New York Police Academy

ARCHITECTURAL ENGINEERING SENIOR THESIS 2010-2011

Thesis Final Report
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Structural Option
Faculty Consultant – Dr. Boothby
Submitted – April 7, 2011
AE 897G/497G

NEW YORK POLICE ACADEMY
COLLEGE POINT, NEW YORK

ARCHITECTURE
- AN ACADEMIC BUILDING HOUSING CLASSROOMS, SOME OF WHICH CONTAIN A MOCK ENVIRONMENT FOR IMMERSION LEARNING
- A PHYSICAL TRAINING BUILDING - HOUSES A 1/8 MILE TRACK AND SPECIAL TACTICAL GYMNASIUMS
- A FACADE MADE UP OF GLAZED ALUMINUM CURTAINWALLS WITH ALUMINUM PANELING WHICH ACT AS LOUVERS ABOVE WINDOWS
- DESIGNED TO REACH A LEED SILVER RATING

STRUCTURE
- STEEL MOMENT RESISTING FRAME
- UPPER 7 STORIES CONTAIN 4" CONCRETE SLAB ON METAL DECK
  - *FLOORS SUPPORTED BY WIDE FLANGE BEAMS SPACED AT 10'
  - *BEAMS SUPPORTED BY WIDE FLANGE GIRDERS SPACED AT 30'
  - *GIRDERS SUPPORTED BY WIDE FLANGE COLUMNS ON 30'x30' GRID
- HSS MEMBERS USED FOR LATERAL BRACING
- GROUND FLOOR COMPRISED OF 14" CAST-IN-PLACE SLAB

MEP SYSTEMS
- A CENTRAL UTILITY PLANT FOR ALL MEP SERVICES
- MECHANICAL: VARIABLE VOLUME AIR HANDLING UNITS AND DECDICATED RETURN FANS
  - *CONSISTS OF MIXING BOXES, FILTER SECTIONS, PRE-HEAT COIL SECTIONS, COOLING COIL SECTIONS, ACCESS SECTIONS AND SUPPLY FAN SECTIONS
- ELECTRICAL: 208Y/120V 3 PHASE SYSTEM, 480Y/277V 3 PHASE SYSTEM
- LIGHTING: FLUORESCENT LIGHTING THROUGHOUT

GENERAL INFORMATION

LOCATION: QUEENS, NY
FUNCTION: ACADEMY
SIZE: 1 MILLION SQUARE FEET
NUMBER OF STORIES: 8
CONSTRUCTION: OCTOBER 2010 - DECEMBER 2013
COST: $656 MILLION
PROJECT DELIVERY: DESIGN-BID-BUILD

PROJECT TEAM MEMBERS

OWNER: NEW YORK DEPARTMENT OF DESIGN AND CONSTRUCTION
GENERAL CONTRACTOR: TURNER CONSTRUCTION
ACOUSTICS: CERAMI & ASSOCIATES
ARCHITECT: PERKINS + WILL
BLAST CONSULTANT: WEIDLINGER ASSOCIATES
COST CONTROL: GARDINER & THEOBALD INC.
COST ESTIMATING: DAVIS LANGDON
CIVIL ENGINEER: LAGAN ENGINEERING AND ENVIRONMENTAL SERVICES
CONSTRUCTION MANAGER: TURNER CONSTRUCTION / STV
GEOTECHNICAL: URS CORPORARTION
IT CONSULTANT: TM TECHNOLOGY PARTNERS, INC.
LANDSCAPE ARCHITECT: BLAMORI ASSOCIATES
LIGHTING: BARTENBACH LICHTLABOR GMBH / HDLC
MEPF ENGINEER: WSP FLAC + KURTZ
PARKING: WALKER PARKING CONSULTANTS
STRUCTURAL ENGINEER: ROBERT SILMAN ASSOCIATES
URBAN DESIGNER: FXFOWLE
VERTICAL TRANSPORTATION: VAN DEUSEN ASSOCIATES

JAKE POLLACK | STRUCTURAL OPTION | AE

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EXECUTIVE SUMMARY

The New York Police Academy is a building that consolidates the New York City Police recruit training into one facility. This building is located in College Point, New York and is 536' long, 95' wide and 150' high. The building has a gravity system consisting of lightweight concrete on metal deck. In the East/West direction, the "X-Direction," the building has moment connections and one double bay of HSS cross bracing to resist lateral loads. In the North/South direction, the "Y-Direction," the lateral resisting system consists of HSS cross bracing in two of the three bays.

This report focuses on showing the changes that occur when altering the lateral resisting systems of the New York Police Academy. The thesis redesign removed the 468 moment connections from the original design and added 128 concentrically braced connections to resist lateral load in the X-direction. This connection was used strictly for constructability purposes. The thesis redesign also removed the 168 concentrically braced HSS connections and replaced them with 136 chevron braced W-shape connections. The chevron system was chosen because the bay sizes were rather large. The W-shapes were chosen because they are cheaper to fabricate than HSS shapes. HSS shapes are typically chosen for their aesthetic appeal. However, in this project, those frames are hidden within the walls. Because HSS shapes are aesthetically pleasing the double bay of HSS bracing in the X-direction remained intact during the redesign. These connections were designed using the controlling lateral loads for the New York Police Academy.

The concentrically braced frames in the X-direction greatly influenced the architecture. The moment frames allowed for a glazed curtainwall façade, which was altered in order to compensate for the concentrically braced frames. Rather than hide the braces, the structure was accentuated with the intention of integrating and embracing the system within the façade.

The construction studies of cost and scheduling provide optimal results. The original lateral system costs $1,606,325.97. The thesis lateral system in conjunction with the changes made to the façade of the New York Police Academy would cost $827,408.92, a savings of more than 48%. The redesign would also limit the number of man hours needed to assemble the lateral system from 4,116 man hours in the original design to 1,320 man hours. This accounts for approximately 68% savings in time, allowing the building to be constructed faster.

In sum, the thesis redesign would save both time and money without sacrificing the building aesthetics.

ACKNOWLEDGEMENTS

I would like to thank the following professionals and The Pennsylvania State University Architectural Engineering Faculty for their guided assistance and generosity throughout the 2010 -2011 academic year with my thesis project.

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Steve Fisher – Tom Mullen – Derrin Sample

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Professor Robert Holland – Professor M. Kevin Parfitt

The Entire AE Faculty and Staff

I would also like to thank my mother, Brenda; father, Alan; brothers, Matt and Alex; and all of my friends who have supported me over the last five years. I could not have done it without your help and guidance.

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. This thesis report will focus on the east campus.

The New York Police Academy can be viewed within its surroundings in Figure 1 to the right. 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

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

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 began in October 2010 and culminates in December 2013.

The purpose of the Final Thesis Report is to modify facets of the existing building to improve it. A different lateral resisting system was designed in this report. The New York Police Academy lateral system redesign increases the rate of construction while limiting the cost. This alteration also changes the façade of the New York Police Academy. These topics will be discussed in more detail throughout this report.

ARCHITECTURAL OVERVIEW

This 8-story 1,000,000 square-foot 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 while others such as the building's green roofs 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 3).

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.

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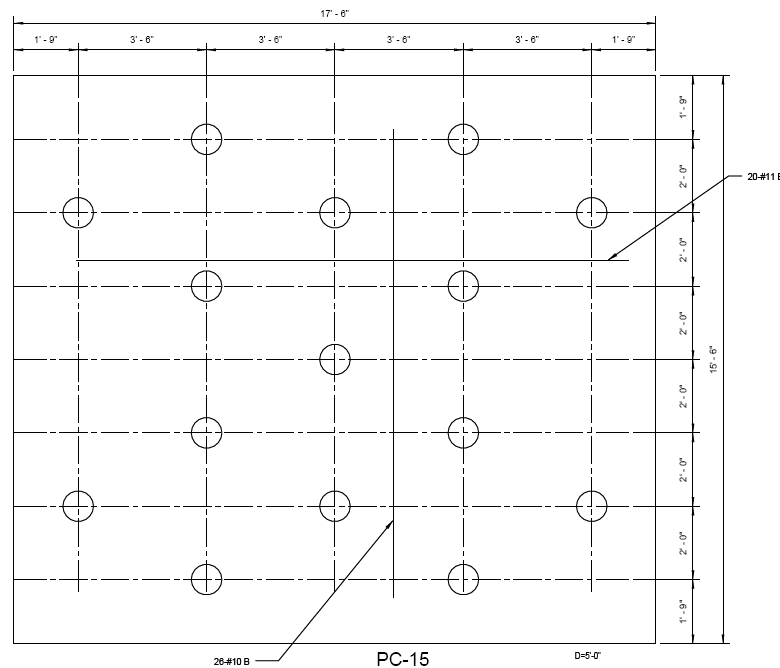
FIGURE 3: THIS IS PLAN OF A SAMPLE PILE CAP. DETAIL COURTESY OF TURNER CONSTRUCTION.

EXISTING FLOOR SYSTEM

The floor system is made up of 3.25" lightweight concrete slab on 3" - 18 gage metal decking. This forms 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.

TYPICAL DETAIL
COMPOSITE FLOOR DECK
PERPENDICULAR EDGE CONDITION

TYPICAL DETAIL
COMPOSITE FLOOR DECK
PARALLEL EDGE CONDITION
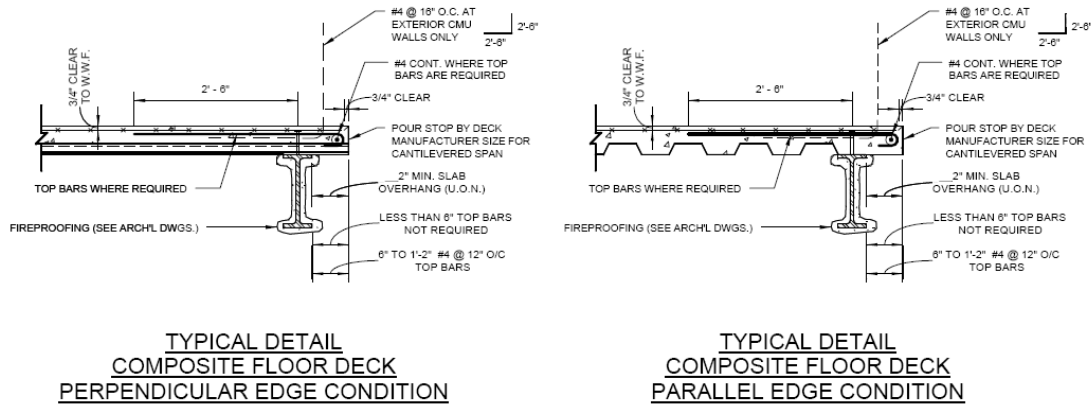
FIGURE 4: TYPICAL SLAB ON DECK FLOOR SECTIONS. DRAWINGS NOT TO SCALE. DETAIL COURTESY OF TURNER CONSTRUCTION.

EXISTING FRAMING SYSTEM

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.
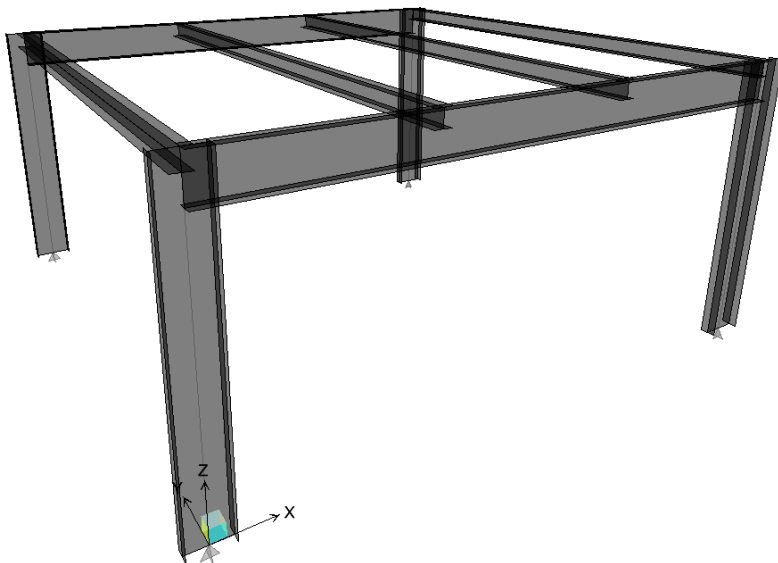
FIGURE 5: THIS IS AN ETABS MODEL OF THE TYPICAL BAY FRAMING.

EXISTING LATERAL SYSTEM

EXISTING X-FRAME

There are two lateral force resisting systems in the New York Police Academy. One system is demonstrated by the X-Frame in Figure 6 below and consists of moment connections throughout the building with a double bay of HSS cross bracing. The HSS cross bracing is where the bridge connects one section of the building to another.

FIGURE 6: THIS IS AN IMAGE OF THE X-FRAME LATERAL RESISTING SYSTEM IN THE NEW YORK POLICE ACADEMY.
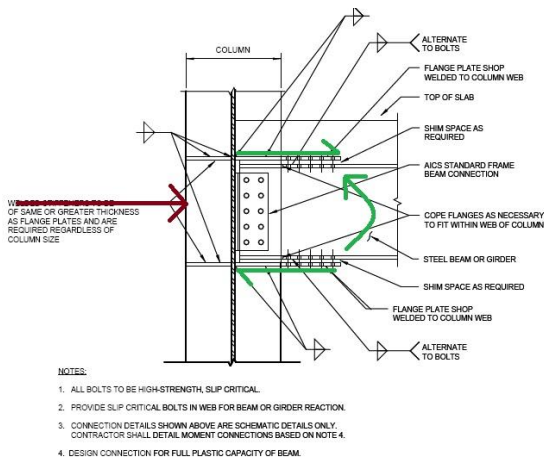
FIGURE 7: THIS IMAGE SHOWS THE LATERAL LOAD PATH THROUGH A MOMENT CONNECTION.

Figure 7 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.

EXISTING Y-FRAME
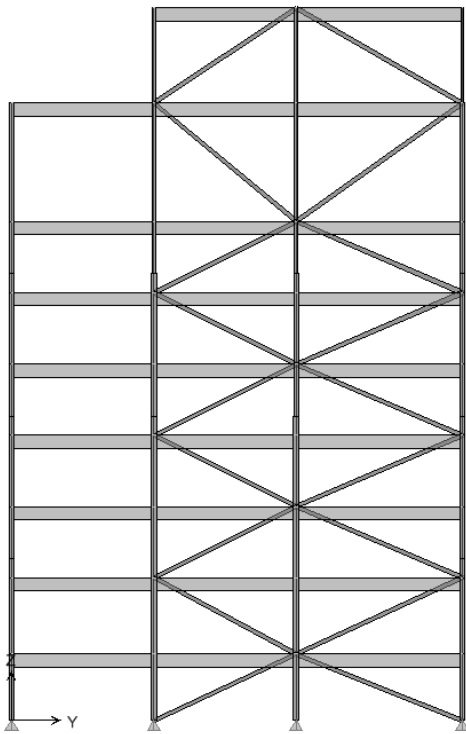
The other lateral force resisting system is in the direction orthogonal to the X-frame and is referred to as the Y-frame. This frame has HSS cross bracing to resist lateral loads and all connections are pinned. This can be seen in Figure 8 to the left.

Figure 9 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.
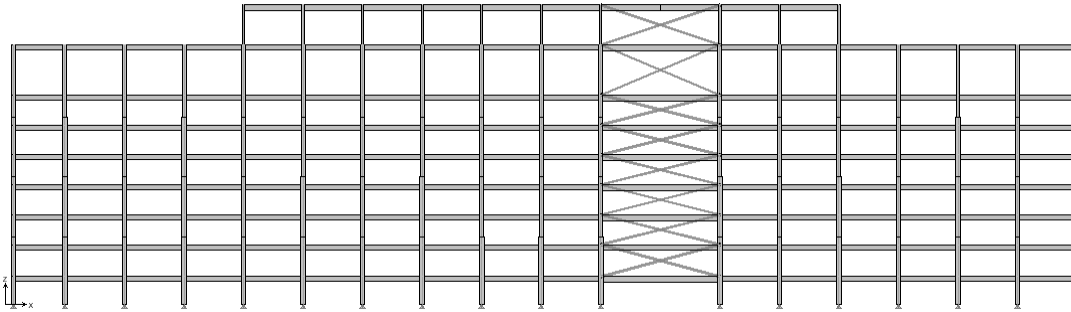
FIGURE 8: THIS IS AN IMAGE OF THE Y-FRAME LATERAL RESISTING SYSTEM IN THE NEW YORK POLICE ACADEMY.
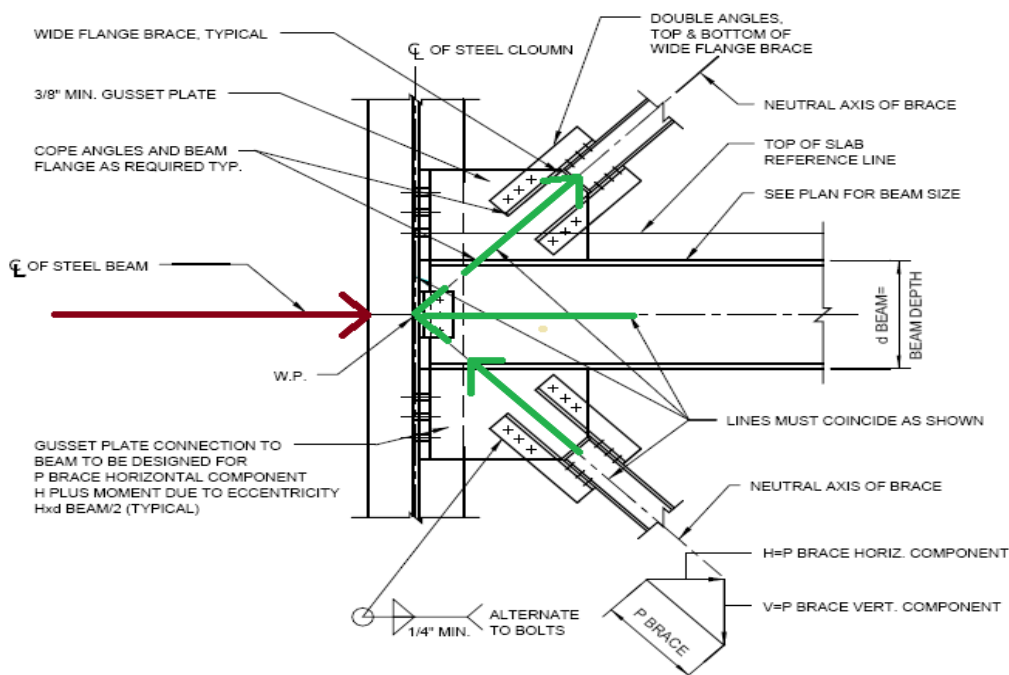
FIGURE 9 THIS IMAGE SHOWS THE LATERAL LOAD PATH THROUGH AN HSS CROSS BRACED CONNECTION.

