TECHNICAL REPORT III

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PENN COLLEGE OF TECHNOLOGY

Lateral System Analysis

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Table of Contents:

Executive Summary……………………………………………………………………3

Building Introduction………………………………………………………………4

Existing System Structural Overview………………………………………………5

- Foundation……………………………………………………………………5
- Floor Systems…………………………………………………………………8
- Framing System………………………………………………………………9
- Lateral System…………………………………………………………………11
- Roof System……………………………………………………………………15
- Design Codes…………………………………………………………………16
- Materials Used…………………………………………………………………17

Gravity Loads………………………………………………………………………19

- Dead and Live Loads…………………………………………………………19
- Snow Loads……………………………………………………………………19

Lateral Loads………………………………………………………………………21

- Wind Loads……………………………………………………………………21
- Seismic Loads…………………………………………………………………24

Lateral System Analysis……………………………………………………………25

- Computer Model………………………………………………………………25
- Load cases………………………………………………………………………27
- Drift……………………………………………………………………………28
- Torsion…………………………………………………………………………30
- Spot Checks……………………………………………………………………31

Conclusion…………………………………………………………………………32

Appendices…………………………………………………………………………33

- Appendix A: Wind Load Calculations ………………………………………34
- Appendix B: Seismic Load Calculations……………………………………40
- Appendix C: Spot Checks……………………………………………………41
- Appendix D: Floor Plans………………………………………………………45

EXECUTIVE SUMMARY

The purpose of this report was to complete an in-depth analysis on the lateral system of the dauphin Hall (DH). The DH, located in Williamsport, Pennsylvania, is 70 feet high, 196 feet wide and 362 feet long. This 4 story student housing, completed in August, 2010, has a gravity system consisting of lightweight concrete on metal deck and Concrete Masonry Units (CMU). The metal deck rests on k-series steel joists. The lateral resisting system of the DH consists of moment connections in both the East-West and North-South direction.

In this technical report, the lateral system of the DH was analyzed under various conditions. This was accomplished through a combination of methods including hand calculations and a 3D ETABs computer model. Some assumptions were made to simplify these calculations. The building was made more rectilinear with "pinned" support conditions in the analysis model.

Both wind and seismic loads were calculated for the building using the Main Wind Force Resisting procedure and the Equivalent Lateral Force procedure given in chapter 6 and 12 of the ASCE 7-10 respectively.

The report also included a study of the combinations of loads that might control design in the structure. It was found that wind case 4 from ASCE 7-10 would be the controlling wind load on the structure. Torsional effects were analyzed and it was found to have a small contribution to the building.

Lastly, a spot check was undertaken to insure that drift met industry standards. It was found that critical members were appropriately sized, overturning was restricted, and drift did not control.

BUILDING INTRODUCTION

The Pennsylvania College of Technology is located in the 200 block of Rose Street in Williamsport, PA. Dauphin Hall, the newest dormitory on campus, is constructed in August 2010 by Murray Associates Architects, P.C in collaboration with IMC as the general contractor; Woodburn & Associates, INC as the food service designer; Whitney, Bailey, Cox & Magnani, LLC as the civil engineering firm; and Gatter & Diehl, INC as the MEP firm. This new structure costs approximately $ 26,000,000 and was delivered using the design-bid-build project delivery method.

At approximately 123,676 GSF, this latest addition to the student housing, provides 268 students with suites and single rooms. A 40-50 student seating commons enclosed with glass provides a social space for student collaboration. Located within the dormitory are other amenities such as: a 460 seat dining room, two private dining rooms for faculties, a 40 station satellite fitness center, two large leisure rooms, a student grocery store, laundry facilities, student mail boxes, Resident Life Offices, campus police office, and a Hall Coordinator apartment.

Figure 2: South facade

Figure 3: South facade

STRUCTURAL OVERVIEW

The structure of the DH is a combination of shallow foundation and stone piers, and composite steel decking with steel framing. The exterior and interior walls are composed primarily of brick and concrete masonry.

FOUNDATIONS

CMT Laboratories, Inc, performed several test borings of the DH. According to their analysis for this site, the geotechnical engineers have determined that the site was filled with brown silty clay, and brown silty sand with gravel. Furthermore, it was found that the cohesive alluvial soils beneath the fill materials have low shear strength.

In light of these conditions, the conventional spread/column and continuous footing foundations will not provide adequate allowable bearing capacity to support the building. Deep foundations such as concrete filled tapered piles could support the structure but are not the most economical approach. Therefore, a practical solution is subsurface improvement with the use of shallow foundation.

Lastly, the final decision comes down to using stone piers which were considered the most technically sound and economically feasible method. Those stone piers are typically eighteen (18) to thirty-six (36) inches in diameter depending on their loading and settlement criteria.

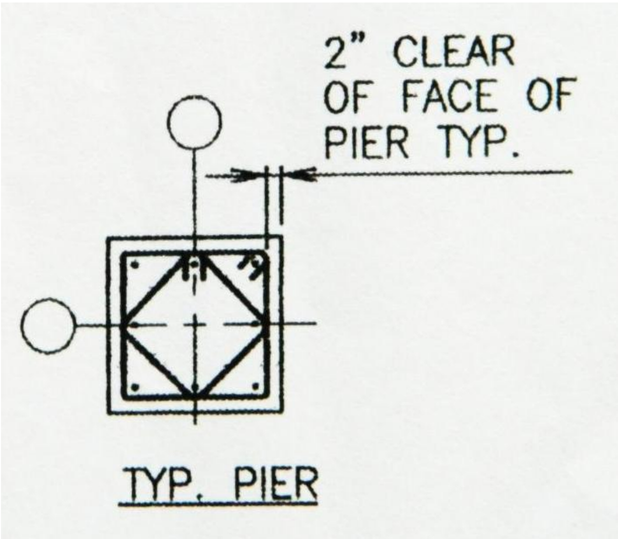


Figure 4: Typical Pier Detail

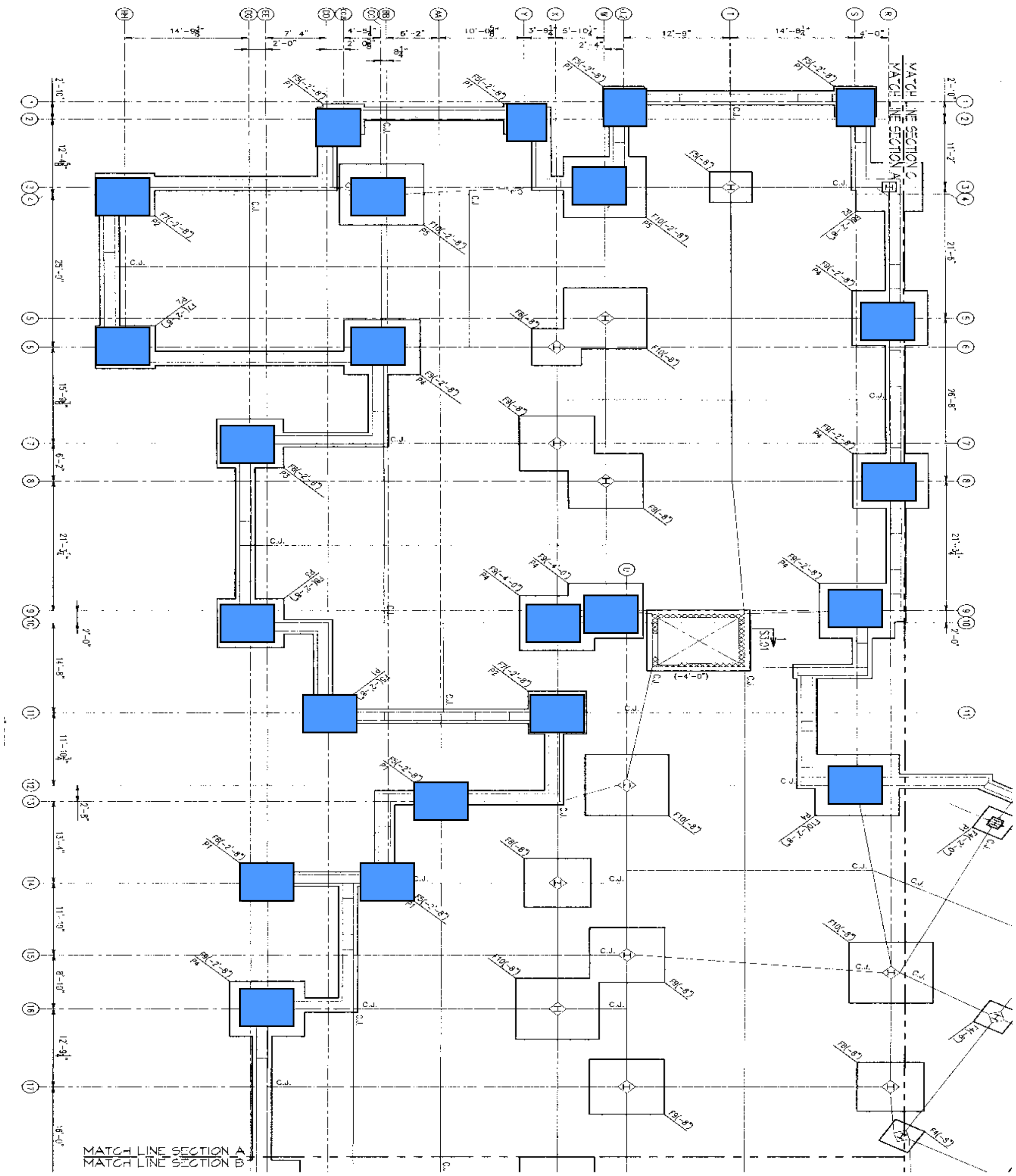
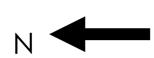


