New York Police Academy

Architectural Engineering Senior Thesis 2010

Technical Report III – Lateral System Analysis and Confirmation Design

Jake Pollack
Structural Option
Faculty Consultant – Dr. Boothby
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EXECUTIVE SUMMARY

The New York Police Academy is located in College Point, New York and is 536 feet long, 95 feet wide and 150 feet high. The building has a gravity system consisting of lightweight concrete on metal deck. In the East/West direction the building has moment connections and one double bay of HSS cross bracing to resist lateral loads. In the North/South direction the lateral resisting system consists of HSS cross bracing in two of the three bays.

In this technical report the lateral system of The New York Police Academy was analyzed under various conditions. Some assumptions were made to simplify thesis calculations. The building was made more rectilinear and typical beam, girder, and column sizes were used in the analysis model.

When reading through this report, please note that in ASCE 7-10 the wind load combinations are different from ASCE7-05. The wind load factors have been altered from 0.8W to 0.5W and 1.6W to 1.0W. This may be the result of the increased wind speeds (V) when performing wind load analysis. It was found that the combination: 1.2D + 1.0W + L + 0.5S controlled in the North/South direction and the combination: 1.2D + 1.0E + L + 0.2S controlled in the East/West direction.

An ETABS model was created for the analysis of the New York Police Academy. Results were obtained from this model and by hand throughout this technical report.

The overturning moment was controlled by wind with a magnitude of 94,182.1 ft-k. The resisting moment due to weight was 2,560,487.5 ft-k and therefore overturning was not an issue.

Direct shear calculations show the distribution of lateral loads to the lateral resisting systems. In The New York Police Academy there are two types of lateral resisting systems and they are classified into two frames: the X-Frame and the Y-Frame. The X-Frame is predominantly made up of moment connections. There are four X-Frames in this building. The exterior frames receive approximately 32% of the controlling lateral load and the interior frames receive approximately 18% of the controlling lateral load. There are ten

Y-Frames and they consist of pinned HSS cross bracing. Each Y-Frame receives approximately 10% of the controlling lateral load in the North/South direction.

The centers of pressure and rigidity were very similar in location to the center of mass which limited the amount of torsion on The New York Police Academy. The largest amount of torsional shear was applied to Frame A with a magnitude of 0.538 kips.

The New York Police Academy's lateral system performed well under lateral loads. The maximum story drift in the North/South direction was due to seismic loads and was 0.121", which was much less than the allowed 3.36". The maximum displacement due to seismic loads was 0.543" which was well within the maximum total displacement. The maximum story drift in the East/West direction was due to wind loads and was 0.179", which was less than the allowed 0.424". The maximum displacement due to wind loads was 0.767", which was less than the total displacement of 4.5".

A typical W24x162 column and a typical HSS10x10x½ cross brace were checked for strength on the 6^{th} floor. This was done to ensure that these members could resist the applied lateral loads. Both members were sufficient to support the necessary loads.

INTRODUCTION

The New York Police Academy is located in College Point, a neighborhood in Queens, New York. This building is an 8-story structure with a west and east campus. It is the first and largest phase of a multiphase project. The west campus houses a physical training facility and a central utility plant while the east campus houses an academic building. The east campus will be analyzed in this technical report. The physical training facility includes a 1/8 mile running track and special tactical gymnasiums. The academic building has a wide variety of classrooms ranging from a capacity of 30 to 300 cadets. Some classrooms create a mock environment for the cadets to experience immersion learning. This phase is expected to cost $656 million. Construction is to begin in October 2010 and culminate in December 2013.

FIGURE 1: THIS IMAGE SHOWS THE LOCATION OF THE NEW YORK POLICE ACADEMY IN ITS SURROUNDINGS.

The purpose of Technical Report III, *the Lateral Systems Analysis and Confirmation Design,* is to gain a better understanding of the current lateral system and compute the effects of wind and seismic loads on the New York Police Academy structure. The lateral systems will be analyzed and conclusions will be determined about their effects on the building.

DESIGN CODES AND STANDARDS

DESIGN CODES:

Design Codes:

- American Concrete Institute (ACI) 318-08, Building Code Requirements for Structural Concrete
- American Concrete Institute (ACI) 315-08, Details and Detailing of Concrete Reinforcement
- American Institute of Steel Construction Manual, 13^{th} Edition
- American Welding Society D1.1-08: Structural Welding Code

Model Codes:

- New York City Building Codes 2008

Structural Standards:

- American Society of Civil Engineers (ASCE) 7-98, Minimum Design Loads for Building and Other Structures

THESIS CODES:

Design Codes:

- American Concrete Institute (ACI) 318-05, Building Code Requirements for Structural Concrete
- AISC Steel Construction Manual, 13^{th} Edition

Model Codes:

- 2006 International Building Code (IBC)

Structural Codes:

- American Society of Civil Engineers (ASCE) 7-10, Minimum Design Loads for Building and Other Structures

DESIGN CRITERIA

DEFLECTION

Floor Deflection:

- Live Load
    - ◊ $< \frac{L}{360}$
- Total Load
    - ◊ $< \frac{L}{240}$

Lateral Drift:

- Wind
    - ◊ Total Building Drift: $< \frac{L}{400}$
    - ◊ Story Wind Drift: $< \frac{L}{600}$
- Seismic Loads
    - ◊ Story Seismic Drift: $< 0.020h_{sx}$

Main Structural Elements Supporting Components and Cladding:

- At Screen Walls
    - ◊ $< \frac{L}{240}$
- At Floors Supporting Curtain Walls
    - ◊ $< \frac{L}{600}$
- At Roof Parapet Supporting Curtain Walls
    - ◊ $< \frac{L}{600}$
- At Non-Brittle Finishes
    - ◊ $< \frac{L}{240}$

*Please note that the deflection criteria have been altered since Technical Reports I and II.

MATERIAL PROPERTIES

STEEL

Wide Flanges, Tees	F_y = 50 ksi (A992)
Hollow Structural Sections	F_y = 50 ksi (A500 Grade B)
Structural Pipe Sections	F_y = 36 ksi (A36)
Channels and Angles	F_y = 36 ksi (A36)
Plates	F_y = 50 ksi (A572 Grade 50)
Plates	F_y = 42 ksi (A572 Grade 42 for t_{steel}>4")
Bolts	F_u = 105 ksi (A325)
	F_u = 150 ksi (A490)
Anchor Bolts	F_y = 36 ksi (F1554 Grade 36)
Metal Deck	F_y = 33 ksi (A653)
Weld Strength	F_y = 70 ksi (E70XX)

CONCRETE

Foundations, Int. Slab on Grade	NWC f'c = 4000 psi
Slab on Metal Deck	LWC f'c = 4000 psi

REINFORCING

Welded Wire Fabric	70 ksi
Bars to be Welded	60 ksi
Epoxy Coated Bars	60 ksi
All Other Bars (unless otherwise noted)	60 ksi

EXISTING STRUCTURAL SYSTEM OVERVIEW

The New York Police Academy's East Campus is 536 feet long and 95 feet wide. The floor to floor height ranges from 14 feet to 16 feet. A green roof system is present on the top of the building. The structure of the New York Police Academy consists predominantly of steel framing with a 14" concrete slab on grade on the first floor. All other floors have a lightweight concrete on metal deck floor system. All concrete is cast-in-place.

FOUNDATION SYSTEM

The geotechnical engineering study was conducted by the URS Corporation. The study showed a variety of soil composition, with bedrock reasonably close to the surface. The building foundations for the New York Police Academy bear on piles with a minimum bearing capacity of 100 tons as specified by the URS Corporation. All piles are driven to bedrock. All exterior pile caps are placed a minimum of 4'-0" below final grade. Please see Figure 3 for example pile cap. Concrete piers, walls, structural slabs on grade, pile caps and grade beams are placed monolithically. Piles are 16" in diameter.

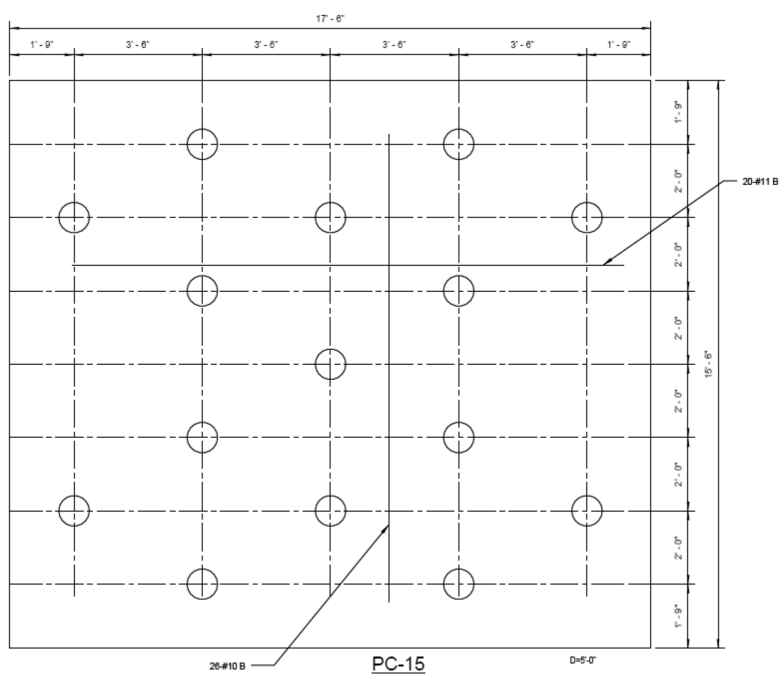


FIGURE 2: THIS IS PLAN OF A SAMPLE PILE CAP. DETAIL COURTESY OF TURNER CONSTRUCTION.

FLOOR SYSTEM

The floor system is made up of 3.25" lightweight concrete slab on 3" - 18 gage metal decking. This will form a one-way composite floor slab system. Units are continuous over three or more spans except where framing does not permit. Shear stud connectors are welded to steel beams or girders in accordance to required specifications. See Figure 4 for details.

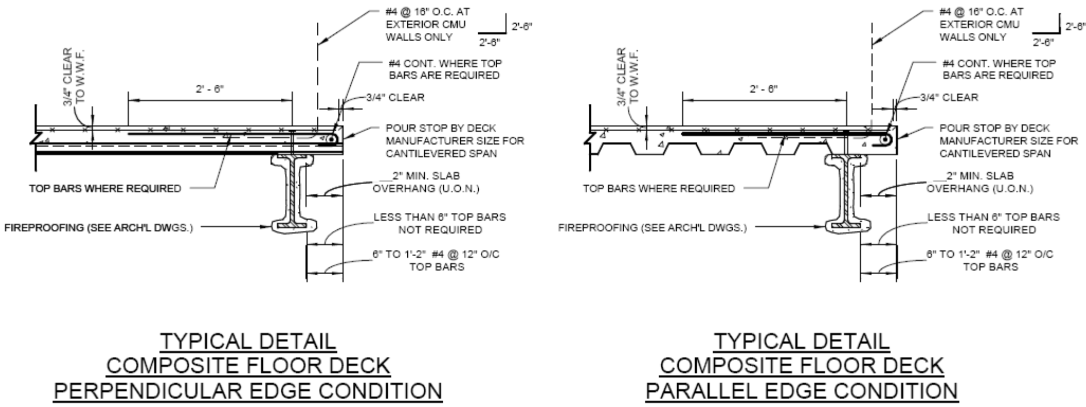


Figure 3: Typical slab on deck floor sections. Drawings not to scale. Detail courtesy of Turner Construction.

