New York Police Academy

Architectural Engineering Senior Thesis 2010

Technical Report I
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AE 481W

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EXECUTIVE SUMMARY

The structural concepts and existing structural conditions report describes the physical conditions for the structure and relative design concepts of the New York Police Academy. All of the structural elements were examined so that an overview could be presented on how each component works with one another.

Existing drawings, specifications and geotechnical conditions were provided by Turner Construction, the general contractor on the project. These items were compared to the applicable codes and standards. Calculations for typical conditions are included to clarify the thesis design analysis that was performed. In the event that direct design information was not present, an educated assumption was made based on previous knowledge and consultant clarification.

Calculations were performed according to ASCE 7-10 and IBC 2006 to obtain gravity and lateral loads. The loads included in this analysis are dead, live, snow, seismic and wind loads. These calculations are compared to those of Robert Silman Associates, the structural engineer of record on the project, who used design codes ASCE 7-98 and the BCNYC 2008. Thesis calculations provided that wind loads in the North/South direction controlled over loads in the East/West direction, producing a greater base shear. This is due to the oblong dimensions of the building. Because the building facing the East/West direction has a greater surface area, greater base shear is produced. The wind speed value used by Robert Silman Associates to calculate wind pressure and forces was almost 20% lower than the wind speed used in this report. This creates a large difference in calculations between those done by the structural engineer of record and those shown in this report.

Seismic analyses were performed in both directions as well because there is a difference in lateral bracing systems based on building orientation. The seismic loads calculated in this report were very similar to those calculated by the engineer of record despite the fact that the edition of codes used to obtain seismic loads differed. The seismic loads controlled base shear in the East/West direction, however the wind controlled base shear in the North/South direction. This is more likely due to the large surface area of the façade facing the North and South directions.

Upon completion of these analyses, spot checks were performed to verify the validity of gravity loads on the structure. Spot check calculations may differ because these members were checked in isolation as the engineer of record analyzed these members using a computer analysis program. This could yield different results because computer analysis programs incorporate how members interact together while the spot checks performed in this report refer to sole member loading.

INTRODUCTION

The New York Police Academy will serve as a consolidated recruit training facility at this one location. Prior to this project recruit training was spread throughout different facilities in the greater New York City area. This building is located in College Point, which is a neighborhood in Queens, New York City. The site that this building lies on was originally submerged under water, but with the aid of New York City garbage a landfill was created and compacted so that it can support large buildings such as this one. As seen in Figure 1, the site is quite large. The building is located just south of the MTA Bus Service Station and just north of the Full Gospel Christian School.

FIGURE 1: SITE PLAN OF NEW YORK POLICE ACADEMY (SHOWN IN BLUE). SATELLITE PHOTO COURTESY OF GOOGLE MAPS.

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.

ARCHITECTURAL OVERVIEW

This 8-story 1,000,000 SF structure is used as an academy to train New York Police Department recruits. The building was designed for LEED Silver Certification as designated by the United States Green Building Council (USGBC). This is accomplished by using numerous tactics to minimize its carbon footprint. Certain features encourage environmentally friendly means of commuting. This building also utilizes green roofs among various other strategies to create a healthier environment.

FIGURE 2: THIS IMAGE SHOWS THE GLAZED ALUMINUM CURTAINWALLS WITH ALUMINUM PANELING. THIS RENDERING IS COURTESY OF TURNER CONSTRUCTION.

The façade of this building is embellished with glazed curtain walls and shimmering aluminum paneling. The aluminum panels act as louvers above the windows both to shade and channel natural light into the building (See Figure 2).

STRUCTURAL 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 utilized 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, but was predominantly gray silty clay with sand. 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 sample pile cap. Concrete piers, walls, structural slabs on grade, pile caps and grade beams are placed monolithically. Pile caps are 16" in diameter.

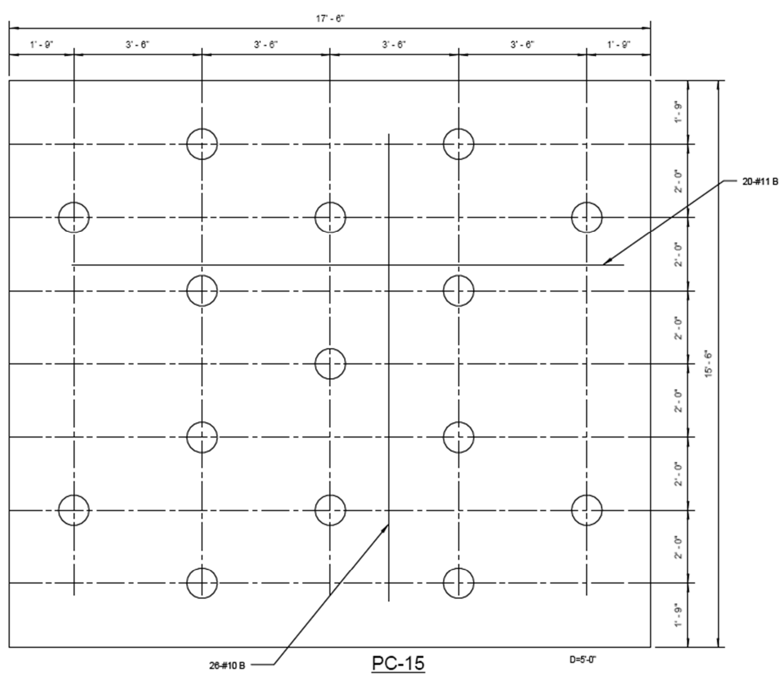


FIGURE 3: THIS IS PLAN OF A SAMPLE PILE CAP. DETAIL COURTESY OF TURNER CONSTRUCTION.

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.

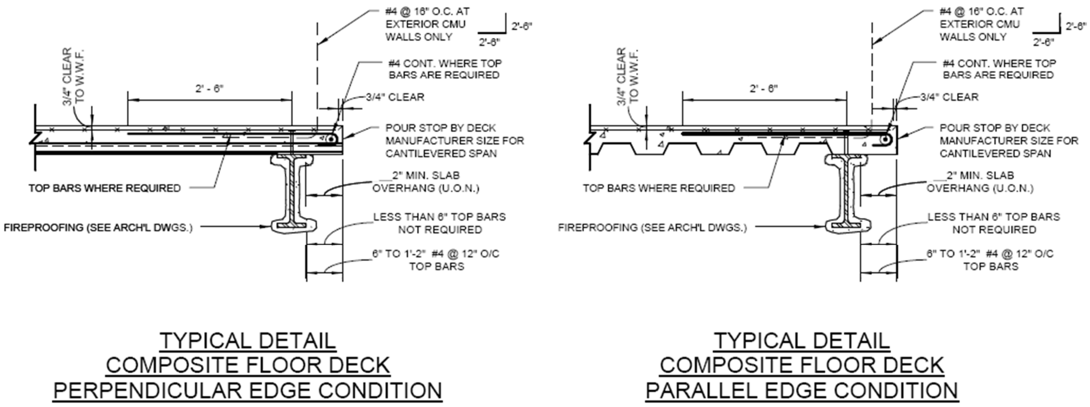


Figure 4: Typical slab on deck floor sections. Drawings not to scale. Detail courtesy of Turner Construction.

FRAMING SYSTEM

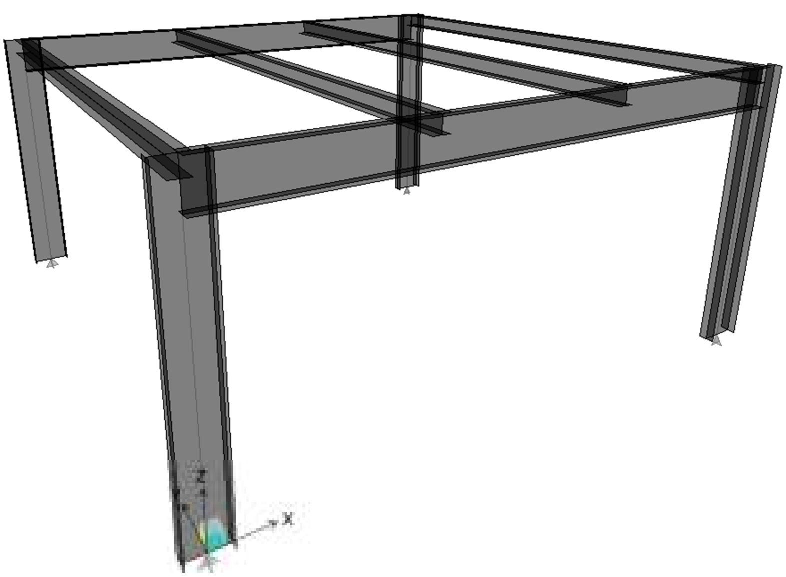


Figure 5: This is an ETABS model of the typical bay framing.

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 consists of steel moment connections in addition to lateral HSS and wide flange bracing (see Figure 6). Lateral HSS bracing is found predominantly in the North/South direction to oppose seismic and wind forces. The HSS bracing ranges in size from HSS 6.625x0.375 to 16x0.625. The HSS bracing in the East/West direction is solely used in the bridge to connect two parts of the campus.

