very small. All drift values were deemed acceptable according to the drift limitation of H/400.

Controlling Wind Drift: East-West Direction										
Story	Story Height (ft)	Story Drift (in.)	Allowable Story Drift Δ_{wind}=H/400 (in.)			Total Drift (in.)	Allowable Total Drift Δ_{wind}=H/400 (in.)			
8	118	0.004915	<	0.29500	Acceptable	0.020761	<	1.40000	Acceptable	
7	104	0.005805	<	0.26000	Acceptable	0.015846	<	1.10500	Acceptable	
6	91	0.005454	<	0.22750	Acceptable	0.010041	<	0.84500	Acceptable	
5	78	0.001129	<	0.19500	Acceptable	0.004587	<	0.61750	Acceptable	
4	64	0.001007	<	0.16000	Acceptable	0.003458	<	0.42250	Acceptable	
3	50	0.000827	<	0.12500	Acceptable	0.002451	<	0.26250	Acceptable	
2	36	0.000927	<	0.09000	Acceptable	0.001624	<	0.13750	Acceptable	
1	19	0.000697	<	0.04750	Acceptable	0.000697	<	0.04750	Acceptable	

Figure 28. Allowable wind drifts in the East-West direction

Controllind Wind Drift: North-South Direction									
Story	Story Height (ft)	Story Drift (in.)	Allowable Story Drift Δ_{wind}=H/400 (in.)			Total Drift (in.)	Allowable Total Drift Δ_{wind}=H/400 (in.)		
8	118	0.004835	<	0.29500	Acceptable	0.018972	<	1.40000	Acceptable
7	104	0.005807	<	0.26000	Acceptable	0.014137	<	1.10500	Acceptable
6	91	0.005118	<	0.22750	Acceptable	0.008330	<	0.84500	Acceptable
5	78	0.000725	<	0.19500	Acceptable	0.003212	<	0.61750	Acceptable
4	64	0.000733	<	0.16000	Acceptable	0.002487	<	0.42250	Acceptable
3	50	0.000694	<	0.12500	Acceptable	0.001754	<	0.26250	Acceptable
2	36	0.000602	<	0.09000	Acceptable	0.001060	<	0.13750	Acceptable
1	19	0.000458	<	0.04750	Acceptable	0.000458	<	0.04750	Acceptable

Figure 29. Allowable wind drifts in the North-South direction

Seismic drifts

Seismic drift values were determined by applying the seismic forces determined in technical report 1. Unlike the wind drift requirements, seismic drift is not a serviceability requirement, it is a requirement that protects against building collapse. The limitation was taken to be $\Delta_{seismic}=0.015h_{sx}$ (in.) based on ASCE 7-05. As is shown in the following tables, seismic drift was acceptable at all story levels in both East-West and North-South directions.

Seismic Drift: East-West Direction									
Story	Story Height (ft)	Story Drift (in.)	Allowable Story Drift $\Delta_{seismic}=0.015h_{sx}$ (in.)			Total Drift (in.)	Allowable Total Drift $\Delta_{seismic}=0.015h_{sx}$ (in.)		
8	118	0.006357	<	1.77000	Acceptable	0.021403	<	8.40000	Acceptable
7	104	0.00674	<	1.56000	Acceptable	0.015046	<	6.63000	Acceptable
6	91	0.00553	<	1.36500	Acceptable	0.008306	<	5.07000	Acceptable
5	78	0.000686	<	1.17000	Acceptable	0.002776	<	3.70500	Acceptable
4	64	0.000611	<	0.96000	Acceptable	0.00209	<	2.53500	Acceptable
3	50	0.000544	<	0.75000	Acceptable	0.001479	<	1.57500	Acceptable
2	36	0.000567	<	0.54000	Acceptable	0.000935	<	0.82500	Acceptable
1	19	0.000368	<	0.28500	Acceptable	0.000368	<	0.28500	Acceptable

Figure 30. Allowable seismic drift in the East-West direction

Seismic Wind Drift: North-South Direction									
Story	Story Height (ft)	Story Drift (in.)	Allowable Story Drift $\Delta_{seismic}=0.015h_{sx}$ (in.)			Total Drift (in.)	Allowable Total Drift $\Delta_{seismic}=0.015h_{sx}$ (in.)		
8	118	0.00916	<	1.77000	Acceptable	0.032258	<	8.40000	Acceptable
7	104	0.009777	<	1.56000	Acceptable	0.023098	<	6.63000	Acceptable
6	91	0.008041	<	1.36500	Acceptable	0.013321	<	5.07000	Acceptable
5	78	0.001219	<	1.17000	Acceptable	0.00528	<	3.70500	Acceptable
4	64	0.001215	<	0.96000	Acceptable	0.004061	<	2.53500	Acceptable
3	50	0.001158	<	0.75000	Acceptable	0.002846	<	1.57500	Acceptable
2	36	0.001034	<	0.54000	Acceptable	0.001688	<	0.82500	Acceptable
1	19	0.000654	<	0.28500	Acceptable	0.000654	<	0.28500	Acceptable

Figure 31. Allowable seismic drift in the North-South direction

Overturning analysis and foundation impact

Overturning moment due to seismic loads is counteracted by the dead load of the building's weight. However, when this is not enough, additional measures need to be taken to resist this moment. Designing the foundation to assist in counteracting the overturn is a popular way to do this.

Values for overturning moment were calculated by multiplying the base shear by the frame height relative to ground level. Overturning was found to be resisted by all frames except the five-story braced frame at grid 1. This indicates an impact on the foundation. However, since seismic forces used were those determined using ASCE 7-05, they do not accurately represent the values used by the structural engineer. It is very possible that a "no impact on foundation" conclusion was found by the structural engineer.

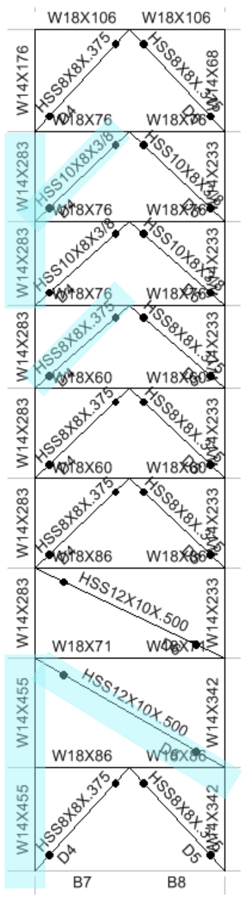
Story	East-West Frames : Forces (kips)					North South Frames : Forces (kips)				Total Story Shear (kips)
	At Grid 1	At Grid 2	At Grid 3	At Grid 4	At Grid 8	At Grid A	At Grid F	At Grid H	At Grid J	
8	0.00	0.00	25.86	87.59	0.00	0.00	0.00	28.84	-28.87	113.41
7	0.00	0.00	47.75	174.56	0.00	0.00	0.00	58.18	-58.28	222.21
6	0.00	0.00	69.83	254.88	0.00	0.00	0.00	85.01	-85.22	324.50
5	0.00	271.65	-21.32	169.10	0.00	6.24	-15.93	-67.57	77.06	419.23
4	64.75	266.07	16.87	165.57	0.00	-4.60	-38.86	4.34	39.40	513.54
3	49.26	417.91	-2.07	141.78	0.00	-10.41	-36.83	-9.85	56.99	606.78
2	99.35	382.43	18.73	191.38	0.00	-21.27	-17.84	-4.31	36.00	684.47
1	64.33	335.91	19.49	141.56	216.62	58.83	-11.40	-7.71	-41.31	776.32

Figure 32. Story Forces due to Controlling load combination

	East-West Frames : Forces (kips)					North South Frames : Forces (kips)			
	At Grid 1	At Grid 2	At Grid 3	At Grid 4	At Grid 8	At Grid A	At Grid F	At Grid H	At Grid J
Overturning Moment (ft-k)	9856	12012	18480	18480	2926	12012	12012	18480	18480
Base Dimension (ft)	16.5	120	30	30	30	28	26	17	17
Force at edge column (k)	597.3	100.1	616	616	97.5	429	462	1087.1	1087.1
Edge Column DL (k)	430	1010	1390	1240	265	530	750	1300	1390
Overturning	NG	OK	OK	OK	OK	OK	OK	OK	OK

Figure 33. Story forces and Overturning Analysis

Lateral member spot checks



Member spot checks were performed for braced frame at grid 4. Figure 33 displays the members which were checked for strength by attaining forces from the ETABS model. Gravity loads were not accounted for in the ETABS model, so a gravity load takedown was performed for each column in the lateral system (a gravity load takedown is available upon request).

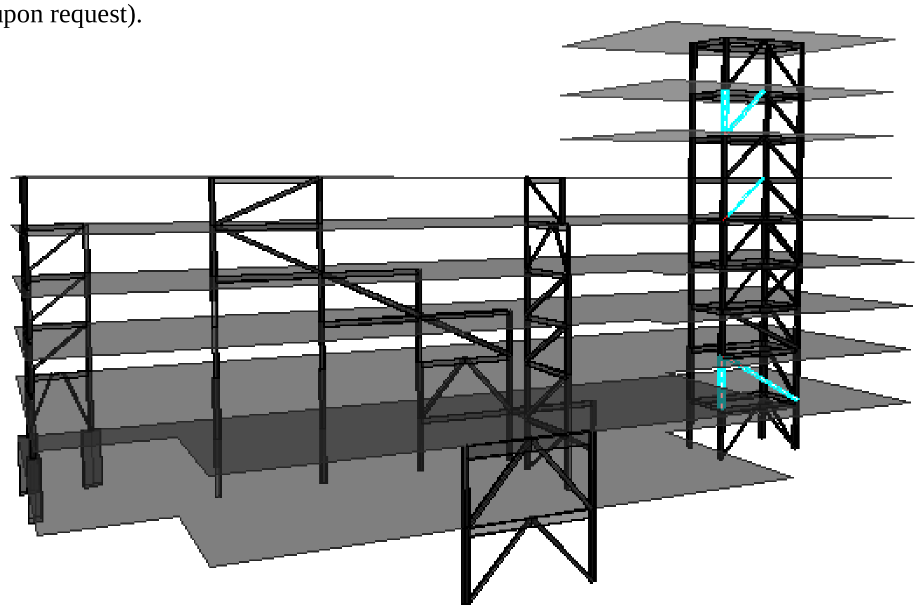


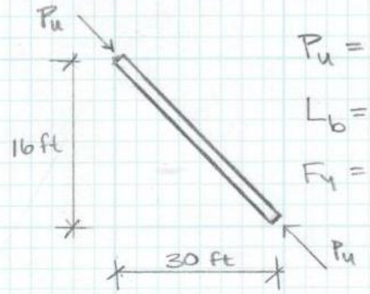
Figure 33. Story Forces due to Controlling load combination

The controlling load combination was used in determining the axial forces in the bracing members. As seen in the table below, the HSS members were more than adequate in resisting the controlling lateral forces. This indicates that these members were probably sized to resist drift rather than for strength. The column member were found to be only thirty percent stressed, indicating that they were not sized for strength requirements, but instead may have been sized for drift as well as the bracing members.