Figure 5: Stone Pier locations



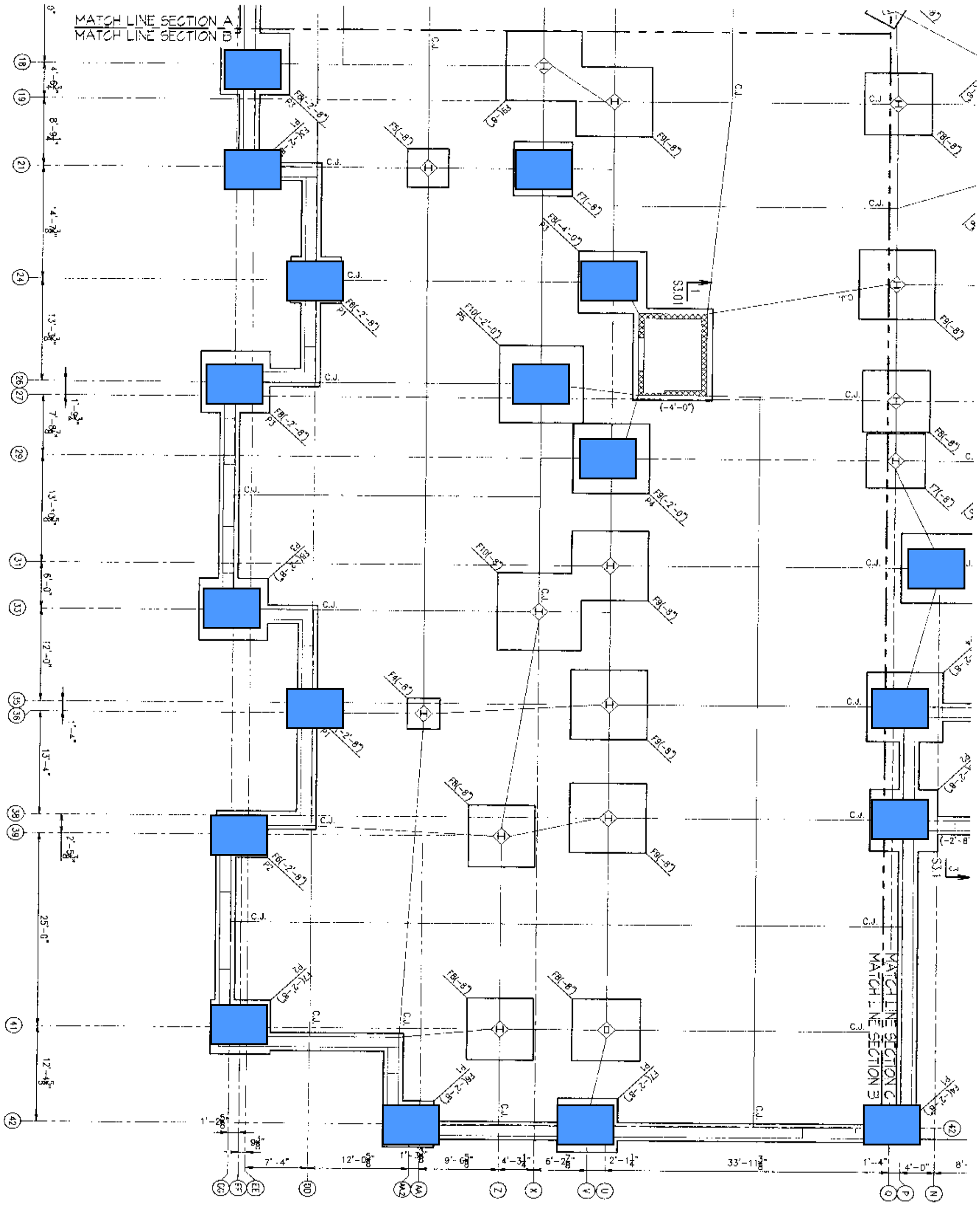


Figure 6: Stone Pier locations

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FLOOR SYSTEMS

The floor system of the DH is composed of 4” Light weight concrete slab, reinforced with 6”×6” –W2.9×W2.9 welded wire mesh, on 1 ½” - 20 gage Vulcraft composite deck. The joists, supporting the floor system, are spaced equally in column bays with a maximum spacing of 2’-0” O.C in areas of floor framing.

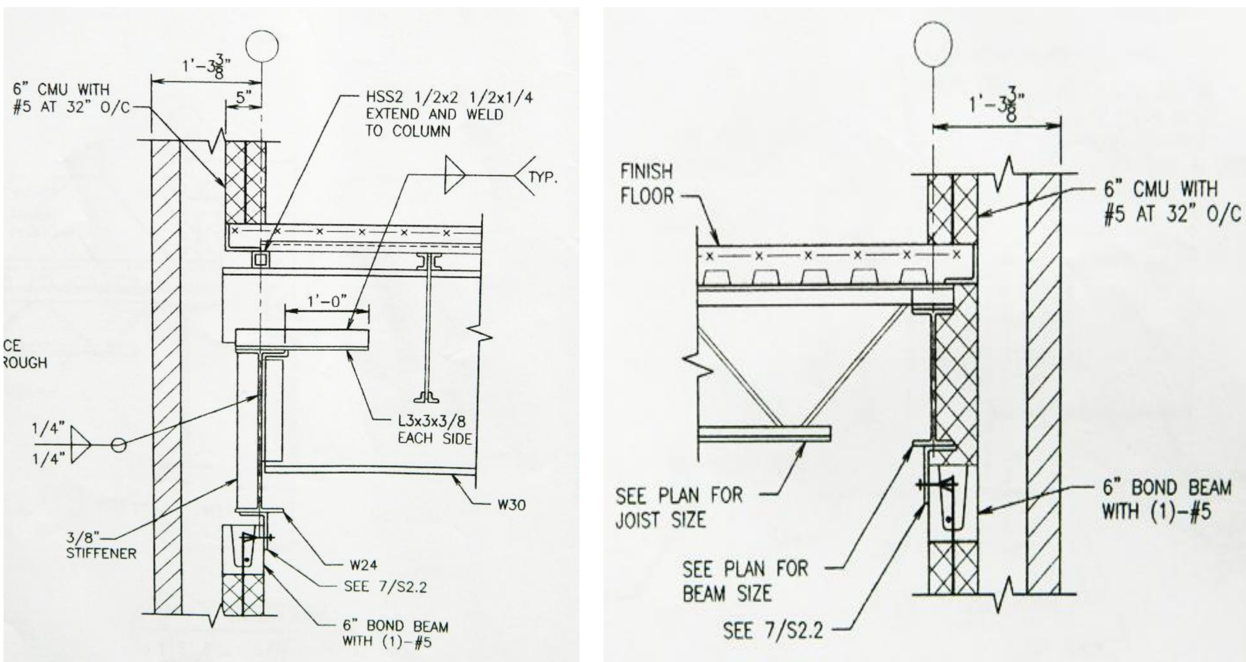

Figure 7: Typical Floor Section showing beam and columns relationship

FRAMING SYSTEM

The superstructure of the DH is primarily a combination of K-series joists, W24 girders, steel columns raging in size from W8's to W10's, and light gage metal framing. The K-series joists are spaced 2'-0" on O.C. The columns are typically on a 25'x30' grid and encased by 5/8" Gypsum board or 6" painted CMU. HSS columns were used in locations near the stairwells. Interior partitions consist of Concrete Masonry Units (CMU).

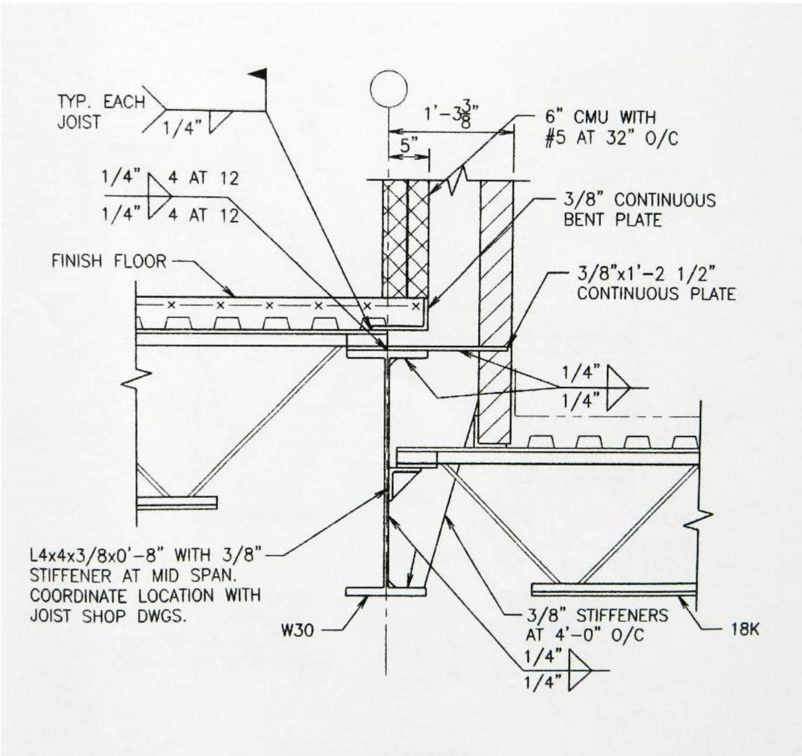

Figure 8: Joists and beam interaction

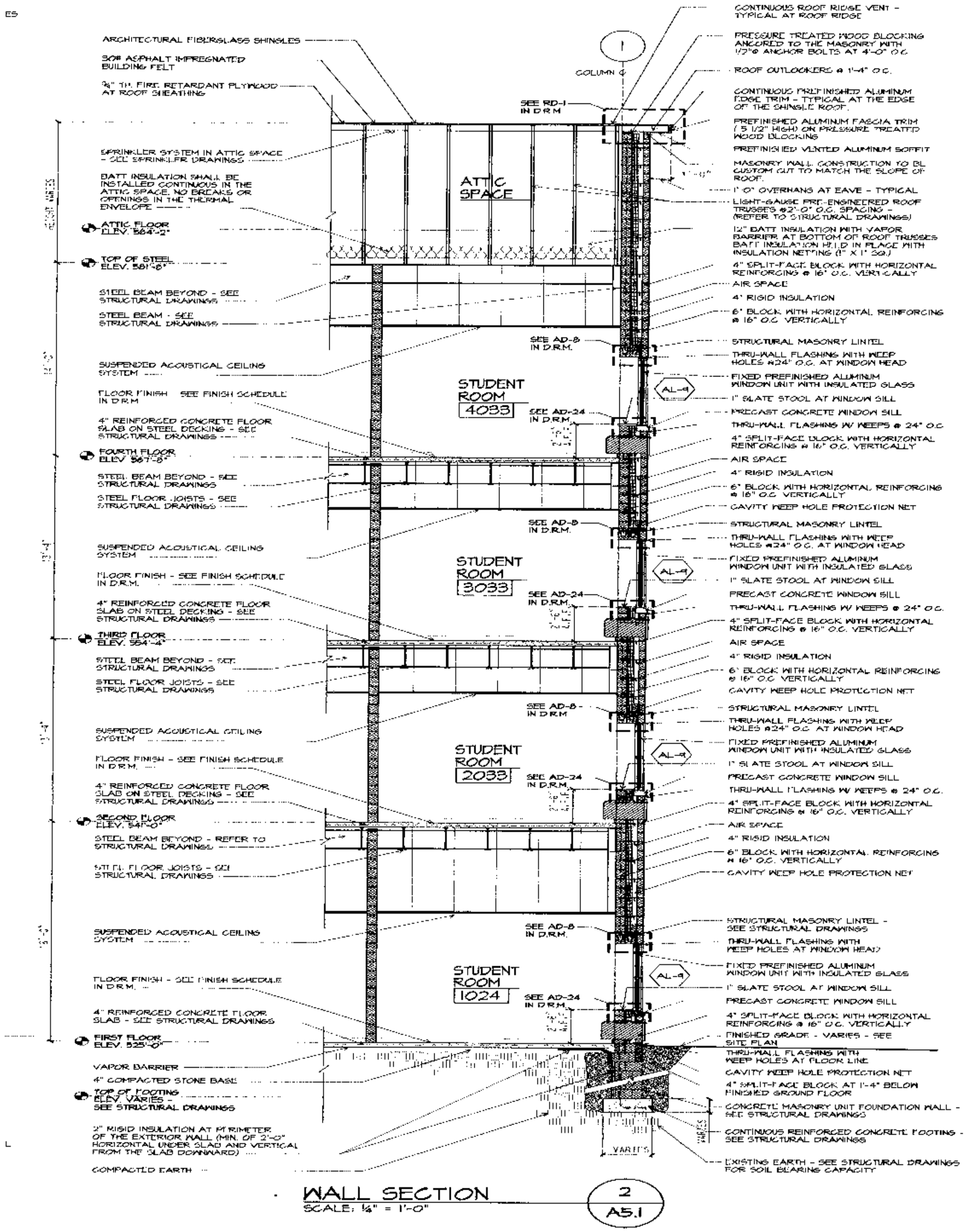

Figure 9: Wall Section

LATERAL SYSTEM

The lateral resisting system in the DH consists of steel moment connections in both the East-West direction and the North-South direction. The lateral resisting connections can be seen in figure 10 below.