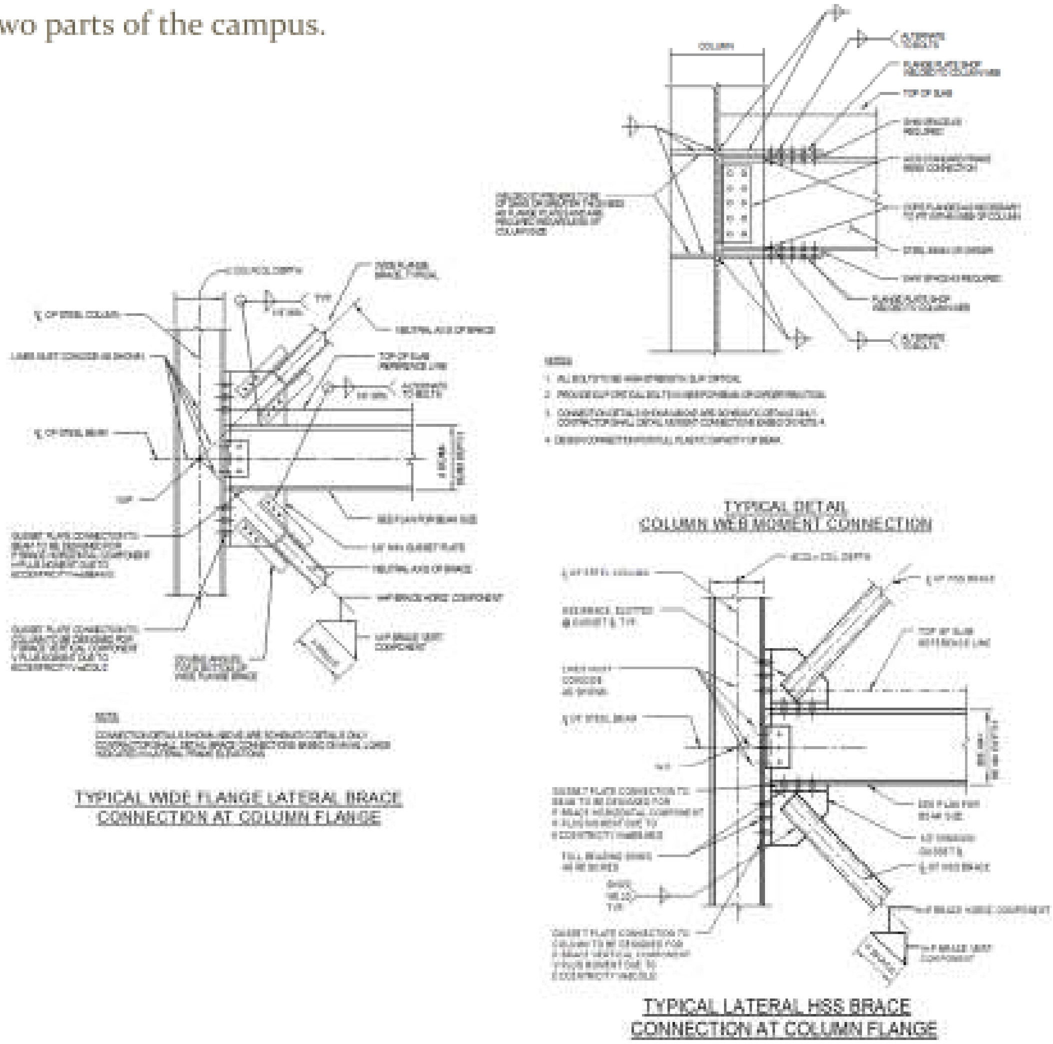


FIGURE 6: TYPICAL COLUMN WEB MOMENT CONNECTION (TOP RIGHT). TYPICAL LATERAL HSS BRACE CONNECTION (BOTTOM RIGHT). TYPICAL WIDE FLANGE LATERAL BRACE CONNECTION (LEFT). ALL DRAWINGS ARE NOT TO SCALE. DETAILS COURTESY OF TURNER CONSTRUCTION.

DESIGN 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, 13th 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, 13th Edition

Model Codes:

- 2006 International Building Code (IBC)

Structural Codes:

- American Society of Civil Engineers (ASCE) 7-08, Minimum Design Loads for Building and Other Structures

DESIGN CRITERIA

DEFLECTION

Horizontal Framing:

- Live Load
  - $< \frac{L}{600}$
- Total Load Excluding Self Weight
  - $< \frac{L}{480}$

Lateral Drift:

- Wind Loads
  - $< \frac{L}{400}$
- Seismic Loads
  - $< \frac{L}{76}$

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
  - $< \frac{L}{240}$

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

Note: Material strengths are based on American Society for Testing and Materials (ASTM) standard rating.

DESIGN LOADS

DEAD AND LIVE 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	20 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 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	100 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	40 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.

DESIGN ANALYSES AND CONCLUSIONS

WIND LOAD ANALYSIS

In order to perform wind load calculations the assumption that the façade and geometry of the New York Police Academy was entirely regular with no protrusions. Figures 7 and 9 below illustrate the geometry analyzed in this assumption. It is also assumed that there are no channeling effects or buffeting in the wake of upwind obstructions. Table 3 outlines variables and classifications needed to perform wind load calculations in the North/South direction. Table 4 displays the calculations and results in this direction as Figures 7 and 8 illustrate these effects.

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)**	33.97
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 3: THIS TABLE SHOWS THE VARIABLES AND CLASSIFICATIONS NECESSARY TO CALCULATE WIND PRESSURES IN THE NORTH/SOUTH DIRECTION.

NORTH/SOUTH 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)
			WIND-WARD	LEE-WARD				
BULK-HEAD	20	150	33.97	-13.11	47.08	196.8	0.0	5095.66
ROOF	10	120	32.22	-13.11	45.33	213.9	196.8	3865.99
8	15	105	31.72	-13.11	44.83	249.0	410.7	3330.10
7	15	90	30.21	-13.11	43.32	237.9	659.7	2719.03
6	15	75	28.96	-13.11	42.07	226.8	897.6	2171.87
5	15	60	27.45	-13.11	40.56	214.7	1124.4	1647.26
4	15	45	25.95	-13.11	39.06	199.6	1339.1	1167.78
3	15	30	23.69	-13.11	36.8	178.5	1538.7	710.84
2	16	14	19.43	-13.11	32.54	83.3	1717.2	272.07
G	14	0	-	-	0	0.0	1800.5	0.00
						∑	**1800.5 K**	**5095.66 FT-K**

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

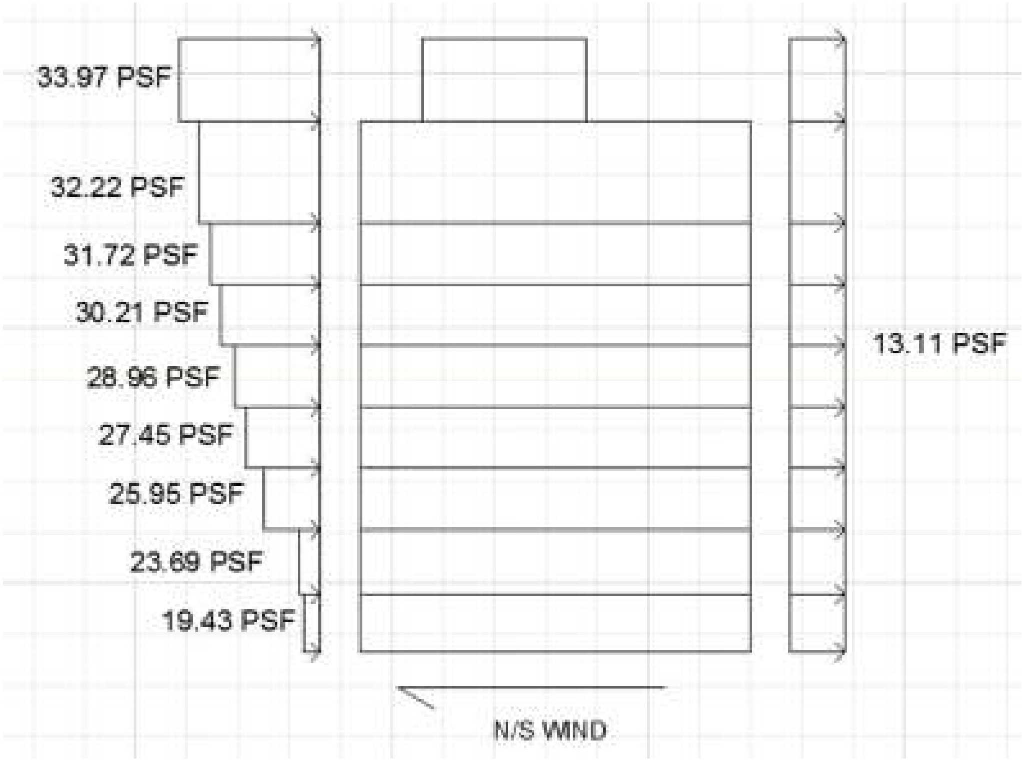


FIGURE 7: THIS FIGURE GRAPHICALLY SHOWS THE WIND PRESSURES ON THE BUILDING IN THE NORTH/SOUTH DIRECTION.