Lateral Elem.	Location	Eff. Length (ft)	Pu (kips)	Mu (ft-kips)	Pn (kips)	Acceptable?
Hss12x10x1/2	Level 1	34.00	133.0	-	386.0	YES
Hss8x8x3/8	Level 5	21.00	83.7	-	275.0	YES
Hss10x8x3/8	Level 7	20.00	85.7	-	333.0	YES
W14x455	Level 1	26.72	1193.0	22.0	-	YES
W14x283	Level 7	21.19	546.0	70.0	-	YES

Figure 34. Summary of the Lateral Member Spot Checks

BRACE MEMBER AT LEVEL 1 HSS 12 × 10 × 1/2



$P_u = 133\,K \quad (1.2D + 1.6W + 1.0L + 0.5L_r)$

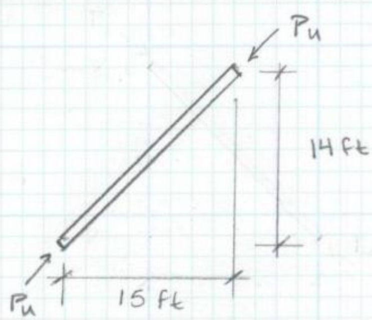
$L_b = \sqrt{16^2 + 30^2} = 34\ ft$

$F_y = 42\ ksi$

FROM TABLE 4-3 : $\phi P_n = 386$ kips $> P_u \therefore OK$

$$\frac{P_u}{\phi P_n} = \frac{133}{386} = 0.34 < 1.0 \therefore OK$$

BRACE MEMBER AT LEVEL 5 HSS 8×8 × 3/8



$P_u = 83.8\,K \quad (1.2D + 1.6W + 1.0L + 0.5L_r)$

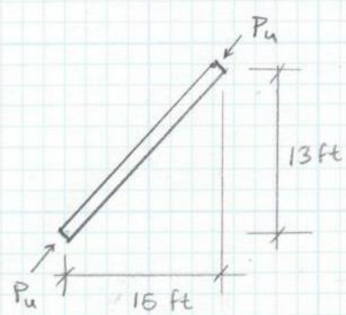
$L_b = 21\ ft$

$F_y = 42\ ksi$

FROM TABLE 4-4 : $\phi P_n = 275$ kips $> P_u \therefore OK$

$$\frac{P_u}{\phi P_n} = \frac{83.8}{275} = 0.30 < 1.0 \therefore OK$$

BRACE MEMBER AT LEVEL 7 HSS 10 × 8 × 3/8



$P_u = 85.7\,K \quad (1.2D + 1.6W + 1.0L + 0.5L_r)$

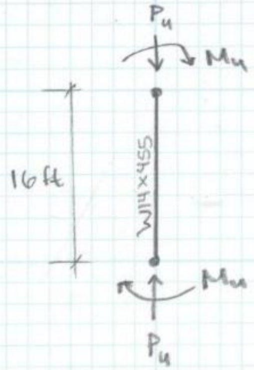
$L_b = 20\ ft$

$F_y = 42\ ksi$

FROM TABLE 4-3 : $\phi P_n = 333\,K > P_u \therefore OK$

$$\frac{P_u}{\phi P_n} = \frac{85.7}{333} = 0.26 < 1.0 \therefore OK$$

COLUMN MEMBER AT LEVEL 1 W14X455



$P_u = 1193$ Kips

$M_{uy} = 22$ ft-kips $(1.2D + 1.6W + 1.0L + 0.5L_r)$

FROM TABLE 6-1

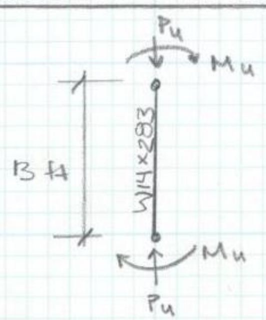
$KL = (16)\frac{r_x}{r_y} = (16)(1.67) = 26.72$ ft

$p \times 10^{-3} = 0.247$

$b_x \times 10^{-3} = 0.261$

$$\frac{1193(0.247)}{1000} + \frac{22(0.261)}{1000} = 0.30 < 1.0 \therefore OK$$

COLUMN MEMBER AT LEVEL 7 W14X283



$P_u = 546$ kips

$M_{uy} = 70$ ft-kips

FROM TABLE 6-1

$KL = (13)(1.63) = 21.19$ ft

$p \times 10^{-3} = 0.349$

$b_x \times 10^{-3} = 0.449$

$$\frac{(546)(0.349)}{1000} + \frac{(70)(0.449)}{1000} = 0.22 < 1.0 \therefore OK$$

Conclusions

Lateral forces used throughout this report were obtained in technical report 1 following ASCE 7-05. Because the structure is located in New York, the New York Building Code was used in the design of the lateral system. The controlling wind case was found to be case 1 and the controlling load combination was
1.2 (Dead) +1.6 (Wind) +1.0(Live) +0.5(Roof Live). In calculating wind and seismic drifts, the unfactored wind loads from case 1 and the unfactored seismic loads were applied respectively. The controlling load combination was used in lateral spot checks and story shear determination.

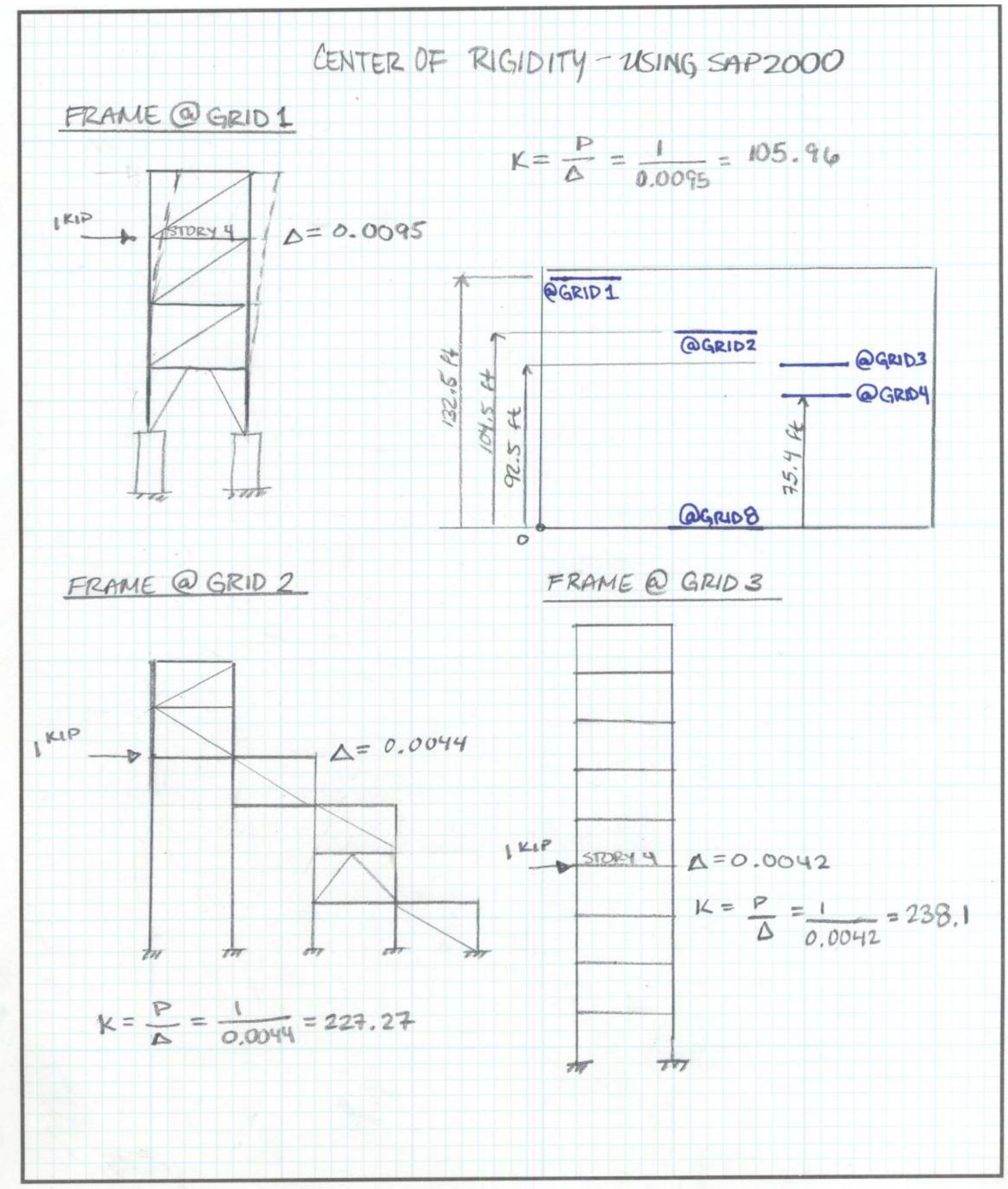
Drift limits were found to be well above the actual drift calculated through ETABS for both wind and seismic loads. This was expected because the building is short and stocky. Drift values for wind were serviceability requirements taken to be a max of H/400. Unlike the wind drift requirements, seismic drift is not a serviceability requirement, and the limitation was taken to be $\Delta_{seismic}=0.015h_{sx}$ (in.) based on ASCE 7-05.

