Dormitory

Northeast USA

Technical Report 1

Rendering Courtesy of WTW Architects

Cadell G. Calkins

Structural Option

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Table of Contents

Executive Summary **3**
Building Introduction **4**
Structural Overview **5**
- Foundation 5
- Floor Construction 7
- Lateral System 8
- Materials Used 9

Design Codes and Standards **10**
Gravity Loads **11**
- Dead Load 11
- Live Load 12
- Snow Load 12
- Built-Up Column Spot Check 13
- Corridor Joist Spot Check 14
- Corridor Core Beam Spot Check 14

Lateral Loads **15**
- Wind Load 15
- Soil Load 17
- Seismic Load 17

Conclusion **19**
Appendices **20**
- Appendix A: Gravity Load Calculations 20
- Appendix B: Wind and Soil Load Calculations 29
- Appendix C: Seismic Load Calculations 32
- Appendix D: Typical Plans 41
- Appendix E: Building Sections 44

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Executive Summary

The following technical report details the structural systems of Dormitory Building A located in Northeast USA. The plans were provided through the owner and WTW Architects. The report details the gravity, wind, seismic, and soil loads that the building encounters and how it deals with them.

The gravity loads of the building are carried through floor and roof trusses to bearing walls that transfer the loads to concrete masonry units and into the ground. To do this, rammed aggregate piers had to be used to provide a greater structure to the soil.

ASCE 7-10 was used extensively in determining the lateral loads of the building. Per the MWFRS (Envelope Procedure), the windward pressure on the building was 22.43 psf on the brick walls under the hip roof and 19.37 psf on the glass walls of the central core. Because part of the building is underground, a soil load determination was made and at the deepest portion underground, a 495 psf load must be withheld. For seismic load calculations, the Equivalent Lateral Force procedure was utilized which resulted in a Seismic Design Category A. Thus the seismic base shear was 1% of the building weight (3227 kips) or 32.27 kips. Distributing the shear to each floor, the entire overturning moment was calculated to be 830 foot kips.

To check the adequacy of the design, spot checks of certain members were calculated. For a typical floor joist in a public corridor, a 2x10 wood joist at 16" on center was found to be the most economical, also matching the design. For a typical floor beam in the central core, a W10x22 was found to be the most economical W10 shape, also matching the design. For a typical built-up column supporting the attic mechanical space, the spot check determined only (2) 2x6 wood boards needed to be used whereas the design calls for (5) 2x6 wood boards. This difference can be attributed to a false assumption that the attic mechanical live load of 60 psf included the mechanical equipment.

Building Introduction

Located in a rural Northeast United States university campus, Dormitory consists of two buildings, Building A and Building B, to be built simultaneously. These new buildings, to be built where tennis courts and a parking lot once sat, will house suite style dorm rooms in each wing with a study lounge and gathering space in the central glass core. The two buildings are nearly identical except mirrored about a North-South axis. For design analysis, only Building A will be considered. However, both buildings will be considered for sitework and cost.

Building A is a 4 story building primarily consisting of a wood frame structure sitting atop a concrete masonry foundation. For lateral load analysis, the building is considered to be a 5 story building due to the walkout basement / ground floor.

To adhere to the architecture of the surrounding university, the majority of the façade of Building A consists of face brick with a base of ground face concrete masonry units. To complement the brick and masonry units, precast window heads and sills can be seen at each suite window and maroon and gray metal panels can be seen throughout the building as well. In the central core, glass storefront walls can be seen complementing the façade of the brick wings. Traditional to the brick wings, a hip roof with asphalt shingles was used and sticking with the modern feel of the glass storefront walls, a flat roof was utilized

Figure 1: Rendering Courtesy of WTW Architects

over the central core.

Structural Overview

Dormitory Building A rests on rammed aggregate piers at a depth of about 30'. Above this, the basement rests on spread footings and a slab on grade. The primary structural system for the gravity loads in the ground floor consists of concrete masonry units and from the first floor and above, the structural system for gravity loads is wood columns and walls.

An Occupancy Class of II was used for all Importance Factors per IBC 2009. Occupancy Class II was used because the occupancy load of the building is under 5000 and it does not fall into the other categories.

Foundation

Empire Geo-Services, Inc. performed the subsurface exploration of the site. This included 8 test borings for Building A completed by SJB Services, Inc. (affiliated drilling company of Empire). The findings concluded that the first 0.5 feet below the surface was either asphalt or topsoil. Below this, fill soils were found to a depth of 2 feet in some bores and at least 22 feet in others. By use of a Standard Penetration Test, it was found that the fill soils were probably installed in an uncontrolled manner. At depths between 8.4 feet and 61.5 feet, the top of bedrock is believed to exist. Per Empire's findings and recommendations, with the given fill conditions, a slab on grade and spread foundations were not a viable option and they suggested using micro-piles or drilled piers. In addition, Empire also found that groundwater conditions do not appear to be within 15 feet of the surface.