The building façade collects wind forces that are then transferred to the respective floor diaphragm. These forces then travel through the diaphragm until the moment connections are engaged. The remaining of the technical report will discuss the lateral system in more detail.

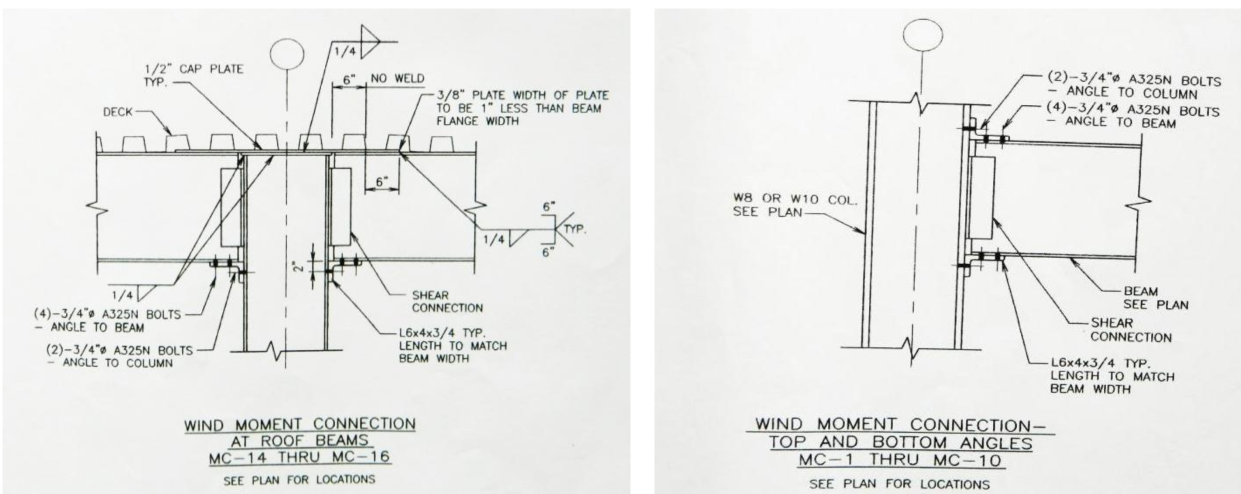

Figure 10: Moment Connections

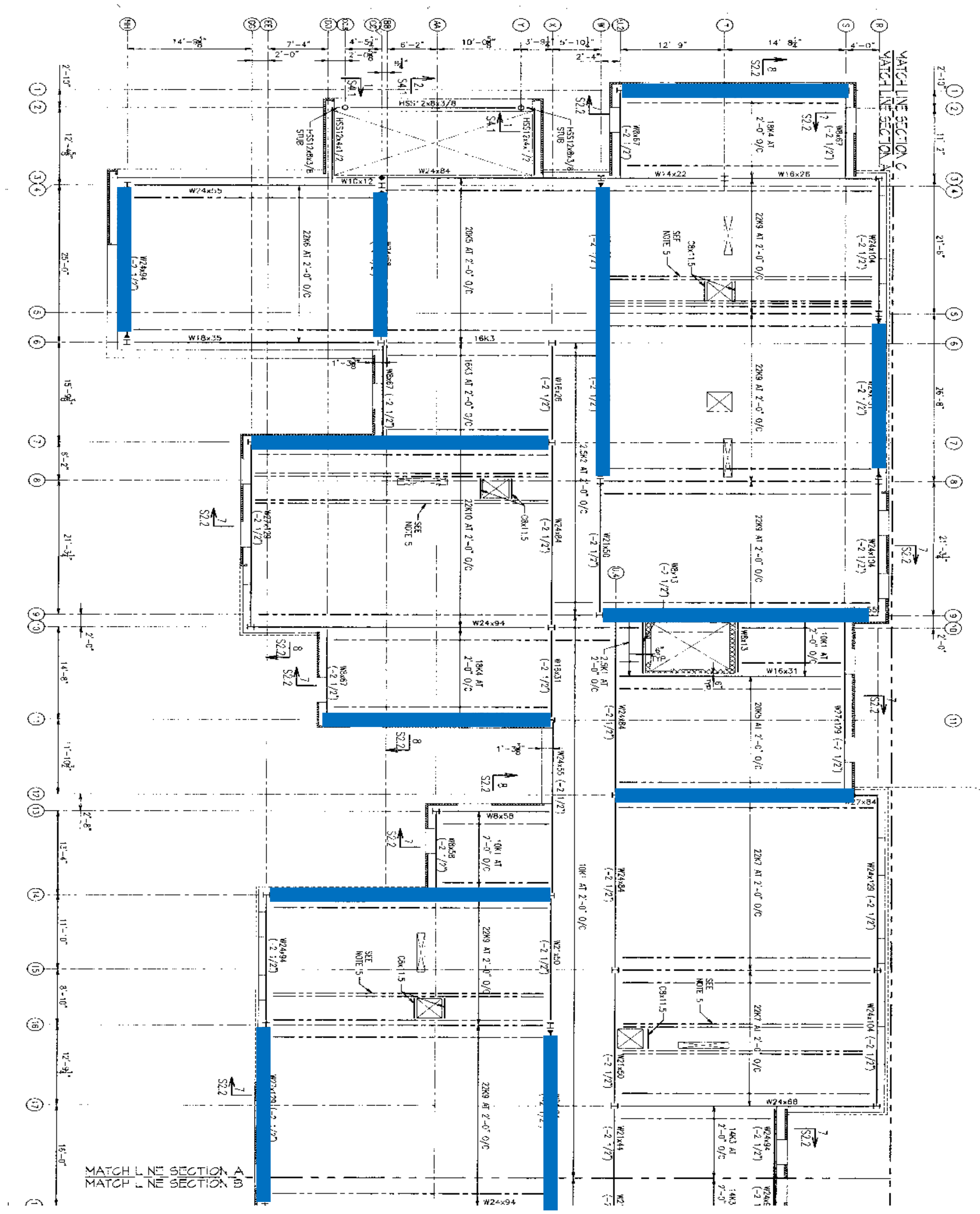

Figure 11: Moment Connections Location on the Building

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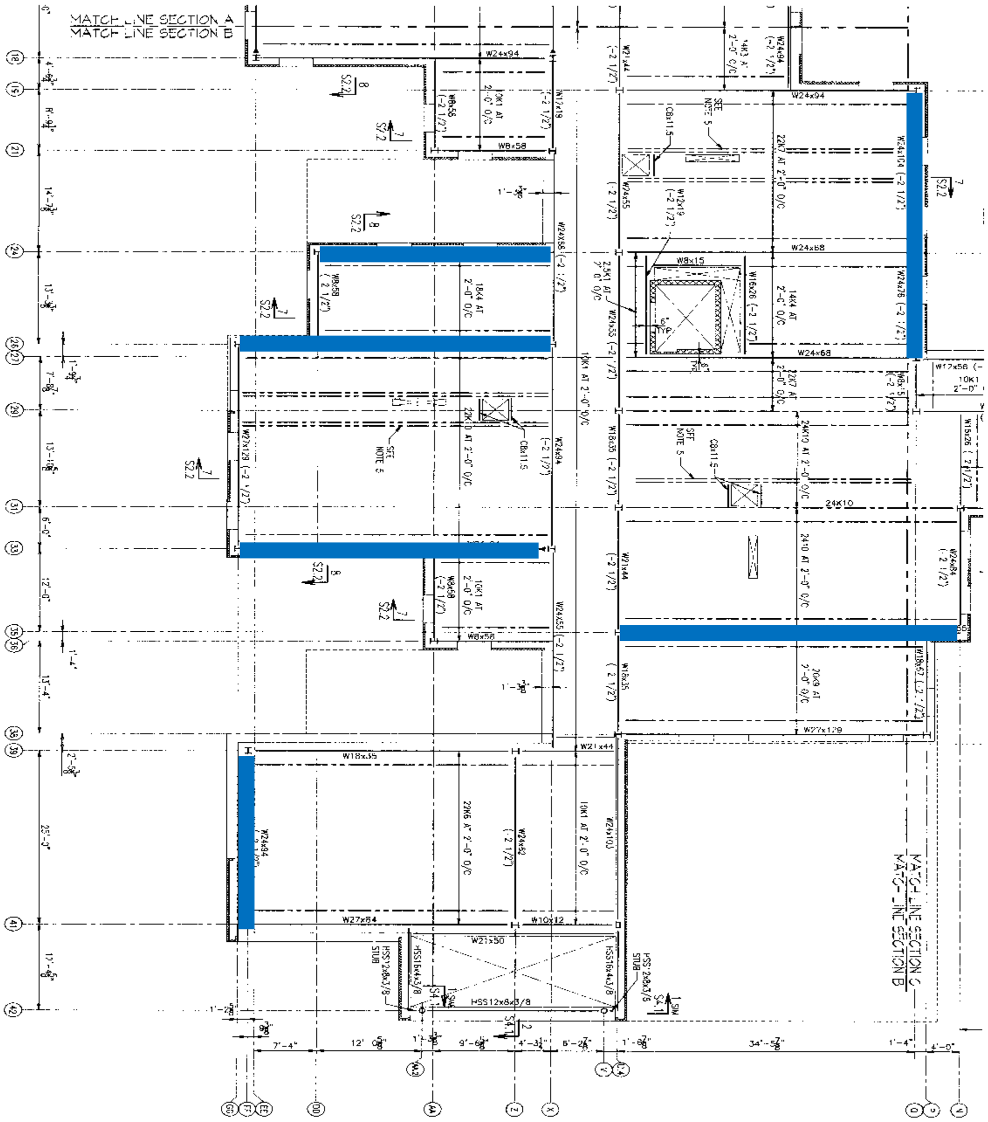
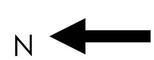


