Technical Report #1

John Jay College Expansion Project

New York, NY

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Structural Option

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Executive Summary

In the first technical report for the John Jay College Expansion Project the building is introduced through a brief explanation of function and a detailed description of the structural system including foundations, floor framing systems, columns, perimeter plate hangers, and lateral systems. A list of materials and governing building codes are also provided. Gravity loads are calculated using ASCE 7-05 and are very close to the Building Code of the City of New York's required gravity loads. Wind and Seismic loads are also calculated using ASCE 7-05 and are compared to the structural engineer's lateral loads. It was determined that the seismic loads were fairly close, while the wind loads calculated in this report are larger than wind loads determined by the NYC Building Code. A typical bay was analyzed and designed for gravity loading. This resulted in verification of the member sizes listed in the structural drawings.

Introduction

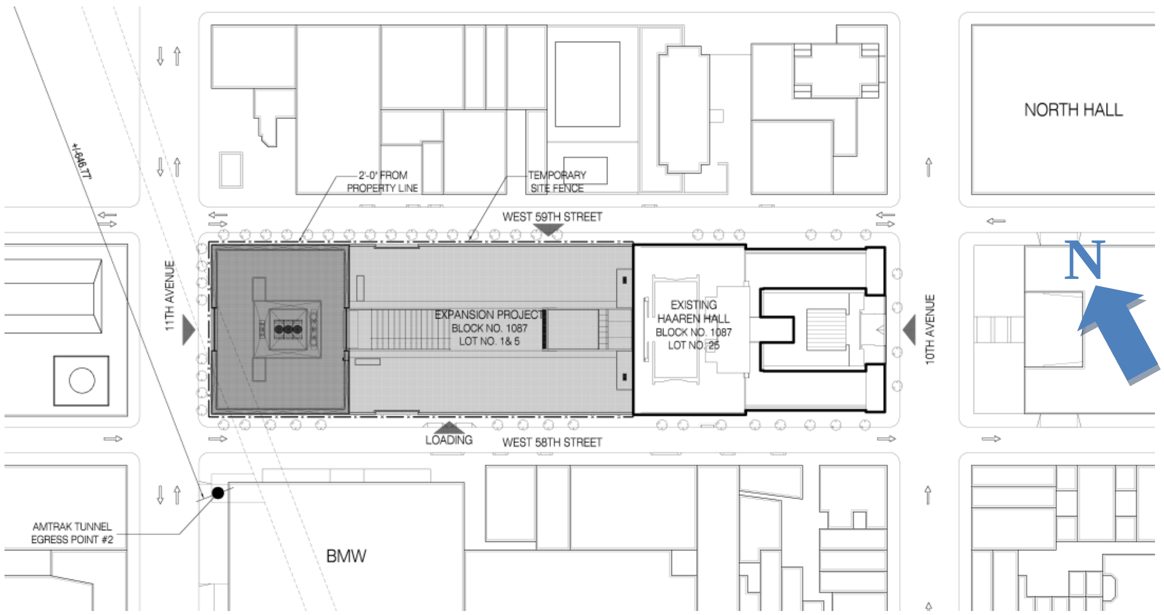


Figure 1 – Site plan

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. 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.

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. These trusses then transfer the loads to a braced frame core.

See Figure 1 and Figure 2 for elevation views of the John Jay College Expansion Project.

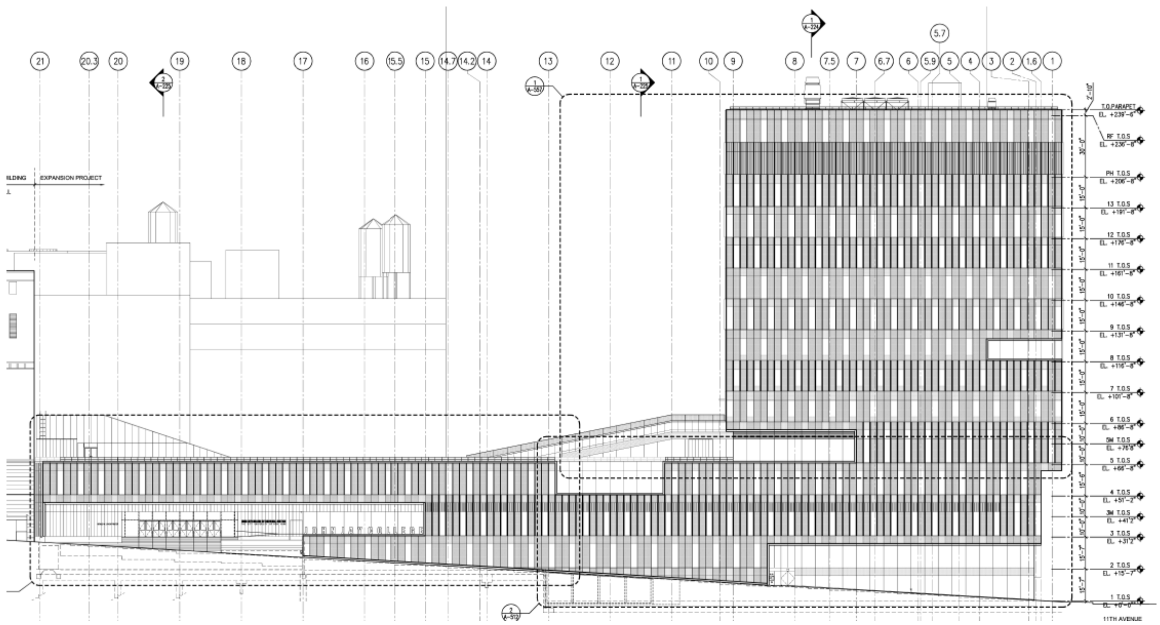


Figure 2 – North Elevation

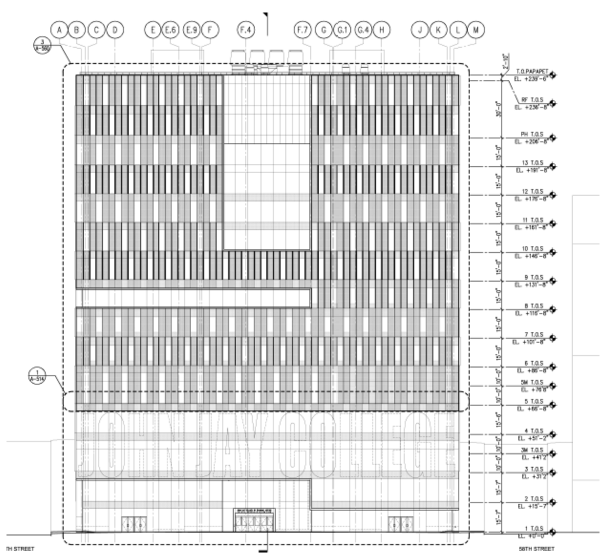


Figure 3 – West Elevation

Structural Systems

Foundation:

The site of the John Jay College Expansion Project is sloping from the east down to the west, and therefore the foundation system is split into two levels. This caused the designers to use various types of foundation systems to support the structure. The northern half and south-eastern corner of the building is primarily supported on drilled caissons ranging from 18" to 36" in diameter. These caissons are embedded up to 14'-0" into the bedrock below. On the south-western corner of the site, columns are supported by reinforced concrete piers of dimensions ranging from 20"x20" to 72"x42". These concrete piers are then supported by individual column footings ranging in sizes of 3'-0"x3'-0" to 9'-0"x9'-0" that are bearing on bedrock. See Figure 4 and 5 for locations of concrete piers and caissons.