LATERAL SYSTEM DESIGN PROCESS

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', the width of the building is 95' and the height from the ground to the tallest point is 150'. These dimensions are the same dimensions that were provided by Turner Construction; however certain architectural protrusions and indentations were neglected.

Due to limitations provided by the given drawing sets 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 the Lateral Movement section of this report on page 51. 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.
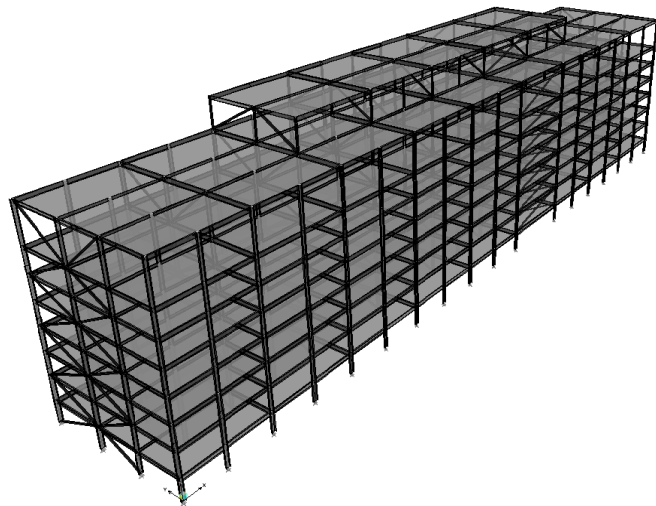
FIGURE 10: THIS IS AN IMAGE OF THE NEW YORK POLICE ACADEMY MODELED IN ETABS.

PROBLEM STATEMENT

There are two different types of steel connections in the lateral resisting systems of the New York Police Academy, most of which are rather complex. As seen in Figure 11 to the right, to construct the concentrically braced lateral resisting systems in both the X- and Y- Frames, HSS braces must be welded to plates, which are welded to double angles. The double angles are then bolted to both the supporting columns and beams. Please note that weld arrows were erased from this detail for viewing purposes.
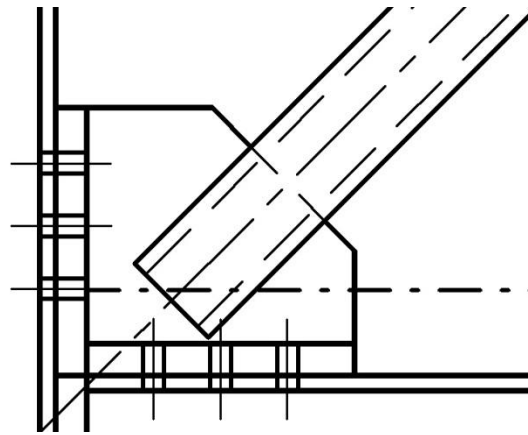
FIGURE 11: THIS FIGURE SHOWS A CLOSE UP VIEW OF THE LATERAL HSS BRACED CONNECTIONS.
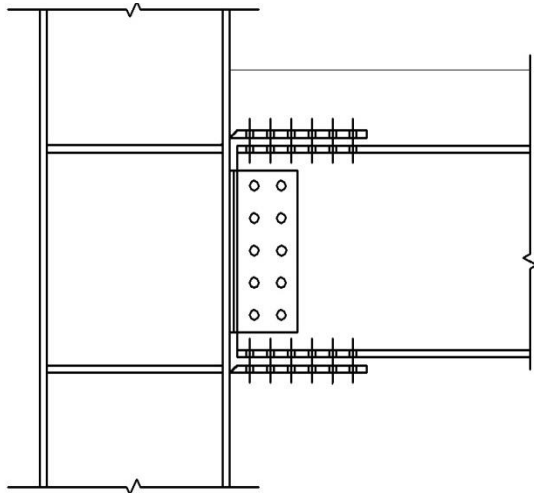
FIGURE 12: THIS FIGURE SHOWS A CLOSE UP VIEW OF THE MOMENT CONNECTIONS.

The steel connections in the X-Frame are moment connections, which incorporate both welds and bolts in various locations. This connection also includes column stiffeners as seen in Figure 12 to the left. Please note that weld arrows were erased from this detail for viewing purposes. Both of these connections are very complicated to fabricate and labor intensive. It is time consuming to construct these connections and skilled workers must be used.

PROBLEM SOLUTION

If the lateral resisting system in the X- and Y- Frames are adjusted from moment connections to a more basic connection then these substantial connections would no longer be needed. The connections would be simpler to fabricate and easier to construct. Because a standard connection would be used in each frame then the builders could construct the connections at a more rapid pace. The types of connections that were analyzed were moment, concentrically and eccentrically braced connections. It was predicted that either concentric or eccentric connections would be the most efficient. A comparison was done relating the following lateral frames in both East/West (X-Frame) and North/South (Y-Frame) directions in Table 1 below.
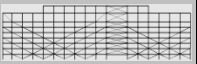
LATERAL SYSTEM POSSIBILITIES		
COMBINTION #	X-FRAME	Y-FRAME
1	MOMENT FRAMES	CONCENTRICALLY BRACED FRAMES
2	MOMENT FRAMES	ECCENTRICALLY BRACED FRAMES
3	MOMENT FRAMES	MOMENT FRAMES
4	CONCENTRICALLY BRACED FRAMES	CONCENTRICALLY BRACED FRAMES
5	CONCENTRICALLY BRACED FRAMES	ECCENTRICALLY BRACED FRAMES
6	CONCENTRICALLY BRACED FRAMES	MOMENT FRAMES
7	ECCENTRICALLY BRACED FRAMES	CONCENTRICALLY BRACED FRAMES
8	ECCENTRICALLY BRACED FRAMES	ECCENTRICALLY BRACED FRAMES
9	ECCENTRICALLY BRACED FRAMES	MOMENT FRAMES

TABLE 1: THIS TABLE SHOWS THE DIFFERENT TYPES OF LATERAL SYSTEMS WILL BE ANALYZED.

This assessment was done in ETABS and compared frame stiffnesses, lateral movement, and torsional shear as calculated in the Lateral System Analysis and Confirmation Design Technical Report. Furthermore once the bracing systems were selected as the most efficient connections they were further optimized. Where cross bracing was exposed HSS shapes were used because they are more aesthetically appealing; however, they are not exposed in the majority of the building and thus double angles and W-shapes were used because they are more easily fabricated and installed.

DESIGN GOALS

The overall design goal of this project is to use the most cost efficient lateral system in the New York Police Academy without negatively effecting on performance. Additional goals to be met throughout this project include:

◊ Do not affect the interior architecture and floorplan lay out
◊ Reduce the amount of lateral connections needed
◊ Adjust the exterior façade as appropriate while remaining aesthetically pleasing
◊ Compare the costs and scheduling differences between systems
◊ Use ETABS to perform in-depth lateral analysis to create a more efficient structure and verify by hand
◊ Design the most efficient connections for the New York Police Academy

STRUCTURAL DEPTH STUDY

The structural depth study includes a design and analysis of the proposed lateral system for the New York Police Academy as defined in the problem statement. For this to occur, nine different models were designed and analyzed to ensure that the optimal lateral system was used. Once optimal systems were chosen they were modified to yield ideal results. Final conclusions and recommendations are based on the impact that the structural depth study had on structural performance, architecture and constructability.

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 Standards:

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

DESIGN CRITERIA

DEFLECTION

Floor Deflection:

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

Lateral Drift:

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

Main Structural Elements Supporting Components and Cladding:

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

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

LATERAL SYSTEM OPTIMIZATION DESIGN

DESIGN LOADS

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 2 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. The multipliers for the wind equation vary between editions as well so the resulting pressure on the building is comparable.

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 L = LIVE LOAD R = RAIN LOAD W = WIND LOAD E = EARTHQUAKE LOAD L_r = ROOF LIVE LOAD S = SNOW LOAD		

TABLE 2: 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.

GRAVITY 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 3 and 4 below. Table 3 shows dead loads while Table 4 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 3: 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 4: THIS TABLE COMPARES LIVE LOADS BETWEEN NYCBC-08 AND ASCE 7-10

WIND DESIGN LOADS

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 13, 14, 15 and 16 on the following pages 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 5 outlines variables and classifications needed to perform wind load calculations in the North/South direction. Table 6 displays the calculations and results in this direction as Figures 13 and 14 illustrate these effects. For a more in depth look at wind load calculations please refer to Appendix C.

NORTH/SOUTH WIND VARIABLES 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 5: THIS TABLE SHOWS THE VARIABLES AND CLASSIFICATIONS NECESSARY TO CALCULATE WIND PRESSURES IN THE NORTH/SOUTH DIRECTION.
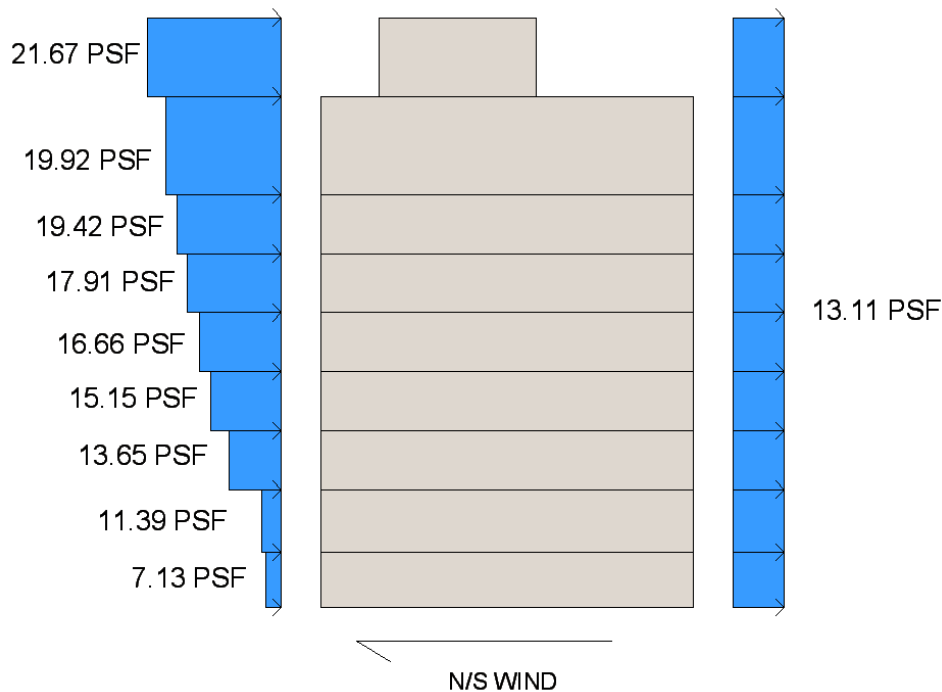
FIGURE 13: THIS FIGURE GRAPHICALLY SHOWS THE WIND PRESSURES ON THE BUILDING IN THE NORTH/SOUTH DIRECTION.

FLOOR	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

Caption for table: **NORTH/SOUTH WIND LOADS**

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

116.2 K →

169.5 K →

131.5 K →

150.1 K →

139.0 K →

127.9 K →

115.8 K →

100.7 K →

76.4 K →

BASE SHEAR = 1126.9 K

FIGURE 14: THE FIGURE TO THE LEFT GRAPHICALLY SHOWS THE WIND SHEAR FORCE ON EACH STORY IN THE NORTH/SOUTH DIRECTION.

Table 7 outlines variables and classifications needed to perform wind load calculations in the East/West direction. Table 8 displays the calculations and results in this direction as Figures 15 and 16 illustrate these effects.

EAST/WEST WIND VARIABLES 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 7: THIS TABLE SHOWS THE VARIABLES AND CLASSIFICATIONS NECESSARY TO CALCULATE WIND PRESSURES IN THE EAST/WEST DIRECTION.
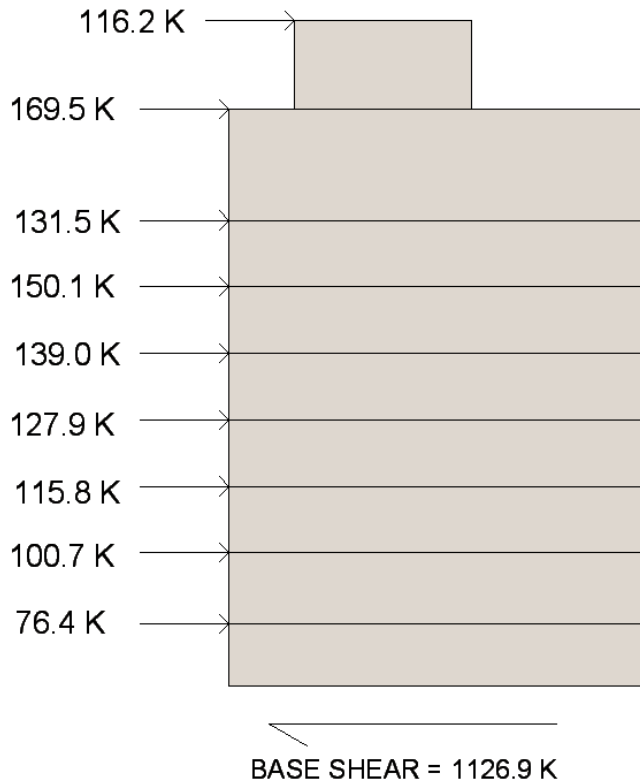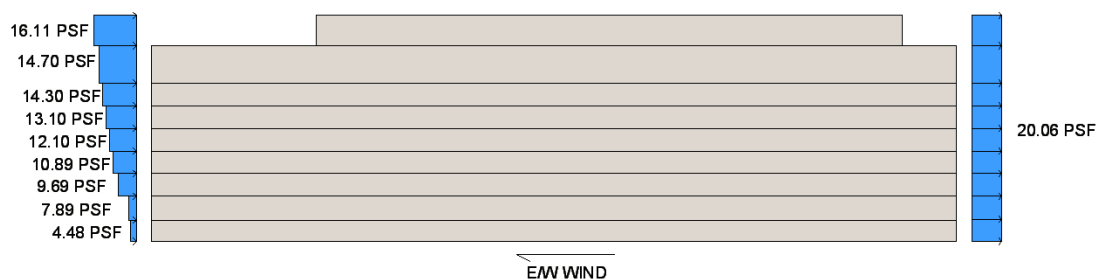

16.11 PSF
14.70 PSF
14.30 PSF
13.10 PSF
12.10 PSF
10.89 PSF
9.69 PSF
7.89 PSF
4.48 PSF

20.06 PSF

E/W WIND

FIGURE 15: THIS FIGURE GRAPHICALLY SHOWS THE WIND PRESSURES ON THE BUILDING IN THE EAST/WEST DIRECTION.

			EAST/WEST WIND LOADS					
FLOOR	STORY HEIGHT (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	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 8: THE TABLE ABOVE SHOWS THE FLOOR WIND PRESSURES AND FORCES ALONG WITH SHEAR/MOMENT FORCES IN THE EAST/WEST DIRECTION.
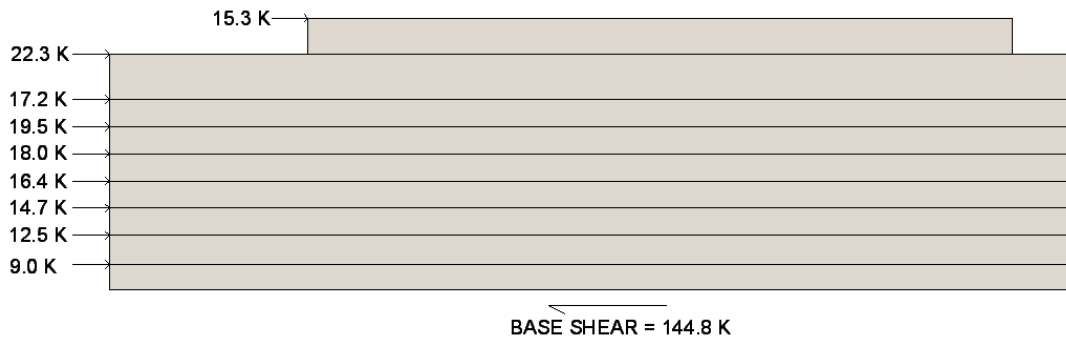

FIGURE 16: THIS FIGURE GRAPHICALLY SHOWS THE WIND SHEAR FORCE ON EACH STORY IN THE EAST/WEST DIRECTION.

SEISMIC DESIGN LOADS

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 seismic variables are organized in Tables 9 and 10.

SEISMIC VARIABLES		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

TABLE 9: THIS TABLE SHOWS THE VARIABLES AND CLASSIFICATIONS NECESSARY TO CALCULATE SEISMIC FORCES.

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 10: SEISMIC CALCULATION RESULTS

The summary of results for the North/South Seismic Forces is found in Table 11 and can be seen in Figure 17 on the following pages.

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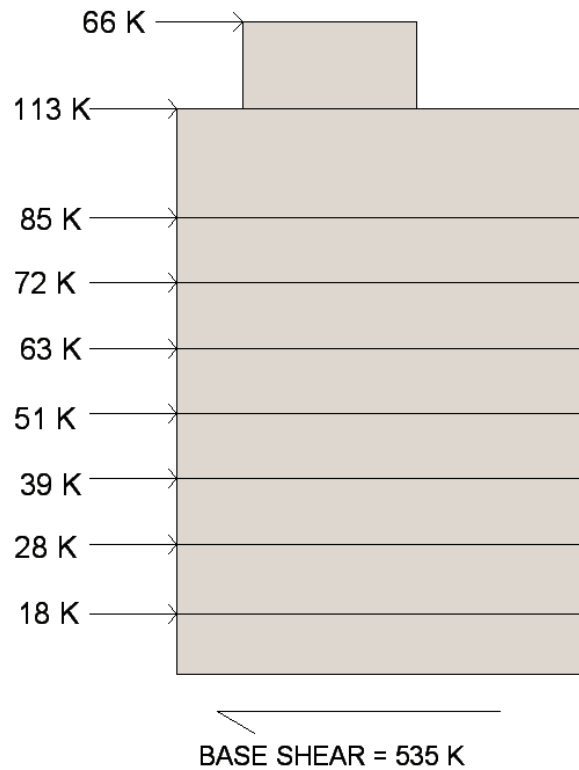

FIGURE 17: THIS FIGURE SHOWS THE SEISMIC SHEAR FORCE ON EACH STORY IN THE NORTH/SOUTH DIRECTION.