FRAMING SYSTEM

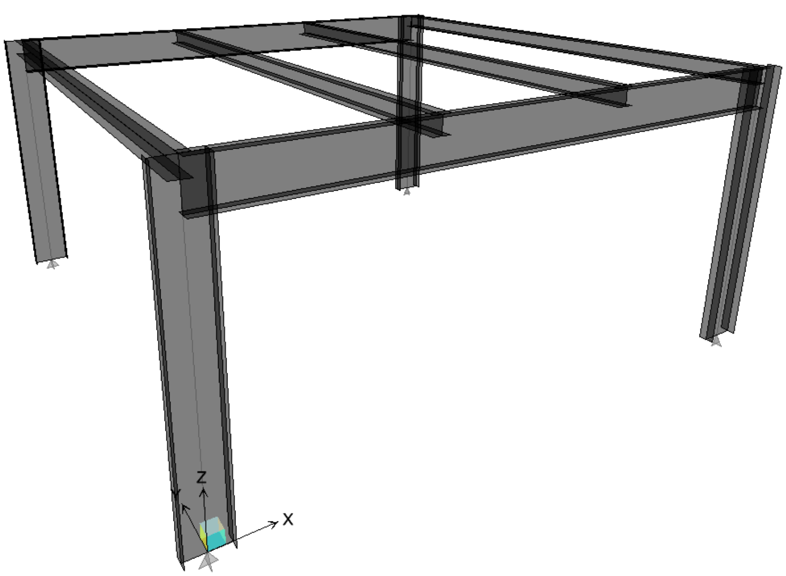
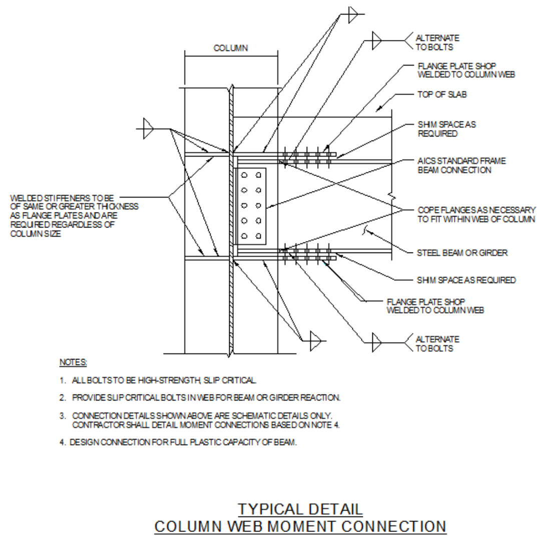


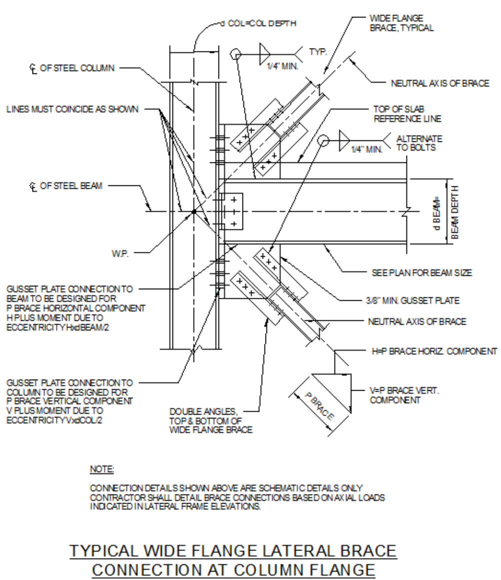
Figure 4: This is an ETABS model of the typical bay framing.

The superstructure is primarily composed to W18 beams, W24 girders and W24 columns. Beams are spaced at 10' increments while girders are spaced at 30' increments. Columns are on a 30'x30' grid. The columns are spliced at 4' above every other floor level and typically span from 30' to 34'. A typical bay is shown in Figure 5.

LATERAL SYSTEM

The lateral resisting system in the New York Police Academy consists of steel moment connections in the East/West direction with HSS and wide flange cross bracing in the North/South direction. The lateral resisting connections can be seen in Figure 5 below. The rest of the technical report will discuss the lateral system in more detail. The lateral load paths, load distribution, torsion, drift and overturning moments will be covered. The building was modeled in ETABS, a structural analysis program, to compare to hand calculations for verification.





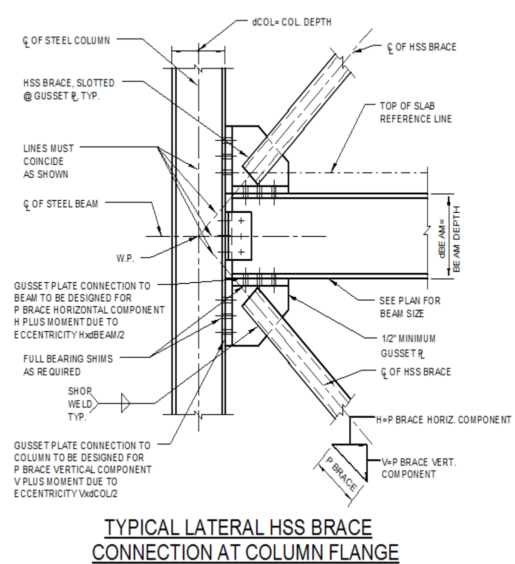


FIGURE 5: TYPICAL COLUMN WEB MOMENT CONNECTION (TOP RIGHT). TYPICAL LATERAL HSS BRACE CONNECTION (BOTTOM RIGHT). TYPICAL WIDE FLANGE LATERAL BRACE CONNECTION (LEFT). ALL DRAWINGS ARE NOT TO SCALE. DETAILS COURTESY OF TURNER CONSTRUCTION.

DESIGN LOADS

Robert Silman Associates, the structural engineer of record on this project, used ASCE 7-98 and the BCNYC 2008 as the main reference for dead and live loads on this project. These loads are compared to the most recent applicable standards, ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures*. The load differences per respective codes can be compared in Tables 1 and 2 below. Table 1 shows dead loads while Table 2 outlines the live loads for this building. The loads used for thesis analyses are from ASCE 7-10 unless not specified in the code.

SUPERIMPOSED DEAD LOADS			
DESCRIPTION	**LOCATION**	**NYCBC 2008**	**ASCE 7-10**
CEILING	FLOORS 2-8, ROOF, MEP	5 PSF	--
MEP	FLOORS 2-8, ROOF, MEP	5 PSF	5 PSF
FLOOR FINISHED	FLOORS G-8	5 PSF	--
ROOFING AND INSULATION	FLOORS 3, ROOF, MEP	8 PSF	15 PSF
PARTITIONS	FLOORS G-8	20 PSF	15 PSF
CURTAIN WALL	FLOORS G-ROOF	NOT SPECIFIED	15 PSF
GREEN ROOF	ROOF	NOT SPECIFIED	100 PSF

TABLE 1: THIS TABLE COMPARES SUPERIMPOSED DEAD LOADS BETWEEN NYCBC-08 AND ASCE 7-10.

LIVE LOADS			
DESCRIPTION	LOCATION	NYCBC 2008	ASCE 7-10
ARMORIES AND DRILL ROOMS	FLOOR G	150 PSF	150 PSF
FIXED SEAT ASSEMBLY AREA	FLOORS 2-5, 8	60 PSF	60 PSF
LOBBIES	FLOORS G-8	100 PSF	100 PSF
CORRIDORS (TYP.)	FLOORS 2-8	100 PSF	100 PSF
1ST FLOOR OFFICE CORRIDORS	FLOORS G	100 PSF	80 PSF
UPPER FLOOR OFFICE CORRIDORS	FLOORS 2-8	80 PSF	80 PSF
EQUIPMENT ROOMS	FLOORS G, 2, 7-8	75 PSF	75 PSF
LIBRARY READING ROOMS	FLOOR 8	60 PSF	60 PSF
LIBRARY STACKS	FLOOR 8	150 PSF	150 PSF
OFFICES	FLOOR 2-8	50 PSF	50 PSF
FILE AND COMPUTER ROOMS	FLOOR 7	150 PSF	100 PSF
CLASSROOMS	FLOORS 2-8	50 PSF	50 PSF
STAIRS AND EXITS	FLOORS G-MEP	100 PSF	100 PSF
LIGHT STORAGE	FLOORS G-7	125 PSF	125 PSF
HEAVY STORAGE	FLOORS 7, MEP	250 PSF	250 PSF
SNOW	FLOORS 3, MEP, ROOF	22 PSF	22 PSF
*LIVE LOADS REDUCED WHERE APPLICABLE **SNOW DRIFT INCLUDED WHERE APPLICABLE			

TABLE 2: THIS TABLE COMPARES LIVE LOADS BETWEEN NYCBC-08 AND ASCE 7-10

LATERAL LOADS - WIND

Wind load calculations were performed with the assumptions that the façade and geometry of the New York Police Academy was regular with no protrusions. It was also assumed that there are no channeling effects or buffeting in the wake of upwind obstructions. The summary of results is found in Tables 3 and 4, and can be seen in Figures 6 and 7. For a more in depth look at wind load calculations please refer to Appendix B.

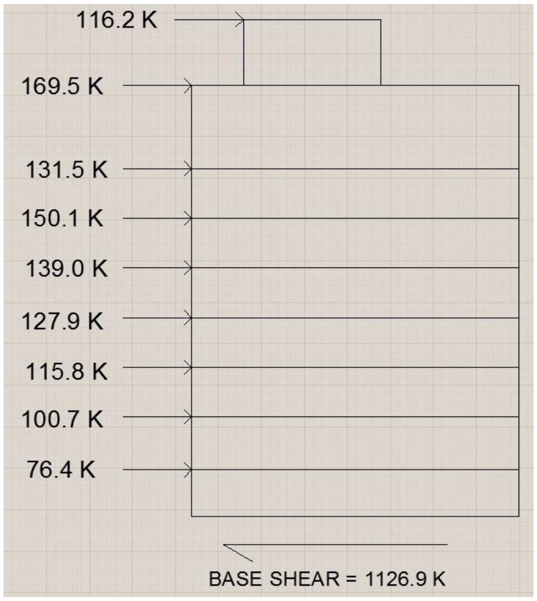


FIGURE 6: THE FIGURE TO THE RIGHT GRAPHICALLY SHOWS THE WIND SHEAR FORCE ON EACH STORY IN THE NORTH/SOUTH DIRECTION.

* Please note that internal wind pressures have been updated since Technical Report I

NORTH/SOUTH WIND LOADS								
FLR	STORY HIEGHT (FT)	HEIGHT ABOVE GROUND (FT)	CONTROLLING WIND PRESSURE (PSF)		TOTAL CONTROLLING PRESSURE (PSF)	FORCE OF WIND-WARD PRESSURE (K)	STORY SHEAR WIND-WARD (K)	MOMENT WIND-WARD (FT-K)
			WIND-WARD	LEE-WARD				
BULK-HEAD	20	150	21.67	-13.11	34.78	116.2	0.0	3250.66
ROOF	10	120	19.92	-13.11	33.03	169.5	116.2	2389.99
8	15	105	19.42	-13.11	32.53	131.4	285.7	2038.60
7	15	90	17.91	-13.11	31.02	150.1	417.1	16121.03
6	15	75	16.66	-13.11	29.77	139.0	567.2	1249.37
5	15	60	15.15	-13.11	28.26	127.9	706.1	909.26
4	15	45	13.65	-13.11	26.76	115.8	834.0	614.28
3	15	30	11.39	-13.11	24.50	100.7	949.8	341.84
2	16	14	7.13	-13.11	20.24	76.4	1050.5	99.87
G	14	0	0.00	0.00	0.00	0.0	1126.9	0.00
						Σ	1126.9 K	3250.66 FT-K

TABLE 3: THE TABLE ABOVE SHOWS THE FLOOR WIND PRESSURES AND FORCES ALONG WITH SHEAR/MOMENT FORCES IN THE NORTH/SOUTH DIRECTION.

EAST/WEST WIND LOADS								
FLOOR	STORY HIEGHT (FT)	HEIGHT ABOVE GROUND (FT)	CONTROLLING WIND PRESSURE (PSF)		TOTAL CONTROLLING PRESSURE (PSF)	FORCE OF WINDWARD PRESSURE (K)	STORY SHEAR WINDWARD (K)	MOMENT WINDWARD (FT-K)
			WINDWARD	LEEWARD				
BULK-HEAD	20	150	16.11	-20.06	36.17	15.3	0.0	2416.02
ROOF	10	120	14.70	-20.06	34.76	22.3	15.3	1764.39
8	15	105	14.30	-20.06	34.36	17.2	37.6	1501.73
7	15	90	13.10	-20.06	33.16	19.5	54.8	1178.92
6	15	75	12.10	-20.06	32.16	18.0	74.3	907.24
5	15	60	10.89	-20.06	30.95	16.4	92.2	653.61
4	15	45	9.69	-20.06	29.75	14.7	108.6	436.07
3	15	30	7.89	-20.06	27.95	12.5	123.3	236.58
2	16	14	4.48	-20.06	24.54	9.0	135.8	62.68
G	14	0	0.00	0.00	0.00	0.0	144.8	0.00
						Σ	**144.8 K**	**2416.02 FT-K**

TABLE 4: THE TABLE ABOVE SHOWS THE FLOOR WIND PRESSURES AND FORCES ALONG WITH SHEAR/MOMENT FORCES IN THE EAST/WEST DIRECTION.