Figure 12: Moment Connections Location on the Building



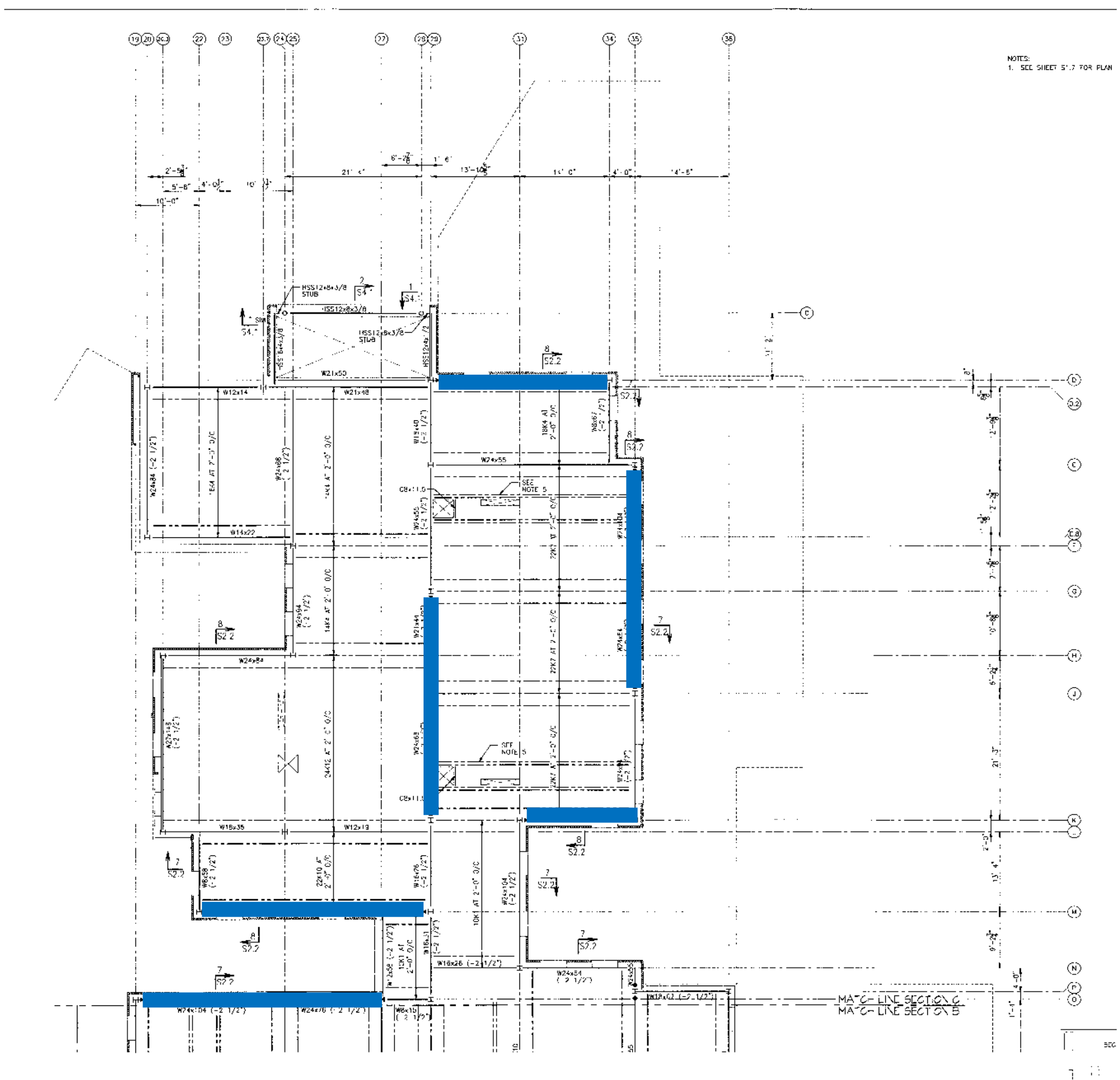


Figure 13: Moment Connections Location on the Building

ROOF SYSTEMS

There is only one roof system on the DH dormitory. It consists of 1 1/2" – 20 gage type B roof deck. The roof deck is then supported by joists spaced at a maximum distance of 4'-0" O.C. between the column bays.

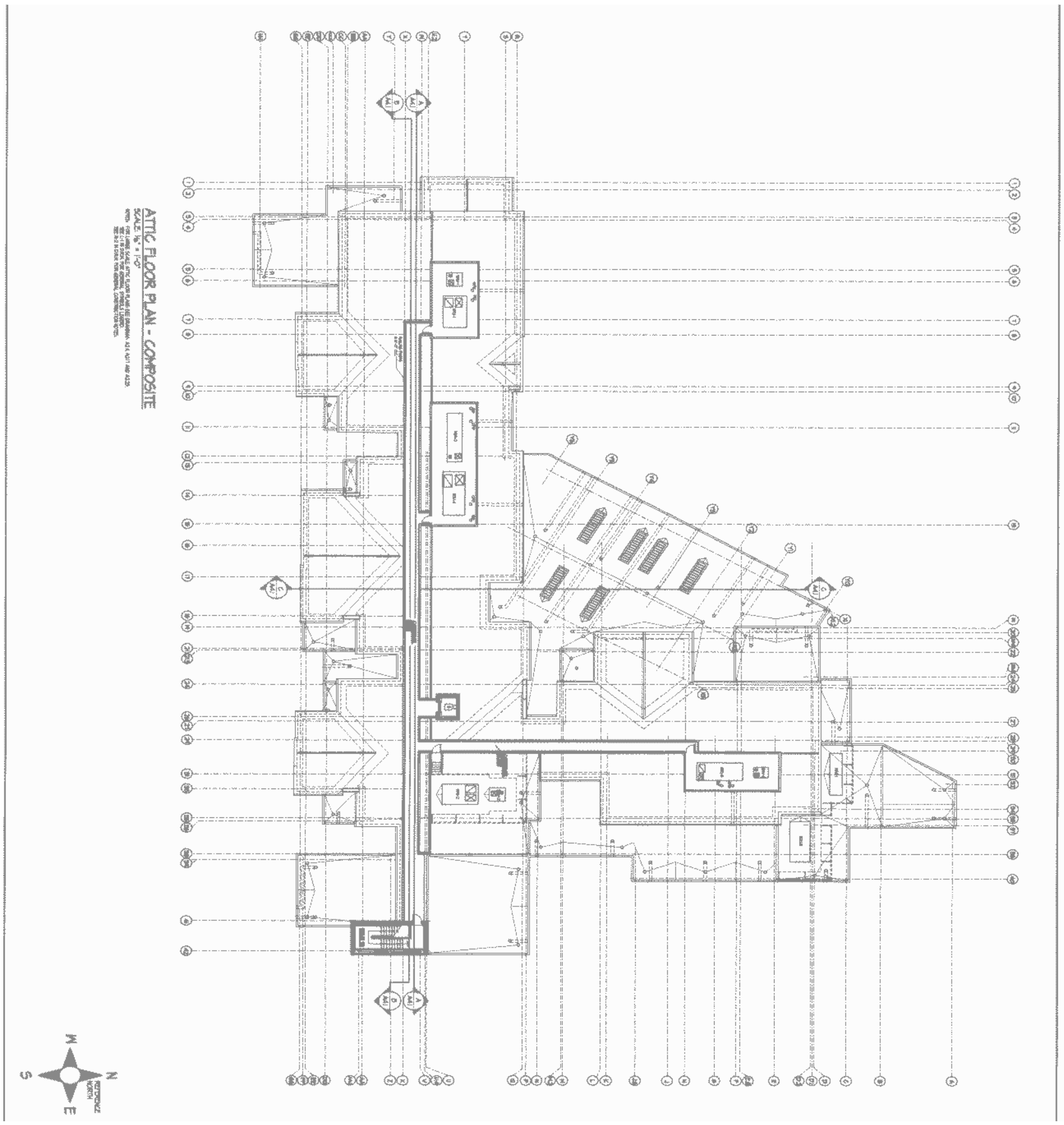


Figure 14: Roof plans

DESIGN CODES

All equipment and components of the DH are designed to comply with all applicable latest editions of articles and sections of the following codes in compliances with all Federal, State, County, and Local ordinances and regulations:

- 2006 International Building Code (IBC)
- National Electrical Code (NEC),
- Uniform Plumbing Code (UPC),
- National Sanitation Foundation (NSF)
- Specifications for structural concrete for buildings (ACI 301)
- Building Code Requirements for Reinforced Concrete (ACI 318-08)
- Recommended Practice for Hot Weather Concreting (ACI 305R)
- Recommended Practice for Cold Weather Concreting (ACI 306R)
- Recommended Practice for Concrete Formwork (ACI 347)
- American Society of Civil Engineers (ASCE 7- 10)

MATERIALS USED

The following tables provide a list of materials used in the design of this building. Those values were found in the structural drawing and the specifications.

Concrete		
Usage	**Weight**	**Strength (psi)**
Footings	Normal	4000
Foundation alls	Normal	4000
Slab-on-Grade	Normal	4000
Suspended Slabs	Normal	4000
Toppings	Normal	5000
Piers	Normal	4000

Table 1: Concrete materials

Steel		
Type	**Standard**	**Grade**
W-Shaped Structural Steel	ASTM A 572/A 572M	50
Channels, Angles-Shapes	ASTM A 36/A 36M	36
Plate and Bar	ASTM A 36/A 36M	36
Cold-Formed Hollow SS	ASTM A 500	B
Steel Pipe	ASTM A 53/A 53M	B
Bolts, Nuts, and Washers	ASTM A325/ASTM F 1852	N/A
Steel Deck	ASTM A 653	A
Reinforcing Bars	ASTM A 615/A 615M	60
Deformed Bars	ASTM 767	A
Welded Wire Fabric	ASTM A 615	65

Table 2: Steel materials

Masonry		
Type	**Standard**	**Strength (psi)**
Concrete Block	ASTM C 90/ ASTM C 145	1900
Split Face CMU	ASTM C 90lightweight	1900
Bond Beam	N/A	3000
Precast Stone	N/A	5000-7000
Concrete Brick	ASTM C 1634/ASTM C 55	N/A
Mortar	ASTM C 979	Type II
Grout	ASTM C 404	N/A

Table 3: Masonry materials

Miscellaneous	
Type	**Strength (psi)**
Concrete Fill	3000
Non-Shrink Nonmetallic Grout	ASTM C 1107

Table 4: Miscellaneous materials

GRAVITY LOADS

Included in this report is a summary of dead, live, and snow loads used in the thesis design. These values were compared to the actual design loads in the structural drawings.

DEAD AND LIVE LOADS

Superimposed Dead Loads		
Description	**Design Loads**	**Thesis Loads**
Roof		
Roofing	3 PSF	3 PSF
Framing	5 PSF	10 PSF
Insulation	3 PSF	3 PSF
Ceiling	2 PSF	2 PSF
Elec./Lights	3 PSF	3 PSF
Mechanical	5 PSF	5 PSF
Sprinklers	3 PSF	3 PSF
Miscellaneous	1 PSF	1 PSF
Total	**25 PSF**	**30 PSF**
Floor		
4” Slab and Deck (LWC)	44 PSF	57 PSF
Framing	5 PSF	15 PSF
Mechanical	5 PSF	5 PSF
Elec./Lights	3 PSF	3 PSF
Ceiling	2 PSF	2 PSF
Sprinklers	3 PSF	3 PSF
Miscellaneous	3 PSF	3 PSF
Total	**65 PSF**	**88 PSF**
Superimposed DL		**30 PSF**
Snow	**35 PSF**	**30 PSF**

Table 5: Design Dead Loads

Design Live Loads		
Description	**Design Loads**	**Thesis Loads**
Roof	35 PSF	30 PSF
First Floor	100 PSF	100 PSF
Stairs	100 PSF	100 PSF
Dorm Rooms	40 PSF	40 PSF
Corridors	100 PSF	100 PSF
Storage	125 PSF	125 PSF
Mechanical room	150 PSF	125 PSF
Common Areas	100 PSF	100 PSF

Table 6: Design Live Load

LATERAL LOADS

In order to better understand the lateral systems, wind loads and seismic loads were calculated in this technical report. These loads were calculated by hands, and then applied to a lateral model of the structure created in ETABs.

WIND

ASCE 7-10 Main Wind Force Resisting System (MWFRS) was used for the determination of the wind loads applied on the DH. Wind load calculations were performed with the assumptions that the façade and geometry of the DH was rectangular with no protrusions. The summary of results is found in table 7 and 8. For a more in depth look at wind load calculations please refer to Appendix A.

The wind loads on this structure are collected by the brick facades on the exterior of the building. The bricks then transfer these loads to the floor system, which in return transfers the loads to the columns through the moment connections. These columns return the loads to the foundations, and therefore to the grade. This load path is illustrated in Figure 14.

To simplify the repetitive process, most calculations were performed using Microsoft Excel spread sheet. The story forces at each level were calculated after wind pressures, including windward, leeward, and internal pressures were found. Wind loads were the largest in the N-S direction resulting in a base shear of 314 kips and an overturning moment of 11,533 ft-kips.