The summary of results for the East/West Seismic Forces is found in Table 12 and can be seen in Figure 18 on the following pages. For additional information please look at seismic load calculations please refer to Appendix C.

EAST/WEST SEISMIC FORCES							
FLOOR	WEIGHT w_x (K)	HEIGHT h_x (FT)	$w_kh_x^k$	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 12: THIS TABLE SHOWS THE CACULATIONS AND PROCESSES NEEDED IN ORDER TO CALCULATE SEISMIC BASE SHEAR IN THE NORTH/SOUTH DIRECTION

77 K
134 K
101 K
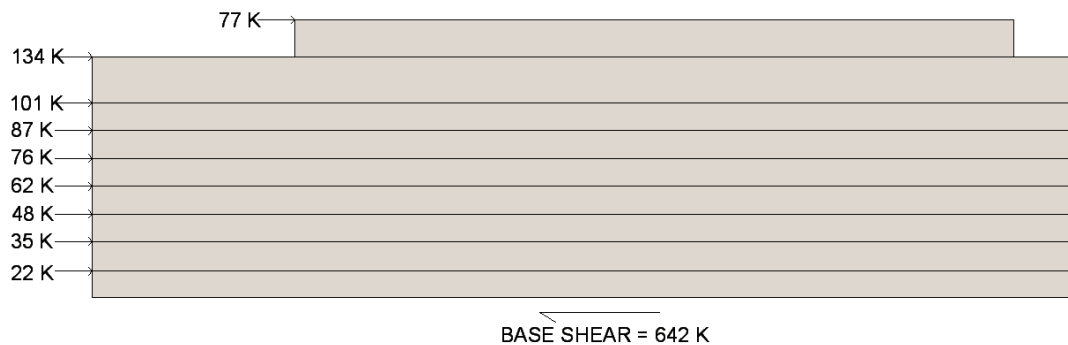87 K
76 K
62 K
48 K
35 K
22 K

BASE SHEAR = 642 K

FIGURE 18: THIS FIGURE GRAPHICALLY SHOWS THE SEISMIC SHEAR FORCE ON EACH STORY IN THE EAST/WEST DIRECTION.

LATERAL SYSTEM DESIGN PROCESS

ETABS COMPUTER MODEL

A computer model was created using ETABS, Computer and Structures Inc. structural modeling program. This model included all structural components in the New York Police Academy including columns, beams, girders, slabs and lateral bracing as designed. All of these elements participate in the distribution of lateral forces. Figure 19 to the right shows columns in magenta, beams and lateral bracing in blue and slabs in grey. Results from the model helped determine the frame stiffness of certain elements, the story displacements and the effects of torsion on the building. Analysis assumptions that were included in the ETABS model include, but are not limited to:

FIGURE 19: THIS FIGURE SHOWS THE STRUCTURE OF THE NEW YORK POLICE ACADEMY REDESIGN.
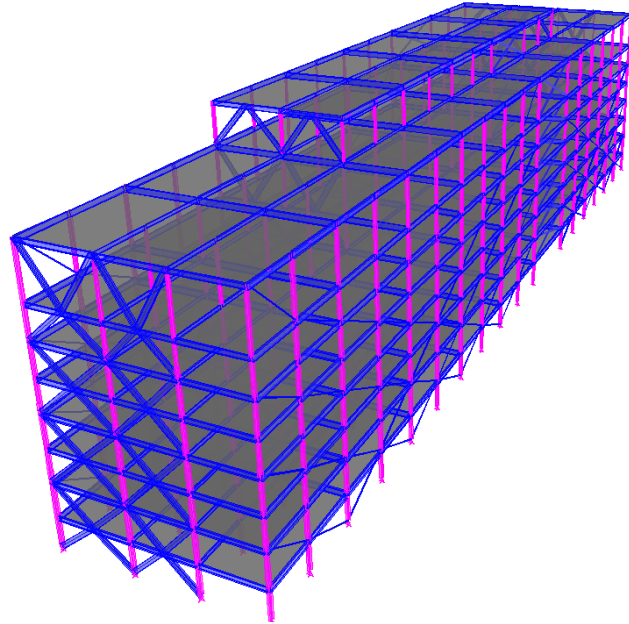
◊ Rigid diaphragms modeled at each floor
◊ All restraints at the ground level were pinned
◊ All lateral displacement include P-Delta Effects
◊ Structural members were modeled with their material properties
◊ Beams, columns and bracing were modeled as line elements
◊ Seismic loads were applied to the center of mass of each floor diaphragm
◊ Wind loads were applied at the center of pressure

LOAD PATH AND DISTRIBUTION

Loads travel through the structure of a building laterally and vertically until they reach the ground. The paths in which loads are distributed are determined based on frame relative stiffnesses. The larger the relative stiffness, the greater the load that frame receives proportionately. In the X-Direction there are two identical concentrically braced frames as shown in Figure 20. These frames both receive approximately 46% of the applied load. In the Y-Direction there are four identical chevron braced frames as shown in Figure 21. These frames each receive approximately 25% of the load. The New York Police Academy is fairly symmetric and thus the center of rigidity and the center of mass are very closely located. Not only does this evenly distribute lateral forces throughout the building, but it also reduces the effect of torsion. Additional load path and distribution can be viewed in the Graduate Course Integration Section on page 38.

X-FRAME

The lateral resisting system in the X-Frame was altered from moment connections to a concentrically braced system strictly for constructability and fabrication purposes. It was then adjusted as necessary for performance, in order to eliminate the substantial moment connections. This alteration replaced a total of 468 moment connections throughout all four frames spanning the X-Direction with 128 concentrically braced connections in only the two exterior frames. These frames were used in the New York Police Academy thesis redesign for the combined purposes of fabrication and constructability. A more in depth look at the connections are on pages 39 through 45, and a more in depth look at the effects on fabrication and constructability are on page 57.
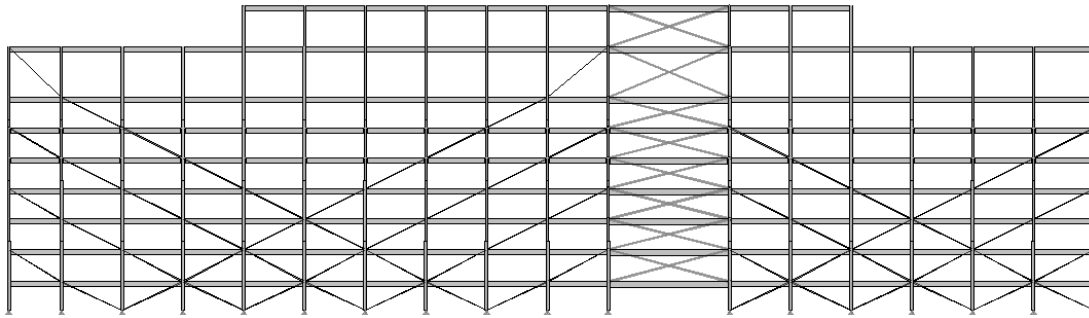

FIGURE 20: THIS IS AN IMAGE OF THE NEW X-FRAME LATERAL RESISTING SYSTEM IN THE NEW YORK POLICE ACADEMY BASED ON THE THESIS LATERAL SYSTEM OPTIMIZATION.
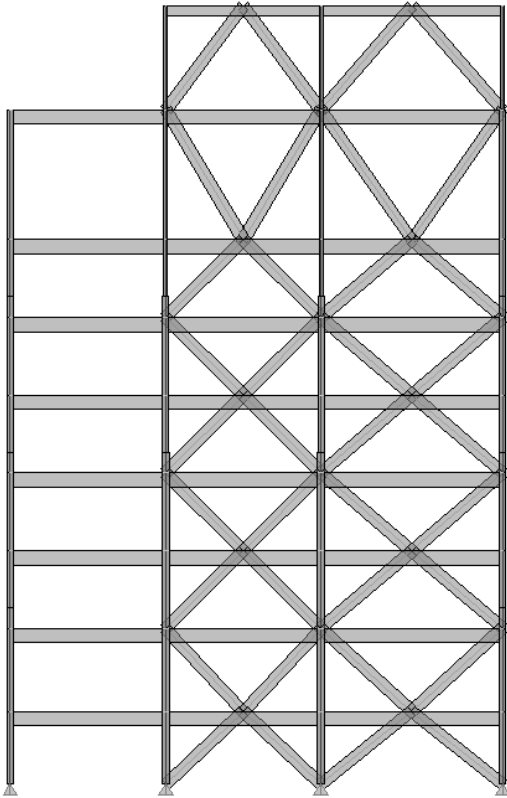
Y-FRAME

FIGURE 21: THIS IS AN IMAGE OF THE Y-FRAME LATERAL RESISTING SYSTEM IN THE NEW YORK POLICE ACADEMY.

The lateral resisting system in the Y-Frame was altered from HSS concentrically braced frames to W-shape concentrically braced chevron frames for constructability and fabrication purposes. The change from HSS to W-shapes was done because HSS shapes are more difficult to fabricate and construct in the field. The alignment was altered from concentric framing to chevron framing because the long spans created peculiar gusset plates. The use of the chevron bracing allowed The New York Police Academy the ability to use only four Y-Frames with symmetric formations. The original design called for concentrically braced connections in all ten Y-Frames, which were spread throughout the structure in an inconsistent pattern. This alteration replaced a total of 168 concentrically braced connections with 136 chevron braced connections. A more in depth look at the connections is on pages 39 through 45. The placement of the two interior frames considered the architectural plans as they are aligned with partitions between auditoriums as not to interrupt the interior space. The exterior frames are along the exterior of the building where there are no windows.

GRADUATE COURSE INTEGRATION

The thesis topic chosen directly correlates to two graduate level courses. This building was modeled and analyzed in ETABS, which reflects the information that was taught in AE 597A, Computer Modeling. This model was used to evaluate the building under wind and seismic loads. This model can be viewed on page 34. The alteration to steel connections also includes material that was taught in AE 534, Steel Connections.

The following section explains how typical connections were designed and detailed. There were three typical connections that were designed, one in the X-Frame and two in the Y-Frame. These connections were all designed to resist lateral forces. To view connection details please refer to pages 43 through 45.

X-FRAME CONCENTRICALLY BRACED CONNECTION

The system redesign utilized concentrically braced frames. The braces are in line with the center of the beam to column connection and thus induce no moment on the connection. Because the controlling load is not extreme the bracing members carry relatively light forces. This allows the diagonal braces to be smaller members because the load they must resist is not large. The concentrically braced frame is demonstrated in Figure 22 below.

Figure 22 shows the load path through an interior concentrically braced connection. The red arrow indicates the controlling lateral load acting on the lateral system of the New York Police Academy while the green arrows show the loads within the connection. Notice
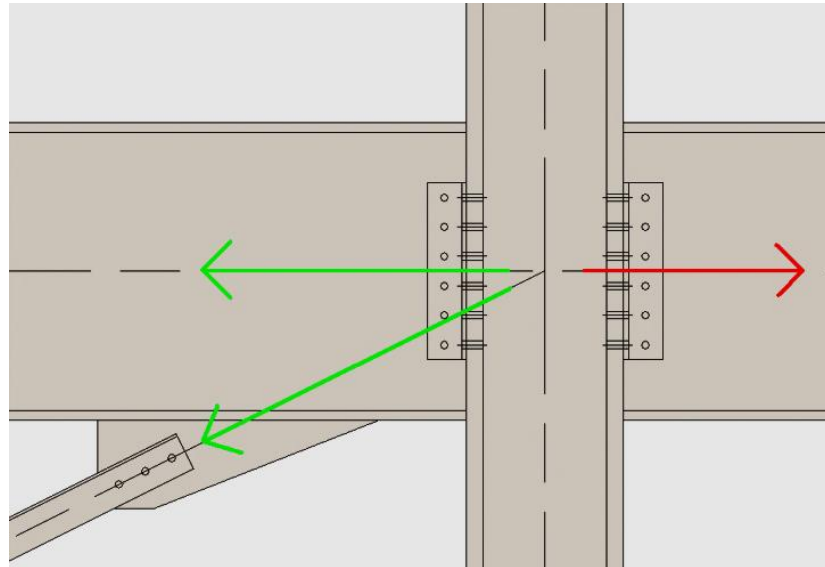
FIGURE 22: THIS IMAGE SHOWS THE LATERAL LOAD PATH THROUGH AN CONCENTRICALLY BRACED CONNECTION.

that the brace is in tension. This was done intentionally in order to maximize the strength of the member. If the members were designed in compression, buckling would control and larger members would be needed. A more in depth explanation of this is in the "Diagonally Braced Connection" section on page 41.

Y-FRAME CHEVRON BRACED CONNECTION

In the Y-direction a similar problem arose where the 30' bays with 15' typical floor height yielded inordinately proportioned gusset plates when using the Uniform Force Method. However, in this direction there is an alternative solution. Chevron bracing is used at 15' increments so that the Uniform Force Method can still be applied. In this instance the gusset-to-beam connection was designed as if each brace were the sole brace resisting the lateral load. Each brace framed into the ideal centroid locations in order to avoid inducing a moment on the gusset-to-beam interface. This method is similar to Uniform force Method Special Case 3, No Gusset-to-Column Web Connection.

The thesis redesign replaced the 168 concentrically braced HSS connections with 136 chevron braced W-shape connections. The chevron system was chosen because the bay sizes were large. The W-shapes were chosen because they are cheaper to fabricate than HSS shapes. HSS shapes are typically chosen for their aesthetic appeal. However, in this project, those frames are hidden within the walls. Because HSS shapes are aesthetically pleasing the double bay of HSS bracing in the X-direction remained intact during the redesign.

Please note that the beam-to-column connection was not designed because it was not a topic of interest. Instead the focus was on the "inverted v" or chevron connection. Please refer to Appendix D for detailed calculations of the chevron connection design. The load path of the chevron braced connection can be seen in Figure 23 below.
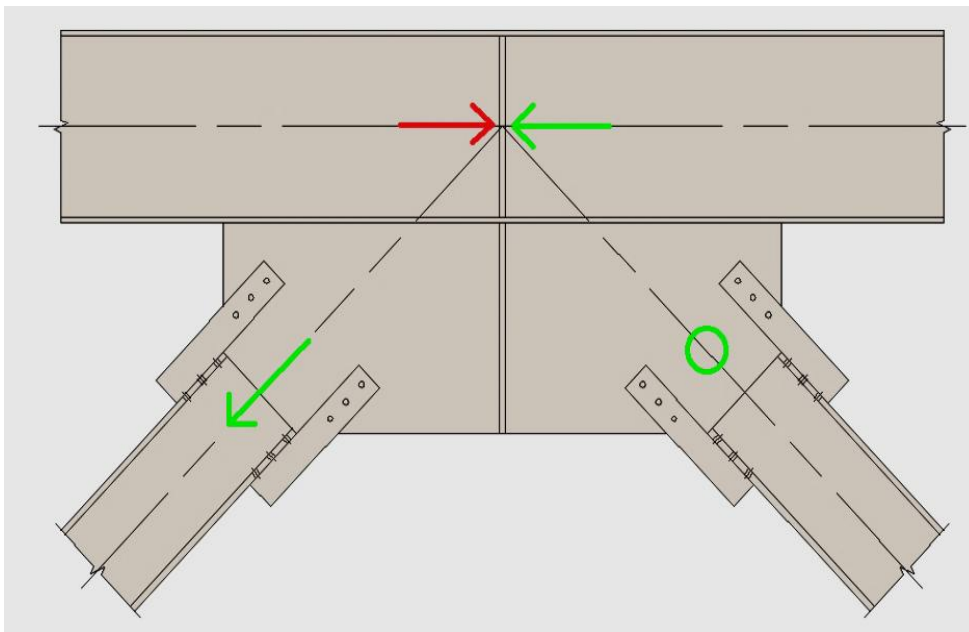

FIGURE 23: THIS IMAGE SHOWS THE LATERAL LOAD PATH THROUGH A CHEVRON BRACED CONNECTION.

Y-FRAME DIAGONALLY BRACED CONNECTION

The diagonally braced connection was designed by using the Uniform Force Method. This method eliminates moments on the working plane (noted as W.P. in Figure 24 below) by selecting a connection geometry where moments do not occur on the three connection interfaces. The three interfaces are beam-to-column, gusset-to-column and gusset-to-beam. By eliminating moments it allows the connection to be designed for strictly shear and tension. The Uniform Force Method takes into account the beam depth, column depth, bracing angle, distance to the centroid of the gusset-to-beam connection and the distance to the centroid of the gusset-to-column connection.
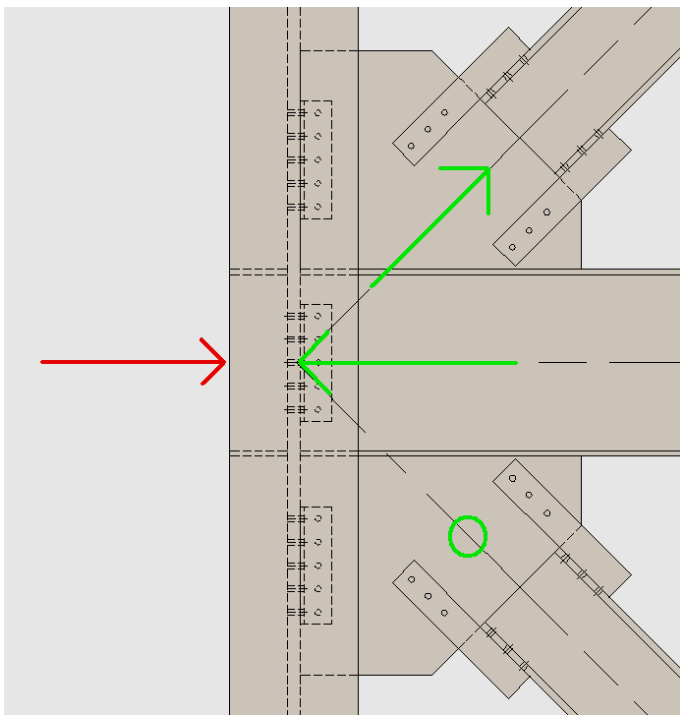

FIGURE 24: THIS IMAGE SHOWS THE LATERAL LOAD PATH THROUGH AN HSS CROSS BRACED CONNECTION.

Figure 24 to the left shows the load path through an exterior concentrically braced connection (which frames into the chevron connection in the beam above. Please refer to Figure 23 on page 40 for an image of the chevron 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 W-shaped brace at the top is in tension while the lower W-shaped brace and gravity member are in compression. As in the X-Frame, twice as many tension members as needed were provided. Assuming that any member needed to resist this load in compression would buckle and therefore be a zero force member. This permits all bracing to be designed as tension-only members and permit them to have a smaller cross section than if they were designed in compression (Geschwindner pp. 93-94).