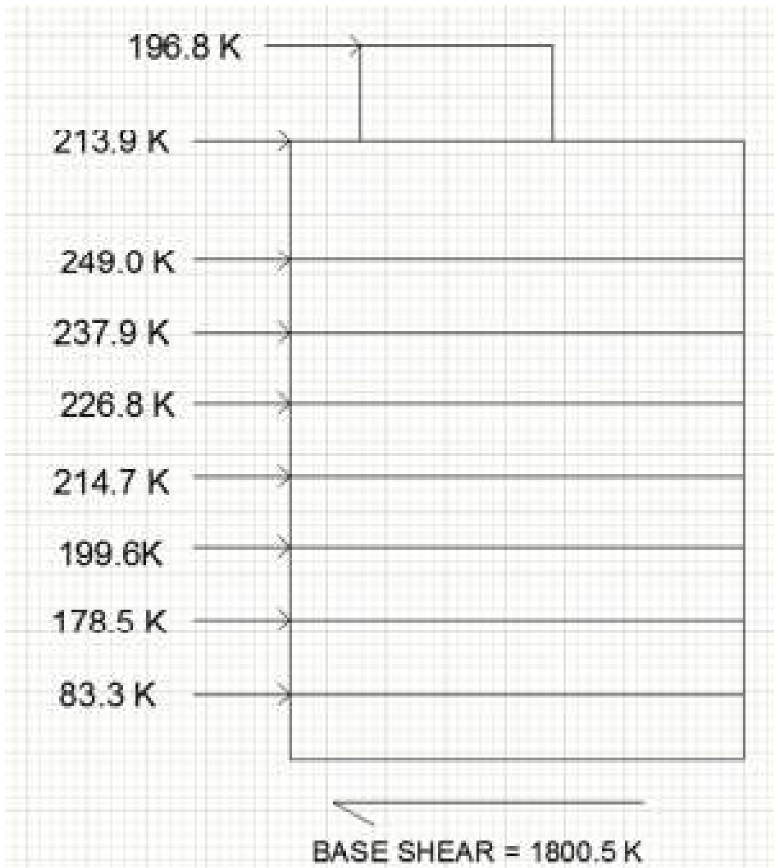


Figure 8: This figure graphically shows the wind shear force on each story in the North/South direction.

Table 5 outlines variables and classifications needed to perform wind load calculations in the East/West direction. Table 6 displays the calculations and results in this direction as Figures 9 and 10 illustrate these effects.

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)	28.41
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 5: THIS TABLE SHOWS THE VARIABLES AND CLASSIFICATIONS NECESSARY TO CALCULATE WIND PRESSURES IN THE EAST/WEST 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)
			WIND WARD	LEE WARD				
BULK-HEAD	20	150	28.41	-20.06	48.47	39.8	0.0	4261.02
ROOF	10	120	27.00	-20.06	47.06	31.8	39.8	3240.39
8	15	105	26.60	-20.06	46.66	37.1	71.6	2793.23
7	15	90	25.40	-20.06	45.46	35.5	108.6	2285.92
6	15	75	24.40	-20.06	44.46	33.9	144.1	1829.74
5	15	60	23.19	-20.06	43.25	32.2	178.0	1391.61
4	15	45	21.99	-20.06	42.05	30.1	210.2	989.57
3	15	30	20.19	-20.06	40.25	27.1	240.3	605.58
2	16	14	16.78	-20.06	36.84	12.8	267.4	234.88
G	14	0	-	-	0.00	0.0	280.2	0.00
						∑	**280.2 K**	**4261.02 FT-K**

TABLE 6: THE TABLE ABOVE SHOWS THE FLOOR WIND PRESSURES AND FORCES ALONG WITH SHEAR/MOMENT FORCES IN THE EAST/WEST DIRECTION.

FIGURE 9: THIS FIGURE GRAPHICALLY SHOWS THE WIND PRESSURES ON THE BUILDING IN THE EAST/WEST DIRECTION.

FIGURE 10: THIS FIGURE GRAPHICALLY SHOWS THE WIND SHEAR FORCE ON EACH STORY IN THE EAST/WEST DIRECTION.

WIND LOAD CONCLUSIONS

The wind loads calculated by the structural engineer of record were performed using ASCE 7-98. There is a large difference in wind speed used in the original design and the wind speed used in this report. The wind speed for Queens, New York in ASCE 7-98 was 98 mph while the wind speed used in this report and in ASCE 7-10 is 120 mph. The use of ASCE 7-10 creates a more conservative approach to wind calculations and creates a larger base shear. The wind pressures were greater in the East/West direction, but the base shear in the

North/South direction controlled because the surface area in which the wind contacts the building in this direction is significantly larger. Differences in surface area can be seen by comparing Figures 8 and 10. For a more in depth look at the calculations please look at Appendix B.

SEISMIC LOAD ANALYSIS

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. These calculations are to be checked on a computer analysis program in a following report. Table 7 outlines variables and classifications needed to perform seismic load calculations in both North/South and East/West directions. Table 8 displays the calculations and results in the North/South direction as Figure 11 illustrates these effects. For a more in depth review of calculations please refer to Appendix C.

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	1.46 s	1.542 s	SECTION 12.8.2.1
R	7	8	TABLE 12.2-1
C_s	0.01	0.01	EQ 12.8-5
W	53905 K	53905 K	SEE SPREADSHEET
V	539 K	502 K	SEE SPREADSHEET
k	1.18	1.52	SECTION 12.8.3

TABLE 7: THIS TABLE SHOWS THE VARIABLES AND CLASSIFICATIONS NECESSARY TO CALCULATE SEISMIC FORCES.

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	1,227,969	0.122	66	50	1,761
ROOF	6,753	130	2,108,385	0.209	113	163	3,024
8	5,574	120	1,583,437	0.157	85	248	2,271
7	5,574	105	1,352,603	0.134	72	320	1,940
6	5,847	90	1,182,876	0.117	63	383	1,696
5	5,847	75	953,906	0.095	51	434	1,368
4	5,847	60	733,080	0.073	39	473	1,051
3	5,920	45	528,582	0.052	28	502	758
2	5,920	30	327,586	0.033	18	519	470
G	3,301	14	74,315	0.007	4	523	107
TOTAL	53,905		10,072,739		539		14,445

TABLE 8: THIS TABLE SHOWS THE CACULATIONS AND PROCESSES NEEDED IN ORDER TO CALCULATE SEISMIC BASE SHEAR IN THE NORTH/SOUTH DIRECTION

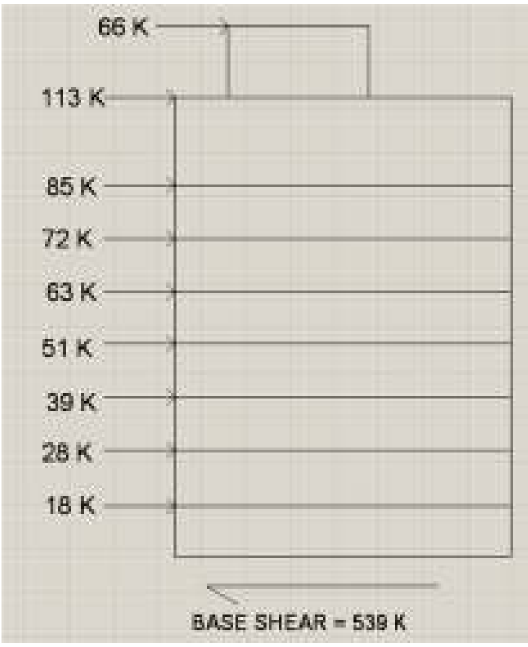


FIGURE 11: THIS FIGURE GRAPHICALLY SHOWS THE SEISMIC SHEAR FORCE ON EACH STORY IN THE NORTH/SOUTH DIRECTION.

Table 9 displays the seismic calculations and results in the East/West direction as Figure 12 illustrates these effects as shown below.

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	1,227,969	0.122	61	41	291
ROOF	6,753	130	2,108,385	0.209	105	146	499
8	5,574	120	1,583,437	0.157	79	225	375
7	5,574	105	1,352,603	0.134	67	292	320
6	5,847	90	1,182,876	0.117	59	351	280
5	5,847	75	953,906	0.095	48	399	226
4	5,847	60	733,080	0.073	37	435	174
3	5,920	45	528,582	0.052	26	462	125
2	5,920	30	327,586	0.033	16	478	78
G	3,301	14	74,315	0.007	4	482	18
TOTAL	53,905		10,072,739		502		2,385

TABLE 9: THIS TABLE SHOWS THE CACULATIONS AND PROCESSES NEEDED IN ORDER TO CALCULATE SEISMIC BASE SHEAR IN THE NORTH/SOUTH DIRECTION

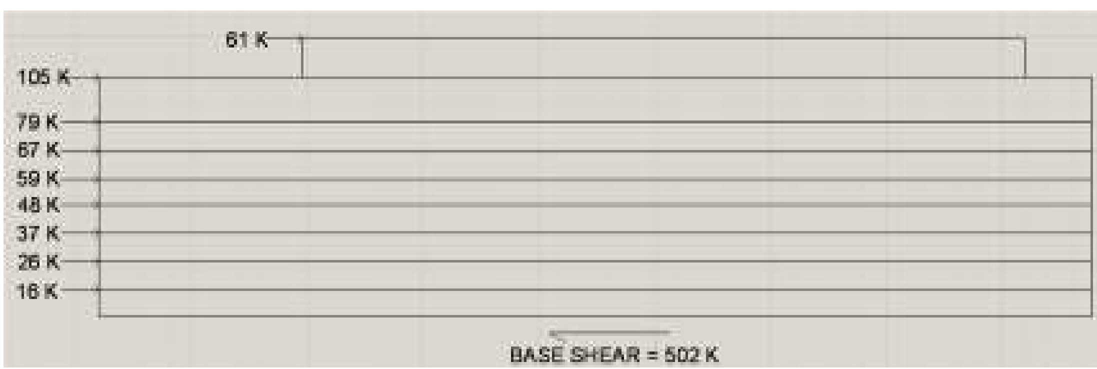


FIGURE 12: THIS FIGURE GRAPHICALLY SHOWS THE SEISMIC SHEAR FORCE ON EACH STORY IN THE EAST/WEST DIRECTION.

