Technical Report #3

John Jay College Expansion Project

New York, NY

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Structural Option

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Executive Summary

The John Jay College Expansion project is an academic building for the John Jay College of Criminal Justice located in Manhattan. A midrise tower includes classroom, laboratory, and office spaces and reaches a maximum height of approximately 240 feet above 11th avenue. This tower is connected to the existing building by a 5 story "grand cascade".

In the third technical report of the John Jay College Expansion Project, an in-depth lateral analysis was performed. Existing lateral force-resisting systems are ordinary steel braced frames. A 14 story braced frame core is utilized in the tower of the expansion project, and a 5 story braced frame resists lateral forces in the cascade. There is a series of trusses at the penthouse level of the 14 story tower, which transfer gravity loads from hanging plate hangers supporting floors 6 through 12 to the braced frame core.

A computer model of the John Jay College Expansion Project was created using ETABS. This model included the existing braced frame members - with modifications to the penthouse level - and rigid diaphragms connecting the frames at each level. Wind and seismic loads were calculated using ASCE 7-05 and were applied to the ETABS model.

The distribution of lateral forces to the braced frames was based on the relative stiffness of each frame. This method of distributing lateral forces was verified by determining the amount of direct and torsional shear each frame would experience at the 8th level. Discrepancies between the hand calculations and the ETABS model for story shear can be attributed the inaccurate calculation of the center of rigidity for the hand calculations.

Several combinations of lateral loads were considered to find the worst case for drift and strength requirements. It was determined that applying wind forces in the East-West and North-South direction separately created the maximum building drift. Although wind forces created the maximum displacements, strict seismic drift limitations required by ASCE 7-05 determined that the seismic drift values were unacceptable in the North-South direction.

After verifying that the ETABS model was distributing forces accurately based on relative stiffness, the model was used to obtain the forces for certain lateral members. Lateral braces were analyzed for strength at several levels, and were determined to be adequate for strength. Columns were also analyzed for strength when subjected to lateral and gravity loads. They were also determined to be sufficient for the loading conditions.

Introduction

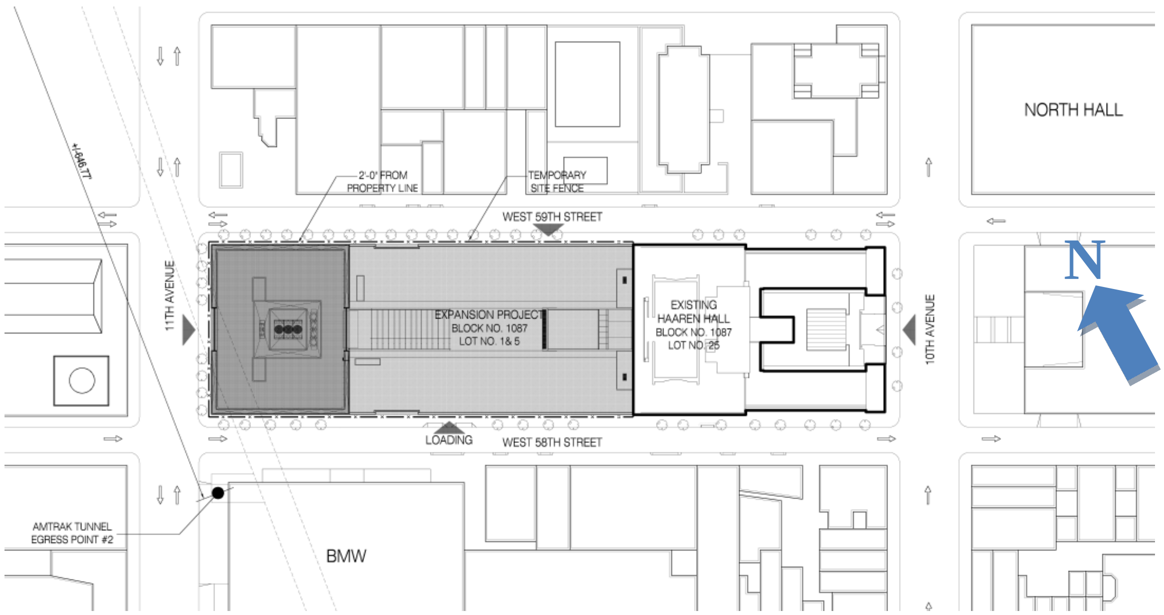


Figure 1 – Site plan

This major expansion project in Manhattan will unify the City University of New York's John Jay College of Criminal Justice into a one block campus that will "demonstrate the transparency of justice". The design includes a mid-rise tower situated on the west side of the site, which will contain classrooms, forensic laboratories, department offices, several student lounge spaces, a "moot" courtroom, a café, and a student bookstore.

A mid-rise structure connects the expansion to Haaren Hall (the existing building) and calls for a multi-level grand cascade, which also serves as a main lounge space for students (see picture 1 below). The connection also contains classrooms, a black box theater, and two cyber cafes. A landscaped roof accommodates outdoor lounge and dining areas, and an outdoor commons.

Technical Assignment #3

Picture 1 – Rendering of the Grand Cascade

Amtrak tracks cross the south-west corner of the site, which is beneath the mid-rise tower. This restriction led to a unique structural solution to transfer over the tracks. Floors 1 through 5 are transferred over the tracks using built-up steel transfer girders and floors 6 through 14 are hanging from perimeter plate hangers supported at the penthouse level by transfer trusses that are one-story tall (see figure 3 below). These trusses then transfer the loads to the lateral force-resisting system, which is a steel braced frame. This braced frame wraps around a centralized service core located in the 14 story tower. A braced frame is also utilized in the service core of the 5 story cascade.

The remainder of this report investigates the existing lateral force-resisting system. A detailed lateral analysis was performed using ETABS and forces were verified using hand calculations. Members of the braced frame were checked for strength and drift requirements. Throughout this report, braced frames will be referred to as shown below in figure 2.

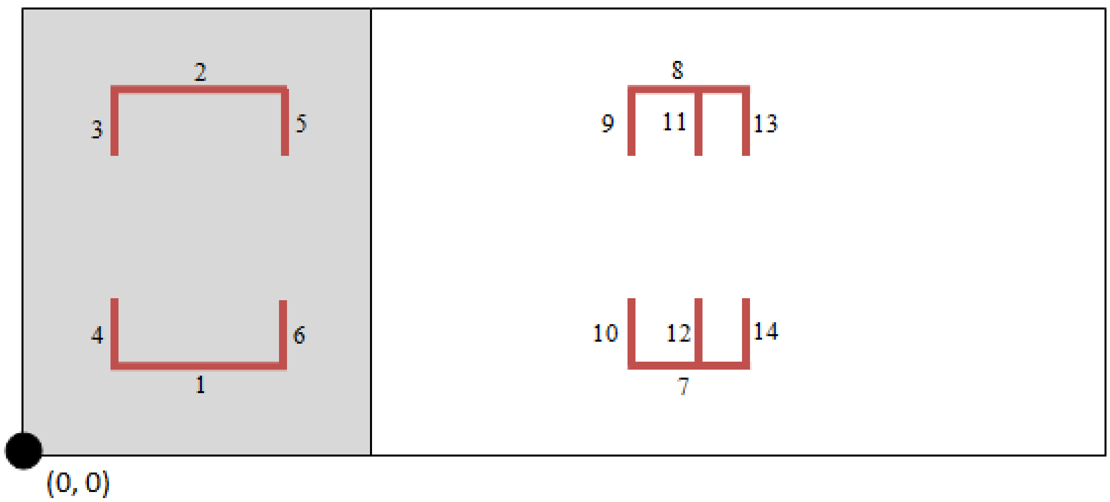


Figure 2 – Plan view of labeled braced frames

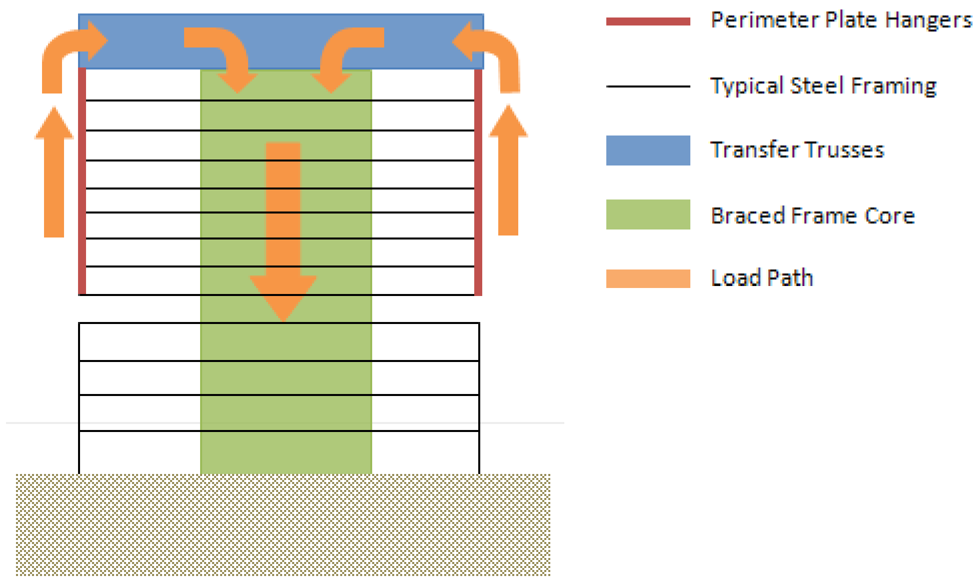


Figure 3 – Schematic diagram of the load path for the 14 story tower

Existing Composite Steel System

Floor System:

The floor system of the John Jay College Expansion Project is a composite system with the most typical bay size being 30’-0”x 37’-10”. 3 ½” light weight concrete and 3” metal decking typically span 12’-2” to W14x22 or W16x26 infill beams. ¾” diameter x 5 ½” long shear studs allow composite action between the floor system and beams. Infill beams span into W-shape girders of varying sizes or two back-to-back MC-shapes. Framing of the cascade, which connects the tower to the existing building (Haaren Hall), consists of W36 girders spanning 68’-4” with infill beams spaced typically at 11’-4” on center. See Appendix A for typical floor framing plans.

Lateral system:

The 14 story tower of the expansion project has a large centralized braced frame core (see Figure 4). This braced frame surrounds the vertical shafts of the building, such as elevator shafts, stairwells, mechanical shafts, and plumbing. Columns of the braced frames are heavy W14 sections and the beams are typically W16 sections. HSS 6x6x3/8 are typically used for diagonal bracing at the 13th level and HSS 8x8x3/8 are used for the diagonal bracing at the 1st level. Reinforced concrete walls span between caissons and concrete piers at the foundation, which support the braced frame.

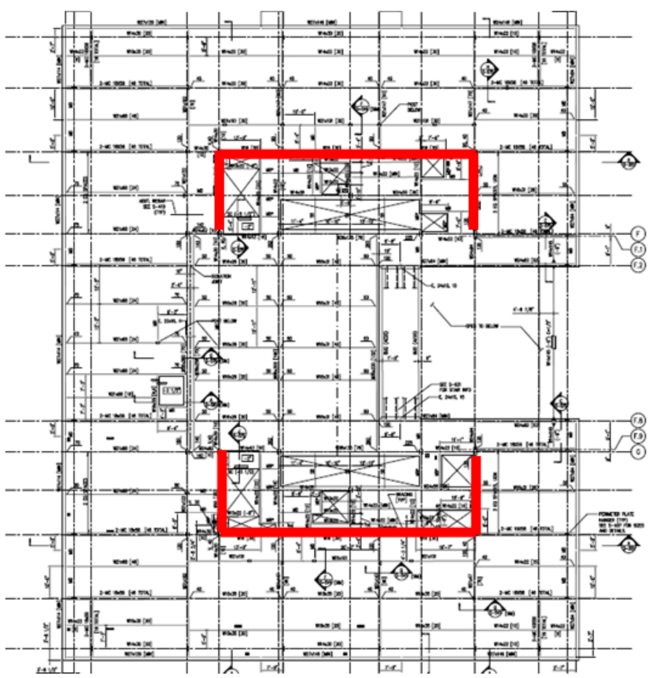


Figure 4 – Location of the Braced Frames in the tower

The lateral system for the 5 story cascade is also a braced frame which encases the building's vertical circulation shafts (see Figure 5). Columns of these braced frames are lighter W14 sections than the 14 story braced frame and the beams are W16x31's and W21x94's. Diagonal braces are typically 2L 6x4's with varying thicknesses.