Special Case 2 of the Uniform Force Method, Minimizing Shear in the Beam-to-Column Connection (AISC p. 13-7), is used in this connection design. This connection was redesigned for constructability and fabrication. The HSS concentrically braced connections are more complicated to fabricate and install than the W-shaped diagonally braced connection shown in Figure 27 on page 45.

INTERFACE FORCES		
INTERFACE	SHEAR (KIPS)	AXIAL (KIPS)
GUSSET-TO-COLUMN	36.5	4.4
GUSSET-TO-BEAM	63.3	95.3
BEAM-TO-COLUMN	31.9	93.8

TABLE 13: UNIFORM FORCE METHOD INTERFACE FORCES

INTERFACE FORCES AFTER APPLYING SPECIAL CASE 2			
INTERFACE	SHEAR (KIPS)	AXIAL (KIPS)	MOMENT (FT-K)
GUSSET-TO-COLUMN	99.8	4.4	0.0
GUSSET-TO-BEAM	0.0	95.3	95.0
BEAM-TO-COLUMN	27.9	93.8	0.0

TABLE 14: UNIFORM FORCE METHOD INTERFACE FORCES AFTER APPLYING SPECIAL CASE 2

The tables above show the interface forces that are calculated using the Uniform Force Method and the changes that occur when Special Case 2 is applied. Note that this 95.0 ft-k moment added to the internal beam moment of 702 ft-k is still less than the 1170 ft-k designed moment of the W30x99

As stated in the previous section the lateral bracing in this system was designed for tension only. All bracing members can be assumed to buckle having a force of zero and the system would still perform adequately.

CONNECTION DETAILS

Figures 25, 26 and 27 show the connection details. These details provide information on how to construct each connection. Calculations supporting these detail configurations can be viewed in Appendix D.
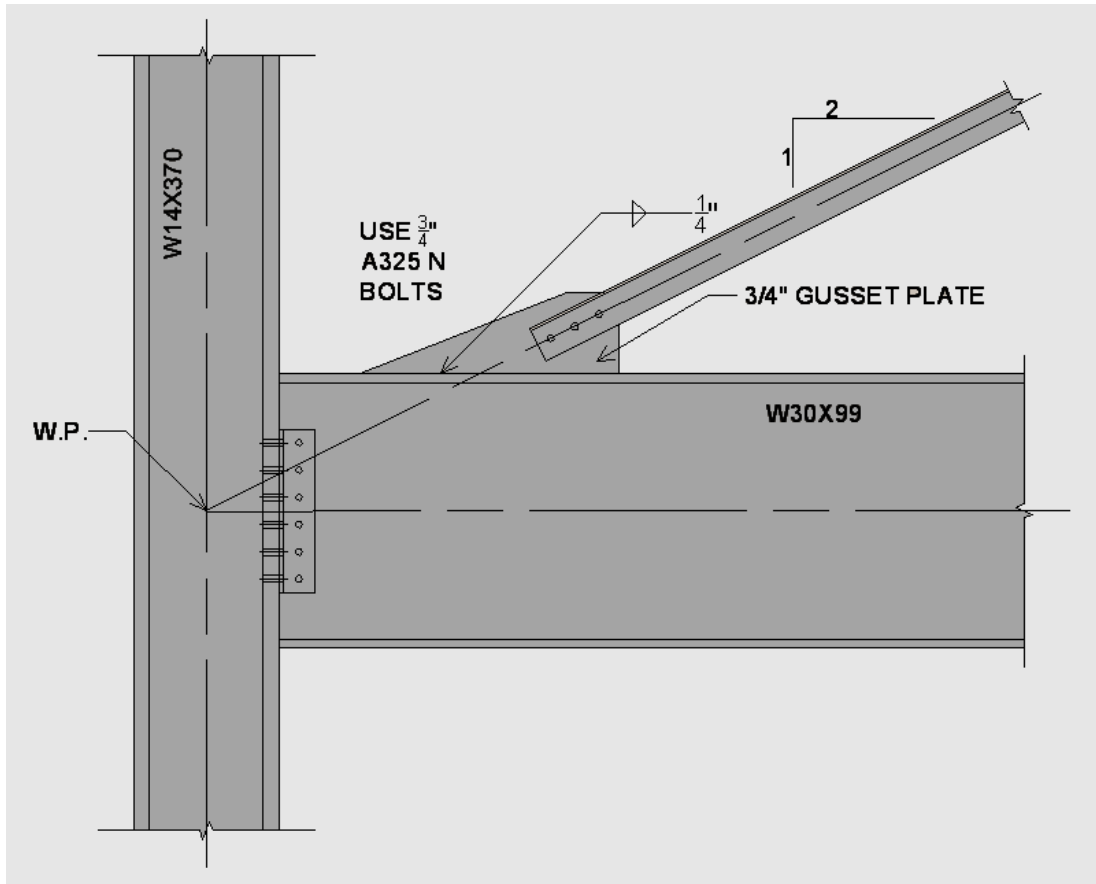
FIGURE 25: CHEVRON BRACED CONNECTION DETAIL

The axial load in each concentric brace is 45.9 kips. One row of three ¾" φ A325 bolts in double shear was needed to connect the gusset plate to the brace. The gusset plate is ½" thick and requires a ¼" fillet weld according to AISC minimum size of fillet welds (Table J2.4). The brace is in line with the working plane (indicated as W.P. in Figure 25) in order to eliminate the moment that would be induced on the connecting beam.

Y-FRAME CHEVRON BRACED CONNECTION

This connection was designed for an axial load of 140.9 kips. The brace-to-gusset and the gusset-to-beam weld size were designed to be ½" fillet welds. The gusset plate it bolted to two double angles (2L4x4x½) which hold the W14x82 brace in place. The braces form a 45° angle with the connecting girders. ¾" gusset and stiffener plates are used in this connection. Please refer to Figure 26 to view this detail.

FIGURE 16: CHEVRON BRACED CONNECTION DETAIL

Y-FRAME DIAGONALLY BRACED CONNECTION

This connection was designed for an axial load of 140.9 kips and has very similar characteristics to the chevron braced connection on the previous page. The brace-to-gusset and the gusset-to-beam weld size were designed to be ½" fillet welds. The gusset plate it bolted to two double angles (2L4x4x½) which hold the W14x82 brace in place. The braces form a 45° angle with the connecting girder and column. ¾" gusset and stiffener plates are used in this connection. Please refer to Figure 27 to view this detail.
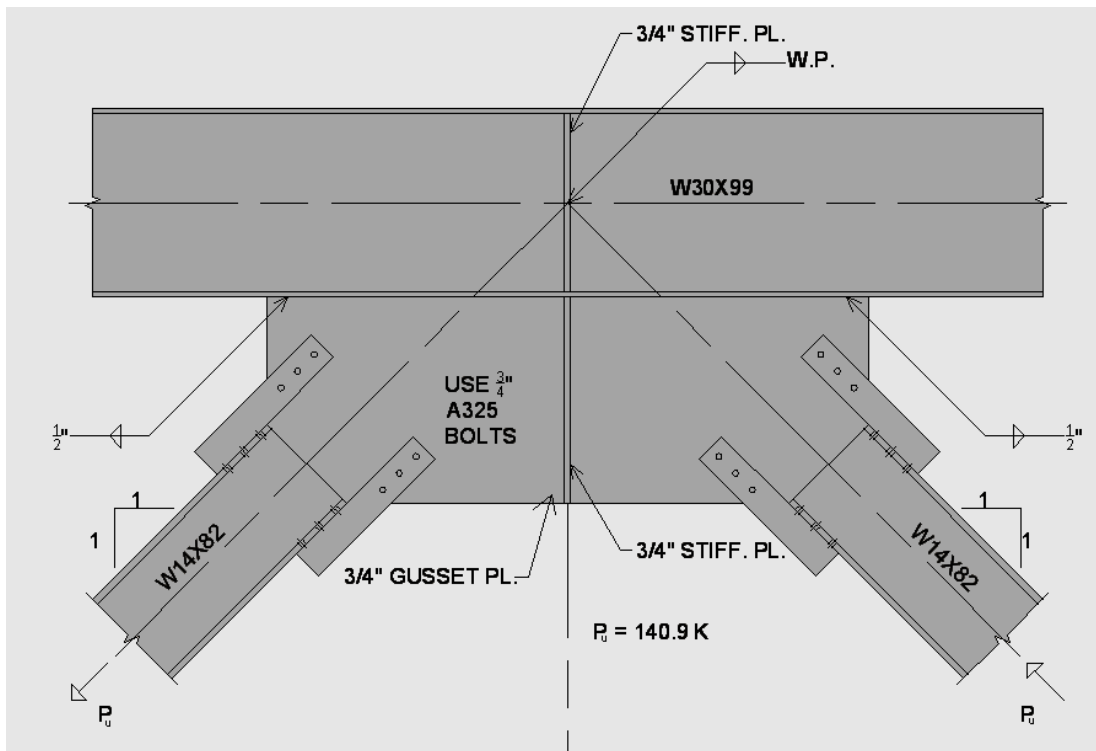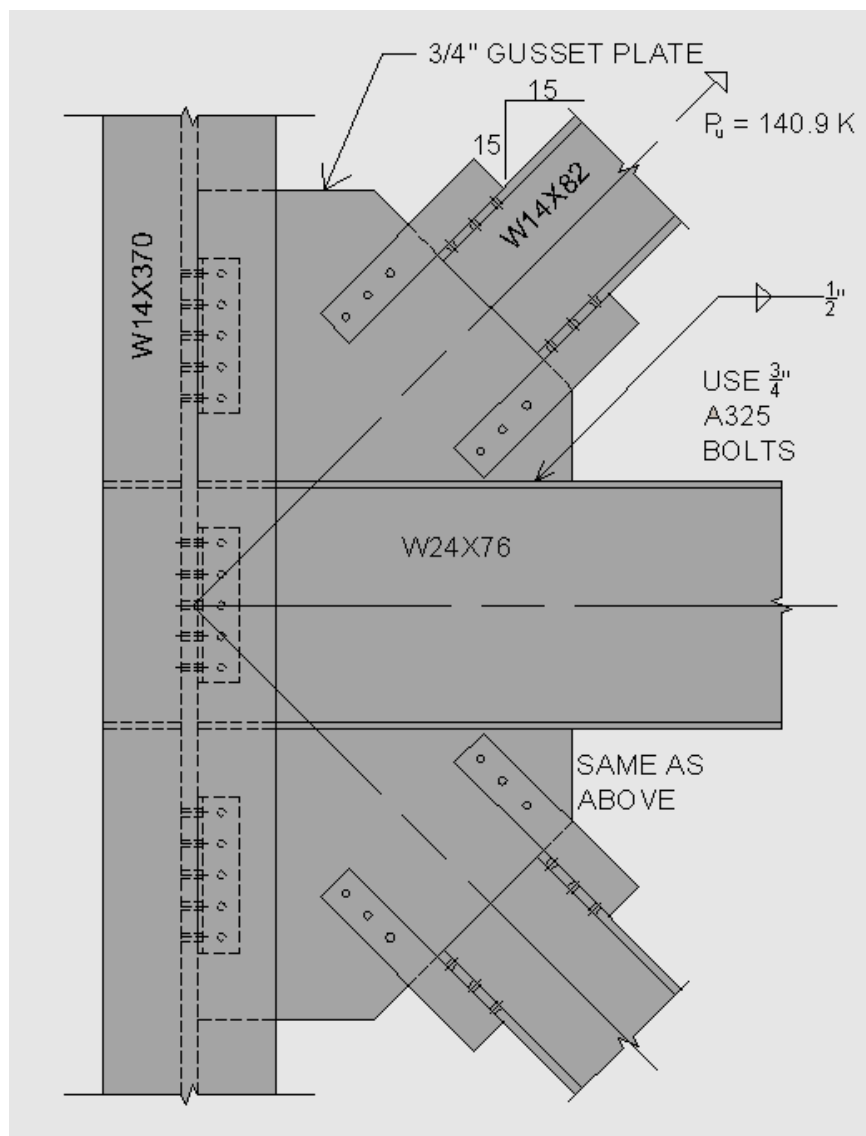

FIGURE 27: DIAGONALLY BRACED CONNECTION DETAIL

THESIS DESIGN ANALYSIS

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 15 and 16 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 direct shear in the X-direction was altered due to the change in connections. The interior frames received far less load in the thesis design. This is because the lateral bracing system is focused on the two exterior bays.

DIRECT SHEAR – EAST/WEST – X-FRAMES (KIPS)					
FLOOR	FORCE	FRAME 1	FRAME 2	FRAME 3	FRAME 4
BULKHEAD	15.3	7.0	0.6	0.6	7.0
ROOF	22.3	10.3	0.9	0.9	10.3
8	17.2	7.9	0.7	0.7	7.9
7	19.5	9.0	0.8	0.8	9.7
6	18.0	8.3	0.7	0.7	8.3
5	16.4	7.5	0.7	0.7	7.5
4	14.7	6.8	0.6	0.6	6.8
3	12.5	5.8	0.5	0.5	5.8
2	9.0	4.1	0.4	0.4	4.1
Σ	144.9	66.7	5.8	5.8	66.7

TABLE 15: THIS TABLE SHOWS THE DIRECT SHEAR IN EACH X-FRAME BY FLOOR.

The direct shear in the Y-direction was also altered from having HSS concentrically braced frames spread throughout ten different frames to having four chevron braced frames with W-Shaped members. Because of this only these four frames receive lateral load. Each frame receives 25% of the total load which explains why each floor load is the same. This can be seen in Table 16 below.

DIRECT SHEAR – NORTH/SOUTH – Y-FRAMES (K)					
FLOOR	FORCE	FRAME A	FRAME E	FRAME O	FRAME S
BULKHEAD	66	16.5	16.5	16.5	16.5
ROOF	113	28.3	28.3	28.3	28.3
8	85	21.3	21.3	21.3	21.3
7	72	18.0	18.0	18.0	18.0
6	63	18.8	18.8	18.8	18.8
5	51	12.8	12.8	12.8	12.8
4	39	9.8	9.8	9.8	9.8
3	28	7.0	7.0	7.0	7.0
2	18	4.5	4.5	4.5	4.5
Σ	535.0	133.8	133.8	133.8	133.8

TABLE 16 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 (FT)	Y(FT)	X-DIFFERENCE	Y-DIFFERENCE
CENTER OF MASS	267.9	48.5	-	-
CENTER OF PRESSURE	267.8	49	0.1	0.5
CENTER OF RIGIDITY	267.0 (269.0)	47.3 (46.6)	0.9	1.2

TABLE 17: 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 C. Please note for the center of rigidity that the numbers in parentheses are the numbers from the original design while the numbers not in parentheses represent the thesis redesign.

Table 18 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:

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

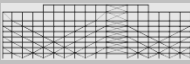These values are also shown in Table 18 on the next page for all stories.

FRAME STIFFNESSES				
	X-FRAME		Y-FRAME	
FLOOR	δ (IN)	STIFFNESS, k	δ (IN)	STIFFNESS, k
BULKHEAD	1.435	696.9	0.875	1142.9
ROOF	1.319	758.2	0.754	1326.3
8	1.036	965.3	0.517	1934.2
7	0.843	1186.2	0.414	2415.5
6	0.684	1462.0	0.312	3205.1
5	0.538	1858.7	0.233	4291.8
4	0.413	2421.3	0.155	6451.6
3	0.298	3355.7	0.096	10416.7
2	0.175	5714.3	0.034	29411.8

TABLE 18: 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 19 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	134.4	0.46	10.8	567.6	148198	0.06
2	N/S	134.4	0.04	10.8	207.6	1724	0.00
3	N/S	134.4	0.04	10.8	152.4	929	0.00
4	N/S	134.4	0.46	10.8	572.4	150715	0.06
A	E/W	385.0	0.25	14.4	3204	2566404	4.18
E	E/W	385.0	0.25	14.4	1812	820836	0.76
O	E/W	385.0	0.25	14.4	1782	793881	0.72
S	E/W	385.0	0.25	14.4	3228	2604996	4.27
					J=	7087683	

TABLE 19: THIS TABLE SHOWS THE SAMPLE CALCULATION FOR TORSIONAL SHEAR FOR THE 6TH FLOOR.

The torsion in the X-Direction is negligible. The torsion in the Y-direction is marginal with a little over 4 kips acting on frames A and S. This is most likely due to the change in rigidity from the original design to the thesis redesign. The torsional shear for the 6th floor was calculated to show that torsional shear has very little effect on the net shear of the building.

LATERAL MOVEMENT

In the thesis redesign of the New York Police Academy drift controlled the member sizes in the Y-Frame. If a smaller W-shape member was used then drift limitations would be breached. The lateral movement of the thesis redesign is tabulated below. The performance is comparable with the exception of the wind in the Y-direction. The maximum displacement in the Y-direction for the original design was 0.767" compared to 2.013". Each story displacement remains within the necessary floor and total displacement criteria. For a more direct comparison please refer to Appendix E to view the original design lateral movement.

	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	0.071	4.8	0.061	0.6	0.049	4.8	0.328	0.6
ROOF	0.103	6	0.098	0.75	0.064	6	0.427	0.75
8	0.069	3.6	0.070	0.45	0.033	3.6	0.207	0.45
7	0.074	3.6	0.069	0.45	0.036	3.6	0.219	0.45
6	0.081	3.6	0.075	0.45	0.032	3.6	0.180	0.45
5	0.089	3.6	0.073	0.45	0.033	3.6	0.190	0.45
4	0.096	3.6	0.076	0.45	0.028	3.6	0.170	0.45
3	0.109	3.84	0.094	0.48	0.031	3.84	0.192	0.48
2	0.117	3.36	0.144	0.42	0.019	3.36	0.118	0.42
Σ	0.809	36	0.760	4.5	0.325	36	2.031	4.5

TABLE 20: THIS TABLE SHOWS THE STORY DRIFTS AND DISPLACEMENTS 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.

FOUNDATION IMPACTS

OVERTURNING AND CHANGE IN BUILDING WEIGHT

The critical overturning moment results in the direction with least depth. In this case it is 95', 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.

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.

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 = 53,919K x 95'/2 =			2,561,152.5$^{'k}$

TABLE 21: THIS TABLE SHOWS THE MAGNITUDE OF THE MAXIMUM OVERTURNING MOMENT AND THE RESISTING

Calculations for overturning and resisting moments are in Table 21 above.

