HUNTER'S POINT SOUTH INTERMEDIATE & HIGH SCHOOL

TECHNICAL REPORT III

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EXECUTIVE SUMMARY

Technical Report III builds on Technical Report I, which gave an overview of the current lateral design of the structure. In this report, an in-depth analysis of the lateral system design is performed. Hunter's Point South is a steel frame structure with a rigid steel deck floor system and a lateral force support system that is comprised of vertical truss bracing and steel moment frames. The foundation system consists of grade beams, steel H piles, and deep caissons.

To perform a more detailed lateral system analysis, a STAAD and ETABS model are created. This model is simplified to show only the lateral system of the structure (No beams or gravity columns are included). Furthermore, each floor is modeled as a rigid diaphragm that is loaded with a mass based off of the total floor weight. Because an in-depth foundation analysis is not done, it is difficult to specify the column to foundation connections as either fixed or pinned. Therefore, this report will give results for both cases to distinguish the differences and find how much different each assumption is.

The in-depth analysis first studies the stiffness of each individual lateral resisting frame, and distinguishes the percentage of the lateral force it will take. Then an overall building stiffness is obtained, a center of rigidity is mapped, and a force eccentricity is determined.

Next, ASCE7-10 is employed to set up 7 different load combinations to find the controlling forces for strength design. Using the ETABS model for analysis, it is determined that combination 5: (**1.2 *Dead* + 1.0 *Earthquake* + 1.0 *Live* + 0.2 *Snow***) was the controlling load combination for design.

Serviceability checks are then performed to limit story drift and prevent damage to the nonstructural components. The seismic story drift limit is found in ASCE7-10 as $\Delta_{seismic} = 0.015h_{sx}$, and wind story drift limit is taken as H/400. Using the ETABS model output, it is determined that the story drifts of each case are well within code limits.

Next, the foundation is analyzed to determine whether it can support the uplift forces from the overturning moments caused by the lateral loads. This is important to prevent the building from tipping over under loading. This report finds that the foundation is more than adequate to support the uplift forces.

Finally, a lateral system member spot check is performed to ensure proper member design. Forces were determined from the compression side (critical side) of Truss 2 in the E-W direction and Truss 6 in the N-S direction. Analysis shows that each member is sufficient in size to carry the lateral loads applied from the controlling load combination.

INTRODUCTION

Hunter's Point South School is a new 5 story educational building being constructed as part of the first phase of New York City's new mixed-use development plan on a 30 acre site of waterfront properties in Long Island City, NY. The new development focuses on creating an affordable middle-income area that includes several new mixed use housing towers, along with supporting retail spaces, a school, and new waterfront park. Hunter's Point South School is being developed by the NYC School Construction Authority (SCA) along with Skanska contracting and FXFowle Architects. The structural engineer on the project is Ysreale A. Seinuk, PC. Construction of the school will last from January 2011 to October 2013, and cost approximately $61Million to complete. Project delivery is lump sum bid. It will open its doors to students in the fall of 2013.

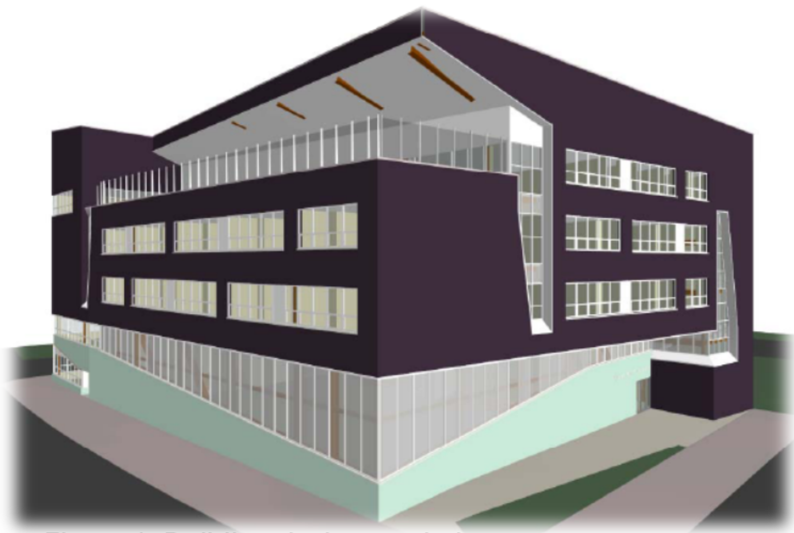


Figure 1: Building design rendering
Rendering by FXFowle Architects

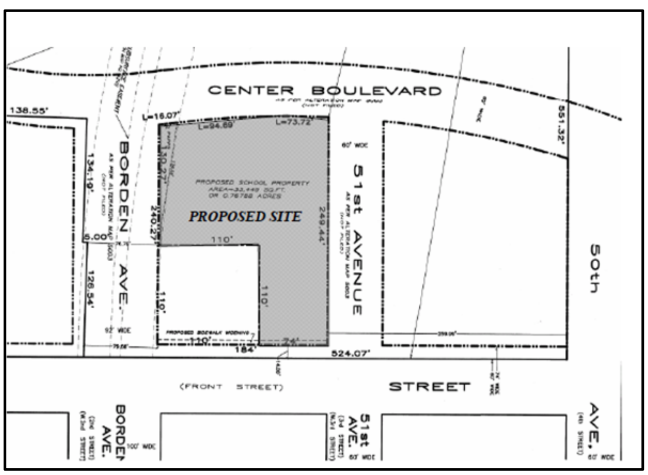


Figure 2: Building site plan
Drawing by FXFowle Architects

The mixed use intermediate and high school will be nearly 154,500 square feet and house roughly 1100 students from grades 6-12 and District 75 (special needs) from the Queens School District. Being constructed on 51st Avenue, Hunter's Point will take up almost a full city block between 2nd Street and Center Boulevard with space in the corner of the lot reserved for the construction of a new 30 story housing tower to be built right next to the school. The site layout can be seen in Figure 2. It should also be noted that the site sits right across the street from the bay.

Following along with other city development ideals, the school building has a modern architectural feel as it incorporates interesting shapes, cantilevers, and sense of solids and voids together. The cubic shape of the building is broken up with vertical shafts, horizontal windows, and slanted edges. In addition, the SCA is aiming to achieve LEED Silver certification for this building through several different sustainable features and construction procedures.

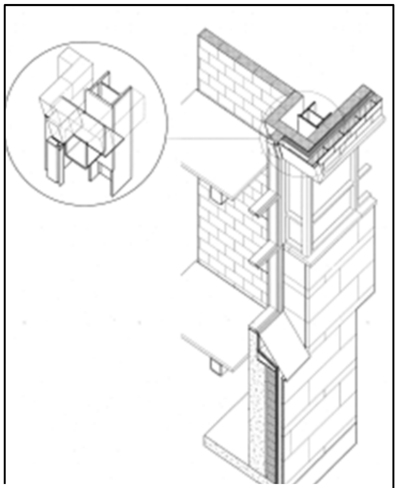


Figure 3: Typical Wall Section Axonometric Detail
Drawing by FXFowle Architects

The 5 story school rises roughly 75 feet off finished grade, with an irregular parapet rising as high as 98 feet on some elevations. It is mainly a structural steel building, with concrete on metal deck floors and an assorted exterior. The exterior façade is comprised of a unique blend of grey brick, slate veneer, concrete block, orange aluminum composite panels, and different types of glazing including translucent panels. Much of the shell is part of a curtain wall system that is supported by the floor above. There is, however, some load bearing masonry used in the design.

Inside, the building is vertically stacked to separate the schools, but includes ties to each other using shared spaces. The first floor contains athletic space, including a 2 story tall gymnasium and locker rooms for all grades. There are also support rooms/offices for the intermediate school and general storage areas. The second floor contains an auxiliary gym, library, and special education rooms for the District 75 students. The third floor contains a full sized 2 story auditorium that links the high school (HS) and intermediate school (IS) together, along with IS classrooms and IS support rooms/offices. The fourth floor contains high school classrooms with support rooms/offices and access to the auditorium. The fifth floor contains HS and IS cafeterias with a central kitchen space, a connecting 4000sf roof terrace, science labs, and support rooms/offices for the high school. There is a small mechanical penthouse on the top roof.

Figure 4: Building Section
Rendering by FXFowle Architects

STRUCTURAL SYSTEMS

This section provides a brief overview of the different structural systems implemented in the Hunter's Point design. The structure consists of a steel framing system with concrete on metal deck floors. There are no subgrade levels, and structural height of the building is 72.3 feet to the roof level with a 13.5 foot parapet wall extending above. All exterior walls are non-loadbearing brick, slate, aluminum panel, or glazing. CMU masonry infill walls are used as a backup wall and are grout filled and reinforced against lateral forces. The steel frame makes up both the gravity and lateral load systems of this building.

Foundation

The foundation consists of a 12 inch 4000 psi reinforced slab on grade supported by a system of grade and strap beams, 14 inch caissons, and steel H-piles. All of these different foundation systems are required due to the poor soil properties on site. A geotechnical survey performed by Langan Engineering showed soil type ranges from grey silty sand fill to clay, with bedrock consisting of gneiss starting at about 40 feet below grade. Deep foundations are installed to at

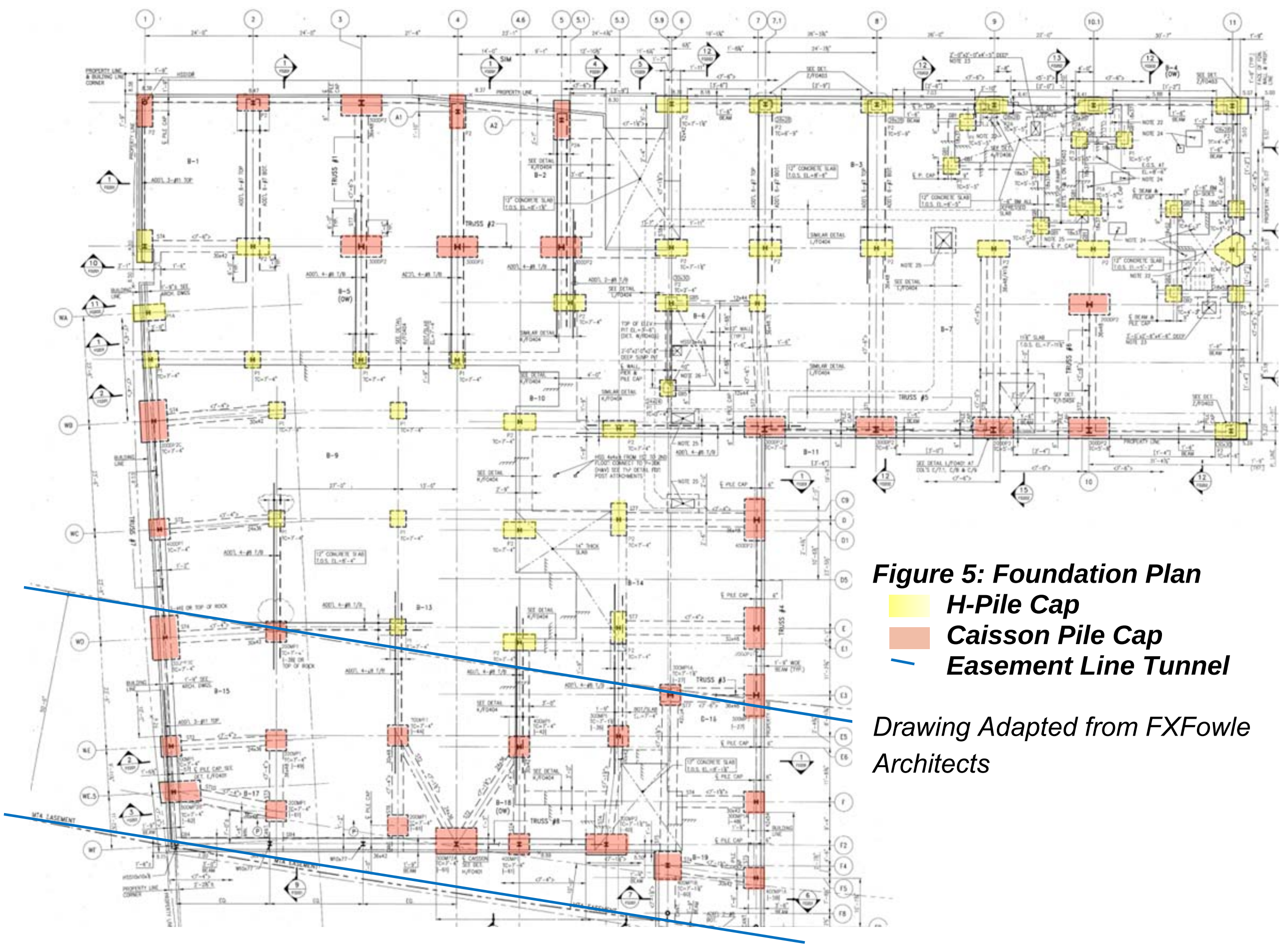


Figure 5: Foundation Plan

H-Pile Cap

Caisson Pile Cap

Easement Line Tunnel

Drawing Adapted from FXFowle Architects

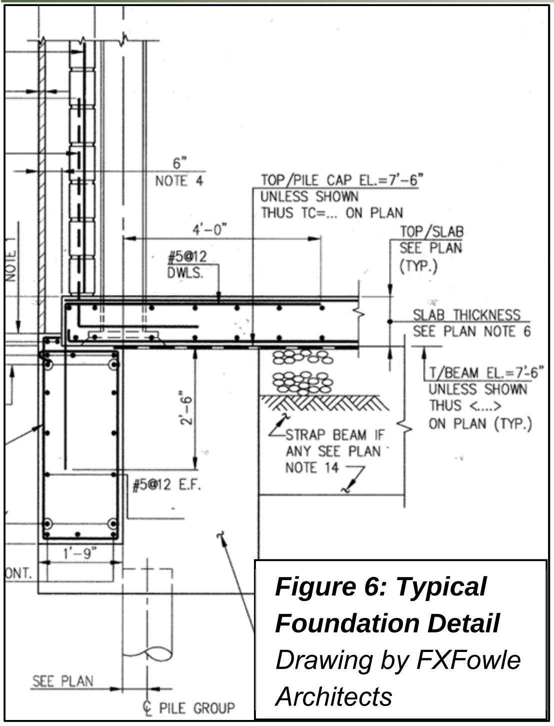


Figure 6: Typical Foundation Detail
Drawing by FXFowle Architects

least this level. H-piles are used mainly within the interior and in the upper north east corner of the site where soil conditions are better. Caissons are installed around the perimeter to help stabilize the building and take the majority of the dead load as it passes down and outward through the structural system. Special isolation caissons, as seen in ***Figure 7***, were used for locations within 50 feet of two subsurface tunnels used for the Queens-Midtown Tunnel easement lines that run E-W through the site. Each caisson has three 20 inch 75 ksi steel threadbars within 8000 psi grout, and can support up to 800kips of compressive force. Ground and strap beams are used to connect pile caps to help prevent lateral column base movement.

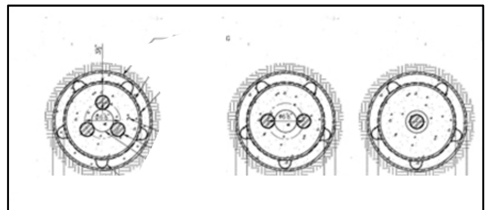


Figure 7: Isolation caisson cross section
Drawing Adapted from FXFowle

Floor and Roof Systems

As seen in ***Figure 8***, the floor system consists typically of 3-¼ inch. thick 3500 psi lightweight concrete on 3 inch deep composite 18 gage galvanized metal deck (6-¼ inch total depth) supported by a steel framing system. Concrete is reinforced with 6x6 W2.0xW2.0 WWF. The floor system above the gymnasium uses acoustical metal deck in place of typical deck. The auditorium stadium seating floor will have 16 gage deck in place of typical deck. The typical unsupported span length for the floor deck is 12'. All cast-in-place concrete slabs are reinforced by #4 reinforcing bars spaced 12 inches in both directions. The top roof and terrace roof will have 2 inch thick lightweight concrete pavers over hot applied asphalt roofing membrane on top of the concrete slab.

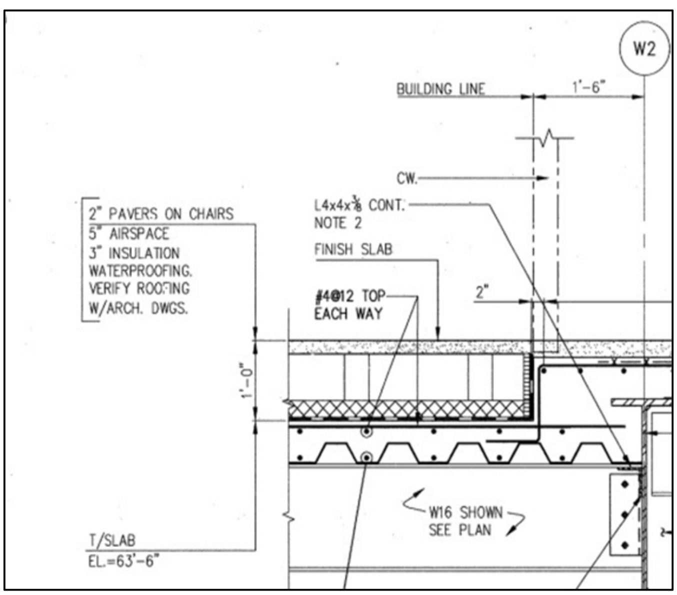


Figure 8: Typical floor system
Drawing by FXFowle Architects

Framing System

The superstructure of Hunter's Point is typically comprised of W10-W14 steel columns supporting W24 girders and either W14 beams at the building core or W16 beams towards the perimeter of the structure. Overall, sizes and span lengths vary greatly throughout the building and across every floor. The third floor includes special long span plate girders over the gymnasium space (red box, ***Figure 10***). Spanning roughly 80feet each with a flange thickness of 2-4 inches and overall depth of up to 3 feet, these large transfer beams allow for open gym space while adequately supporting the load transferred from the auditorium and cafeteria space in the floors directly above. Gravity loads are transferred from the floor slab to the wide flange beams then to interior and exterior columns down to the foundation system. Exterior walls and cladding transfer their weight to exterior beams.