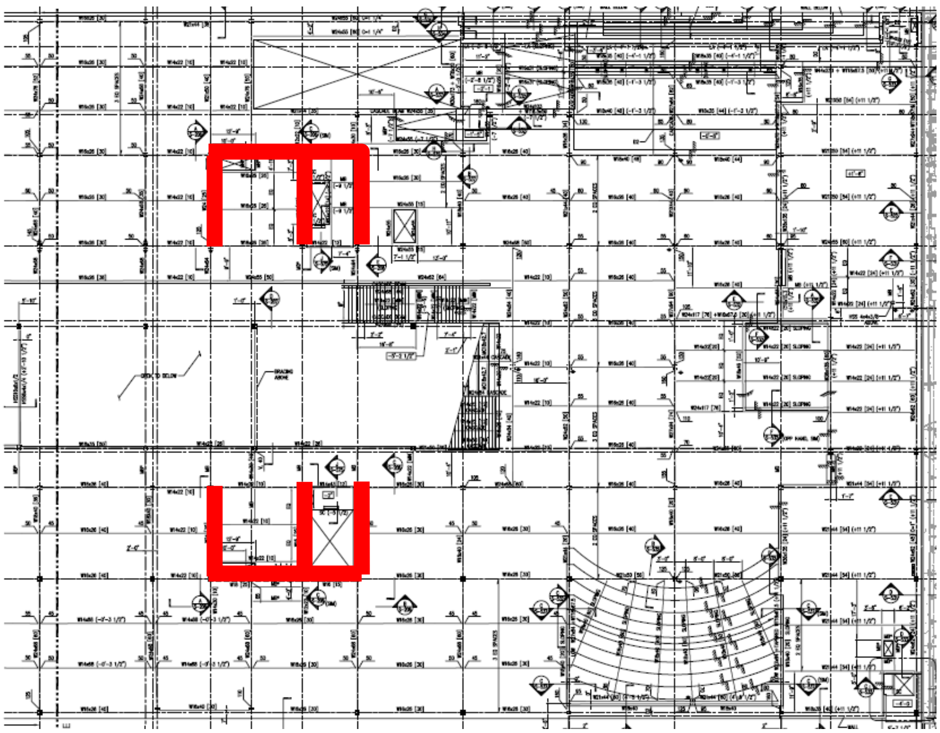


Figure 5 – Location of the Braced Frames in the 5 story cascade

Codes, Design Criteria, and Load Combinations

Codes:

It should be noted that the original design for the John Jay College Expansion project used the Building Code of the City of New York. The designers also used ASD for sizing the members of the lateral force-resisting system. This report will use the following codes and standards, as well as LRFD to perform spot checks:

National Model Code:

2006 International Building Code

Structural Standards:

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

Design Codes:

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

Deflection Criteria:

Allowable Building Drift: $\Delta_{wind} = H/400$

Allowable Story Drift: $\Delta_{seismic} = 0.015h_{sx}$

Load Cases and Combinations:

The following load combinations are considered in this report and are taken from section 2.3 of ASCE 7-05.

$1.4D$
$1.2D + 1.6L + 0.5L_r$
$1.2D + 1.6L_r + (1.0L \text{ or } 0.8W)$
$1.2D + 1.6W + 1.0L + 0.5L_r$
$1.2D + 1.0E + 1.0L$
$0.9D + 1.6W$
$0.9D + 1.0E$

Due to the rectangular geometry of the building, wind load combinations one and three in Figure 6-9 of ASCE 7 -05 were applied. Wind load cases two and four were not analyzed and will be considered in future studies. Snow loads were also not considered in this analysis.

Building Loads

The following loads were determined in technical report one. ASCE 7 – 05 was used to determine both gravity and lateral loads.

Gravity Loads:

Construction Dead Loads:

Typical floor Construction:

3” Metal Decking: 20 Gage Minimum	3 psf
3 ½” Lightweight Concrete Slab (115 psf)	48 psf
Allowance for Self Weight of Steel Framing	7 psf
Total CDL for Floor System Design:	51 psf
Total CDL for Seismic Calculations:	58 psf

Mechanical and Mezzanine floor Construction:

3” Metal Decking: 20 Gage Minimum	3 psf
4 ½” Normal weight Concrete Slab	75 psf
Allowance for Self Weight of Steel Framing	7 psf
Total CDL for Floor System Design:	78 psf
Total CDL for Seismic Calculations:	85 psf

Superimposed Dead Loads:

Typical floor Construction:

Fireproofing	2 psf
Finishes	5 psf
Partitions	20 psf
Ceiling	5 psf
Mech. & Electrical Distribution	5 psf
Total SDL:	37 psf

Live Loads:

Space	Load
Classrooms	40 psf
Offices	50 psf
Lobbies & Corridors	100 psf
Cascade	100 psf (assume corridor/lobby/bleachers)
Stairs	100 psf
Assembly areas (moot court and quad spaces)	60 psf (fixed seats) 100 psf (movable seats)
Roof	20 psf

Heavy Mechanical Equipment:

Location	Load
6th, 7th, & 8th Floor: Increased loads in laboratory spaces	100 psf (assumed)
Penthouse Mezzanine Level	63 kips (Total load)
Penthouse Level	853 kips (Total Load)

Lateral Loads:

Wind Loads:

See figure 6 and 7 below for the wind loads which were applied at the center of pressure for each level of the John Jay College Expansion Project. For detailed calculations and the assumptions used when determining the wind forces, see appendix C.

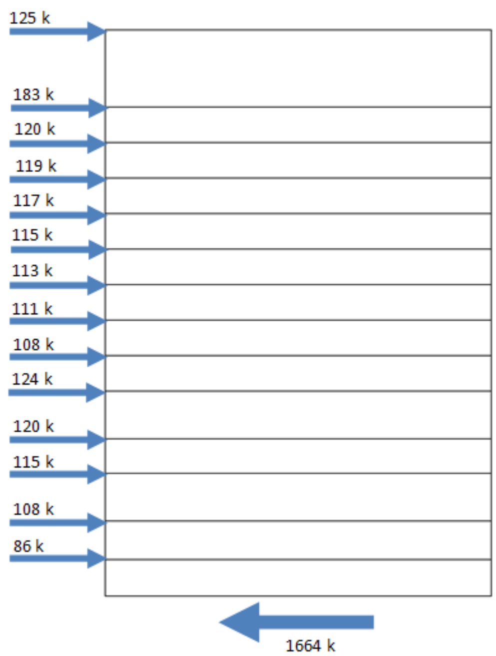


Figure 6 – East-West Wind Force Diagram

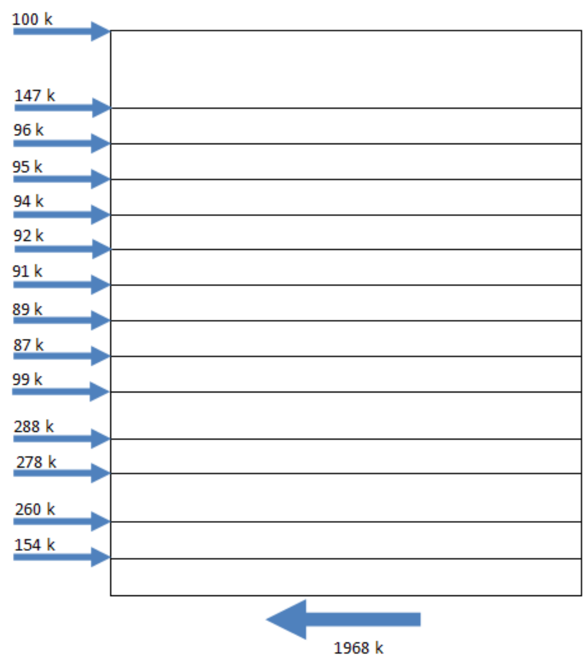


Figure 7 – North-South Wind Force diagram.
Note: The first 5 levels of the N-S direction are approximately 500 ft wide. See appendix C for calculations.

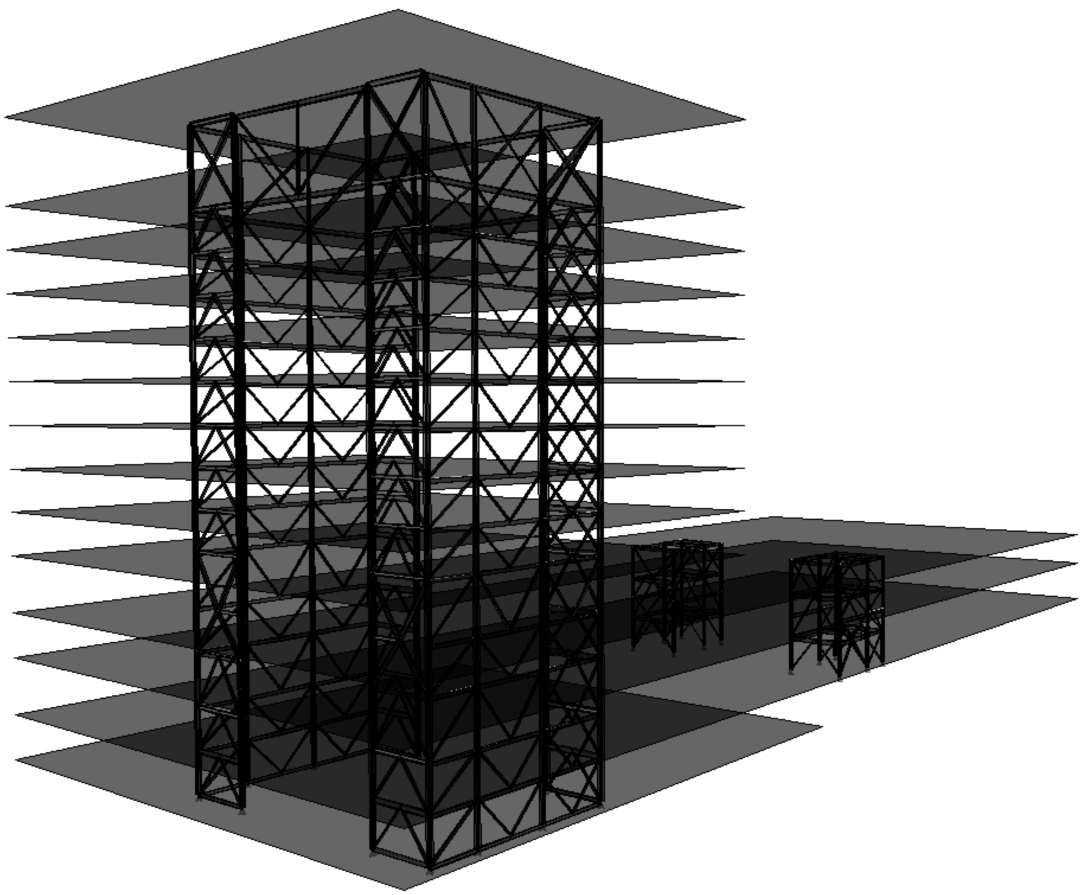
Seismic Loads:

Seismic loads were determined in technical report one with a few minor changes. Forces were calculated using chapters 11 and 12 of ASCE 7 – 05. Listed below in table 1 are the seismic loads which were applied in both directions for the ETABS model at the center of mass. Please see appendix D for the assumptions used when calculating seismic loads.