In the X-direction, moment connections were changed to the exterior concentrically braced connections. This brace was a 2L4x4x¼, which weighs approximately 12.2 pounds per linear foot (MC² Construction Reference). This added weight was miniscule when compared to each total floor weight accounting for a total of 28 kips. In the Y-direction 136 W14X82 replaced 168 various HSS members. The total weight added was approximately 2 kips. When the building is over 50,000 kips an added 30 kips has a minimal effect on the building as a whole. Because the net change in weight is marginal the foundation impact on the 100 ton piles is essentially negligible. Furthermore, since the weight was added to the building it creates more moment to resist overturning. Hand calculations for this can be seen in Appendix F.

ARCHITECTURAL BREADTH

Moment connections were used to resist lateral loads in the original X-Frame design so that the entire façade in the North and South facing directions could be glazed. Concentrically braced frames are used in the thesis X-Frame redesign which means that the façade needed to be changed in order to accommodate the new lateral resisting system. If it were not changed then the diagonal bracing would be seen through the existing glass façade windows. This would not be aesthetically pleasing because the sight of the structural system through the windows would seem unintentional. Instead the structure should be emphasized.

The Y-Frame connection alteration did not affect the East and west facades.

FAÇADE REDESIGN

DESIGN PROCESS

Window shapes and sizes were altered to maintain aesthetics and emphasize the change in structure that is beyond the façade. In the original design there were aluminum louvers used to channel light into and out of the New York Police Academy (this can be seen in Figure 3 on page 9). These louvers remain because not only is it aesthetically pleasing, but it channels light and gains LEED certification points. The aluminum louvers are included in the new design, but it reveals the structure behind it while keeping its aesthetic integrity. Because the lateral bracing is located throughout the X-Frame the whole façade must be redesigned to accommodate the new bracing. This can be seen in the North and South Elevations located in Figures 28 and 29 below.
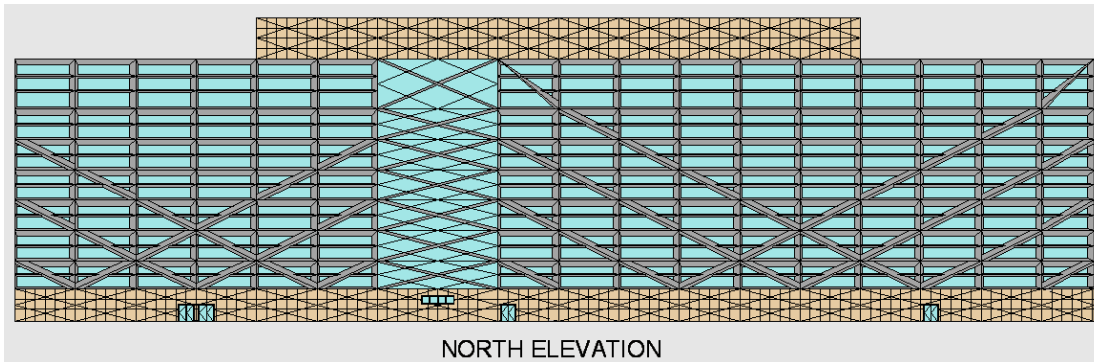

NORTH ELEVATION

FIGURE 28: THE NEW YORK POLICE ACADEMY REDESIGN NORTH ELEVATION
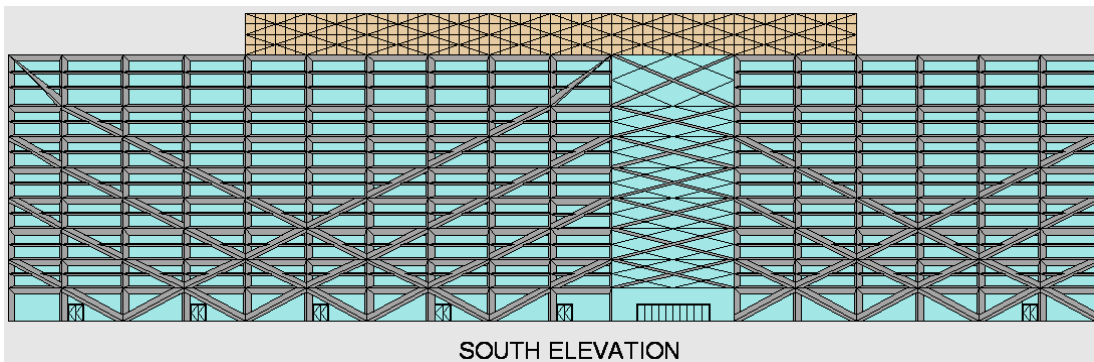

SOUTH ELEVATION

FIGURE 29: THE NEW YORK POLICE ACADEMY REDESIGN SOUTH ELEVATION

To view the changes that were made between the original façade design and the thesis redesign please refer to Appendix G.
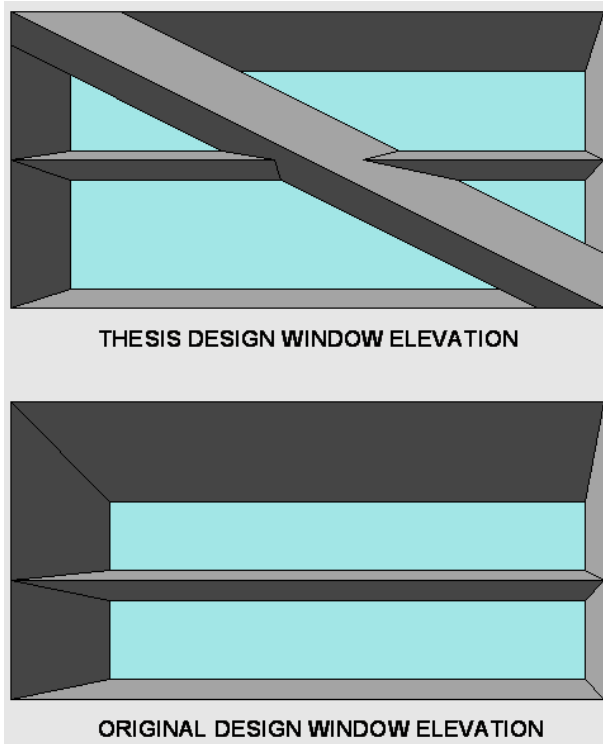
FIGURE 20: THIS PICTURE COMPARES THE THESIS WINDOW REDESIGN (TOP) TO THE ORIGINAL WINDOW DESIGN (BOTTOM)

In order to leave the interior space unaffected the braced frames were placed solely on the exterior two frames. This is undesirable from the architectural standpoint because it interrupts the purity of the exterior glazing. However this design embraces the architectural integrity by accentuating the structure with additional aluminum louvers as seen in the window detail in Figure 30 to the left. This can be compared to the original design directly below it.

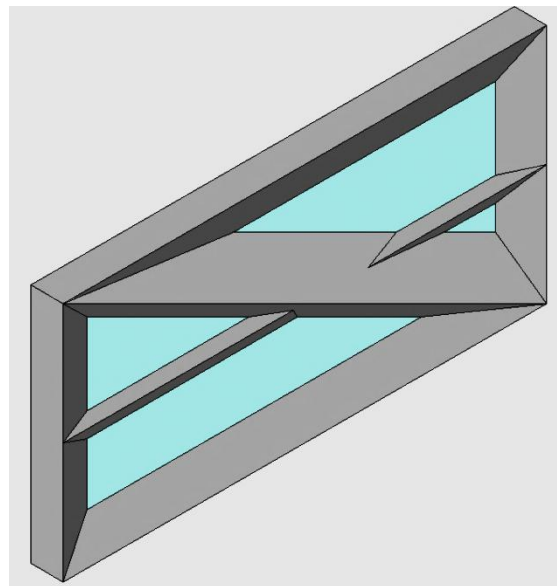This image shows how the architectural aesthetics and structural system are integrated to form the façade of the New York Police Academy. Please note that these are not the actual colors of the building. Colors were added to emphasize the glazing and aluminum louvers.

It appears as if these aluminum louvers are now blocking sunlight from entering instead of channeling them, however, this is not the case. The old windows used larger louvers around the perimeter with approximately 170SF of glazing per window bay. The perimeter was reduced when the cross louvers were input into the design. The windows with cross louvers allow for approximately 200SF of glazing per window bay actually allowing more natural light to be channeled into the building. The new window can be seen in Figure 31 to the right.

FIGURE 31: THIS IS AN ISOMETRIC VIEW OF THE NEW YORK POLICE ACADEMY WINDOW REDESIGN.

CONSTRUCTABILITY

Fabricating glazing in various triangular and quadrilateral shapes is extremely expensive. The resulting window assembly can be seen in Figure 32 below. In order to avoid this issue the windows will remain rectangular (1) with the perimeter aluminum louvers (2) holding the glass in place while the inner aluminum louvers (3) rest on the exterior of the glass. The structural bracing is behind the glass on the interior of the building. This will increase the amount of total glass used on the North and South facades while limiting the fabrication costs immensely. These windows will be unitized so that it does not have a big impact on the construction schedule because the additional diagonal louver is assembled during fabrication.
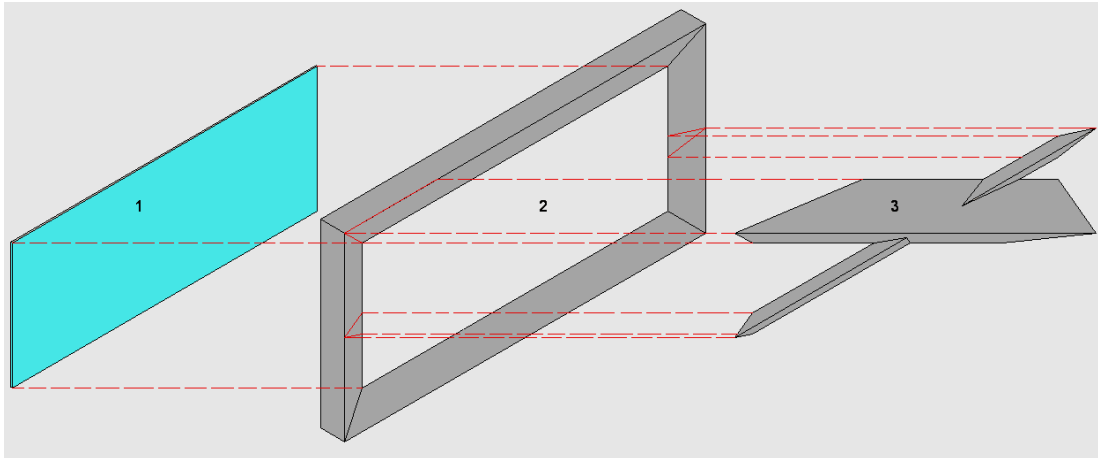

FIGURE 32: THIS FIGURE SHOWS THE ORDER THAT THE WINDOW SYSTEM WOULD BE ASSEMBLED DURING FABRICATION.

CONSTRUCTION MANAGEMENT BREADTH

The change in connections for the structural aspect of this project directly correlates to the construction facet of the building design. A time and cost analysis was compiled in this section to demonstrate the changes that have taken place in the New York Police Academy redesign.

To determine the effects that the new connections have on fabrication and labor was a three-part process. First the connections were designed. Then experienced professionals within the industry were contacted to inspect and price both the original and proposed connections. The fabrication, labor and material costs were taken into account and then compared and contrasted. In addition to the information and guidance provided by experienced professionals, RSMeans 2011 Building Construction Cost Data Book was also used in comparison calculations.

COST STUDY

FABRICATION AND MATERIAL COSTS

The following table provides the necessary information regarding the material and fabrication needs for each connection used in the New York Police Academy. The costs associated for each fabrication and material "needs" are also listed in Table 22 below.

CONNECTION	TYPICAL MOMENT	TYPICAL HSS CONCENTRICALLY BRACED	TYPICAL W-SHAPE CHEVRON / DIAGONALLY BRACED	TYPICAL DOUBLE ANGLE CONCENTRICALLY BRACED
FABRICATION NEEDS	CUT: • PLATE • ANGLE PUT HOLES IN: • BEAM • ANGLE • PLATE	FABRICATE: • CONNECTION CLIPS • GUSSET PLATES NOTCH AND WELD HSS BRACES	CUT: • GUSSET PLATE • ANGLE PUT HOLES IN: • BEAM • ANGLE • PLATE WELD	CUT: • PLATE • ANGLE PUT HOLES IN: • ANGLE • PLATE WELD
FABRICATION COST	19 HOURS @ $69.14	32 HOURS @ $69.14	18 HOURS @ $69.14	26 HOURS @ 69.14
MATERIAL NEEDS	• BOLTS • PLATE • CONNECTION CLIP	• BOLTS • PLATE • ANGLES	• BOLTS • GUSSET PLATES • ANGLES	• GUSSET PLATE • BOLTS
MATERIAL COST	$259.80	$1,261.70	$631.43	$782.88

TABLE 22: FABRICATION AND MATERIAL NEEDS/COSTS

LABOR COSTS

According to RSMeans 2011 Assembly Crew E-2 with 4 steel workers on each beam would construct the necessary connections. Labor costs $48.55 per hour for crew E-2. When taking into account that this project is in New York City the labor must be multiplied by a 1.433 location modifier making the costs $69.57 per hour. The labor needs are shown in Table 23 below. The full effects of labor costs on the construction of all lateral bracing connections for the New York Police Academy are tabulated in Tables 24 and 25 on pages 61 and 62.

CONNECTION	TYPICAL MOMENT	TYPICAL HSS CONCENTRICALLY BRACED	TYPICAL W-SHAPE CHEVRON BRACED	TYPICAL DOUBLE ANGLE CONCENTRICALLY BRACED
LABOR NEEDS	INSTALL BOLTS WELD ANGLE TO COLUMN	BOLT UP CONNECTIONS	BOLT UP CONNECTIONS	BOLT UP CONNECTIONS
LABOR COST	7 HOURS @ $69.57	5 HOURS @ $69.57	5HOURS @ $69.57	5 HOURS @ $69.57

TABLE 23 – LABOR NEEDS/COSTS

SCHEDULING

Although an actual schedule could not be provided for this project, the effects of the New York Police Academy redesign are tabulated below. This data was obtained from contacts at Baltimore Steel Erectors and Steel Fab Enterprises. The thesis connections save much more time both in the fabrication shop and in the field. The amount of labor time affects the schedule more than fabrication because fabrication can start before construction begins. Other trades are dependent on the amount of time that is spent in the field assembling connections. A breakdown of exact hours can be viewed in Appendix H. The thesis design demands 1,320 labor hours while the original design demands 4,116 labor hours. The thesis design accounts for approximately 68% savings in time. This information is can be viewed graphically in Figure 33 below.
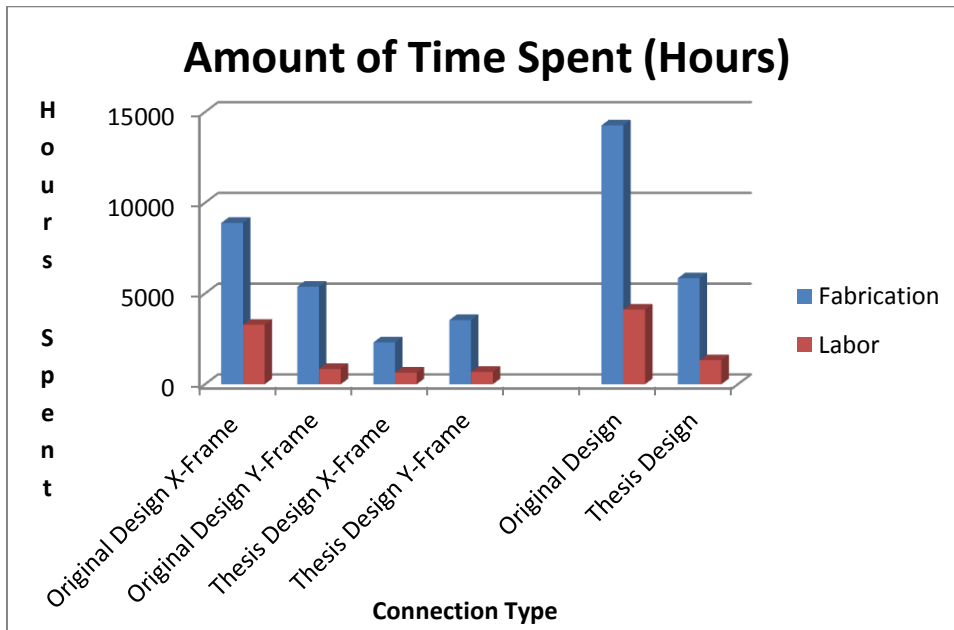

FIGURE 33: THIS FIGURE COMPARES THE TIME SPENT FOR LABOR AND FABRICATION BETWEEN THE ORIGINAL DESIGN AND THE THESIS REDESIGN.

TOTAL SYSTEM SAVINGS

All of the above information can be consolidated into Tables 24 and 25 below to calculate the total system costs for the original design of the New York Police Academy and the thesis redesign. These tables incorporate material, labor and fabrication costs into the total system savings.

ORIGINAL DESIGN

This table summarizes the lateral system costs for the original design of the New York Police Academy. Please notice that there are 468 moment connection and 168 HSS connections totaling **$1,606,325.97.**

ORIGINAL DESIGN X-FRAME MOMENT CONNECTION CALCULATION				
	COST	MULTIPLIER	HOURS	SUBTOTAL
MATERIAL	$223.00	1.165	N/A	$259.80
LABOR	$48.55	1.433	7	$487.01
FABRICATION	$55.00	1.257	19	$1,313.57
			SUBTOTAL	$2,060.37
CONNECTIONS (#)	468		TOTAL	$964,250.84
ORIGINAL DESIGN Y-FRAME HSS CONNECTION CALCULATION				
	COST	MULTIPLIER	HOURS	SUBTOTAL
MATERIAL	$1,083.00	1.165	N/A	$1,261.70
LABOR	$48.55	1.433	5	$347.86
FABRICATION	$55.00	1.257	32	$2,212.32
			SUBTOTAL	$3,821.88
CONNECTIONS (#)	168		TOTAL	$642,075.13
TOTAL ORIGINAL SYSTEM COST				**$1,606,325.97**

TABLE 24: ORIGINAL SYSTEM COST CALCULATIONS

This table summarizes the lateral system costs for the thesis redesign of the New York Police Academy. Please notice that there are 128 moment connection and 136 HSS connections totaling **$827,408.92**. Also note that in the thesis redesign the additional aluminum paneling that is needed is accounted for.