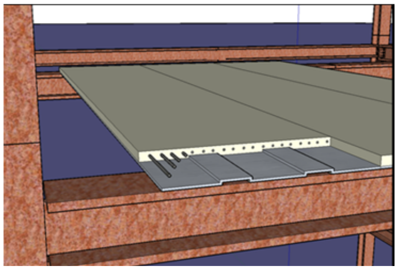


Figure 9: Typical frame layout

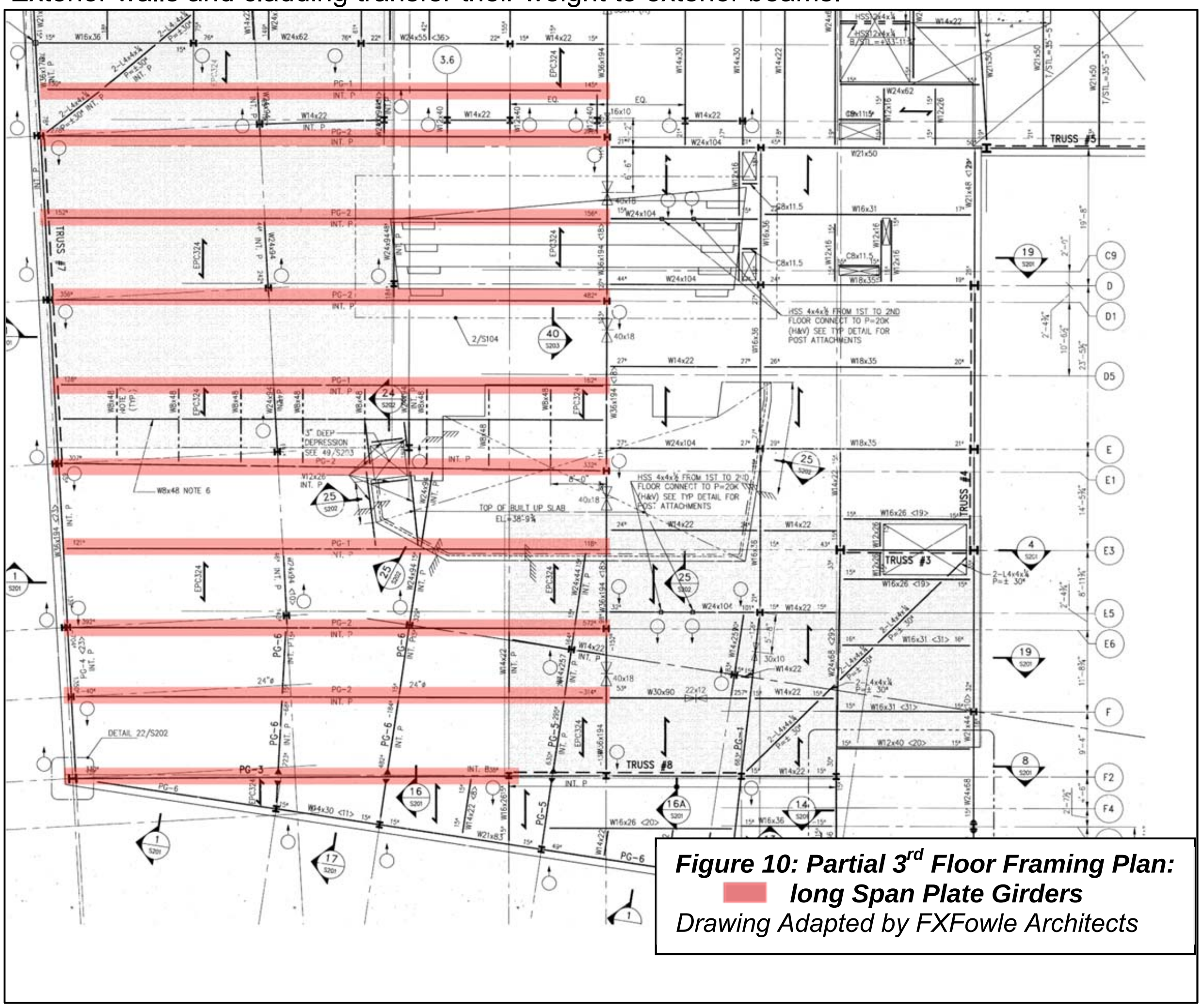


Figure 10: Partial 3rd Floor Framing Plan:
long Span Plate Girders
Drawing Adapted by FXFowle Architects

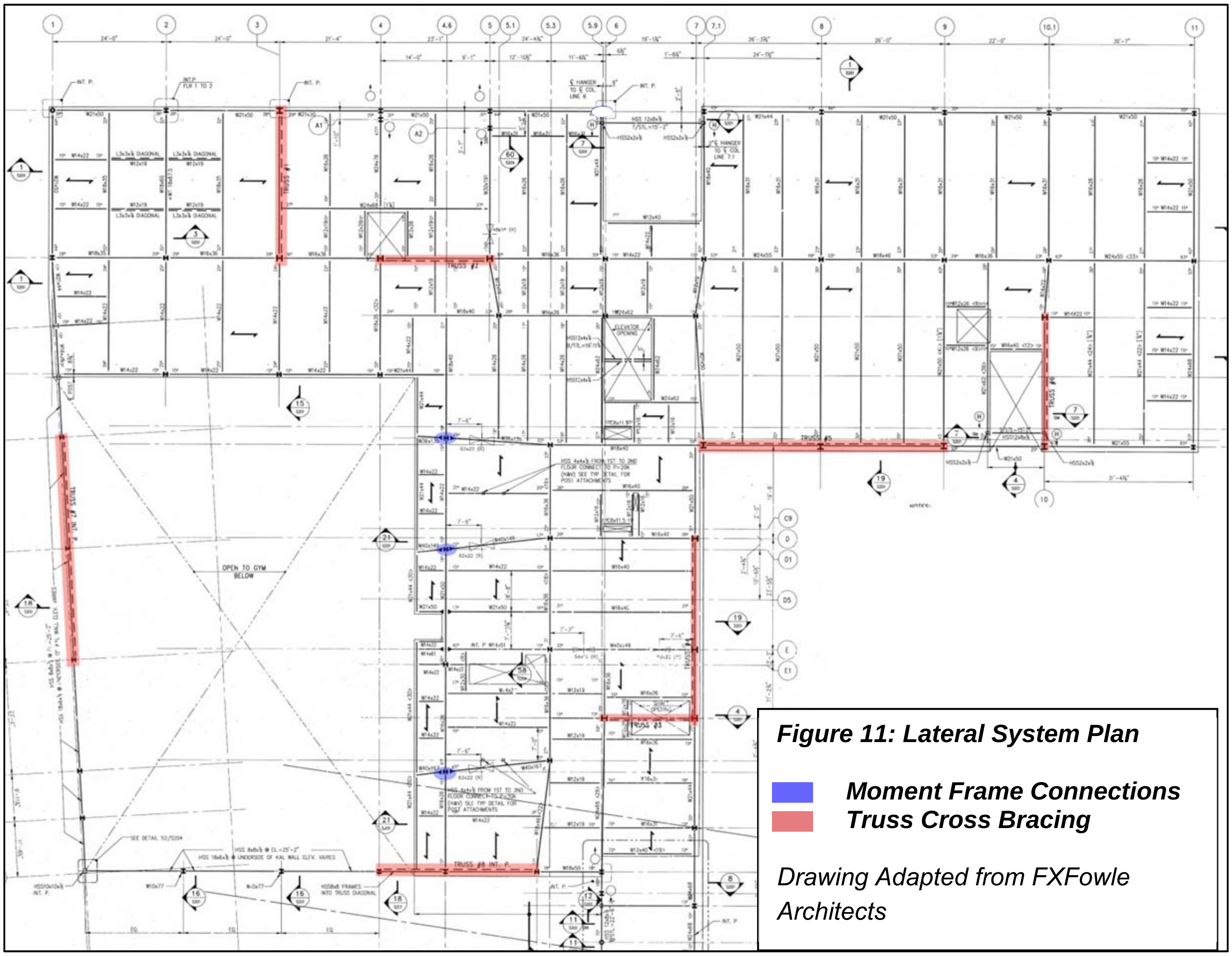


Figure 11: Lateral System Plan

Moment Frame Connections
Truss Cross Bracing

Drawing Adapted from FXFowle Architects

Lateral System

The lateral force resisting system consists of both HSS and wide flange lateral truss bracing (red box, ***Figure 10***), along with steel moment connections at columns around the gymnasium space (blue circles, ***Figure 11***). There are six different types of truss bracing systems, two of which are shown in **Figure 12** to the right. Single bay trusses are primarily used along interior spaces, while stronger double bay trusses are implemented along the exterior wall where there is more room. Several of the truss systems allow for architectural use and have odd cross bracing, such as the left truss in **Figure 12.** Trusses run in both the N-S and E-W directions. The first floor implements lateral force resisting systems the most. This is due to the 2 story cavity formed in the framing system to allow for open gym space.

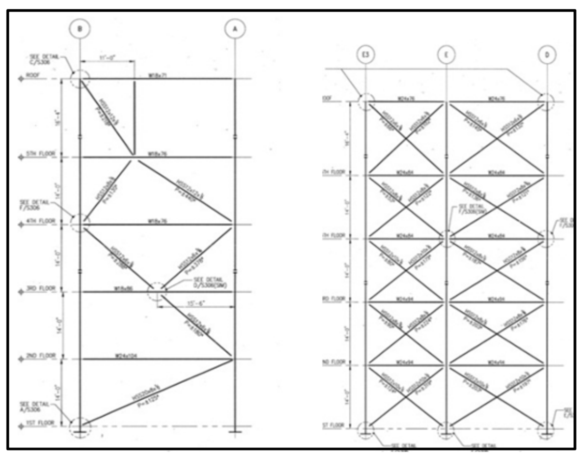


Figure 12: Two types of lateral bracing used in the design
Drawing by FXFowle Architects

DESIGN CRITERIA

This section provides data regarding codes, materials, and gravity loads for the design of Hunter's Point South. This thesis project will differ from the original design in that it will implement design criteria from ASCE7-10 and IBC 2009 rather than the NYCBC 2008 building code. There are several reasons for doing this. First of all, obtaining outdated copies of the NYCBC and other code books is not an option due to availability. The NYCBC also references the IBC and ASCE7 throughout; so much of the design will be the same. The only issue with using newer codes is that they may have different design procedures, which may change the design slightly. However, I feel using codes up to today's standards will be most beneficial to me as I go from analysis to redesign.

CODES & REFERENCES

Design Codes

Building Code

- New York City Building Code, NYCBC 2008, (2008)

Reference Codes

- American Concrete Institute Building Code, ACI 318-02, (2002)
- American Institute of Steel Construction, AISC 9th edition (1989)

Thesis Codes

Building Code

- International Building Code, IBC 2009 (2009)

Reference Codes

- American Concrete Institute Building Code, ACI 318-08 (2008)
- American Institute of Steel Construction, AISC 14th edition (2011)
- American Society of Civil Engineers, ASCE 7-10 (2010)

MATERIAL STRENGTHS

Design Materials and strengths were found in the construction drawings on page S001.

Table 1: Material Strengths

Material Strengths			
Material	**Element**	**Type**	**Strength**
Cast-in-Place Concrete	Pile Caps under Columns	Normal Weight Concrete	f'c= 5950 psi
	Grade & Strap Beams	Normal Weight Concrete	f'c= 4000 psi
	Column Pier and Buttress	Normal Weight Concrete	f'c= 4000 psi
	Slab on Grade	Normal Weight Concrete	f'c= 4000 psi
	Floor Slab	Light Weight Concrete	f'c= 3500 psi
Reinforcing Steel	Concrete Reinforcing bars		FY= 60 ksi
	Caisson Steel threadbars		Fy= 75 ksi
Structural Steel	Steel Wide Flange Members	ASTM A992	Fy= 50 ksi
	Steel HSS Tubes	ASTM A500	Fy= 46 ksi
	Steel Base Plates	ASTM A572 gr 50	Fy= 50 ksi
	Steel Deck	ASTM A653	Fy= 40 ksi
	Steel Bolts	ASTM A325	Fu= 120 ksi
		ASTM A490	Fu= 150 ksi

DESIGN LOADS

Hunter's Point South was designed for gravity loads using the Allowable Strength Design (ASD) Method. This thesis project will implement the Load and Resistance Factor Design (LRFD) Method instead due to the fact that it is becoming the industry standard. All thesis design loads have been taken from tables out of ASCE7-10 unless original design loads controlled.

Table 2: Dead Loads

Dead Load		
	Design (psf)	**Thesis (psf)**
NW Concrete	150	150
LW Concrete + Deck	49	49
Masonry Wall	90	90
Roof Paver	15	15
MEP	20	25
Ceiling	10	
Partitions	12	12
Curtain Wall	20	20

Table 3: Live Loads

Live Load		
	Design (psf)	ASCE7-10
first floor, lobby, stair, corridor	100	100
classrooms	40	40
art room/ science lab	60	60
office	50	50
library stacks	100	**150**
library reading	60	60
mechanical space	75	**100**
book storage	150	150
roof (main)	45	45
Gymnasium	100	100
Cafeteria	100	100
Kitchen	150	150
Auditorium Stage	150	150
toilets	60	60
terrace	100	**1.5LL<100psf**
corridor 2nd floor+	80	80
Auditorium	100	100
stadium seating	60	60

Table 4: Snow Loads

Snow Load		
	Design	ASCE7-10
Ground Snow Load:	25 psf	25
Flat Roof Snow Load	22 psf	22
Snow Exposure Factor CB	1.1	1.1
Snow Load Importance IS	1.1	1.1
Thermal Factor Ct	1.0 main roof/terrace 1.1 mech. bulkhead	1

DESIGN ANALYSIS

WIND LOAD SUMMARY

Wind load analysis of the Main Wind Force Resisting System (MWFRS) was determined using ASCE7-10 Chapter 26 and 27. Per this chapter, the building was designed as an enclosed building in Exposure Category C. The building was simplified to a rectangular shape with legs the size of the longest dimensions in each direction for this analysis. This simplification prevents the need to go into further analysis to determine the effects different floor shapes will have in wind loading. Hand calculations and Microsoft Excel were used to come up with net wind pressures, story shear forces, and overturning moments for both the North-South and East-West directions. Windward, leeward, and internal pressures were taken into account when calculating wind pressures. ***Table 5 and 6*** summarize the process used in AISC7-10 to come up with the values chosen. See **Appendix A** for all wind load calculations.

North-South Direction

Results of wind load analysis in the N-S direction can be found in ***Table 7 and 8*** and in ***Figure 13 and 14*** on the next several pages. Due to the simplified shape, wind forces are equal in both the N-S and S-N directions. Once pressures for windward, internal, and leeward sides are calculated, wind load forces can be obtained. Resultant wind forces are shown on the windward side of ***Figure 14.*** The total base shear force due to wind loading is 1322 kip, and the overturning moment in this direction is about 61,324 k-ft. Although a wind load analysis was included in the original design drawings, no force results are included. Therefore, I am unable to check my numbers to see how they compare to the original design

East-West Direction

Results of wind load analysis in the E-W direction can be found in ***Table 9 and 10*** and in ***Figure 15 and 16*** on pages 15-16. Due to the simplified shape, wind forces are equal in both the E-W and W-E directions. Once pressures for windward, internal, and leeward sides are calculated, wind load forces can be obtained. Resultant wind forces are shown on the windward side of ***Figure 16.*** Total base shear force due to wind in this direction is 924 kip, and the overturning moment is 44,259 k-ft. This is slightly lower than the wind load forces in the N-S direction due to the shorter building length in that direction.

(Note: forces are calculated assuming a rectangular building, but forces are shown on actual building elevations to help show force locations).

Table 5: Wind Load Design Criteria

Windload Design Criteria			
Per ASCE7-10		N-S	E-W
Risk Category		III	
Importance Factor		1	
Exposure		C	
Surface Roughness		B	
V		130	
K_d		0.85	
K_{zt}		1	
n_a		1.03	
G		0.85	
K_h		1.19	
h		72.3	
L		175	240.5
B		240.5	175
L/B		0.728	1.374
h/l		0.413	0.301
C_p Windward		0.8	
C_p Leeward		-0.5	-0.425
C_p Side		-0.7	
C_p Roof		0 to h/2	-0.9
		h/2 to h	-0.9
		h to 2h	-0.5
		>2h	-0.3
Reduction Factor		0.8	
GC_{pi}		+/-0.18	
K_h		1.179	
q_z		43.36	
q_p		45.30	
GC_{pn} Windward		1.5	
GC_{pn} Leeward		-1	

Table 6: Velocity Pressure

Velocity Pressure			
Level	Height	K_z	q_z
Parapet	87.3	1.232	45.30
Roof	72.3	1.179	43.36
5	56	1.114	40.97
4	42	1.050	38.61
3	28	0.964	35.45
2	14	0.850	31.26
1	0	0.850	31.26

Notes:

- Due to its location on the Bay, NYC Building Code requires this structure to be Risk Category III and Exposure C.
- Using the velocity maps in ASCE7-10, a design wind velocity of 130mph is used.
- Due to its location near the shore, the original design calls for protected glazing on the entire building. Therefore, the building is assumed to be enclosed and a GC_{pi} of +/-0.18 is chosen for calculations.
- Using AISC7-10 design guide, the other factors are chosen and plugged into the story pressure equation.