Level	**Story Weight** w_x (Kips)	**Height** h_x (ft)	$w_x h_x^k$	C_{vx}	**Lateral Force** F_x (kips)	**Story Shear** V_x (kips)	**Moment** M_x (ft-k)
Roof	3286	236.67	5529107	0.134	139	0	32803
Penthouse	6502	206.67	9101073	0.221	228	139	47151
13	2874	191.67	3631231	0.088	91	367	17447
12	2822	176.67	3191293	0.077	80	458	14134
11	3040	161.67	3047887	0.074	76	538	12352
10	2638	146.67	2317053	0.056	58	614	8519
9	3040	131.67	2306064	0.056	58	672	7612
8	2870	116.67	1847361	0.045	46	730	5403
7	2929	101.67	1563720	0.038	39	776	3985
6	3785	86.67	1626559	0.039	41	816	3534
5	12565	66.67	3780295	0.092	95	856	6318
4	8483	51.17	1781485	0.043	45	951	2285
3	10119	31.17	1083535	0.026	27	996	847
2	10932	15.58	456219	0.011	11	1023	178
Total	81866	236.67	41262883	1.000	1034	1034	162568

Table 1 – Lateral forces, story shears, and overturning moments due to seismic forces

ETABS Model



ETABS was used to perform a detailed lateral analysis of the John Jay College Expansion Project. This analysis was performed by modeling the existing lateral systems, as well as treating each floor as a rigid diaphragm. Gravity members were neglected in this analysis, but were accounted for when calculating the building's weight for seismic loads. Wind loads were applied at the center of pressure of each level and seismic loads were placed at the center of mass of each level. Five lateral load cases were investigated:

- 100 percent of the wind forces in the East-West direction
- 100 percent of the wind forces in the North-South direction
- 75 percent of the East-West and North-South wind forces applied simultaneously
- Seismic loads in the East-West direction
- Seismic loads in the North-South direction

As mentioned above, all of the required combinations of lateral loads were not studied in this report. Wind load cases two and four in Figure 6-9 of ASCE 7 – 05 will be investigated in more detail in the future.

A few important assumptions were used during the modeling procedure. The penthouse level has several cantilevered trusses which were not completely modeled. To account for the stiffness of these additional trusses at the penthouse level, members within the braced frames were modified by an area factor of 5. Trusses connecting frames 3 to 4 and 5 to 6 were modeled with no property modifiers to allow for a realistic shear transfer between frames. The differences between the existing braced frames and those modeled in ETABS can be found in appendix B.

Lateral Force Distribution and Analysis

Distribution of lateral forces to the braced frames was determined based on the relative stiffness of each braced frame. Floor diaphragms are assumed to be infinitely rigid, and therefore distribute lateral loads to each frame based on their stiffness. Two hand calculation methods were used to calculate the center of rigidity at the 8th level using the South-West corner of the building as the origin. From this point, direct shear and shear due to torsion about the center of rigidity were calculated and compared to story shear results provided by ETABS.

The first method to calculate the center of rigidity was to directly calculate the x and y-coordinates from the approximate stiffness of each frame. To do this, a separate ETABS model was created by removing the rigid diaphragms. A one kip load was then applied to each frame at the 8th level. Torsion was neglected to avoid any additional shear from torsion being applied to the frames. Finally, this stiffness was calculated by taking the inverse of the displacement of each frame at the 8th level. Table 2 below displays the approximate stiffness of each frame using method one.

Approximate Frame Stiffness Calculation: Method 1						
Location	Direction	Braced Frame	Load (kips)	Displacement (in)	Stiffness (kips/in)	Stiffness Factor
Level 8	E-W	1	1	0.0043	233	0.282
	E-W	2	1	0.0043	233	0.282
	N-S	3	1	0.011682	86	0.104
	N-S	4	1	0.011706	85	0.104
	N-S	5	1	0.010665	94	0.114
	N-S	6	1	0.010667	94	0.114
				Total =	824	

Table 2 – Approximate frame stiffness

From this information, the center of rigidity was found to be at X_r = 81.84 ft and Yr = 100 ft. This method neglects considerations of the center of rigidities of the stories below, and therefore is fairly inaccurate due to the contributions of levels 5 and below, where the x-coordinate of the center of rigidity significantly changes (see table 5 and figure 8). A summary of direct shear forces and shear forces generated from torsion about the center of rigidity when a North-South wind force of 714 kips was applied at the center of pressure can be found below in table 3.

Level 8 Shear Forces: Method 1					
Frame	Direct Shear (kips)	Shear from Torsion (kips)	Net Shear (kips)	ETABS Shear (kips)	% Error
1	0.00	-8.15	-8.15	-84.10	-90.30
2	0.00	8.15	8.15	84.10	-90.30
3	170.47	-2.09	168.38	149.11	12.92
4	170.12	-2.09	168.03	149.08	12.71
5	186.72	2.09	188.81	208.44	-9.42
6	186.69	2.09	188.78	208.36	-9.40

Table 3– Story shear forces at level 8

The second method used to calculate the center of rigidity was more accurate than the first. Another ETABS model was created with rigid diaphragms at each story and story shears due to wind loads in each direction were applied at the center of pressure of the 8th level. Then, the load distributed to each frame at the 8th level was determined as a percentage of the total story shear. These percentages were multiplied by the specified frames distance to the origin. This method is a simple way to find the center of rigidity without directly calculating the stiffness of each frame. Findings for this method are presented below in table 4, which resulted in a center of rigidity at X_r = 96.41 ft and Yr = 98.5 ft.

Level 8 Approximate COR Check: Method 2				
Frame (dir.)	Load Applied in Diaphragm (kips)	Distribution (kips)	Percentage	Distance to Origin (ft)
1 (E-W)	892	439.2	0.492	40
2 (E-W)	892	439.2	0.492	160
3 (N-S)	714	162.5	0.228	45
4 (N-S)	714	167.3	0.234	45
5 (N-S)	714	218.2	0.306	125
6 (N-S)	714	213.8	0.299	125

Table 4 – Approximate method for determining the center of rigidity at level 8

Table 5 below displays the x and y-components of the center of rigidity for each level as calculated by the ETABS model. As you can see, the x-coordinate varies with the height of the building due to the location of the braced frames in the five story cascade, while the y-coordinate remains constant due to symmetrically placed braced frames with identical stiffness.

ETABS Center of Rigidity		
Level	Xr (ft)	Yr (ft)
14	103	100
13	103	100
12	104	100
11	105	100
10	107	100
9	108	100
8	110	100
7	113	100
6	118	100
5	129	100
4	227	100
3	240	100
2	269	100

Table 5 – COR locations as calculated by ETABS from the South-West corner of the building

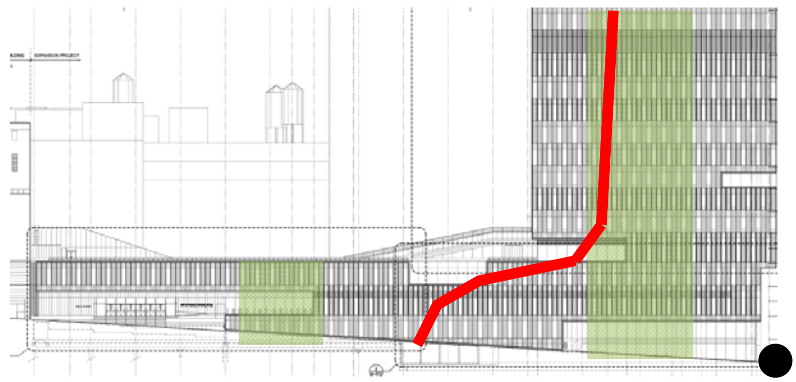


Figure 8– North elevation showing the location of Xr (0, 0)

- Braced Frames
- Location of Xr

The differences in the 14 story tower between the center of rigidity and the center of pressure, as well as the center of rigidity and the center of mass, create a significant amount of shear to be resisted by the braced frames. This is verified by the ETABS model and by the hand calculations above in method 1. When viewing the animation of the natural period about the vertical axis, the tower twists about the center of rigidity, while the lower levels remain relatively stationary.

Analysis Results

The three dimensional model created in ETABS was built with the intention of obtaining more realistic results than those calculated by hand. After performing hand calculations and comparing them to the results from the detailed lateral analysis from ETABS, the discrepancies in story shear can be explained by the differences in the shear due to torsion. More accurate centers of rigidity were calculated with ETABS, by accounting for the effect shown in figure 8 caused by the braced frames in the 5-story cascade.

Story shears were determined to control in the analysis by using wind loads from wind load case 1 of figure 6-9 in ASCE 7 -05. Tables 6 and 7 present story shears for each frame in the John Jay College Expansion Project. Discrepancies between total base shears and the base shear listed in the wind loading table in appendix C is due to the contribution of the out-of-plane frames.

Tower Braced Frames: Story shear						
Level	East-West Frames (kips)		North-South Frames (kips)			
	1	2	3	4	5	6
Roof	0	0	0	0	0	0
14	63.1	63.1	83.6	83.6	17.0	17.0
13	153.7	153.8	60.8	60.8	61.7	58.7
12	214.2	213.9	76.9	76.9	96.0	96.5
11	273.6	273.6	92.7	92.7	125.0	125.1
10	329.5	329.5	111.4	111.4	155.4	155.3
9	390.4	390.9	128.9	128.9	182.4	182.5
8	446.1	446.1	149.1	149.1	208.4	280.4
7	501.3	501.4	168.0	168.0	233.5	233.5
6	559.8	559.8	188.2	188.2	258.4	258.4
5	606.1	606.1	272.3	272.3	217.4	217.4
4	407.6	408.6	348.1	348.1	137.4	137.4
3	386.5	386.5	270.0	270.0	235.4	235.4
2	334.5	344.5	295.9	295.8	230.7	230.7
BASE	375.1	375.1	343.8	343.8	253.7	253.6

Table 6 – Tower Braced Frame story shears due to wind loading

Cascade Braced Frames: Story Shear								
Level	East-West Frames (kips)		North-South Frames (kips)					
	7	8	9	10	11	12	13	14
5	0	0	0	0	0	0	0	0
4	0	0	74.1	73.9	58.5	59.5	33.9	33.1
3	335.2	335.3	108.3	108.3	85.8	85.7	68.4	68.5
2	448.0	448.4	197.8	197.8	118.4	118.4	71.5	71.2
BASE	448.0	448.4	197.8	197.8	118.4	118.4	71.5	71.2

Table 7 – Cascade Braced Frame story shear due to wind loading

Serviceability Check:

Wind and Seismic drifts computed by ETABS were compared to code drift limitations. Drift due to wind was compared to Δ_{wind} = H/400 for the entire building in both directions (see table 8 and 9). Seismic drift was compared to $\Delta_{seismic} = 0.015h_{sx}$ for each floor level in both directions (see table 10 and 11).