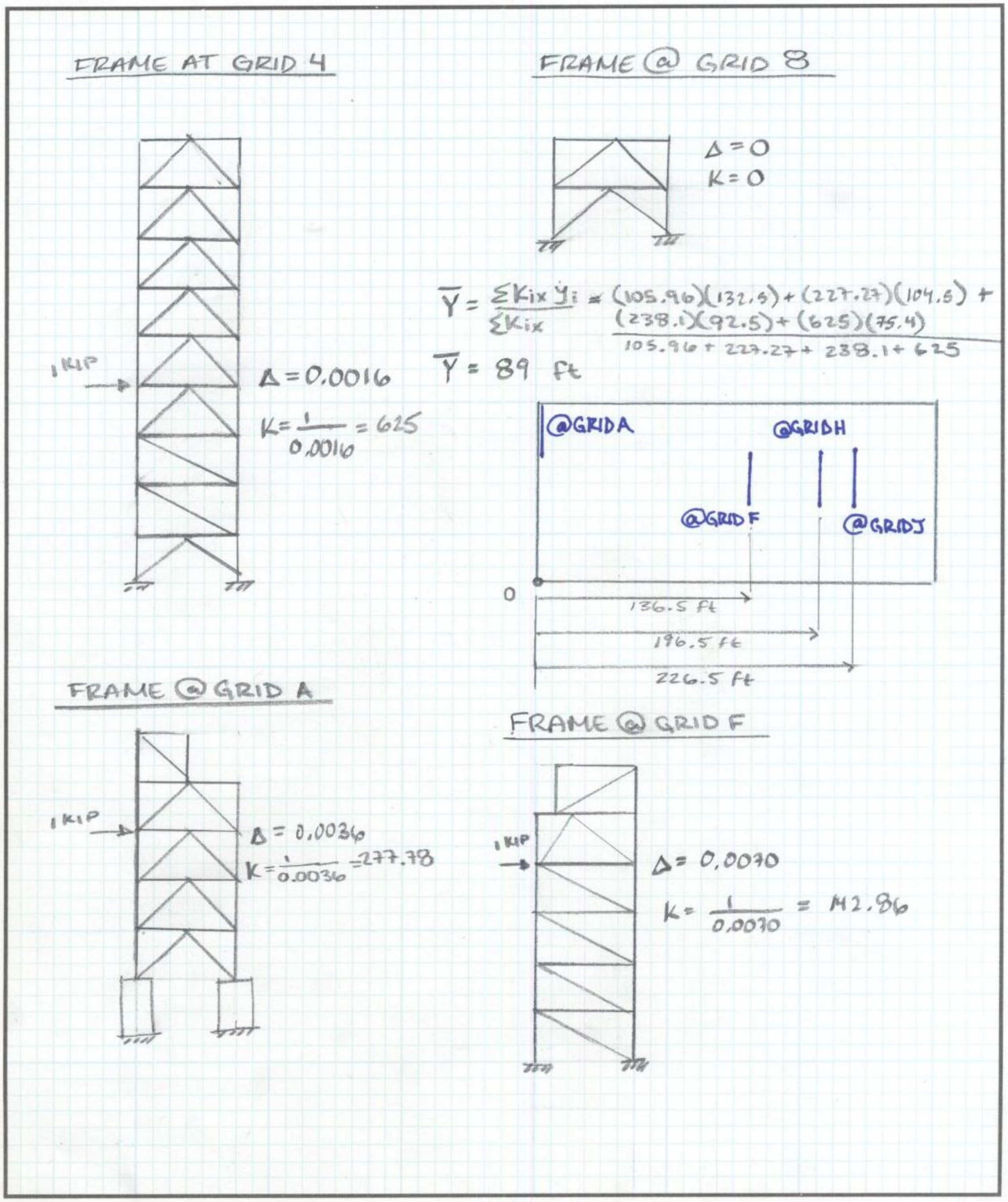
Overturning was found to be resisted by all frames except the five-story braced frame at grid 1. This indicates an impact on the foundation. However, since seismic forces used were those determined using ASCE 7-05, they do not accurately represent the values used by the structural engineer. It is very possible that a “no impact on foundation” conclusion was found by the structural engineer.

Strength spot checks of three braces and two columns which were elements of the frame at grid 4 were found to be adequate. These members experience approximately thirty percent of the stresses they can actually withstand. This leads to the conclusion that they were not sized for strength requirements. It is unclear what controlled the design of these members, further investigation is needed to determine this. A possibility is that induced moments were not taken into account when checking strength requirements.

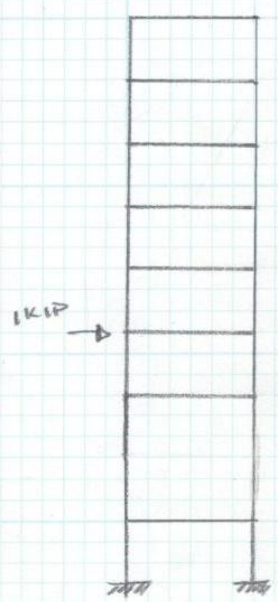
Appendix A – Calculations

Center of Rigidity Checks using SAP2000




FRAME AT GRID 4
1 KIP
Δ=0.0016
K= 1/0.0016 = 625
FRAME @ GRID 8
Δ=0
K=0
Ȳ = ΣKix yi / ΣKix = (105.96)(132.5)+(227.27)(104.5)+(238.1)(92.5)+(625)(75.4) / 105.96+227.27+238.1+625
Ȳ = 89 ft
@GRID A
@GRID H
@GRID F
@GRID J
0
136.5 ft
196.5 ft
226.5 ft
FRAME @ GRID A
1 KIP
Δ = 0.0036
K = 1/0.0036 = 277.78
FRAME @ GRID F
1 KIP
Δ= 0.0070
k= 1/0.0070 = 142.86

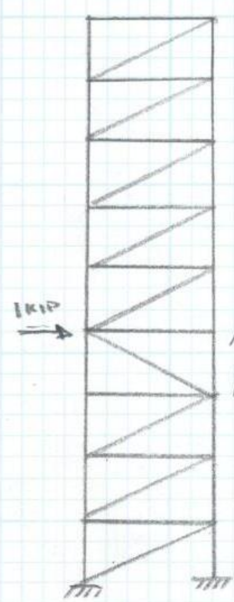
FRAME @ GRID H



$\Delta = 0.0997$

$k = \frac{1}{0.0997} = 10.03$

FRAME @ GRID J



$\Delta = 0.0062$

$K = \frac{1}{0.0062} = 161.29$

$$\bar{X} = \frac{(277.78)(0)+(142.86)(136.5)+(10.03)(196.5)+(161.29)(226.5)}{277.78+142.86+10.03+161.29}$$

$\bar{X} = 98$ ft

CENTER OF RIGIDITY – USING ETABS

$$\bar{Y} = \frac{(41)(132.5)+(165.31)(104.5)+(10.54)(92.5)+(103.01)(75.4)+0(0)}{41+165.31+10.54+103.01}$$

$\bar{Y} = 98$ ft

$$\bar{X} = \frac{(9.95)(0)+(33.63)(136.5)+(2.75)(196.5)+(47.84)(226.5)}{9.95+33.63+2.75+47.84}$$

$\bar{X} = 169$ ft

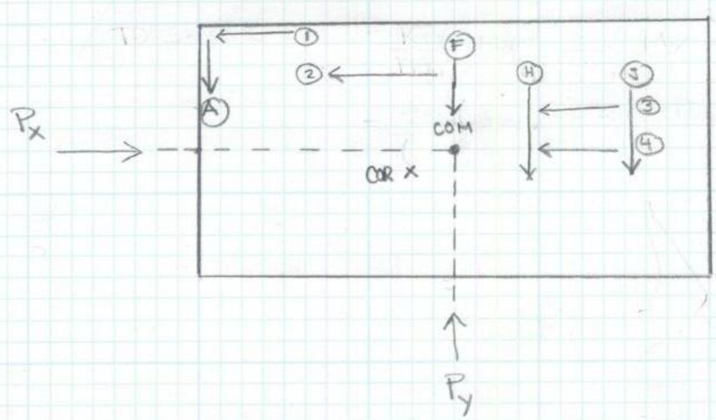
Torsion Analysis

FROM ETABS: C.O.M. = (131.6', 97.3')

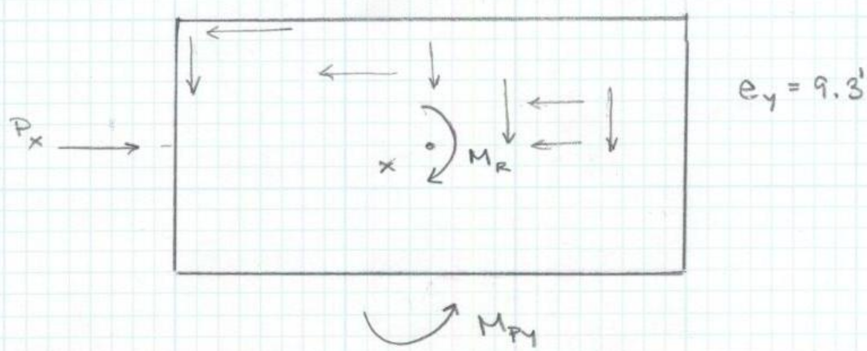
C.O.R. = (112.9', 88.0')

e = (18.7', 9.3')

DIRECTION OF FORCES



DIRECTION OF X TORSIONAL FORCES (DUE TO P_x)



DIRECT SHEAR FORCES (X DIRECTION)

FRAME 1: $F_{ix} = \frac{K_{ix}}{\Sigma K_{ix}} P_x = \frac{18 \cdot P_x}{18+57+5+20} = 0.18 P_x$

FRAME 2: $F_{ix} = \frac{57}{18+57+5+20} = 0.57 P_x$

FRAME 3: $F_{ix} = \frac{5}{18+57+5+20} = 0.05 P_x$

FRAME 4: $F_{ix} = \frac{20}{18+57+5+20} = 0.20 P_x$

TORSIONAL EFFECT (X-DIRECTION)

d_i = distance from C.O.R. to each wall

FRAME ①: $d_i = 132.5 - 88 = 44.5$

FRAME ②: $d_i = 104.5 - 88 = 16.5$

FRAME ③: $d_i = 92.5 - 88 = 4.5$

FRAME ④: $d_i = 75.4 - 88 = 12.6$

FRAME ⑧: $d_i =$ n/a

FRAME Ⓐ: $d_i = 112.9$

FRAME Ⓔ: $d_i = 136.5 - 112.9 = 23.6$

FRAME Ⓗ: $d_i = 196.5 - 112.9 = 83.6$

FRAME Ⓙ: $d_i = 226.5 - 112.9 = 113.6$

$$J = \sum K_i d_i^2 = (18)(44.5)^2 + (57)(16.5)^2 + (5)(4.5)^2 + (20)(12.6)^2 + (4)(112.9)^2 + (3)(83.6)^2 + (46)(23.6)^2 + (55)(113.6)^2$$

$$J = 861785 \ (k/in) ft^2$$

FRAME ①: $$f_{it} = \frac{K_i d_i P_x e_y}{\sum K_i d_i^2} = \frac{(18)(44.5) P_x (9.3)}{861785} = 0.008644 P_x$$

FRAME ②: $$f_{it} = \frac{(57)(16.5) P_x (9.3)}{861785} = 0.010149 P_x$$

FRAME ③: $$f_{it} = \frac{(5)(4.5) P_x (9.3)}{861785} = 0.000243 P_x$$

FRAME ④: $$f_{it} = \frac{(20)(12.6) P_x (9.3)}{861785} = 0.002719 P_x$$

TOTAL FORCES (X-DIRECTION)

$F_i = F_{i\,DIRECT} \pm F_{i\,torsion}$

*NOTE: THE ± DEPENDS ON WHAT SIDE OF THE C.O.R. THE LATERAL ELEMENT OF INTEREST IS.