Table 7: Wind Pressure: North-South Direction

Wind Pressure: North-South Direction						
Story Level	Floor to Floor Height (ft)	Story Height (ft)	Wind Pressure (psf)	Internal Pressure (psf)	Net Pressure -GCpi (psf)	Net Pressure +GCpi (psf)
Roof	15	72.3	29.488	+/- 7.80	21.68	37.29
5	16.3	56	27.857	+/- 7.80	20.05	35.66
4	14	42	26.257	+/- 7.80	18.45	34.06
3	14	28	24.106	+/- 7.80	16.30	31.91
2	14	14	21.256	+/- 7.80	13.45	29.06
1	14	0	21.256	+/- 7.80	13.45	29.06
Parapet	Windward	87.3	67.954	-	-	-
	Leeward	87.3	-45.302	-	-	-
Leeward	-	-	-18.430	+/- 7.80	-26.23	-10.62
Roof	0 to 36.15ft	-	-33.174	+/- 7.80	-40.97	-25.36
	36.15-72.3ft	-	-33.174	+/- 7.80	-40.97	-25.36
	72.3-144.6ft	-	-18.430	+/- 7.80	-26.23	-10.62
	144.6-175ft	-	-11.058	+/- 7.80	-18.86	-3.25

Table 8: Wind Loads: North-South Direction

Wind Loads: North-South Direction							
Story Level	Floor to Floor Height (ft)	Story Height (ft)	Windward (kip)	Leeward (kip)	Total Story Force (kip)	Total Story Shear (kip)	Overturning Moment (ft-k)
Parapet	15	87.3	122.6	-81.7	204.3	1322.3	16302.0
Roof	16.3	72.3	135.9	-95.6	231.5	1118.0	16735.4
5	14	56	120.1	-88.3	208.4	886.5	11671.1
4	14	42	114.7	-88.3	203.0	678.1	8527.0
3	14	28	107.4	-88.3	195.8	475.1	5481.9
2	14	14	97.8	-88.3	186.2	279.3	2606.6
1	14	0	48.9	-44.2	93.1	93.1	0.0
			∑			1322.3	61323.9

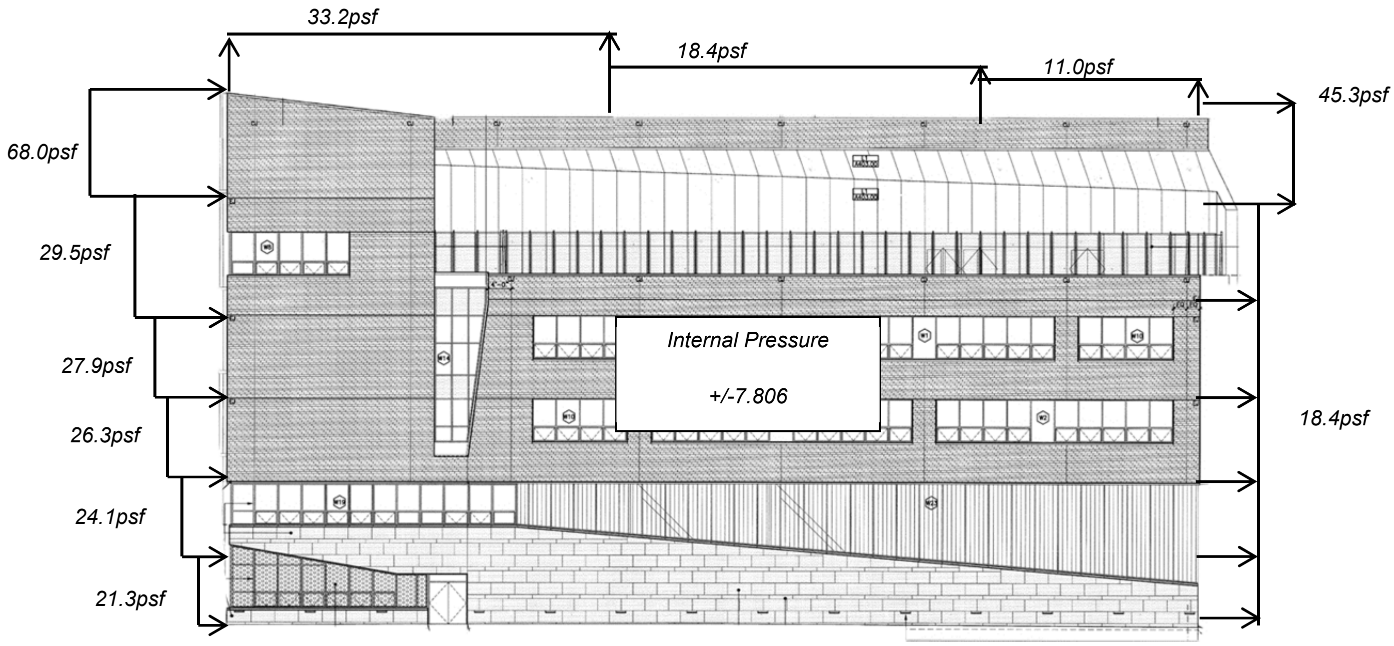


Figure 13: Wind Pressures, N-S Direction *Drawing Adapted from FXFowle Architects*

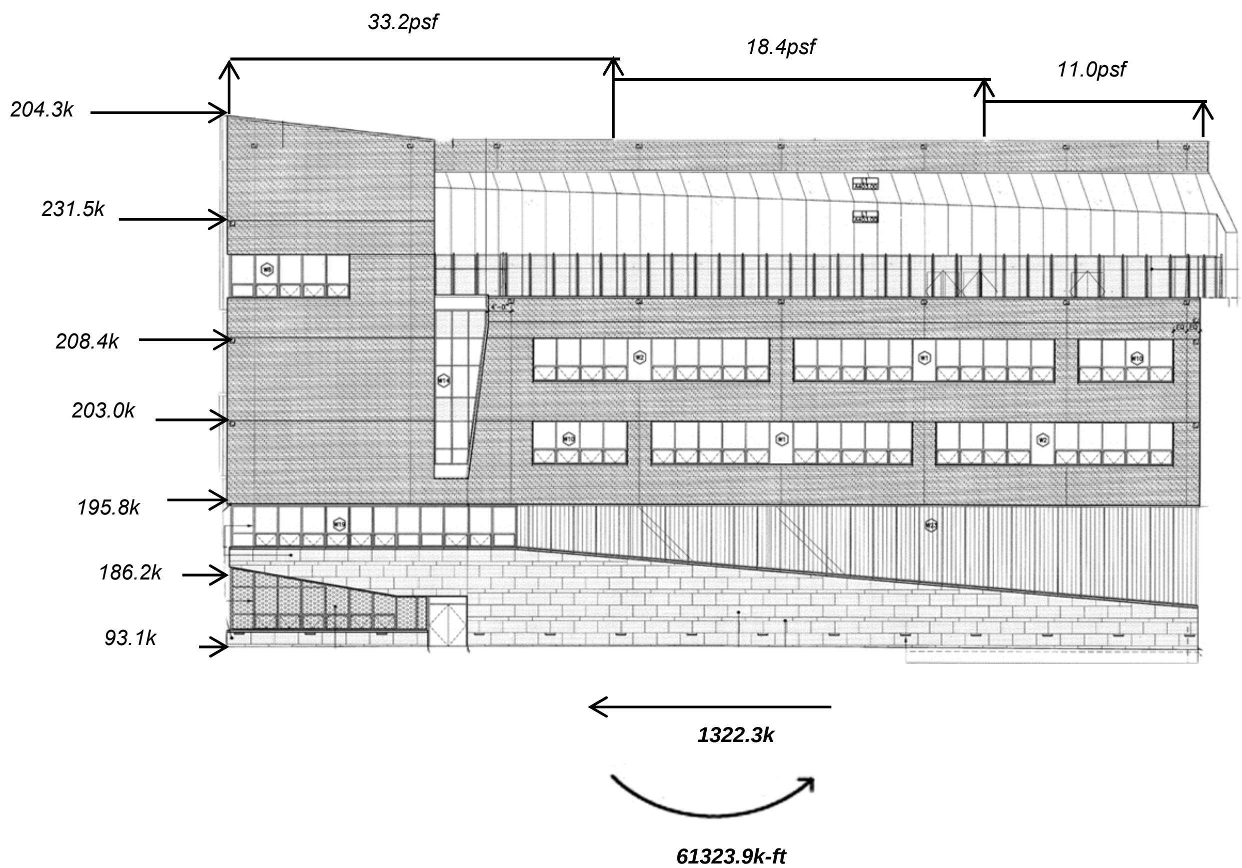


Figure 14: Wind Forces, N-S Direction
Drawing Adapted from FXFowle Architects

Table 9: Wind Pressure: East-West Direction

Wind Pressure: East-West Direction						
Story Level	**Floor to Floor Height (ft)**	**Story Height (ft)**	**Wind Pressure (psf)**	**Internal Pressure (psf)**	**Net Pressure -GCpi (psf)**	**Net Pressure +GCpi (psf)**
Roof	15	72.3	29.488	+/- 7.806	21.682	37.293
5	16.3	56	27.857	+/- 7.80	20.05	35.66
4	14	42	26.257	+/- 7.806	18.451	34.063
3	14	28	24.106	+/- 7.80	16.30	31.91
2	14	14	21.256	+/- 7.806	13.450	29.061
1	14	0	21.256	+/- 7.80	13.45	29.06
Parapet	Windward	87.3	67.954	-	-	-
	Leeward	87.3	-45.302	-	-	-
Leeward	-	-	-15.665	+/- 7.807	-23.471	-7.860
Roof	0 to 36.15ft	-	-33.174	+/- 7.80	-40.97	-25.36
	36.15-72.3ft	-	-33.174	+/- 7.807	-40.979	-25.368
	72.3-144.6ft	-	-18.430	+/- 7.80	-26.23	-10.62
	144.6-240.5ft	-	-11.058	+/- 7.807	-18.864	-3.252

Table 10: Wind Loads: East-West Direction

Wind Loads: East-West Direction							
Story Level	**Floor to Floor Height (ft)**	**Story Height (ft)**	**Windward (kip)**	**Leeward (kip)**	**Total Story Force (kip)**	**Total Story Shear (kip)**	**Overturning Moment (ft-k)**
Parapet	15	87.3	89.2	-59.5	148.6	924.3	12977.0
Roof	16.3	72.3	98.9	-62.2	161.1	775.7	11647.6
5	14	56	87.4	-57.5	144.9	614.6	8113.2
4	14	42	83.5	-57.5	141.0	469.7	5920.2
3	14	28	78.2	-57.5	135.7	328.7	3799.3
2	14	14	71.2	-57.5	128.7	193.1	1801.9
1	14	0	35.6	-28.8	64.4	64.4	0.0
			∑			924.3	44259.1

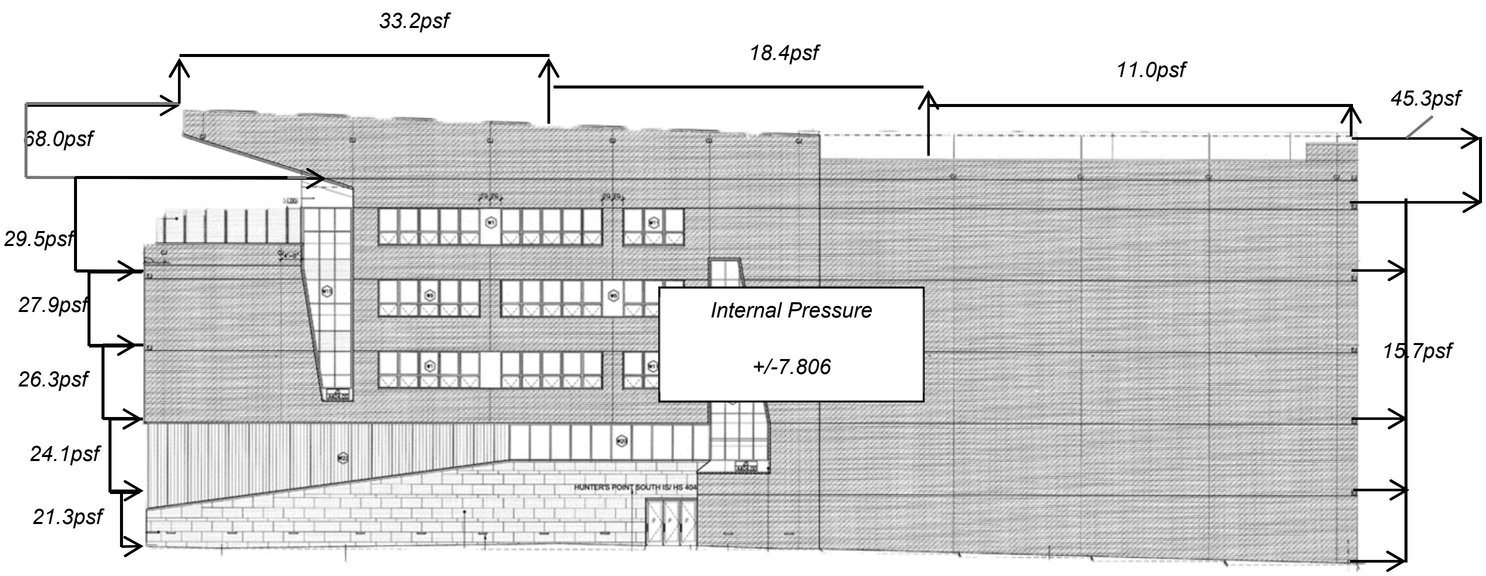


Figure 15: Wind Pressures, E-W Direction
Drawing Adapted from FXFowle Architects

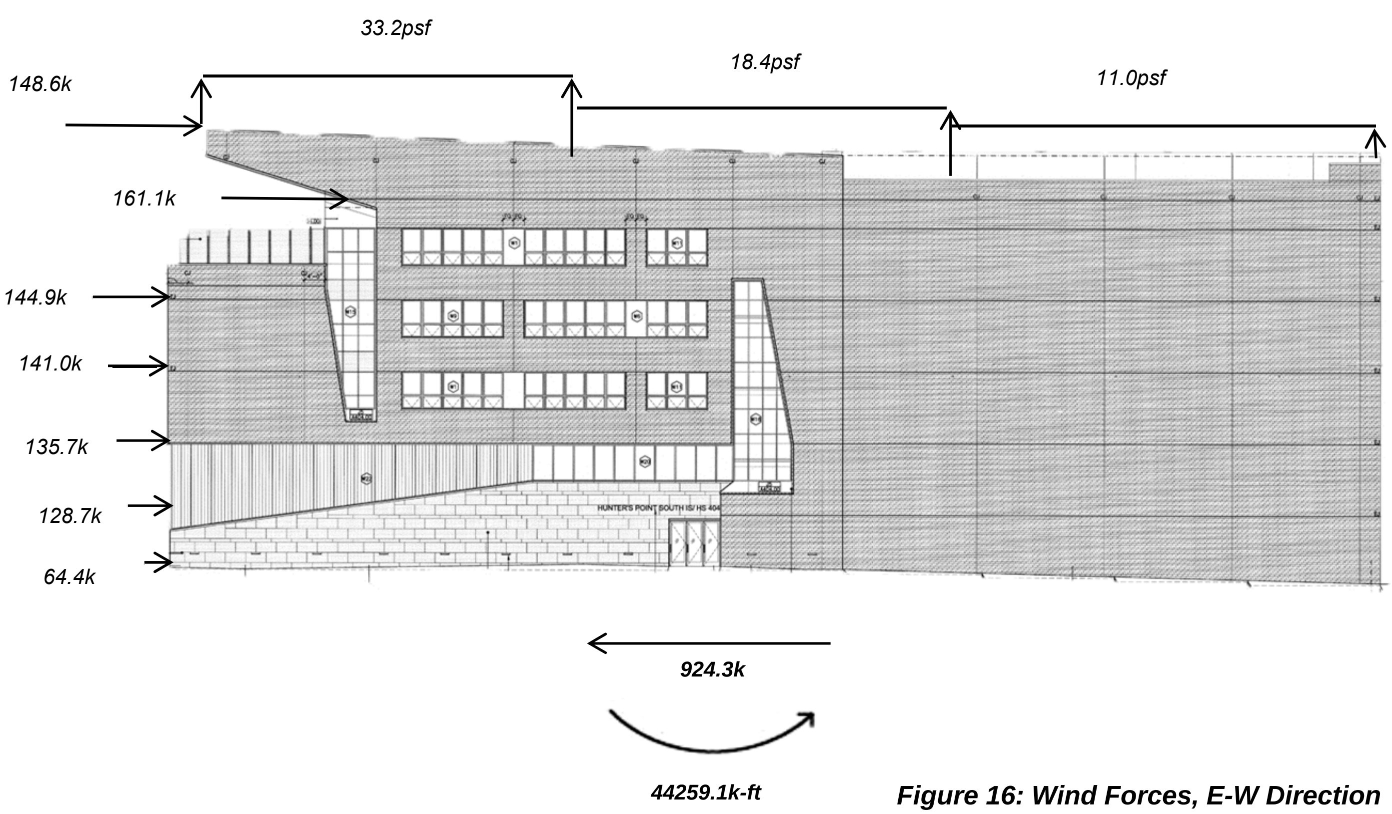


Figure 16: Wind Forces, E-W Direction
Drawing Adapted from FXFowle Architects

DESIGN ANALYSIS

SEISMIC LOAD SUMMARY

Seismic load analysis was done following the Equivalent Lateral Force Procedure (ELFP) in Chapter 12 of ASCE7-10. Building weight was determined using the structural floor plan drawings, then entered into an Excel file to calculate individual story forces and shear and overturning moment at the base. Using the method prescribed in ELFP , a building period of 0.794 seconds was determined. However, after doing an in-depth study of stiffness using a 3D computer model of the lateral system in ETABS, a building period of 0.882 seconds is found to be more accurate. Total building weight of the structure is roughly 13,300 kips (***Table 11)***. It should be noted that the weight of the third floor is on the high side due to heavy plate girders placed at long spans over the gymnasium. Seismic load calculations can be found in ***Appendix B.***

Using figure 22 of ASCE7-10, the design spectral response accelerations for short periods and period = 1 are determined. Then all other factors required for ELFP are found. It is important to note that the ductility of steel connections (detailing) in this design are not adequate to satisfy a special seismic resistance factor, so an R=3 is used. Using calculated floor weights, the base shear is determined. Story forces and moment are also found.

North-South Direction

After correcting building for stiffness and period, results are formulated. ***Table 12*** shows a base shear of 1067 kips and overturning moment of 6986 kip-feet in the N-S direction. A breakdown of individual story forces can be found in ***Figure 17***. The original analysis done for this building came up with a base shear of 1061 k. This means the analysis in this report differs by 0.6%. This small difference can be attributed to several reasons. The original design analysis used the 2008 NYC Building Code which could give different values when completing the reference analysis. Also, when determining floor weights, this report took slightly higher dead load weights than the original design reported (along with a more detailed analysis of weight), which could increase story forces and ultimately the base shear.

Table 11: Floor Weights

Floor Weight:	
floor	weight
roof	2944.57
5th	2563.12
4th	2277.47
3rd	3499.68
2nd	1977.5
Total	**13262.3**

East-West Direction

Table 13 shows a base shear of 1067 kips and overturning moment of 9491 kip-feet in the E-W direction. A breakdown of individual story forces can be found in ***Figure 18***. The increase of the overturning moment can be attributed to a longer effective building width in that direction.