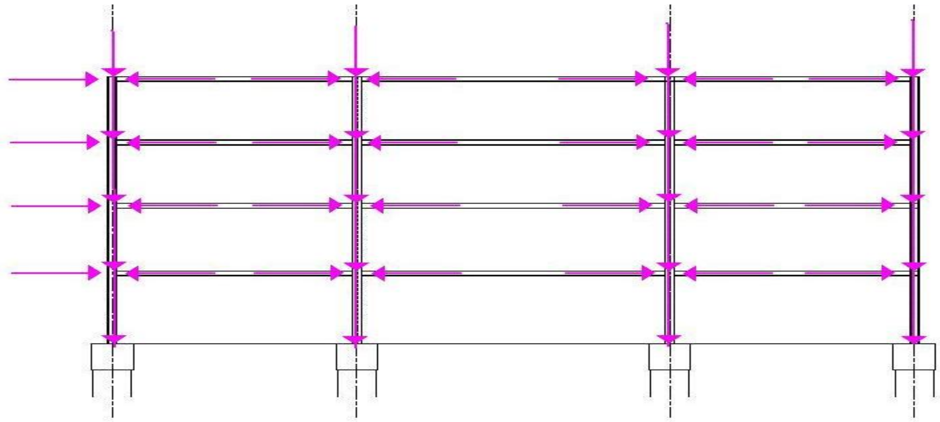


Figure 14: Lateral Load Distribution on the Frames

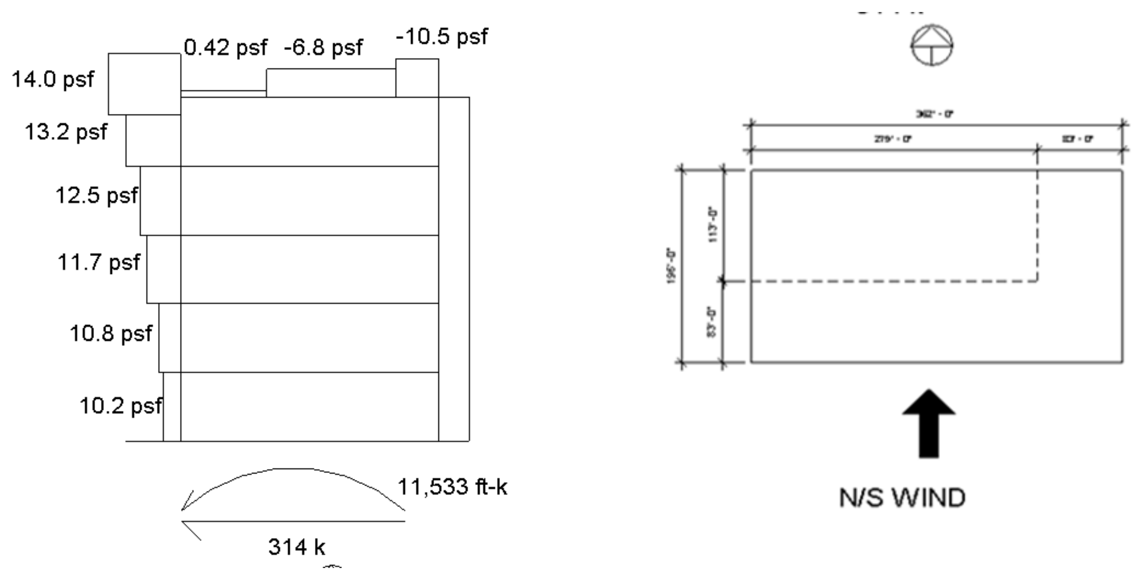


Figure 15: Wind Pressure diagram

Wind Pressures, Shear, Moment in N-S Direction			
Floor	**Force of windward pressure (k)**	**Windward Story Shear (k)**	**Windward Moment (ft-K)**
Ground	0	357	0
2nd	77	280	1229
3rd	74	206	2179
4th	80	126	3403
Attic Space	83	43	4722
Base	**314**		**11533**

Table 7: Wind Pressures, Shear, and Moment in N-S Direction

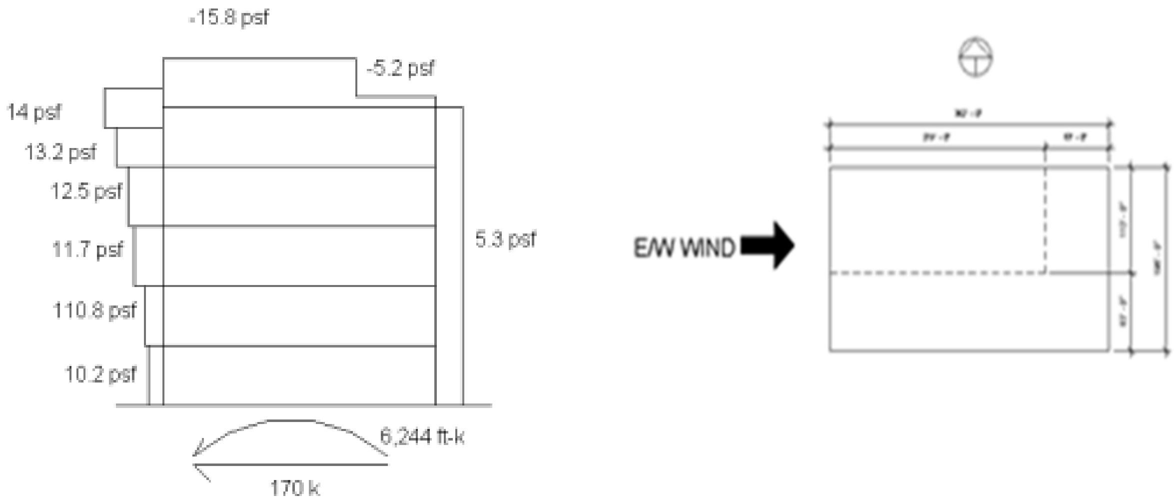


Figure 16: Wind Pressure diagram

Wind Pressure, Shear, Moment in E-W Direction			
Floor	**Force of windward pressure (k)**	**Windward Story Shear (k)**	**Windward Moment (ft-K)**
Ground	0	193	0
2nd	42	151	666
3rd	40	111	1180
4th	43	68	1842
Attic Space	45	23	2557
Base	**170**		**6244**

Table 8: Wind Pressures, Shear, and Moment in E-W Direction

SEISMIC

Seismic loads for the DH were performed using chapter 11 and 12 of ASCE 7-10 under the Equivalent Lateral Force Procedure (ELF). This procedure also assumes a simple building footprint. Various area square footages were assumed and approximated in the seismic hand calculations.

Since the DH used moment frames in both directions, the code specified period, T_a is independent of direction for this structure. Therefore a single analysis holds for both directions. This analysis resulted in a base shear of 335K and an overturning moment of 13,285 ft-kips. Please refer to Appendix B for a more in depth look at seismic load calculations.

Vertical Distribution of Seismic Forces in N/S Direction

Floor	Height (ft)	W_x	k	$hi^{\wedge k}$	$W_i*h_i^{\wedge K}$	Cv_x	F_x (k)	Story Shear V_x (k)	Moment M(ft-k)
Ground	0	8165	1.17	0.0	0.0	0.00	0	0	0
2nd	16	8165	1.17	25.6	209303.5	0.13	44	335	703.2
3rd	29.3	8165	1.17	52.0	424806.6	0.27	89	291	2613.7
4th	42.6	8165	1.17	80.6	658211.4	0.41	138	202	5888.1
Attic Space	56.6	1181	1.17	112.4	132754.0	0.08	28	64	1577.9
Roof	70	1181	1.17	144.1	170221.6	0.11	36	36	2502.2
				Σ (Wi*hi^K)=	**1595297.2**	1.00	335		13285.1

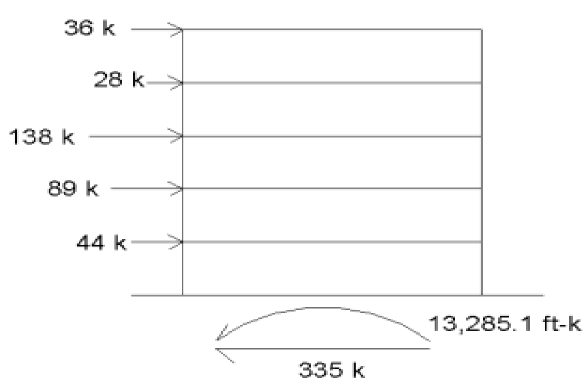


Table 9: Seismic Forces and Moment in N-S Direction

COMPUTER MODEL

A sectional 3D model was created using ETABs for the purpose of determining drift in order to obtain the relative stiffness of each frame element and determining the effects of loads on the complete lateral system. Each lateral element was modeled then connected by rigid diaphragm. The columns were modeled as “pinned” connections in order to achieve a conservative approximation of the column base fixity.

A hand calculation of the center of rigidity was done to determine the accuracy of the model.

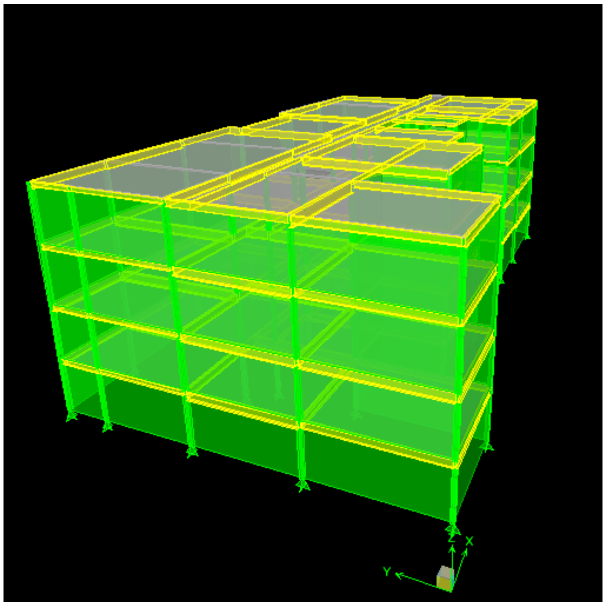


Figure 17: ETABs Model

The center of rigidity of each floor was determined using the relative stiffness of each frame element based on the ratio of the applied load to horizontal displacement caused.

$$ki = \frac{p}{\Delta p}$$

The center of rigidity was then found by dividing the sum of each elements stiffness times its location by the total stiffness in that direction. The summary of results can be found in table 10.

$$\mathrm{X} = \frac{\sum Kiy\, Xi}{\sum Kiy}$$

$$Y = \frac{\sum KixYi}{\sum Kix}$$

Center of Mass and Rigidity (ft)					Eccentricity	
	ETABS Output		Hand Calculations		ETABS Values	
	X	Y	X	Y	ex	ey
Center of Mass	2196.3	1153.7	-	-	54.2	28.4
Center of Rigidity	2250.5	1182.1	2290.2	1152.9	-	-