SEISMIC LOAD CONCLUSIONS

Seismic loads calculated above were similar to those calculated by Robert Silman Associates despite differences in ASCE code edition. One reason that the results may vary slightly is because of the square footage approximations made to simplify the analysis. The engineer of record did not make these assumptions. Another possible source of error is that the calculations above were done by hand while the engineer of record used a computer analysis program. Note that if the seismic period of vibration is shorter (in time) in a computer analysis model than it is in hand calculations then the period calculated by the analysis program must be used. The shorter the duration of the seismic period of vibration the more severe the seismic loading and thus the more conservative an analysis it would yield.

SNOW LOAD ANALYSIS AND DISCUSSION

Snow loads were calculated using various charts and tables from ASCE 7-10. Table 10 displays the snow loads and variables between designer loads and thesis loads. For more information and calculations please see Appendix D.

SNOW LOADS		
DESCRIPTION	DESIGNER LOADS	CALCULATED LOADS
P_g	20 PSF	20 PSF
I_s	1.0	1.0
C_e	1.0	1.0
C_t	1.0	1.0
P_f	22 PSF	22 PSF
P_{DRIFT}	--	64 PSF

TABLE 10: THIS TABLE COMPARES THE SNOW LOADS BETWEEN THE DESIGNER AND LOADS USED IN THIS THESIS REPORT.

Designer loads and calculated loads are the same. Due to bulkhead on the roof of the building snow drift needed to be computed. The structural engineer did not label snow drift on any drawings. Because the bulkhead extends 25 feet above the rooftop the weight of snow drift is rather high. In column spot checks below there is excess axial compressive force that can be used. Snow drift may be the reason for this larger design load.

GRAVITY SYSTEM SPOT CHECKS

TYPICAL SLAB ON METAL DECK

Each of the upper floors in the New York Police Academy utilized a 3.25" lightweight concrete slab on the 3" – 18 gage metal decking. Typical loads were applied to this system and calculations found that this slab is sufficient in strength; however it would need shoring during construction because the 3-span limit was breached. Figure 13 illustrates a section of the concrete slab on metal deck.

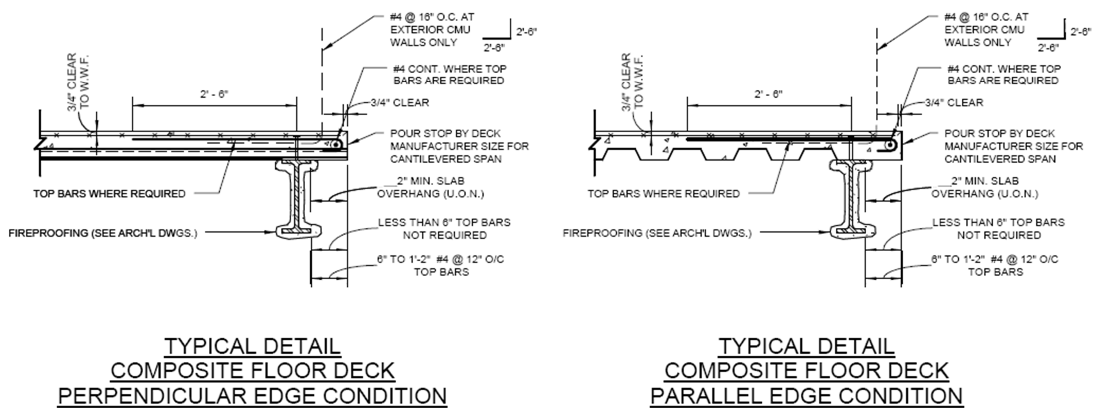


FIGURE 13: THIS FIGURE SHOWS THE TYPICAL SLAB/DECK ON FRAMING MEMBERS.

TYPICAL COMPOSITE BEAM AND GIRDER

Based on composite beam and girder typical spot check calculations the designer on this project was conservative when choosing framing member sizes. The calculations were performed using an office as the live load because this building is an academy that houses many offices and classrooms. To be conservative the office live load was utilized in these calculations. Office loads however, are not as extreme when compared to atrium, lobby and library loads. This could be a reason for the difference in member sizing. It is also more efficient for fabrication procedures for the designer to choose one typical member and use it throughout the building rather than sizing each specific area. The latter method is quite inefficient and leaves very little room for changing the function of a space in the future. A large amount of shear studs are needed to ensure the strength of the shear connection between the slab and the large members used. Figure 12 illustrates how typical framing members interact with typical floor systems.

TYPICAL COLUMN

The column analyzed extended from the ground floor to the roof and was spliced just above floors 3, 5 and 7. The column analyzed was at ground level so it would be carrying the greatest load. This column was an interior W14x145 and was located at gridline intersection A3-AQ. This column supports classrooms and offices throughout the building, but in order to be more conservative the live load of office space was used because it is larger in magnitude. The unbraced length was assumed to be the floor-to-floor height. It was also assumed that the column was pinned at the top and bottom. To be consistent with typical beam and girder spot checks the live load reduction was neglected in this calculation. As stated, this column was designed to be a W14x145. It is designed to carry an axial load of 1,690 kips at 14’ (the floor height of the first floor). This column had greater axial capacity than the W14x132 that was found to be sufficient in the calculations in Appendix E. This conservative approach could be due to the expectation of snow drift at the roof of the building or an anticipated change in function of the space it supports.

For all spot check calculations please refer to Appendix E.

FINAL SUMMARY AND CONCLUSIONS

Although different editions for ASCE 7 were utilized by the designer and this report the majority of the loads were very similar. Other load discrepancies were due to the difference in use of The Building Code of New York City and The International Building Code.

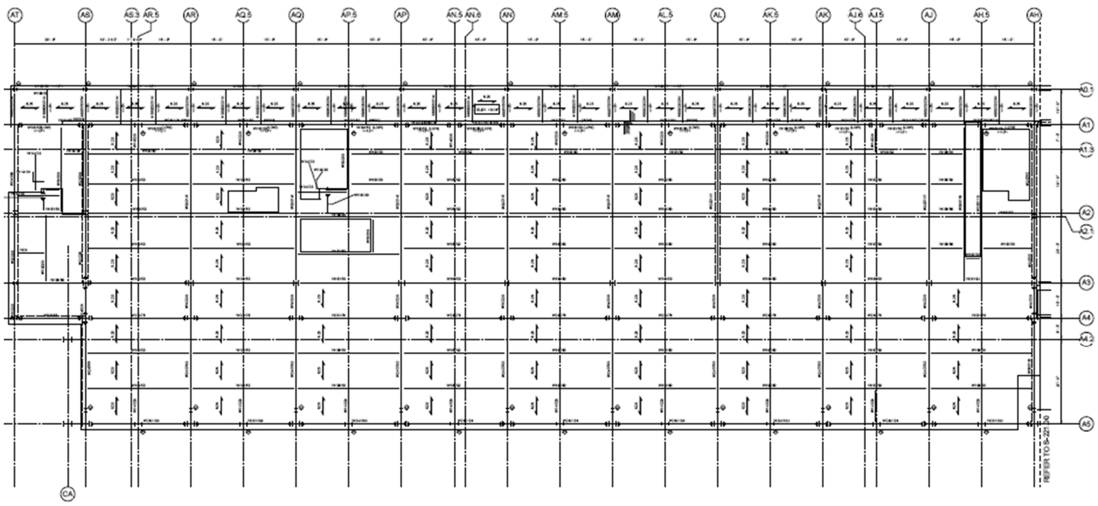
Wind was found to control in the North/South direction although wind pressures were more severe in the East/West direction. This is due to the large surface area of the building that faces the North/South winds. This load was more severe than the designer's load because the editions of ASCE used were different. The change in wind speed in the area is approximately a 20% increase.

Seismic loads controlled the lateral bracing system in the East/West direction because it was stronger than the wind in this direction. The seismic calculations were similar to those done by the designer despite the difference in codes used. Lateral systems varied based on direction. The lateral system resisting wind was predominantly HSS cross bracing, while the lateral system resisting seismic loading was primarily moment connections.

When performing spot checks it was found that the slab and deck system chosen by designer and this report were the same. Typical framing members and columns however were deemed to be conservative. This may be due to foresight in a change of function for certain areas. Another possible reason is the use of computer analysis programs. These programs analyze a structure as a whole while spot checks analyze each member individually.