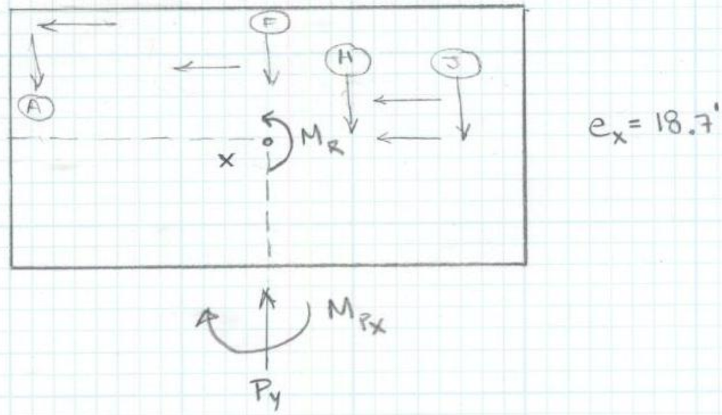
FRAME ①: $F_i = 0.18 P_x + 0.008644 P_x = 0.188644 P_x$

FRAME ②: $F_i = 0.57 P_x + 0.010149 P_x = 0.580149 P_x$

FRAME ③: $F_i = 0.05 P_x + 0.000243 P_x = 0.050243 P_x$

FRAME ④: $F_i = 0.20 P_x + 0.002719 P_x = 0.202719 P_x$

DIRECTION OF Y - TORSIONAL FORCES (DUE TO P_y)



DIRECT SHEAR FORCES (Y DIRECTION)

FRAME (A) : $F_{iy} = \dfrac{K_{iy}}{\Sigma K_{iy}} P_y = \dfrac{45\,P_y}{45+34+463+549} = 0.04\,P_y$

FRAME (H) : $F_{iy} = \dfrac{34\,P_y}{45+34+463+549} = 0.03\,P_y$

FRAME (F) : $F_{iy} = \dfrac{463}{45+34+463+549} = 0.42\,P_y$

FRAME (J) : $F_{iy} = \dfrac{549}{45+34+463+549} = 0.50\,P_y$

TORSIONAL EFFECT (Y-DIRECTION)

FRAME (A) : $f_{it} = \dfrac{K_i d_i P_y e_x}{\Sigma K_i d_i^2} = \dfrac{(45)(112.9)P_y(18.7)}{861785} = 0.110243\,P_y$

FRAME (H) : $f_{it} = \dfrac{(34)(83.6)P_y(18.7)}{861785} = 0.061678\,P_y$

FRAME (F) : $f_{it} = \dfrac{(463)(23.6)P_y(18.7)}{861785} = 0.237102\,P_y$

FRAME (J) : $f_{it} = \dfrac{(549)(113.6)P_y(18.7)}{861785} = 1.353298\,P_y$

TOTAL FORCES (Y-DIRECTION)

$$F_i = F_{i\,DIRECT} \pm F_{i\,TORSION}$$

FRAME A : $F_i = 0.04P_y + 0.110243P_y = 0.150243P_y$

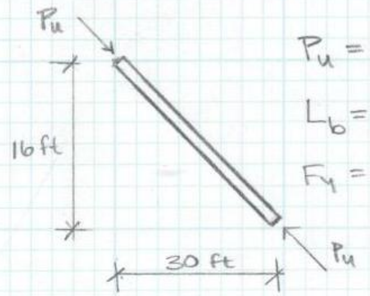
FRAME H : $F_i = 0.03P_y + 0.061678P_y = 0.091678P_y$

FRAME F : $F_i = 0.42P_y + 0.237102P_y = 0.657102P_y$

FRAME J : $F_i = 0.50P_y + 1353298P_y = 1.853298P_y$

Lateral Members

BRACE MEMBER AT LEVEL 1 HSS 12 × 10 × 1/2



$P_u = 133K$ $(1.2D + 1.6W + 1.0L + 0.5L_r)$

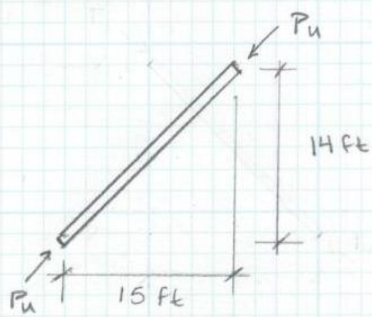
$L_b = \sqrt{16^2 + 30^2} = 34$ ft

$F_y = 42$ ksi

FROM TABLE 4-3 : $\phi P_n = 386$ kips $> P_u$ ∴ OK

$\frac{P_u}{\phi P_n} = \frac{133}{386} = 0.34 < 1.0$ ∴ OK

BRACE MEMBER AT LEVEL 5 HSS 8 × 8 × 3/8



$P_u = 83.8K$ $(1.2D + 1.6W + 1.0L + 0.5L_r)$

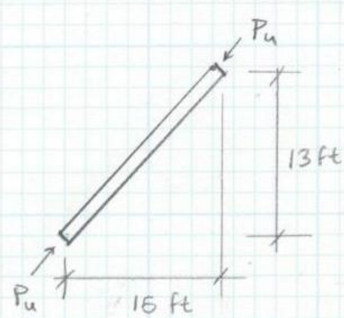
$L_b = 21$ ft

$F_y = 42$ ksi

FROM TABLE 4-4 : $\phi P_n = 275$ kips $> P_u$ ∴ OK

$\frac{P_u}{\phi P_n} = \frac{83.8}{275} = 0.30 < 1.0$ ∴ OK

BRACE MEMBER AT LEVEL 7 HSS 10 × 8 × 3/8



$P_u = 85.7K$ $(1.2D + 1.6W + 1.0L + 0.5L_r)$

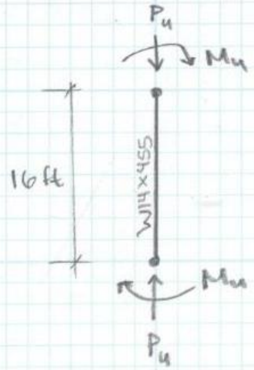
$L_b = 20$ ft

$F_y = 42$ ksi

FROM TABLE 4-3 : $\phi P_n = 333K > P_u$ ∴ OK

$\frac{P_u}{\phi P_n} = \frac{85.7}{333} = 0.26 < 1.0$ ∴ OK

COLUMN MEMBER AT LEVEL 1 W14X455



$P_u = 1193$ Kips

$M_{uy} = 22$ ft-kips $(1.2D + 1.6W + 1.0L + 0.5L_r)$

FROM TABLE 6-1

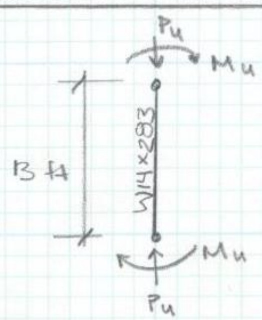
$KL = (16)\frac{r_x}{r_y} = (16)(1.67) = 26.72$ ft

$p \times 10^{-3} = 0.247$

$b_x \times 10^{-3} = 0.261$

$$\frac{1193(0.247)}{1000} + \frac{22(0.261)}{1000} = 0.30 < 1.0 \therefore OK$$

COLUMN MEMBER AT LEVEL 7 W14X283



$P_u = 546$ kips

$M_{uy} = 70$ ft-kips

FROM TABLE 6-1

$KL = (13)(1.63) = 21.19$ ft

$p \times 10^{-3} = 0.349$

$b_x \times 10^{-3} = 0.449$

$$\frac{(546)(0.349)}{1000} + \frac{(70)(0.449)}{1000} = 0.22 < 1.0 \therefore OK$$

Figure A-1" Calculated Wind Pressures in North/South Direction

Distribution of Windward and Leeward Pressures								
Level	Height Above Ground (ft)	q (psf)	Wind Pressures (psf)					
			N-S windward	N-S leeward	N-S side wall	E-W windward	E-W leeward	E-W side wall
Penthouse	134	20.75	23.10	-7.29	-20.18	23.36	-9.36	-20.41
T.O. Parapet	120	20.16	22.55	-7.29	-20.18	22.81	-9.36	-20.41
Roof	118	20.16	22.55	-7.29	-20.18	22.81	-9.36	-20.41
8	104	19.39	21.82	-7.29	-20.18	22.07	-9.36	-20.41
7	91	18.61	21.09	-7.29	-20.18	21.33	-9.36	-20.41
6	78	17.84	20.37	-7.29	-20.18	20.60	-9.36	-20.41
5	64	16.87	19.46	-7.29	-20.18	19.67	-9.36	-20.41
4	50	15.70	18.37	-7.29	-20.18	18.57	-9.36	-20.41
3	36	14.35	17.09	-7.29	-20.18	17.28	-9.36	-20.41
2	19	11.83	14.73	-7.29	-20.18	14.88	-9.36	-20.41
Ground	0	11.05	14.00	-7.29	-20.18	14.14	-9.36	-20.41

Figures A2 & A3: Coefficients used to calculate Wind Loading and Gust Effect Factor Respectively

Design Category	III
V (mph)	90
K_d	0.85
Importance Factor (I)	1.1
Exposure Category	B (urban areas)
K_{zt}=	1
n_1=	0.75
Gf	1.173 (N-S)
	1.189 (E-W)
q_p	20.16
GC_{pn}	+1.5 windward
	-1.0 leeward
P_p	21.56 windward
	19.16 leeward
GC_{pi}	n/a
z_g=	1200 ft
α=	7

Cp Value	N-S	E-W
Windward wall	0.8	0.8
Leeward Wall	-0.155	-0.239
Side Wall	-0.7	-0.7

Gust Effect Factors		
	N-S	E-W
B (ft)	260	80.5
L (ft)	80.5	260
h (ft)	134	134
n_1	0.75	0.75
Structure:	Flexible	Flexible
g_q	3.4	3.4
g_v	3.4	3.4
g_r	4.12	4.12
z bar	80.4	80.4
ε bar	0.33	0.33
L bar	320	320
b bar	0.45	0.45
α bar	0.25	0.25
Iz bar	0.259	0.259
Lz bar	430.6	430.6
Q	0.792	0.843
Vz bar	74.21	74.21
N_1	4.352	4.352
n_h	6.23	6.23
n_b	12.087	3.742
n_l	12.529	40.466
R_h	0.148	0.148
R_b	0.079	0.232
R_L	0.077	0.024
R_n	0.055	0.055
R	0.06	0.101
G_f	1.173	1.189