Table 10: Center of Mass and Rigidity

LOAD CASES

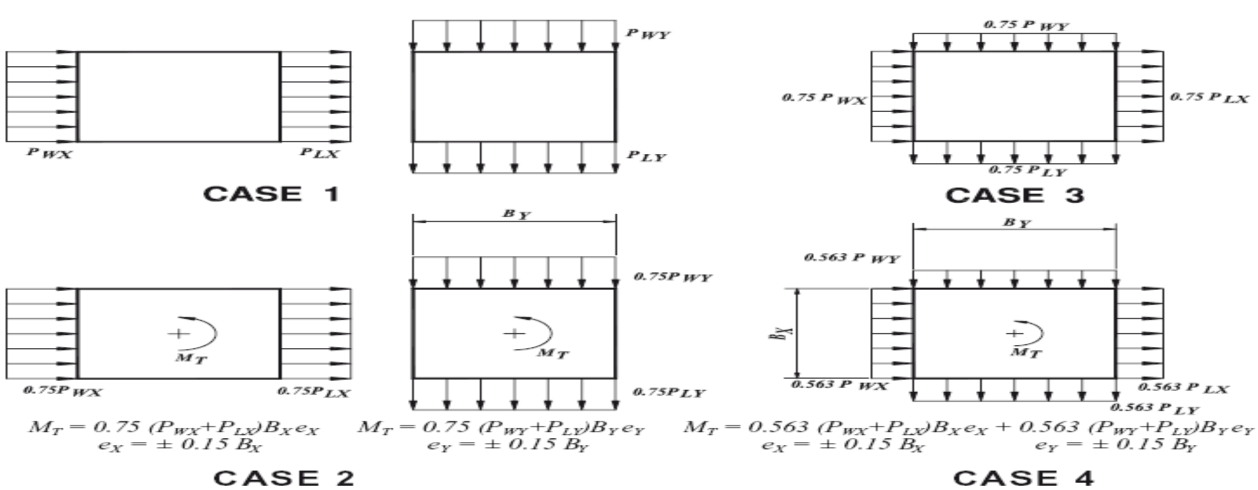
The lateral systems analyzed in this report are governed by the load combinations in ASCE 7-10. The following table shows the tabulated value of the load combinations taken under consideration.

Typically, only load case 2 will control for gravity loads. However, when lateral forces are being analyzed, load case 4 will control in this case.

Basic Load Combination		
Applicable Load Types		Lateral Load Types Only
1	1.4D	-
2	1.2D + 1.6L+ 0.5(Lr or S or R)	-
3	1.2 D + 1.6(Lr or S or R) + (L or 0.5W)	0.5W
4	1.2D + 1.0W + L + 0.5(Lr 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 Lr= Roof Live Load S= Snow Load		

Table 11: ASCE 7-10 Basic Wind Combination

ASCE 7-10 Figure 27.4 describes the different loading conditions for wind on a building. All four cases for the Main Wind Force Resisting System must be considered in the analysis of the lateral system. Case 2 and 4 consider the torsional loads that can be induced by wind loading.



DRIFT

The 3D ETABs model was used to determine the maximum drift for both wind and seismic forces. These values were then compared to maximum allowable drift to prevent cracking in the brick façade and other serviceability issues.

Only load case 4 shown previously was used to analyze wind loads under service loads. These deflections were then compared to H/600 to be more conservative. Table 12 and 13 show the tabulated values.

Maximum Drift NS - Wind (in)			
Floor	Height (hx)	Δ (ETABS)	Δ allowable = L/600
Roof	840	0.025866	1.4
Attic Space	679.2	0.031214	1.132
Story 4	511.2	0.038214	0.852
Story 3	351.6	0.04062	0.586
Story 2	192	0.04127	0.32

Table 12: Maximum Drift in N-S Direction under Wind Loads

Maximum Drift EW - Wind (in)			
Floor	Height (hx)	Δ (ETABS)	Δ allowable = L/600
Roof	840	0.02866	1.4
Attic Space	679.2	0.032136	1.132
Story 4	511.2	0.042136	0.852
Story 3	351.6	0.04475	0.586
Story 2	192	0.045686	0.32

Table 13: Maximum Drift in E-W Direction under Wind Loads

In the calculation of drift under seismic loads, factored loads were used. The drifts were then compared to a maximum drift of 0.02hx which is specified in ASCE 7-10 Table 12.12-1. These seismic drifts met all necessary deflection criteria.

Maximum Drift NS - Seismic (in)			
Floor	Height (hx)	Δ (ETABS)	Δ allowable = 0.02hsx
Roof	840	0.059925	16.8
Attic Space	679.2	0.054466	13.584
Story 4	511.2	0.050566	10.224
Story 3	351.6	0.040924	7.032
Story 2	192	0.033154	3.84

Table 14: Maximum Drift in N-S Direction under Seismic Loads

Maximum Drift EW - Seismic (in)			
Floor	Height (hx)	Δ (ETABS)	Δ allowable = 0.02hsx
Roof	840	0.060002	16.8
Attic Space	679.2	0.051222	13.584
Story 4	511.2	0.050667	10.224
Story 3	351.6	0.041012	7.032
Story 2	192	0.033204	3.84

Table 15: Maximum Drift in E-W Direction under Seismic Loads

TORSION

When the center of mass and the center of rigidity are not located at the same point, the lateral loads applied to the building will induce torsion. The induced eccentricity multiply by the force will produce a moment. The center of mass and rigidity are tabulated in the table below.

Center of Mass and Rigidity (ft)					Eccentricity	
	ETABS Output		Hand Calculations		ETABS Values	
	X	Y	X	Y	ex	ey
Center of Mass	2196.3	1153.7	-	-	54.2	28.4
Center of Rigidity	2250.5	1182.1	2290.2	1152.9	-	-

The center of mass and rigidity are not too far apart which will result in a negligible moment in this case. However, for the purpose of this report, a torsional analysis procedure will be elaborated and a full analysis will be completed in the proposal to ensure that the lateral load effects on the building are minimal.

The direct force and torsional force in each element is calculated using the following equation:

$$Fiy = \frac{Kiy}{\Sigma \mathrm{Kjy}}\,(Py)$$

$$Fit = \frac{\mathrm{Ki} * \mathrm{di} * \mathrm{Py} * \mathrm{e} * \mathrm{dx}}{\Sigma \mathrm{kj} * (\mathrm{dj})^\wedge 2}$$

d_j = perpendicular distance to centroid

F_{jt} = Forces due to torsion

e = Eccentricity

k= Stiffness

P_y = Loads

Fi_y = Direct force

After determining the direct and torsional force on each element, the force in each frame due to the lateral force is evaluated as following:

$$F_i = F_{idirect} \pm F_{itorsion}$$

SPOT CHECKS

In order to verify the validity of member sizes in this analysis, two spot checks were completed. A typical girder and a typical column on the ground floor were checked for strength under controlling wind and seismic loads. The members were more than sufficient to support the given controlling loads. View Appendix C for calculations supporting this data.

CONCLUSION

After reviewing the structural system of the Dauphin Hall, three major conclusions can be drawn from this report.

It was shown that wind loads were the controlling load factor in both North-South and East-West direction. In addition, load case 4 (ASCE 7-10) was the governing load case combination.

Drift and torsion were checked respectively. The lateral drift resulting from both wind and seismic forces were found to meet industry standards. Torsional effects were assumed to have a minimal effect on each frame due to a relatively small eccentricity between the center of rigidity and the center of mass. However, these torsional effects will be fully investigated in the proposal to ensure the stability of the building. A sectional ETABs model was developed and its results were compared to hand calculations.

Lastly, a girder and a column on the ground floor were checked and it was found that both the girder and the column have adequate capacity.

APPENDICES

APPENDIX A: WIND LOAD CALCULATIONS

General Wind Design Criteria		
Design wind Speed (V)	90 MPH	ASCE 7-10
Directionality Factor (kd)	0.85	ASCE 7-10
Important factor (Iw)	1.15	ASCE 7-10
Exposure Category	C	ASCE 7-10
Topographic Factor (Kzt)	1	ASCE 7-10
Internal Pressure (Gcpi)	0.18	ASCE 7-10

Velocity Pressures Coeff. And Velocity Pressure			
Level	Elevation	Kz	Qz
Gound	0	0.85	15.0
2nd	16	0.9	15.9
3rd	29.3	0.98	17.3
4th	42.6	1.04	18.3
Attic Space	56.6	1.1	19.4
Roof	70	1.17	20.6

External Pressure coeff. (Cp)		
Description	N-S Wind	E-W Wind
L/B	0.54	1.84
Winward Wall	0.8	0.8
Leeward Wall	-0.5	-0.3
Side Walls	-0.7	-0.7
h/L	0.179	0.33
Roof	h/L < 0.5	h/L < 0.5
	-0.388	0.8
	0.024	-0.3
	-0.6	-0.7

Wind Pressures -N-S Direction						
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressure (psf)	Net Pressure(psf)	
				(+/-)(Gcpi)	(+)(Gcpi)	(-)(Gcpi)
Windward Walls	Ground	0	10.2	0.18	6.5	13.9
	2nd	16	10.8	0.18	7.1	14.5
	3rd	29.3	11.7	0.18	8.0	15.4
	4th	42.6	12.5	0.18	8.8	16.2
	Attic Space(AS)	56.6	13.2	0.18	9.5	16.9
	Roof	70	14.0	0.18	10.3	17.7
Leeward Walls	All	All	-8.8	0.18	-12.5	-5.1
Side Walls	All	All	-12.3	0.18	-16.0	-8.6
Roof	-	0-31.7	0.42	0.18	-3.3	4.1
	-	31.7-63.4	-6.8	0.18	-10.5	-3.1
	-	63.4-70	-10.5	0.18	-14.2	-6.8

Wind Pressures -E-W Direction						
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressure (psf)	Net Pressure(psf)	
				(+/-)(Gcpi)	(+)(Gcpi)	(-)(Gcpi)
Windward Walls	Ground	0	10.2	0.18	6.5	13.9
	2nd	16	10.8	0.18	7.1	14.5
	3rd	29.3	11.7	0.18	8.0	15.4
	4th	42.6	12.5	0.18	8.8	16.2
	Attic Space	56.6	13.2	0.18	9.5	16.9
	Roof	70	14.0	0.18	10.3	17.7
Leeward Walls	All	All	-5.3	0.18	-9.0	-1.6
Side Walls	All	All	-12.3	0.18	-16.0	-8.6
Roof	-	0-31.7	-15.8	0.18	-19.5	-12.1
	-	31.7-63.4	-15.8	0.18	-19.5	-12.1
	-	63.4-70	-5.2	0.18	-8.9	-1.5

Wind Load 4/4

Design criteria ASCE 7-10

Design wind Pressures for MWFRS

Table 1.5-1 → Risk category III

Fig 26.5-1 → Basic wind speed $V = 90\ mph$

Table 26.6-1 → Directionality factor: $K_d = 0.85$

Section 26.7 → Exposure category C

Section 26.8 → Topographic factor = $K_{zt} = 1.0$

Section 26.9 → Gust effect factor: $G = 0.85$ (Rigid building)

Section 26.10 → Enclosure classification: Enclosed

Table 26.11-1 → Internal pressure coeff: $(GC_{pi}) = \pm 0.18$

Table 27.3-1 → Velocity pressure exposure coefficient $K_z = 1.17$ @ 70 ft.