Table 12: North-South Direction Loading

North-South Direction Loading

$T=$ 0.882 s
$k=$ 1.191
$V_b=$ 1067 kips

i	h_i	h	w	$w*h^k$	C_{VX}	f_i	v_i	B_X	$5\%B_y$	A_x	M_Z
	ft	ft	kips			kips	kips	ft	ft		k-ft
6	16.33	72.33	2945	482573	0.396	423	423	131	7	1	2766
5	14	56	2563	309691	0.254	271	694	131	7	1	1775
4	14	42	2277	195314	0.160	171	865	131	7	1	1120
3	14	28	3500	185228	0.152	162	1027	131	7	1	1062
2	14	14	1978	45848	0.038	40	1067	131	7	1	263
1											
		∑	13263	1218654		1067	=V				6986

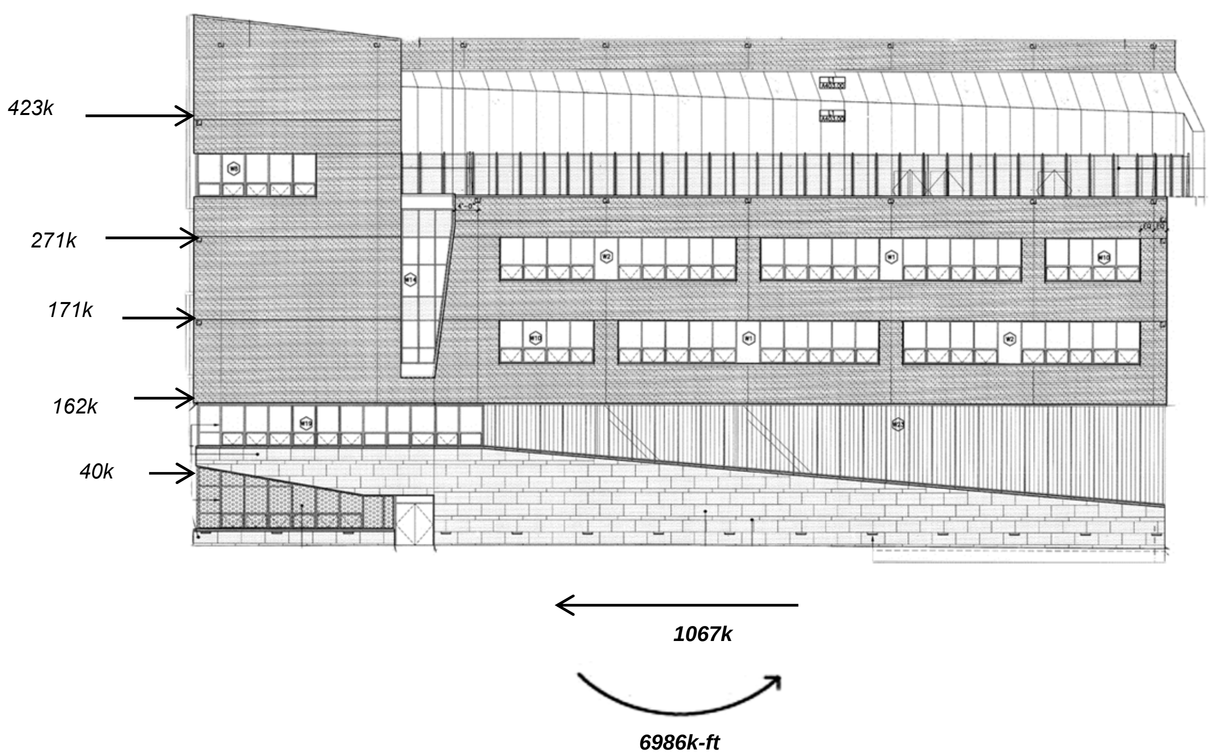


Figure 17: Seismic Forces, N-S Direction
Drawing Adapted from FXFowle Architects

Table 13: East-West Direction Loading

East-West Direction Loading											
									T=	0.882 s	
									k=	1.191	
									V_b=	1067 kips	
i	h_i	h	w	$w*h^k$	C_{VX}	f_i	v_i	B_y	$5\%B_y$	A_x	M_Z
	ft	ft	kips			kips	kips	ft	ft		k-ft
6	16.33	72.33	2945	482573	0.396	423	423	178	9	1	3759
5	14	56	2563	309691	0.254	271	694	178	9	1	2412
4	14	42	2277	195314	0.160	171	865	178	9	1	1521
3	14	28	3500	185228	0.152	162	1027	178	9	1	1443
2	14	14	1978	45848	0.038	40	1067	178	9	1	357
1											
		∑	**13263**	**1218654**		**1067**	**=V**				**9491**

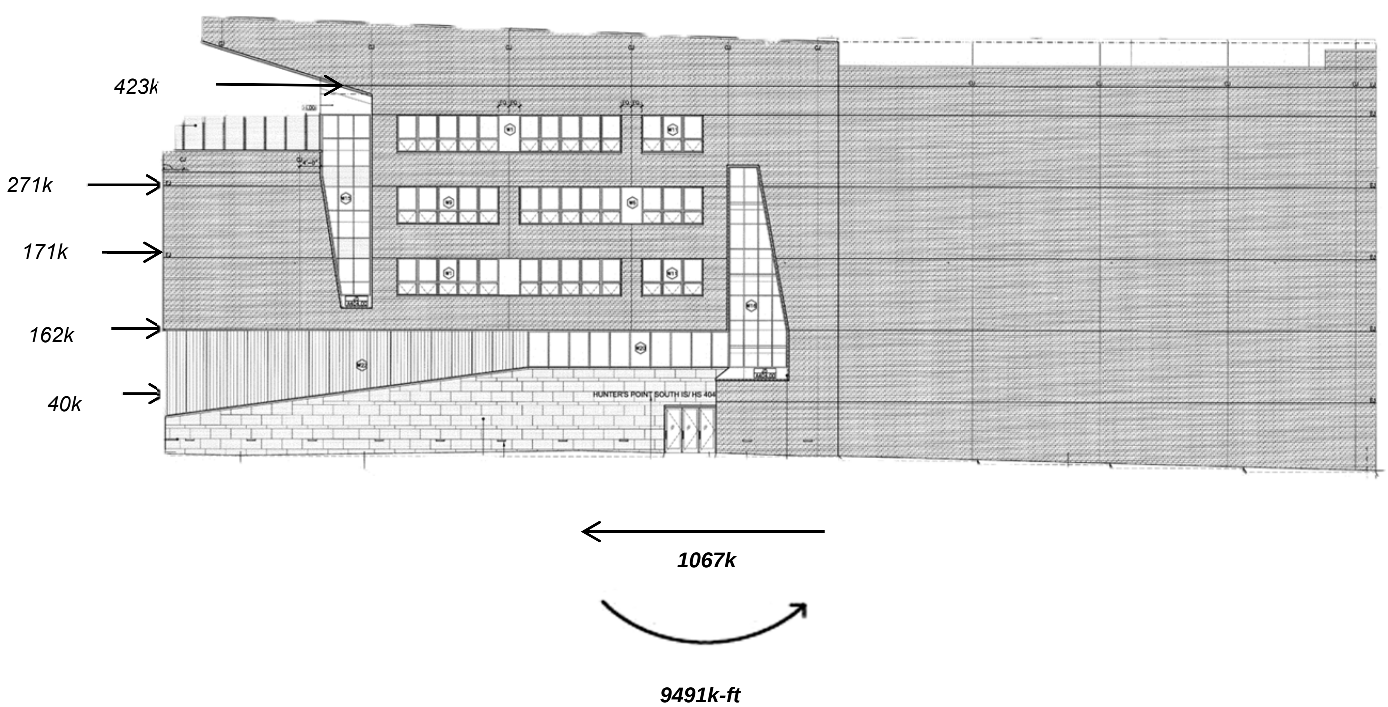


Figure 18: Seismic Forces, E-W Direction
Drawing Adapted from FXFowle Architects

LATERAL SYSTEM IN-DEPTH ANALYSIS

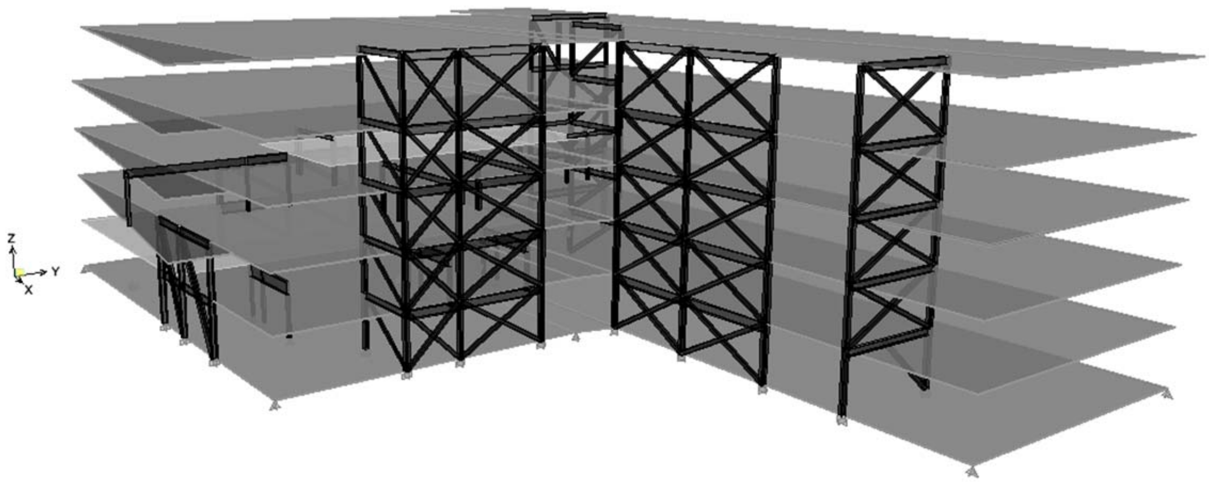


Figure 19: ETABS MODEL: Lateral Force Resisting System

LATERAL SYSTEM IN-DEPTH ANALYSIS

RELATIVE STIFFNESS OF LATERAL ELEMENTS

The relative stiffness of each lateral element is important because it correlates to the amount of contribution that specific system will have in the overall lateral force resisting system. Relative stiffness is equal to the force applied to a system divided by the displacement caused by that force. To calculate stiffness (k), the following equation can be used:

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

Using STAADPro, each lateral system from ***Figure 20*** was modeled with a 1 kip load acting at the top story, and the maximum displacement of the top right corner was recorded. This is done for fixed and pinned base truss systems in ***Figures 22-25*** on pages 24-25. Then, this is done for fixed/ pinned base moment frames in ***Figures 26-29*** on pages 26-27. Using the above equation, the stiffness of each system was obtained. ***Table 14*** and ***Table 15*** on page 28 show relative stiffness for all the truss elements.

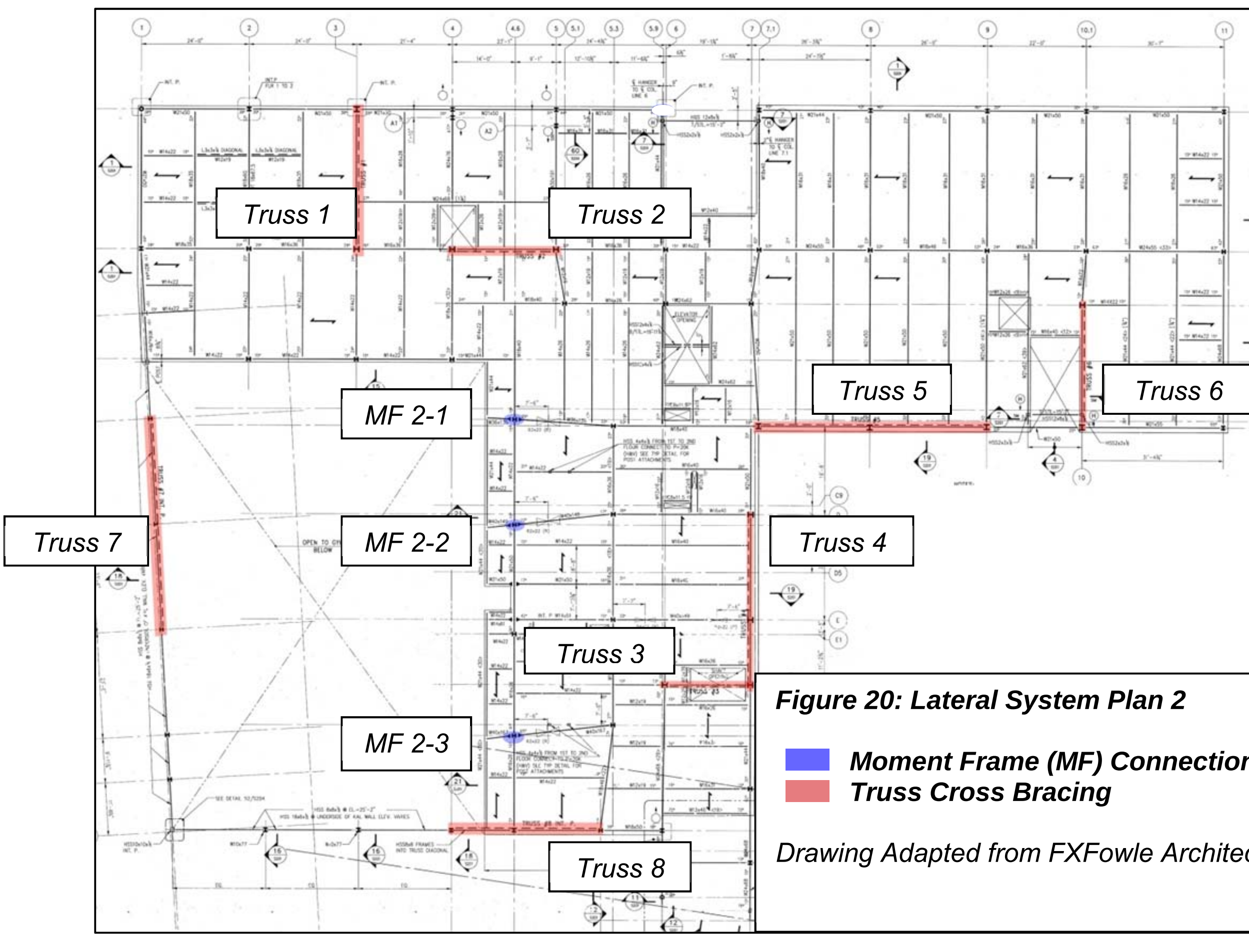


Figure 20: Lateral System Plan 2

Moment Frame (MF) Connections
Truss Cross Bracing

Drawing Adapted from FXFowle Architects

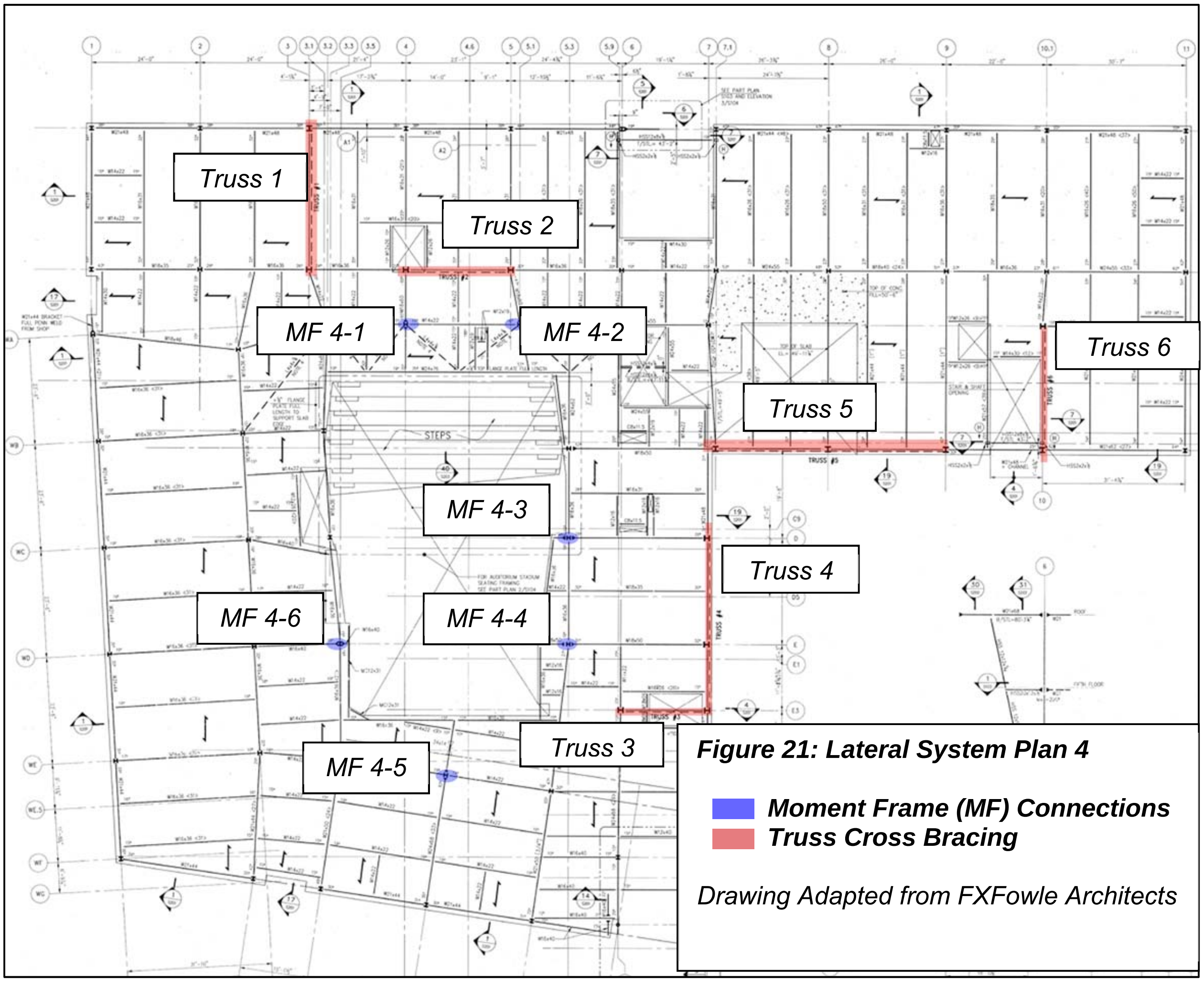


Figure 21: Lateral System Plan 4

Fixed Base Assumption:

North-South Trusses

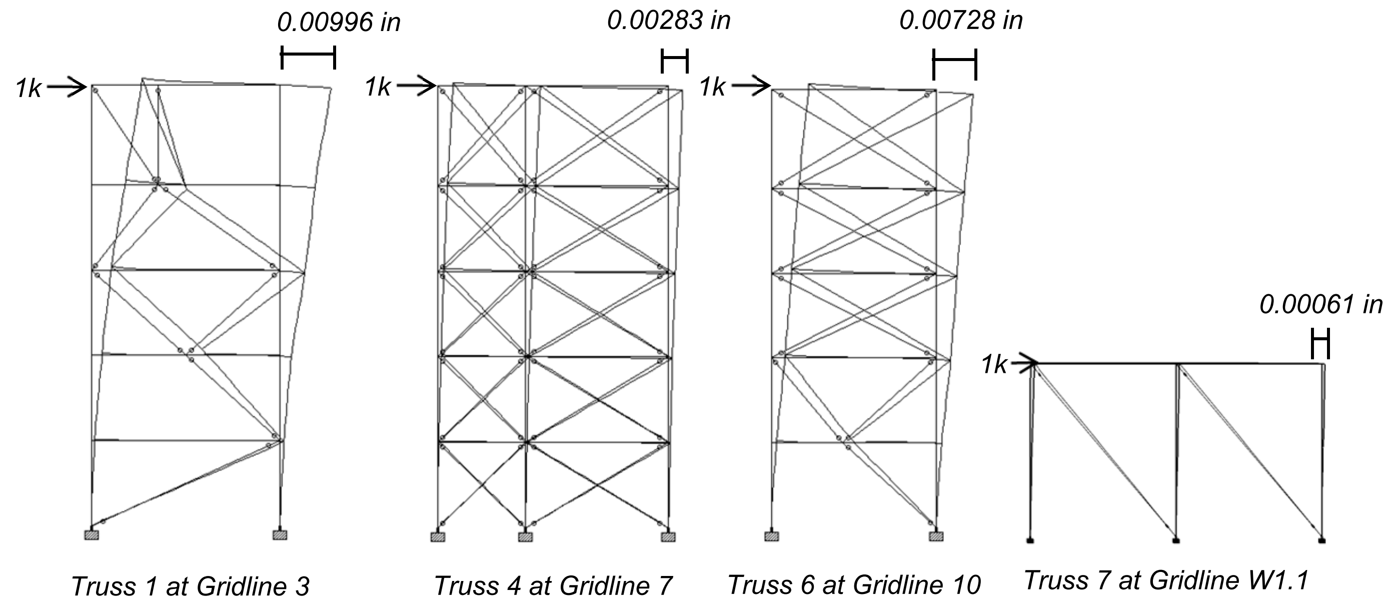


Figure 22: *P & Δ: North-South Frames (fix)*

Fixed Base Assumption:
East-West Trusses

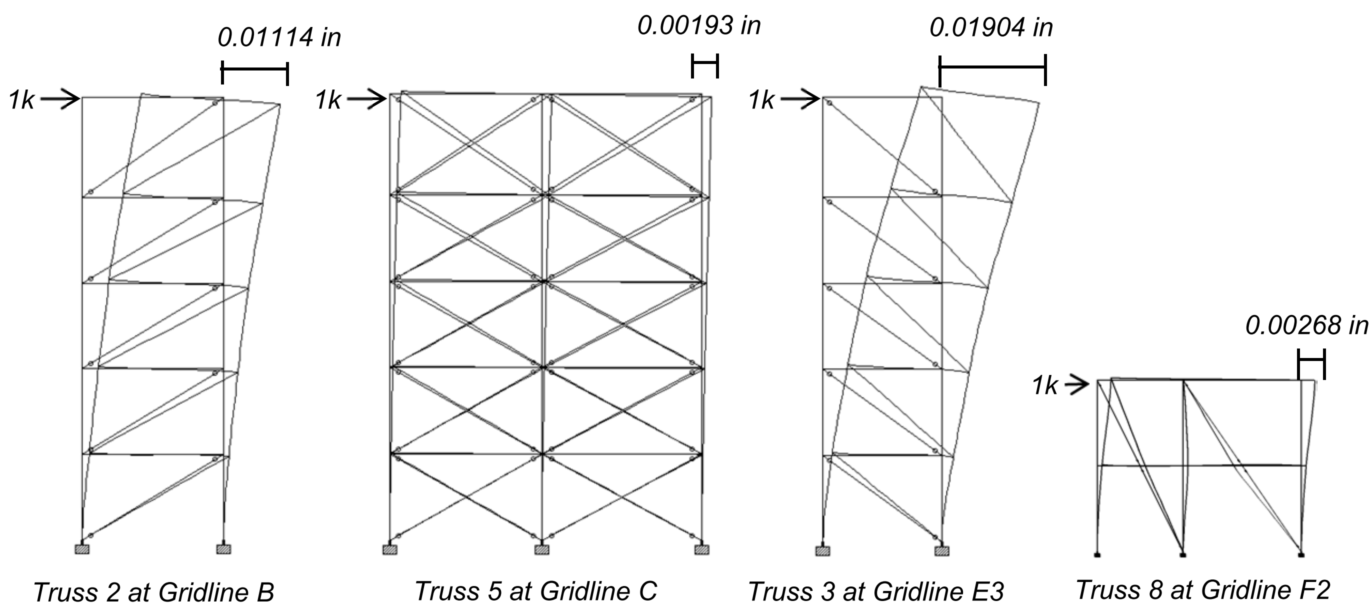


Figure 23: *P & Δ: East-South Frames (pin)*

Pinned Base Assumption:
North-South Trusses

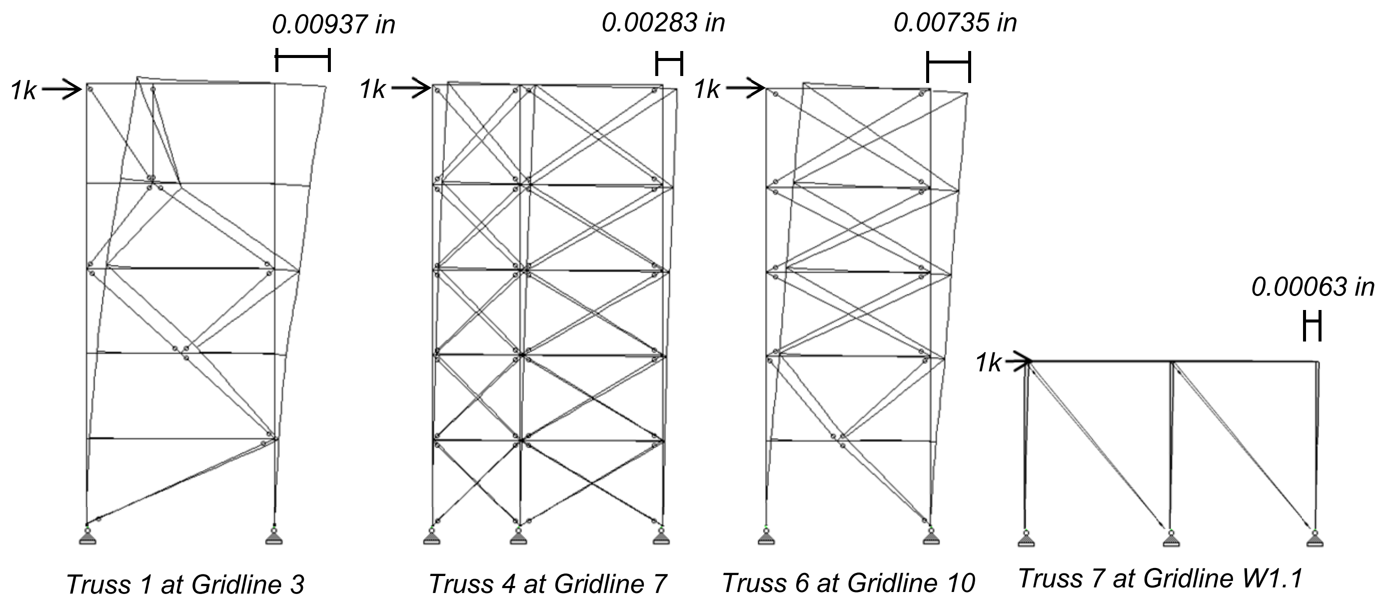


Figure 24: *P & Δ: North-South Frames (fix)*

Pinned Base Assumption:
East-West Trusses

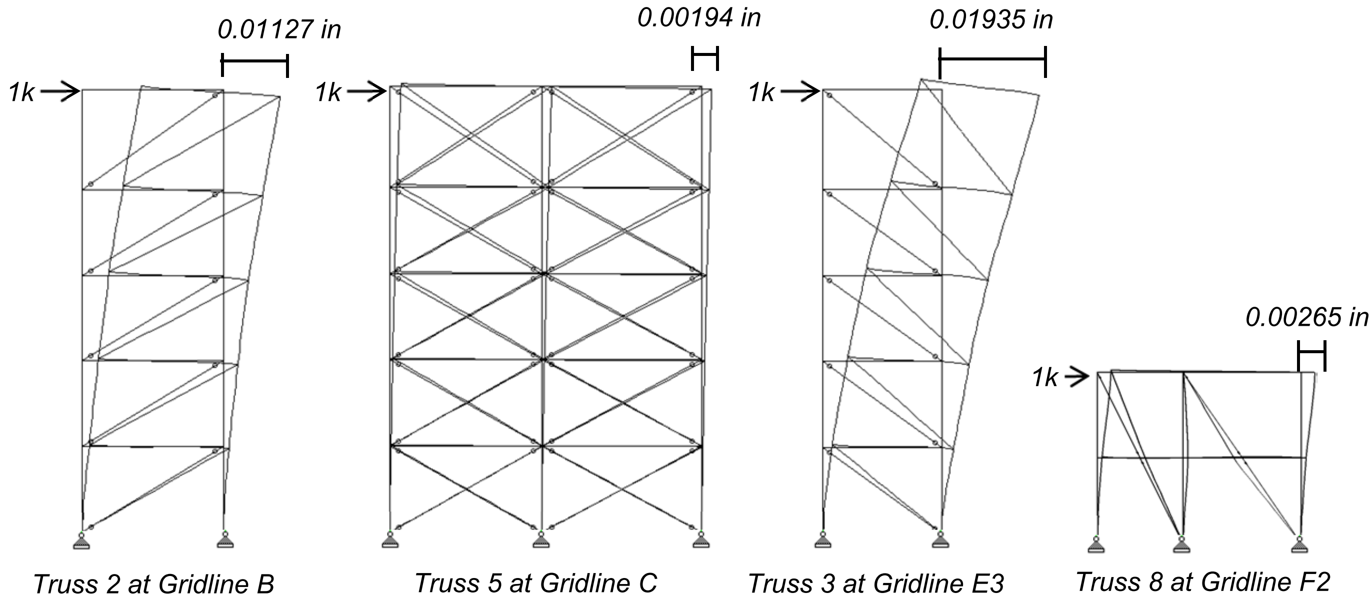


Figure 25: *P & Δ: East-South Frames (pin)*

Fixed Base Assumption:
Floor 2 Moment Frames

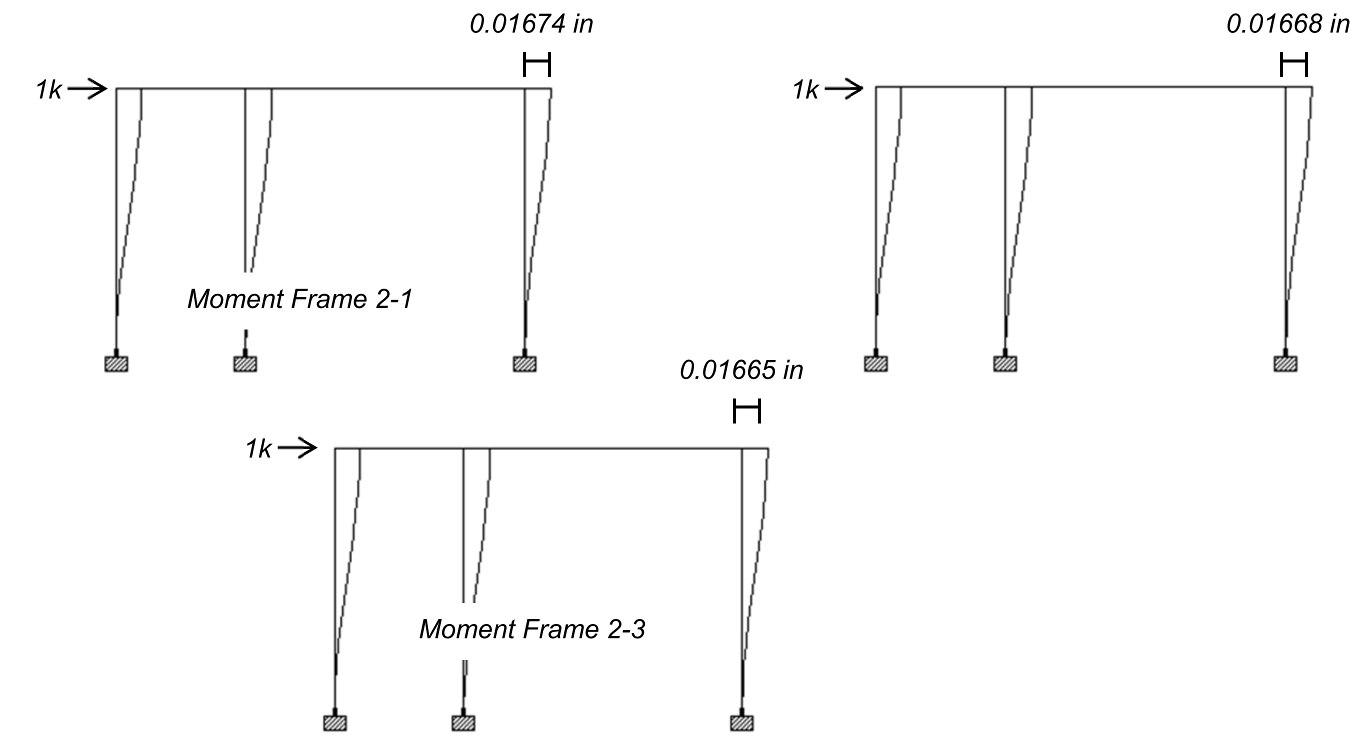


Figure 26: *P & Δ: 2nd Floor Mom. Frames (fix)*

Pinned Base Assumption:
Floor 2 Moment Frames

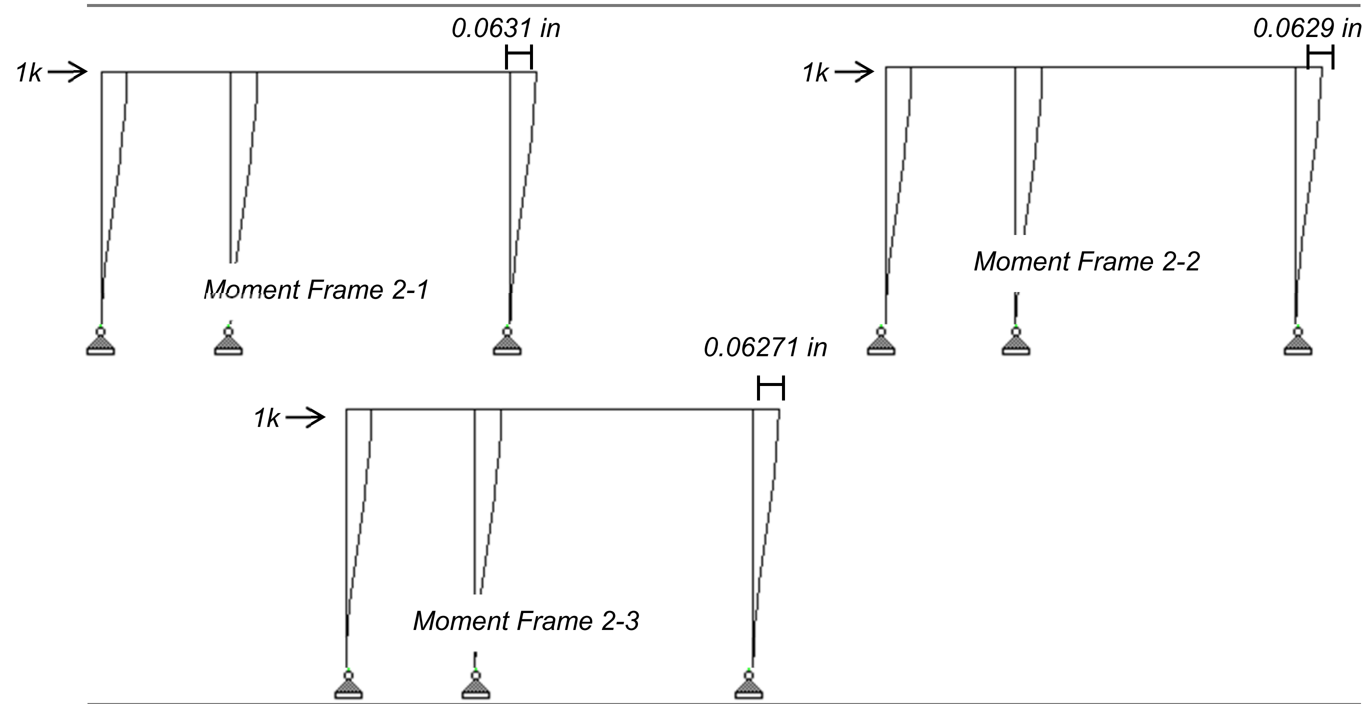


Figure 27: *P & Δ: 2nd Floor Mom. Frames (pin)*

Fixed Base Assumption:
Floor 4 Moment Frames

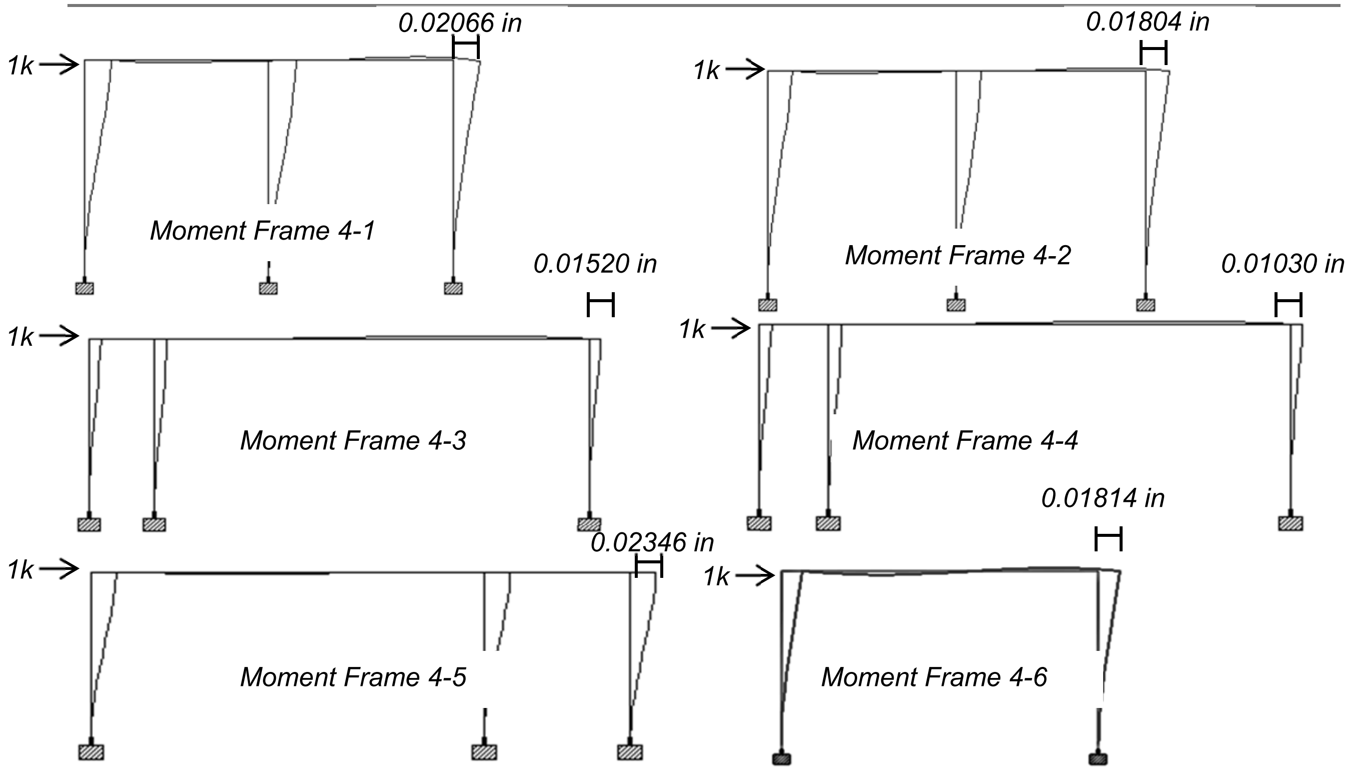


Figure 28: *P & Δ: 4th Floor Mom. Frames (fix)*

Pinned Base Assumption:
Floor 2 Moment Frames

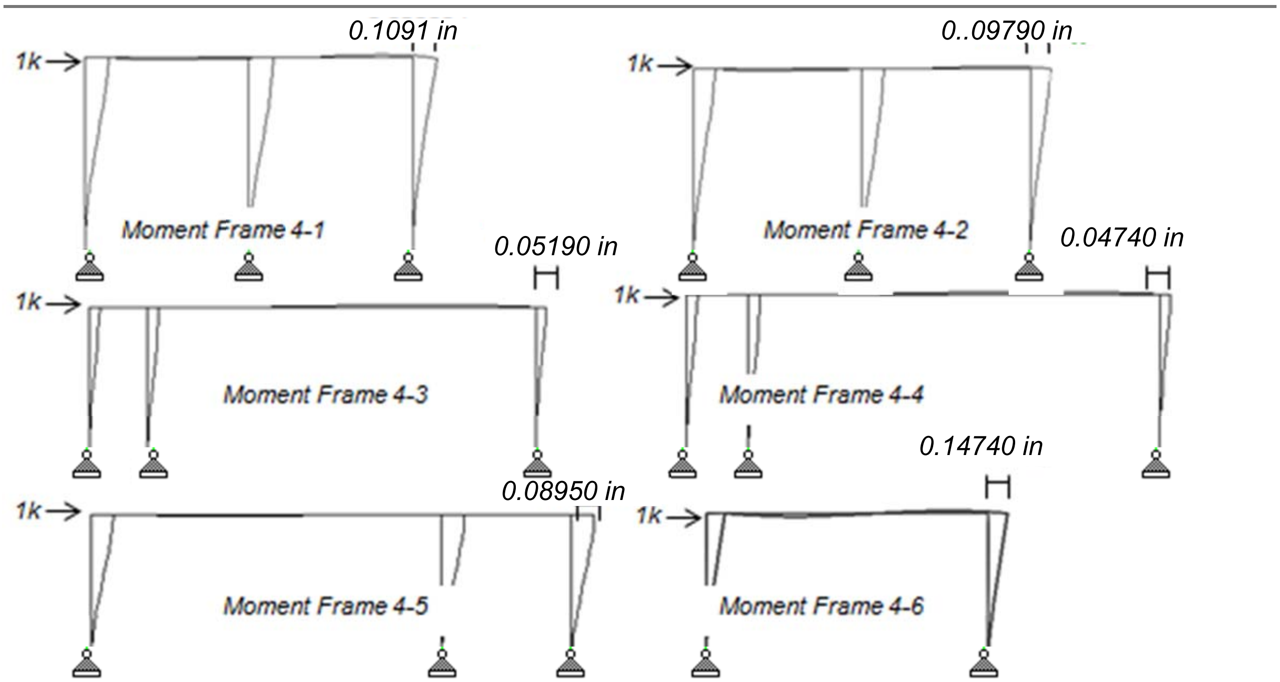


Figure 29: *P & Δ: 4th Floor Mom. Frames (pin)*

Table 14: Relative Stiffness for Truss Braces (Fixed Base Assumption)

Relative Stiffness				
Truss	**Load (P)**	**Displacement (Δ)**	**Stiffness (K)**	**% Contribution**
North-South	**Kip (k)**	**Inches (in)**	**(k/in)**	**in Lateral System**
Truss 1 at Grid 3	1.0	0.00996	100	10.65
Truss 4 at Grid 7	1.0	0.00283	353	37.49
Truss 6 at Grid 10	1.0	0.00728	137	14.57
Truss 7 at Grid W1.1	1.0	0.00065	205	21.75
Moment Frame 4-1	1.0	0.02066	48	5.14
Moment Frame 4-2	1.0	0.01804	55	5.88
Moment Frame 4-5	1.0	0.02346	43	4.52
		Σ=	943	100.00
East-West	**Load (P)**	**Displacement (Δ)**	**Stiffness (K)**	**% Contribution**
Truss 2 at Grid B	1.0	0.01114	90	6.27
Truss 5 at Grid C	1.0	0.00193	518	36.20
Truss 3 at Grid E3	1.0	0.01904	53	3.67
Truss 8 at Grid F2	1.0	0.00268	373	26.07
Moment Frame 2-1	1.0	0.01674	60	4.17
Moment Frame 2-2	1.0	0.01668	60	4.19
Moment Frame 2-3	1.0	0.01665	60	4.20
Moment Frame 4-3	1.0	0.01520	66	4.60
Moment Frame 4-4	1.0	0.01030	97	6.78
Moment Frame 4-6	1.0	0.01814	55	3.85
		Σ=	1431	100.00

Table 15: Relative Stiffness for Truss Braces (Pinned Base Assumption)

Relative Stiffness				
Truss	**Load (P)**	**Displacement (Δ)**	**Stiffness (K)**	**% Contribution**
North-South	**Kip (k)**	**Inches (in)**	**(k/in)**	**in Lateral System**
Truss 1 at Grid 3	1.0	0.00937	107	12.83
Truss 4 at Grid 7	1.0	0.00283	353	42.49
Truss 6 at Grid 10	1.0	0.00735	136	16.36
Truss 7 at Grid W1.1	1.0	0.00063	205	24.65
Moment Frame 4-1	1.0	0.10910	9	1.10
Moment Frame 4-2	1.0	0.09789	10	1.23
Moment Frame 4-5	1.0	0.08950	11	1.34
		Σ=	832	100.00
East-West	**Load (P)**	**Displacement (Δ)**	**Stiffness (K)**	**% Contribution**
Truss 2 at Grid B	1.0	0.01127	89	7.87
Truss 5 at Grid C	1.0	0.00194	515	45.69
Truss 3 at Grid E3	1.0	0.01935	52	4.58
Truss 8 at Grid F2	1.0	0.00265	377	33.45
Moment Frame 2-1	1.0	0.06310	16	1.40
Moment Frame 2-2	1.0	0.06285	16	1.41
Moment Frame 2-3	1.0	0.06271	16	1.41
Moment Frame 4-3	1.0	0.05190	19	1.71
Moment Frame 4-4	1.0	0.04740	21	1.87
Moment Frame 4-6	1.0	0.14740	7	0.60
		Σ=	1128	100.00

After calculating stiffness for each member, several comparisons can be made. First, a comparison is made between member types. It is shown that lateral truss bracing has a much higher stiffness. This can be attributed to the fact that they have bracing to help hold them together when a lateral load is applied, creating a more rigid system. Also, the stiffness is quite comparable in each direction on most floors, but floor 2 and floor 4 have much higher stiffness in the E-W direction. Because the gym and auditorium spaces take out so much of the structure on these floors, steel moment frames need to be added to help support the structure in the E-W direction where little framing is left.

After comparing the different stiffness of the vertical truss systems, it can be seen that much of the stiffness in the building is focused on the perimeter around the gym cavity space and at the reentrant corner on the Southeast side of the structure. This is done purposely to help resist the added force and torsion prone to these areas.

Last, the differences from the base fixity assumption are looked at. When the building is modeled as pinned, it is roughly 88% as stiff in the N-S direction and 79% as stiff in the E-W direction as the fixed assumption. Moment frames seem to lose the most amount of stiffness when a pinned assumption is made. These frames become about 40% as stiff, while braced frames do not lose more than 5% stiffness. Because it does not have bracing to help hold it together when a lateral load is applied, a moment frame is more likely to rotate around the pinned connection, causing the large loss of stiffness.

Note: After analysis is performed, it is found that torsional Irregularity exists. Because the structure is seismic design category C, static analysis is still allowed by code. However, there is a penalty. Accidental eccentricity from the center of mass must be increased, and, therefore, lateral forces are increased for member design.

LATERAL SYSTEM IN-DEPTH ANALYSIS

CENTER OF RIGIDITY

The center of rigidity (COR) was found for both the X and Y direction of each floor using the following two equations:

$$X_r = \frac{\sum k_{iy} X_i}{\sum k_{iy}} \qquad Y_r = \frac{\sum k_{ix} X_i}{\sum k_{ix}}$$

Multiplying the stiffness of each system in a certain direction by the distance from a determined origin and then dividing by the total stiffness of the systems in that direction will give the location of the center of rigidity. See **Appendix C** for hand calculations. After finding the X and Y component of the center of rigidity, the center of mass (COM) can be found and the difference between the two will give the structure's torsional eccentricity. When seismic forces are applied to the structure, they act on the COM and resistive forces act on the COR, causing a torsional moment around the COM. An accidental eccentricity of 5% building width times an amplification factor is also applied.

The center of rigidity is also found using ETABS (***See Table 16)***. The average COR for each floor is then tabulated. The average COR found in ETABS is less than 2 inches off the manually calculated value. Also, the difference of eccentricity from the COM in the pinned base assumption differs from the fixed base assumption by less than 2 inches as well. Therefore, even though the individual stiffness of each system changes when a pinned base assumption is used, the center of rigidity stays relatively the same.