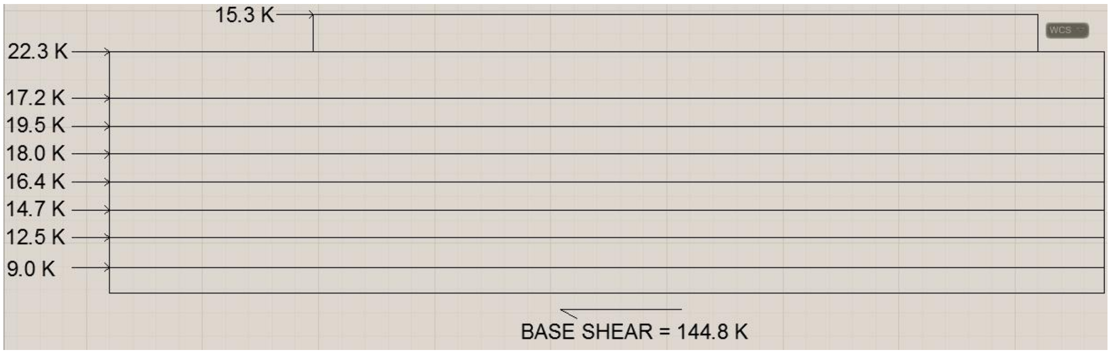


FIGURE 7: THIS FIGURE GRAPHICALLY SHOWS THE WIND SHEAR FORCE ON EACH STORY IN THE EAST/WEST DIRECTION.

LATERAL LOADS - SEISMIC

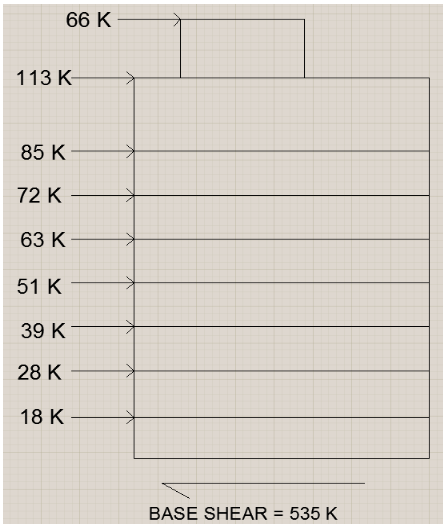


FIGURE 8: THIS FIGURE GRAPHICALLY SHOWS THE SEISMIC SHEAR FORCE ON EACH STORY IN THE NORTH/SOUTH DIRECTION.

Seismic loads for the New York Police Academy were performed using Chapters 11 and 12 of ASCE 7-10 using the Equivalent Lateral Force Procedure. Included in the analysis were the dead loads from floor slabs, steel framing, glass curtain walls and superimposed dead loads. An additional allowance was also used for roof gardens and mechanical equipment upon the rooftop as applicable. Seismic calculations were performed by hand and various area square footages were assumed and approximated. The summary of results is found in Tables 5 and 6, and can be seen in Figures 8 and 9. For a more in depth look at seismic load calculations please refer to Appendix C.

NORTH/SOUTH SEISMIC FORCES							
FLOOR	WEIGHT w_x (K)	HEIGHT h_x (FT)	$w_k h_x^k$	C_{vx}	LATERAL FORCE F_x (k)	STORY SHEAR V_x (k)	MOMENT M_x (K)
BULKHEAD	3,322	150	1,004,948	0.120	66	50	1730
ROOF	6,753	130	1,735,370	0.207	113	161	2988
8	5,574	120	1,307,476	0.156	85	245	2251
7	5,574	105	1,122,853	0.134	72	318	1933
6	5,847	90	988,029	0.118	63	381	1701
5	5,847	75	802,607	0.096	51	433	1382
4	5,847	60	622,337	0.074	39	473	1071
3	5,920	45	453,925	0.054	28	502	781
2	5,920	30	285,917	0.034	18	520	492
TOTAL	50604		8,323,461		535		14,332

TABLE 5: THIS TABLE SHOWS THE CACULATIONS AND PROCESSES NEEDED IN ORDER TO CALCULATE SEISMIC BASE SHEAR IN THE NORTH/SOUTH DIRECTION.

EAST/WEST SEISMIC FORCES							
FLOOR	WEIGHT w_x (K)	HEIGHT h_x (FT)	$w_k h_x^k$	C_{vx}	LATERAL FORCE F_x (k)	STORY SHEAR V_x (k)	MOMENT M_x (K)
BULKHEAD	3,322	150	1,004,948	0.120	77	41	368
ROOF	6,753	130	1,735,370	0.207	134	175	636
8	5,574	120	1,307,476	0.156	101	276	479
7	5,574	105	1,122,853	0.134	87	362	411
6	5,847	90	988,029	0.118	76	438	362
5	5,847	75	802,607	0.096	62	500	294
4	5,847	60	622,337	0.074	48	548	228
3	5,920	45	453,925	0.054	35	583	166
2	5,920	30	285,917	0.034	22	605	105
TOTAL	**50604**		8,323,461		**642**		3,049

TABLE 6: THIS TABLE SHOWS THE CACULATIONS AND PROCESSES NEEDED IN ORDER TO CALCULATE SEISMIC BASE SHEAR IN THE NORTH/SOUTH DIRECTION

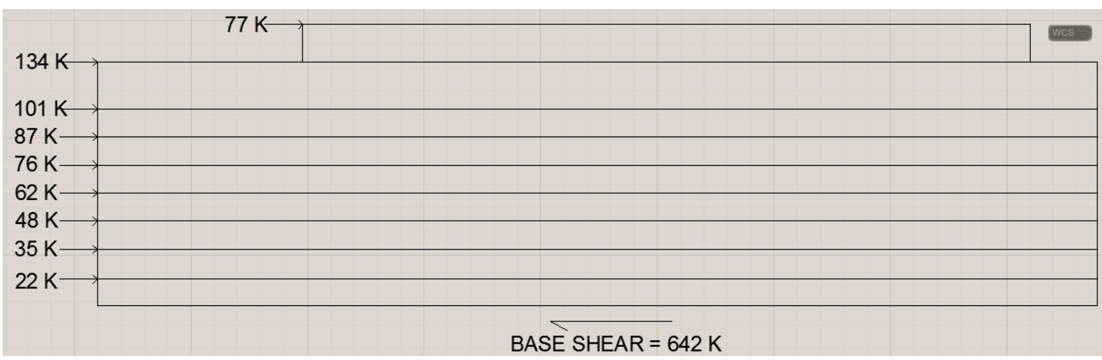


FIGURE 9: THIS FIGURE GRAPHICALLY SHOWS THE SEISMIC SHEAR FORCE ON EACH STORY IN THE EAST/WEST DIRECTION.

LOAD COMBINATIONS

The lateral systems analyzed in this report are governed by the load combinations found in ASCE 7-10 and can be seen in Table 3 below. Please note that the wind load factor has changed since the ASCE 7-05 edition. In ASCE 7-10 the wind load factor is 0.5W in Case 3 and 1.0W in Cases 4 and 6. In ASCE 7-05 the wind load factor was 0.8W in Case 3 and 1.6W in Cases 4 and 6. However, also note that the wind speeds are larger. For example, in ASCE 7-10 the wind speed for Queens, NY is 120 MPH while the wind speed at the same location in ASCE 7-05 is approximately 100 MPH.

BASIC LOAD COMBINATIONS		
	APPLICABLE LOAD TYPES	**LATERAL LOAD TYPES ONLY**
1	1.4D	–
2	1.2D + 1.0L + 0.5(L_r or S or R)	–
3	1.2D + 1.0(L_r or S or R) + (L or 0.5W)	0.5W
4	1.2D + 1.0W + L + 0.5(L_r or S or R)	1.0W
5	1.2D + 1.0E + L + 0.2S	1.0E
6	0.9D + 1.0W	1.0W
7	0.9D + 1.0E	1.0E
D = DEAD LOAD **E = EARTHQUAKE LOAD**	**L = LIVE LOAD** **L_r = ROOF LIVE LOAD** **R = RAIN LOAD** **S = SNOW LOAD**	**W = WIND LOAD**

TABLE 7: SUMMARY OF LOAD COMBINATIONS FROM ASCE 7-10

After analyzing wind and seismic loads it appears that Case 4 (1.2D+1.W+L+0.5S) controls in North/South direction and Case 5 (1.2D + 1.0E + L + 0.2S) controls in the East/West Direction. This is confirmed later in the *Lateral Movement* section of this technical report.

LATERAL ANALYSIS

ASSUMPTIONS

Before hand calculations or computer analysis could be performed assumptions have been made to simplify thesis calculations. The geometry of the building was altered slightly to be more rectilinearly shaped with dimensions as follows: the length of the building is 536 feet, the width of the building is 95 feet and the height from the ground to the tallest point is 150 feet. These dimensions are the same dimensions that were provided by Turner Construction; however certain architectural protrusions and indentations were neglected.

When the structural drawings were obtained from Turner Construction, the project was still in the design phase. The structural drawings provided were not complete and because of this various assumptions regarding member sizes have been made and simplified in order to complete thesis calculations and analyses. Typical beam, girder and column sizes were used in the analysis model for simplification.

ETABS MODEL

A model of the gravity and lateral framing systems was modeled in ETABS and analyzed. From this program the relative story drifts were obtained from the ETABS model which can be seen in Appendix E. The ETABS output is compared to the accepted allowable drift later in this report. A snapshot of the ETABS model can be seen in Figure 10 to the right. Notice only elements contributing to lateral systems were modeled.

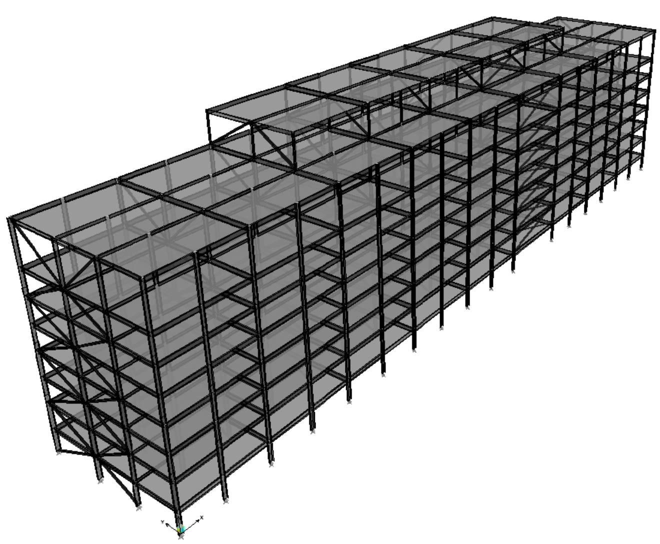


FIGURE 10: THIS IS AN IMAGE OF THE NEW YORK POLICE ACADEMY MODELED IN ETABS.

X-FRAME

As stated earlier in this report there are two lateral force resisting systems in the New York Police Academy. One system is demonstrated by the X-Frame in Figure 11 below and consists of moment connections throughout the building except where the HSS cross bracing can be seen. The HSS cross bracing is where the bridge from one part of the building to another part of the building is located.