Controlling Wind Drift: East -West Direction									
Story	Story Height	Story Drift	Allowable Story Drift Δ_{wind} = H/400			Total Drift	Allowable Total Drift Δ_{wind} = H/400		
	(ft)	(in)	(in)			(in)	(in)		
Roof	236.67	0.165	<	0.900	Acceptable	2.85	<	7.10	Acceptable
14	206.67	0.133	<	0.450	Acceptable	2.69	<	6.20	Acceptable
13	191.67	0.160	<	0.450	Acceptable	2.56	<	5.75	Acceptable
12	176.67	0.187	<	0.450	Acceptable	2.40	<	5.30	Acceptable
11	161.67	0.210	<	0.450	Acceptable	2.21	<	4.85	Acceptable
10	146.67	0.210	<	0.450	Acceptable	2.00	<	4.40	Acceptable
9	131.67	0.219	<	0.450	Acceptable	1.79	<	3.95	Acceptable
8	116.67	0.230	<	0.450	Acceptable	1.57	<	3.50	Acceptable
7	101.67	0.248	<	0.450	Acceptable	1.34	<	3.05	Acceptable
6	86.67	0.368	<	0.600	Acceptable	1.09	<	2.60	Acceptable
5	66.67	0.185	<	0.465	Acceptable	0.73	<	2.00	Acceptable
4	51.17	0.232	<	0.600	Acceptable	0.54	<	1.54	Acceptable
3	31.17	0.142	<	0.468	Acceptable	0.31	<	0.94	Acceptable
2	15.58	0.166	<	0.467	Acceptable	0.17	<	0.47	Acceptable

Table 8 – Calculated vs. allowable wind drift in the East-West direction

Controlling Wind Drift: North -South Direction									
Story	Story Height	Story Drift	Allowable Story Drift Δ_{wind} = H/400			Total Drift	Allowable Total Drift Δ_{wind} = H/400		
	(ft)	(in)	(in)			(in)	(in)		
Roof	236.67	0.153	<	0.900	Acceptable	4.85	<	7.10	Acceptable
14	206.67	0.206	<	0.450	Acceptable	4.70	<	6.20	Acceptable
13	191.67	0.245	<	0.450	Acceptable	4.49	<	5.75	Acceptable
12	176.67	0.306	<	0.450	Acceptable	4.25	<	5.30	Acceptable
11	161.67	0.340	<	0.450	Acceptable	3.94	<	4.85	Acceptable
10	146.67	0.377	<	0.450	Acceptable	3.60	<	4.40	Acceptable
9	131.67	0.396	<	0.450	Acceptable	3.22	<	3.95	Acceptable
8	116.67	0.408	<	0.450	Acceptable	2.83	<	3.50	Acceptable
7	101.67	0.413	<	0.450	Acceptable	2.42	<	3.05	Acceptable
6	86.67	1.292	>	0.600	Unacceptable	2.01	<	2.60	Acceptable
5	66.67	0.156	<	0.465	Acceptable	0.72	<	2.00	Acceptable
4	51.17	0.299	<	0.600	Acceptable	0.56	<	1.54	Acceptable
3	31.17	0.017	<	0.468	Acceptable	0.26	<	0.94	Acceptable
2	15.58	0.276	<	0.467	Acceptable	0.28	<	0.47	Acceptable

Table 9 - Calculated vs. allowable wind drift in the North-South direction

Seismic Drift: East -West Direction									
Story	Story Height	Story Drift	Allowable Story Drift $\Delta_{seismic} = 0.015h_{sx}$			Total Drift	Allowable Total Drift $\Delta_{seismic} = 0.015h_{sx}$		
	(ft)	(in)	(in)			(in)	(in)		
Roof	236.67	0.154	<	0.450	Acceptable	2.29	<	3.55	Acceptable
14	206.67	0.137	<	0.225	Acceptable	2.14	<	3.10	Acceptable
13	191.67	0.158	<	0.225	Acceptable	2.00	<	2.88	Acceptable
12	176.67	0.176	<	0.225	Acceptable	1.84	<	2.65	Acceptable
11	161.67	0.190	<	0.225	Acceptable	1.67	<	2.43	Acceptable
10	146.67	0.181	<	0.225	Acceptable	1.48	<	2.20	Acceptable
9	131.67	0.180	<	0.225	Acceptable	1.30	<	1.98	Acceptable
8	116.67	0.180	<	0.225	Acceptable	1.12	<	1.75	Acceptable
7	101.67	0.186	<	0.225	Acceptable	0.94	<	1.53	Acceptable
6	86.67	0.262	<	0.300	Acceptable	0.75	<	1.30	Acceptable
5	66.67	0.131	<	0.233	Acceptable	0.49	<	1.00	Acceptable
4	51.17	0.160	<	0.300	Acceptable	0.36	<	0.77	Acceptable
3	31.17	0.095	<	0.234	Acceptable	0.20	<	0.47	Acceptable
2	15.58	0.102	<	0.234	Acceptable	0.10	<	0.23	Acceptable

Table 10 – Calculated vs. allowable seismic drift in the East-West direction

Seismic Drift: North -South Direction										
Story	Story Height	Story Drift	Allowable Story Drift $\Delta_{seismic} = 0.015h_{sx}$			Total Drift	Allowable Total Drift $\Delta_{seismic} = 0.015h_{sx}$			
	(ft)	(in)	(in)			(in)	(in)			
Roof	236.67	0.165	<	0.450	Acceptable	4.55	>	3.55	Unacceptable	
14	206.67	0.231	>	0.225	Unacceptable	4.39	>	3.10	Unacceptable	
13	191.67	0.269	>	0.225	Unacceptable	4.16	>	2.88	Unacceptable	
12	176.67	0.335	>	0.225	Unacceptable	3.89	>	2.65	Unacceptable	
11	161.67	0.363	>	0.225	Unacceptable	3.55	>	2.43	Unacceptable	
10	146.67	0.390	>	0.225	Unacceptable	3.19	>	2.20	Unacceptable	
9	131.67	0.399	>	0.225	Unacceptable	2.80	>	1.98	Unacceptable	
8	116.67	0.397	>	0.225	Unacceptable	2.40	>	1.75	Unacceptable	
7	101.67	0.389	>	0.225	Unacceptable	2.00	>	1.53	Unacceptable	
6	86.67	1.151	>	0.300	Unacceptable	1.62	>	1.30	Unacceptable	
5	66.67	0.115	<	0.233	Acceptable	0.47	<	1.00	Acceptable	
4	51.17	0.191	<	0.300	Acceptable	0.35	<	0.77	Acceptable	
3	31.17	0.018	<	0.234	Acceptable	0.16	<	0.47	Acceptable	
2	15.58	0.176	<	0.234	Acceptable	0.18	<	0.23	Acceptable	

Table 11 – Calculated vs. allowable seismic drift in the North-South direction

As displayed above in the tables, the total drift and story drift in the East-West direction is acceptable for both wind and seismic loads. However, several calculated story drifts and the total building drift in the North-South direction were unacceptable for seismic loading. Although the building displaced more under wind loading, strict drift limitations for an Occupancy Category of III caused seismic loads to govern the serviceability design of the North-South Direction.

These unacceptable drift calculations were not expected, since wind forces in New York City usually control the lateral force-resisting system design. This major discrepancy was justified by looking into the original design criteria of the John Jay College Expansion project and the Building Code of the City of New York. It was found that lateral drifts for seismic loads are limited to H/260 for the total building height and story height. By comparing the drift calculations above for seismic forces to the limit of H/260, it was determined that the total building drift and each story drift was acceptable and that wind forces governed the design for both strength and serviceability.

Overturning Analysis and Foundation Impact

An overturning analysis was performed for the 14 story braced frame core. It was expected that overturning would not be an issue in the North-South direction due to the coupling action through the transfer trusses at the penthouse level between braced frames 3 and 4, as well as braced frames 5 and 6. This prediction was verified in table 12 below when the uplift forces at edge columns due to overturning moments were compared to the dead load in the same edge columns. After analyzing the East-West direction, it was also determined that overturning will not be an issue.

Since the 14 story braced frame carries additional gravity loads from the hanging floors, it was very unlikely that overturning would be an issue. This theory is verified through the calculations in table 12 and therefore the foundation system does not need to be designed to resist overturning forces.

Tower Braced Frames: Story forces						
Level	East-West Frames (kips)		North-South Frames (kips)			
	1	2	3	4	5	6
Roof	63.1	63.1	83.6	83.6	17.0	17.0
14	90.6	90.6	-22.7	-22.7	44.7	41.7
13	60.4	60.1	16.1	16.1	34.3	37.7
12	59.4	59.7	15.8	15.8	29.0	28.6
11	55.9	55.9	18.7	18.7	30.4	30.2
10	60.8	61.4	17.5	17.5	27.0	27.2
9	55.7	55.2	20.3	20.3	26.1	25.9
8	55.3	55.3	18.9	18.9	25.1	25.1
7	58.4	58.4	20.2	20.2	24.9	24.8
6	46.3	46.3	84.1	84.1	-41.0	-41.0
5	-198.5	-197.6	75.8	75.8	-80.0	-80.0
4	-21.1	-22.1	-78.1	-78.1	98.0	98.0
3	-52.1	-42.0	25.8	25.8	-4.7	-4.6
2	40.6	30.6	47.9	47.9	22.9	22.9
BASE	375.1	375.1	343.8	343.8	253.7	253.6

Tower Braced Frames: Overturning Check				
	East-West Frame		North-South Frames	
	1	2	3 & 4 Coupled	5 & 6 Coupled
Overturning Moment (ft-k)	82154	82325	86720	78075
Base Dimension (ft)	80	80	120	120
Force at Edge Column (k)	1027	1029	723	651
Edge Column DL (k)	1083	1083	1083	1083
Overturning:	OK	OK	OK	OK

Table 12 – Story forces and overturning analysis for the 14 story braced frame core

At the 5th level of the John Jay College Expansion Project the braced frames in the cascade are discontinued. Therefore, there is a sudden change in stiffness of the overall lateral force-resisting systems at the same level. This is apparent in the story force data above in table 12 where there is a large shear reversal in the 5th level of the East-West frames.

Lateral Member Spot Checks

Member spot checks were performed for braced frame 1. Figure 9 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). These gravity loads included the loads transferred from the perimeter plate hangers to the braced frame core.

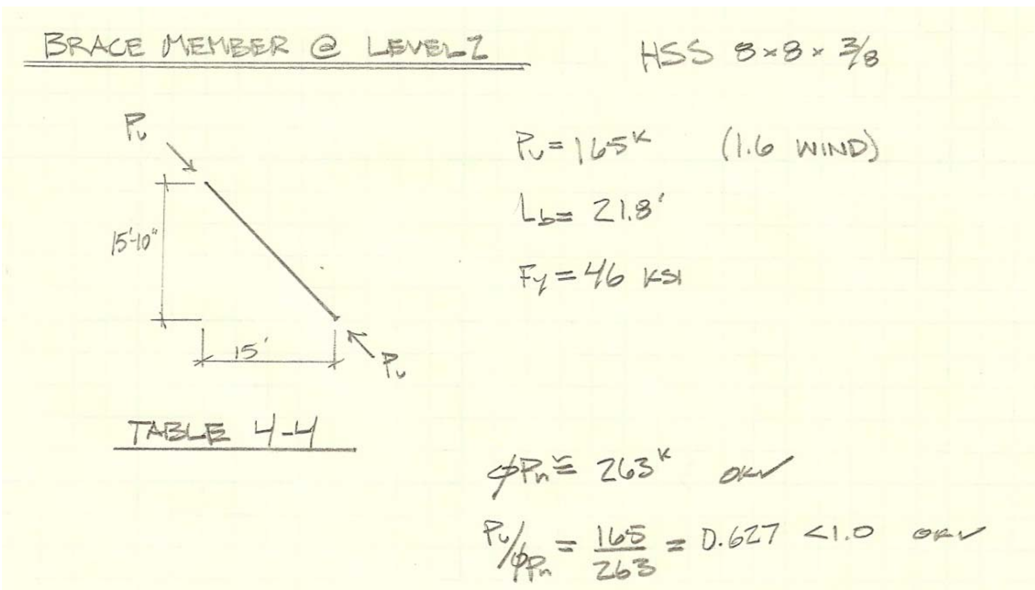
Figure 9 – Analyzed members of the lateral force-resisting system

Bracing Members:

Below are the calculations for each bracing member analyzed. Design of braced frame 1 was controlled by wind in the East-West direction, and therefore the controlling load combination was 1.6W.