THESIS DESIGN X-FRAME CONCENTRIC CONNECTION CALCULATION				
	COST	MULTIPLIER	HOURS	SUBTOTAL
MATERIAL	$542.00	1.165	N/A	$631.43
LABOR	$48.55	1.433	5	$347.86
FABRICATION	$55.00	1.257	18	$1,244.43
			SUBTOTAL	2,223.72
CONNECTIONS (#)	128		TOTAL	**$284,636.26**
THESIS DESIGN Y-FRAME W-SHAPE CONNECTIONS CALCULATION				
	COST	MULTIPLIER	HOURS	SUBTOTAL
MATERIAL	$672.00	1.165	N/A	$782.88
LABOR	$48.55	1.433	5	$347.86
FABRICATION	$55.00	1.257	26	$1,797.51
			SUBTOTAL	$2,928.25
CONNECTIONS (#)	136		TOTAL	**$398,242.10**
ADDITIONAL ALUMINUM PANELING				
	COST	MULTIPLIER	HOURS	SUBTOTAL
MATERIAL	$791.19	1.165	N/A	$921.74
LABOR	$48.55	1.433	N/A	$0.00
FABRICATION	$55.00	1.257	3	$ 207.41
			SUBTOTAL	$1,129.15
CONNECTIONS (#)	128		TOTAL	**$144,530.56**
TOTAL THESIS SYSTEM COST				**$827,408.92**

TABLE 25: THESIS SYSTEM COST CALCULATIONS

By comparing the above two tables it is apparent that the thesis redesign is **$778,917.05** cheaper including the façade redesign. This equates to roughly **48% savings**. In addition to the cost savings there are also savings in scheduling. The thesis redesign saves **2,796** man hours in the field. This equates to approximately **68%** less man hours.

CONSTRUCTION MANAGEMENT CONCLUSION

It is clear from the above analysis that the goal of the New York Police Academy redesign was successful from a construction standpoint. The redesign saved both time and money making all of the changes worthwhile.

FINAL CONCLUSIONS

By altering structural, construction and architectural aspects of the New York Police Academy, this project demonstrates the well rounded building-engineers that are created by the Pennsylvania State University's renowned Architectural Engineering program. In addition, the alteration of connections and the use of computer modeling demonstrate the skills learned in courses at the graduate level.

The goals of this thesis project were to alter the lateral system of the New York Police Academy, examine its effects on the architecture and account for them. Once this was accomplished the effects of the new system on both the overall cost and scheduling of construction were analyzed.

The original lateral system design of the New York Police Academy used moment frames in the X-direction and concentrically braced HSS frames in the Y-direction.

The new lateral system removed the 468 moment connections from the original design and added 128 concentrically braced connections to resist lateral load in the X-direction. It also removed the 168 concentrically braced HSS connections and replaced them with 136 chevron braced W-shape connections.

The alteration of the lateral system in the X-direction had vast effects on the architecture of the New York Police Academy. Instead of having large open glazing there are now braces in their places. Although the structural braces are behind the windows on the interior of the building, they would be seen through these large open window spaces. Rather than hide the structure behind the façade the thesis façade redesign embraced it by accentuating it with aluminum paneling.

The construction studies of cost and scheduling provide optimal results. The original lateral system costs $1,606,325.97. The thesis lateral system in conjunction with the changes made to the façade of the New York Police Academy, would cost only $827,408.92, a savings of more than 48%. The redesign would also limit the number of man hours needed to assemble the lateral system from 4,116 man hours in the original design to 1,320 man hours. This accounts for approximately 68% savings in time, allowing the building to be constructed faster.

In sum, the thesis redesign would save both time and money without sacrificing the building aesthetics.

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APPENDIX A: LIST OF FIGURES

APPENDIX B: LIST OF TABLES

APPENDIX C: LATERAL SYSTEM DESIGN

WIND DETERMINATION CALCULATIONS

Use ASCE 7-10 - MWFRS (Directional Procedure)

Table 27.2-1

1. Risk Category:

Institution/Academy ⇒ $\boxed{II}$ (Table 1.5-1)

2. Basic Wind Speed (Table 26.5-1A)

Located in Queens, NY ⇒ $\boxed{V = 120 \text{ MPH}}$

3. Wind Directionality Factor (§26.6, Table 26.6-1)

MWFRS ⇒ $\boxed{k_d = 0.85}$

Exposure Category (§26.7)

Urban Area ⇒ $\boxed{\text{Exposure B}}$

Topographic Factor (§26.8, Table 26.8-1)

No hills/escarpments ⇒ $\boxed{k_{zt} = 1.0}$

Gust Effect Factor (§26.9)

Limitations for Approximate Natural Frequency (§26.9.2.1)

1. Building Height = ~~150~~ 150' < 300' ∴ ok

2. $L_{eff} = \dfrac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = \dfrac{[130(536) + 20(390)]}{150}$

   = 508'

   = 4 × 508 > 150 ∴ ok

Approximate Natural Frequency

Steel w/ HSS lateral bracing ~~moment connections~~

$n_a = 75/n \left\{\dfrac{3.50(in)}{536(390 \times in)}\right\} / 536 = \boxed{142 = h}$ = Mean Roof Height

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

(left margin, vertical text) North / South Wind

3 CONTINUED

GUST EFFECT FACTOR

FLEXIBLE BUILDINGS (26.9.5)

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

$$I_{\bar{z}} = c \left(\frac{33}{\bar{z}} \right)^{1/6} = 0.3 \left(\frac{33}{45.2} \right)^{1/6} = \boxed{0.26 = I_{\bar{z}}}$$
(TABLE 26.9-1) (0.6h)

$$Q = \left[\frac{1}{1 + 0.63 \left(\frac{B+h}{\bar{z}} \right)} \right]^{-1/2} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{35 + 142}{439} \right)}} = \boxed{0.86 = Q}$$

$$L_{\bar{z}} = \ell \left(\frac{\bar{z}}{33} \right)^{\bar{c}} = 320 \left(\frac{85.2}{33} \right)^{1/5} = \boxed{439 = L_{\bar{z}}}$$
TABLE 26.9-1

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

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

$n_1 = n_a$

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

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

$$N_1 = \frac{n_1 L_{\bar{z}}}{V_{\bar{z}}} = \frac{0.53 \cdot 439}{100.39} = \boxed{2.32 = N_1}$$
PLUG INTO R_n

NORTH / SOUTH WIND

3) CONTINUED

GUST EFFECT FACTOR

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V \quad \text{TABLE 26.9-1}$$

$$= 0.45\left(\frac{85.2}{33}\right)^{1/4}\left(\frac{88}{60}\right)120 = \boxed{100.33 \text{ FT/S} = \bar{V}_{\bar{z}}}$$
PLUGING INTO N.

$$R_n = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{2.07} - \frac{1}{2(2.07)}(1-e^{-2\eta}) = \boxed{0.25 = R_n}$$
PLUG INTO **R**

$$\eta_n = \frac{4.6n_1h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.53)(142)}{100.33} = \boxed{3.45\ \eta_n}$$
PLUG INTO Rn

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

$$\eta_B = \frac{4.6n_1B}{\bar{V}_{\bar{z}}} = \boxed{2.31 = \eta_B}$$
PLUG INTO RB

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

$$\eta_L = \frac{15.4n_1L}{\bar{V}_{\bar{z}}} = \boxed{45.58 = \eta_L}$$
PLUG INTO RL

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

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

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

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

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

NORTH / SOUTH WIND

5. VELOCITY PRESSURE (EQ. 27.3-1)

$$q_z = 0.00256 \, (k_z \, k_{zt} \, k_d \, V^2) = 0.00256 \, (1.11)(1.0)(0.85)(120^2)$$

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

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

N/S PLAN
WIND

$q_h \, GC_p$

$q_z \, GC_p$

$q_h \, GC_p$

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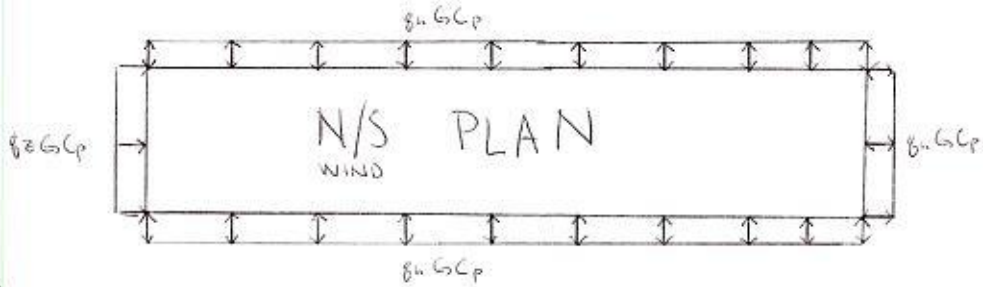
$$L/B = 536/95 = 5.64 > 4 \Rightarrow C_p = 0.8 \text{ WINDWARD}$$
$$C_p = -0.2 \text{ LEEWARD} \Big\} \text{ WALL PRESSURE}$$
$$C_p = -0.7 \text{ SIDE}$$

N/S ELEVATION
WIND

$q_h \, GC_p$

$q_z \, GC_p$

$q_h \, GC_p$
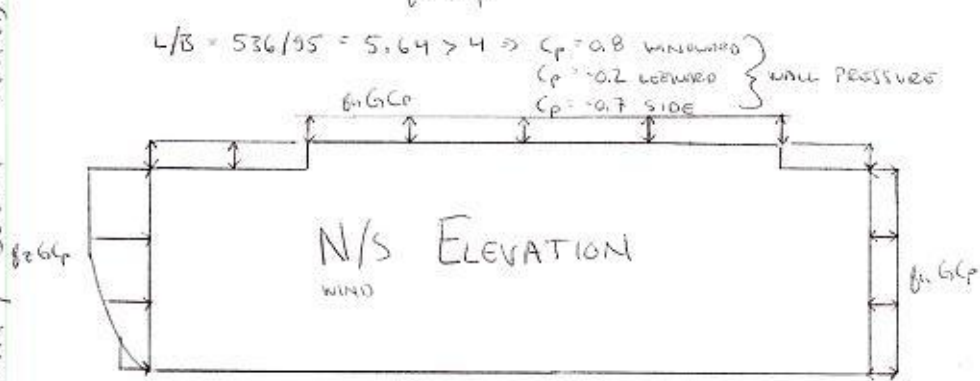
$$h/L = 142/536 = 0.26 \leq 0.5 \quad \text{FROM } 0' \text{ TO } 71' \quad C_p = -0.9, \; -0.18$$
$$\text{FROM } 71' \text{ TO } 142' \quad C_p = -0.9, \; -0.18$$
$$\text{FROM } 142' \text{ TO } 150' \quad C_p = -0.5, \; -0.18$$

$$\text{AREA} = B \times L = 95 \times 536 = 50,920 \text{ SF} \Rightarrow \text{REDUCTION FACTOR} = 0.8$$
$$\text{HOWEVER, DOES NOT APPLY}$$

7. WIND PRESSURE

WINDWARD WALL:

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

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

7 CONTINUED

LEEWARD WALL:

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

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':
$$\left. \right\} \quad p = 34.78(1.00)(-0.9) - 34.15(\pm 0.18) = -31.30 + 6.15 = \boxed{-25.15 \text{ PSF}}$$

142' to 158': $\quad p = 34.78 (1.00)(-0.5) - 34.15 (\pm 0.18) = -16.69 + 6.15 = \boxed{-10.54 \text{ PSF}}$

8 FORCE OF WINDWARD PRESSURE

$$\frac{1}{1000 \text{ lbs}} \left[\frac{h_{RW}}{2} (\text{WIND PRESSURE}_a) + \frac{h_{RW}}{2} (\text{WIND PRESSURE}_b) \right]$$

NORTH / SOUTH WING

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 OR

    2 $L_{eff} = 88' \times 4 > 150 \therefore$ OR

    Approximate Natural Frequency

        Steel Moment Connections

        $n_a = 0.43 < 1 \therefore$ __Flexible__

    Find G_f

        $Q = 0.71$

          $I_{\bar{z}} = 0.26 \;,\; L_{\bar{z}} = 435 \;,\; B = 556' \;,\; g_Q, g_v = 3.4$

      $g_R = 3.98$

        $n_1 = n_a = 0.43$

    $R = 0.27$

        $R_n = 0.08$

          $N_1 = 2.32 \;,\; \bar{V}_{\bar{z}} = 100 \, ft/s$

        $R_h = 0.30$

          $\eta_h = 2.73$

       $R_B = 0.09$

          $\eta_B = 10.60$

       $R_L = 0.15$

          $\eta_L = 6.29$

    $\boxed{G_f = 0.80}$

East / West Wind

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)

$q_z = 34.79$ PSF

$q_h = 34.15$ PSF

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

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

FROM 0' TO 63' $C_p = -1.3^{**}, -0.18$
FROM 63' TO 150' $C_p = -0.7, -0.18$

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

$L/B = 95/536 = 0.18 > 0.1$

WINDWARD $= 0.8$
LEEWARD $= -0.5$
SIDE $= -0.7$

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:

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

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

EAST / WEST WIND)

SEISMIC DETERMINATION CALCULATIONS

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 (at 0.2 seconds)

S_1 = 7.0%g (at 1.0 seconds)

F_a = 1.0 (Table 11.4-1)

F_v = 1.0 (Table 11.4-1)

$S_{MS} = F_a S_s = 1.0(0.356) = 0.356$ (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 = 6 seconds (Figure 22-12)

Values of Approximate Period Parameters:

System Resisting in North/South Direction = Steel Concentrically Braced Frame

C_t = 0.02   x = 0.75   (Table 12.8-2)

System Resisting in East/West Direction = Steel Concentrically Braced Frame

C_t = 0.02   x = 0.75   (Table 12.8-2)

	North/South Direction	East/West Direction
$T_a = C_t h_n^x$ (§12.8.2.1)	$T_a = 0.02(150)^{0.75}$	$T_a = 0.02(150)^{0.75}$
	$T_a = 0.857$ seconds	$T_a = 0.857$ seconds
$T = C_u T_a$	$T = 1.7(0.857)$	$T = 1.7(0.857)$
	$T = 1.46$ seconds	$T = 1.46$ seconds
$T = min \begin{cases} C_u T_a \\ T_b \end{cases}$	$T_b = 0.7803$ seconds (controls)	$T_b = 1.1532$ seconds (controls)
Note: T_b obtained from ETABS model		
R-Value (Table 12.2-1)	R = 3.0	R = 3.0

Equivalent Lateral Force Procedure		
	North/South Direction	East/West Direction
$C_s = min \begin{cases} \frac{S_{D1}}{T(^R/_{I_e})} \ For\ T \le T_L\ (Eq.\,12.8-3) \\ \frac{S_{DS}}{(^R/_{I_e})} \ (Eq.\,12.8-2) \\ 0.044 S_{DS} I_e \ge 0.01 \end{cases}$	$\frac{0.047}{(0.7803)(^{3.0}/_{1.0})} = 0.020$	$\frac{0.047}{(0.1.1532)(^{3.0}/_{1.0})} = 0.014$
	$\frac{0.237}{(^{3.0}/_{1.0})} = 0.079$	$\frac{0.237}{(^{3.0}/_{1.0})} = 0.079$
	$0.044(0.237)(1) = 0.010$ (controls)	$0.044(0.237)(1) = 0.010$ (controls)

Use $C_s = 0.01$

Seismic Base Shear

$V = C_s W$ (Eq. 12.8-1)

W = seismic weight (calculated in spreadsheet on page 79)

$V = 0.01\,(53,919^k) = 539^k$

Vertical Distribution of Seismic Forces

$F_x C_{VX} V$ (Eq. 12.8-11)

$C_{VX} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ (Eq. 12.8-12)

K = 1 for 0.5 seconds and k=2 for 2.5 seconds

	North/South Direction	East/West Direction
Interpolate:	T = 0.7803 seconds	T = 1.1532 seconds
	$k = 2 - (1)\left(\dfrac{2.5 - 0.7803}{2.5 - 0.5}\right)$	$k = 2 - (1)\dfrac{2.5 - 1.1532}{2.5 - 0.5}$
	$k = 1.14$	$k = 1.33$

PERIOD OF VIBRATION RESULTS*

Mode	Period (2)
1	1.1532
2	0.7803
3	0.5367

*Results obtained from ETABS model

SEISMIC SPREADSHEET CALCULATIONS

Noth/South Direction Loading		
T=	0.7803	s
k=	1.14	
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	1004948	0.120	65	50	536	27	1.0	1730
ROOF	10	130	6753	1735370	0.207	111	161	536	27	1.0	2988
8	15	120	5574	1307476	0.156	84	245	536	27	1.0	2251
7	15	105	5574	1122853	0.134	72	318	536	27	1.0	1933
6	15	90	5847	988029	0.118	63	381	536	27	1.0	1701
5	15	75	5847	802607	0.096	52	433	536	27	1.0	1382
4	15	60	5847	622337	0.074	40	473	536	27	1.0	1071
3	15	45	5920	453925	0.054	29	502	536	27	1.0	781
2	16	30	5920	285917	0.034	18	520	536	27	1.0	492
G	14	14	3301	66870	0.008	4	524	536	27	1.0	115
		⌑	53905	8390331		539					14445

TABLE 26: THIS TABLE SUMMARIZES SEISMIC STORY SHEAR

East/West Direction Loading		
T=	1.1532	s
k=	1.33	
V_b=	647	kips

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
BULK HEAD	20	150	3322	1004948	0.120	77	41	95	5	1.0	368
ROOF	10	130	6753	1735370	0.207	134	175	95	5	1.0	636
8	15	120	5574	1307476	0.156	101	276	95	5	1.0	479
7	15	105	5574	1122853	0.134	87	362	95	5	1.0	411
6	15	90	5847	988029	0.118	76	438	95	5	1.0	362
5	15	75	5847	802607	0.096	62	500	95	5	1.0	294
4	15	60	5847	622337	0.074	48	548	95	5	1.0	228
3	15	45	5920	453925	0.054	35	583	95	5	1.0	166
2	16	30	5920	285917	0.034	22	605	95	5	1.0	105
G	14	14	3301	66870	0.008	5	611	95	5	1.0	24
		S	53905	8390331		647					3073

TABLE 27: THIS TABLE SUMMARIZES SEISMIC STORY SHEAR

TORSION CALCULATIONS

CENTER OF MASS CALCULATIONS

FLOOR	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 28: 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 29: 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	460	46%	0	0
2	1000	40	4%	30	1.2
3	1000	40	4%	60	2.4
4	1000	460	46%	95	43.7
					y = 47.3'

TABLE 30: 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	250	25%	0	0
E	1000	250	25%	116	29
O	1000	250	25%	416	104
S	1000	250	25%	536	134
					x = 267.0'

TABLE 31: THIS TABLE SHOWS THE CALCULATION OF THE X COORDINATE FOR THE CENTER OF RIGIDITY.