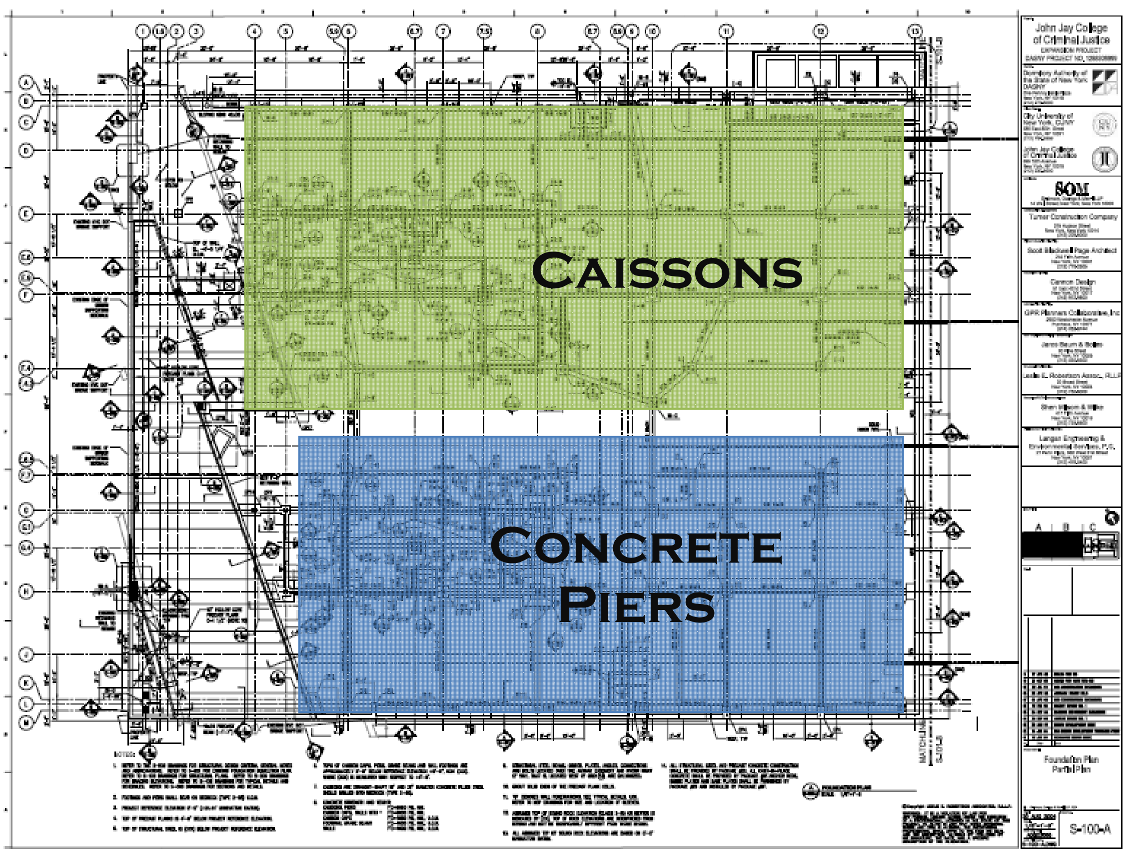


Figure 4 – West Expansion Foundation Plan

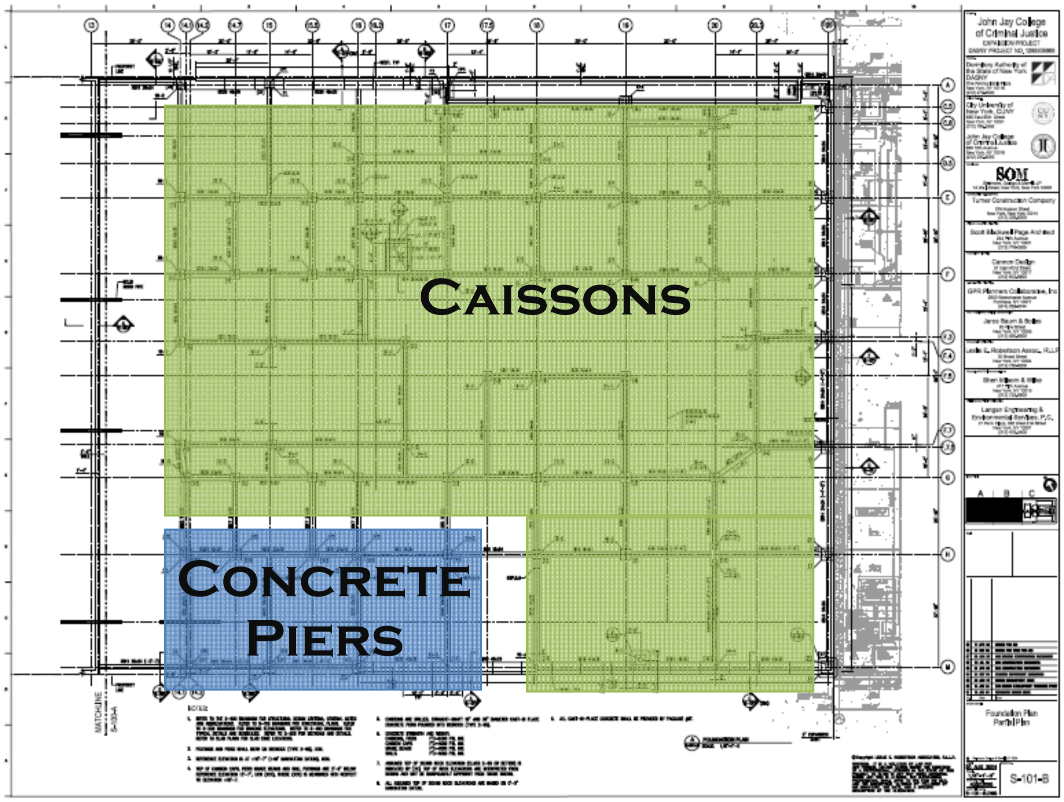


Figure 5 – East Expansion Foundation Plan

The first floor framing system is constructed using a two-way reinforced concrete slab-on-ground that is 6” thick. The slab is spanning to grade beams, which then frame into the concrete pier caps or concrete caissons. The perimeter of the building is enclosed with a reinforced concrete wall that varies in thickness from 12” to 20”.

The John Jay College Expansion Project also has a major site restriction: Amtrak tracks cross the south-western corner and west side of the site (see Figure 6). Loads from the 1st through 5th levels are transferred over the tracks by built-up box girders of up to 3’-2” deep with 4” thick flanges. The tracks are enclosed with 10” thick hollow core pre-cast planks to minimize the amount of time the tracks are delayed for construction.

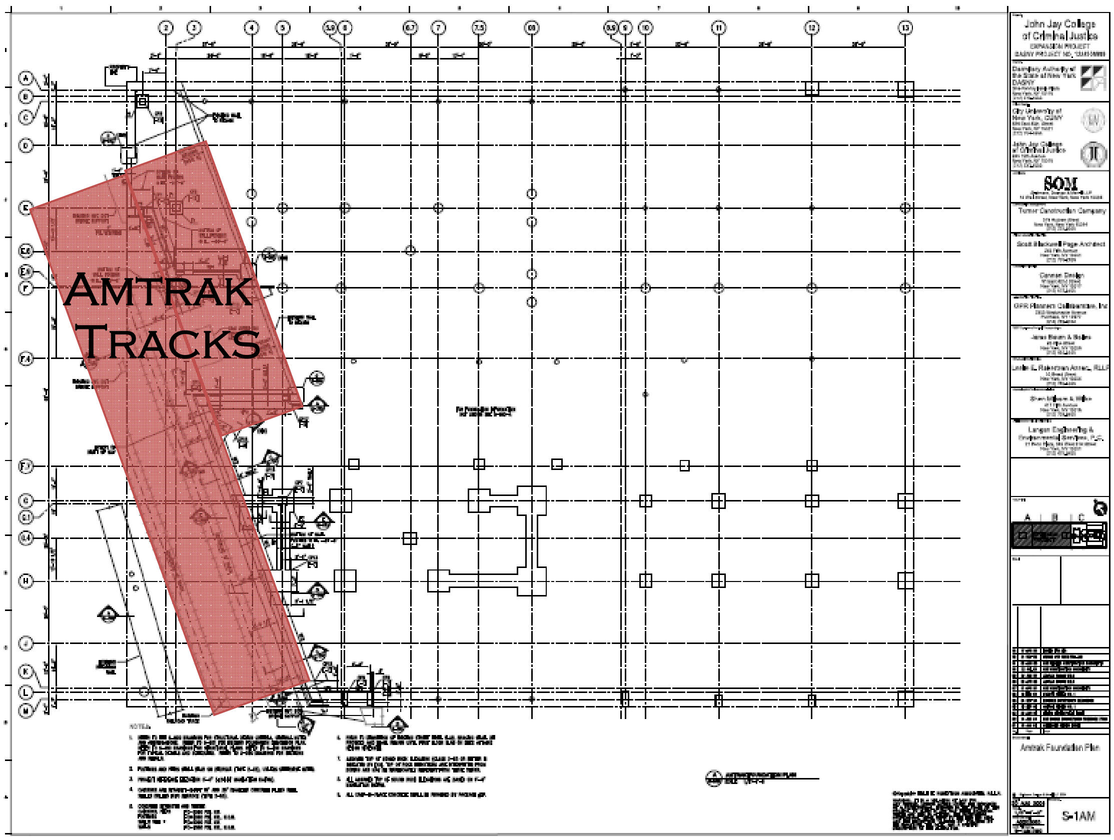


Figure 6 – Amtrak Foundation Plan

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"x37'-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.

Columns:

Typical gravity columns for the John Jay College Expansion Project are W14's. Lateral columns have a significantly heavier W14 section than the gravity columns due to the perimeter tensile loads transferring to the braced core at the penthouse level and to resist lateral loads. Perimeter plate hangers supporting the 6th through 14th floors range in size from 1"x12" to 2"x20". Splices of the plate hangers occur at every two levels using 1 1/8" diameter A490 bolts.

Lateral system:

The 14 story tower of the expansion project has a large centralized braced frame core (see Figure 7 and 8). 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 the caissons and concrete piers at the foundation of the lateral system.