Table 16: Center of Rigidity and Eccentricity

Center of Rigidity and Eccentricity								
	Hand Calculations		ETABS Calculations					
Floor	Center of Rigidity (COR)		Center of Rigidity (COR)		Center of Mass (COM)		Eccentricity	
Fixed	Xr (in)	Yr (in)	Xr (in)	Yr (in)	Xm (in)	Ym (in)	X (ft)	Y (ft)
6	1664.4	1352.1	1676.4	1346.4	1271.4	1321.3	33.7	2.1
5	1664.4	1352.1	1627.5	1342.0	1182.5	1320.9	37.1	1.8
4	1524.8	1352.1	1582.3	1342.4	1182.0	1318.9	33.4	2.0
3	1235.8	1332.0	1210.6	1320.4	1182.8	1299.2	2.3	1.8
2	1235.8	1270.3	1220.0	1302.6	1180.5	1342.5	3.3	-3.3
Pinned	Xr (in)	Yr (in)	Xr (in)	Yr (in)	Xm (in)	Ym (in)	X (ft)	Y (ft)
6	1651.0	1352.4	1676.1	1346.4	1271.4	1321.3	33.7	2.1
5	1651.0	1352.4	1582.1	1342.0	1182.5	1320.9	33.3	1.8
4	1617.1	1352.4	1627.4	1342.4	1182.0	1318.9	37.1	2.0
3	1228.5	1332.1	1210.5	1320.0	1182.8	1299.2	2.3	1.7
2	1228.5	1326.4	1213.6	1312.2	1180.5	1342.5	2.8	-2.5

LATERAL SYSTEM IN-DEPTH ANALYSIS

LOAD COMBINIATIONS

The following are the 7 basic load combinations prescribed by ASCE7-10 Chapter 2.3 for use in "combining factored loads using strength design":

1.) $1.4D$

2.) $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$

3.) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$

4.) $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$

5.) $1.2D + 1.0E + L + 0.2S$

6.) $0.9D + 1.0W$

7.) $0.9D + 1.0E$

(D=Dead, L=Live, L_r=Roof Live, S=Snow, R=Rain, W=Wind, E=Earthquake)

It was determined earlier in this report that roof live load was the controlling load over rain and snow load. Also, for every combination that includes wind or seismic loads, the loading in both the N-S and E-W directions must be considered. Therefore, 13 different load cases were tested in the ETABS model.

After comparing deflection at the roof level for each case, it is determined that seismic load combination 5 ($1.2D + 1.0E_x + L + 0.2S$ and $1.2D + 1.0E_y + L + 0.2S$) and 7 ($0.9D + 1.0E_x$ and $0.9D + 1.0E_y$) will control the design in both the X and Y directions. When comparing deflections found in the ETABS model, it can be seen that load combination 5a and 7a are both equal in controlling in the E-W direction and 5b and 7b are equal in controlling the N-S direction. This is because only seismic forces cause lateral deflection. However, because load case 5 includes both seismic and all gravity loads, it would have a greater impact on the gravity system as well. Because case 5 includes this combination loading, it is considered the controlling combination for design. ***Table 17*** on the next page lists the 13 different load combinations used, and shows roof point displacements for each case. Maximum deflections on both the X and Y direction are bolded along with the controlling load combination.

Table 17: Controlling Load Combination

Controlling Load Combination			
Load Combination	**Floor**	**X-Displacement (in)**	**Y-Displacement (in)**
Load Combo 1	Roof	0.0	0.0
Load Combo 2	Roof	0.0	0.0
Load Combo 3A (L)	Roof	0.0	0.0
Load Combo 3B ($W_{E\text{-}W}$)	Roof	0.946	-0.076
Load Combo 3C ($W_{N\text{-}S}$)	Roof	-0.095	1.358
Load Combo 4A ($W_{E\text{-}W}$)	Roof	1.894	-0.154
Load Combo 4B ($W_{N\text{-}S}$)	Roof	-0.189	2.714
Load Combo 5A ($E_{E\text{-}W}$)	**Roof**	**2.828**	-0.057
Load Combo 5B ($E_{N\text{-}S}$)	**Roof**	-0.170	**2.813**
Load Combo 6A ($W_{E\text{-}W}$)	Roof	1.889	-0.168
Load Combo 6B ($W_{N\text{-}S}$)	Roof	-0.194	2.700
Load Combo 7A ($E_{E\text{-}W}$)	Roof	**2.823**	-0.070
Load Combo 7B ($E_{N\text{-}S}$)	Roof	-0.175	**2.800**

LATERAL SYSTEM IN-DEPTH ANALYSIS

DRIFT ANALYSIS

Total building drift needs to be limited to prevent structural strength failure. Story drift analysis is also important when analyzing lateral system design. Structures need to be checked for serviceability limits for story to story displacements to prevent cracking and other damage to nonstructural components. The lateral system is checked for both seismic and wind loading to determine story drifts.

Using the ETABS model, drift values were found for each story under earthquake loads. Using ASCE7-10 Table 12.12-1, an allowable story drift of $0.015h_{sx}$ is determined for occupancy category III. Looking at ***Table 18*** and ***Table 19***, all story drifts under earthquake loads in each direction are within allowable limits.

Table 18: Allowable Seismic Drift: E-W Direction (Fixed Base Assumption)

Allowable Seismic Drift: E-W Direction							
Floor	Story Height	Story Drift	Allowable Story Drift		Total Drift	Allowable Total Drift	
Fixed	(ft)	(in)	Δ_{EQ} (in)=$0.015h_{sx}$	Acceptable	(in)	Δ_{EQ} (in)=$0.015h_{sx}$	Acceptable
6	72.3	0.00142	0.24495	Yes	0.0069	1.08495	Yes
5	56.0	0.00156	0.21000	Yes	0.0055	0.84000	Yes
4	42.0	0.00151	0.21000	Yes	0.0039	0.63000	Yes
3	28.0	0.00129	0.21000	Yes	0.0024	0.42000	Yes
2	14.0	0.00111	0.21000	Yes	0.0011	0.21000	Yes
Pinned	(ft)	(in)	Δ_{EQ} (in)=$0.015h_{sx}$	Acceptable	(in)	Δ_{EQ} (in)=$0.015h_{sx}$	Acceptable
6	72.3	0.00142	0.24495	Yes	0.0070	1.08495	Yes
5	56.0	0.00156	0.21000	Yes	0.0056	0.84000	Yes
4	42.0	0.00151	0.21000	Yes	0.0040	0.63000	Yes
3	28.0	0.00126	0.21000	Yes	0.0025	0.42000	Yes
2	14.0	0.00123	0.21000	Yes	0.0012	0.21000	Yes

Table 19: Allowable Seismic Drift: N-S Direction (Fixed Base Assumption)

Allowable Seismic Drift: N-S Direction							
Floor	Story Height	Story Drift	Allowable Story Drift		Total Drift	Allowable Total Drift	
Fixed	(ft)	(in)	Δ_{EQ} (in)=$0.015h_{sx}$	Acceptable	(in)	Δ_{EQ} (in)=$0.015h_{sx}$	Acceptable
6	72.3	0.00277	0.24495	Yes	0.0090	1.08495	Yes
5	56.0	0.00169	0.21000	Yes	0.0063	0.84000	Yes
4	42.0	0.00213	0.21000	Yes	0.0046	0.63000	Yes
3	28.0	0.00127	0.21000	Yes	0.0025	0.42000	Yes
2	14.0	0.00119	0.21000	Yes	0.0012	0.21000	Yes
Pinned	(ft)	(in)	Δ_{EQ} (in)=$0.015h_{sx}$	Acceptable	(in)	Δ_{EQ} (in)=$0.015h_{sx}$	Acceptable
6	72.3	0.00278	0.24495	Yes	0.0092	1.08495	Yes
5	56.0	0.00169	0.21000	Yes	0.0064	0.84000	Yes
4	42.0	0.00213	0.21000	Yes	0.0047	0.63000	Yes
3	28.0	0.00125	0.21000	Yes	0.0026	0.42000	Yes
2	14.0	0.00131	0.21000	Yes	0.0013	0.21000	Yes

Using the ETABS model, drift values were also found for each story under wind loading. Story drift values for wind loading are commonly limited to H/400. Looking at ***Table 20*** and ***Table 21***, all story drifts under wind loads in each direction are also within allowable limits.