Brace at Level 2:

Location (Member Size)	Load Case		P (k)	M (k-ft)
Level 2 Brace (HSS 8 x 8 x3/8)	Service	Wind East-West	103.03	
	Factored	1.6W	165	



BRACE MEMBER @ LEVEL 2

HSS 8×8×3/8

$P_u = 165^k$ (1.6 WIND)

$L_b = 21.8'$

$F_y = 46$ KSI

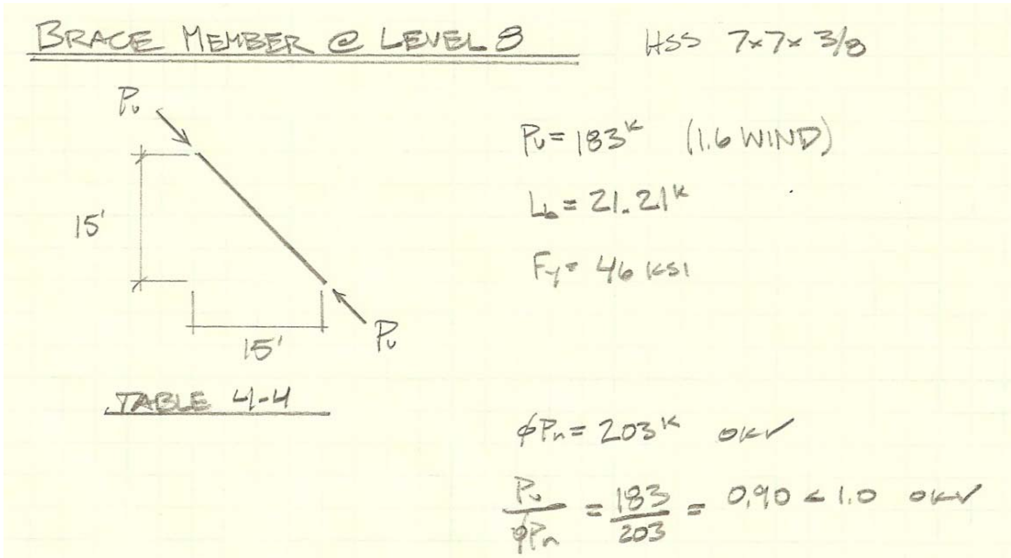
TABLE 4-4

$\phi P_n = 263^k$ OK✓

$P_u/\phi P_n = \frac{165}{263} = 0.627 < 1.0$ OK✓

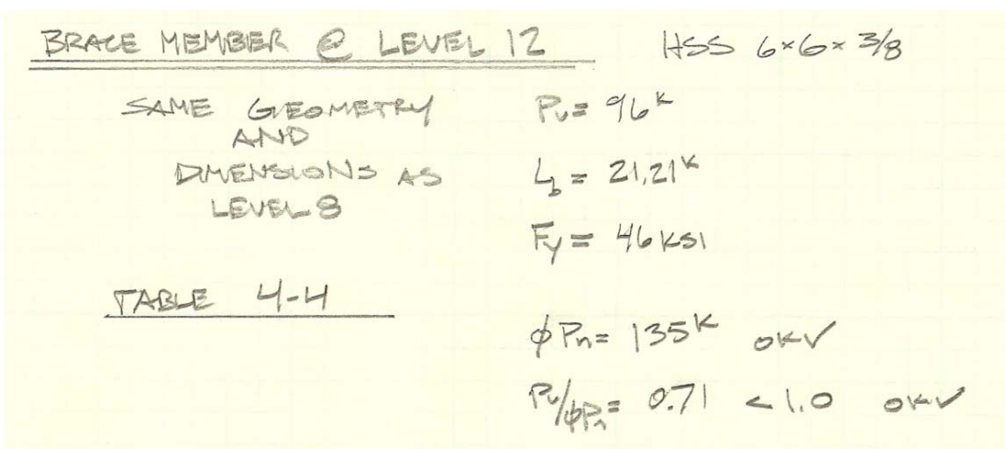
Brace at Level 8:

Location (Member Size)	Load Case		P (k)	M (k-ft)
Level 8 Brace (HSS 7 x 7 x3/8)	Service	Wind East-West	114.2	
	Factored	1.6W	183	



Brace at Level 12:

Location (Member Size)	Load Case		P (k)	M (k-ft)
Level 12 Brace	Service	Wind East-West	60.13	
(HSS 6 x 6 x3/8)	Factored	1.6W	96	



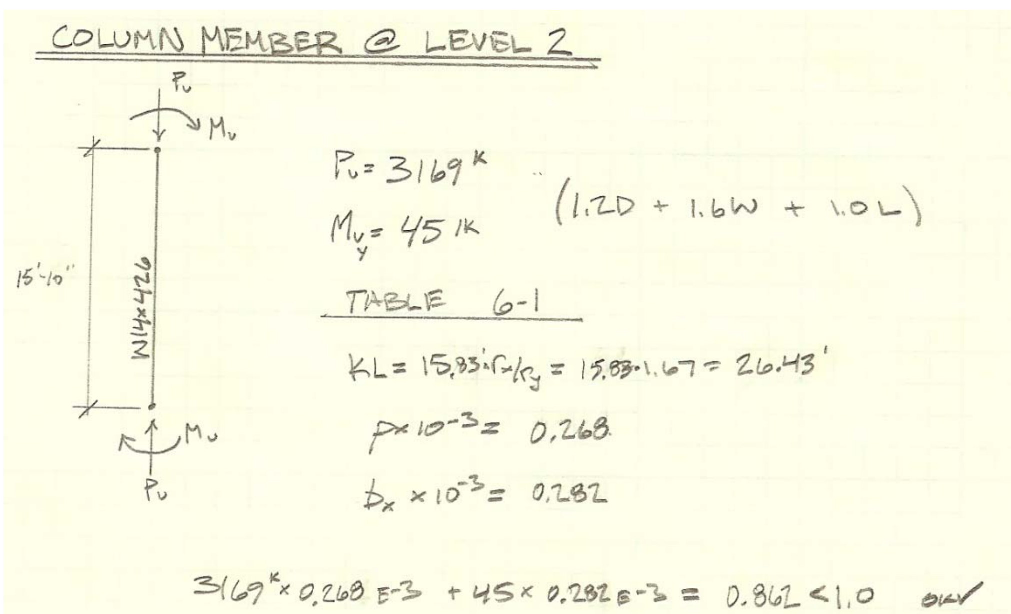
All of these members were determined to be more than adequate for strength requirements. Since none of these members are more than 90% stressed, it is believed they were sized based on drift requirements rather than strength.

Column Members:

The following calculations are for the column members analyzed in this report. Since these columns are end columns for braced frames 1 and 4, wind forces from the East-West and North-South directions were considered when determining the controlling load combination.

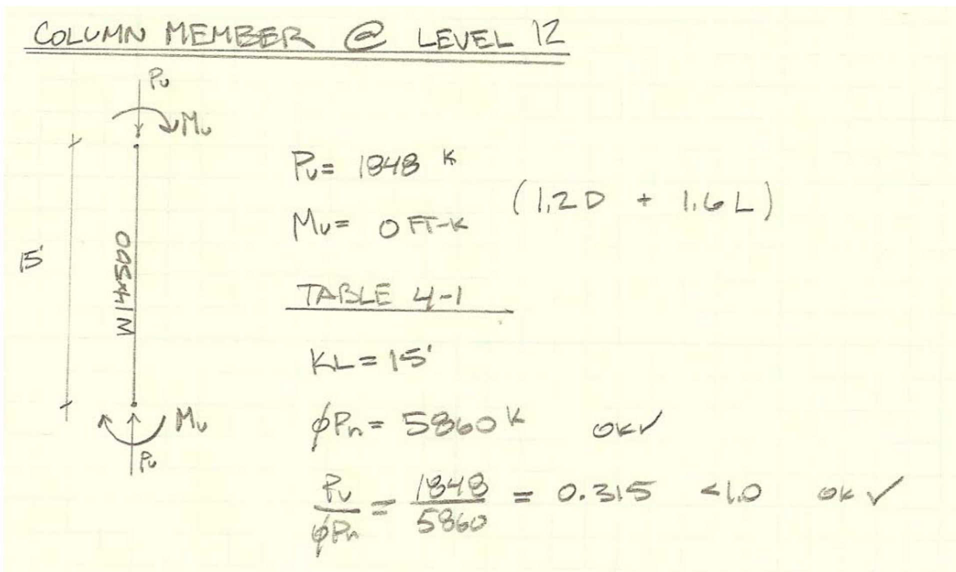
Column at Level 2:

Location (Member Size)	Load Case		P (k)	M (k-ft)
Level 2 column (W14 x 426)	Service	Wind East-West	564.83	27.5
		Wind North-South	679.5	28.325
		DL	999	0
		LL	884	0
	Factored	1.4D	1399	0
		1.2D + 1.6L	2612	0
		1.2D + 1.6W +1.0L	3169	45
		.9D + 1.6W	1986	45



Column at Level 12:

Location (Member Size)	Load Case		P (k)	M (k-ft)
Level 12 Column (W14 x 500)	Service	Wind East-West	13.67	17.024
		Wind North-South	133.8	15.2
		DL	718	0
		LL	616	0
	Factored	1.4D	1005	0
		1.2D + 1.6L	1848	0
		1.2D + 1.6W +1.0L	1692	24
		.9D + 1.6W	860	24



Both of these columns were determined to be adequate for the loads determined through a detailed lateral analysis. Design forces for the column at level 12 were controlled by 1.2D + 1.6L. This is justified by the large gravity loads at the 12th level, which is a unique situation for this building due to the braced frame core supporting gravity loads from perimeter plate hangers. Design forces for the column at the 2nd level were controlled by 1.2D + 1.6W +1.0L, which is the expected load combination for columns in a braced frame.

When analyzed for strength, the W14x500 at level 12 was determined to be 31.5 percent stressed, and therefore was sized to meet drift requirements. The W14x426 at the 2nd level was determined to be 86 percent stressed, and it is also believed that it is designed for drift requirements.

Conclusions

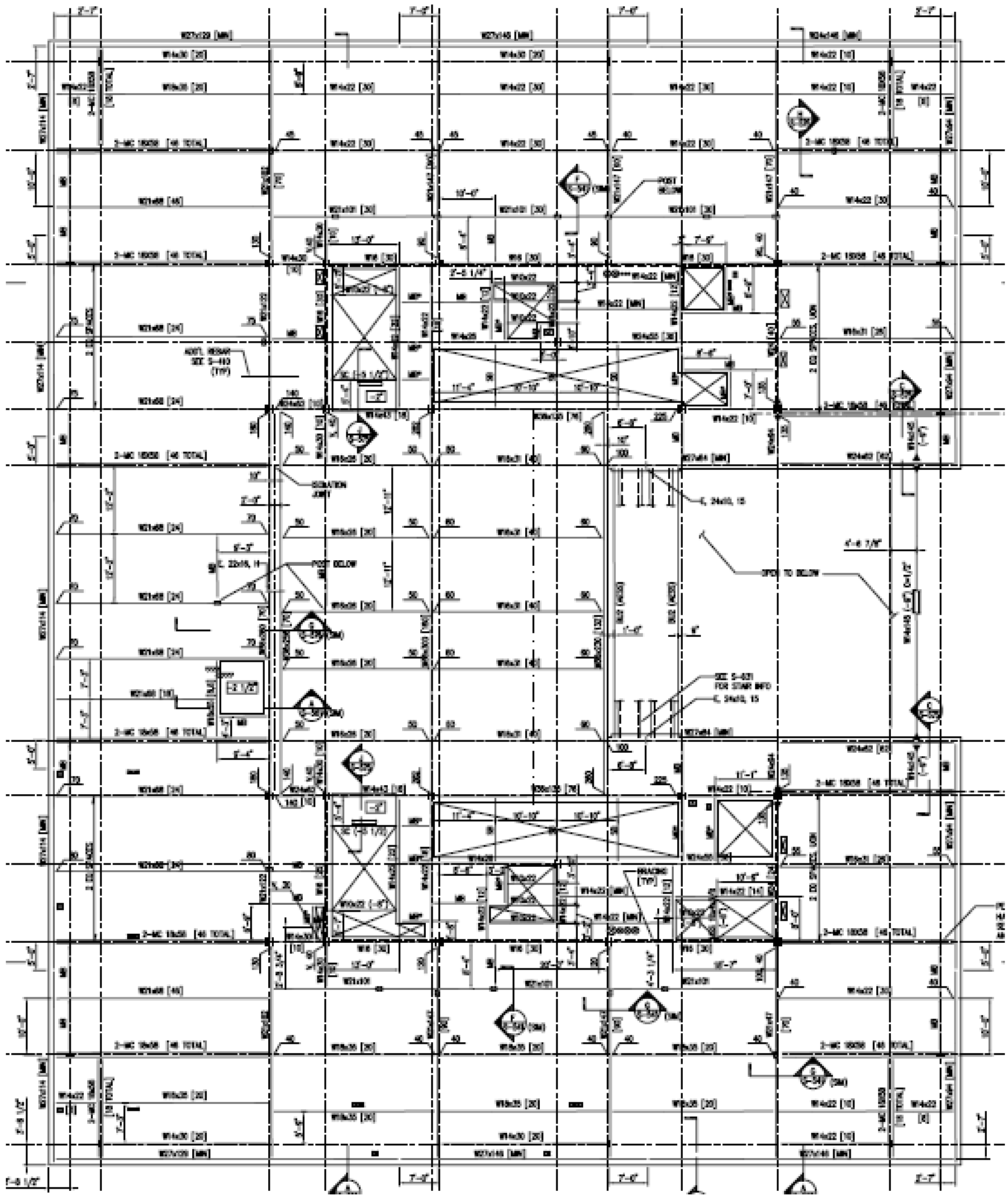
In the third technical report of the John Jay College Expansion project a detailed lateral analysis was performed and lateral members were analyzed for strength requirements. An ETABS model was created to ensure a realistic distribution of story forces to the lateral force-resistant systems was achieved. This was accomplished by modeling the existing braced frames and treating the floor systems as infinitely rigid diaphragms. Lateral forces were distributed by the relative stiffness of each frame with respect to the stiffness of the other frames.