APPENDIX D:

CONNECTIONS DESIGN & CALCULATIONS

X-FRAME CONCENTRICALLY BRACED CONNECTION

Lateral System Design

Limit States for Brace: 2L4x4x1/4

1. Tension Yield

    Assume 1 row of (3) bolts ¾" ϕ A325N in double shear

    $\phi R_n = \phi F_y A_g = 0.9(36)(7.74) = $ **250.8 kips > 45.9 kips** $\therefore$ **OK**

    Note: Angle sizes based on deflections in ETABS model

2. Tension Rupture

    $\phi R_n = \phi F_u A_e$

    Note $A_e = A_n U$

    $U = 1 - \dfrac{\bar{x}}{L}$, $\bar{x} = 1.08$, L = 9" [Assumes (3) ¾" ϕ bolts]

    $U = 1 - \dfrac{1.08}{9} = 0.88$

    $A_g = 3.87$ in²

    $A_n = 3.87 - 1(\frac{3}{4} + \frac{1}{8}) = 3.00$ in²

    $A_e = 0.88(3.00) = 2.64$ in² $< 0.85 A_g = 3.29$ in²

    $\phi R_n = 0.75(58)(3.29)(2) = $ **286.2 kips > 45.9 kips** $\therefore$ **OK**

3. Block Shear

    Using AISC Table 9-3 $L_{eh} = 1\frac{1}{2}$", $L_{ev} = 1\frac{1}{2}$"

       9-3a: 46.2 kips/in

       9-3b: 121 kips/in

       9-3c: 139 kips/in

    $\phi R_n = 2[0.5(46.2)(0.75) + 0.5(121)(0.75)] = $ **125.4 kips > 45.9 kips** $\therefore$ **OK**

    Note: 0.5 is multiplied in the equation above because of the non-uniform load

Limit States for Bolts

1. Bolt Shear

    $\phi R_n = 29.8 \times 2 = 59.6$ kips

    Note $\phi R_n = 29.8$ because bracing was designed in tension

2. Bearing on Angle

    $\phi R_n = \phi 2.4 F_u t d_b$

    $\phi R_n = 0.75(2.4)(58)(0.25)(0.75) \times 2 = 39.2$ kips

3. Bearing on Plate (Assume ½" thick)

$\phi R_n = \phi 2.4 F_u t d_b$

$\phi R_n = 0.75(2.4)(58)(0.5)(0.75) = 39.2$ kips

4. Tearout Angle Edge

$\phi R_n = \phi 1.2 F_u t l_e$

$\phi R_n = 0.75(1.2)(58)(0.25)[1.5 - 0.5(0.75 + 0.125)]x2 = 27.7$ kips

5. Tearout Angle Middle

$\phi R_n = \phi 1.2 F_u t l_e$

$\phi R_n = 0.75(1.2)(58)(0.25)[3 - 1(0.75 + 0.125)]x2 = 55.5$ kips

6. Tearout Plate Edge

$\phi R_n = \phi 1.2 F_u t l_e$

$\phi R_n = 0.75(1.2)(58)(0.5)[1.5 - 0.5(0.75 + 0.125)] = 27.7$ kips

7. Tearout Plate Middle

$\phi R_n = \phi 1.2 F_u t l_e$

$\phi R_n = 0.75(1.2)(58)(0.5)[3 - 1(0.75 + 0.125)] = 55.5$ kips

$\phi R_n = 27.7 + 2(39.2) = $ **106.1 > 45.9 kips** $\therefore$ **OK**

Limit States for Gusset Plate

1. Block Shear

Using AISC Table 9-3 $L_{eh} = 1\frac{1}{2}"$, $L_{ev} = 1\frac{1}{2}"$

9-3a: Does not occur

9-3b: 121 kips/in x (0.5) = **60.5 kips [controls] > 45.9 kips** $\therefore$ **OK**

9-3c: 139 kips/in x (0.5) = 69.5 kips

2. Gusset Yielding

Whitmore Section

9[tan(30)] = 5.2"

$A_w = l_w t = 2(5.2)(0.5) = 5.2$ in²

$\phi R_n = \phi F_y A_w = 0.9(36)(5.2) = $ **168.5 kips > 45.9 kips** $\therefore$ **OK**

3. Gusset buckling

$r = \dfrac{t}{\sqrt{12}} = \dfrac{0.5}{\sqrt{12}} = 0.144$ in

$\dfrac{kl}{r} = \dfrac{1.2(9)}{0.144} = 75$

Limiting Slenderness Ratio:

$4.71 \times \sqrt{E/F_y} = 134 > 75$

$F_e = \dfrac{\pi^2 E}{(kl/r)^2} = \dfrac{\pi^2 29000}{(75)^2} = 50.9$ ksi

$F_{CR} = [0.658^{F_y/F_e}]F_y = [0.658^{36/50.9}] \times 36 = 26.8$ ksi

$\phi P_n = \phi F_{CR} A_w = 0.9(26.8)(5.2) = $ **125.4 kips > 45.9 kips** $\therefore$ **OK**

Determine Required Gusset-to-Beam Weld Size

$$D_{Required} = \frac{1.25 P_u}{1.392 l} \qquad = \frac{1.25(45.9)}{1.392(60)(2)} = 0.34 < 1/8"$$

Use W = ¼" for ½" thick plate [AISC Table J2.4]

Check Gusset Thickness (Against Weld Size for Required Strength)

$$t_{min} = \frac{6.19 D}{F_u} = \frac{6.19(4)}{58} = \textbf{0.43"} < ½" \therefore \textbf{OK}$$

Check Local Web Yielding of the Beam

$$\phi R_n = \phi \ (N + 5k)F_y t_w = 1.00[60 + 5(1.08)](50)(0.615) = 2011.1 \text{ kips}$$

2011.1 kips >> 45.9 kips ∴ OK

Y-FRAME CHEVRON BRACE CONNECTION

$$Pu = \frac{\Sigma F}{Braces} = \frac{(116.2 + 169.5 + 131.5 + 150.1 + 139 + 127.9 + 115.8 + 100.7 + 76.4)}{(4)(2)} = \textbf{140.9 kips}$$

1. Determine Required Gusset Plate Thickness

$$W_e = W_w - 3/8 - 1/16 = 5/16\text{"}$$

$$t_{required} = \frac{\varphi(0.60F_{EXX}W_e)(0.707)(2)}{\varphi(0.60F_{yi})} = \frac{0.75(0.60)(70)(0.3125)(0.707)(2)}{1.00(0.60)(36)} = 0.644\text{"}$$

Use a ¾ inch gusset plate

2. Check Gusset Plate Buckling

$$r = \frac{t_1}{\sqrt{12}} = \frac{3/4}{\sqrt{12}} = 0.217\text{"}$$

Since the gusset is attached by one edge, the buckling mode could be sideways. In this case k = 1.2

$$\frac{kl}{r} = \frac{1.2(6)}{0.217} = 33.2$$

Limiting Slenderness Ratio:

$$4.71 \times \sqrt{E/F_y} = 134 > 33.2$$

$$F_e = \frac{\pi^2 E}{(kl/r)^2} = \frac{\pi^2 29000}{(33.2)^2} = 259.7 \text{ ksi}$$

$$F_{CR} = [0.658^{F_y/F_e}]F_y = [0.658^{36/259.7}] \times 36 = 34.0 \text{ ksi}$$

$$l = B + 2[(\text{connection length})\tan(30°)] = 6\text{"} + 2(6\tan(30°)) = 12.9\text{"}$$

Note: The Whitmore section is assumed to be entirely in the gusset. The Whitmore section can spread across the joint into adjacent connected material of equal or greater thickness or adjacent connected material of lesser thickness provided that a rational analysis is performed.

$$A_w = l_w t_1 = 12.9(0.75) = 9.7\text{"}$$

$$\phi P_n = \phi F_{CR} A_w = (0.90)(34)(9.7) = \textbf{296.8 kips} > \textbf{140.9 kips} \therefore \textbf{OK}$$

3. Check Tension Yielding of Gusset Plate (Tension Brace)

$$\phi R_n = \phi F_{yi} A_w = 0.9(36)(9.7) = \textbf{314.3 kips} > \textbf{140.9 kips} \therefore \textbf{OK}$$

4. Tension Rupture

$$\phi R_n = \phi F_u A_e$$

Note $A_e = A_n U$

$$b_f \geq 2/3(d) = 2/3(14.3) = 9.53\text{"} < 10.1\text{"} \therefore \text{Use } U = 0.90$$

$$A_g = 24.0 \text{ in}^2$$

$$A_n = 24.0 - 4\left(\frac{3}{4} + \frac{1}{8}\right) = 20.5 \text{ in}^2$$

$A_e = 0.90(20.5) = 18.45 \text{ in}^2$

$\phi R_n = 0.75(58)(18.45) = \textbf{802.6 kips} > \textbf{140.9 kips} \therefore \textbf{OK}$

5. Calculate Interface Forces

Design the gusset-to-beam connection as if each brace were the only brace and locate each brace's connection centroid at the ideal centroid location to avoid inducing a moment on the gusset-to-beam interface, similarly to the uniform force method special case 3

$e_b = \textbf{11.95"}$

$\beta = e_c = \textbf{0"}$

$\tan\theta = 15/15 = 1 = \textbf{45}°$

Let $\alpha = e_b\tan\theta = 11.95\tan(45) = 11.95" \rightarrow$ **Use 12"**

$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(12.0 + 0)^2 + (0 + 11.95)^2} = \textbf{16.9"}$

Limit States for Gusset Plate

$H_{ub} = \dfrac{\alpha P_u}{r} = \dfrac{12(140.9)}{16.9} = 100.0 \text{ kips}$

$V_{ub} = \dfrac{e_b P_u}{r} = \dfrac{11.95(140.9)}{16.9} = 99.6 \text{ kips}$

6. Determine Required Gusset-to-Beam Weld Size

The weld length is twice the horizontal distance from the work point to the centroid of the gusset-to-beam connection, α, for each brace.
Therefore $L = 2\alpha = 2(12) = 24"$

$D_{required} = \dfrac{1.25P_u}{1.392L} = \dfrac{1.25(140.9)}{1.392(24)(2)} = 2.64$

Theoretically use a 3/16" fillet weld, 48" long total.
According to AISC Table J2.4 – **use ¼" fillet weld, 48" long total**.

7. Check Gusset Thickness (Against Weld Size for Required Strength)

$t_{min} = \dfrac{6.19D}{F_u} = \dfrac{6.19(2.64)}{58} = \textbf{0.28"} < \textbf{¾"} \therefore \textbf{OK}$

8. Check Local Web Yielding of the Beam

$\phi R_n = \phi(N + 5k)F_y t_w = 1.00[36 + 5(1.08)](50)(0.44) = \textbf{910.8 kips} > \textbf{140.9 kips} \therefore \textbf{OK}$

Limit States for Bolts

1. Bolt Shear

$\phi R_n = 29.8 \times 4 = 119.2 \text{ kips}$

2. Bearing on Angle (Use 2L4x4x¼)

$\phi R_n = \phi 2.4 F_u t d_b$
$\phi R_n = 0.75(2.4)(58)(0.25)(3/4)\times4 = 78.3 \text{ kips}$

3. Bearing on Plate (Assume 3/4" thick)

$\phi R_n = \phi 2.4 F_u t d_b$
$\phi R_n = 0.75(2.4)(58)(0.75)(3/4)\times2 = 117.5 \text{ kips}$

4. Tearout Angle Edge

$\phi R_n = \phi 1.2 F_u t l_e$

$\phi R_n = 0.75(1.2)(58)(0.25)[1.5 - 0.5(0.75 + 0.125)] x4 = 55.5$ kips

5. Tearout Angle Middle

$\phi R_n = \phi 1.2 F_u t l_e$

$\phi R_n = 0.75(1.2)(58)(0.25)[3 - 1(0.75 + 0.125)] x4 = 110.9$ kips

6. Tearout Plate Edge

$\phi R_n = \phi 1.2 F_u t l_e$

$\phi R_n = 0.75(1.2)(58)(0.75)[1.5 - 0.5(0.75 + 0.125)] x2 = 83.2$ kips

7. Tearout Plate Middle

$\phi R_n = \phi 1.2 F_u t l_e$

$\phi R_n = 0.75(1.2)(58)(0.75)[3 - 1(0.75 + 0.125)] x2 = 166.4$ kips

$\phi R_n = 55.5 + 2(78.3) = \mathbf{212.1 > 140.9\ kips} \therefore \mathbf{OK}$

Y-FRAME LATERAL BRACE CONNECTION

$$Pu = \frac{\Sigma F}{Braces} = \frac{(116.2 + 169.5 + 131.5 + 150.1 + 139 + 127.9 + 115.8 + 100.7 + 76.4)}{(4)(2)} = \mathbf{140.9\ kips}$$

Limit States for Brace: W14x82

1. Tension Yield
   Assume 4 sets of 1 row of (3) bolts 3/4" ϕ A325N in double shear
   $\phi R_n = \phi F_y A_g = 0.9(36)(24) = \mathbf{777.6\ kips} > \mathbf{140.9\ kips}\ \therefore \mathbf{OK}$
2. Tension Rupture
   $\phi R_n = \phi F_u A_e$
   　　Note $A_e = A_n U$
   　　$b_f \geq 2/3(d) = 2/3(14.3) = 9.53" < 10.1"\ \therefore Use\ U = 0.90$
   $A_g = 24.0\ in^2$
   $A_n = 24.0 - 4(\frac{3}{4} + \frac{1}{8}) = 20.5\ in^2$
   $A_e = 0.90(20.5) = 18.45\ in^2$
   $\phi R_n = 0.75(58)(18.45) = \mathbf{802.6\ kips} > \mathbf{140.9\ kips}\ \therefore \mathbf{OK}$
3. Block Shear
   Using AISC Table 9-3 $L_{eh} = 1\frac{1}{2}"$, $L_{ev} = 1\frac{1}{2}"$
   　9-3a: 46.2 kips/in
   　9-3b: 121 kips/in
   　9-3c: 139 kips/in
   $\phi R_n = 4[0.5(46.2)(0.75) + 0.5(121)(0.75)] = \mathbf{250.8\ kips} > \mathbf{140.9\ kips}$
   　$\therefore \mathbf{OK}$
   Note: 0.5 is multiplied in the equation above because of the non-uniform load

Limit States for Bolts

1. Bolt Shear
   $\phi R_n = 29.8\ x\ 4 = 119.2\ kips$
2. Bearing on Angle (Use 2L4x4x¼)
   $\phi R_n = \phi 2.4 F_u t d_b$
   $\phi R_n = 0.75(2.4)(58)(0.25)(3/4)x4 = 78.3\ kips$
3. Bearing on Plate (Assume ¾" thick)
   $\phi R_n = \phi 2.4 F_u t d_b$
   $\phi R_n = 0.75(2.4)(58)(0.75)(3/4)x2 = 117.5\ kips$
4. Tearout Angle Edge
   $\phi R_n = \phi 1.2 F_u t l_e$
   $\phi R_n = 0.75(1.2)(58)(0.25)[1.5 - 0.5(0.75 + 0.125)]x4 = 55.5\ kips$
5. Tearout Angle Middle
   $\phi R_n = \phi 1.2 F_u t l_e$
   $\phi R_n = 0.75(1.2)(58)(0.25)[3 - 1(0.75 + 0.125)]x4 = 110.9\ kips$

6. Tearout Plate Edge

$$\phi R_n = \phi 1.2 F_u t l_e$$
$$\phi R_n = 0.75(1.2)(58)(0.75)[1.5 - 0.5(0.75 + 0.125)] \times 2 = 83.3 \text{ kips}$$

7. Tearout Plate Middle

$$\phi R_n = \phi 1.2 F_u t l_e$$
$$\phi R_n = 0.75(1.2)(58)(75)[3 - 1(0.75 + 0.125)] \times 2 = 166.3 \text{ kips}$$

$$\phi R_n = 55.5 + 2(78.3) = \textbf{212.1} > \textbf{140.9 kips} \therefore \textbf{OK}$$

Connection Design Using Uniform Force Method

      Special Case 2: Shear in Beam-to-Column Connection Minimized

Goal: Do not transfer moment to horizontal members. To achieve this use the following equation:

$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c$$

Given:

$$e_b = \textbf{11.95"}$$
$$e_c = \textbf{0.83"}$$
$$\tan\theta = 15/15 = \textbf{1}$$
$$\text{Assume } \alpha = \textbf{18"}$$
$$\alpha - \beta\tan\theta = e_b\tan\theta - e_c \rightarrow \beta = 18 - 11.95 + 0.83 = \textbf{6.88"}$$
$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} =$$
$$\sqrt{(18.0 + 0.83)^2 + (6.88 + 11.95)^2} = \quad \textbf{26.6"}$$

Limit States for Gusset Plate (Plate Dimensions: 14"x36"x¾")

1. Block Shear

  Using AISC Table 9-3 $L_{eh} = 1\frac{1}{2}$", $L_{ev} = 1\frac{1}{2}$"

    9-3a: Does not occur

    9-3b: 121 kips/in x (0.75) x 2 = **181.6 kips [controls]** > **140.9 kips** ∴ **OK**

    9-3c: 139 kips/in x (0.75) = 104.3 kips

2. Gusset Yielding

    Whitmore Section

      $9[\tan(30)] = 5.2$"

      $A_w = l_w t = 2(5.2)(0.5) = 5.2 \text{ in}^2$

      $\phi R_n = \phi F_y A_w = 0.9(36)(5.2) = \textbf{168.5 kips} > \textbf{140.9 kips} \therefore \textbf{OK}$

3. Gusset buckling

$$r = {}^t/_{\sqrt{12}} = {}^{0.625}/_{\sqrt{12}} = 0.180 \text{ in}$$
$$^{kl}/_r = {}^{1.2(9)}/_{0.180} = 60$$

    Limiting Slenderness Ratio:

$$4.71 \times \sqrt{E/F_y} = 134 > 75$$

$$F_e = \frac{\pi^2 E}{(kl/r)^2} = \frac{\pi^2 29000}{(60)^2} = 79.5 \text{ ksi}$$

$$F_y/F_e$$

$$F_{CR} = [0.658\qquad]F_y = [0.658^{36/79.5}] \times 36 = 29.8 \text{ ksi}$$
$$\phi P_n = \phi F_{CR}A_w = 0.9(29.8)(5.2) = \textbf{139.5 kips} \sim \textbf{140.9 kips} \therefore \textbf{OK}$$

Calculate Interface Connections

1. Gusset-to-Column Connection:

$$V_{uc} = \beta/r \cdot P_u = 6.9/26.6 \cdot 140.9 = \textbf{36.5 kips}$$

$$H_{uc} = e_c/r \cdot P_u = 0.83/26.6 \cdot 140.9 = \textbf{4.4 kips}$$

2. Gusset-to-Beam Connection:

$$H_{ub} = \alpha/r \cdot P_u = 18/26.6 \cdot 140.9 = \textbf{95.3 kips}$$
$$V_{ub} = e_b/r \cdot P_u = 11.95/26.6 \cdot 140.9 = \textbf{63.3 kips}$$