Table 20: Allowable Seismic Drift: E-W Direction (Pinned Base Assumption)

Allowable Wind Drift: E-W Direction							
Floor	Story Height	Story Drift	Allowable Story Drift		Total Drift	Allowable Total Drift	
Fixed	(ft)	(in)	Δ_w(in)=h/400	Acceptable	(in)	Δ_w(in)=h/400	Acceptable
6	72.3	0.00088	0.04083	Yes	0.0047	0.18083	Yes
5	56.0	0.00113	0.03500	Yes	0.0038	0.14000	Yes
4	42.0	0.00120	0.03500	Yes	0.0027	0.10500	Yes
3	28.0	0.00078	0.03500	Yes	0.0015	0.07000	Yes
2	14.0	0.00074	0.03500	Yes	0.0007	0.03500	Yes
Pinned	(ft)	(in)	Δ_w(in)=h/400	Acceptable	(in)	Δ_w(in)=h/400	Acceptable
6	72.3	0.00088	0.04083	Yes	0.0048	0.18083	Yes
5	56.0	0.00113	0.03500	Yes	0.0039	0.14000	Yes
4	42.0	0.00120	0.03500	Yes	0.0028	0.10500	Yes
3	28.0	0.00076	0.03500	Yes	0.0016	0.07000	Yes
2	14.0	0.00082	0.03500	Yes	0.0008	0.03500	Yes

Table 21: Allowable Seismic Drift: N-S Direction (Pinned Base Assumption)

Allowable Wind Drift: N-S Direction							
Floor	Story Height	Story Drift	Allowable Story Drift		Total Drift	Allowable Total Drift	
Fixed	(ft)	(in)	Δ_w(in)=h/400	Acceptable	(in)	Δ_w(in)=h/400	Acceptable
6	72.3	0.00197	0.04083	Yes	0.0079	0.18083	Yes
5	56.0	0.00124	0.03500	Yes	0.0059	0.14000	Yes
4	42.0	0.00170	0.03500	Yes	0.0047	0.10500	Yes
3	28.0	0.00143	0.03500	Yes	0.0030	0.07000	Yes
2	14.0	0.00152	0.03500	Yes	0.0015	0.03500	Yes
Pinned	(ft)	(in)	Δ_w(in)=h/400	Acceptable	(in)	Δ_w(in)=h/400	Acceptable
6	72.3	0.00198	0.04083	Yes	0.0080	0.18083	Yes
5	56.0	0.00124	0.03500	Yes	0.0060	0.14000	Yes
4	42.0	0.00171	0.03500	Yes	0.0048	0.10500	Yes
3	28.0	0.00141	0.03500	Yes	0.0026	0.42000	Yes
2	14.0	0.00168	0.03500	Yes	0.0013	0.21000	Yes

LATERAL SYSTEM IN-DEPTH ANALYSIS

OVERTURNING AND IMPACT ON FOUNDATIONS

When a structure is loaded with lateral seismic and wind loads, an overturning moment occurs at the base of the structure. It is necessary to design the structure so that the foundation can support the uplifting force caused by this moment and prevent the building from falling over.

Using ETABS and the controlling load combination, the uplift force, *FZ,* is found for the base supports in the structure. Referring to the foundation plan (***Figure 5***), pile caps that support the columns of the lateral systems are identified.

Table 21: Base Reactions and Foundation Capacity (Fixed Base Assumption)

Base Reactions and Foundation Capacity (Fixed Base Assumption)									
N-S Loading Direction					E-W Loading Direction				
Point	Fz	Pile Cap	Axial Capacity	Adequate?	Point	Fz	Pile Cap	Axial Capacity	Adequate?
#	(k)		(k)	Y/N	#	(k)		(k)	Y/N
17	130.8	-	-	-	**17**	**-218.9**	**300DP2**	**1200.000**	**Y**
27	222.2	-	-	-	27	571.9	-	-	-
28	**-301.0**	**300DP2**	**1200.000**	**Y**	28	89.9	-	-	-
29	455.5			-	29	64.6	-	-	-
30	**-168.5**	**300MP1A**	**600.000**	**Y**	**30**	**-17.0**	**300MP1A**	**600.000**	**Y**
31	333.9	-	-	-	**31**	**-625.0**	**300MP2**	**1200.000**	**Y**
44	154.4	-	-	-	44	162.8	-	-	-
45	43.4	-	-	-	45	842.4	-	-	-
46	**-613.2**	**300DP2**	**1200.000**	**Y**	46	189.9	-	-	-
47	196.0	-	-	-	47	212.5	-	-	-
48	931.8	-	-	-	48	112.2	-	-	-
49	181.1	-	-	-	**49**	**-239.2**	**300DP2**	**1200.000**	**Y**
50	**-9.1**	**200DP2**	**800.000**	**Y**	50	411.1	-		-
52	**-36.8**	**300MP2C**	**1200.000**	**Y**	**52**	**-132.1**	**300MP2C**	**1200.000**	**Y**
53	53.2	-	-	-	53	52.2	-	-	-
54	97.9	-	-	-	54	194.3	-	-	-
55	**-134.7**	**300MP2A**	**1200.000**	**Y**	**55**	**-26.5**	**300MP2A**	**1200.000**	**Y**
56	180.5	-	-	-	56	64.1	-	-	-
57	15.9	-	-	-	57	24.1	-	-	-
60	0.0	-	-	-	60	0.0	-	-	-
61	0.0	-	-	-	61	0.0	-	-	-
62	0.0	-	-	-	62	0.0	-	-	-
63	0.0	-	-	-	63	0.0	-	-	-
79	0.1	-	-	-	79	3.9	-	-	-
80	15.2	-	-	-	80	14.0	-	-	-
81	10.7	-	-	-	81	8.2	-	-	-
82	0.9	-	-	-	82	3.8	-	-	-
83	14.8	-	-	-	83	13.9	-	-	-
84	10.7	-	-	-	84	8.7	-	-	-
85	1.9	-	-	-	85	3.5	-	-	-
86	14.5	-	-	-	86	14.1	-	-	-
87	10.5	-	-	-	87	9.4	-	-	-

Table 21 and ***Table 22*** show the uplift forces at each lateral system column base for both fixed and pinned base assumptions. Each table then lists the pile cap that is supporting that column and its axial force capacity. Pile caps are called out by strength in tons per pile and either containing 1 or 2 micro/rock piles. Each of these tables shows that the foundation is more than adequate in supporting the uplift force at all points. Foundation design was clearly for supporting gravity loads and adding support on a weak soil base. Rock pile caissons are drilled to at least a depth of 40 feet until good rocky soil is found to help support the structure.

Table 22: Base Reactions and Foundation Capacity (Pinned Base Assumption)

Base Reactions and Foundation Capacity (Pinned Base Assumption)									
N-S Loading Direction					E-W Loading Direction				
Point	Fz	Pile Cap	Axial Capacity	Adequate?	Point	Fz	Pile Cap	Axial Capacity	Adequate?
#	(k)		(k)	Y/N	#	(k)		(k)	Y/N
17	129.3	-	-	-	**17**	**-220.9**	**300DP2**	**1200.000**	**Y**
27	223.7	-	-	-	27	573.8	-	-	-
28	**-308.8**	**300DP2**	**1200.000**	**Y**	28	92.1	-	-	-
29	463.3	-	-	-	29	62.5	-	-	-
30	**-171.9**	**300MP1A**	**600.000**	**Y**	**30**	**-18.5**	**300MP1A**	**600.000**	**Y**
31	342.5	-	-	-	**31**	**-642.8**	**300MP2**	**1200.000**	**Y**
44	153.2	-	-	-	44	168.4	-	-	-
45	39.4	-	-	-	45	856.1	-	-	-
46	**-629.5**	**300DP2**	**1200.000**	**Y**	46	191.2	-	-	-
47	197.1	-	-	-	47	212.7	-	-	-
48	947.1	-	-	-	48	110.8	-	-	-
49	182.6	-	-	-	**49**	**-242.7**	**300DP2**	**1200.000**	**Y**
50	**-10.7**	**200DP2**	**800.000**	**Y**	50	414.7	-	-	-
52	**-39.7**	**300MP2C**	**1200.000**	**Y**	**52**	**-132.2**	**300MP2C**	**1200.000**	**Y**
53	53.2	-	-	-	53	52.2	-	-	-
54	100.8	-	-	-	54	194.4	-	-	-
55	**-139.0**	**300MP2A**	**1200.000**	**Y**	**55**	**-29.4**	**300MP2A**	**1200.000**	**Y**
56	185.7	-	-	-	56	67.8	-	-	-
57	15.1	-	-	-	57	23.4	-	-	-
60	0.0	-	-	-	60	0.0	-	-	-
61	0.0	-	-	-	61	0.0	-	-	-
62	0.0	-	-	-	62	0.0	-	-	-
63	0.0	-	-	-	63	0.0	-	-	-
79	1.5	-	-	-	79	3.7	-	-	-
80	14.8	-	-	-	80	14.0	-	-	-
81	9.7	-	-	-	81	8.3	-	-	-
82	2.1	-	-	-	82	3.8	-	-	-
83	14.5	-	-	-	83	14.0	-	-	-
84	9.9	-	-	-	84	8.7	-	-	-
85	2.8	-	-	-	85	3.7	-	-	-
86	14.3	-	-	-	86	14.0	-	-	-
87	9.8	-	-	-	87	9.2	-	-	-

LATERAL SYSTEM IN-DEPTH ANALYSIS

LATERAL MEMBER SPOT CHECKS

Lateral member spot checks were performed on members of Truss 2 supporting the X direction and members of Truss 6 supporting the Y direction. The controlling load combinations (5a and 5 b) were applied to the structure in ETABS, and forces were found in select members. Axial, shear and moment forces were found and multiplied by an amplification factor to account for torsional irregularity, then the 14th edition AISC Steel Manual was used to determine if sizes were adequate to carry the loads applied.

Figure 30 and ***Figure 31*** show which members are checked in each truss. Column sizes are checked by using AISC Table 6-1 to find combined axial and bending moment limitations. Beams are checked using Table 3-2 to find bending moment capacity. Lastly, Table 4-1 is used to check axial compression of braced framing.

After calculations are done, it is determined that all members are adequate in carrying the load applied to them. See **Appendix D** for all hand calculations for member spot checks.

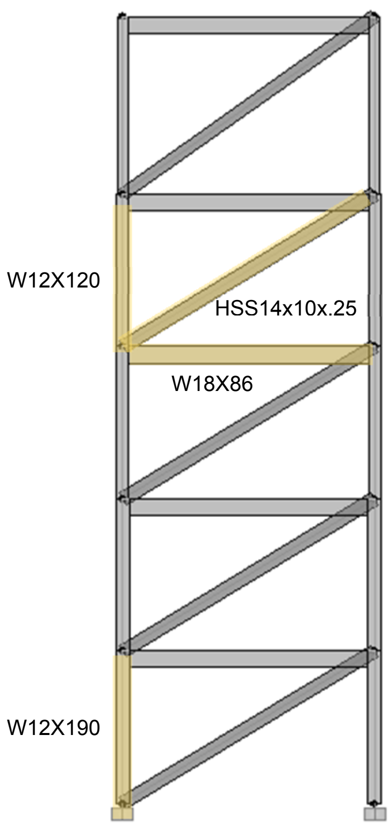


Figure 30: Truss 2 Elevation (X-Direction)
ETABS Model

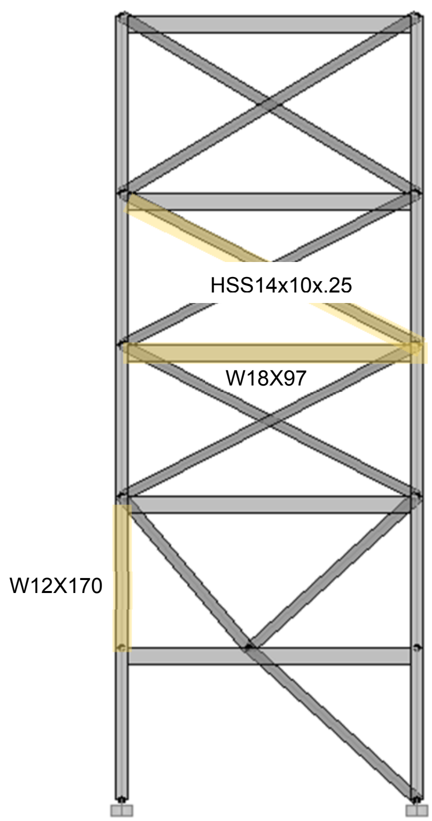


Figure 31: Truss 6 Elevation (Y-Direction)
ETABS Model

EVALUATION AND SUMMARY

Technical Report III builds on the information presented in Technical Report I, as it explores the lateral system of Hunter's Point South in an in-depth strength and serviceability analysis. Lateral system information presented in Technical Report I has been updated and included in this report.

A 2D STAAD model and 3D ETABS structural model are designed for a better in-depth analysis. The ETABS model includes only lateral system members with rigid diaphragm floor areas to carry the floor mass and create a rigid story. Bracing is modeled with a moment release so only axial forces act on it. Columns are modeled as both fixed and pinned base, and analyzed to determine differences. This report shows that lateral loads/design does not change much with either base assumption.

Per ASCE7-10, seven basic load combinations are considered to determine the controlling load case for this structure. To consider both X and Y directions for wind and earthquake loads, 13 total load combinations are input into the ETABS model. After running the computer analysis, it is determined that load case 5, which includes earthquake loads and gravity loads, is the controlling case in E-W and N-S directions.

The drift analysis included in this report focuses on total story displacement and single story drifts. Total displacement needs to be limited to prevent structural failure, and story drift needs to be limited to prevent damage to walls and other nonstructural components. Story drifts due to earthquake loading are found in ETABS, and checked against an allowable drift of $\Delta_{seismic}=0.015h_{sx}$. Wind loads are also found from the ETABS model and checked against the allowable wind drift value of H/400. Analysis shows that all story drifts and total displacements are within limits for this design.

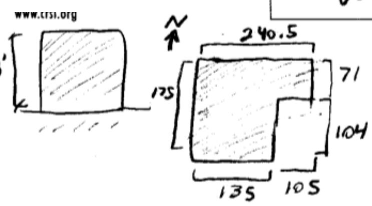
A foundation analysis is performed to analyze the effect of overturning moment forces on the pile caps, and to see if foundation design is sufficient to prevent uplift and building topple. Using the ETABS model, the uplift forces, Fz, are found at base points. Then, referencing the foundation plan, pile caps under lateral systems are identified and checked to see if they have enough axial force strength. Analysis shows that the foundation is capable of supporting the uplift forces and is sufficiently designed.

To check lateral member design is sufficient, manual spot checks are performed at different floors for Truss 2 in the E-W direction and Truss 6 in the N-S direction. Loading the ETABS model with the controlling load combination, forces in columns, beams, and cross bracing are found. Using the AISC Steel Manual (14^{th} Ed.) member sizes were checked for sufficient strength. Analysis shows each member is adequately designed.

APPENDIX A

WIND ANALYSIS

Project: Hunters Point South		Sheet No. of
Made By: Michael Payne	Checked By: Tech III	Date
Subject: Wind Loads (per ASCE 7-10)		Project No.



Risk Category : $\boxed{III}$
Wind speed : Fig 26-5-1B $V = 130$ mph
Directionality Factor : table 26-6-1 $K_d = 0.85$
↳ MWFRS
↳ components/cladding

Surface Roughness : $\boxed{B}$
topographic Factor : $\boxed{K_{zt} = 1.0}$
Exposure : $\boxed{C}$
↳ on upper Bay

Internal pressure Coefficient : table 26-11-1 $\boxed{GC_{pi} = \pm 0.18}$
↳ enclosed : Protected glazing per 26.10.3.1 & Design

Natural Frequency $N_a = 75/h = 75/72.3 \simeq 1.0$ Hz → Gust Factor = 0.85

Velocity Pressure Exposure Coefficients K_h or K_z : $\boxed{1.179}$

$q_h = 0.00256 K_z K_{zt} K_d V^2$ Assume $q_z = q_h$
$= 0.00256(1.179)(1.0)(0.85)(130)^2$
$= \boxed{43.362 \text{ lb/Ft}^2}$

$\boxed{P = qGC_p - q_i(GC_{pi}) \text{ lb/Ft}^2}$

$\boxed{N\text{-}S}$
$q = 43.362$
$L/B = 0.73$ $G = 0.85$
$C_p =$ → windward : 0.8 ; → -0.5 ; → -0.7
$GC_{pi} = \pm 0.18$

$\boxed{E\text{-}W}$
$q = 41.302$
$L/B = 1.35$ $G = 0.85$
$C_p =$ → 0.8 ; → -0.425 ; → -0.7
$GC_{pi} = \pm 0.18$

Roof :
0 to h/2 → 0.9
h/2 to h → 0.9
h to 2h → 0.5
2h or greater → 0.3

Parapet $P_p = q_p(GC_{pn})$ lb/Ft²
$q_p = 0.00256(1.232)(1.0)(0.85)(130)^2 = 45.3$
mean parapet height : $72.3 + 15 = 87.3'$
$P_{p\,windward} = (45.3)(1.5) = 67.95$
$P_{p\,leeward} = -45.3(1.0) = -45.3$ (See table)

Figure 32: Wind Load Hand Calculations

APPENDIX B

SEISMIC ANALYSIS

Project: Hunters Point South	Sheet No. of
Made By: Michael Payne / Checked By: Tech III	Date
Subject: Seismic Loads (Per ASCE 7-10)	Project No.