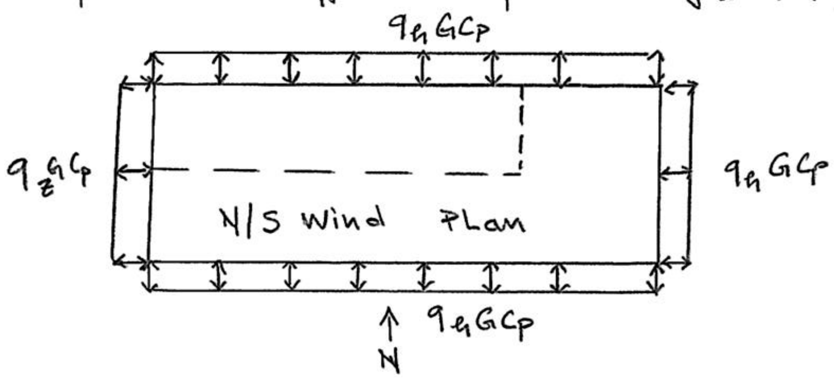
Eq. 27.3-1 ⇒ velocity pressure q_z:

$$q_z = 0.00256\, K_z K_{zt} K_d V^2$$

$$q_z = (0.00256)(1.17)(1.0)(0.85)(90)^2 = \boxed{20.6\ psf = q_z}$$

$$\boxed{q_h = 20.6\ psf}$$

External pressure coefficient C_p. (Fig 27.4-1)

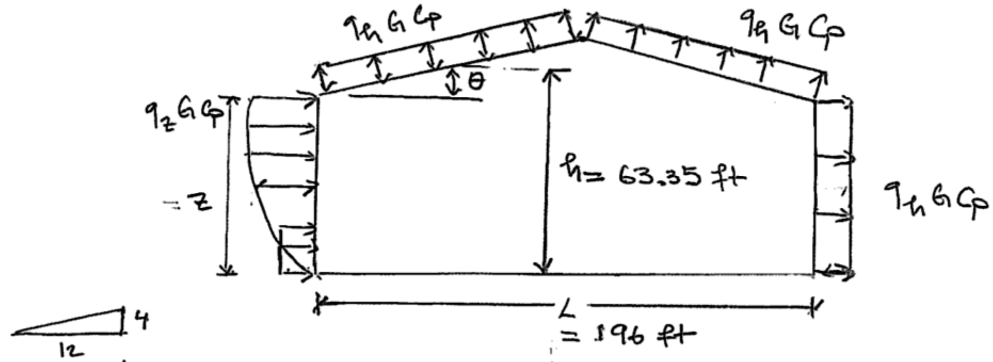


$\frac{L}{B} = \frac{196}{362} = 0.54 < 1.0$ ⇒ winward wall: $C_p = 0.8$

Leeward wall: $C_p = -0.5$

Side walls: $C_p = -0.7$

2/4



$\theta = \tan^{-1}\left(\frac{5}{12}\right) = 22.6°$ N/S wind elevation

$\frac{h}{L} = \frac{63.35}{196} = 0.32 \le 0.5 \Leftrightarrow$ windward roof: $C_p = -0.388$; $C_p = 0.024$ interpolation

wind $\perp$ to ridge

leeward roof: $C_p = -0.6$

Wind Pressure

Windward wall: $P = q G_f C_p - q_i (G C_{pi})$ Eq. 27.4.2

$P = (20.6)(0.85)(0.8) - (20.6)(\pm 0.18) = \boxed{(14 \pm 3.7)\text{ psf}}$

Leeward wall:

$P = (20.6)(0.85)(-0.5) - (20.6)(\pm 0.18) = \boxed{(-8.8 \pm 3.7)\text{ psf}}$

Side wall:

$P = (20.6)(0.85)(-0.7) - (20.6)(\pm 0.18) = \boxed{(-12.3 \pm 3.7)\text{ psf}}$

Roof:

- Windward (positive):

$P = (20.6)(0.85)(0.024) - (20.6)(\pm 0.18) = \boxed{(0.42 \pm 3.7)\text{ psf}}$

Windward (negative):

$P = (20.6)(0.85)(-0.388) - (20.6)(\pm 0.18) = \boxed{(-6.8 \pm 3.7)\text{ psf}}$

Leeward:

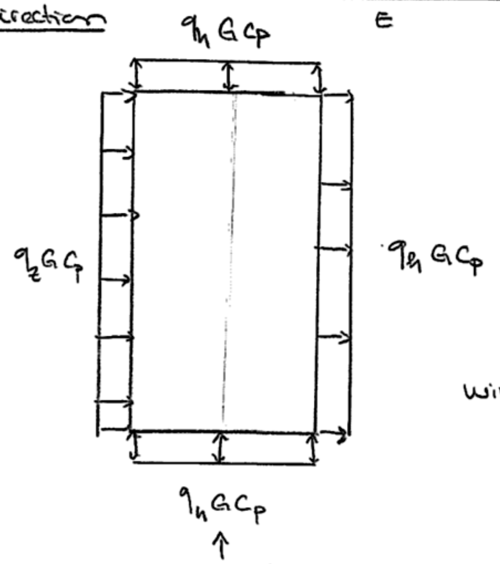
$P = (20.6)(0.85)(-0.6) - (20.6)(\pm 0.18) = \boxed{(-10.5 \pm 3.7)\text{ psf}}$

Force of windward Pressure

$$\frac{1}{1000\text{ lbs}}\left[\frac{h}{2}(\text{wind pressure a}) + \frac{h}{2}(\text{wind pressure b})\right]$$

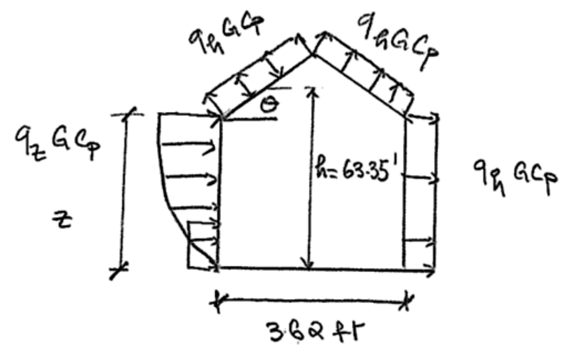
3/4

East/West direction



wind // to ridge

E/W wind Plan



$\theta = 22.6°$

E/W wind elevation

$\frac{L}{B} = \frac{362}{196} = 1.84 \geq 1.0$

windward wall: $C_p = 0.8$

leeward wall: $C_p = -0.3$

side walls: $C_p = -0.7$

Roof: $h/L = \frac{63.35}{362} = 0.175 \leq 0.5$

windward roof: $C_p = -0.9;\ -0.18$ for $0 - \frac{63.35}{2} = 31.7\ ft$

$C_p = -0.9;\ -0.18$ for $31.7\ ft - 63.4\ ft$

$C_p = -0.3;\ -0.18$ for $> 2(63.4) = 126.8\ ft$

4/4

Wind Pressure (E/W)

$P = qG_fC_p - q_i(GC_{p_i})$

Windward wall:

$P = (20.6)(0.85)(0.8) - (20.6)(\pm 0.18) = \boxed{(14 \pm 3.7)\ psf}$

Leeward wall:

$P = (20.6)(0.85)(-0.3) - (20.6)(\pm 0.18) = \boxed{(-5.3 \pm 3.7)\ psf}$

Side wall:

$P = (20.6)(0.85)(-0.7) - (20.6)(\pm 0.18) = \boxed{(-12.3 \pm 3.7)\ psf}$

Roof pressures:

windward: 0 to 32 ft

$P = (20.6)(0.85)(-0.9) - (20.6)(\pm 0.18) = \boxed{(-15.8 \pm 3.7)\ psf}$

windward: 32 to 63.4 ft

$P = (20.6)(0.85)(-0.9) - (20.6)(\pm 0.18) = \boxed{(-15.8 \pm 3.7)\ psf}$

Leeward: 63.4 ft to 70 ft

$P = (20.6)(0.85)(-0.3) - (20.6)(\pm 0.18) = \boxed{(-5.2 \pm 3.7)\ psf}$

APPENDIX B: SEISMIC LOAD CALCULATIONS

Design Criteria ASCE7-10	
Description	**Value**
Ss	18% g
S1	6% g
Fa	1.6
Fv	2.4
Sms	0.288
Sm1	0.144
SDs	0.192
SD1	0.096
Ie	1.25
Seismic Design categoy	B
Ct	0.028
x	0.8
T	0.84
Cs	0.041
W (k)	8165
V (K)	335

Vertical Distribution of Seismic Forces in N/S Direction									
Floor	**Height (ft)**	**Wx**	**k**	**hi^k**	**Wi*hi^K**	**Cvx**	**Fx (k)**	**Story Shear Vx (k)**	**Moment M(ft-k)**
Ground	0	8165	1.17	0.0	0.0	0.00	0	0	0
2nd	16	8165	1.17	25.6	209303.5	0.13	44	335	703.2
3rd	29.3	8165	1.17	52.0	424806.6	0.27	89	291	2613.7
4th	42.6	8165	1.17	80.6	658211.4	0.41	138	202	5888.1
Attic Space	56.6	1181	1.17	112.4	132754.0	0.08	28	64	1577.9
Roof	70	1181	1.17	144.1	170221.6	0.11	36	36	2502.2
				Σ (Wi*hi^K)=	**1595297.2**	1.00	335		13285.1

SEISMIC ANALYSIS 1/2

Design Criteria ASCE7-10

Assumption : site Class D (stiff soil)

Fig. 22.1 → $S_S = 18\% g$ for PA

Fig. 22.2 → $S_1 = 6\% g$ for PA

Eq. 11.4-1 → $S_{MS} = F_a S_S$

Eq. 11.4-2 → $S_{M1} = F_v S_1$

Table 11.4-1 → $F_a = 1.6$ for PA ($S_s < 0.25$ & site class D)

⇒ $S_{MS} = 1.6(0.18) = 0.288$

Table 11.4-2 → $F_v = 2.4$ For PA ($S_1 < 0.1$ & site class D)

⇒ $S_{M1} = 2.4(0.06) = 0.144$

Eq. 11.4-3 → $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.288) = 0.192$

Eq. 11.4-3 → $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.144) = 0.096$

Section 11.5.1 → $I_e = 1.25$ (Risk category III)

Table 11.6-1 → Seismic design Category (SDC) = "B"

($S_{DS} = 0.192$ & occupancy category II)

Table 11.6-2 → Seismic design Category (SDC) = "B"

($S_{D1} = 0.096$ & O.C II)

Equivalent Lateral Force Procedure (Both directions)

Seismic Base Shear

Eq 12.8-1 → $V = C_s W$

Table 12.2-1 → $R = 3.5$ for Steel ordinary moment frame

Table 12.8-2 → $C_t = 0.028$ (steel moment resisting frame)

$x = 0.8$

2/0

Fundamental Period: $T_a = C_t h_n^x$ (Eq. 12.8-7)

$T_a = (0.028)(70)^{0.8} = 0.84$ sec

$T = C_u T_a$ with $C_u = 1.7$ Table 12.8-1

$T = (0.84)(1.7) = 1.428$ sec ← upper limit.

Fig. 22-15 → $T_L = 6$ sec > $T = 0.84$ sec

$C_s =$ (seismic response coeff)
$= \dfrac{S_{DS}}{(R/I_e)} = \dfrac{0.192}{(3.5/1.25)} = 0.069$ Eq. 12.8-2

For $T < T_L$, Eq. 12.8-3

$= \dfrac{S_{D1}}{(R/I_e)T} = \dfrac{0.096}{(3.5/1.25)(0.84)} = 0.041$

$C_s = 0.069 > 0.041$ ∴ Not good.

eq 12.8-5 → $C_s = 0.044 S_{DS} I_e = 0.044(0.192)(1.25) = 0.011$ ← Min

use $C_s = 0.041$

Seismic Base shear

Eq 12.8-1 → $V = C_s W$

$W =$ Seismic weight calculated in spread sheet.