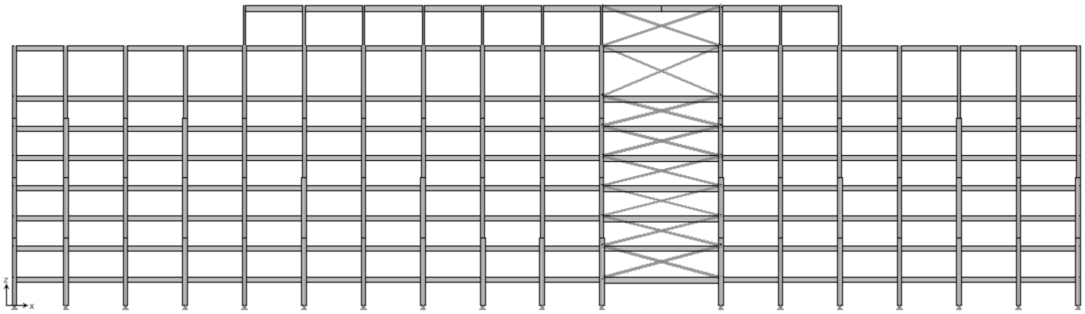


FIGURE 11: THIS IS AN IMAGE OF THE X-FRAME LATERAL RESISTING SYSTEM IN THE NEW YORK POLICE ACADEMY.

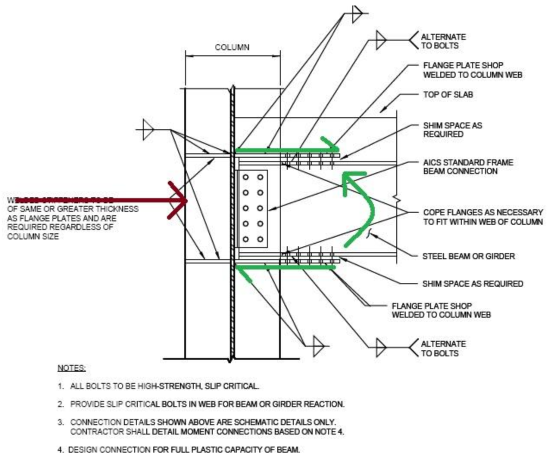


FIGURE 12: THIS IMAGE SHOWS THE LATERAL LOAD PATH THROUGH A MOMENT CONNECTION.

Figure 12 to the left shows the load path through an exterior moment connection. The red arrow indicates the exterior lateral load on the façade of the New York Police Academy while the green arrows show the loads within the connection. The top plate is in tension, while the bottom plate is in compression. This creates a moment on the connection, which is shown on the W-shaped member. This is what happens when lateral forces are applied to the X-Frame moment connections.

Y-FRAME

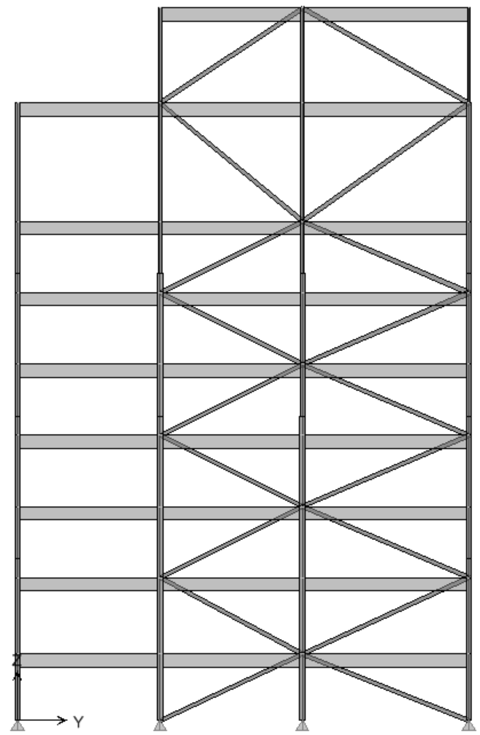


Figure 13: This is an image of the Y-frame lateral resisting system in the New York Police Academy.

The other lateral force resisting system is in the direction orthogonal to the X-frame and is referred to as the Y-frame. This frame has HSS cross bracing to resist lateral loads and all connections are pinned. This can be seen in Figure 13 to the left.

Figure 14 below shows the load path through an exterior HSS laterally braced connection. The red arrow indicates the exterior lateral load on the façade of the New York Police Academy while the green arrows show the loads within the members. The HSS brace at the top is in tension while the lower HSS brace and W shaped member are in compression. This is what happens when lateral forces are applied to the Y-Frame.

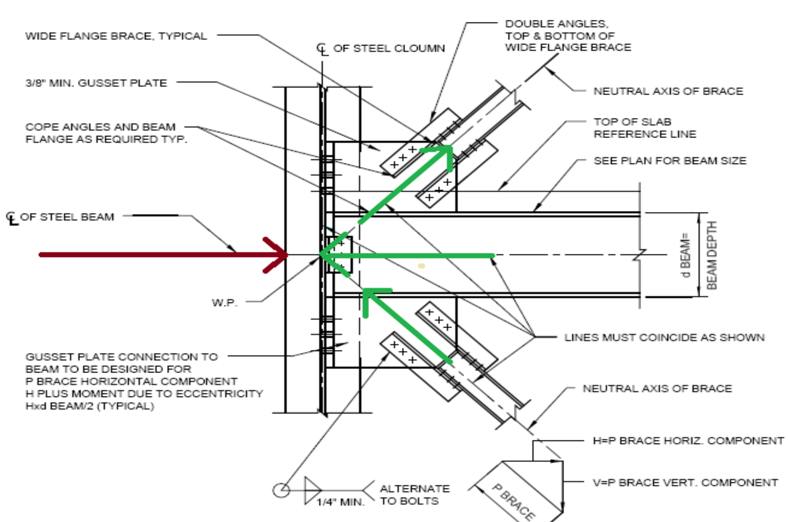


Figure 14: This image shows the lateral load path through an HSS cross braced connection.

OVERTURNING MOMENT

The critical overturning moment results in the direction with least depth. In this case it is 95 feet, the width of The New York Police Academy. This is the length of the lateral resisting Y-Frame. Wind loads control in this direction. The resisting moment is calculated by multiplying the weight of the building by the moment arm of half the width of the building. To stop the structure from overturning the resisting moment of the building must be greater than the moment that wind loads put on the building.

OVERTURNING MOMENTS			
FLOOR	WIND FORCE (K)	ELEVATION (IN)	MOMENT (IN-K)
ROOF	116.2	1800	209160
BULK HEAD	169.5	1560	264420
8	131.5	1260	165690
7	150.1	1080	162108
6	139	900	125100
5	127.9	720	92088
4	115.8	540	62532
3	100.7	360	36252
2	76.4	168	12835.2
		Σ	1,130,185.2/12 = **94,182.1'k**
		M_N = 53,905K x 95'/2 =	**2,560,487.5'k**

TABLE 8: THIS TABLE SHOWS THE MAGNITUDE OF THE MAXIMUM OVERTURNING MOMENT AND THE RESISTING MOMENT.

Though lightweight concrete is used in this structure, the building is rather heavy and the moment created by the wind is not near the magnitude of the resisting moment created by the dead load of the New York Police Academy. Calculations for overturning and resisting moments are in Table 8 above. Hand calculations for this can be seen in Appendix F.

DIRECT SHEAR

Direct shear is caused by lateral forces acting on a building and distributed to the lateral resisting system. Direct shear for each frame by story is calculated in Tables 9 and 10 below. This is calculated by multiplying the story force by the relative stiffness. This allows the engineer to know what force is being applied to what members throughout the building. The engineer can then check the strength of members accordingly. The strength checks can be seen on page 28.

DIRECT SHEAR – EAST/WEST – X-FRAMES (k)					
FLOOR	**FORCE**	**FRAME 1**	**FRAME 2**	**FRAME 3**	**FRAME 4**
BULKHEAD	15.3	4.9	2.8	2.8	4.9
ROOF	22.3	7.1	4.0	4.0	7.1
8	17.2	5.5	3.1	3.1	5.5
7	19.5	6.2	3.5	3.5	6.2
6	18.0	5.8	3.2	3.2	5.8
5	16.4	5.2	3.0	3.0	5.2
4	14.7	4.7	2.6	2.6	4.7
3	12.5	4.0	2.3	2.3	4.0
2	9.0	2.9	1.6	1.6	2.9
Σ	144.9	46.4	26.1	26.1	46.4

TABLE 9: THIS TABLE SHOWS THE DIRECT SHEAR IN EACH X-FRAME BY FLOOR.

DIRECT SHEAR – NORTH/SOUTH – Y-FRAMES (k)						
FLOOR	FORCE	FRAME A	FRAME C	FRAME E	FRAME G	FRAME I
BULKHEAD	66	6.6	6.6	6.6	6.6	6.6
ROOF	113	11.3	11.3	11.3	11.3	11.3
8	85	8.5	8.5	8.5	8.5	8.5
7	72	7.2	7.2	7.2	7.2	7.2
6	63	6.3	6.3	6.3	6.3	6.3
5	51	5.1	5.1	5.1	5.1	5.1
4	39	3.9	3.9	3.9	3.9	3.9
3	28	2.8	2.8	2.8	2.8	2.8
2	18	1.8	1.8	1.8	1.8	1.8
Σ	535.0	53.5	53.5	53.5	53.5	53.5
FLOOR	FORCE	FRAME K	FRAME M	FRAME O	FRAME Q	FRAME S
BULKHEAD	66	6.6	6.6	6.6	6.6	6.6
ROOF	113	11.3	11.3	11.3	11.3	11.3
8	85	8.5	8.5	8.5	8.5	8.5
7	72	7.2	7.2	7.2	7.2	7.2
6	63	6.3	6.3	6.3	6.3	6.3
5	51	5.1	5.1	5.1	5.1	5.1
4	39	3.9	3.9	3.9	3.9	3.9
3	28	2.8	2.8	2.8	2.8	2.8
2	18	1.8	1.8	1.8	1.8	1.8
Σ	535.0	53.5	53.5	53.5	53.5	53.5

TABLE 10: THIS TABLE SHOWS THE DIRECT SHEAR IN EACH Y-FRAME BY FLOOR.

TORSION

Lateral loads applied to a building will induce torsion when the centers of pressure or rigidity and the center of mass are not located at the same point. Seismic loads act on the center of rigidity of the structure while wind loads act at the center of pressure. If either the centers of pressure or rigidity are not equal with the center of mass then there will be a moment equal to the force multiplied by the eccentricity induced. The centers of mass, pressure and rigidity for the New York Police Academy are tabulated below.

CENTERS OF MASS, PRESSURE, AND RIGIDITY (FT)				
	X	Y	X-DIFFERENCE	Y-DIFFERENCE
CENTER OF MASS	267.9	48.5	-	-
CENTER OF PRESSURE	267.8	49	0.1	0.5
CENTER OF RIGIDITY	269	46.6	1.1	1.9

TABLE 11: THIS TABLE COMPARES THE CENTERS OF PRESSURE AND RIGIDITY TO THE CENTER OF MASS.

The centers of pressure and rigidity are very similar in location to the center of mass meaning that torsion does not have too large of an effect on the building as a whole. However, torsion must be considered and analyzed to ensure that its effects on the building are minimal. For the calculation of the centers of mass, pressure and rigidity please refer to Appendix D.

Table 12 on the following page shows the stiffness of the two lateral resisting frames. This was calculated by applying a unit load on each frame and recording the resulting displacement of each floor. Using the equation:

$$k = \frac{P}{\delta}$$

the stiffness, k, was calculated. Relative stiffness was calculated by using the equation:

$$Relative\ Stiffness = \frac{R}{\Sigma R}$$

These values are also shown in Table 12 on the next page for all stories.

FRAME STIFFNESSES						
	X-FRAME			Y-FRAME		
FLR	δ (IN)	STIFFNESS, k	% OF k	δ (IN)	STIFFNESS, k	% OF k
BULK HEAD	4.393	227.6	10%	8.105	123.4	25%
ROOF	3.484	287.0	10%	6.737	148.4	25%
8	2.667	375.0	10%	5.043	198.3	25%
7	2.284	437.8	10%	4.158	240.5	25%
6	1.928	518.7	10%	3.277	305.2	25%
5	1.595	627.0	10%	2.359	423.9	25%
4	1.294	772.8	10%	1.804	554.3	25%
3	0.986	1014.2	10%	1.167	856.9	25%
2	0.569	1757.5	10%	0.507	1972.4	25%

TABLE 12: THIS TABLE SHOWS THE STIFFNESS, DISPLACEMENT AND PERCENT STIFFNESS OF THE FRAMES IN THE X AND Y DIRECTIONS

TORSIONAL SHEAR

Torsional shear is calculated using the equation:

$$T = \frac{V_{tot} \cdot e \cdot d_i \cdot R_i}{J}$$

where V_{tot} = Story Shear

e = distance from the center of mass to the center of rigidity

d_i = distance from frame to the center of rigidity

R_i = relative stiffness of the frame

J = torsional moment of inertia $[\sum(R_i {d_i}^2)]$

The torsional shear for the 6th floor is calculated in Table 13 on the following page.