Fig 22-1 → $S_S = 0.25g$
Fig 22-2 → $S_1 = 0.073g < 0.75$ ✓
Fig 22-7 → MCE_G PGA = 15%g

Fig 22-12 → $T_L = 6$ sec
Fig 22-17 → $CR_S = 0.9$
Fig 22-18 → $CR_1 = 0.9$

Table 12.2.1 → R=3
Table 20.3.1 – Site Class E (Soft clay soil)
$I_E = 1.25$

table 11.4.1 $F_a = 2.5$
table 11.4.2 $F_v = 3.5$

$S_{MS} = F_a S_S = (2.5)(0.25) = 0.625$
$S_{M1} = F_v S_1 = (3.5)(0.073) = 0.2555$
$S_{DS} = 2/3\, S_{MS} = 2/3(0.625) = 0.4167$
$S_{D1} = 2/3\, S_{M1} = 2/3(0.2555) = 0.1703$

$T_0 = 0.2 \frac{S_{D1}}{S_{DS}} = 0.2\left(\frac{.1703}{.4167}\right) = 0.0817$

$T_S = \frac{S_{D1}}{S_{DS}} = 0.4087$

Seismic Design Category : C
(table 11.6-1 & 11.6-2)
Use Equivelant lateral Force Analysis Method (ELFA)

$\boxed{\text{ELFA}}$
table 12.8.2

$C_S = \min \begin{cases} S_{DS}/(R/I_e) = 0.4167/(3/1.25) = 0.1736 \\ S_{D1}/(T(R/I_e)) = 0.1703/.7936(3/1.25) = 0.0894 \\ S_{D1}T_L/(T^2(R/I_e)) = .1703(6)/.7936^2(3/1.25) = 0.6759 \end{cases}$

$T_a = C_t h_n^x = (0.02)(72.33)^{0.75} = 0.4960425$
$T = C_u T_a = (1.6)(0.49604) = 0.7936674$

Floor weights

Floor	
R	3047.24 lb
5	2959.28 lb
4	2631.63 lb
3	3895.83 lb
2	2284.97 lb
	14,820 lb

$V_{base} = C_S W = (0.0894055 7)(14820)$
$V = 1325^k$

$F_x = C_{vx} V$ (see table)

Figure 33: Seismic Load Hand Calculations

Roof																		
		weight/ft	length	weight			weight/ft	length	weight				Area	DL	LL	SL	Tot	weight
Column					Beam					Floor								
10	X	49	17	833	24	X	76	24	1824	31.1	X	232.45	7229.195	85	45	22	85	614481.6
10	X	54	17	918	24	X	76	24	1824	39.25	X	198.45	7789.163	85				662078.8
12	X	96	17	1632	24	X	68	21.3	1448.4	101.75	X	104.66	10649.16	85				905178.2
10	X	54	17	918	24	X	68	23.08333	1569.667	TOTAL								2181739
10	X	54	17	918	24	X	68	24.39583	1658.917									2181.739
12	X	96	17	1632	24	X	68	19.10417	1299.083									
10	X	68	14	952	24	X	68	26.3125	1789.25									
10	X	54	14	756	24	X	68	26	1768	PERIMETER								
10	X	54	14	756	24	X	68	22	1496	19	X	592	11248	20				224960
10	X	54	17	918	30	X	99	30.58333	3027.75	11	X	172	1892	20				37840
12	X	53	17	901	14	X	22	12	264		X		0					262800
12	X	79	7	553	12	X	26	12	312									262.8
10	X	54	17	918	12	X	26	10.65	276.9									
12	X	40	17	680	14	X	22	10.19444	224.2778									
12	X	79	7	553	14	X	22	12	264									
12	X	79	7	553	12	X	26	12	312									
12	X	79	7	553	12	X	26	10.65	276.9			TOTAL	2944.57					
10	X	33	7	231	14	X	22	10.19444	224.2778									
10	X	33	7	231	12	X	26	11.54165	300.0829									
12	X	40	7	280	12	X	26	8.133333	211.4667									
12	X	40	7	280	14	X	22	11.72917	258.0417									
10	X	33	7	231	24	X	76	24	1824									
12	X	50	17	850	21	X	101	24	2424									
10	X	33	7	231	14	X	233	21.3	4962.9									
10	X	33	7	231	16	X	36	23.08333	831									
10	X	33	7	231	16	X	36	24.39583	878.25									
10	X	33	7	231	16	X	36	19.10417	687.75									
12	X	79	7	553	21	X	50	26.3125	1315.625									
10	X	33	7	231	21	X	50	26	1300									
12	X	50	7	350	21	X	50	22	1100									
12	X	79	7	553	24	X	62	30.58333	1896.167									
12	X	79	7	553	4	X	13	8	104									
12	X	79	7	553	4	X	13	8.5	110.5									
12	X	79	7	553	4	X	13	9	117									
14	X	53	15	795	4	X	13	10	130									
10	X	33	7	231	4	X	13	10.5	136.5									
12	X	40	7	280	4	X	13	11	143									
12	X	79	7	553	4	X	13	12	156									
10	X	33	7	231	4	X	13	12.5	162.5									
12	X	40	7	280	4	X	13	13	169									
12	X	79	7	553	4	X	13	14	182									
12	X	79	7	553	4	X	13	14.5	188.5									
12	X	79	7	553	4	X	13	15	195									
10	X	33	7	231	4	X	13	16	208									
14	X	61	7	427	4	X	13	16.5	214.5									
14	X	74	7	518	4	X	13	17	221									
HSS			7	0	4	X	13	18	234									
HSS			7	0	4	X	13	18.5	240.5									
14	X	109	14.25	1553.25	4	X	13	19	247									
14	X	193	13.5	2605.5	4	X	13	20	260									
14	X	233	12.75	2970.75	4	X	13	20.5	266.5									
14	X	283	12	3396	4	X	13	21	273									
14	X	342	11.25	3847.5	4	X	13	22	286									
14	X	342	10.75	3676.5	4	X	13	22.5	292.5									
10	X	49	7	343	4	X	13	24	312									
10	X	33	7	231	12	X	55	20	1100									
10	X	49	7	343	12	X	35	23.5	822.5									
10	X	33	14	462	12	X	35	23.5	822.5									
10	X	33	14	462	12	X	35	23.5	822.5									
10	X	33	14	462	12	X	35	20	700									
TOTAL				46874.5	12	X	35	23.5	822.5									
				46.8745	12	X	35	23.5	822.5									
					12	X	35	22.75	796.25									
					12	X	35	22.75	796.25									
					12	X	35	22.75	796.25									
					12	X	35	23.5	822.5									
					12	X	35	23.5	822.5									
					12	X	35	23.5	822.5									
					12	X	35	23.5	822.5									
					21		101	20	2020									
					21	X	44	23.5	1034									
					21	X	44	23.5	1034									
					21	X	44	23.5	1034									
					21	X	44	23.5	1034									
					21	X	44	23.5	1034									
					12	X	35	23.5	822.5									
					21	X	44	22.75	1001									
					18	X	76	22.75	1729									
					21	X	73	20	1460									
					14	X	22	30	660									
					14	X	53	30	1590									
					14	X	22	30	660									
					14	X	82	30	2460									
					16	X	31	30	930									
					14	X	90	30	2700									
					16	X	40	30	1200									
					14	X	109	28	3052									
					14	X	22	26	572									
					14	X	90	24	2160									
					14	X	22	20	440									
					14	X	82	15	1230									

Figure 34: Part of Story Weight Calculations using Microsoft Excel

21	X	57	40	2280
24	X	68	10	680
24	X	62	22	1364
30	X	99	30.58333	3027.75
16	X	40	11	440
16	X	40	25	1000
24	X	55	23	1265
16	X	40	18	720
30	X	99	46	4554
30	X	99	45	4455
30	X	99	44	4356
16	X	36	21	756
16	X	36	20	720
16	X	36	20	720
16	X	36	30	1080
16	X	31	30	930
16	X	31	30	930
16	X	31	32	992
16	X	31	32	992
16	X	36	32	1152
16	X	36	32	1152
16	X	36	32	1152
16	X	36	32	1152
21	X	50	40	2000
21	X	50	40	2000
14	X	22	20	440
14	X	22	20	440
14	X	22	20	440
14	X	22	20	440
24	X	68	20	1360
24	X	68	20	1360
24	X	68	23.5	1598
24	X	68	23.5	1598
24	X	117	45	5265
24	X	162	45	7290
24	X	117	45	5265
24	X	162	45	7290
24	X	117	42	4914
24	X	117	40	4680
14	X	22	20	440
21	X	50	23.5	1175
14	X	22	23.5	517
14	X	22	23.5	517
14	X	22	23.5	517
14	X	22	23.5	517
14	X	22	23.5	517
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
16	X	36	11.5	414
24	X	62	27	1674
12	X	50	35	1750
24	X	62	35	2170
21	X	57	35	1995
24	X	62	35	2170
24	X	55	25	1375
24	X	55	20	1100
24	X	55	28	1540
24	X	55	12	660
24	X	55	22	1210
24	X	55	35	1925
24	X	68	25	1700
14	X	22	12	264
14	X	68	12	816
24	X	68	15	1020
24	X	68	20	1360
24	X	76	20	1520
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190

	Weight
Misc	
AHU1	37200
AHU2	39600
AHU3	39600
AHU4	34900
AHU5	21400
AHU6	20700
	193400
	193.4

12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
12	X	19	10	190
30	X	99	31	3069
16	X	40	31	1240
16	X	40	31	1240
16	X	40	31	1240
16	X	31	31	961
16	X	31	31	961
16	X	40	31	1240
16	X	40	31	1240
16	X	40	31	1240
16	X	40	31	1240
16	X	40	31	1240
16	X	40	31	1240
16	X	40	31	1240
16	X	26	31	806
16	X	26	31	806
16	X	26	31	806
16	X	26	31	806
16	X	26	31	806
16	X	31	31	961
16	X	31	31	961
16	X	31	31	961
16	X	31	31	961
16	X	31	31	961
30	X	99	31	3069
40	X	167	40	6680
18	X	35	28	980
21	X	50	40	2000
21	X	50	40	2000
21	X	50	40	2000
21	X	50	40	2000
21	X	50	40	2000
21	X	57	40	2280
21	X	57	40	2280
21	X	57	40	2280
21	X	57	40	2280
18	X	22	12	264
14	X	22	10	220
14	X	22	10	220
14	X	22	10	220
14	X	22	10	220
16	X	40	15	600
16	X	40	15	600
	X			0
	X			0
	X			0
	X			0
	X			0
	X			0
	X			0
	X			0
	X			0
TOTAL				259757.2
				259.7572

Figure 35: Part of Story Weight Calculations using Microsoft Excel

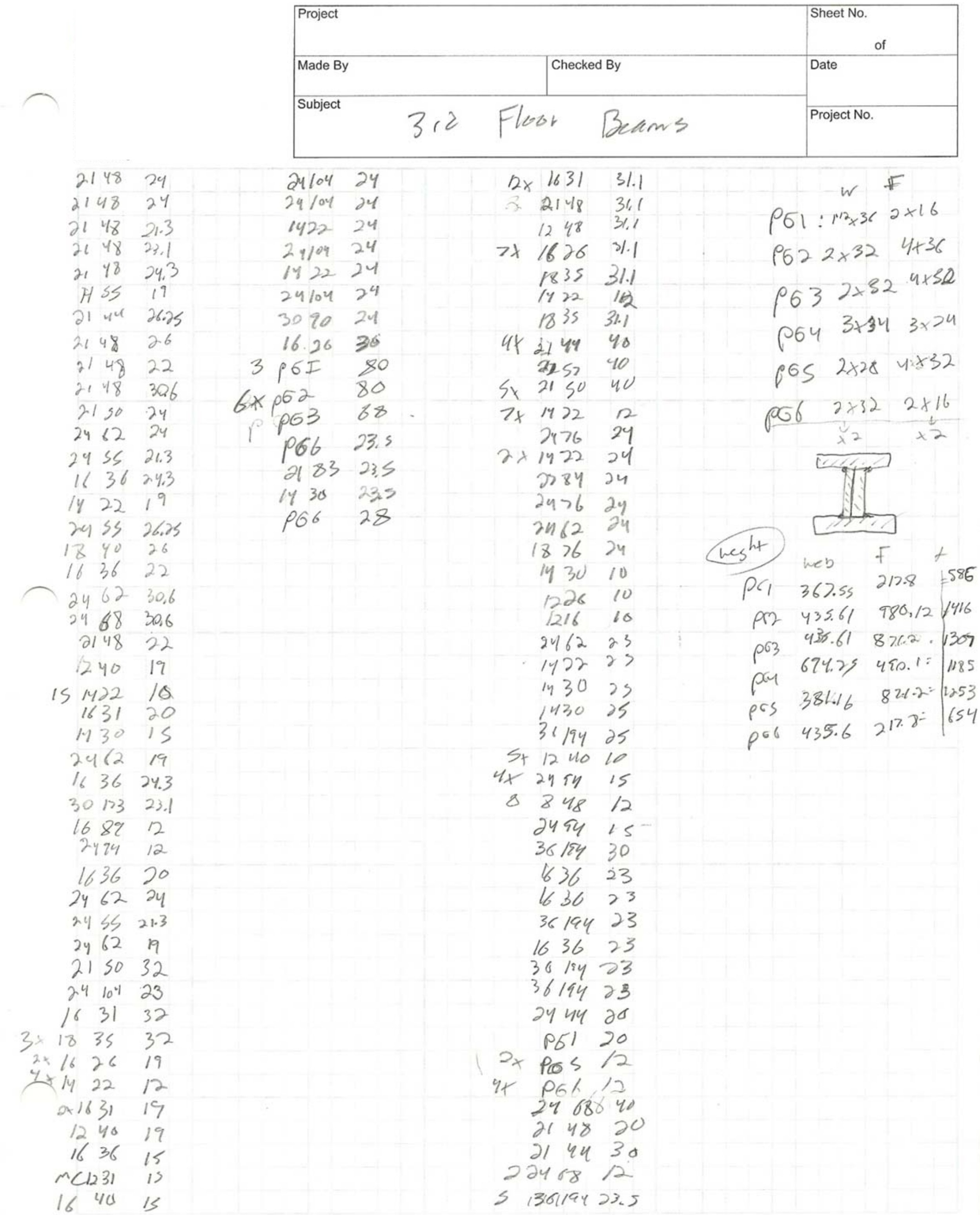


Figure 36: Part of Story Weight Hand Calculations

APPENDIX C

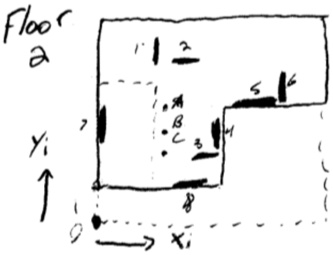
CENTER OF RIGIDITY MANUAL CHECK

Project	Sheet No. 1 of 1
Made By Michael Payne — [redacted]	Date
Subject: Center of rigidity check	Project No.

Fixed Assumption

$$X_r = \frac{\Sigma R_i x_i}{\Sigma R_i} \qquad Y_R = \frac{\Sigma R_i y_i}{\Sigma R_i}$$

Floor 2



$$X_r = [(100)(576) + (353)(1637.5) + (137)(2529.3) + (205)(0)] \div (100+353+137+205)$$
$$= \boxed{1235.8 \text{ in}}$$

$$Y_r = [(90)(1816) + (518)(1345) + (53)(629.75) + (373)(0) + (60)(1362) + (60)(1056.3) + (60)(493.25)] \div (90+518+53+373+60\times 3)$$
$$\boxed{1270.3 \text{ in}}$$

Pinned Assumption. Other Floors Done in Excell

Floor 2

$$X_r = [(107)(576) + (353)(1637.5) + (136)(2529.3)] \div (107+353+136+205)$$
$$= \boxed{1228.5 \text{ in}}$$

$$Y_r = [(89)(1816) + (515)(1345) + (52)(629.75) + (377)(0) + (16)(1362) + (16)(1056.3) + (16)(493.25)] \div (89+515+52+377+16+3)$$
$$= \boxed{1326.4 \text{ in}}$$

Figure 37: COR Hand Calculations

APPENDIX D

LATERAL FORCE MEMBER SPOT CHECK

Project: Hunters Point South	Sheet No. 1 of 2
Made By: Michael Payne / Checked By: Tech Report III	Date
Subject: Lateral Force Member Spot check	Project No.

Loading in X-Direction

1.) column member @ Truss 2 : Floor 1

W12x190 column
$P_u = 216.2^k$
$M_u = 5353.7^{k\text{-}in} = 446.2^{k\text{-}Ft}$

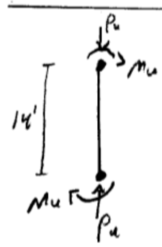
Use table 6-1, AISC: Combined Axial & Bending

$KL = (14)\frac{r_x}{r_y} = (14)(1.79) = 25^{Ft}$

$P \times 10^{-3} = 0.742 \quad b_x \times 10^{-3} = 0.822$

$$\frac{216.2(.742)}{1000} + \frac{446.2(.822)}{1000} = 0.53 < 1.0 \quad \text{ok}$$

2.) column member @ Truss 2 : Floor 4



W12x120 column
$P_u = 29.6^k$
$M_u = 1354.7^{k\text{-}in} = 112.9^{k\text{-}Ft}$

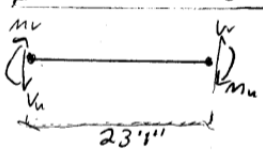
From table 6-1 : Combined Axial/Bnd.

$KL = (14)(1.76) = 24.65$

$P \times 10^{-3} = 1.20 \quad b_x \times 10^{-3} = 1.44$

$$\frac{29.6(1.2)}{1000} + \frac{112.9(1.44)}{1000} = 0.20 < 1.0 \quad \text{ok}$$

3.) Beam member @ Truss 2 : Floor 3



W18x86 beam
$V_u = 88^k$
$M_u = 6080.1^{k\text{-}in} = 506.7^{k\text{-}Ft}$

Use table 3-2, AISC W-shapes

$\phi M_p = 698 \quad L_b = 23'1"$
$BF = 13.6 \quad L_p = 9.29$
$\phi V_n = 265$

$M_n = M_p - BF(L_b - L_p)$
$= 698 - 13.6(23.08 - 9.29)$
$= 510^{k\text{-}Ft} > 506.7^{k\text{-}Ft}$ ok

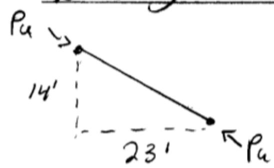
$V_n = 265^k \gg 88^k$ ok

Figure 38: Lateral Member Spot Check

Project: Hunters Point South		Sheet No. 2 of 2
Made By: Michael Payne	Checked By:	Date:
Subject: Lateral Load Member Spot Check		Project No.

Loading in x-direction (cont.)

4.) Bracing Member @ Truss 2 : Floor 4

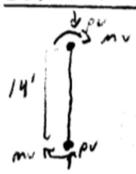


HSST14x10x0.375
$P_u = 108.6^k$
$L_b = 27$ feet

⇒ use table 4-1 in AISC
Strength in Axial compression
$\phi P_n = 276 > 108.6$ ok

Loading in Y-Direction

5.) column Member @ Truss 6 : Floor 2

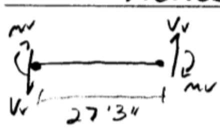


W12X170 column
$P_u = 242.6^k$
$M_u = 4288.9^{k\text{-in}}$
$= 357.5^{k\text{-ft}}$

⇒ table 6-1, AISC
$KL = (14)(1.78) = 25$
$P_{x \times 10^{-3}} = 0.84$ $\quad b_{x \times 10^{-3}} = 0.938$

$\frac{242.6(.84)}{1000} + \frac{357.5(.938)}{1000} = 0.54 < 1.0$ ok

6.) Beam member @ Truss 6 : floor 3

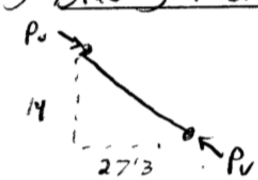


W18 X97 beam
$V_u = 13.8^k$
$M_u = 2355.9^{k\text{-in}}$
$= 196$ k-ft

⇒ use table 3-2, AISC
$\phi M_p = 791$ $\quad L_b = 27'3$
$BF = 14.1$ $\quad L_p = 9.36$
$\phi V_n = 299$

$M_n = M_p - BF(L_b - L_p)$
$= 791 - 14.1(27.25 - 9.36) = 539 > 196$ ok
$\phi V_n = 299 >> 13.8$ ok

7.) Bracing member @ Truss 6 : Floor 4



HSST12X8X0.375
$P_u = 62.7^k$
$L_b = 30.6'$

⇒ use table 4-1 AISC
$\phi P_n = 250^k > 62.7^k$ ok

Figure 39: Lateral Member Spot Check

APPENDIX E

REFERENCES

The following is a list of reference materials used in this thesis project for research, analysis, and design aids.

1. ASCE7-10 (2010)
2. AISC Steel Specification Manual 14th Edition (2010)
3. "Seismic Design of Building Structures" Ninth Edition. Lindeburg, Michael. Professional Publications Inc. 2008.
4. "Steel Plate Weight Calculator". Portland Bolt & Manufacturing Company. 2011 http://www.portlandbolt.com/steel-plate-weight.html
5. ETABS Integrated Building Design Software. Technical Knowledgebase. CSI. 2008

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