$V = (0.041)(8165 \text{ Kips}) = 335^k$

Vertical Distribution of seismic forces

Eq. 12.8-11 → $F_x = C_{vx} V$

where $C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ → Eq. 12.8-12

For $0.5 < T < 2.5$ ($k=1$, $k=2$)

$K = 1.17$ ← Interpolation.

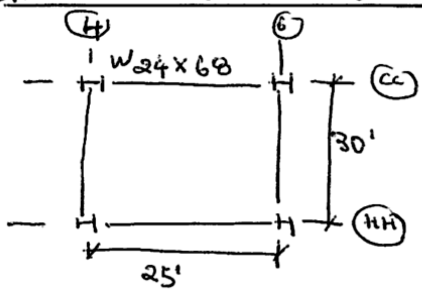
See spreadsheet for forces, shears, and moments

$\boxed{V_x = \sum_{i=x}^{n} F_i}$ (Eq 12.8-13)

APPENDIX C: SPOT CHECKS

Spot check

Ground Floor Girder spot check



LL = 100 psf

SDL = 30 psf

Floor DL = 30 psf

Controlling load combination

$1.2D + 1.0W + L + 0.5S = 187\ psf$

Trib. width $= \frac{30}{2} = 15\ ft$

$w_u = (187)(15) = 2.8\ KLF$

$M = \frac{w_u l_n^2}{8} = \frac{(2.8)(25)^2}{8} = 219\ ft-K$

Using Steel Manual Table 3-2

$\phi M_p = 664\ ft-K >> 219\ ft-K$

Check serviceability

$\Delta_{Total\ L} = \frac{5wl^4}{384EI} = \frac{(5)(2.8)(25)^4(1728)}{384(29000)(1830)} = 0.46 < 1.25 = \frac{L}{240}$

∴ ok

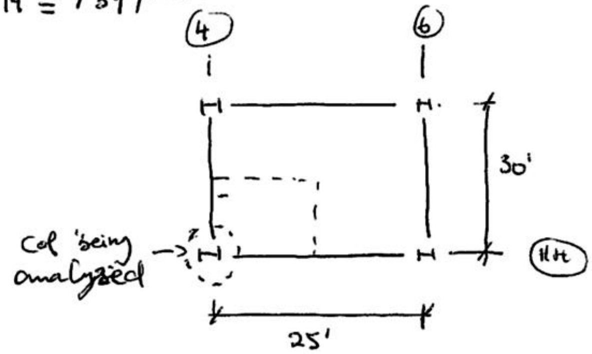
$\boxed{\text{W24X68 girder works}}$

Spot check

Check column @ Ground floor (col. line (HH) + 4)

W10x49

$M = 139$ ft-K



$$A_T = \left(\frac{30}{2}\right)\left(\frac{25}{2}\right) = 187.5 \text{ ft}^2$$

$$A_I = 4(187.5) = 750$$

$$L_{reduced} = 100\left[0.25 + \frac{15}{\sqrt{750}}\right] = 79.77 \text{ psf}$$

$0.4L = 0.4(100) = 40 < 79.77$ ∴ use 79.77

$P_{LL} = 750(79.77) = 59.8^K$

$P_{DL} = 60(750) = 45^K$

$P_u = 1.2D + 1.0L = 1.2(45) + 59.8 = 113.8^K$

Using Table 6-1

$p = 2.34 \times 10^3$ $\qquad$ $KL = 16'$

$b_x = 4.43 \times 10^3$ K-ft

$$\frac{P_u}{P_c} = \frac{113.8}{113} = 1.0 \geq 0.2$$

$$p P_u + b_x M_{ux} = (2.34\times10^{-3})(113.8) + (4.43\times10^{-3})(139'k)$$

$$= 0.27 + 0.615 = 0.88 < 1.0$$

so OK

APPENDIX D: FLOOR PLANS

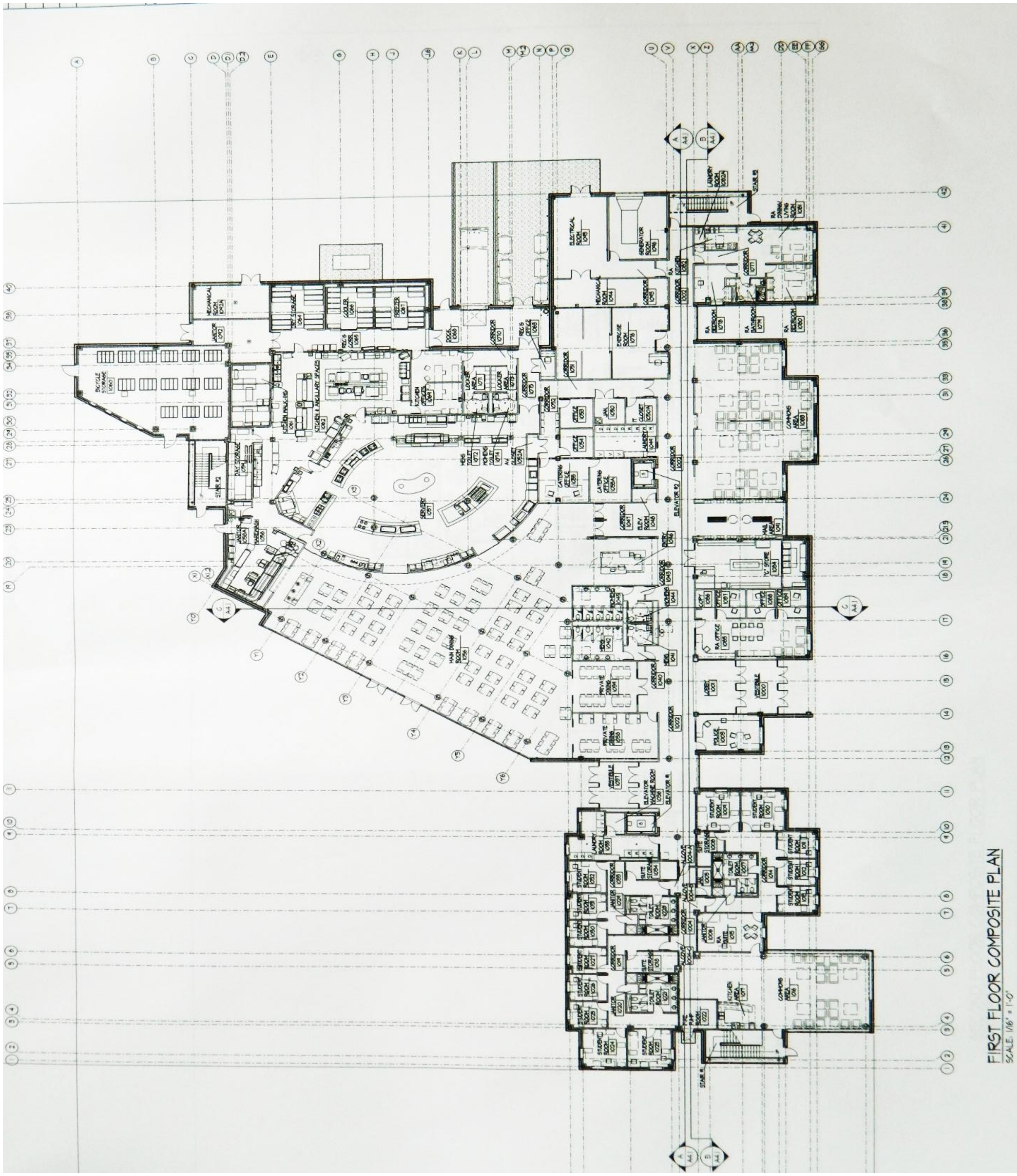


Figure 18: Ground floor

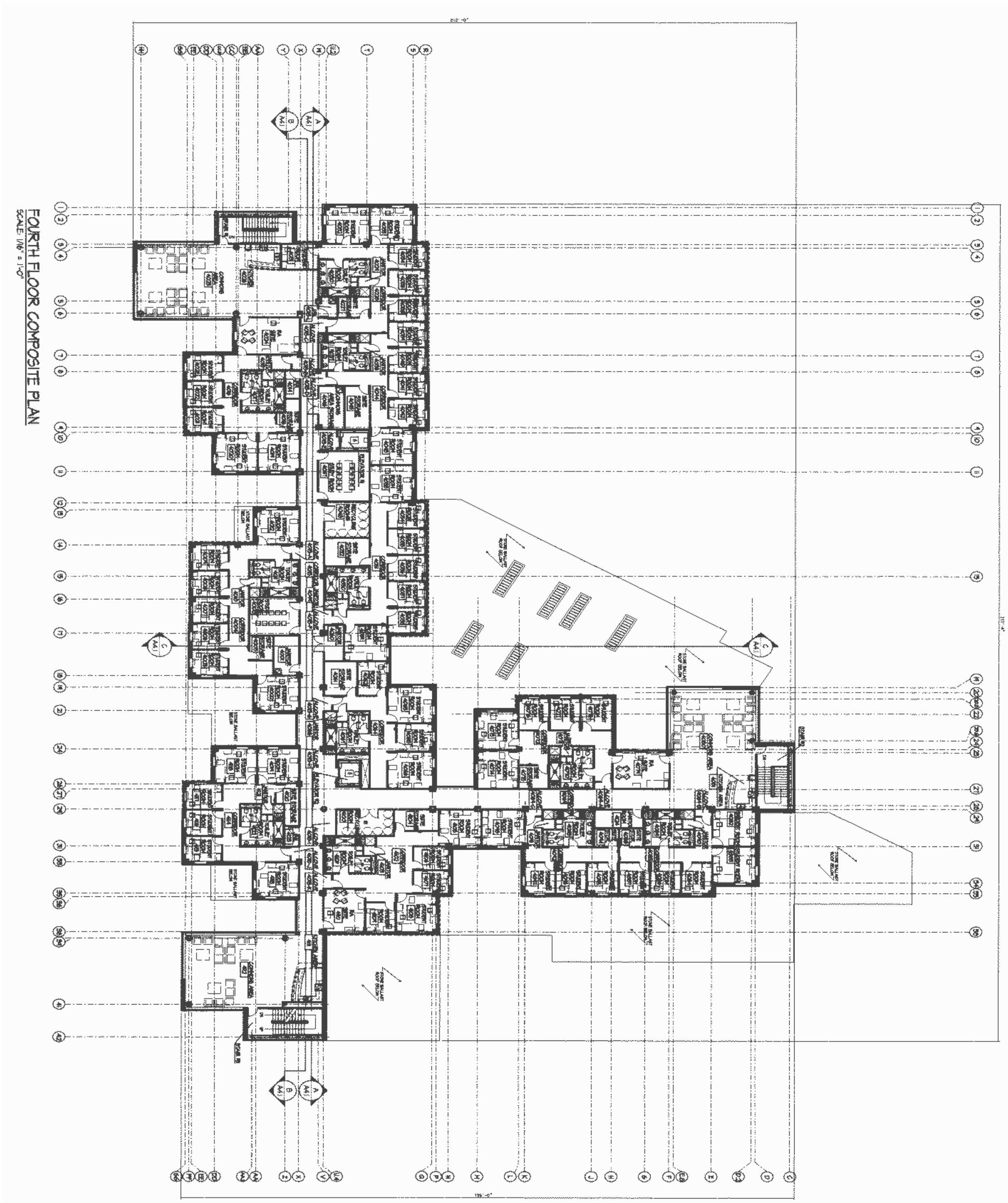


Figure 19: Upper Floors