Figure A-4: Kz and q_z Factors

	Level	Height Above Ground (ft)	Floor Height (ft)	K_z	q_z
windward	Penthouse	134	14	1.07	20.75
	T.O. Parapet	120	0.25	1.04	20.16
	Roof	118	1.7	1.04	20.16
	8	104	14	1	19.39
	7	91	13	0.96	18.61
	6	78	13	0.92	17.84
	5	64	14	0.87	16.87
	4	50	14	0.81	15.70
	3	36	14	0.74	14.35
	2	19	17	0.61	11.83
	Ground	0	19	0.57	11.05
Leeward	All	All	All	1.04	20.16

Figure A-5: Wind Story Forces, Shears, and Moments

Level	Height Above Ground (ft)	Floor Height (ft)	h/2 above	h/2 below	Wind Forces					
					Load (kips)		Shear (kips)		Moment (ft-kips)	
					N-S	E-W	N-S	E-W	N-S	E-W
Pent house	134	14	14	0.125	71	21	71	21	9580	2783
T.O. Parapet	120	0.25	0.125	0.9	5	1	77	22	605	176
Roof	118	1.7	0.9	7.0	39	11	115	33	4557	1324
8	104	14	7	6.5	64	19	179	52	6641	1930
7	91	13	6.5	6.5	59	17	238	69	5372	1561
6	78	13	6.5	7	59	17	297	86	4583	1331
5	64	14	7	7	58	17	354	103	3687	1071
4	50	14	7	7	54	16	408	119	2682	779
3	36	14	7	8.5	54	16	462	134	1953	568
2	19	17	8.5	9.5	52	15	514	149	987	287
Ground	0	19	9.5	7	44	13	559	162	0	0

FIND VELOCITY PRESSURES, q_z AND q_h:

DETERMINE BASIC WIND SPEED V FROM FIG. 6-1

$V = 90$ mph

DETERMINE WIND DIRECTIONALITY FACTOR K_d FROM TABLE 6-4 (ASCE 7-05)

$K_d = 0.85$

DETERMINE IMPORTANCE FACTOR I FROM TABLE 6-1 (ASCE 7-05)

CATEGORY III, $I = 1.1$

DETERMINE EXPOSURE CATEGORY FROM § 6.5.6 (ASCE 7-05)

CATEGORY B, URBAN AREA

ARE ALL 5 CONDITIONS OF §6.5.7.1 MET? NO

TOPOGRAPHIC FACTOR $K_{zt} = 1.0$

DETERMINE VELOCITY PRESSURE EXPOSURE COEFFICIENTS K_z AND K_h FROM TABLE 6-3 (ASCE 7-05)

$z_g = 1200$ ft
$\alpha = 7.0$
$z = 148$ ft ← NOTE: THIS IS THE MOST CRITICAL BUILDING HT.
EXPOSURE B, CASE 2

* REFER TO WIND ANALYSIS SPREADSHEET
$K_z = 1.07$ @ 134' (TOP OF PENTHOUSE)

DETERMINE VELOCITY PRESSURE AT HEIGHT z AND h

SAMPLE CALCULATION AT HT. = 134 ft (TOP OF PENTHOUSE)

$$K_z = 2.01 \left(\frac{134}{1200}\right)^{(2/7)} = 1.07$$

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$
$$= 0.00256(1.07)(1.0)(0.85)(90^2)(1.1)$$
$$= 20.75$$

GUST EFFECT FACTORS, G & G_f :

DETERMINE B, L, and H

B (N-S) = 260ft, L(N-S) = 80.5 ft
B (E-W) = 80.5ft, L (E-W) = 260 ft
H = 134 ft

DETERMINE n_1 & β

$n_1 = 100/H$ (ft) AVERAGE VALUE
$= 100/134 = 0.75$ Hz

$\beta = 1.0$ PER ISO

IS $n_1 > 1$ Hz ? NO
STRUCTURE IS FLEXIBLE

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} \quad \text{[EQ. 6-9]}$$

$$g_R = \sqrt{2\ln(3600 \times 0.75)} + \frac{0.577}{\sqrt{2\ln(3600\times 0.75)}} = 4.120$$

$\bar{z} = 0.6h \geq z_{min}$

$z_{min} = 30$ ft [TABLE 6-2 ASCE 7-05]

$\bar{z} = 0.6(134) = 80.4\text{ft} > 30\text{ft}$ $\therefore$ OK

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} \quad \text{[EQ. 6-5]}$$

$c = 0.30$ [TABLE 6-2 ASCE 7-05]

$$I_{\bar{z}} = 0.30\left(\frac{33}{80.4}\right)^{1/6} = 0.259$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$$

$\ell = 320$ ft, $\bar{\epsilon} = 1/3.0$ [TABLE 6-2 ASCE 7-05]

$$L_{\bar{z}} = 320\left(\frac{80.4}{33}\right)^{(1/3)} = 430.6 \text{ ft}$$

GUST EFFECT FACTORS, G & G_f CONTINUED:

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \quad [EQ.\ 6\text{-}6]$$

$$Q_{(N-S)} = \sqrt{\frac{1}{1+0.63\left(\frac{260+134}{430.6}\right)^{0.63}}} = 0.792$$

$$Q_{(E-W)} = \sqrt{\frac{1}{1+0.63\left(\frac{80.5+134}{430.6}\right)^{0.63}}} = 0.843$$

DETERMINE BASIC WIND SPEED: $V = 90$ mph [FIG. 6-1 ASCE 7-05]

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) \quad [EQ.\ 6\text{-}14]$$

$$\bar{b} = 0.45, \quad \bar{\alpha} = 1/4.0$$

$$\bar{V}_{\bar{z}} = 0.45\left(\frac{80.4}{33}\right)^{(1/4)} 90\left(\frac{88}{60}\right) = 74.21$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.75)(430.6)}{74.21} = 4.352 \quad [EQ.\ 6\text{-}12]$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(4.352)}{(1+10.3(4.352))^{5/3}} = 0.055 \quad [EQ.\ 6\text{-}11]$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \text{ for } \eta > 0$$

$$\eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.75)(134)}{74.21} = 6.230$$

$$R_h = \frac{1}{6.230} - \frac{1}{2(6.230)^2}\left(1-e^{-2(6.230)}\right) = 0.148$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \text{ for } \eta > 0$$

$$\eta = 4.6 n_1 B/\bar{V}_{\bar{z}} = 4.6(0.75)(260)/74.21 = 12.087 \quad (N-S)$$
$$= 4.6(0.75)(80.5)/74.21 = 3.742 \quad (E-W)$$

$$R_B(N-S) = \frac{1}{12.087} - \frac{1}{2(12.087)^2}\left(1-e^{-(2\times 12.087)}\right) = 0.079 \quad (N-S)$$

$$R_B(E-W) = \frac{1}{3.742} - \frac{1}{2(3.742)^2}\left(1-e^{-(2\times 3.742)}\right) = 0.232 \quad (E-W)$$

GUST EFFECT FACTORS, G & GF CONTINUED:

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) \text{ for } \eta > 0$$

$$\eta = 15.4\, n_1 L / \bar{V}_{\bar{z}} = 15.4(0.75)(80.5)/74.21 = 12.529 \quad (N\text{-}S)$$

$$= 15.4(0.75)(260)/74.21 = 40.466 \quad (E\text{-}W)$$

$$R_L\,(N\text{-}S) = \frac{1}{12.529} - \frac{1}{2(12.529)^2}\left(1 - e^{-(2\times 12.529)}\right) = 0.077 \quad (N\text{-}S)$$

$$R_L\,(E\text{-}W) = \frac{1}{40.466} - \frac{1}{2(40.466)^2}\left(1 - e^{-(2\times 40.466)}\right) = 0.024 \quad (E\text{-}W)$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B \left(0.53 + 0.47 R_L\right)} \quad \text{[EQ. 6-10]}$$

$$= \sqrt{\frac{1}{1.0}(0.55)(0.148)(0.079)\left(0.53 + 0.47(0.077)\right)} = 0.060 \quad (N\text{-}S)$$

$$= \sqrt{\frac{1}{1.0}(0.55)(0.148)(0.232)\left(0.53 + 0.47(0.024)\right)} = 0.101 \quad (E\text{-}W)$$

$$G_f = 0.925\left[\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right] \quad \text{[EQ. 6-8]}$$

$$= 0.925\left[\frac{1 + 1.7(0.259)\sqrt{(3.4)^2(0.792)^2 + (4.120)^2(0.060)^2}}{1 + 1.7(3.4)(0.259)}\right] = 1.173 \quad (N\text{-}S)$$

$$= 0.925\left[\frac{1 + 1.7(0.259)\sqrt{(3.4)^2(0.843)^2 + (4.120)^2(0.101)^2}}{1 + 1.7(3.4)(0.259)}\right] = 1.189 \quad (E\text{-}W)$$

BUILDINGS, MAIN WIND-FORCE RESISTING SYSTEMS

IS THE BUILDING ENCLOSED OR PARTIALLY ENCLOSED? YES

DOES THE BUILDING HAVE A PARAPET? YES

VELOCITY PRESSURE $q_p = 20.16$ mph

DETERMINE COMBINED NET PRESSURE COEFFICIENT GC_{pn}

$GC_{pn} = +1.5$ WINDWARD

$GC_{pn} = -1.0$ LEEWARD

DETERMINE COMBINED NET DESIGN PRESSURE ON THE PARAPET

$P_p = q_p GC_{pn}$ [EQ. 6-20]

$= (20.16) + 1.5 = 21.56$ (WINDWARD)

$= (20.16) - 1.0 = 19.16$ (LEEWARD)

IS THE BUILDING A LOW-RISE BUILDING AS DEFINED IN 6.2? NO

IS THE BUILDING RIGID? NO

DETERMINE VELOCITY PRESSURE q_z FOR WINDWARD WALLS ALONG THE HT. OF THE BUILDING AND q_h FOR LEEWARD WALLS, SIDE WALLS, & ROOF (SEE SPREADSHEETS)

DETERMINE PRESSURE COEFFICIENTS C_p FOR THE ROOF FROM FIG 6-6

$\frac{L}{B} = \frac{80.5}{260} = 0.310$ (N-S)

$\frac{L}{B} = \frac{260}{80.5} = 3.230$ (E-W)

C_p VALUE		
	N-S	E-W
WINDWARD WALL	0.8	0.8
LEEWARD WALL	-0.155	-0.239
SIDE WALL	-0.7	-0.7

WINDWARD WALLS: $P_z = q_z G_f C_p$ ← SAMPLE CALCULATION (SEE SPREADSHEET)