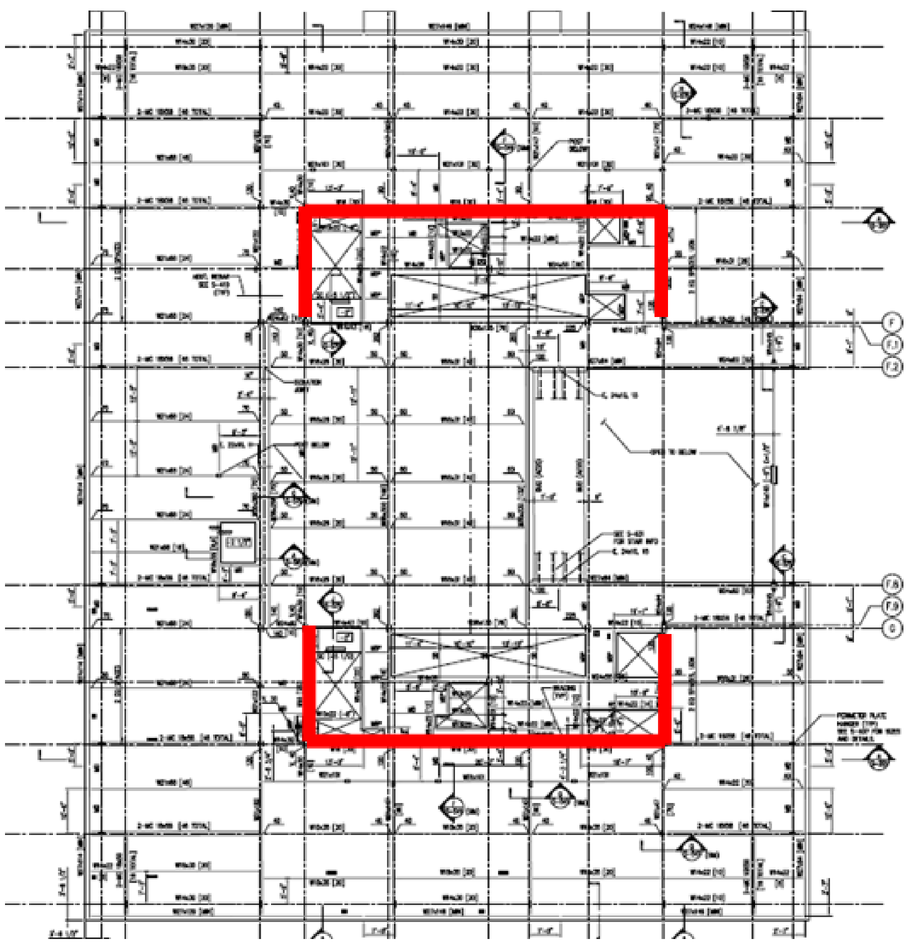


Figure 7 – Location of Lateral Force Resistance Systems (Braced Frames) in tower

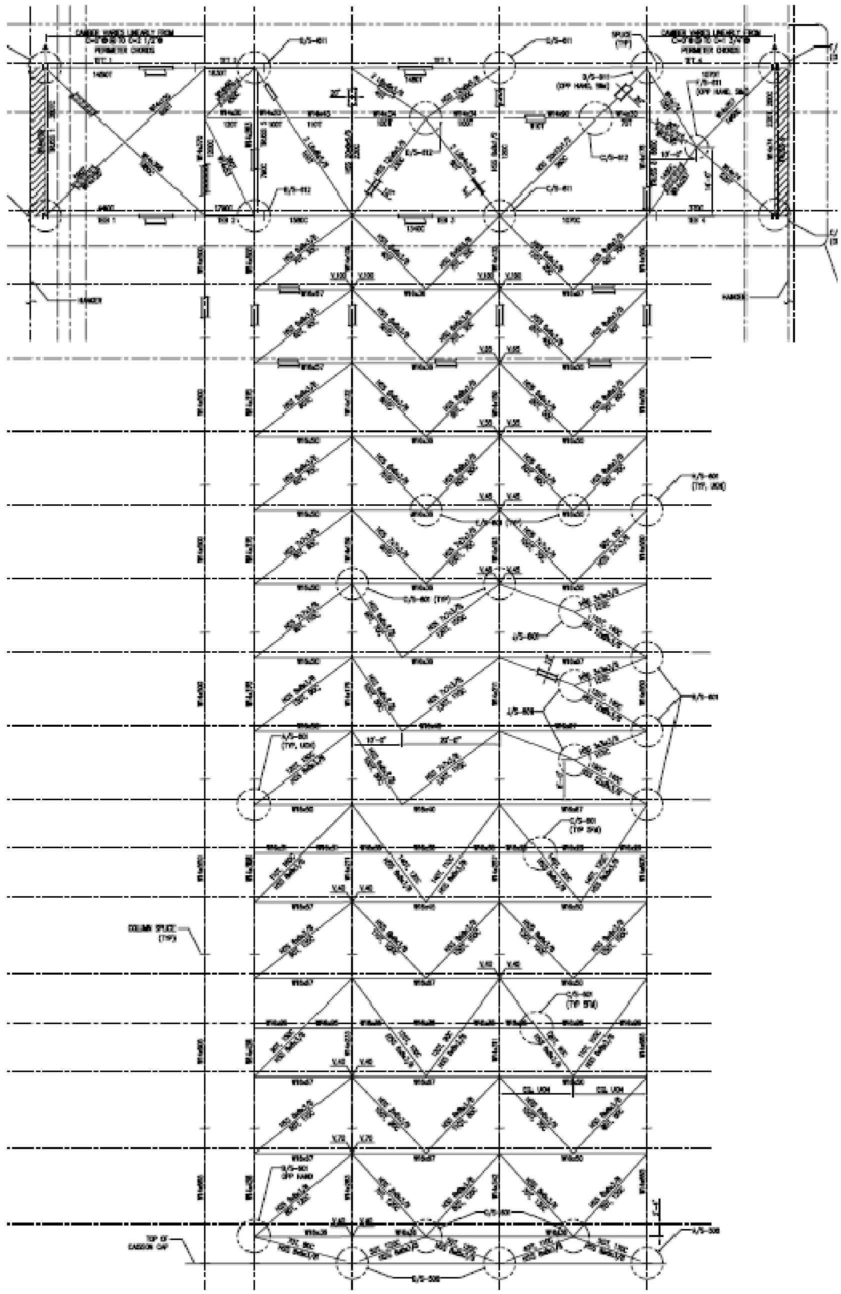


Figure 8 – South Elevation of the Braced Core in the 14 story tower

The lateral system for the 5 story cascade is also a braced frame which encases the buildings vertical circulation (see Figure 9). 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.

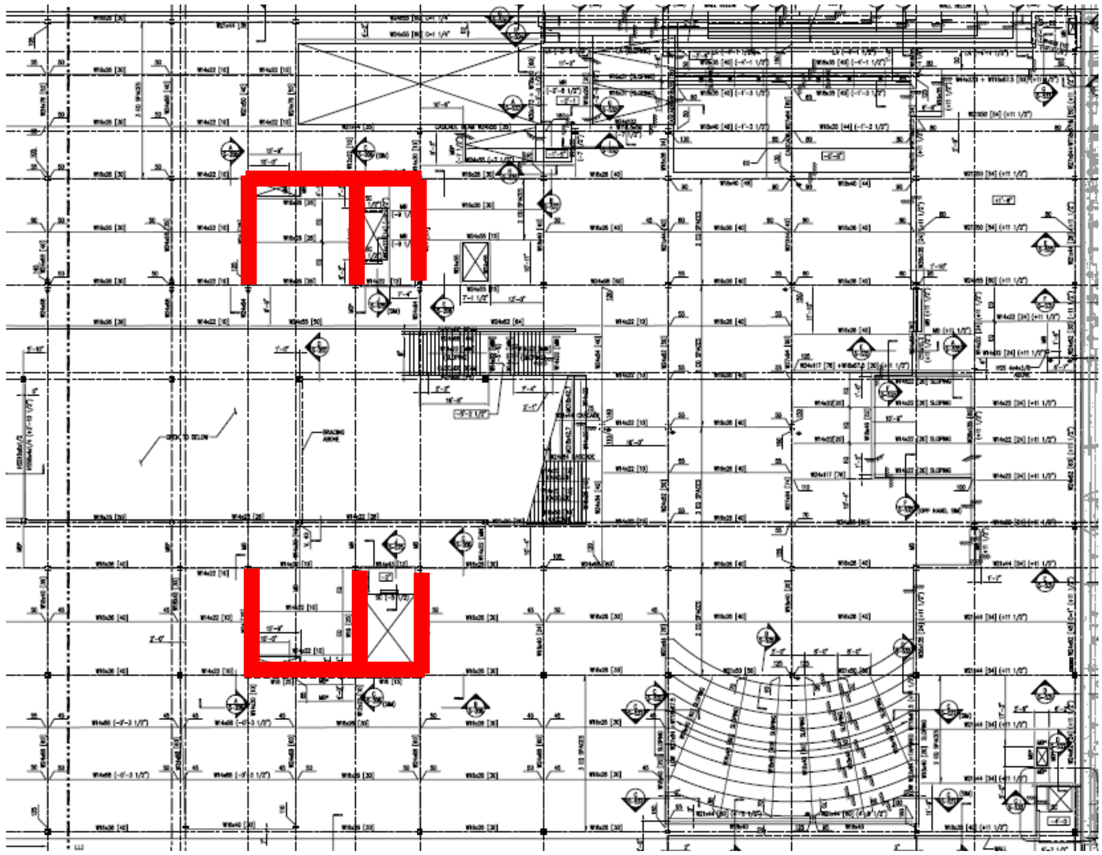


Figure 9 – Location of Lateral Force Resistance Systems (Braced Frames) in the 5 story cascade

Braced frames were chosen to resist the lateral forces because they are more efficient than moment frames (relatively stiff system without the high cost of moment connections). The centralized core, which is very typical in high-rise construction, allows for the diagonal braces to be enclosed by partitions. Reinforced concrete shearwalls could be used around the core of the building in place of braced frames, but in New York City steel workers will not work with any crews above them, which would lead to complicated scheduling for construction.

Codes and References

Design Codes:

National Model Code:

The Building Code of the City of New York with latest supplements

Structural Standards:

ASCE 7-02, Minimum Design Loads for Buildings and other Structures (used for cladding wind loads)

Design Codes:

AISC –LRFD 1999, Load and Resistance Factor Design Specification for Structural Steel Buildings

AISC-ASD 1989, Specifications for Structural Steel Buildings – Allowable Stress Design and Plastic Design (used for the design of Braced Frames and Penthouse level Transfer Trusses)

ACI 318-95, Building Code Requirements for Structural Concrete

Deflection Criteria:

Gravity Deflections:

Live load deflections of beams < 60' are limited to L/500 or ¾", whichever is smaller

Live load deflections of beams ≥ 60' are limited to L/500 or 1-3/8", whichever is smaller

Live load deflections of beams supporting elevator sheave beams are limited to L/1666

Total load deflections of beams and lintels supporting masonry is limited to L/600 or 0.3", whichever is smaller

Lateral Deflections:

Total building sway deflection for wind loading is limited to H/500

Total building sway deflection for seismic loading is limited to H/260

Interstory shear deformation for wind loading is limited to (story H)/400

Interstory shear deformation for seismic loading is limited to (story H)/260

Thesis Codes:

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

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

Materials

Structural Steel:

Wide Flanges and Tee Shapes……………………………….ASTM A572 or A992, Grade 50
Channels and Built-Up Sections……………………………………...ASTM A572, Grade 50
Pipes…………………………………………………ASTM A501 or A53, Types E or S, Grade B
Tubes………………………………………………………………………………….ASTM A500 Grade B
Angles…………………………………………………………………………………………………..ASTM A36
Connection Plates…………………………………………………………………………….ASTM A36

Metal Decking:

3" and 2" Composite Deck……………………………………..Fy = 40 ksi, 20 Gage Minimum

Headed Shear Studs:

¾" diameter…………………………………………………………………….....ASTM A108, Type B

Welding Electrodes:

E70XX……………………………………………………………………………tensile strength of 70 ksi

High Strength Bolts:

¾" and 7/8" Bolts…………………………………………………………………………….ASTM A325
1" and 1 1/8" Bolts……………………………………………………………………….ASTM A490

Cast-in-Place Concrete:

Caisson Caps and Grade Beams…………………………………………………….f'c = 4000 psi
Caissons and Piers………………………………………………………………………f'c = 6000 psi
Slabs on Ground and Footings…………………………………………………….f'c = 4000 psi
Walls……………………………………………………………………………………………..f'c = 4000 psi
Slabs on Deck……..f'c = 4000 psi – light weight concrete unless noted on drawings

Reinforcement:

Reinforcing Bars…………………………………………………………………..ASTM A615, Grade 60
Caisson #18 Reinforcing Bars……………………………………………ASTM A615, Grade 75

Welded Wire Fabric:

D4.0 and larger……………………………………………………..ASTM A497, Fy = 70 ksi

W4.0 and smaller………..ASTM A185 (Fy = 65 ksi $\geq$ W1.2, Fy=56 ksi < W1.2)

Deformed Bar Anchors……………………………………………………..ASTM A496, Fy = 70 ksi

Gravity and Lateral Loads

ASCE 7-05 was used for 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:

Typical Spaces:

	ASCE 7 -05	Design (NYC Building Code)
Classrooms	40 psf	60 psf
Offices	50 psf	50 psf
Lobbies & Corridors	100 psf	100 psf
Cascade	100 psf (assume corridor/lobby/bleachers)	100 psf
Stairs	100 psf	100 psf
Assembly areas (moot court and quad spaces)	60 psf (fixed seats) 100 psf (movable seats)	100 psf
Roof	20 psf	30 psf

Heavy Mechanical Equipment:

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

Wall Loads:

Curtain Wall	25 psf
1'-6" Thick Reinf. Conc. Wall (@ Foundation)	225 psf

Snow Loads:

Ground Snow Load	20 psf
Flat Roof Snow Load	22 psf
Rain-on-Snow Surcharge	5 psf
Tower Roof Drift	25 psf
Commons Drift	70.6 psf

(For calculation of snow loads, see Appendix E)

Lateral Loads:

Wind Analysis:

The wind loads for the John Jay College Expansion Project were analyzed using Method 2 listed in Chapter 6 of ASCE 7-05. Details of the analysis can be found in Appendix C of this report. Loads were calculated using the height and widths of the 14 story tower, and then the lower level pressures were applied to the 5 story cascade section of the project. It was determined that the pressures in the North-South direction were slightly larger than in the East-West direction. The base shear also controlled in the North-South direction due to the large façade area of the cascade connecting the tower to Haaren Hall.

Below in Table 1 are tabulated values of wind pressures at each floor level. Table 2 displays lateral loads, shears, and moments created at each level by the wind pressures.

	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

Table 1 – Wind Pressures

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

Table 2 – Wind loads, shears, and moments at each level

As you can see in table 2 above, the base shear of 1968 kips in the north-south direction controls. This is expected due to the large façade area in the north-south direction.

Wind Pressure Diagrams:

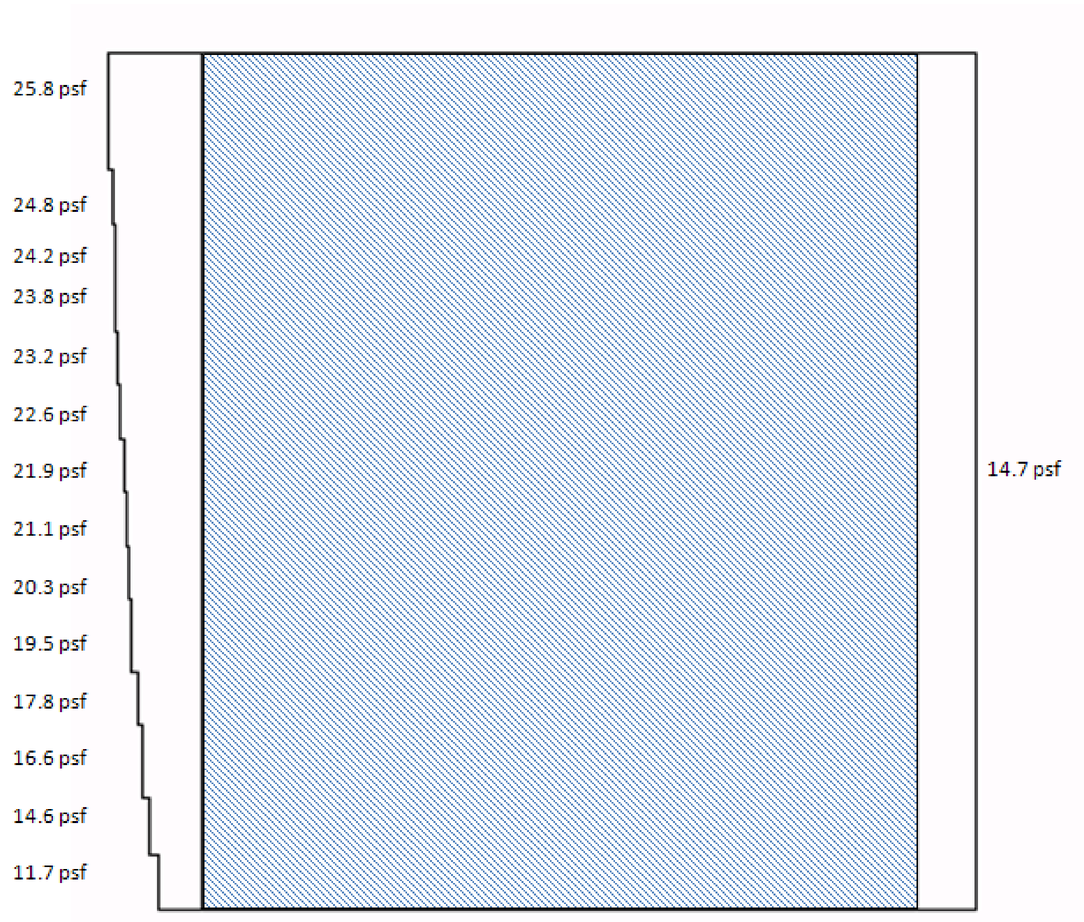


Figure 10 – North-South wind pressure diagram

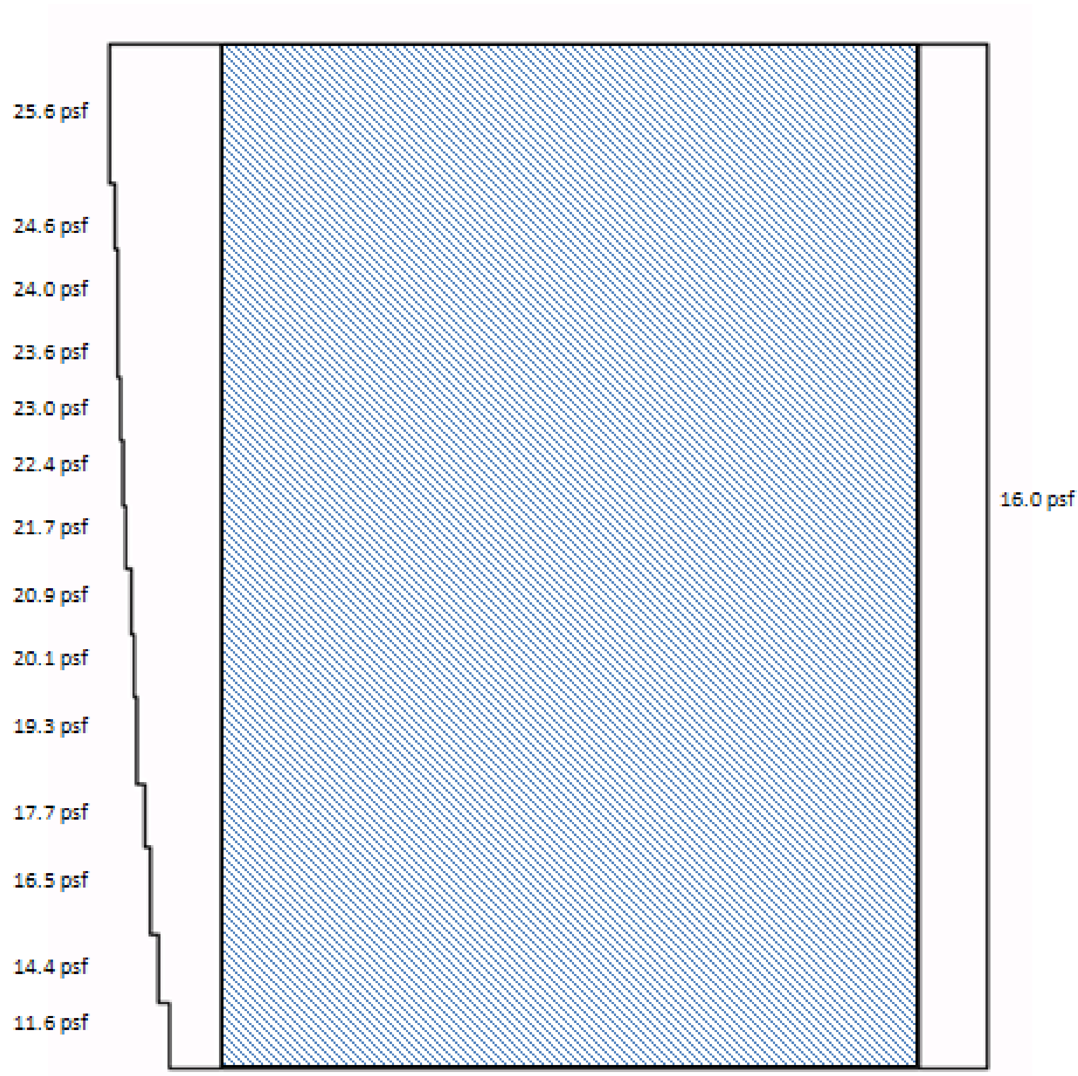


Figure 11 – East-West wind pressure diagram

Wind Load Diagrams:

Wind pressures were applied to the building's façade and distributed to floor levels by tributary area. Floor diaphragms are assumed to be rigid and transfer wind loads to the braced frames at the building's core. See figures 12 and 13 for the distribution of wind loads to the lateral systems.