Several combinations of loads were applied to the ETABS model to find the worst case for strength and serviceability requirements. Due to the rectangular geometry of the building, wind forces applied separately in the East-West and North-South directions controlled the overall drift of the building. However, when using ASCE 7-05, strict drift limitations for seismic loading led to seismic drift controlling the design for the North-South direction. This was an unexpected result, as the drift in the North-South was unacceptable for the majority of the levels with the existing braced frames. This discrepancy was clarified when looking into the original design criteria, which used H/260 for seismic drift limitations.

Since the lateral force-resisting systems for the John Jay College Expansion Project are braced frames, the only forces present in the lateral braces are wind forces. Therefore, the braces were controlled for strength requirements by the load combination 1.6W. After analyzing the existing braces, it was determined that they were sized for drift requirements. Two columns were also checked for their controlling load combinations and strength. At the 12th level, a corner column of braced frame 1 and 4 was found to be controlled by 1.2D + 1.6L. This combination is usually unlikely to control for a member with lateral loads present, but the column supports approximately twice the gravity load from the 6th to 14th levels due to the transfer of the perimeter plate hanger loads at the penthouse level. This same column was also analyzed at the 2nd level, but the column had a substantial amount of weak axis bending due to wind forces and was controlled by 1.2D + 1.6W + 1.0L. Both of the columns analyzed in this report were determined to be controlled by drift rather than strength.

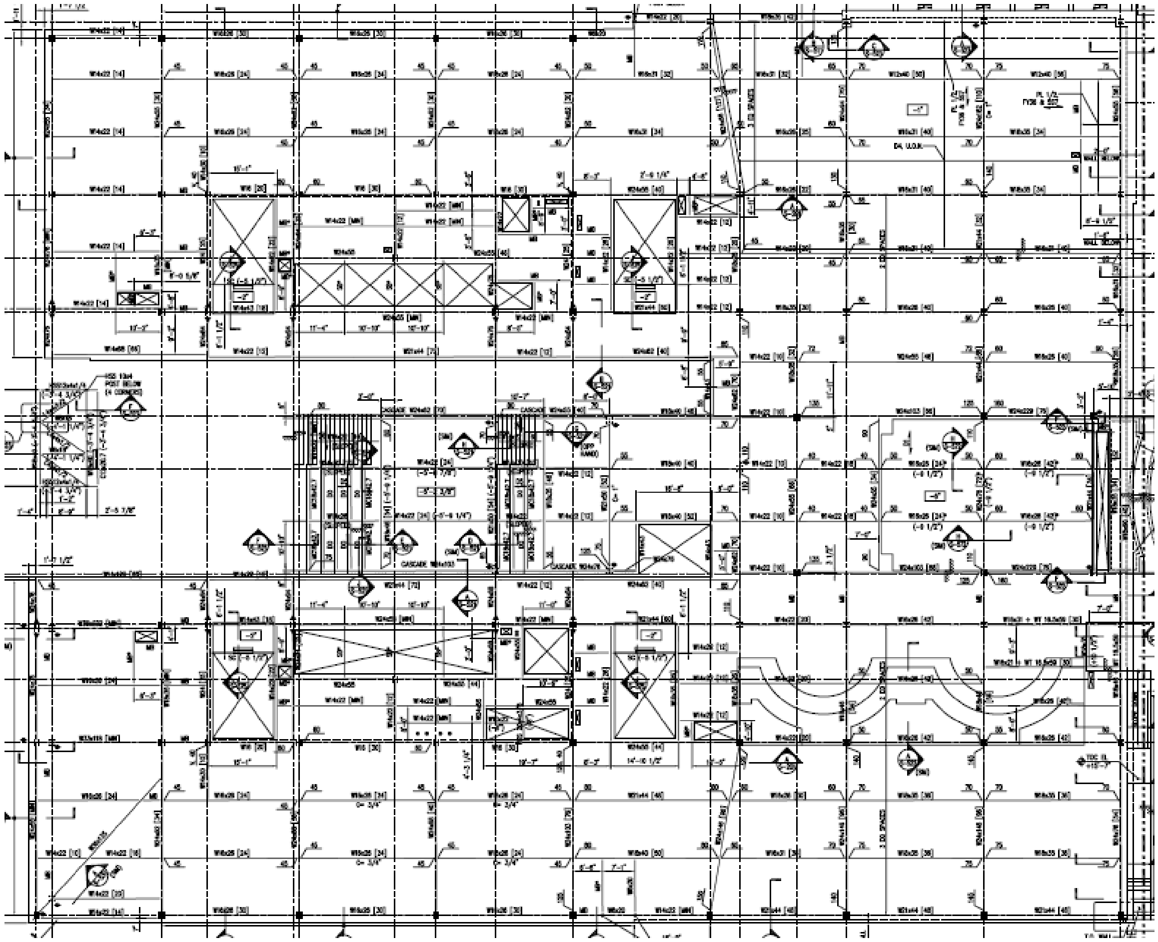
From this report, it can be concluded that the existing braced frames are more than adequate for both strength and drift requirements for the Building Code of the City of New York. When using ASCE 7-05, seismic drift is an issue in the North-South direction. This issue, along with a more advanced torsion analysis will be further investigated throughout the rest of the year.