TORSIONAL SHEAR – 6TH FLOOR SAMPLE CALCULATION							
FRAME	DIRECTION	STORY SHEAR	RELATIVE STIFFNESS	DISTANCE FROM COM TO COR e (IN)	DISTANCE FROM FRAME TO COR d_i (IN)	$R_i d_i^2$	TORSIONAL SHEAR (k)
1	N/S	567.2	0.32	22.8	559.2	100065.5	0.518
2	N/S	567.2	0.18	22.8	199.2	7142.5	0.104
3	N/S	567.2	0.18	22.8	160.8	4654.2	0.084
4	N/S	567.2	0.32	22.8	580.8	107945.2	0.538
A	E/W	74.3	0.10	13.2	3228.0	1041998.4	0.071
C	E/W	74.3	0.10	13.2	2556.0	653313.6	0.056
E	E/W	74.3	0.10	13.2	1836.0	337089.6	0.040
G	E/W	74.3	0.10	13.2	1116.0	124545.6	0.025
I	E/W	74.3	0.10	13.2	396.0	15681.6	0.009
K	E/W	74.3	0.10	13.2	324.0	10497.6	0.007
M	E/W	74.3	0.10	13.2	1044.0	108993.6	0.023
O	E/W	74.3	0.10	13.2	1764.0	311169.6	0.039
Q	E/W	74.3	0.10	13.2	2484.0	617025.6	0.055
S	E/W	74.3	0.10	13.2	3204.0	1026561.6	0.070
					J=	4466684.16	

Table 13: This table shows the sample calculation for torsional shear for the 6th floor.

LATERAL MOVEMENT

Drift is a serviceability consideration that is taken into account during building design. Drift is inversely proportionate to rigidity. The lateral displacement in this report has been limited to $1/400$th of the building height for wind and $1/50$th of the building height of seismic considerations. As shown in Table 14 below the maximum wind drift for The New York Police Academy was 0.767" which is less than the allowable 4.5" and is therefore an acceptable wind drift. The maximum seismic drift for the New York Police Academy was 0.543" which was much less than the allowable 36" and is also an acceptable seismic drift.

X – FRAME STORY DRIFTS (INCHES)					Y – FRAME STORY DRIFTS (INCHES)			
FLR	1.0E ETABS	SEISMIC $\Delta_{ALLOWABLE}$	1.0W ETABS	WIND $\Delta_{ALLOWABLE}$	1.0E ETABS	SEISMIC $\Delta_{ALLOWABLE}$	1.0W ETABS	WIND $\Delta_{ALLOWABLE}$
BULK HEAD	0.043	4.8	0.020	0.6	0.159	4.8	-.307	0.6
ROOF	0.057	6	0.025	0.75	0.165	6	-.066	0.75
8	0.04	3.6	0.023	0.45	0.039	3.6	0.036	0.45
7	0.045	3.6	0.026	0.45	-0.054	3.6	0.042	0.45
6	0.05	3.6	0.030	0.45	0.004	3.6	0.107	0.45
5	0.052	3.6	0.033	0.45	-0.004	3.6	0.114	0.45
4	0.056	3.6	0.037	0.45	0.013	3.6	0.139	0.45
3	0.079	3.84	0.055	0.48	0.004	3.84	0.150	0.48
2	0.121	3.36	0.087	0.42	0.04	3.36	0.179	0.42
Σ	0.543	36	0.336	4.5	0.366	36	0.394	4.5
							Max = 0.767 @8th FLOOR	

TABLE 14: THIS TABLE SHOWS THE STORY DRIFT OF THE FRAME IN THE X- AND Y- DIRECTIONS OF THE NEW YORK POLICE ACADEMY. EMPHASIZED IN RED IS THE CONTROLLING DRIFT IN EACH DIRECTION.

STRENGTH CHECK

Columns and lateral bracing members were checked for strength when controlling wind and seismic loads were applied. The members checked were more than sufficient to support the given controlling loads. Calculations supporting this data can be viewed in Appendix G.

CONCLUSION

The New York Police Academy was modeled in ETABS to get a better idea of how the building behaved as a whole when gravity and lateral loads were applied. The controlling lateral loads induce an overturning moment that the gravity system must resist to prevent uplift. The controlling overturning moment on The New York Police Academy was the wind load in the North/South direction with a magnitude of 94,182.1 ft-k. The resisting moment was much greater than the overturning moment with a magnitude of 2,560,487.5 ft-k.

The direct shear calculations show how the lateral system frames resist the controlling lateral loads. The four X-Frames must resist the seismic loads in the East/West direction. The X-Frame predominantly uses moment connections to resist this load. The exterior frames receive 32% of the load while the interior frames receive 18% of the load. The ten Y-Frames must resist the wind loads in the North/South direction. The Y-Frame uses HSS cross bracing to resist this load. All frames receive about 10% of the load.

The weak links in the X-Frame is the double bay of HSS cross bracing. The cross bracing is HSS16x16x½ and spans just under 62 feet diagonally between floors. The weak link in the Y-Frame is the bar without HSS cross bracing. All connections in this frame are pinned and therefore the bay with no cross bracing is the weakest link when resisting lateral loads.

The centers of rigidity and pressure were very close to the center of mass which means that the effect of torsion on the New York Police Academy is very limited. This became even more apparent when the torsional shear calculations were performed. The largest amount of shear added to a lateral resisting frame due to torsion was 0.538 k, which was added to Frame A. This force is very small in magnitude and has little effect on the structure as a whole.

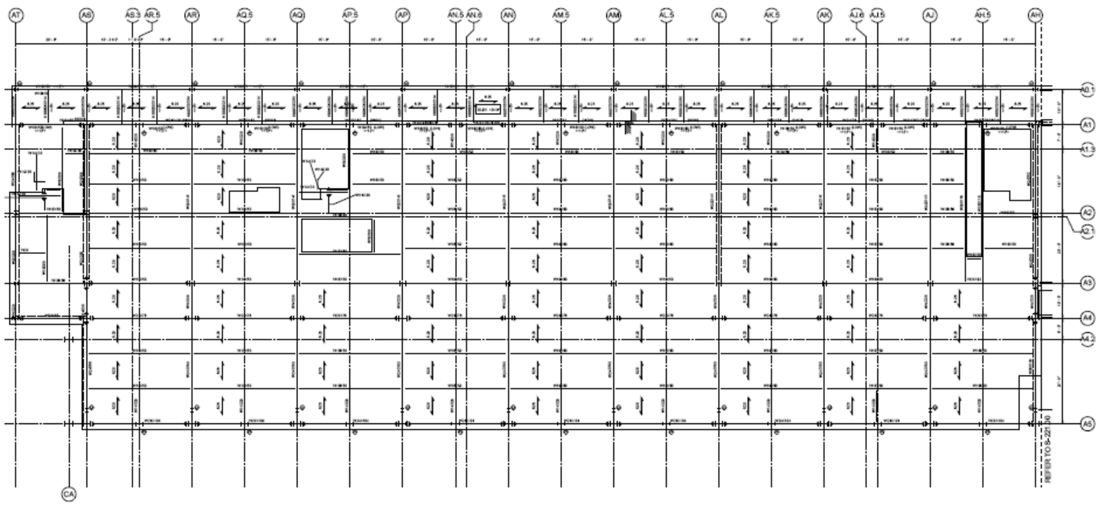
The maximum drifts and displacements in The New York Police Academy were minimal. The maximum story drift on the X- Frame was due to the controlling seismic load, which was 0.121". This is much less than the allowable 3.36" for that story. The maximum displacement due to seismic loads was 0.543", which was also much less than the allowable total displacement. The maximum story drift on the Y-Frame due to the controlling wind load was 0.179", which was less than the allowable 0.42". The maximum displacement due to wind loads was 0.767", which was less than the allowable total displacement of 4.5".

The strength of a W24x162 column and an HSS10x10x½ were checked to make sure that they could support the applied lateral loads. Both members were found to be sufficient in strength.

The lateral system of the New York Police Academy performed well when lateral loads were applied to the structure. Due to the analysis done in this report it is confirmed that the lateral resisting system in this building is sufficient under controlling site conditions.

APPENDIX A: FRAMING PLANS AND ELEVATIONS

FRAMING PLAN PART 1 (WEST END)



FRAMING PLAN PART 2 (EAST END)

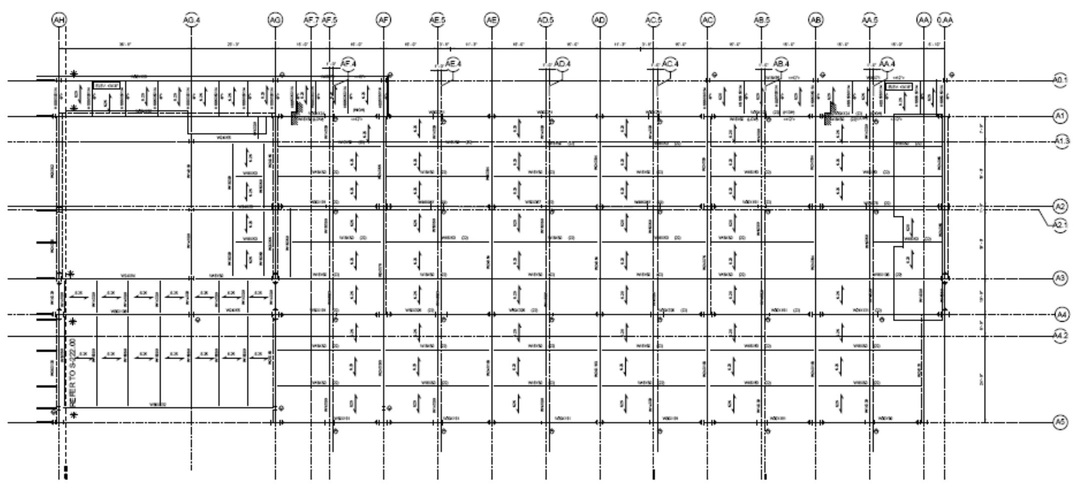
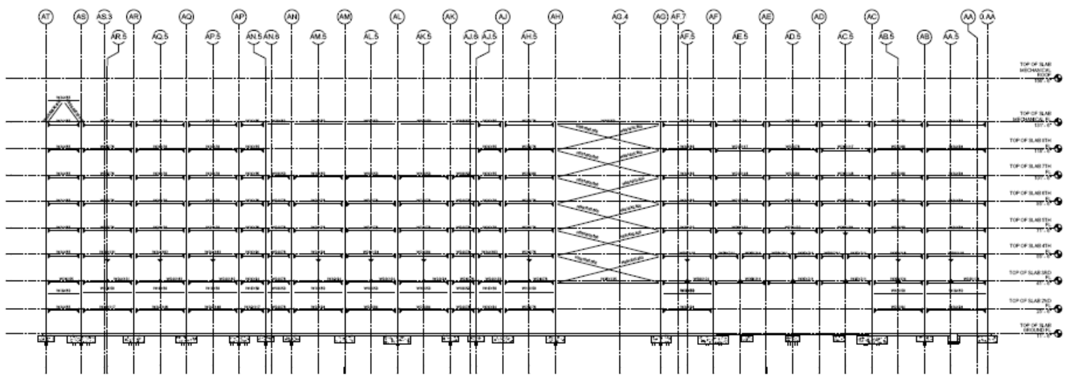
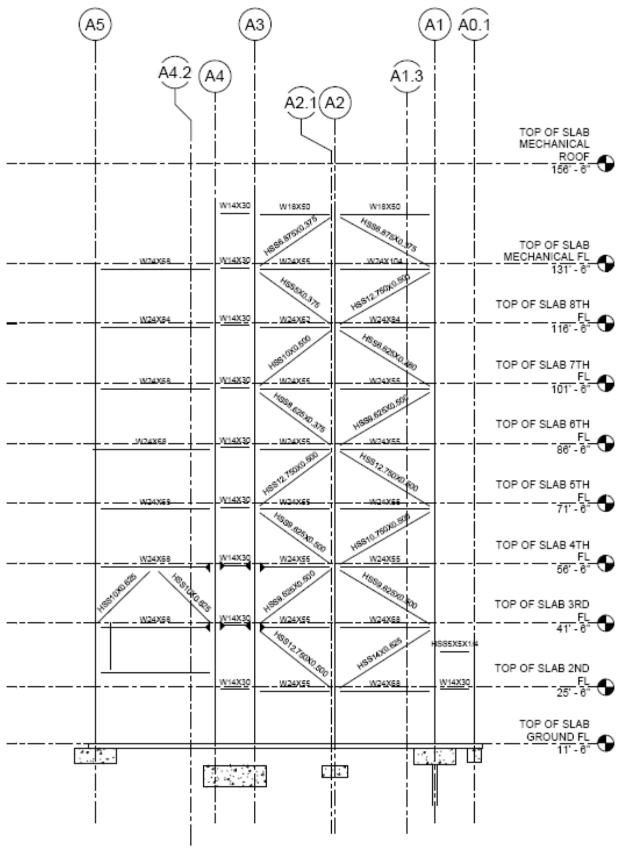


FIGURE 15: THIS IS THE TYPICAL FRAMING PLAN OF ONE FLOOR OF THE NEW YORK POLICE ACADEMY. PLEASE NOTE THAT THE BUILDING IS SO OBLONG THAT EACH FLOOR PLAN IS SPLIT INTO TWO SHEETS WITH PART 1 (THE WEST END) AND PART 2 (THE EAST END).