APPENDIX A: FRAMING PLANS AND ELEVATIONS

FRAMING PLAN PART 1 (WEST END)



FRAMING PLAN PART 2 (EAST END)

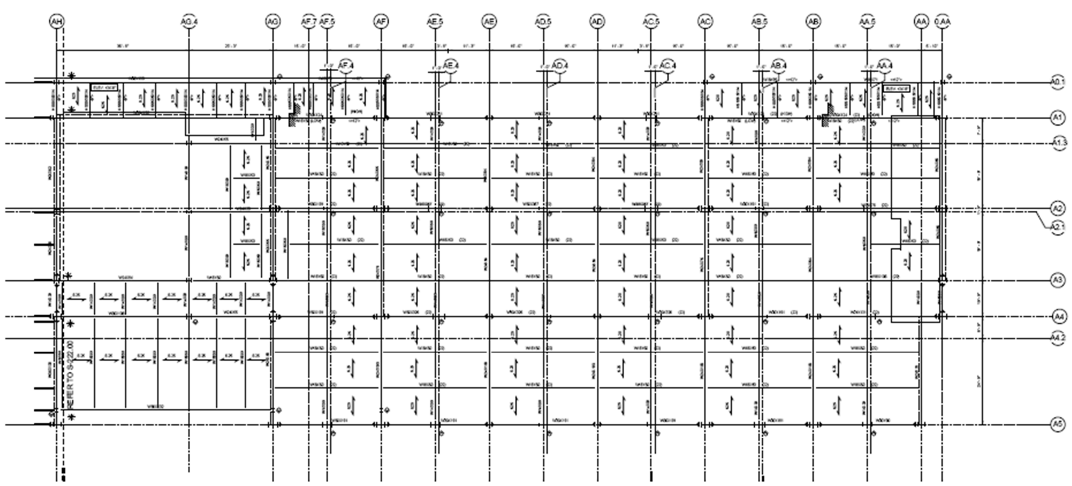


FIGURE 14: THIS IS THE TYPICAL FRAMING PLAN OF ONE FLOOR OF THE NEW YORK POLICE ACADEMY. PLEASE NOTE THAT THE BUILDING IS SO OBLONG THAT EACH FLOOR PLAN IS SPLIT INTO TWO SHEETS WITH PART 1 (THE WEST END) AND PART 2 (THE EAST END).

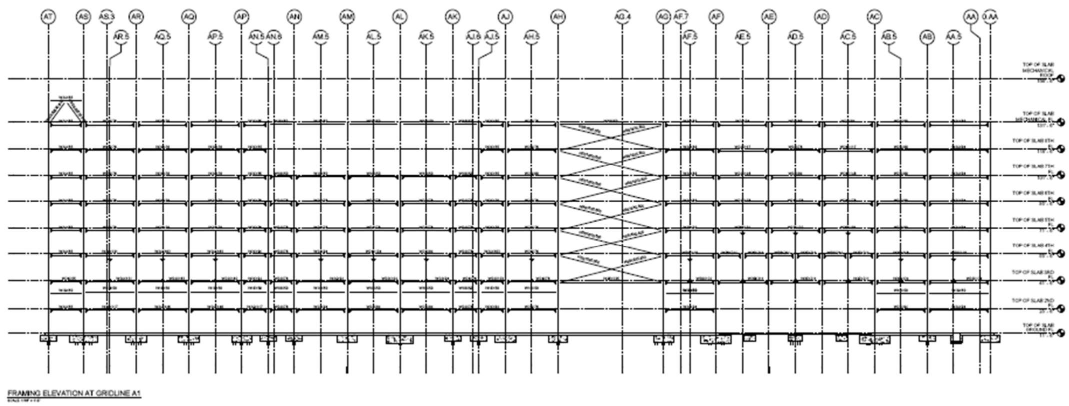
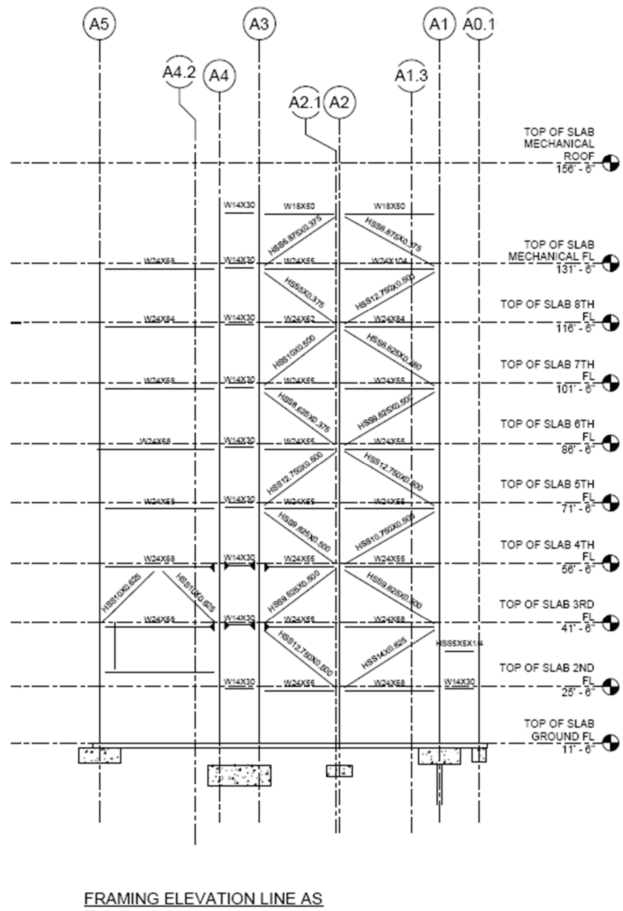


Figure 15: Above is an elevation of the framing system looking in the North/South direction. Notice only moment connections except for the cross bracing on the bridge. Below is an elevation of the framing system looking in the East/West dirction. Notice the majority of cross bracing in this direction compared to few moment connections.



APPENDIX B: WIND CALCULATIONS

The following procedure was used to calculate wind loads based on ASCE 7-10 Standards:

Table 27.2-1 Steps to Determine MWFRS Wind Loads for Enclosed, Partially Enclosed and Open Buildings of All Heights
Step 1: Determine risk category of building or other structure, see Table 1.4-1
Step 2: Determine the basic wind speed, V, for the applicable risk category, see Figure 26.5-1A, B or C
Step 3: Determine wind load parameters:

- ➢ Wind directionality factor, K_d , see Section 26.6 and Table 26.6-1
- ➢ Exposure category, see Section 26.7
- ➢ Topographic factor, K_{zt}, see Section 26.8 and Table 26.8-1
- ➢ Gust Effect Factor, G, see Section 26.9
- ➢ Enclosure classification, see Section 26.10
- ➢ Internal pressure coefficient, (GC_{pi}), see Section 26.11 and Table 26.11-1

Step 4: Determine velocity pressure exposure coefficient, K_z or K_h, see Table 27.3-1
Step 5: Determine velocity pressure q_z or q_h Eq. 27.3-1
Step 6: Determine external pressure coefficient, C_p or C_N

- ➢ Fig. 27.4-1 for walls and flat, gable, hip, monoslope or mansard roofs

Step 7: Calculate wind pressure, p, on each building surface

- ➢ Eq. 27.4-2 for flexible buildings

Please see attached hand calculations for a more in depth look at how wind calculations were performed.

NORTH/SOUTH DIRECTION WIND LOAD HAND CACLULATIONS

JAKE POLLACK | TECH REPORT #1 | WIND ANALYSIS | ①

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 = 150' < 300' $\therefore$ OK

$\boxed{2}$

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = \frac{[130(536) + 20(325)]}{150}$$

$= 508'$

$4 \times 508 > 150 \therefore$ OK

APPROXIMATE NATURAL FREQUENCY

PRELIMINARILY STEEL w/ HSS LATERAL BRACING SYSTEM

$n_a = 75/h \left\{ \begin{bmatrix} 322(150) \\ + \\ 20(325)(150) \end{bmatrix} \right\} / 536 = 142$ [illegible]

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

JAKE POLLACK TECH REPORT #1 WIND ANALYSIS ③

NORTH/SOUTH WIND

3 CONTINUED

GUST EFFECT FACTOR

FLEXIBLE BUILDINGS (26.9.5)

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

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

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{95+142}{459}\right)^{0.63}}} = \boxed{0.86 = Q}$$

$$L_{\bar{z}} = \underbrace{\ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}}_{\text{TABLE 26.9-1}} = 320\left(\frac{85.2}{33}\right)^{1/3} = \boxed{459 = L_{\bar{z}}}$$

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

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

PLUG IN TO Q

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

$n_1 = n_a = 0.53$

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

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

PLUG INTO R

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.53)(459)}{100.33} = \boxed{2.32 = N_1}$$

PLUG INTO R_n

[5] CONTINUED

NORTH / SOUTH WIND

GUST EFFECT FACTOR

TABLE 26.9-1

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

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

PLUG INTO η_h

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) = \frac{1}{2.07} - \frac{1}{2(2.07)^2}\left(1-e^{-2\eta}\right) = \boxed{0.28 = R_h}$$

PLUG INTO R

$$\eta_h = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.55)(142)}{100.39} = \boxed{3.45 = \eta_h}$$