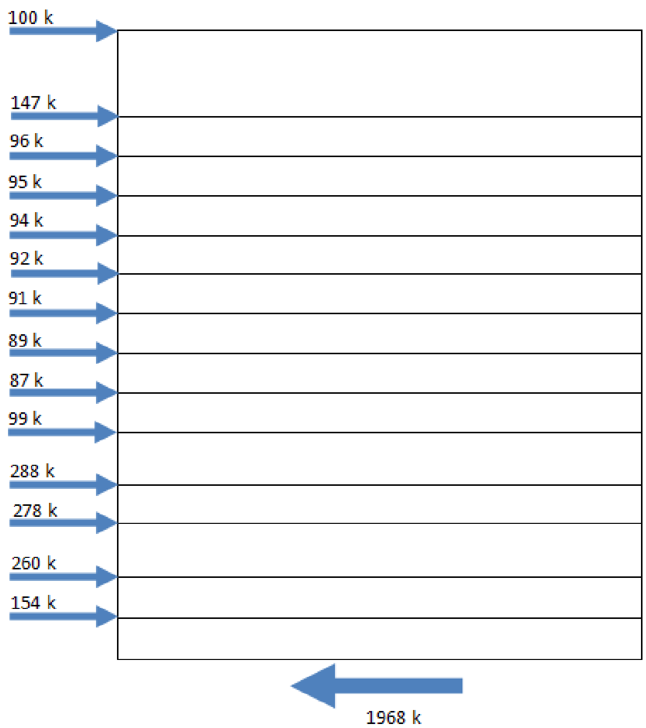


Figure 12 – North-South Wind Force diagram

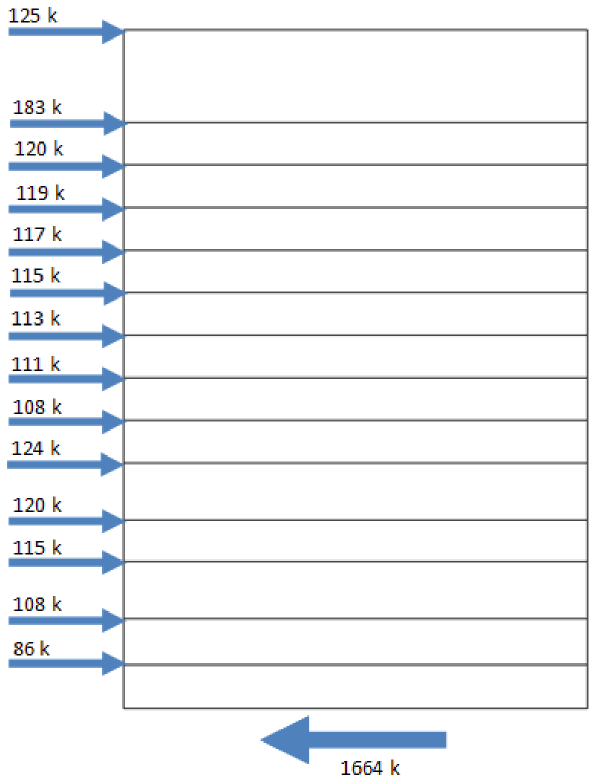


Figure 13 – East-West Wind Force Diagram

Response to Calculated Wind Loads:

The calculated windward pressures using ASCE 7-05 are comparable to those used by the structural engineer of record. The Building Code of the City of New York requires a wind pressure of 20 psf for 0-100 feet above the ground and 25 psf for 101-300 feet above the ground. Calculated pressures at level 7, which is 101'-8" above the ground, is 20.1 psf in the east-west direction using ASCE 7-05 and the pressure at the roof level is 25.6 psf in the east-west direction using ASCE 7-05. These values are very close to the pressures in the New York City Building Code, however, when leeward suction pressures are added, method 2 of ASCE 7-05 becomes a more conservative approach for calculating wind loads. See Table 3 for a comparison between windward pressures and Table 4 for a comparison between total pressures.

Base shear values from the structural engineer verify the prediction that ASCE 7-05 is more conservative than the New York City Building Code. A base shear of 1106 kips in the east-west direction and 1329 kips in the north-south direction were calculated by the designers. These values are significantly smaller than the base shears calculated using ASCE 7-05 for this report, which are presented in Table 2.

Height	ASCE 7-05 Method 2		New York City Building Code
	N-S (psf)	E-W (psf)	(psf)
101'-8"	20.3	20.1	20
236'-8"	25.8	25.6	25

Table 3 – Comparison between **windward pressures** using ASCE 7-05 and the Building Code of the City of New York

Height	ASCE 7-05 Method 2		New York City Building Code
	N-S (psf)	E-W (psf)	(psf)
101'-8"	40.5	41.6	20
236'-8"	35.0	36.1	25

Table 4 – Comparison between **total pressures** using ASCE 7-05 and the Building Code of the City of New York

Seismic Loads:

The seismic loads for this report were calculated using chapters 11 and 12 in ASCE 7-05. It was determined that the equivalent lateral force procedure could be used for the John Jay College Expansion Project. This seismic analysis includes dead loads from typical floor construction, heavy mechanical equipment, the commons roof (green roof on cascade), and wall weights. These dead load calculations are available in Appendix B. See Appendix D for all seismic calculations and assumptions used.

Below in Table 5 is a load distribution table. Lateral forces are assumed to transfer through rigid floor diaphragms to the steel braced frames. Seismic forces for the John Jay College Expansion project are less than the forces generated by wind pressures.

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	110	0	25962
Penthouse	6502	206.67	9101073	0.221	181	110	37318
13	2874	191.67	3631231	0.088	72	290	13809
12	2822	176.67	3191293	0.077	63	362	11186
11	3040	161.67	3047887	0.074	60	426	9776
10	2638	146.67	2317053	0.056	46	486	6742
9	3040	131.67	2306064	0.056	46	532	6024
8	2870	116.67	1847361	0.045	37	578	4276
7	2929	101.67	1563720	0.038	31	614	3154
6	3785	86.67	1626559	0.039	32	645	2797
5	12565	66.67	3780295	0.092	75	678	5000
4	8483	51.17	1781485	0.043	35	753	1809
3	10119	31.17	1083535	0.026	21	788	670
2	10932	15.58	456219	0.011	9	810	141
Total	81866	236.67	41262883	1.000	819	819	128665

Table 5 – Lateral forces, story shears, and overturning moments

Response to Calculated Seismic Loads:

The total base shear from the structural engineer's seismic analysis is 738 kips. This value is comparable to the base shear of 819 kips as calculated above in Table 5. After reviewing the structural engineer's seismic calculations, the following discrepancies account for the difference is base shears:

- The structural engineer used the New York City Building Code for their seismic load calculations, while ASCE 7-05 was used for this report. This resulted in a base shear coefficient of 0.014 from the NYC Building Code, and a base shear coefficient of 0.010 from ASCE 7-05.

- The structural engineer was more accurate when calculating the area of each floor. Total floor areas of the building provided by the structural engineer are approximately 682,000 square feet which resulted in a total weight of 53,832 kips. Floor area calculations for this report resulted in a total of approximately 711,000 square feet. This led to a total weight of 81,866 kips.
- This report included the weight of heavy permanent equipment at the penthouse level, heavy dead loads for the commons roof (green roof), and heavy loadings in laboratory spaces for the seismic weight. These additional loadings were not used by the designers.

It is very typical for lateral systems of tall buildings in New York City to be controlled by wind forces, therefore little efforts are put into seismic calculations by practicing engineers. Even with the conservative approach used in this report, the wind loads still control the lateral forces for the John Jay College Expansion Project.

Typical Floor Framing Spot-Checks

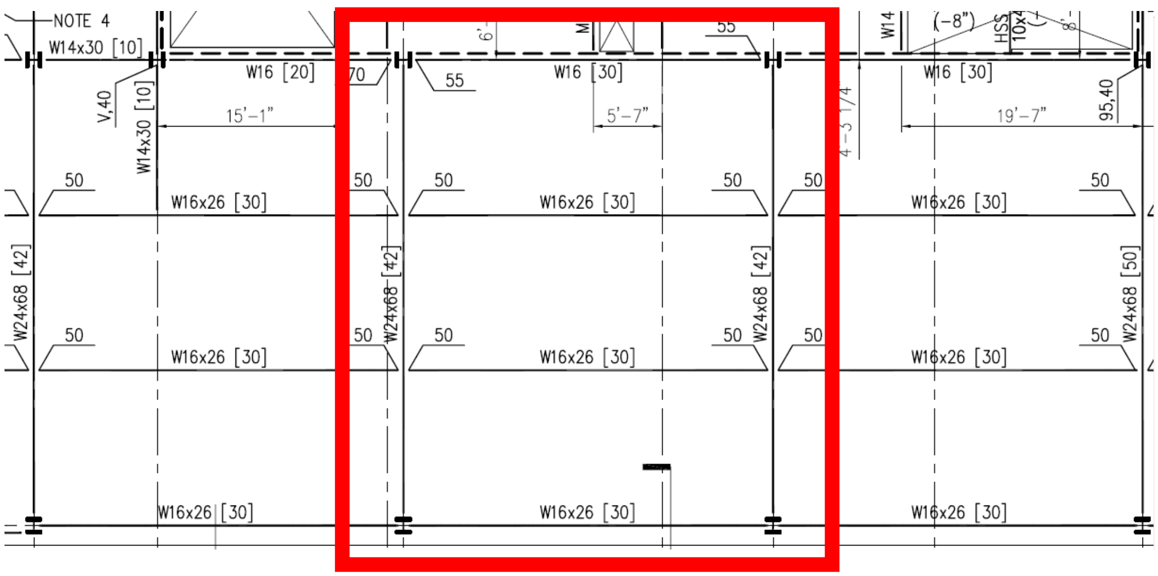


Figure 14 – Typical 30'-0" by 37'-10" bay

Above in Figure 14 is a typical bay that was analyzed. Typical infill beam sizes for the John Jay College Expansion Project are W16x26, which frame into typical girder sizes of W24x68. Girders and spandrel beams are supported by either columns or perimeter plate hangers.