(N-S) $P_z = (20.16)(1.173)(0.8)$

C_P VALUES	N-S	E-W
WINDWARD	0.8	0.8
LEEWARD	-0.155	-0.239
SIDE WALL	-0.700	-0.700

G_f (N-S) = 1.173

G_f (E-W) = 1.189

NOT INCLUDING UPLIFT ON ROOF SINCE ROOF FRAMING MADE UP OF W-SHAPES

$q_i = q_h = q_z$ FOR TOP OF BUILDING = 20.16 psf

INTERNAL PRESSURE COEFFICIENT : $GC_{pi} = \pm 0.18$

DESIGN WIND PRESSURES — P_z + P_h (EQ. 6-17)

WINDWARD WALLS: (psf)

$$P_z = q_z GC_p - q_h (GC_{pi})$$
$$= (1.173)(0.8) q_z \pm 20.16 (0.18)$$
$$= 0.9384 q_z \pm 3.6288 \quad [N-S]$$

$$P_z = (1.189)(0.8) q_z \pm 20.16 (0.18)$$
$$= 0.9512 q_z \pm 3.6288 \quad [E-W]$$

LEEWARD WALLS & SIDE WALLS: (psf)

$$P_z = q_h GC_p - q_h (GC_{pi})$$
$$= (20.16)(1.173) C_p \pm 20.16 (0.18)$$
$$= 23.6477 C_p \pm 3.6288 \quad [N-S]$$

$$P_z = (20.16)(1.189) C_p \pm 20.16 (0.18)$$
$$= 23.9702 C_p \pm 3.6288 \quad [E-W]$$

Seismic

Figure A-6: Coefficients used for Seismic Analysis per ASCE 7-05

Seismic Analysis Coefficients	
Ss=	0.37
S1=	0.07
Occupancy Category=	III
Site Class=	C (very dense soil and soft rock)
Fa=	1.2
Fv=	1.7
Sms=	0.45
Sm1=	0.119
Sds=	0.3
Sd1=	0.079
Ta=	1.182
0.8Ts=	0.211
SDC=	B
Ts=	0.226
R=	7
I=	1.1
Ta=	1.182
Cu=	0.211
TL=	6 sec
Cs=	0.006
Cs=	0.01
k=	1.755
W=	15388 kips
V=	153.88 kips

Figure A-7: Equivalent Lateral Force Procedure

Lateral Seismic Force, Fx							
Level	Floor Height (ft)	Slab Weight (lbs)	Column Weight (lbs)	Beam Weight (lbs)	Curtainwall Weight (lbs)	Total Level Weight (lbs)	Fx (kips)
penthouse	134	80750	0	38245	0	118995	6.76
roof	120	492300	3440	50726	70560	617026	28.87
8	104	403570	15938	37130	61740	518378	18.87
7	91	374170	24463	42135	57330	498098	14.34
6	78	1108370	24463	116396	127335	1376564	30.24
5	64	1201959	16940	169389	144690	1532978	23.80
4	50	1201959	86174	90008.7	144690	1522831.7	15.33
3	36	1201959	76816.5	140824.5	144690	1564290	8.85
2	19	3223770.5	76816.5	220889.5	178755	3700231.5	6.82
1	0	3356119.75	236557.1637	177844	168240	3938760.916	0.00

Figure A-8: Distribution of Shear and Moment on Building

Base Shear and Overturning Moment Distribution							
Level	hx (ft)	Story Weight (k)	hxk Wx	Cvx	Fx=CvxV	Vx (k)	Mx (ft-k)
penthouse	134	119.0	643573	0.044	7	7	906
roof	120	617.0	2749581	0.188	29	36	4276
8	104	518.4	1796967	0.123	19	54	5668
7	91	498.1	1365943	0.093	14	69	6265
6	78	1376.6	2880199	0.197	30	99	7729
5	64	1533.0	2266636	0.155	24	123	7865
4	50	1522.8	1459971	0.100	15	138	6911
3	36	1564.3	842613	0.057	9	147	5294
2	19	3700.2	649294	0.044	7	154	2924
1	0	3938.8	0	0.000	0	154	0
Total	134	15388.2	14654776	1	**154**		**47835**
Base Shear=	154 kips						

SEISMIC GROUND MOTION VALUES & EQUIV. LAT. FORCE PROCEDURE

DETERMINE S_S AND S_1 FROM FIG. 22-1 THROUGH 22-14

$S_1 = 0.07$, $S_S = 0.350$

IS $S_S \leq 0.15$ & $S_1 \leq 0.04$? NO

IS THE STRUCTURE SEISMICALLY ISOLATED OR DOES IT HAVE DAMPING SYSTEMS ON SITE W/ $S_1 \geq 0.6$? NO

DETERMINE THE SITE CLASS IN ACCORDANCE W § 11.4.2 & CH. 20

↳ SITE CLASS "C"

DETERMINE S_{MS} & S_{M1} by EQN 11.4-1 & 11.4-2

$F_a = 1.2$, $F_v = 1.7$

$S_{MS} = F_a S_S = 1.2(0.370) = 0.45$

$S_{M1} = F_v S_1 = 1.7(0.07) = 0.119$

DETERMINE S_{DS} & S_{D1} BY EQN 11.4-3 & 11.4-4 RESPECTIVELY:

$S_{DS} = 2 S_{MS}/3 = 2(0.45)/3 = 0.30$

$S_{D1} = 2 S_{M1}/3 = 2(0.119)/3 = 0.079$

DETERMINE OCCUPANCY CATEGORY: III

IS $S_1 > 0.75$? NO

IS THE SIMPLIFIED DESIGN PROCEDURE OF 12.14 PERMITTED? NO

ARE ALL 4 CONDITIONS OF 11.6 SATISFIED? NO

$T_a = C_t h_n^x = 0.03(134)^{0.75} = 1.182$ (ECCEN. BRACED STEEL FRAMES)

$0.8 T_S = 0.8 \frac{S_{D1}}{S_{DS}} = 0.8\left(\frac{0.079}{0.30}\right) = 0.211$

$T_a \nless 0.8 T_S$ $T = C_u T_a = 1.7(1.182) = 2.009$

DETERMINE SDC AS THE MORE SEVERE OF T. 11.6-1 & T. 11.6-2

SDC = B

DETERMINE R, RESPONSE COEFF. : 7 for truss frames

IMPORTANCE FACTOR : 1.1

DETERMINE T_L FROM FIG 22-15 THROUGH 22-20 : 6 sec.

DETERMINE C_S

$C_S = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \leq \frac{S_{DS}}{\left(\frac{R}{I}\right)}$ $C_S = \frac{0.079}{(2.009)\left(\frac{7}{1.1}\right)} \leq \frac{0.30}{\left(\frac{7}{1.1}\right)} = 0.006$

Is $S_1 \geq 0.6$? NO

IS $C_s < 0.01$? YES

$C_s = 0.01$

DETERMINE EFFECTIVE SEISMIC WEIGHT W : 15,388 KIPS

DETERMINE BASE SHEAR

$V = C_s W = 0.01(15388 \text{ kips}) = 153.88 \text{ kips}$

IS $T \leq 0.5$ sec? NO

IS $T \geq 2.5$ sec? NO

$k = 0.75 + 0.5T = 0.75 + 0.5(2.009) = 1.755$

Figure A-9: Building Weight Calculations

Level	Floor Height (ft)	Slab Weight (lbs)	Column Weight (lbs)	Beam Weight (lbs)	Curtainwall Weight (lbs)	Total Level Weight (lbs)
penthouse	134	80750	0	38245	0	118995
roof	120	492300	3440	50726	70560	617026
8	104	403570	15938	37130	61740	518378
7	91	374170	24463	42135	57330	498098
6	78	1108370	24463	116396	127335	1376564
5	64	1201959	16940	169389	144690	1532978
4	50	1201959	86174	90008.7	144690	1522831.7
3	36	1201959	76816.5	140824.5	144690	1564290
2	19	3223770.5	76816.5	220889.5	178755	3700231.5
1	0	3356119.75	236557.1637	177844	168240	3938760.916
					Total Building Weight:	15388153.12

Floor	Floor Area (sf)	Floor Dead Load (psf)	Floor Weight	Curtainwall length (ft)	Curtainwall height (ft)	Curtainwall weight (ft) (height*weight* 15 psf)
cellar level						
Ground						
loading dock	930	150	139500	701	16	168240
first floor level	14838	130	1928940			
podium	600	200	120000			
archive	900	75	67500			
Offices	1948	71	138308			
roof with garden	1330.84	365	485756.6			
library stacks	6705.847	71	476115.153			
second level						
roof with garden	4560	365	1664400	701	17	178755
classrooms	6784	71	481664			
corridors	7601.5	71	539706.5			
auditorium	2800	85	238000			
roof with pavers on 2	2000	150	300000			

Floor	Floor Area (sf)	Floor Dead Load (psf)	Floor Weight	Curtain wall length (ft)	Curtain wall height (ft)	Curtainwall weight (ft) (height*weight* 15 psf)
third level						
classrooms	11424	71	811104	689	14	144690
corridor	5505	71	390855			
fourth level						
offices	5712	71	405552	689	14	144690
classrooms	1200	71	85200			
corridors	10017	71	711207			
fifth level						
offices	7570.5	71	537505.5	689	14	144690
corridors	9358.5	71	664453.5			
sixth level						
offices	3050	71	216550	653	13	127335
corridors	2220	71	157620			
roof	4757.5	90	428175			
roof with drift	325	85	27625			
mechanical	2320	120	278400			
seventh level						
offices	2635	71	187085	294	13	57330
corridors	2635	71	187085			
eighth level						
offices	2335	71	165785	294	14	61740
corridors	2335	71	165785			
mechanical	600	120	72000			
roof level						
roof	4670	90	420300	294	16	70560
mechanical	600	120	72000			
penthouse level						
roof with drift	950	85	80750	248	0	0
		total:	12644927.3			1098030

Figure A-10: Accumulated Loads on Columns

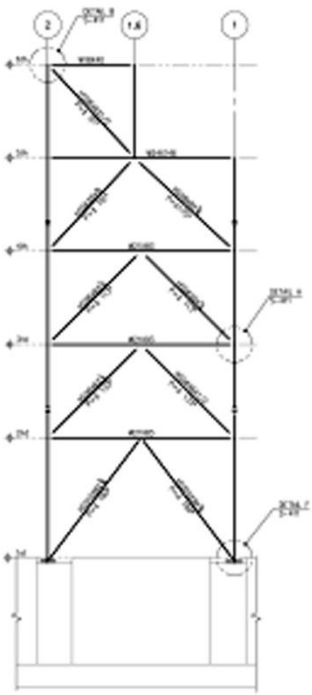
LOCATION J3 : Accumulated Loads on Columns											
Level	tributary area	dead load (psf)	live load (psf)	influence area	LL red. Factor	live load (k)	dead load (k)	load comb.	load at floor (k)	accum. Load (k)	accum. load (k) by Turner
roof	525	90	45	2100	1.00	23.6	47.3	1.2D+0.5Lr	68.5	68.5	80
Eighth	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	161.7	161
seventh	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	255.0	242
sixth	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	348.2	337
fifth	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	460.4	715
fourth	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	572.6	852
third	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	684.8	997
second	675	85	100	3420	0.51	34.2	57.4	1.2D+1.6L	123.6	808.4	1123
Ground	675	130	100	3420	0.51	34.2	87.8	1.2D+1.6L	160.0	968.4	1349

At level 5 there is a large difference between the accumulated loads calculated by that which was provided by Turner Construction Company. This is due to the step- back of the floor levels above. Since the columns located at J1.6 at above levels don't continue to the fifth level, the fifth level is forced to carry the load from the J1.6 column at level 6. Below is a table depicting the adjusted accumulated loads and how they compare to values provided by Turner Construction Company.