PLUG INTO R_h

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

PLUG INTO R

$$\eta_B = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \boxed{2.71 = \eta_B}$$

PLUG INTO R_B

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

PLUG INTO R

$$\eta_L = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \boxed{42.58 = \eta_L}$$

PLUG INTO R_L

$$R = \sqrt{\frac{1}{0.02}(0.08)(0.28)(0.34)(0.53 + 0.47(0.02))} = \boxed{0.42 = R}$$

PLUG INTO G_f

$$G_f = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{2.4^2\,0.86^2 + 4.4^2\,0.42^2}}{1 + 1.7(3.4)(0.26)}\right) = \boxed{1.00 = G_f}$$

[4] VELOCITY PRESSURE EXPOSURE COEFFICIENT (TABLE 27.3-1)

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

FOR MEAN ROOF HEIGHT $\boxed{K_h = 1.02}$

JAKE POLLACK | TECH REPORT #1 | WIND ANALYSIS (9)

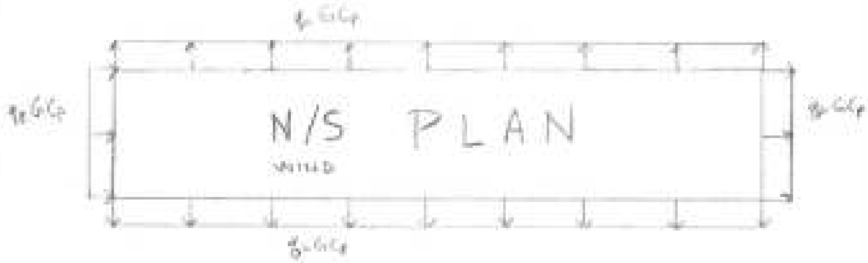
NORTH / SOUTH WIND

[5] VELOCITY PRESSURE (EQ. 27.3-1)

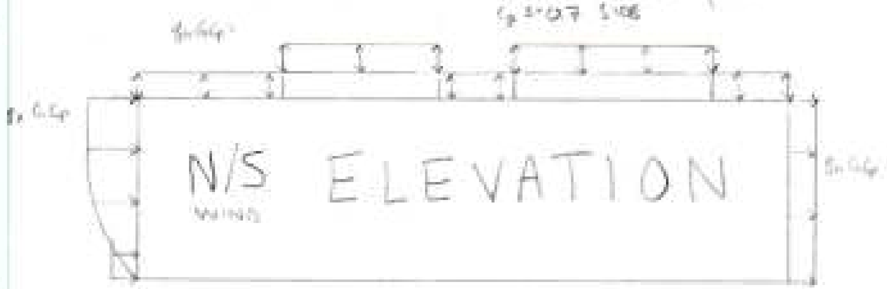
$q_z = 0.00256(K_z K_{zt} K_d V^2) = 0.00256(1.11)(1.0)(0.85)(V^2)$

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

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



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



h/L = 142/536 = 0.26 ≤ 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 153' $C_p = -0.5, -0.18$

AREA = B×L = 95×536 = 50,920 SF ⇒ REDUCTION FACTOR = 0.8
HOWEVER, DOES NOT APPLY

[7] WIND PRESSURE

WINDWARD WALL:

$p = q G C_p - q_i (G C_{pi})$ (EQ 27.4-2)

$p = (34.78)(1.00)(0.8) - (34.15)(\pm 0.18) = 27.82 \pm 6.15$ PSF = $\boxed{+33.97 \text{ PSF}}$

NORTH/SOUTH WIND

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:

$p = 34.78(1.00)(-0.7) - (34.15)(\pm 0.18) = -24.35 \pm 6.15 \text{ PSF} = \boxed{-30.50 \text{ PSF}}$

ROOF:

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

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

$\boxed{-37.5 \text{ PSF}}$

142' TO 150': $p = 34.78(1.00)(-0.5) - 34.15(\pm 0.18) = -17.39 \pm 6.15 \text{ PSF} = \boxed{-23.54 \text{ PSF}}$

8 FORCE OF WINDWARD PRESSURE

$L\left[\frac{h_{FL}}{2}(\text{WIND PRESSURE}) + \frac{h_{FL}}{2}(\text{WIND PRESSURE})\right]/1000 \text{ LBS}$

FLR 5: $\frac{536}{1000}\left[\frac{15}{2}(23.63) + \frac{16}{2}(19.43)\right] = \boxed{178.5^{k}}$

EAST/SOUTH DIRECTION WIND LOAD HAND CACLULATIONS

JAKE POLLACK TECH REPORT #1 WIND ANALYSIS ⑨

EAST / WEST 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.

3 GUST EFFECT FACTOR

1 BLDG HEIGHT OK

2

$$L_{EFF} = \frac{[130(95) + 20(40)]}{150} = 88'$$

$4 \times 88 > 150 \therefore$ OK

APPROXIMATE NATURAL FREQUENCY

PREDOMINANTLY STEEL MOMENT CONNECTIONS

$$n_a = \frac{22.2}{h^{.8}} = \frac{22.2}{138^{.8}} = \boxed{0.43 = n_a} < 1 \therefore \text{FLEXIBLE}$$

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

$$I_{\bar{z}} = 0.3\left(\frac{33}{82.8}\right)^{1/6} = \boxed{0.26 = I_{\bar{z}}}$$

$$L_{\bar{z}} = 320\left(\frac{82.8}{33}\right)^{1/3} = \boxed{435 = L_{\bar{z}}}$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{536 + 138}{435}\right)}} \quad \boxed{0.71 = Q}$$

$\boxed{B = 536'}$, $\boxed{g_Q, g_v = 3.4}$

$$g_R = \sqrt{2 \ln(3600 n_1)} + \frac{0.577}{\sqrt{2 \ln(3600 n_1)}} = \boxed{3.38 = g_R}$$

$\boxed{n_1 = n_a = 0.43}$

JAKE POLLACK | TECH REPORT #1 | WIND ANALYSIS ⑤

[3] CONTINUED

GUST FACTOR

EAST / WEST WIND

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

$$\bar{V}_{\bar{z}} = 0.45\left(\frac{82.9}{33}\right)^{1/4}\left(\frac{88}{60}\right)(120)$$

$$\boxed{\bar{V}_{\bar{z}} = 100 \text{ ft/s}}$$

$$N_1 = \frac{(0.45)(4.35)}{100} = \boxed{1.87 = N_1}$$

$$R_n = \frac{7.47(1.87)}{(1 + 10.3(1.87))^{5/3}} = \boxed{0.09 = R_n}$$

$$R_h: \eta_h = \frac{4.6(0.45)(138)}{100} = \boxed{2.73 = \eta_h}$$

$$\boxed{R_h = 0.30}$$

$$R_B: \eta_B = \frac{4.6(0.45)(53.6)}{100} = \boxed{10.60 = \eta_B}$$

$$\boxed{R_B = 0.09}$$

$$R_L: \eta_L = \frac{15.4(0.45)(95)}{100} = \boxed{6.29 = \eta_L}$$

$$\boxed{R_L = 0.15}$$

$$R = \sqrt{\frac{1}{0.02}(0.09)(0.30)(0.09)(0.53 + 0.47(0.15))} = \boxed{0.27 = R}$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.77^2) + 3.9^2(0.27)^2}}{1 + 1.7(3.4)(0.26)}\right)$$

$$\boxed{G_f = 0.80}$$

JAKE POLLACK | TECH REPORT #1 | WIND ANALYSIS (3)

[4] VELOCITY EXPOSURE COEFFICIENT (TABLE 27.3-1)

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

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

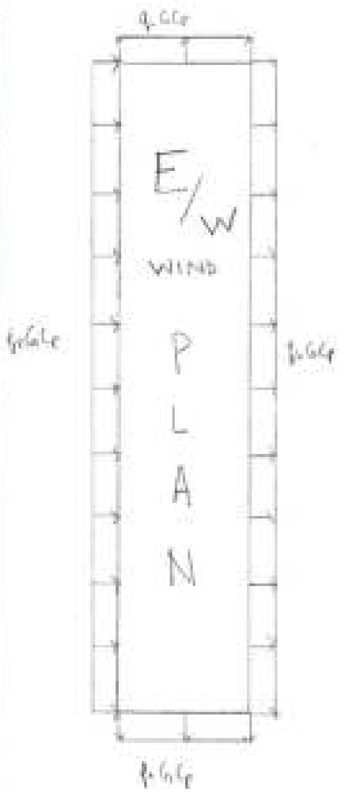
[5] VELOCITY PRESSURE (EQ. 27.3-1)

$q_z = 34.78$ PSF

$q_h = 34.15$ PSF

[6] EXTERNAL PRESSURE COEFFICIENT (FIG 27.4-1 FOR FLAT ROOF)

EAST / WEST WIND



$h/L = 179/35 = 5.45 > 1.0$

FROM 0' TO 63' $C_p = -1.3^{**}, -0.18$

FROM 63' TO 150' $C_p = -0.7, -0.18$

$\Sigma L > 1000$ SF ⇒ REDUCTION FACTOR = 0.8 DOES APPLY**

$L/B = 35/576 = 0.180 \Rightarrow 1$

WINDWARD = 0.8

LEEWARD = -0.5

SIDE = -0.7

JAKE POLLACK | TECH REPORT #1 | WIND ANALYSIS (1)