Metal Decking:

It was determined from the structural design criteria, general notes for each floor, and in the specifications that the metal decking chose by the structural engineer is 3" deep, with a 40 ksi minimum yield strength, and a minimum thickness of 20 gage. The following table was taken from United Steel Deck for a 3" deep deck with a yield strength of 40 ksi:

slab depth	wc psf	Sc in^3	øVt lbs.	Ac in^2	Iav in^4	Max Unshored Spans, ft. 1 span	2 spans	3 spans	WWF
5.50	38	1.50	5249	37.6	8.1	10.25	12.79	13.22	0.023
6.00	43	1.73	5866	42.0	10.4	9.78	12.28	12.68	0.027
6.25	46	1.84	6183	44.3	11.6	9.56	12.04	12.44	0.029
6.50	48	1.96	6506	46.6	13.0	9.36	11.82	12.21	0.032
7.00	53	2.21	7125	51.3	16.1	9.00	11.41	11.78	0.036
7.25	55	2.33	7295	53.8	17.7	8.84	11.21	11.59	0.038
7.50	58	2.46	7468	56.3	19.6	8.68	11.03	11.40	0.041
8.00	62	2.71	7823	61.3	23.5	8.44	10.69	11.05	0.045

Maximum un-shored spans for this metal decking with a 6 ½" thick slab are highlighted in red above. Typical clear spans between the W16x26 infill beams are 12.15 feet and the decking clears 3 spans. Therefore, the decking is adequate to span between infill beams.

The following table was also taken from United Steel deck and displays the maximum service live load per square foot of metal decking:

			Superimposed Live Load, psf												
			Spans, ft.												
Stud Spacing	Slab Depth	ϕMn in.k	9.0	9.5	10.0	10.5	11.0	11.5	12.0	12.5	13.0	13.5	14.0	14.5	15.0
	5.50	74.69	355	315	280	250	225	205	185	170	155	140	130	115	105
	6.00	85.06	400	360	320	290	260	235	210	195	175	160	145	135	125
	6.25	90.25	400	380	340	305	275	250	225	205	185	170	155	145	130
ONE	6.50	95.43	400	400	360	325	290	265	240	215	200	180	165	150	140
FOOT	7.00	105.80	400	400	400	360	325	290	265	240	220	200	185	170	155
	7.25	110.99	400	400	400	375	340	305	280	255	230	210	195	175	165
	7.50	116.17	400	400	400	395	355	320	290	265	240	220	200	185	170
	8.00	126.54	400	400	400	400	385	350	320	290	265	240	220	205	185

As you can see in the table above, the selected metal deck can support 215 pounds per square foot for a 12 ½ foot span with a 6 ½" concrete slab. The total load that the decking needs to support is 188 pounds per square foot and therefore the metal decking is adequate for use. See Appendix F for supporting calculations.

Typical Composite Beam:

Typical composite beam sizes for the John Jay College Expansion Project are W16x26 [30]. Figure 14 displays the location of these beams which are typically spaced at 12.61 feet and span 30 feet. These beams were checked for bending, shear, and deflection and the design calculations are in Appendix F.

After designing a typical composite beam, it was verified that a W16x26 meets all requirements for strength and serviceability. A minimum number of shear studs of 12 was calculated to ensure composite action between the floor slab and steel beam. Although this size meets all design requirements for strength and serviceability, $M_u/\Phi M_n=0.98$ (see Appendix F). This means the beam is adequate, but common engineering practice would be to add more shear studs to increase the bending capacity of the composite beam. This approach was adopted by the structural engineer, as they have provided 30 shear studs for this beam per the construction documents. Also, by supplying 1 shear stud per foot of beam, the metal decking can support more superimposed live load than a beam with greater shear stud spacing.

Typical Composite Girder:

After analyzing a typical composite girder, it was determined that the minimum beam size to resist the required loading is a W24x55. This beam size was selected by determining the minimum beam size required to exceed the minimum construction dead load deflection criteria of span/240. Once this size was selected, 44 shear studs were required to create composite action between the floor slab and beam. An area of concern is the bending capacity of this member, where $M_u/\Phi M_n=0.95$. As mentioned above, it is common in engineering practice to increase the amount of shear studs or increase the beam size to allow for unforeseen conditions. The structural engineer took this approach for design and used a W24x68 [42] for typical composite girders. See Appendix F for all design assumptions and calculations.

Typical Column:

Typical columns for the expansion project support floors 2 through 5 and are spliced at level 3. Column load takedowns are available upon request, but service and factored compressive loads are listed in Appendix F for reduced and unreduced live loads. Table 4-1 of the Steel Construction Manual was used to size columns. Un-braced lengths of the column were determined by floor to floor heights and columns were assumed to be pinned at the top and bottom. Live loads were reduced for design with the exception of level 5 due to an un-reducible cafeteria live load of 100 psf. See Appendix F for the complete design procedure.

Using the assumptions listed above, the top portion (levels 3 through 5) of column M/7 was sized to be a W14x61 and the bottom (levels 1 through 3) was sized to be a W14x61. Column M/7 is listed in the structural drawings as a W14x74 at the top and a W14x82 at the bottom, which are both slightly larger than the columns designed for this report. There are many possible reasons for these discrepancies, but it is most likely that the designer increased the column size to account for any unexpected future loads. For example, typical spaces supported by columns are classrooms, but there are also many laboratory spaces at higher levels in the building supported by plate hangers. If the school needs to move some laboratories to lower levels, the columns may have the capacity to support loads from heavy machinery. Another possible reason for the different column sizes is that the structural engineer could have been conservative and used un-reduced live loads for the entire column because the cafeteria loading at level 5 is un-reducible.

Typical Perimeter Plate Hangers:

Floors 6 through 14 of the tower are hanging from perimeter plate hangers which transfer gravity loads from the floors up to transfer trusses at the penthouse level. The plate hangers are 50 ksi steel and are spliced at every two levels using 1 1/8" diameter ASTM A490 bolts. A typical load "take-up" is listed in Appendix F for service and factored loads with both reduced and un-reduced live loads. To design the plate hangers, required tensile areas of 50 ksi steel were calculated at level 13 (point of maximum load) with reduced live loads. Tensile areas were determined for two limit states: yielding and rupture, in which the effective area for rupture was assumed to be 75% of the gross area to allow for splicing connections. See Appendix F for plate hanger design calculations.

The design of plate hanger L/7 (same location in plan as column M/7) at the 13th level of the John Jay College Expansion Project resulted in a tensile area of 22.56 inches squared, which was controlled by rupture. This same plate hanger is shown on the structural drawings as a 1 ½" by 18" plate, resulting in 27 inches squared.

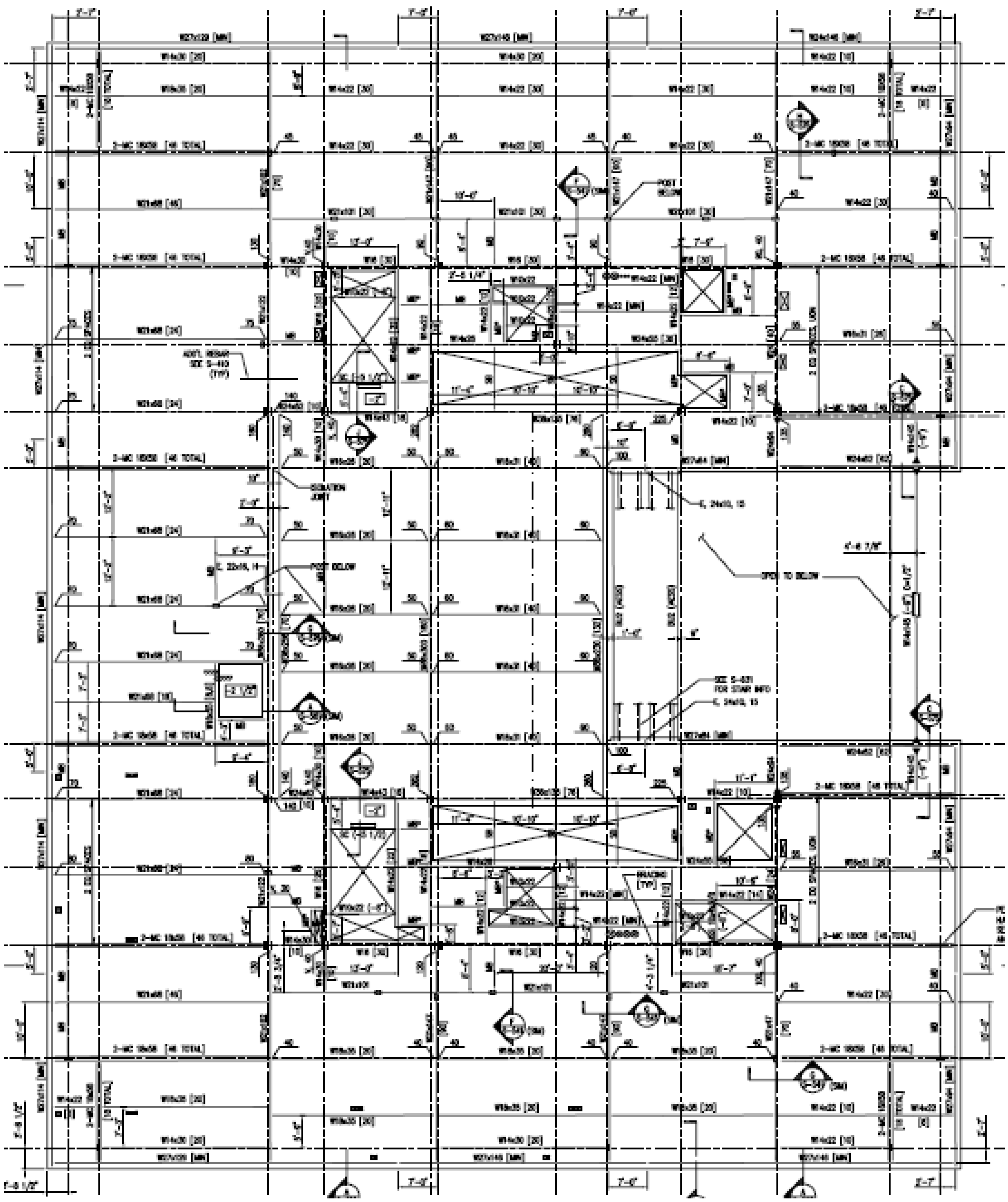
These differences between plate areas can be explained by the progressive collapse analysis performed by the structural engineer. The designers wanted to ensure redundancy in the hanging structure, so the plate hangers and transfer trusses were designed to prevent collapse in the event of the removal of a plate hanger. A similar calculation is listed in Appendix F using the "take-up" loads and resulted in an area of 25.4 inches squared, which is comparable to the 27 inches squared provided by the structural engineer.