FRAMING ELEVATION AT GRIDLINE A1

FIGURE 16: ABOVE IS AN ELEVATION OF THE FRAMING SYSTEM LOOKING IN THE NORTH/SOUTH DIRECTION. NOTICE ONLY MOMENT CONNECTIONS EXCEPT FOR THE CROSS BRACING ON THE BRIDGE. BELOW IS AN ELEVATION OF THE FRAMING SYSTEM LOOKING IN THE EAST/WEST DIRECTION. NOTICE THE MAJORITY OF THE CROSS BRACING IN THIS DIRECTION COMPARED TO FEW MOMENT CONNECTIONS.



FRAMING ELEVATION LINE AS

SCALE: 1/16" = 1'-0"

APPENDIX B: WIND LOAD CALCULATIONS

NORTH/SOUTH WIND VARIABLE AND CALSSIFICATIONS					
BASIC WIND SPEED (V)	120	DAMPING RATIO (β)	2	G_f	1
WIND DIRECTIONALITY FACTOR (K_d)	0.85	NATURAL FREQUENCY (n_a)	0.53	q_z	34.78
IMPORTANCE FACTOR (I)	1	L/B	536/95	q_h	34.15
EXPOSURE CATEGORY	B	I_z	0.26	q_i	34.15
TOPOGRAPHIC FACTOR (K_{zt})	1	L_z	439	GC_{pi}	±0.18
α	7	Q	0.86	P_p (WINDWARD)	21.67
Z_g	1200	V_z	100	P_p (LEEWARD)	-13.11
a	1/7.0	N_1	2.32	C_p (WINDWARD)	0.8
b	0.84	R_n	0.08	C_p (LEEWARD)	-0.2
c	0.3	R_h	0.25	C_p (SIDE WALLS)	-0.7
l	320	R_b	0.34	MEAN ROOF HEIGHT (h)	142
EXPOSURE CATEGORY	1/3.0	R_L	0.02	ENCLSURE TYPE	FULLY ENCLOSED
Z_{min}	30	R	0.42	RIGIDITY	FLEXIBLE
α	1/4.0	g_r	4.04	TOPOGRAPHY	NO HILLS/ ESCARPMENTS

TABLE 15: THIS TABLE SHOWS THE VARIABLES AND CLASSIFICATIONS NECESSARY TO CALCULATE WIND PRESSURES IN THE NORTH/SOUTH DIRECTION.

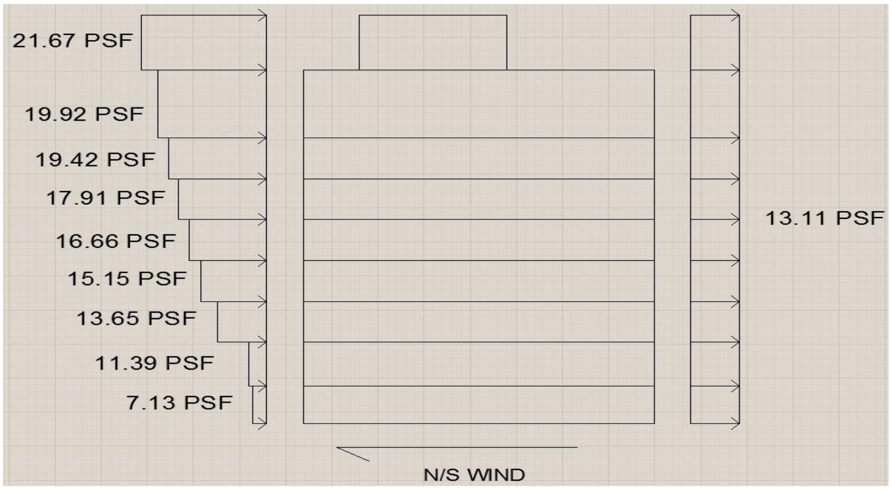


FIGURE 17: THIS FIGURE GRAPHICALLY SHOWS THE WIND PRESSURES ON THE BUILDING IN THE NORTH/SOUTH DIRECTION.

EAST/WEST WIND VARIABLE AND CALSSIFICATIONS					
BASIC WIND SPEED (V)	120	DAMPING RATIO (β)	2	G_f	0.8
WIND DIRECTIONALITY FACTOR (K_d)	0.85	NATURAL FREQUENCY (n_a)	0.43	q_z	34.78
IMPORTANCE FACTOR (I)	1	L/B	95/536	q_h	34.15
EXPOSURE CATEGORY	B	I_z	0.26	q_i	34.15
TOPOGRAPHIC FACTOR (K_{zt})	1	L_z	435	GC_{pi}	±0.18
α	7	Q	0.71	P_p (WINDWARD)	16.11
Z_g	1200	V_z	100	P_p (LEEWARD)	-20.06
a	1/7.0	N_1	1.87	C_p (WINDWARD)	0.8
b	0.84	R_n	0.09	C_p (LEEWARD)	-0.5
c	0.3	R_h	0.30	C_p (SIDE WALLS)	-0.7
l	320	R_b	0.09	MEAN ROOF HEIGHT (h)	138
EXPOSURE CATEGORY	1/3.0	R_L	015	ENCLSURE TYPE	FULLY ENCLOSED
Z_{min}	30	R	0.27	RIGIDITY	FLEXIBLE
α	1/4.0	g_r	3.98	TOPOGRAPHY	NO HILLS/ ESCARPMENTS

TABLE 16: THIS TABLE SHOWS THE VARIABLES AND CLASSIFICATIONS NECESSARY TO CALCULATE WIND PRESSURES IN THE EAST/WEST DIRECTION.

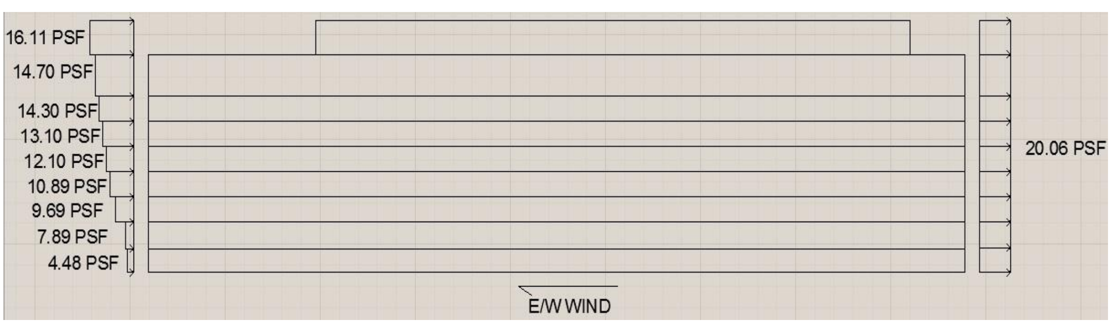


FIGURE 18: THIS FIGURE GRAPHICALLY SHOWS THE WIND PRESSURES ON THE BUILDING IN THE EAST/WEST DIRECTION.

NORTH / SOUTH WIND

USE ASCE 7-10 - MWFRS (DIRECTIONAL PROCEDURE)

TABLE 27.2-1

$\boxed{1}$ RISK CATEGORY:

INSTITUTION / ACADEMY $\Rightarrow$ $\boxed{II}$ (TABLE 1.5-1)

$\boxed{2}$ BASIC WIND SPEED (TABLE 26.5-1A)

LOCATED IN QUEENS, NY $\Rightarrow$ $\boxed{V = 120 \text{ MPH}}$

$\boxed{3}$ WIND DIRECTIONALITY FACTOR (§ 26.6, TABLE 26.6-1)

MWFRS $\Rightarrow$ $\boxed{K_d = 0.85}$

EXPOSURE CATEGORY (§ 26.7)

URBAN AREA $\Rightarrow$ $\boxed{\text{EXPOSURE B}}$

TOPOGRAPHIC FACTOR (§ 26.8, TABLE 26.8-1)

NO HILLS / ESCARPMENTS $\Rightarrow$ $\boxed{K_{zt} = 1.0}$

GUST EFFECT FACTOR (§ 26.9)

LIMITATIONS FOR APPROXIMATE NATURAL FREQUENCY (§ 26.9.2.1)

$\boxed{1}$ BUILDING HEIGHT = $\cancel{150}$ 150' < 300' ∴ OK

$\boxed{2}$

$$L_{EFF} = \frac{\sum_{i=1}^{8} h_i L_i}{\sum_{i=1}^{8} h_i} = \frac{\left[130(536) + 20(390)\right]}{150}$$

$= 508'$

$= 4 \times 508 > 150$ ∴ OK

APPROXIMATE NATURAL FREQUENCY

STEEL W/ HSS LATERAL BRACING ~~& MOMENT CONNECTIONS~~

$$n_a = 75/h \quad \left\{ \begin{bmatrix} 390(132) \\ + \\ (536-390)(150) \end{bmatrix} \right/ 536 = \boxed{142 = h} = \text{MEAN ROOF HEIGHT}$$

$\boxed{n_a = 0.53 \text{ Hz}} < 1$ ∴ FLEXIBLE

NORTH / SOUTH WIND

3 CONTINUED

GUST EFFECT FACTOR

FLEXIBLE BUILDINGS (26.9.5)

$$G_f = 0.925 \left[\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}} \right]$$

$$I_{\bar{z}} = \overset{\text{TABLE 26.9-1}}{c} \left(\frac{33}{\underset{0.6h}{\bar{z}}} \right)^{1/6} = 0.3 \left(\frac{33}{85.2} \right)^{1/6} = \boxed{0.26 = I_{\bar{z}}}$$

$$Q = \left[\frac{1}{1 + 0.63 \left(\frac{B+h}{L_{\bar{z}}} \right)} \right]^{-1/2} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{95+142}{439} \right)}} = \boxed{0.86 = Q}$$

$$L_{\bar{z}} = \underbrace{\ell \left(\frac{\bar{z}}{33} \right)^{\bar{\epsilon}}}_{\text{TABLE 26.9-1}} = 320 \left(\frac{85.2}{33} \right)^{1/3} = \boxed{439 = L_{\bar{z}}}$$

B = HORIZ. DIM. NORMAL TO WIND $\boxed{95' = B}$

$\boxed{g_Q \,\&\, g_v = 3.4}$ (§ 26.9.5, EQ 26.9-11)

PLUG INTO Q

$$g_R = \sqrt{2 \ln (3600 n_1)} + \frac{0.577}{\sqrt{2 \ln (3600 n_1)}} = \boxed{4.04 = g_R}$$

$n_1 = n_a$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(2.32)}{(1 + 10.3(2.32))^{5/3}} = \boxed{0.08 = R_n}$$

PLUG INTO R

$$N_1 = \frac{n_1 L_{\bar{z}}}{V_{\bar{z}}} = \frac{0.53 \times 439}{100.39} = \boxed{2.32 = N_1}$$

PLUG INTO R_n

JAKE POLLACK TECH REPORT #3 WIND ANALYSIS 3/8

NORTH / SOUTH WIND

3 CONTINUED

GUST EFFECT FACTOR

TABLE 26.9-1

$$\bar{V}_{\bar{z}} = \bar{l}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V$$

$$= 0.45\left(\frac{85.2}{33}\right)^{1/4}\left(\frac{88}{60}\right)120 = \boxed{100.39 \text{ FT/S} = \bar{V}_{\bar{z}}}$$

PLUG INTO N.