Appendix A – Typical Framing Plans



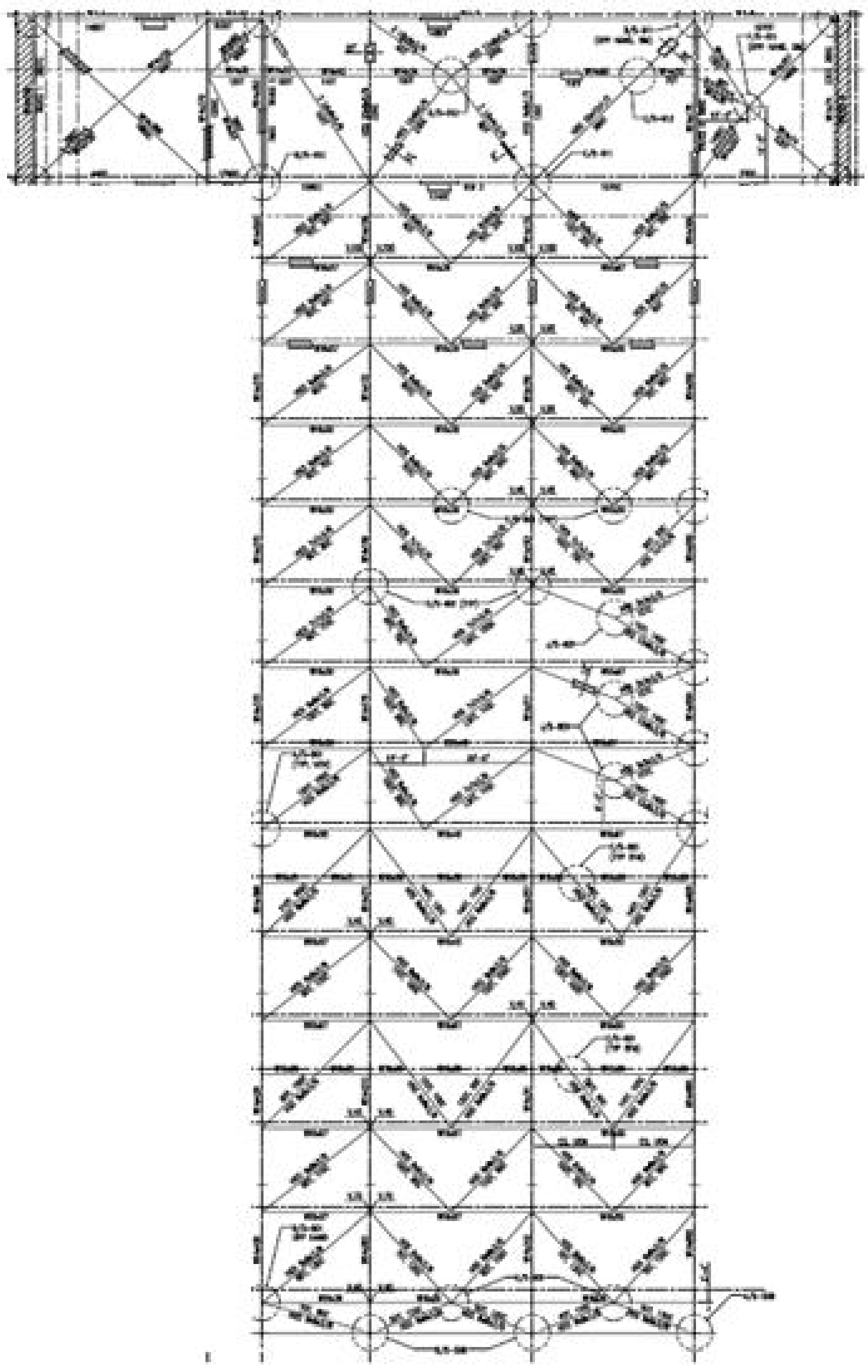
Typical Tower Framing Plan

Technical Assignment #3

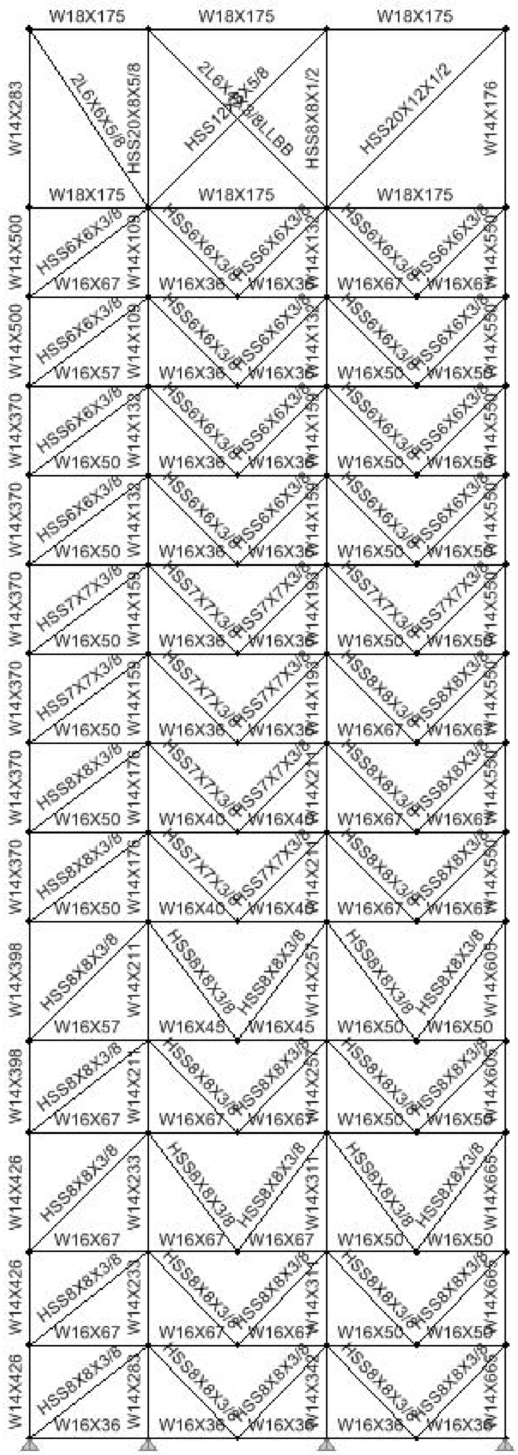


Typical Cascade Area Framing Plan

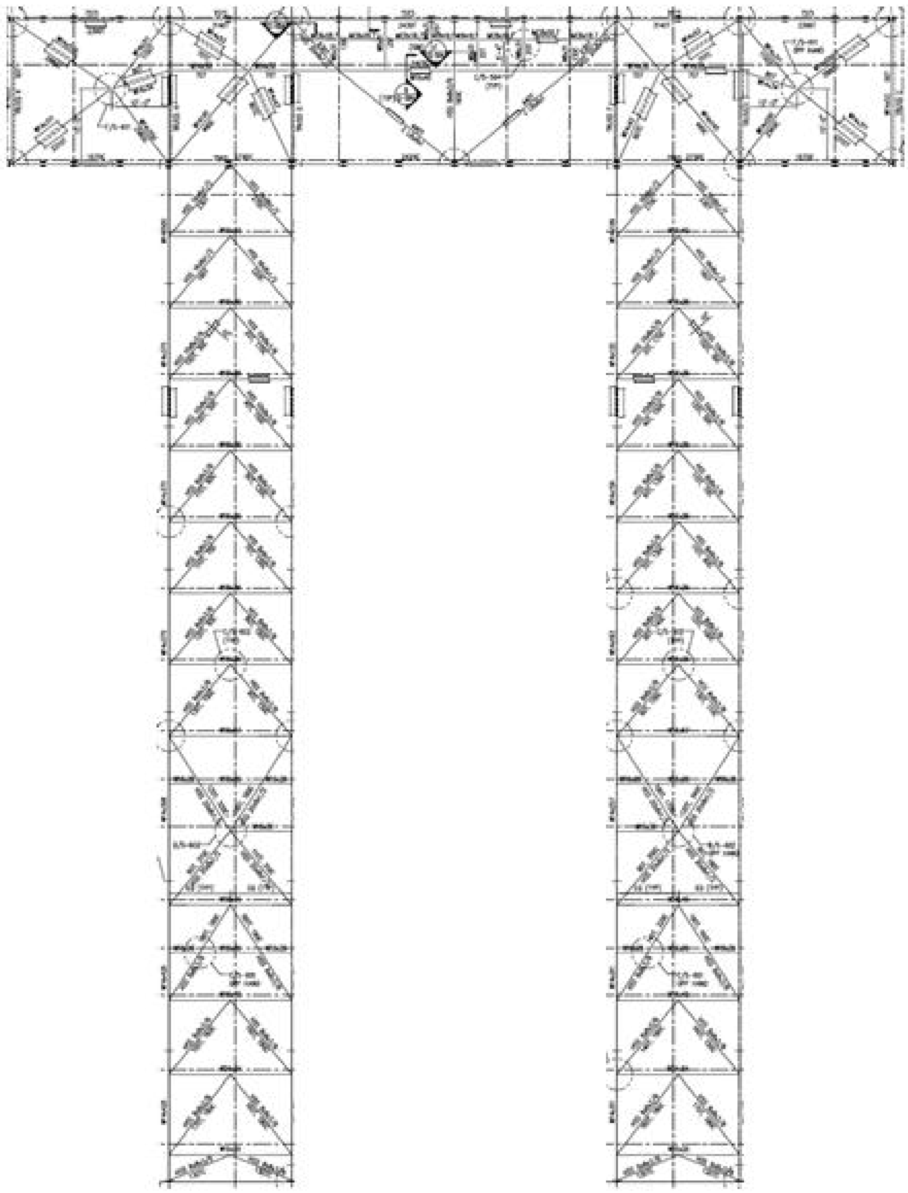
Appendix B – Braced Frames



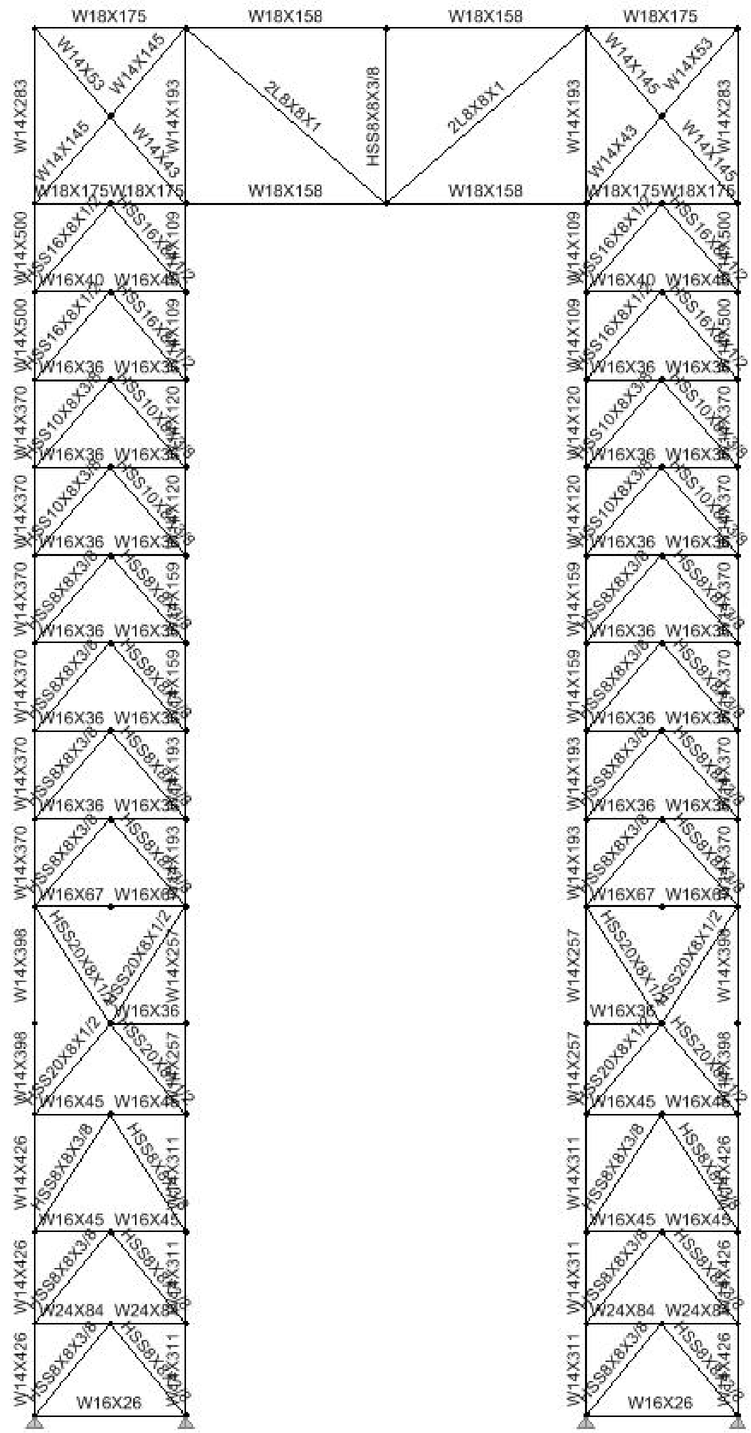
Actual Braced Frames 1 and 2



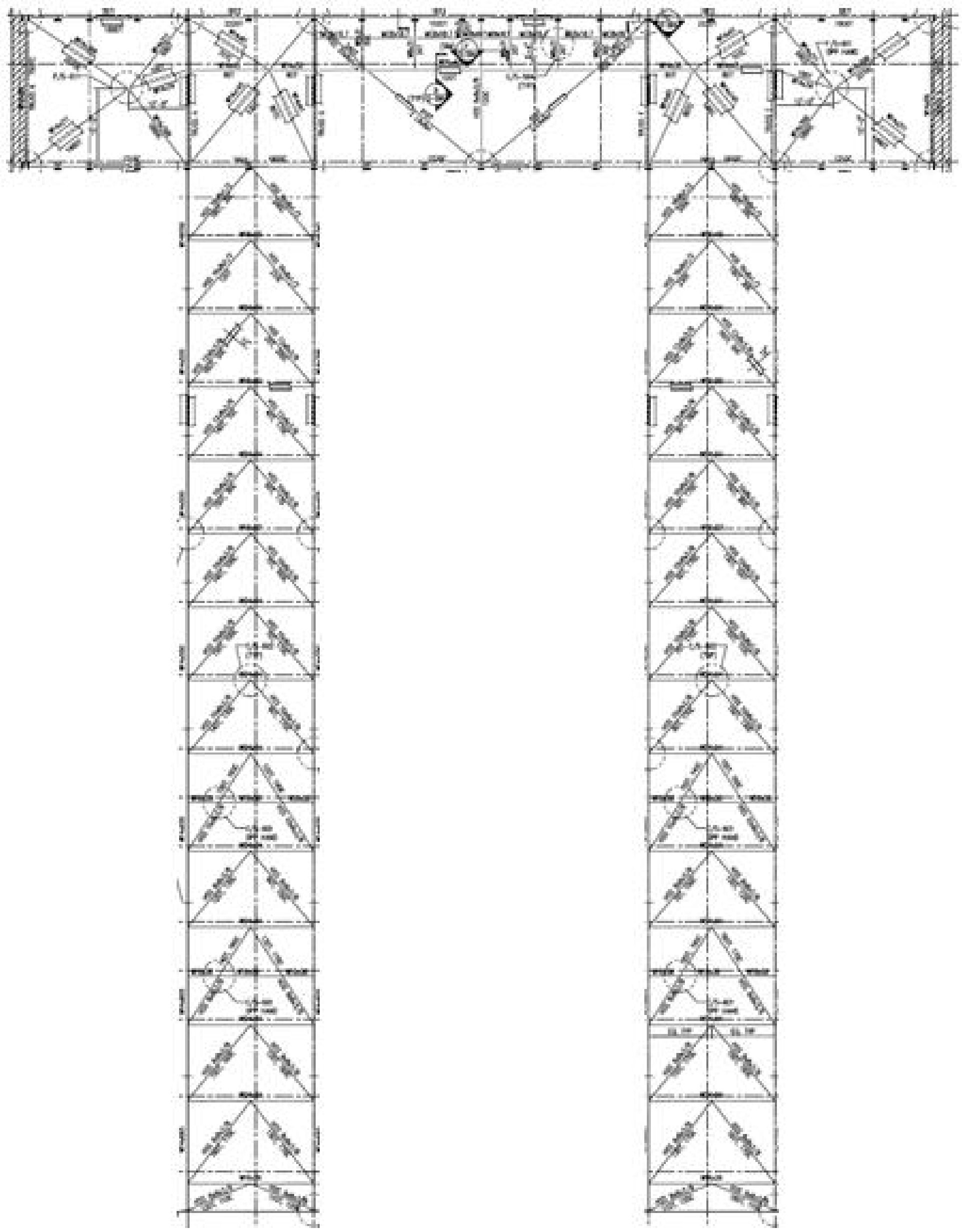
ETABS Model Braced Frames 1 and 2



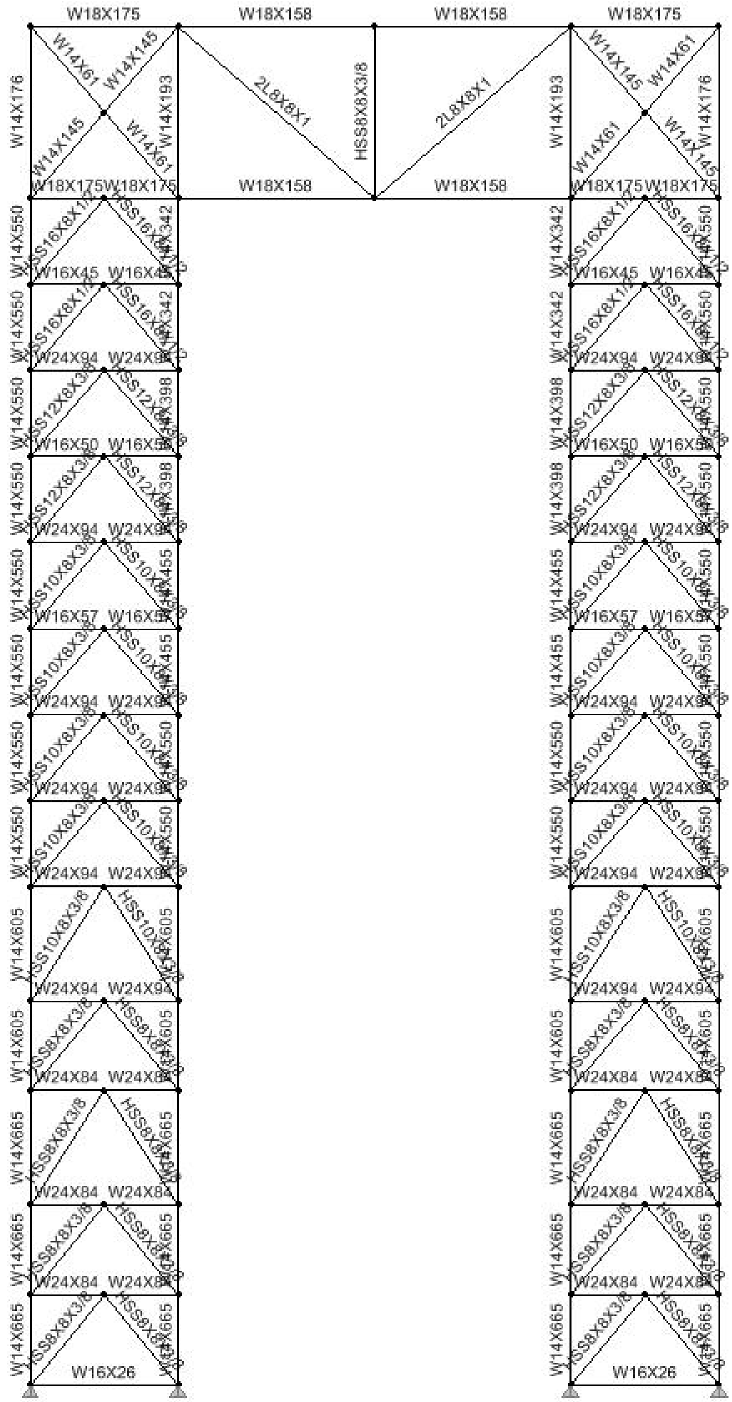
Actual Braced Frame 3 Actual Braced Frame 4



ETABS Model Braced Frame 3 ETABS Model Braced Frame 4

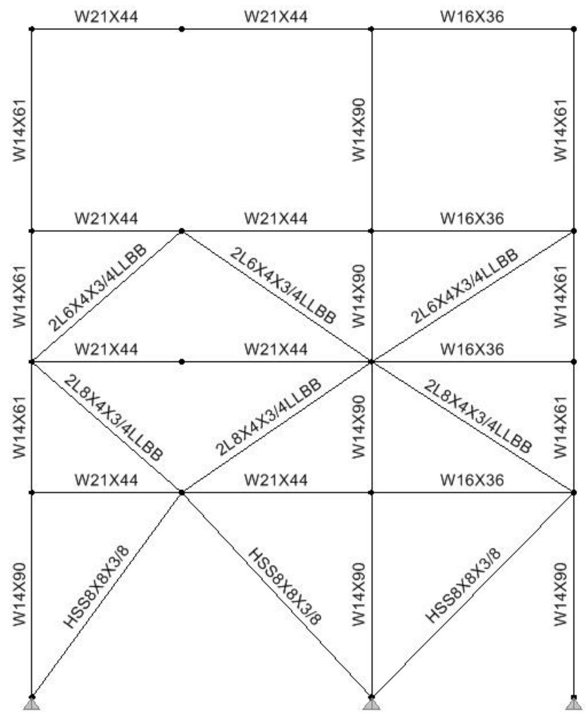


Actual Braced Frame 5 Actual Braced Frame 6

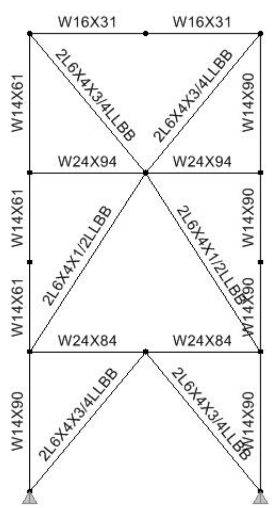


ETABS Model Braced Frame 5

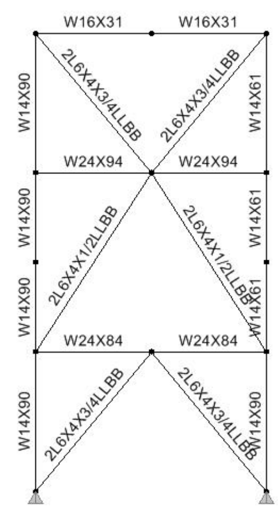
ETABS Model Braced Frame 6



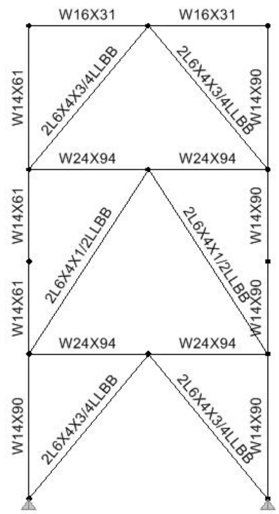
ETABS Model Braced Frame 7 and 8



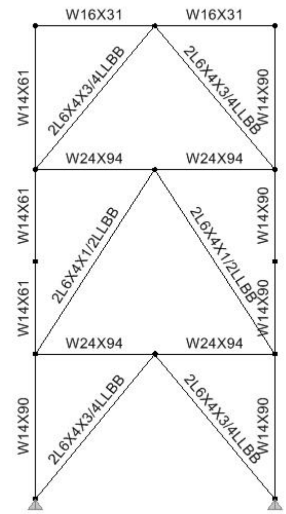
ETABS Model Braced Frame 9



ETABS Model Braced Frame 10



ETABS Model Braced Frame 11 & 13



ETABS Model Braced Frame 12 & 14

Appendix C – Wind Load Calculations

V=	110	mph
K_d=	0.85	
K_d=	1.15	
K_d=	1	
Exposure:	B	

C_p Value	N-S	E-W
Windward wall	0.8	0.8
Leeward Wall	-0.454	-0.5
Side Wall	-0.7	-0.7

Gust Effect Factors		
	N-S	E-W
B	163.33	200.67
L	200.67	163.33
h	239.5	239.5
n_1	0.42	0.42
Structure:	FLEXIBLE	FLEXIBLE
g_R	3.976	3.976
z	143.7	143.7
I_z	0.235	0.235
L_z	520	520
Q	0.807	0.799
V_z	104.88	104.88
N_1	2.070	2.070
R_n	0.087	0.087
R_h	0.202	0.202
n=	4.386	4.386
R_B	0.279	0.235
n=	2.991	3.675
R_L	0.078	0.095
n=	12.303	10.014
R	0.236	0.218
G_f	0.847	0.839

	Level	Height Above ground (ft)	Kz	qz	Wind Pressures N-S (psf)	Wind Pressures E-W (psf)
Windward	T.O. Parapet	239.5	1.26	38.2	25.8	25.6
	Roof	236.67	1.26	38.2	25.8	25.6
	Penthouse	206.67	1.21	36.6	24.8	24.6
	13	191.67	1.18	35.7	24.2	24.0