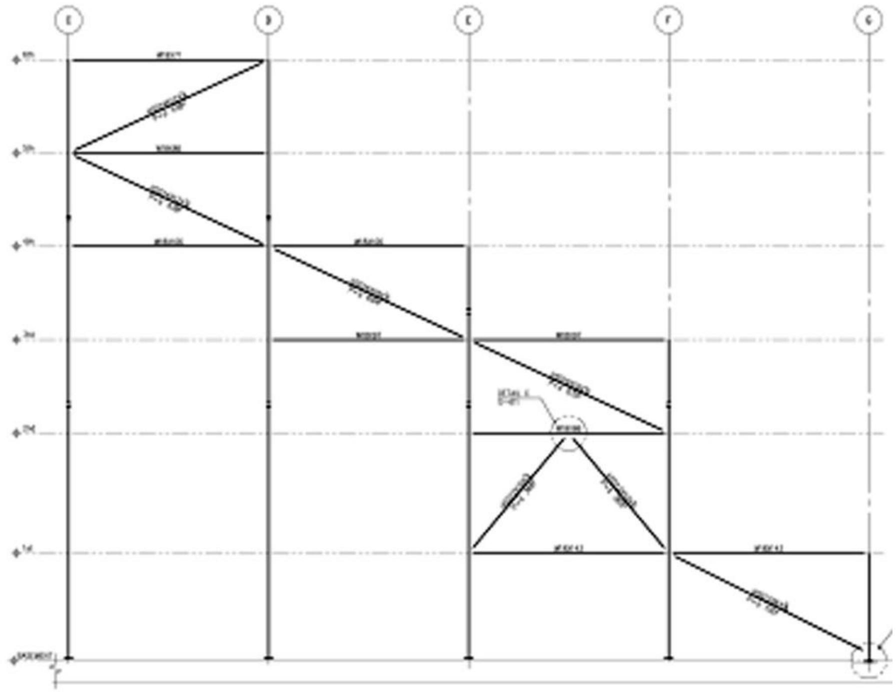
Figure A-2: Adjustment of Accumulated Loads on Columns

	accumulated load (k) by Turner for Loc. J1.6	**LOCATION J3 : Accumulated Loads on Columns**		
Level		Adjusted accumulated load (k)	accumulated load (k) provided by Turner	percent Error = |adj-prov| /adj*100
roof	n/a	68.5	80	17
eighth	n/a	161.7	161	0
seventh	n/a	255.0	242	5
sixth	266	348.2	337	3
fifth	n/a	726.4	715	2
fourth	n/a	838.6	852	2
third	n/a	950.8	997	5
second	n/a	1074.4	1123	5
Ground	n/a	1234.4	1349	9