Conclusion

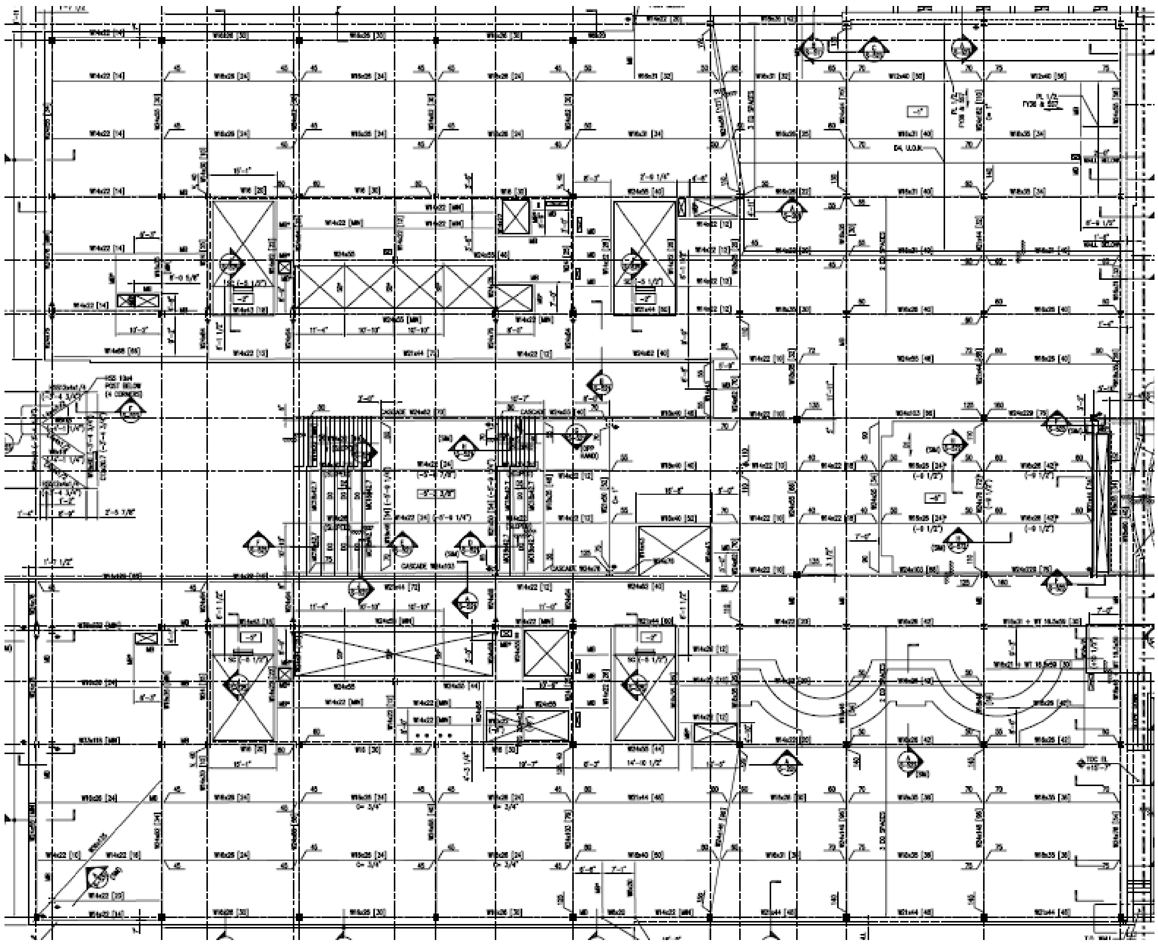
In the first technical report of the John Jay College Expansion Project, the existing building conditions are investigated. A detailed description of the buildings foundations, floors systems, columns, plate hangers, and lateral system, as well as typical floor framing plans and other images were provided to introduce the building and structure. Gravity loads were calculated from ASCE 7-05 and were used to design a typical bay in the expansion project. Lateral loads were also examined using ASCE 7-05 and were compared to the forces used by the original designers.

Spot-checks for the typical bay verified the structural engineer's results shown in the structural drawings. It was determined that the designer was conservative and that each member analyzed in this report had adequate capacity. Seismic loads calculated in this report were close to values calculated by the structural engineer despite having used a different code for this report. However, the base shears caused by wind loading were substantially larger for this report than calculated by the structural engineer. This difference is explained by the difference in codes. Method 2 in ASCE 7-05 is a more conservative approach than the Building Code of the City of New York, and therefore the larger base shear calculated in this report is acceptable.

Appendix A – Typical Floor Plans



Typical Tower Framing Plan



Typical Cascade Area Framing Plan

Appendix B – Dead Load Calculations

CALCULATION OF DEAD LOADS:

TYPICAL FLOOR CONSTRUCTION:

— CONSTRUCTION DEAD LOADS:

- 3" METAL DECK : 20 GAGE MINIMUM: USE 3 PSF
- 3½" LIGHTWEIGHT CONCRETE SLAB (115 PCF):

$$\frac{3\tfrac{1}{2}'' + 3\tfrac{1}{2}}{12''} \times 115 \text{ PCF} = 47.9 \text{ PSF}$$

- STEEL FRAMING (ADD FOR SEISMIC CALCULATIONS) = 7 PSF

$\text{TOTAL CDL}_{\text{FLOOR DESIGN}} = 51 \text{ PSF}$

$\text{TOTAL CDL}_{\text{SEISMIC}} = 58 \text{ PSF}$

— SUPERIMPOSED DEAD LOADS:

FIREPROOFING = 2 PSF

FINISHES = 5 PSF

PARTITIONS:
METAL STUDS = 4 PSF
G.W.B. = 10 PSF (2-SIDES)

14 PSF → USE 20 PSF

CEILING: 5 PSF (ASSUMED)

MECH, AND ELECTRICAL: 5 PSF (ASSUMED)

TOTAL SDL = 37 PSF

MEZZ. FLOOR CONSTRUCTION:

- CONSTRUCTION DEAD LOADS:

3" METAL DECK : 20 GAGE = USE 3 PSF

4½" NORMAL WEIGHT CONCRETE SLAB:

$\frac{4\frac{1}{2}'' + 3''/2}{12''} \times 150 \text{ PCF} = 75 \text{ PSF}$

STEEL FRAMING: (SEISMIC) = 7 PSF

TOTAL CDL (FLOOR DESIGN) = 78 PSF

TOTAL CDL (SEISMIC) = 85 PSF

HEAVY MECHANICAL EQUIPMENT BY LEVEL

- 6TH FLOOR, 7TH FLOOR, & 8TH FLOOR:

INCREASED LOADS IN LABORATORY SPACES FOR HEAVY EQUIPMENT.

ASSUME: 100 PSF *

*VERIFY WITH E.O.R.

- PENTHOUSE MEZZANINE LEVEL:

$15^K + 48^K = 63^K$

- PENTHOUSE LEVEL:

$18^K + 50^K \times 3 \text{ UNITS} + 15^K \times 3 \text{ UNITS} + 100^K \times 2 \text{ UNITS}$

$+ 120^K + 10^K \times 4 \text{ UNITS} + 280^K$

$= 853^K$

COMMONS ROOF

ASSUME 115 PSF AVERAGE.

CURTAIN WALL DEAD LOAD:

ASSUME 25 PSF

1ST LEVEL PERIMETER WALL:

$\frac{18''}{12''} \times 150 \text{ PCF} = 225 \text{ PSF}$

Appendix C – Wind Analysis

V=	110	mph
K_d=	0.85	
K_d=	1.85	
K_d=	2.85	
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

Appendix D – Seismic Analysis

The following table displays the assumptions used to calculate the seismic forces using ASCE 7-05.

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=	6		Special steel concentrically braced frames
I=	1.25		Occupancy Category III
T_a=	1.218		
C_u=	1.7		
T_L=	6	seconds	
C_s=	0.01	<--- Governs	
C_s=	0.058		
k=	1.36		
W=	81866	Kips	
V=	819	Kips	

Appendix E – Snow Loads

The following snow loads were calculated from ASCE 7-05. Snow loads were calculated for drifts at typical parapets and for drifts on the commons roof against the tower.

Snow Loads		
P_g=	20	psf
C_e=	1.1	Terrain Category C
C_t=	1	
I=	1.1	Assume Category III
P_f=	22	psf
Rain-on-Snow Surcharge=	5	psf

Snow Drifts		
γ=	16.60	pcf
$h_{c\ parapet}$=	1.50	ft
$h_{c\ cascade\ roof}$=	10.00	ft
h_d=	4.25	ft
h_d=	1.33	
$h_c/h_{b\ parapet}$=	1.14	
$h_c/h_{b\ cascade\ roof}$=	7.55	
$w_{parapet}$=	48.02	
$w_{cascade\ roof}$=	17.00	
$DRIFT_{parapet}$:	Yes	
$DRIFT_{cascade\ roof}$:	Yes	
Max Drift $Load_{parapet}$=	24.98	psf
Max Drift $Load_{cascade\ roof}$=	70.55	psf

Appendix F – Typical Bay Spot Checks

CHECK METAL DECKING

FROM STRUCTURAL DESIGN CRITERIA:

- 3½" L.W. CONC. SLAB
- 3" DECKING
- 40 KSI YIELD STRENGTH
- MINIMUM 20 GAGE

FROM UNITED STEEL DECK WEBSITE:

3" LOK, 20 GAGE, 115 PCF CONCRETE

MAXIMUM UNSHORED SPAN:

6½" SLAB, 3 SPANS } 12.21'

SPAN BETWEEN BEAMS: (USE W16x26's ON C.D.'S)

$12.61' - 5.5''/12'' = 12.15'$ OK ✓

CHECK SUPERIMPOSED LL:

STUD SPACING = 1', SPAN = 12.5', 6½" SLAB } 215 PSF

TYPICAL FLOOR LOADING:

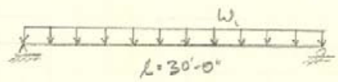
LL = 100 PSF
CDL = 51 PSF
SDL = 37 PSF

TOTAL SERVICE LOAD = 188 PSF OK ✓

USE 3" LOK-FLOOR, 20 GAGE MINIMUM
w/ F_Y = 40 KSI

BEAM CHECK

CLASSROOM LOADING.