3. Beam-to-Column Connection Shear:

$$R_{ub} + V_{ub} = 27.9 + 4.7 = \textbf{31.9 kips}$$

4. Beam-to-Column Connection Axial Force:

$$A_{ub} + H_{uc} = 89.4 + 4.4 = \textbf{93.8 kips}$$

Apply Special Case 2 with $\Delta V_{ub} = V_{ub} = \textbf{63.3 kips}$

5. Gusset-to-Column Connection:

$$V_{uc} = \textbf{36.5} + \textbf{63.3} = \textbf{99.8 kips}$$
$$H_{uc} = \textbf{4.4 kips (unchanged)}$$

6. Gusset-to-Beam Connection:

$$H_{ub} = \textbf{95.3 kips (unchanged)}$$
$$V_{ub} = 63.3 - 63.3 = \textbf{0 kips}$$
$$M_{ub} = (\Delta V_{ub}) \, \alpha = (63.3)(18)/12"/ft = \textbf{95.0 ft-k}$$

7. Beam-to-Column Connection Shear:

$$R_{ub} + \Delta V_{ub} - V_{ub} = 27.9 + 63.3 - 63.3 = \textbf{27.9 kips}$$

8. Beam-to-Column Connection Axial Force:

$$A_{ub} + H_{uc} = 89.4 + 4.4 = \textbf{93.8 kips (unchanged)}$$

Check Local Web Yielding of the Beam

$$\phi R_n = \phi (N + 5k)F_y t_w = 1.00[36 + 5(1.08)](50)(0.44) = 910.8 \text{ kips}$$
$$\textbf{910.8 kips} > \textbf{140.9 kips} \therefore \textbf{OK}$$

APPENDIX E: LATERAL MOVEMENT

ORIGINAL DESIGN

	X – FRAME STORY DRIFTS (INCHES)				Y – FRAME STORY DRIFTS (INCHES)			
FLOOR	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

TABLE 32: ORIGINAL DESIGN LATERAL MOVEMENT

THESIS REDESIGN

	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	0.071	4.8	0.061	0.6	0.049	4.8	0.328	0.6
ROOF	0.103	6	0.098	0.75	0.064	6	0.427	0.75
8	0.069	3.6	0.070	0.45	0.033	3.6	0.207	0.45
7	0.074	3.6	0.069	0.45	0.036	3.6	0.219	0.45
6	0.081	3.6	0.075	0.45	0.032	3.6	0.180	0.45
5	0.089	3.6	0.073	0.45	0.033	3.6	0.190	0.45
4	0.096	3.6	0.076	0.45	0.028	3.6	0.170	0.45
3	0.109	3.84	0.094	0.48	0.031	3.84	0.192	0.48
2	0.117	3.36	0.144	0.42	0.019	3.36	0.118	0.42
Σ	0.809	36	0.760	4.5	0.325	36	2.031	4.5

TABLE 33: THESIS REDESIGN LATERAL MOVEMENT

APPENDIX F: FOUNDATION IMPACTS

OVERTURNING MOMENT CALCULATIONS

RESISTING MOMENT (M_R) = BUILDING WEIGHT × MOMENT ARM

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

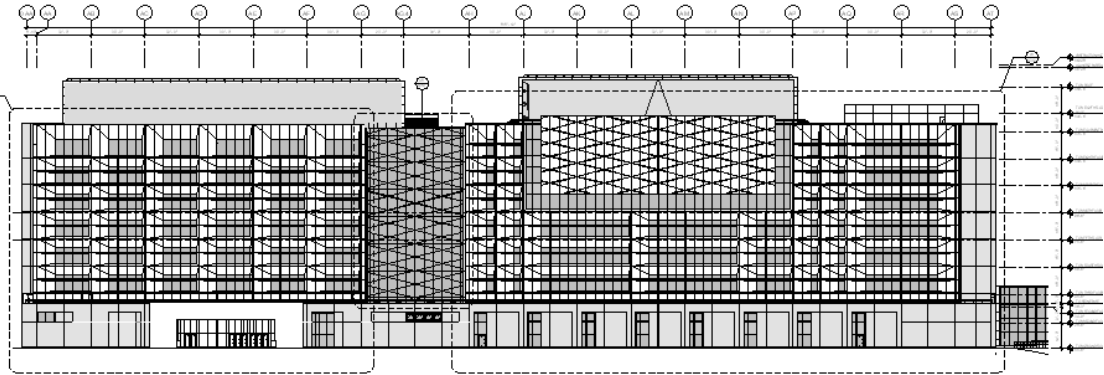$M_R = 53,905^k \times \dfrac{95'}{2} = 2,560,487.5^{1k}$

CONTROLLING LOAD CASE = N/S WIND
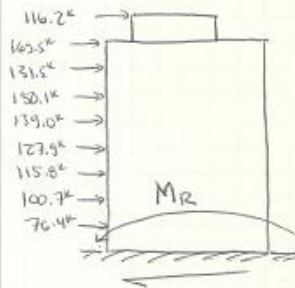
OVERTURNING MOMENT $(M_O) =$

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

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

$= 94,182.1^{1k} \ll M_R \therefore$ OKAY

APPENDIX G: ARCHITECTURE REDESIGN

NORTH ELEVATIONS

ORIGINAL DESIGN
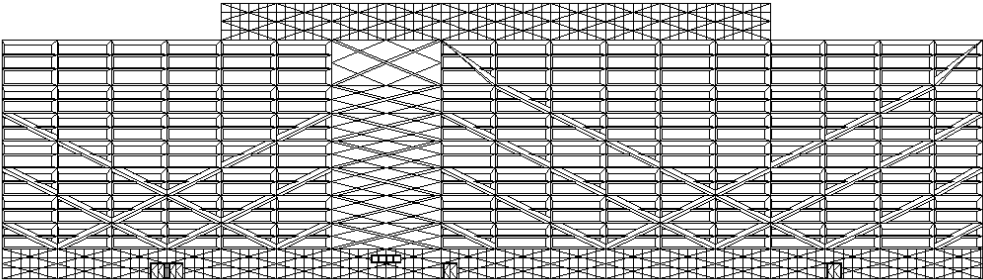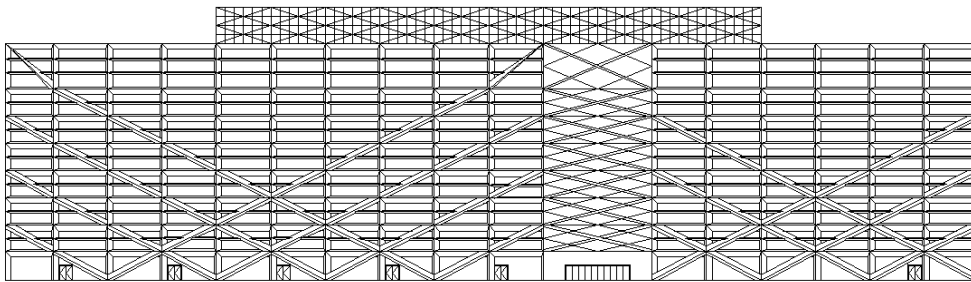

THESIS REDESIGN

SOUTH ELEVATIONS

ORIGINAL DESIGN
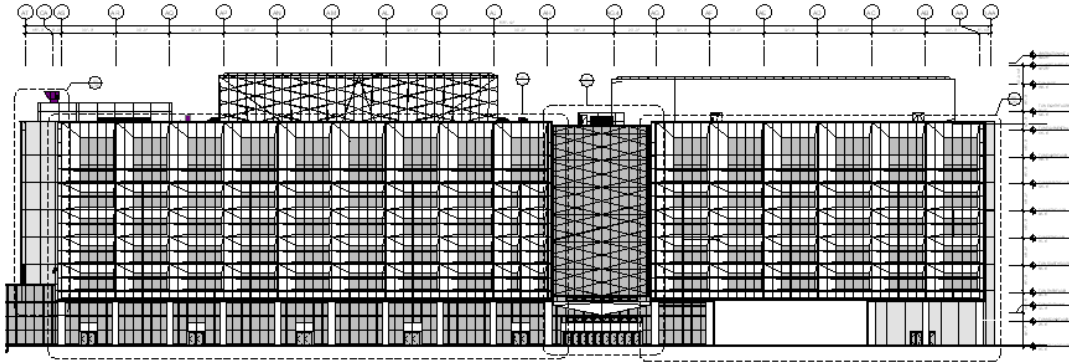
THESIS REDESIGN

APPENDIX H: COST STUDY

SAMPLE CALCULATION

Using RSMeans 2011:

Assume connection assembly crew E-2 with 4 steel workers at each beam.

$48.55 (per steel worker per hour) x 1.433 (Location Multiplier) = $69.57

Original Design Y-Frame HSS Connection Calculation:

Location Multipliers for New York, NY:

Material: 1.165 x $223.00 = $259.80

Labor: 1.433 x $48.55 = $69.57

$69.57 x 2 hours = $139.14

Fabrication: 1.257 x $55.00 = $69.14

$69.14 x 19 hours = $1313.66

($259.80 + $139.15 + $1313.66) x 468 Connections = **$801,452.01**

Additional Aluminum Paneling

Typical Panel Dimensions: 48" x 400" x 0.158" (4mm thick)

Volume of Aluminum = 1.76 ft^3

Density of Aluminum = 169 pcf

1.76 x 169 = 297.44 lbs

128 Aluminum Panels x 297.44 lbs = 38,072.32 lbs

38,072.32 x $2.66 (material cost) x 1.165 (location modifier) = $117,981.72

Additional Fabrication: 1.257 x $55.00 x 3 hrs x 128 panels = $26,547.846

Total = **$144,529.56**

APPENDIX I: SCHEDULING STUDY

Time Spent (Hours)		
Connection	Fabrication	Labor
Original Design X-Frame	8,892	3,276
Original Design Y-Frame	5,376	840
Thesis Design X-Frame	2,304	640
Thesis Design Y-Frame	3,536	680
Original Design	14,268	4,116
Thesis Design	5,840	1,320

TABLE 34: THESIS SCHEDULING STUDY

APPENDIX J: THESIS TASKS

I.   Redesign X- and Y-Frames from moment and concentrically braced frames to ideal braced frames for constructability

    a. Decide the best location for lateral system in both X- and Y-Frames. √

    b. Model in ETABS √

    c. Compare the following combinations of frames √

        i. X: Moment, Y: CBF

        ii. X: Moment, Y: EBF

        iii. X: Moment, Y: Moment

        iv. X: CBF, Y: CBF

        v. X: CBF, Y: EBF

        vi. X: CBF, Y: Moment

        vii. X: EBF, Y: CBF

        viii. X: EBF, Y: EBF

        ix. X: CBF, Y: CBF

        x. Choose the most efficient combination

    d. Detail connections √

II.   Construction impact and cost analysis

    a. Material Cost √

    b. Labor Cost √

    c. Scheduling Cost √

    d. System Savings √

III.   Architectural façade alterations

    a. Alter glazing to better suit the optimum lateral systems √

        i. Aesthetically

        ii. Structurally

    b. Report any load changes and alter ETABS model as necessary √

IV.   Compose Final Presentation and Report √

APPENDIX K: FRAMING PLANS AND ELEVATIONS

FRAMING PLAN PART 1 (WEST END)
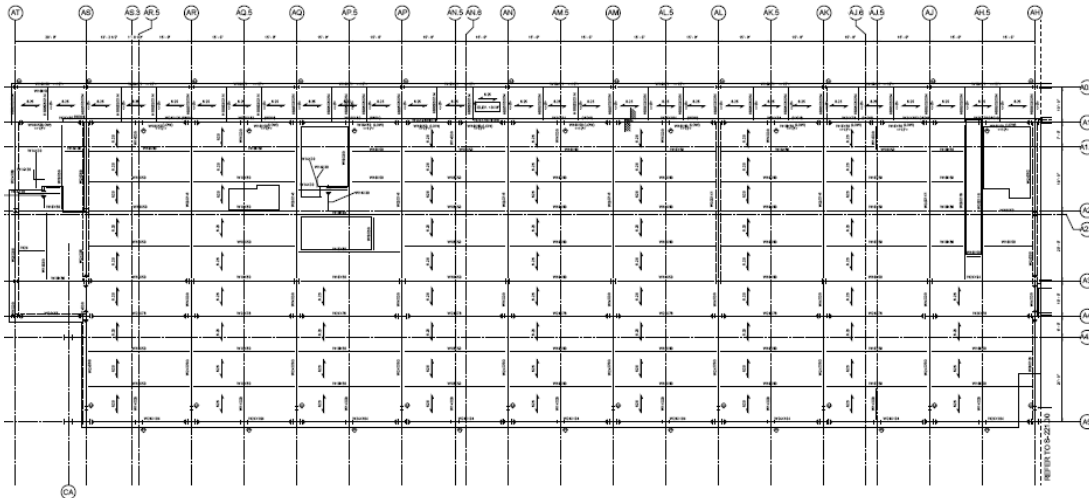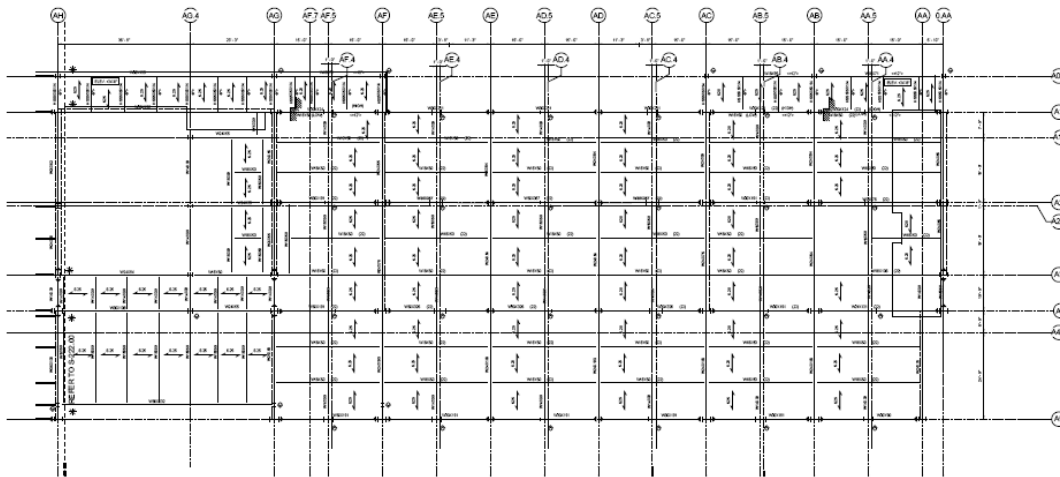

FRAMING PLAN PART 2 (EAST END)

FIGURE 12: 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).
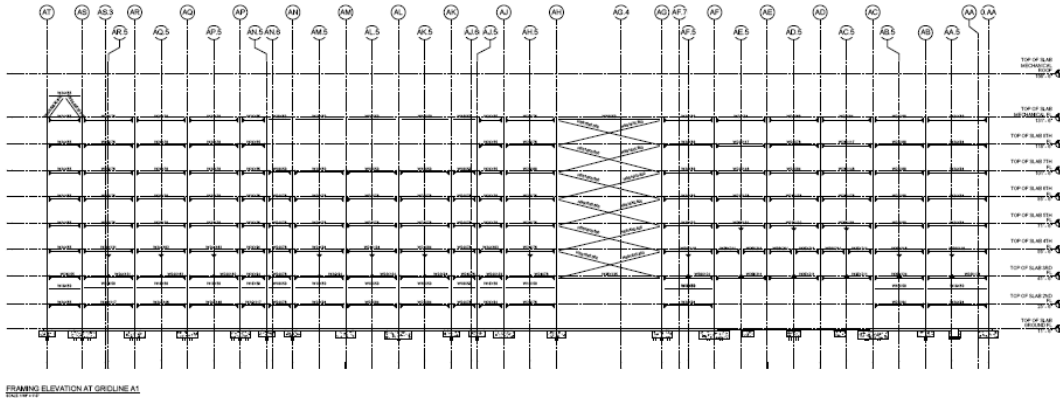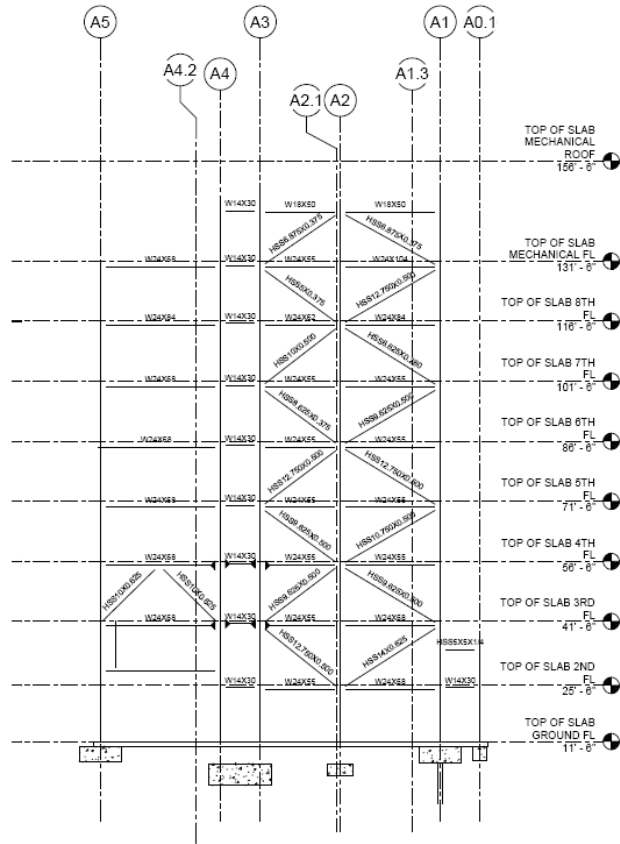
FIGURE 13: ABOVE IS AN ELEVATION OF THE FRAMING SYSTEM LOOKING IN THE NORTH/SOUTH DIRECTION. NOTICE ONLY MOMENT CONNECTIONS EXCEPT FOR THE CROSS BRACING ON THE BRIDGE. BELOW IS AN ELEVATION OF THE FRAMING SYSTEM LOOKING IN THE EAST/WEST DIRECTION. NOTICE THE MAJORITY OF THE CROSS BRACING IN THIS DIRECTION COMPARED TO FEW MOMENT CONNECTIONS.

FRAMING ELEVATION LINE A5
SCALE: 1/16" = 1'-0"