	12	176.67	1.16	35.1	23.8	23.6
	11	161.67	1.13	34.2	23.2	23.0
	10	146.67	1.1	33.3	22.6	22.4
	9	131.67	1.07	32.4	21.9	21.7
	8	116.67	1.03	31.2	21.1	20.9
	7	101.67	0.99	30.0	20.3	20.1
	6	86.67	0.95	28.8	19.5	19.3
	5	66.67	0.87	26.3	17.8	17.7
	4	51.17	0.81	24.5	16.6	16.5
	3	31.17	0.71	21.5	14.6	14.4
	2	15.58	0.57	17.3	11.7	11.6
Leeward	All	All	1.26	38.2	-14.7	-16.0

Level	Height Above ground (ft)	Building Dimensions (ft) H	B N-S	B E-W	Wind Forces Load (kips) N-S	Load (kips) E-W	Shear (kips) N-S	Shear (kips) E-W	Moment (ft-kips) N-S	Moment (ft-kips) E-W
Roof	236.67	15	165	200	100	125	0	0	23724	29547
Penthouse	206.67	22.5	165	200	147	183	100	125	30288	37757
13	191.67	15	165	200	96	120	247	308	18435	22994
12	176.67	15	165	200	95	119	343	428	16813	20979
11	161.67	15	165	200	94	117	438	546	15139	18902
10	146.67	15	165	200	92	115	532	663	13511	16880
9	131.67	15	165	200	91	113	624	778	11929	14913
8	116.67	15	165	200	89	111	714	892	10333	12929
7	101.67	15	165	200	87	108	803	1002	8798	11019
6	86.67	17.5	165	200	99	124	890	1111	8545	10712
5	66.67	17.75	500	200	288	120	988	1234	19232	7973
4	51.17	17.75	500	200	278	115	1277	1354	14202	5898
3	31.17	17.795	500	200	260	108	1554	1469	8104	3376
2	15.58	15.585	375	200	154	86	1814	1578	2399	1340
Total	236.67				1968	1664	1968	1664	201451	215221

Appendix D – Seismic Load Calculations

S_s=	0.35 %g	
S_1=	0.06 %g	
Occupancy Category=	III	
Site Class=	C (Assumed)	
F_a=	1.2	
F_v=	1.7	
S_{ms}=	0.42	
S_{m1}=	0.102	
S_{DS}=	0.28	
S_{D1}=	0.068	
T_a=	1.218	
$0.8T_s$=	0.194	$< T_a$
SDC=	B	Table 11.6-1
SDC=	B	Table 11.6-2
SDC=	**B**	Can use Equivalent Lateral Force Procedure
T_s=	0.243	
R=	3.25	Ordinary steel concentrically braced frames
I=	1.25	Occupancy Category III
T_a=	1.218	
C_u=	1.7	
T_L=	6 seconds	
$C_{s\ min}$=	0.013 <--- Governs	
$C_{s\ max}$=	0.108	
k=	1.36	
W=	81866 Kips	
V=	1034 Kips	