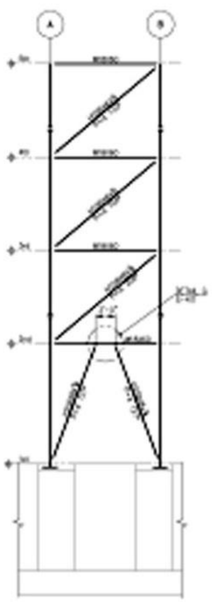
Appendix B - Braced Frames



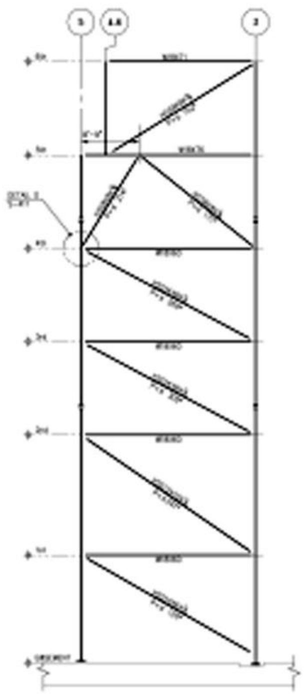
1 TRUSS @ GRID A



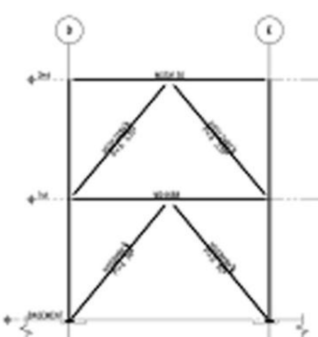
2 TRUSS @ GRID 2



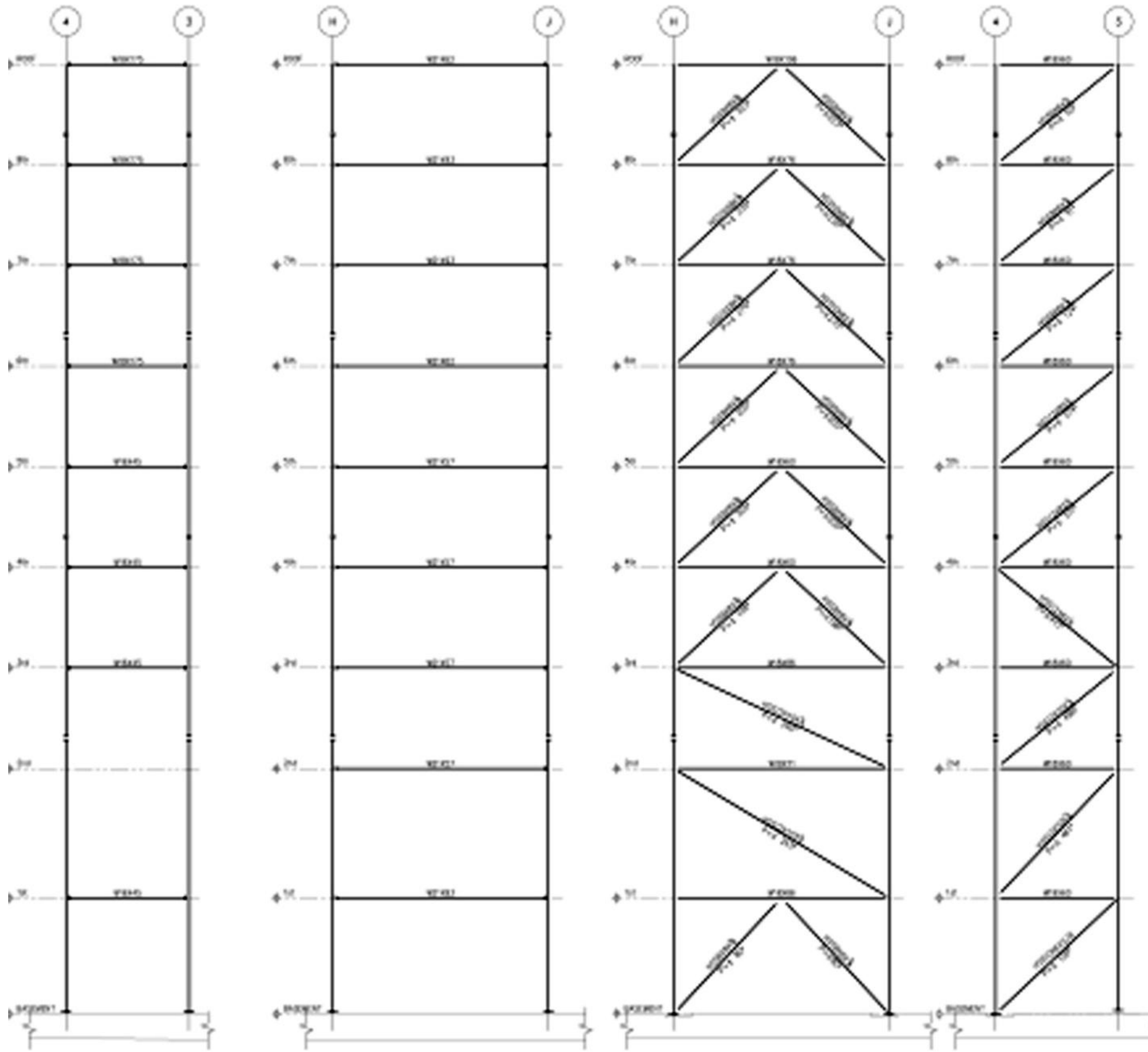
3 TRUSS @ GRID 1



4 TRUSS @ GRID F



5 TRUSS @ GRID B



6 FRAME @ GRID H

7 FRAME @ GRID 3

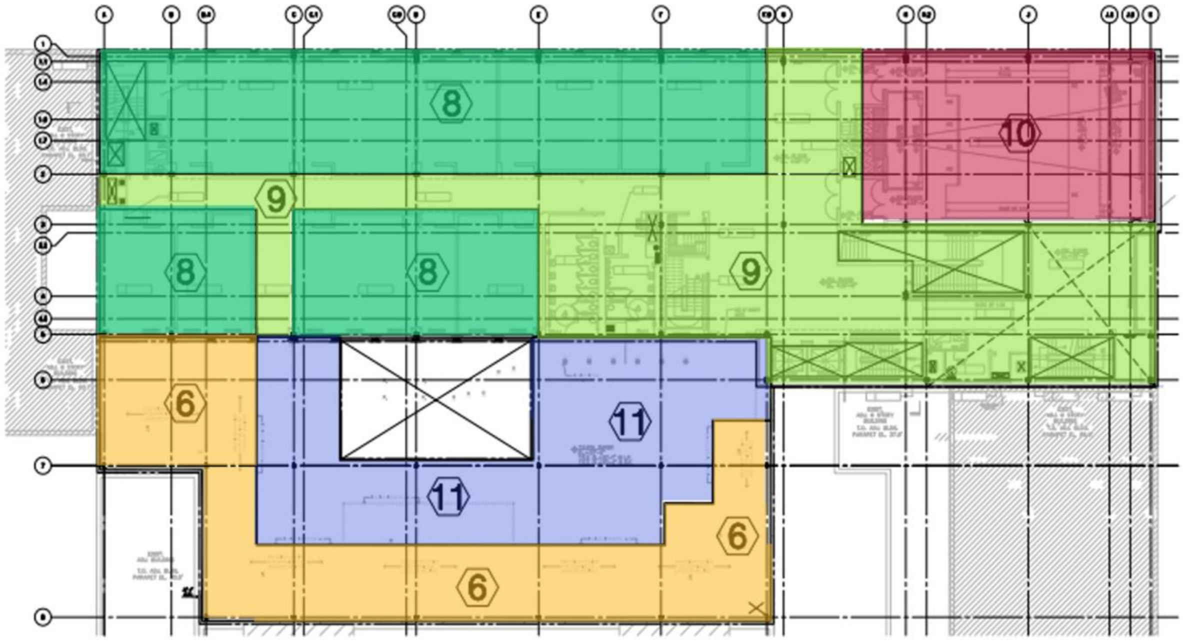
8 TRUSS @ GRID 4

9 TRUSS @ GRID J

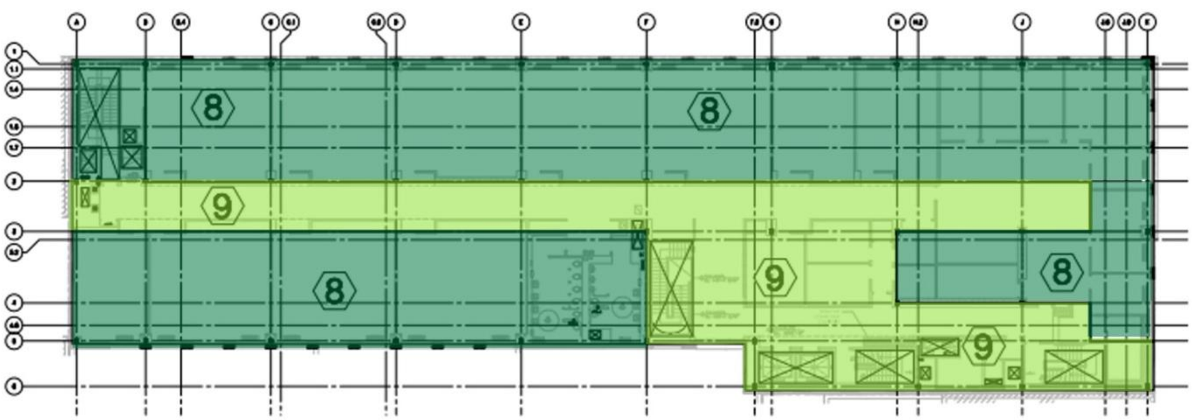
Appendix C. Loading Diagrams

LOADING SCHEDULE		
ID	DL psf	LL psf
1. LOADING DOCK	150.0	600.0
2. 1ST FLOOR	130.0	100.0
3. PODIUM	200.0	100.0
4. ARCHIVE	75.0	350.0
5. OFFICES	71.0	50.0
6. ROOF WITH GARDEN	365.0	100.0
7. LIBRARY STACKS	71.0	100.0
8. CLASSROOMS	71.0	40.0
9. CORRIDOR	71.0	100.0
10. AUDITORIUM	85.0	60.0
11. ROOF WITH PAVERS ON 2	150.0	100.0
12. ROOF	90.0	45.0
13. ROOF WITH DRIFT	85.0	60.0
14. MECHANICAL	120.0	100.0

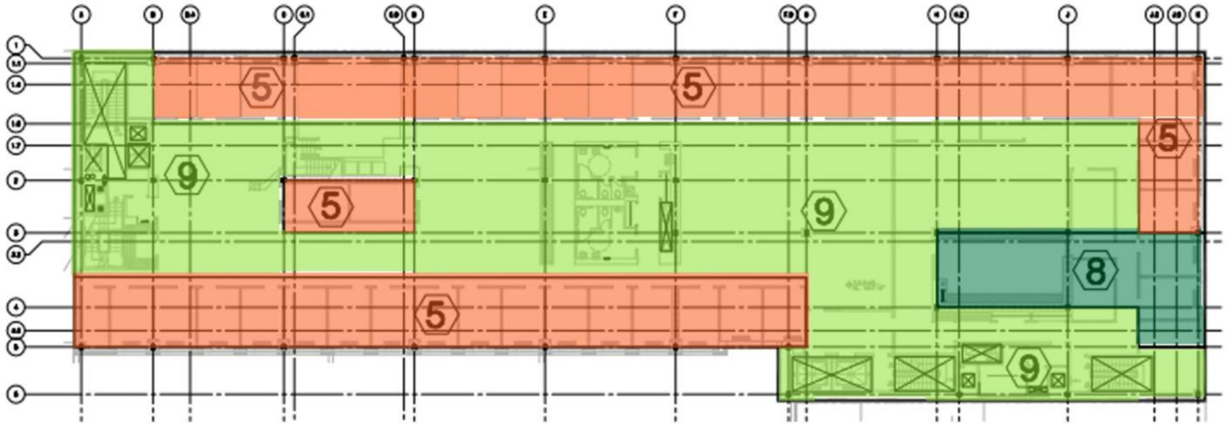
1 S-008 1ST FLOOR LOADING DIAGRAM SCALE: 1/16"=1'-0"



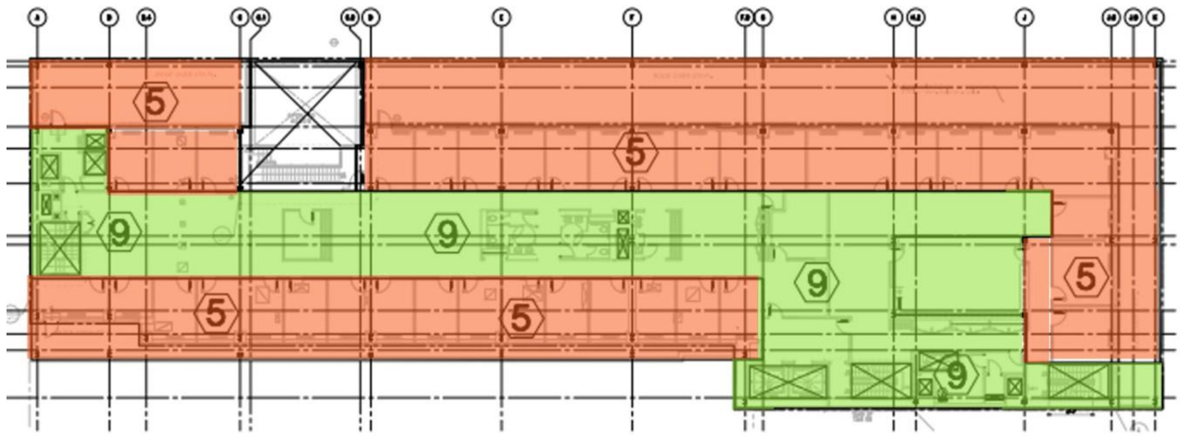
2 2ND FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



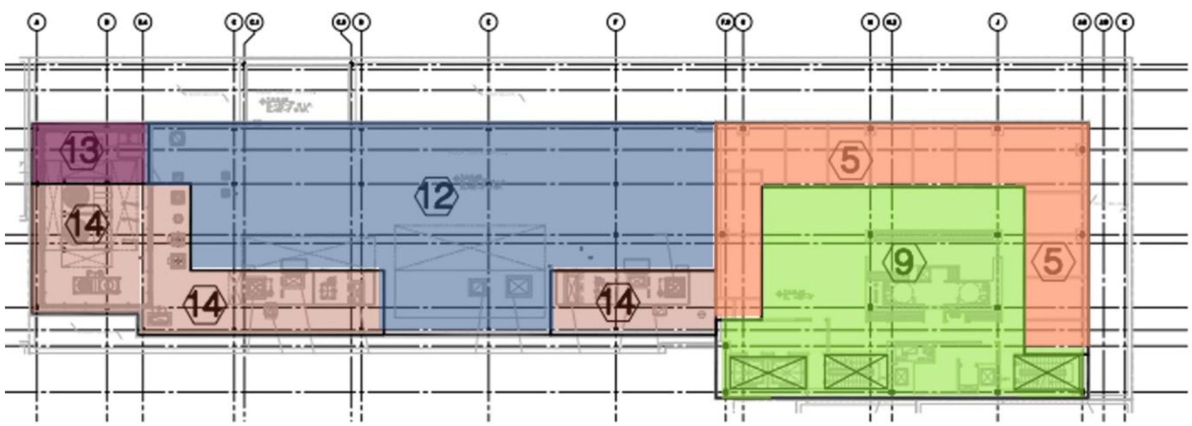
3 3RD FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



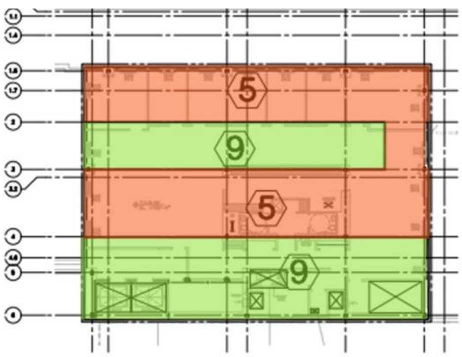
4TH FLOOR LOADING DIAGRAM
SCALE: 1/8"=1'-0"



5TH FLOOR LOADING DIAGRAM
SCALE: 1/8"=1'-0"

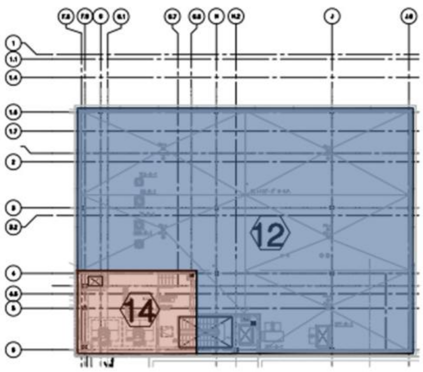


6TH FLOOR LOADING DIAGRAM
SCALE: 1/8"=1'-0"

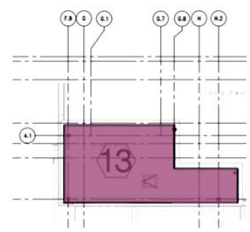


7 7TH FLOOR LOADING DIAGRAM

8 8TH FLOOR LOADING DIAGRAM



9 ROOF LOADING DIAGRAM



10 PENTHOUSE LOADING DIAGRAM