TRIB WIDTH = 13'-0"

W_1

CDL = 51 PSF × 13'-0" = 0.663 KLF
SDL = 37 PSF × 13'-0" = 0.481 KLF
LL = 40 PSF × 13'-0" = 0.520 KLF

FACTORED LOADS

$W_u = 1.2(0.663 + 0.481) + 1.6(0.520) = 2.20$ KLF

DESIGN

$M_u = \frac{2.20(30^2)}{8} = 248$ FT-K

$V_u = 2.20 \times \frac{30}{2} = 33^K$

ASSUME $a = 1''$

$Y_2 = 6\frac{1}{2}'' - \frac{1}{2} = 6''$

TRY W16×26 $\phi M_n = 252'^K$ PNA #7

$b_{eff} = \left| \begin{matrix} 13' \\ 30'/4 = 7.5' \end{matrix} \right| = 7.5' = 90''$

$a = \frac{96}{0.85(4)90} = 0.313 < 1''$ ASSUMPTION OK ✓

$\Sigma Q_n = 96^K$

$\# \text{SHEAR STUDS} = \frac{96^K}{17.2^K/\text{STUD}} = 6$ STUDS × 2 = 12 STUDS

CHECK DEFLECTIONS:

Δ CONSTRUCTION $I = 301$ IN^4

$\Delta_{CDL} = \frac{L}{240}$

$$\Delta = \frac{5}{384}\,\frac{(0.663)\,30^4 \times 1728}{29000 \times 301} = 1.38'' = \frac{L}{260} < \frac{L}{240} \quad \text{ok}\checkmark$$

Δ_{LL} $\quad I = 595\ \text{IN}^4$

$$\Delta = \frac{5}{384}\,\frac{(0.520)\,30^4 \times 1728}{29000 \times 595} = 0.55'' = \frac{L}{655} < \frac{L}{500} \quad \text{ok}\checkmark$$

USE W16×26 [12]

$\phi M_n = 252'^k > M_u = 242'^k$ ok✓

$\phi V_n = 106^k > V_u = 33^k$ ok✓

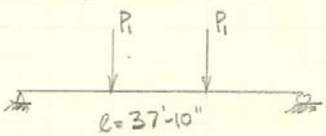
Δ_{CDL} ok✓

Δ_{LL} ok✓

Technical Assignment #1

GIRDER CHECK

LOADING DIAGRAM



P_1 @ 1/3 POINTS

P_1 (FROM BEAM CHECK)

$CDL = 0.633\ KLF \times 15' + 0.035\ KLF \times 15' = 10.5^K$
$SDL = 0.481\ KLF \times 15' = 7.22^K$
$LL = 0.520\ KLF \times 15' = 7.80^K$

FACTORED LOAD

$P_{1u} = 1.2(10.5 + 7.22) + 1.6(7.80) = 33.75^K$

DESIGN

$M_u = 2 P_{1u}\, a \qquad a = 12.61'$

$M_u = 2(33.75)\,12.61 = 852\ \text{FT-K}$

$V_u = P_{1u} \times 2 = 67.5^K$

DESIGN FOR Δ_{CDL}

$\Delta_c = \dfrac{L}{240} \qquad \Delta = \dfrac{Pa}{24EI}(3L^2 - 4a^2)$

$$\frac{37.83' \times 12''}{240} = \frac{2 \times 10.5^K \times 12.61'}{24 \times 29000 \times I_{REQ'D}}\left(3 \times 37.83^2 - 4 \times 12.61^2\right) \times 1728$$

$I_{REQ'D} = 1272\ IN^4$

TRY W24×55 $\qquad I = 1350 \qquad \Delta_{CDL}$ OK ✓

ASSUME $a = 1''$

$Y_2 = 6\tfrac{1}{2}'' - 1''/2 = 6'' \qquad$ PNA # BFL $\qquad \phi M_n = 915\ \text{FT-K}$

$b_{eff} = \left| \begin{matrix} 30' \\ 37'\text{-}10''/4 \end{matrix} \right| = 113.5''$

$\Sigma Q_n = 456^K$

$a = \dfrac{456}{0.85(4)113.5} = 1.18 \qquad$ ASSUMPTION NOT VALID!

ASSUME $a = 2"$

$Y_2 = 6\frac{1}{2}" - 2"/2 = 5\frac{1}{2}"$ PNA # BFL $\phi M_n = 897$ FT-K

$\Sigma Q_n = 456^K$

$a = \frac{456}{0.85(4)113.5} = 1.18 < 2$ ASSUMPTION OK ✓

$\# STUDS = \frac{456^K}{21.2} \times 2 = 44$ STUDS

CHECK Δ_{LL}

$\Delta_{LL} = \frac{2(7.8)12.61}{24 \times 29000 \times 3090}(3 \times 37.83^2 - 4 \times 12.61^2)1728$

$\Delta_{LL} = 0.58" = \frac{L}{785} < \frac{L}{500}$ OK ✓

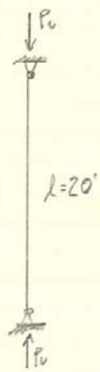
USE W24×55 [44]

$\phi M_n = 897 > M_u = 852$ FT-K OK ✓
$\phi V_n = 67.5^K > V_u = 25.1$ K OK ✓
Δ_{LL} OK ✓
Δ_{CDL} OK ✓

TYPICAL COLUMN CHECK:

- DESIGN OF COLUMN M/7
- COLUMNS ARE SPLICED JUST ABOVE LEVEL 3.
- COLUMNS SUPPORT LEVELS 2-5.
- FB HANGERS SUPPORT LEVELS 6-14.

DESIGN OF UPPER COLUMN.



$P_u = 287^k$ (SEE COLUMN TAKEDOWN SPREADSHEET)

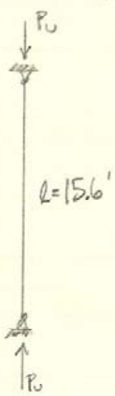
$K = 1.0$

$l = 20'$ → UNBRACED LENGTH IS TAKEN AS FLOOR TO FLOOR HEIGHT

TABLE 4-1

USE W14×61 $\quad \phi P_n = 400$ @ 20'

DESIGN OF LOWER COLUMN



$P_u = 501^k$ (SEE COLUMN TAKEDOWN SPREADSHEET)

$K = 1.0$

$l = 15.6'$ → UNBRACED LENGTH IS TAKEN AS FLOOR TO FLOOR HEIGHT

TABLE 4-1

USE W14×61 $\quad \phi P_n = 515^k$ @ 16'

DESIGNER USED W14×74 @ TOP AND W14×82 @ BASE.
POSSIBILITIES INCLUDE:
- LIVE LOAD REDUCTIONS WERE NOT CONSIDERED BY DESIGNER
- EXTRA CONTINGENCY FOR FUTURE CONDITIONS

PERIMETER PLATE HANGER CHECK

- DESIGN OF PL HANGER L/7 (SAME LOCATION IN PLAN AS M/7)

DESIGN OF UPPER HANGER

P_u = 825 K (SEE PL HANGER TAKE-UP SPREADSHEET)

DETERMINE AREA FOR YIELDING:

$\phi P_n = \phi F_y A_g$

$825^k = 0.9(50 \text{ ksi}) A_g$

$A_g = 18.33 \text{ in}^2$

DETERMINE AREA FOR RUPTURE:

$\phi P_n = \phi F_u A_e$ ASSUME $A_e = 0.75 A_g$ ALLOWANCE FOR CONNECTIONS

$825^k = 0.75(65 \text{ ksi})(0.75 A_g)$

$A_g = 22.56 \text{ in}^2$ ← CONTROLS

ORIGINAL DESIGN:
DESIGNER USED PL HANGER @ LEVEL 13
W/ AREA = 27 IN².

- DESIGNER USED PROGRESSIVE COLLAPSE ANALYSIS AND DESIGNED PL'S TO RESIST COLLAPSE IN THE EVENT OF THE REMOVAL OF ONE PL HANGER.
- LOAD CASE USED FOR COLLAPSE PREVENTION DESIGN IS 0.75(D+L)

$P_u = 0.75(825) = 620\ ^K \times 1.5 = 928\ ^K$
SERVICE LOAD W/ UNREDUCED LL.

Y: $928^k = 0.9(50) A_g$

$A_g = 21 \text{ in}^2$

R: $928^k = 0.75(65) 0.75 A_g$ $A_g = 25.4 \text{ in}^2$ ← CONTROLS

Level	Service loads				Factored loads			
	Total Load	Total Load $_{red}$	Takedown	Takedown$_{red}$	Total Load	Total Load $_{red}$	Takedown	Takedown$_{red}$
	K	K	K	K	K	K	K	K
5	129	129	129	129	180	180	180	180
4	105	84	234	213	140	108	320	287
3	105	84	338	298	140	108	460	395
2	103	83	441	380	138	106	598	501

Column M/7 Take-down

Level	Service loads				Factored loads			
	Total Load	Total Load $_{red}$	Takeup	Takeup$_{red}$	Total Load	Total Load $_{red}$	Takeup	Takeup$_{red}$
	K	K	K	K	K	K	K	K
13	108	86	899	677	144	109	1227	872
12	102	80	791	591	138	102	1082	763
11	102	80	689	511	138	102	945	661
10	102	80	586	431	138	102	807	558
9	102	80	484	351	138	102	669	456
8	127	90	381	270	177	118	531	354
7	127	90	254	180	177	118	354	236
6	127	90	127	90	177	118	177	118

Perimeter Plate Hanger L/7 Take-up