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{2.07} - \frac{1}{2(2.07^2)}(1-e^{-2\eta}) = \boxed{0.25 = R_h}$$

PLUG INTO R

$$\eta_h = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.53)(142)}{100.39} = \boxed{3.45\ \eta_h}$$

PLUG INTO R_h

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \boxed{0.34\ R_B}$$

PLUG INTO R

$$\eta_B = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \boxed{2.31 = \eta_B}$$

PLUG INTO R_B

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \boxed{0.02 = R_L}$$

PLUG INTO R

$$\eta_L = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \boxed{43.58 = \eta_L}$$

PLUG INTO R_L

$$R = \sqrt{\frac{1}{0.02}(0.08)(0.25)(0.34)(0.53 + 0.47(0.02))} = \boxed{0.42 = R}$$

PLUG INTO G_f

$$G_f = 0.925\left[\frac{1 + 1.7(0.26)\sqrt{3.4^2 0.86^2 + 4.04^2 0.52^2}}{1 + 1.7(3.4)(0.26)}\right] = 1.00 = G_f$$

4 VELOCITY PRESSURE EXPOSURE COEFFICIENT (TABLE 27.3-1)

BY LINEAR INTERPOLATION $\boxed{K_z = 1.11}$

FOR MEAN ROOF HEIGHT $\boxed{K_z = 1.09}$

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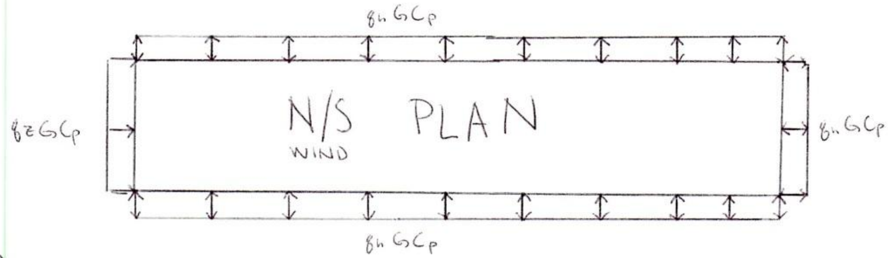
5 VELOCITY PRESSURE (EQ. 27.3-1)

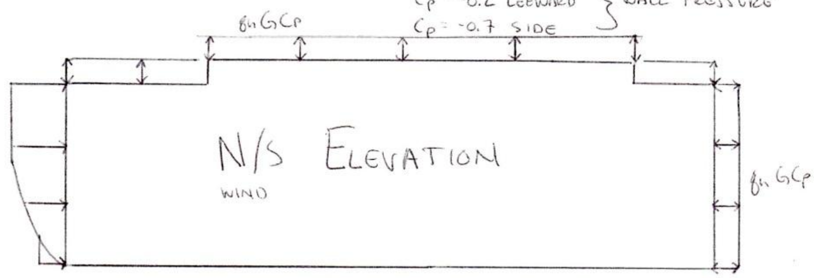
$q_z = 0.00256\,(K_z K_{zt} K_d V^2) = 0.00256\,(1.11)(1.0)(0.85)(120^2)$

$\boxed{q_z = 34.78\ \text{PSF}}$ $\boxed{q_h = 34.15\ \text{PSF}}$

6 EXTERNAL PRESSURE COEFFICIENT (FIG. 27.4-1 FOR FLAT ROOFS)



$L/B = 536/95 = 5.64 > 4 \Rightarrow$ $C_p = 0.8$ WINDWARD, $C_p = -0.2$ LEEWARD, $C_p = -0.7$ SIDE } WALL PRESSURE



NORTH / SOUTH WIND

$h/L = 142/536 = 0.26 \le 0.5$ FROM 0' TO 71' $C_p = -0.9, -0.18$
FROM 71' TO 142' $C_p = -0.9, -0.18$
FROM 142' TO 150' $C_p = -0.5, -0.18$

AREA = B×L = 95 × 536 = 50,920 SF ⇒ REDUCTION FACTOR = 0.9
HOWEVER, DOES NOT APPLY

7 WIND PRESSURE

WINDWARD WALL:

$p = q\,G_f C_p - q_i\,(GC_{pi})$ (§27.4.2)

$p = (34.78)(1.00)(0.8) - (34.15)(\pm 0.18) = 27.82 - 6.15\ \text{PSF} = \boxed{21.67\ \text{PSF}}$

7 CONTINUED

LEEWARD WALL:

$p = 34.78(1.00)(-0.2) - (34.15)(\pm 0.18) = -6.96 \pm 6.15 \text{ PSF} = \boxed{-13.11 \text{ PSF}}$

SIDE WALLS:

$p = 34.78(1.00)(-0.7) - (34.15)(\pm 0.18) = -23.37 \pm 6.15 \text{ PSF} = \boxed{-30.50 \text{ PSF}}$

ROOF:

0 TO 71':
71' TO 142':
$p = 34.78(1.00)(-0.9) - 34.15(\pm 0.18) = -31.30 + 6.15 = \boxed{-25.15 \text{ PSF}}$

142' TO 150': $p = 34.78(1.00)(-0.5) - 34.15(\pm 0.18) = -16.69 + 6.15 = \boxed{-10.54 \text{ PSF}}$

8 FORCE OF WINDWARD PRESSURE

$\frac{L}{1000 \text{lbs}} \left[\frac{h_{FLa}}{2} (\text{WIND PRESSURE}_a) + \frac{h_{FLb}}{2} (\text{WIND PRESSURE}_b) \right]$

NORTH / SOUTH WIND

USE ASCE 7-10 - MWFRS (DIRECTIONAL PROCEDURE)

NOTE: STEPS 1-3 ARE THE SAME UP TO GUST EFFECT FACTOR (§26.9)

PROCEDURE WILL START FROM HERE.

EAST / WEST WIND

[3] GUST EFFECT FACTOR

[1] BLDG HEIGHT OK

[2] $L_{EFF} = 88' \times 4 > 150 \therefore$ OK

APPROXIMATE NATURAL FREQUENCY

STEEL MOMENT CONNECTIONS

$n_a = 0.43 < 1 \therefore$ FLEXIBLE

FIND G_f

$Q = 0.71$

$I_{\bar{z}} = 0.26$, $L_{\bar{z}} = 435$, $B = 536'$, $g_Q, g_v = 3.4$

$g_R = 3.98$

$n_1 = n_a = 0.43$

$R = 0.27$

$R_n = 0.08$

$N_1 = 2.32$, $\bar{V}_{\bar{z}} = 100$ FT/S

$R_h = 0.30$

$\eta_h = 2.73$

$R_B = 0.09$

$\eta_B = 10.60$

$R_L = 0.15$

$\eta_L = 6.29$

$\boxed{G_f = 0.80}$

4 VELOCITY EXPOSURE COEFFICIENT (TABLE 27.3-1)

$\boxed{K_z = 1.11}$ $\boxed{K_z = 1.05}$

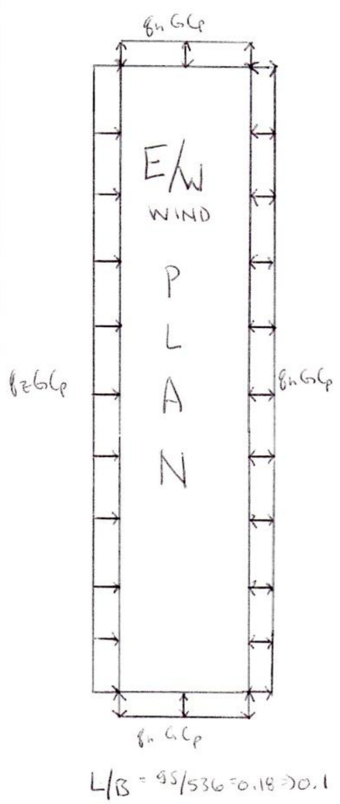
5 VELOCITY PRESSURE (EQ. 27.3-1)

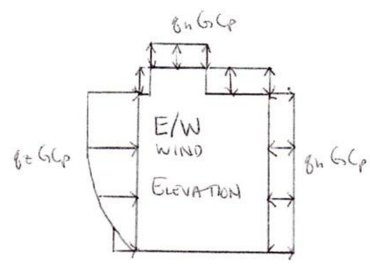
$q_z = 34.78$ PSF

$q_h = 34.15$ PSF

6 EXTERNAL PRESSURE COEFFICIENT (FIG 27.4-1 FOR FLAT ROOFS)

EAST / WEST WIND





$L/B = 95/536 = 0.18 \Rightarrow 0.1$

WINDWARD = 0.8
LEEWARD = -0.5
SIDE = -0.7

$h/L = 138/95 = 1.45 > 1.0$

FROM 0' TO 69' $C_p = -1.3$**, -0.18
FROM 69' TO 150' $C_p = -0.7$, -0.18

$B \times L > 1000$ SF => REDUCTION FACTOR = 0.8
DOES APPLY **

JAKE POLLACK TECH REPORT #3 WIND ANALYSIS 8/8

7 WIND PRESSURE

WINDWARD WALL:

$$p = (34.78)(0.8)(0.8) - (34.15)(\pm 0.18) = 22.26 - 6.15 = \boxed{16.11 \text{ PSF}}$$

LEEWARD WALL:

$$p = (34.78)(0.8)(-0.5) - (34.15)(\pm 0.18) = -13.91 \pm 6.15 = \boxed{-20.06 \text{ PSF}}$$

SIDE WALLS:

$$p = (34.78)(0.8)(-0.7) - (34.15)(\pm 0.18) = -19.48 \pm 6.15 = \boxed{-25.63 \text{ PSF}}$$

ROOF:

$$0 \text{ TO } 69': \ p = 34.78(0.8)(-1.3)(0.8) - (34.15)(\pm 0.18) = -28.94 + 6.15 = \boxed{-22.79 \text{ PSF}}$$

$$69' \text{ TO } 150': \ p = 34.78(0.8)(-0.7) - (34.15)(\pm 0.18) = -19.48 + 6.15 = \boxed{-13.33 \text{ PSF}}$$

EAST / WEST WIND

APPENDIX C: SEISMIC LOAD CALCULATIONS

SEISMIC VARIABLE			ASCE 7-10 REFERENCE
S_S	35.6%g		USGS WEBSITE
S_1	7.00%g		USGS WEBSITE
SITE CLASSIFICATION	B		TABLE 20.3-1
F_A	1.0		TABLE 11.4-1
F_V	1.0		TABLE 11.4-2
S_{MS}	0.356		EQ 11.4-1
S_{M1}	0.070		EQ 11.4-2
S_{DS}	0.237		EQ 11.4-3
S_{D1}	0.047		EQ 11.4-4
OCCUPANCY CATEGORY	II		TABLE 1-1
I	1.00		TABLE 1.5-2
SEISMIC DESIGN CATEGORY	B		TABLE 11.6-1
EQUIVALENT LATERAL FORCE PROVEDURE PERMITTED BY (TABLE 12.6-1)			
	NORTH/SOUTH DIRECTION	EAST/WEST DIRECTION	
T_L	6 s	6 s	FIGURE 22-12
C_t	0.020	0.028	TABLE 12.8-2
x	0.75	0.80	TABLE 12.8-2
T_a	0.857 s	1.542 s	SECTION 12.8.2.1
C_u	1.7	1.7	TABLE 12.8-1
T_b	0.7763 s	1.1101 s	ETABS
C_uT_a	1.46 s	2.62 s	SECTION 12.8.2.1
R	6	3.5	TABLE 12.2-1
C_s	0.010	0.012	EQ 12.8-5
W	53905 K	53905 K	SEE SPREADSHEET
V	539 K	647 K	SEE SPREADSHEET

k	1.14	1.31	SECTION 12.8.3

TABLE 17: THIS TABLE SHOWS THE VARIABLES AND CLASSIFICATIONS NECESSARY TO CALCULATE SEISMIC FORCES.