7 WIND PRESSURE

WINDWARD WALL:

$p = (34.78)(0.8)(0.8) - (34.15)(\pm 0.18) = 22.26 \pm 6.15 = \boxed{28.41 \text{ PSF}}$

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 WALL:

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

ROOF:

0 to 69': $p = 34.78(0.8)(-1.3)(0.8) - (34.15)(\pm 0.18) = -28.94 \pm 6.15 = \boxed{-35.09 \text{ PSF}}$

69' to 138': $p = 34.78(0.8)(-0.7) - (34.15)(\pm 0.18) = -19.48 \pm 6.15 = \boxed{-25.63 \text{ PSF}}$

EAST / WEST WIND

APPENDIX C: SEISMIC CALCULATIONS

	Noth/South Direction Loading				T=	**0.857**	**s**				
					k=	1.180					
					V_b=	539	kips				
i	h_i	h	w	$w*h^k$	C_{VX}	f_i	V_i	By	5%By	Ax	M_z
	ft	ft	kips			kips	kips	ft	ft		k-ft
BULK HEAD	20	150	3322	1227969	0.122	66	50	536	27	1	1761
ROOF	10	130	6753	2108385	0.209	113	163	536	27	1	3024
8	15	120	5574	1583437	0.157	85	248	536	27	1	2271
7	15	105	5574	1352603	0.134	72	320	536	27	1	1940
6	15	90	5847	1182876	0.117	63	383	536	27	1	1696
5	15	75	5847	953906	0.095	51	434	536	27	1	1368
4	15	60	5847	733080	0.073	39	473	536	27	1	1051
3	15	45	5920	528582	0.052	28	502	536	27	1	758
2	16	30	5920	327586	0.033	18	519	536	27	1	470
G	14	14	3301	74315	0.007	4	523	536	27	1	107
		Σ	53905	10072739		539					14445
	East/West Direction Loading				T= 1.540 s		k= 1.52			V_b= 502 K	
i	**h_i**	**h**	**w**	**$w*h^k$**	**C_{VX}**	**f_i**	**V_i**	**Bx**	**5% Bx**	**Ax**	**M_z**
	ft	ft	kips			kips	kips	ft	ft		k-ft
BULKHEAD	20	150	3322	1227969	0.122	61	41	95	5	1.0	291
ROOF	10	130	6753	2108385	0.209	105	146	95	5	1.0	499
8	15	120	5574	1583437	0.157	79	225	95	5	1.0	375
7	15	105	5574	1352603	0.134	67	292	95	5	1.0	320
6	15	90	5847	1182876	0.117	59	351	95	5	1.0	280
5	15	75	5847	953906	0.095	48	399	95	5	1.0	226
4	15	60	5847	733080	0.073	37	435	95	5	1.0	174
3	15	45	5920	528582	0.052	26	462	95	5	1.0	125
2	16	30	5920	327586	0.033	16	478	95	5	1.0	78
G	14	14	3301	74315	0.007	4	482	95	5	1.0	18
		Σ	53905	10072739		502					2385

TYPICAL FLOOR WEIGHT CALCULATION

FLOORS 4, 5, 6			
ITEM	LOAD (PSF)	AREA (SF)	WEIGHT (K)
SLAB ON METAL DECK	46	50920	2342.32
PARTITIONS	20	50920	1018.4
CURTAIN WALLS	15	1262	18.93
STEEL FRAMING	33.5	50920	1705.82
SDL (FIREPROOFING, MEP, FINISH)	15	50920	763.8
TOTAL			5847 KIPS

SEISMIC LOAD HAND CACLULATIONS

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

USE ASCE 7-10 SEISMIC ANALYSIS FOR BUILDING [illegible]

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

SITE CLASS: B (TABLE 20.3-1)

OCCUPANCY CATEGORY: II (TABLE 1-1)

LATITUDE: 40.784098
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)

$S_{DS} = \frac{2}{3} S_{MS} = 0.237$ (EQ 11.4-3)

$S_{D1} = \frac{2}{3} S_{M1} = 0.047$ (EQ 11.4-4)

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

(TABLE 12.8-2) $C_t = 0.02$ $x = 0.75$

SYSTEM RESISTING IN E/W DIRECTION = STEEL SPECIAL MOMENT FRAMES $C_t = 0.028$ $x = 0.8$

	N/S DIRECTION	E/W DIRECTION
$T_a = C_t h_n^x$ (§12.8.2.1)	$0.02(159)^{0.75}$ $T_a = 0.857s$	$0.028(159)^{0.8}$ $T_a = 1.542s$
$T = C_u T_a$	(TABLE 12.8-1) $1.7(0.857)$ $T = 1.46s$	$1.7(1.542)$ $T = 2.62s$

NOTE: $T = \min \begin{cases} C_u T_a \\ T_a \leftarrow \text{FOUND FROM COMPUTER ANALYSIS, WILL BE CALCULATED IN A FUTURE REPORT} \end{cases}$

SEISMIC ANALYSIS CONTINUED

	N/S DIRECTION	E/W DIRECTION
R (TABLE 12.2-1)	7	8
	STEEL CONCENTRICALLY BRACED FRAMES	STEEL SPECIAL MOMENT FRAMES

EQUIVALENT LATERAL FORCE PROCEDURE

		N/S DIRECTION	E/W DIRECTION
C_s = MIN	$\frac{S_{D1}}{T(R/I_e)}$ FOR $T \le T_L$ (EQ 12.8-3)	$\frac{0.047}{1.46(7/1.0)} = \boxed{0.005}$ CONTROLS < 0.01 ∴ NG	$\frac{0.047}{2.44(8/1)} = \boxed{0.002}$ CONTROLS < 0.01 ∴ NG
	$\frac{S_{DS}}{R/I_e}$ (EQ 12.8-2)	$\frac{0.237}{(7/1.0)} = 0.034$	$\frac{0.237}{(8/1.0)} = 0.030$
	$0.044 S_{DS} I_e \ge 0.01$	$0.044(0.237)(1.0) = 0.01$	$= 0.01$

USE 0.01 FOR C_s

SEISMIC BASE SHEAR

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

W = SEISMIC WEIGHT - CALCULATED IN SPREADSHEET

$V = 0.01(53,305^k) = 533^k$

VERTICAL DISTRIBUTION OF SEISMIC FORCES

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

$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$ (EQ 12.8-12)

$k = 1$ FOR $T \le 0.5s$ & $k = 2$ FOR $T \ge 2.5s$

N/S DIRECTION	E/W DIRECTION
$T = 0.85s$ ∴ $\left(\frac{[illegible]}{[illegible]}\right)$	$T = 1.94s$ ∴ $\left(\frac{[illegible]}{[illegible]}\right)$
$k = 1.18$	$k = 1.52$

SEE SPREADSHEET FOR FORCES, SHEARS & MOMENTS

APPENDIX D: SNOW AND DRIFT CALCULATIONS

SNOW LOAD HAND CALCULATIONS

JAKE POLLACK   TECH #1   SNOW CALCULATIONS

USE ASCE 7-10 TO COMPUTE SNOW LOADS

$p_f = 0.7 C_e C_t I_s p_g$

C_e ⇒ EXPOSURE B (PARTIALLY EXPOSED ROOF)

$C_e = 1.0$

C_t ⇒ UNHEATED & OPEN AIR STRUCTURE

$C_t = 1.2$

I_s ⇒ RISK CATEGORY II

$I_s = 1.0$

p_g ⇒ QUEENS, NYC

$p_g = 20$ PSF

$p_f = 0.7(1.0)(1.2)(1.0)(20)$

$\boxed{p_f = 16.8 \text{ PSF}}$

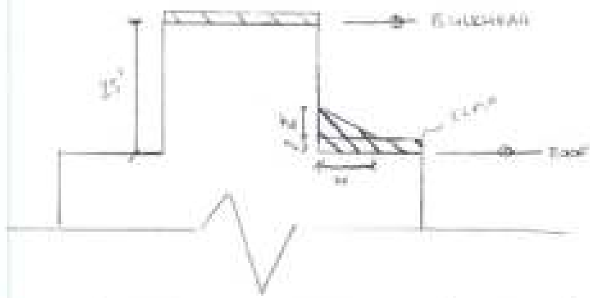
$p_g \leq 20$ PSF so $p_f \geq I p_g = 1.1(20) = \boxed{22 \text{ PSF}}$

CONTROLS

SNOW DRIFT HAND CALCULATIONS

JAKE POLLACK | TECH. #1 | SNOW DRIFT CALCULA…

DRIFTS ON LOWER ROOFS ASCE §7.7



$\gamma = 0.13\,p_g + 14 \le 30\text{ pcf}$ (EQ. 7.7-1)

$= 0.13(20) + 14$

$= 16.6\text{ pcf}$

CALCULATED ABOVE $p_{max} = 22\text{ psf}$

BALANCED SNOW HEIGHT

$h_b = \frac{22}{16.6} = 1.20'$

CLEAR HEIGHT

$h_c = 25' - 1.2' = 23.8'$

$h_c/h_b = \frac{23.8}{1.2} = 19.8' > 0.2$ ∴ DESIGN FOR DRIFT

DRIFT:

$h_d = 0.43\sqrt[3]{l_u}\cdot\sqrt[4]{p_g+10} - 1.5 = 0.43\sqrt[3]{140}\cdot\sqrt[4]{20+10} - 1.5 = 3.85'$

$p_d = h_d\,\gamma = 3.85'(16.6\text{ pcf}) = \boxed{63.9\text{ psf} = p_d}$

$w = 4h_d = 15.4'$

APPENDIX E: SPOT CHECK CALCULATIONS

TYPICAL SLAB/DECK SPOT CHECK

JAKE POLLACK | TECH #1 | TYPICAL SLAB SPOT CHECK | 1

TYPICAL SLAB/METAL DECK:

3.25" LWC FILL ON 3" 18-GAGE COMPOSITE METAL DECK - MATCHES 1.5VL18 IN VULCRAFT STEEL ROOF AND FLOOR DECK CATALOG

MAX UNSHORED SPAN:

9'-3", 3 SPAN → BEAMS TYPICALLY SPACED 10'-0" O.C. CLEARSPAN > 9'-3" ∴ SHORING REQUIRED

UNIFORM LIVE SERVICE LOAD:

STUDS SPACED 1 PER RIB, 4" SLAB, 10' SPAN

⇒ $\boxed{184 \text{ PSF}}$

USE UNFACTORED LOADS TO CHECK:

DL = 46 PSF
SDL = 37 PSF
LL = 80 PSF ← CORRIDORS ABOVE 1ST FLOOR

TOTAL = 163 PSF < 184 PSF ∴ OK

↖ THIS VALUE INCLUDES SELF WEIGHT OF SLAB & METAL DECK

TYPICAL COMPOSITE BEAM SPOT CHECK

JAKE POLLACK | TECH #1 | TYPICAL BEAM SPOT CHECK | ①

EVALUATE A COMPOSITE BEAM

① TRIBUTARY AREA

$A_T = 10 \times 50' = \underline{500\ SF}$

② LIVE LOAD NOT REDUCIBLE B/C

500 SF < 400 SF ... NOT REDUCIBLE

③ w_u:

1.2 (SDL + SM + SLAB) + 1.6 (LL) = PSF

1.2 (17 + 5 + 46) + 1.6 (50) = 185.6 PSF

185.6 PSF × 10' = 1.86 K/FT = w_u

④ FIND M_u

$M_u = \frac{w_u l^2}{8} = \frac{1.86 (30^2)}{8} = 209.3^{K} = M_u$

⑤ FIND b_{eff}

b_{eff} = 1(10)/8×2 = 90" ← CONTROLS

⑥ FIND Y_2

ASSUME a = 1

Y_2 = ... = 5.75"

⑦ USING W18X50

USE Y_2 = ... FOR PNA ... ϕM_n = ... , ΣQ_n = 413 (TABLE 3-19 AISC)

⑧ USING 3/4" STUDS: FIND # STUDS, #, Y_2, WEIGHT

$\frac{\Sigma Q_n}{Q_n} = \frac{413}{21.5} = 20 \times 2 = 40$ STUDS

TABLE 3-21 AISC

JAKE POLLACK | TECH #1 | TYPICAL BEAM SPOT CHECK | 5

(8) CONTINUED

(1) $a = \frac{\Sigma Q_n}{[illegible]} = \frac{415}{[illegible]} = 1.35"$

(2) $Y_2 = 3 + 2.25 - 1.35/2 = 5.58" > Y_2$ [illegible] ∴ OKAY

(3) [illegible]

$50 \times 30 + 40 \times 10 = 1.7"$

(9) CHECK UNSHORED STRENGTH

(a) TABLE 5-2 AISC: $\phi M_n = 415'^k$

(b) FIND W_U:

$W_U = 1.4 D_L = 1.4(83 \times 10 + 50) = 1.232$ KLF

$W_U = 1.2 D_L + 1.6 L_L = 1.2(83 + [illegible]) + 1.6([illegible]) = 1.696$ KLF [illegible]

(c) $M_U = \frac{W_U \ell^2}{8} = \frac{1.696(30^2)}{8} = 190.8'^k < 415'^k$ ∴ OK

(d) $V_U = \frac{w\ell}{2} = \frac{1.696(30)}{2} = 25.4^k < 132^k$ ∴ OK

(10) CHECK Δ_{LL}

(a) FIND W_{LL}:

$W_{LL} = (LL)(W_{TRIB}) = 50 \times 10 = 0.5$ KLF

(b) LOOK UP I_{LB} (TABLE 3-20 AISC)

$I_{LB} = 1120$

(c) $\Delta_{LL} = \frac{5 w_{LL} (\ell^4)(1728)}{384 E I_{LB}} = \frac{5(0.5)(30^4)(1728)}{384(29000)(1120)} = 0.15"$

$L/360 = \frac{\ell(12)}{360} = 1"$

$0.15" < 1"$ ∴ OK

JAKE POLLACK	TECH #1	TYPICAL BEAM SPOT CHECK	5

(11) CHECK WET CONC Δ:

(a) FIND W_{wc}

$W_{wc} = (46 \times 10) + 50 = 510$ PLF

(b) $\Delta_{wc} = \dfrac{5 w_{wc} \ell^4 (1728)}{384 E I_{LB}} = \dfrac{5(0.510)(30)^4(1728)}{(384)(29000)([illegible])} = 0.40''$

*AISC TABLE 3-20

$\ell/240 = \dfrac{30(12)}{240} = \dfrac{360}{240} = 1.5'' > 0.40''$ ∴ OK

(12) $\boxed{\text{W 18×50 w/ (10) 3/4" φ STUDS IS SUFFICIENT}}$

TYPICAL COMPOSITE GIRDER SPOT CHECK

JAKE POLLACK | TECH #1 | TYPICAL GIRDER SPOT CHECK | ③

EVALUATE A COMPOSITE GIRDER

① DRAW A MOMENT DIAGRAM:

② V_u = [illegible] = 51.9

M_u = [illegible] = 529.3

③ USE W24x76 & ASSUME Y_2 = [illegible]

[illegible]

④ [illegible] STUDS [illegible]

$\frac{\Sigma Q_n}{Q_n} = \frac{660}{21.1}$ = 31.1 = 64 STUDS

⑤ FIND WEIGHT

76 x 30' + [illegible] = 2.5

⑥ CHECK UNSHORED STRENGTH

ⓐ (TABLE 3-2 AISC) $\phi_b M_p$ = 750

ⓑ W_u = $\frac{50.9}{30}$ = 1.693

ⓒ M_u = 529.3 < 750 ∴ OK

ⓓ V_u = 51.9 < 316 ∴ OK

⑦ CHECK Δ_{LL}

Δ_{LL} = [illegible] = 0.15

Δ_{LL} = $\frac{\ell \times 12}{360}$ = 1" > 0.15 ∴ OK

(5) Check Δ_{LL}

$$\Delta_{LL} = \frac{5 w_{LL} l^4 (1728)}{384 E I} = \frac{5(1.38)(30')^4(1728)}{384(29000)(2490)} = 0.41''$$

$$w_{LL} = \frac{46 \times 30}{1000} = 1.38 \text{ klf}$$

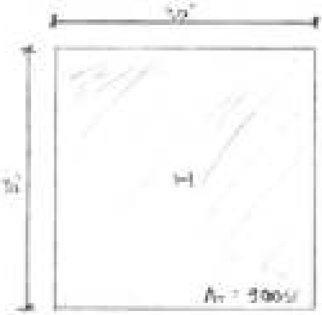
$$l/240 = \frac{30(12)}{240} = 1.5'' > 0.41'' \text{ ok}$$

TYPICAL COLUMN SPOT CHECK

JAKE POLLACK | TECH #1 | TYPICAL COLUMN SPOT CHECK | 0

EVALUATE TYPICAL COLUMN (A3-AQ)

COLUMN EXTENDS FROM GROUND FLOOR TO TOP OF 8TH FLOOR (ROOF) AND IS SPLICED AT FLOORS 3, 5 & 7. COLUMN A3-AQ IS AN INTERIOR W14 COLUMN THAT SUPPORTS OFFICES AND CLASSROOMS FROM TOP TO BOTTOM. LIVE LOADS FOR OFFICES, SINCE GREATER IN MAGNITUDE, WILL BE USED TO BE CONSERVATIVE



$LL_{RED} = 50_{PSF} \times \left[0.25 + \frac{15}{\sqrt{4(900)(8)}} \right] = 20$ ... MAX / 0.4

$P_{DL} = (2+5+20+5+5+46)(8)(900) = 727^k$

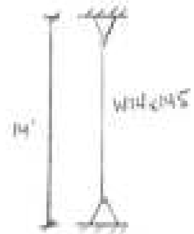
$P_{DEAD} = (22)(900) = 19.8^k$

$P_{LL} = (50)(900)(8) = 360^k$ ← IGNORE LLRED FOR CONSISTENCY

$P_u = 1.2(727) + 1.6(360) + 0.5(19.8) = 1461^k$

$\phi P_{n\,W14\times145} = 1650^k > 1461^k$ ∴ OK

W14 x 145 @ 14' IS SUFFICIENT (AISC TABLE 4-1)



A W14 x 132 @ 14' COULD HAVE BEEN USED W/ $\phi P_n = 1500^k$