JAKE POLLACK | TECH REPORT #3 | SEISMIC ANALYSIS | 1/3

USE ASCE 7-10 SEISMIC ANALYSIS FOR BUILDING STRUCTURES

ADDRESS: 130-30 28TH AVENUE, COLLEGE POINT, NY

SITE CLASS: B (TABLE 20.3-1)

OCCUPANCY CATEGORY: II (TABLE 1-1)

LATITUDE: 40.784088
LONGITUDE: -73.845924

S_s = 35.6% g (@ 0.2 SEC)
S_1 = 7.0% g (@ 1.0 SEC)

FROM USGS WEBSITE

$F_a = 1.0$ (TABLE 11.4-1)

$F_v = 1.0$ (TABLE 11.4-2)

$S_{MS} = F_a S_s = 1.0(0.356) = 0.356$ (EQ 11.4-1)

$S_{M1} = F_v S_1 = 1.0(0.070) = 0.070$ (EQ 11.4-2)

IMPORTANCE FACTOR: $I = 1.00$ (TABLE 1.5-2)

SEISMIC DESIGN CATEGORY: B (TABLE 11.6-1)
A (TABLE 11.6-2)

MAPPED LONG-PERIOD TRANSITION PERIOD: $T_L = 6s$ (FIGURE 22-12)

VALUES OF APPROXIMATE PERIOD PARAMETERS:

SYSTEM RESISTING IN N/S DIRECTION
= STEEL CONCENTRICALLY BRACED FRAMES
$C_t = 0.02$ $\quad x = 0.75$ (TABLE 12.8-2)

SYSTEM RESISTING IN E/W DIRECTION
= STEEL MOMENT-RESISTING FRAMES
$C_t = 0.028$ $\quad x = 0.8$ (TABLE 12.8-2)

	N/S DIRECTION	E/W DIRECTION
$T_a = C_t h_n^x$ (§ 12.8.2.1)	$T_a = 0.02(150)^{0.75}$	$T_a = 0.028(150)^{0.8}$
	$T_a = 0.857s$	$T_a = 1.542s$
$T = C_u T_a$ (TABLE 12.8-1)	$T = 1.7(0.857)$	$T = 1.7(1.542)$
	$T = 1.46s$	$T = 2.62s$
$T = \min \begin{cases} C_u T_a \\ T_b \end{cases}$	$T_b = 0.7763s$	$T_b = 1.1101s$

NOTE: T_b OBTAINED FROM ETABS MODEL

R-VALUE: (TABLE 12.2-1)	$R = 6$	$R = 3.5$

EQUIVALENT LATERAL FORCE PROCEDURE

$C_s =$		N/S	E/W
$\frac{S_{D1}}{T(R/I_e)}$	FOR $T \leq T_L$ (EQ. 12.8-3)	$\frac{0.047}{(0.7763)(6/1.0)} = 0.01$	$\frac{0.047}{1.1101(3.5/1.0)} = 0.012$
$\frac{S_{DS}}{(R/I_e)}$	(EQ. 12.8-2)	$\frac{0.237}{6/1.0} = 0.04$	$\frac{0.237}{3.5/1.0} = 0.068$
$0.044 S_{DS} I_e \geq 0.01$		$0.044(0.237)(1) = 0.01$	$= 0.01$

USE $C_s = 0.01$

SEISMIC BASE SHEAR

$V = C_s W$ (EQ 12.8-1)

W = SEISMIC WEIGHT → CALCULATED IN SPREADSHEET

$V = 0.01(53{,}905^k) = 539^k$

VERTICAL DISTRIBUTION OF SEISMIC FORCES

$F_x = C_{vx} V$ (EQ. 12.8-11)

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \text{(EQ 12.8-12)}$$

$k = 1$ FOR $0.5s$ & $k = 2$ FOR $T = 2.5s$

INTERPOLATE:

N/S DIRECTION	E/W DIRECTION
$T = 0.7763$	$T = 1.1101$
$\Rightarrow \left(\frac{2.5 - 0.7763}{2.5 - 0.5}\right)$	$\Rightarrow \left(\frac{2.5 - 1.1101}{2.5 - 0.5}\right)$
$k = 1.14$	$k = 1.31$

SEE SPREADSHEET FOR FORCES, SHEARS, MOMENTS

APPENDIX D: TORSION CALCULATIONS

CENTER OF MASS CALCULATIONS

FLR	m_i	x_i	y_i	$\Sigma m_i x_i$	$\Sigma m_i y_i$
BULKHEAD	3322	266	62.5	883652	207625
ROOF	6753	268	47.5	1809804	320767.5
8	5574	268	47.5	1493832	264765
7	5574	268	47.5	1493832	264765
6	5847	268	47.5	1566996	277732.5
5	5847	268	47.5	1566996	277732.5
4	5847	268	47.5	1566996	277732.5
3	5920	268	47.5	1586560	281200
2	5920	268	47.5	1586560	281200
Σ	50604			13555228	2453520
		$\frac{\Sigma mx}{m}$	$\frac{\Sigma my}{m}$		
		x = 267.9'	y = 48.5'		

TABLE 18: THIS TABLE SHOWS THE CALCULATION OF THE X AND Y COORDINATES FOR THE CENTER OF MASS.

CENTER OF PRESSURE CALCULATIONS

FLR	STORY HEIGHT	TOTAL HEIGHT	X	Y
BULKHEAD	20	150	266	62.5
ROOF	10	130	268	47.5
8	15	120	268	47.5
7	15	105	268	47.5
6	15	90	268	47.5
5	15	75	268	47.5
4	15	60	268	47.5
3	15	45	268	47.5

2	16	30	268	47.5
1	14	14	268	47.5
			x = 267.8'	y = 49'

Table 19: This table shows the calculation of the X and Y coordinates for the center of pressure.

CENTER OF RIGIDITY CALCULATIONS

X-FRAMES					
FRAME	LOAD APPLIED	DISTRIBUTION	PERCENTAGE	DISTANCE TO ORIGIN	%·DISTANCE
1	1000	320	32%	0	0
2	1000	180	18%	30	5.4
3	1000	180	18%	60	10.8
4	1000	320	32%	95	30.4
					y = 46.6'

Table 20: This table shows the calculation of the Y coordinate for the center of rigidity.

Y-FRAMES					
FRAME	LOAD APPLIED	DISTRIBUTION	PERCENTAGE	DISTANCE TO ORIGIN	%·DISTANCE
A	1000	98.2	10%	0	0
C	1000	98.6	10%	56	5.5
E	1000	99	10%	116	11.5
G	1000	99.5	10%	176	17.5
I	1000	99.9	10%	236	23.6
K	1000	100.4	10%	296	29.7
M	1000	100.8	10%	356	35.9
O	1000	101.3	10%	416	42.1
Q	1000	101.7	10%	476	48.4
S	1000	102.2	10%	536	54.8
					x = 269.0'

Table 21: This table shows the calculation of the X coordinate for the center of rigidity.

APPENDIX E: LATERAL MOVEMENT CALCULATIONS

Wind: $\Delta_{ALLOWABLE} = (l' \times 12'')/400$

Seismic: $\Delta_{ALLOWABLE} = 0.02h_{sx}$

APPENDIX F: OVERTURNING MOMENT CALCULATIONS

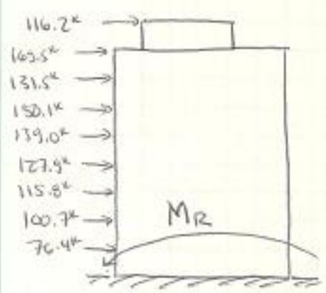
JAKE POLLACK	TECH REPORT #3	OVERTURNING MOMENT	1/1

RESISTING MOMENT (M_R) = BUILDING WEIGHT x MOMENT ARM

USE $\frac{\text{WIDTH OF BLDG}}{2}$

$M_R = 53,905^k \times \frac{95'}{2} = \underline{2,560,487.5^{'k}}$

CONTROLLING LOAD CASE = N/S WIND



OVERTURNING MOMENT (M_O) =

$150'(116.2^k) + 130'(169.5^k) + 90'(150.1^k) + 75'(139^k) + 60'(127.9^k)$

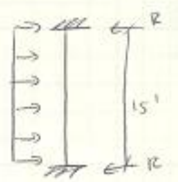
$+ 45'(115.8^k) + 30'(100.7^k) + 14'(76.4^k)$

$= \underline{94,182.1^{'k} << M_R \therefore \text{OKAY}}$

APPENDIX G: STRENGTH CHECKS

Jake Pollack | Tech Report #3 | Strength Check | 1/2

Column in X-Frame @ 6th Floor ⇒ W24x62



$W_{WIND} = 1.0(17.91)(30') = 537.3$ PLF

↑ ASCE 7-10 ↑ TRIB WIDTH

$R = \frac{wl}{2} = \frac{537.3(15)}{2} = 4.03^k$

$M_u = \frac{wl^2}{24} = \frac{537.3(15^2)}{24} = 5.04^{'k}$

$\phi M_{n_{W24x62}} = 1614^{'k} > 5.04^{'k} \therefore OK$ (AISC TABLE 3-10)

$\Delta_{CENTER} = \frac{wl^4}{384EI} = \frac{537.3(15^4)(1728)}{384(29000)(443)} = 0.0095''$

$\Delta_{ALLOW} = \frac{l}{400} = 0.45'' > 0.0095'' \therefore OK$

SEISMIC

$P_{10E} = \frac{76+87}{2} = 81.5^k$ @ CENTER

$R = P/2 = 40.75^k$

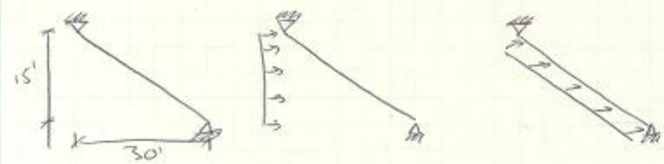
$M_u = \frac{Pl}{8} = \frac{40.75^k(15')}{8} = 76.41^{'k} < \phi M_n \therefore OK$

$\Delta_{CENTER} = \frac{Pl^3}{192EI} = \frac{40.75^k(15')^3(1728)}{192(29000)(443)} = 0.0964'' < \Delta_{ALLOW} \therefore OK$

Jake Pollack | Tech Report #3 | Strength Check | 2/2

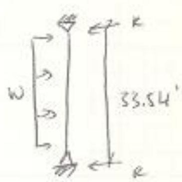
DIAGONAL BRACE IN Y-FRAME @ 6TH FLOOR

$\Rightarrow$ HSS $10 \times 10 \times 1/2$



$W_{WIND} = 1.0(17.91\ PSF)(15/30) = 8.96\ PSF$

$\Rightarrow W_{WIND} = 8.96^{PSF}(30') = 268.8\ PLF$

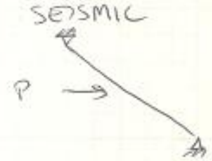


$R = \frac{w\ell}{2} = \frac{268.8(33.54)}{2} = 4.51^k$

$M_u = \frac{w\ell^2}{8} = \frac{268.8(33.54)^2}{8} = 37.8^{'k}$

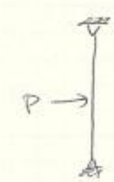
$\phi M_{n\ HSS\ 10\times10\times1/2} = 210^{'k} > 37.8^{'k}$ ∴ OK

SEISMIC



$P_{10E} = \frac{63+72}{2} = 67.5^k$

$P = 67.5^k(15/30) = 33.75^k$



$R = P/2 = 33.75^k/2 = 16.88^k$

$M_u = \frac{P\ell}{4} = \frac{16.88(33.54)}{4} = 141.50^{'k} < \phi M_n$ ∴ OK