To counter the poor soil fill conditions, rammed aggregate piers, as designed by Geopier, were installed by GeoConstructors. The piers utilized a 2 foot diameter drilled hole and the hole was compacted using 2 foot lifts. Placed on a semi-regular grid of 10 feet, the piers were drilled between 8 feet and 50 feet deep depending on bedrock and soil conditions and most were around 30 feet deep. This type of pier also compacted the surrounding soil resulting in a better structure for a slab on grade.

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Below the surface, 12” reinforced concrete masonry units were utilized on spread footings with 8” concrete masonry units above the surface up to beneath the Second Floor. On the sides where soil was to be held back, 12” Ivany blocks grout solid on spread footings were utilized below the surface and 8” Ivany blocks grout solid were used above the Ground Floor up to the First Floor with 8” concrete masonry units to continue up to the Second Floor. A detail of the Ivany block wall can be seen in Figure 2 below. The floor of the Ground Floor was a 4” concrete slab over drainage course. The floor of the First Floor consisted of a 2” concrete cover over 8” hollow core precast concrete planks. This floor was utilized to provide a 2 hour fire rating between the Ground Floor and the First Floor.

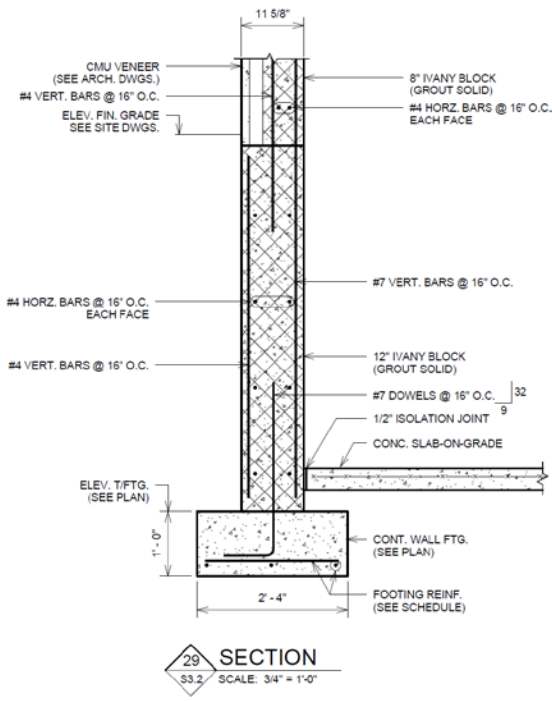

Figure 2: Typical Ivany Block Wall

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Floor Construction

Considering the First Floor as part of the foundation, the Second through Fourth Floors are nearly identical. Each suite rests on 18" deep wood floor trusses spaced at 19.2" on center. On top of the trusses consists of ¾" of Gypcrete on top of ¼" sound mat all resting on ¾" plywood sheathing. The corridors follow a similar structure, except that instead of trusses, the sheathing is supported by 2x10 Spruce-Pine-Fir or Douglas Fir wood joists at 16" on center resting on the corridor walls.

Within the central core, the floor structure consists of 1.75"x9.25" laminated veneer lumber wood joists at 16" on center topped with ¾" Gypcrete on top of ¾" plywood. For sound, 3.5" batt insulation is placed between the joists and the joists rest on W10x22 beams which in turn rest on W10x45 girders.

A typical partial floor plan can be seen below in Figure 3 with the central core outlined with a dash line.

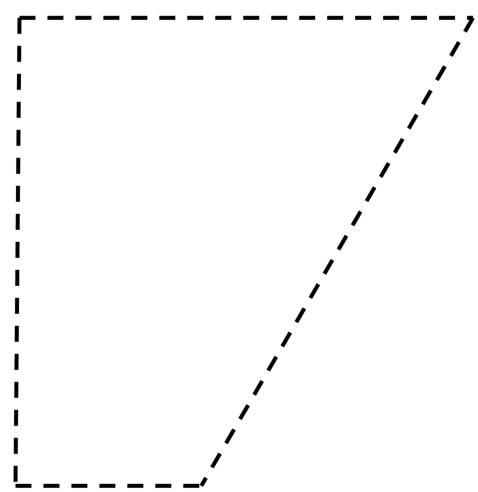

Figure 3: Typical South Wing Floor Plan

Lateral Systems

In regard to handling lateral forces, Building A is basically 3 separate buildings; South Wing, Central Core, North Wing.

In the North-South direction, the wings use shear walls that go from the first floor up to the roof. These shear walls consist of the exterior walls and the corridor walls. The exterior walls use ½" oriented strand board and 5/8" gypsum wall board per wall to resist the lateral forces, while the corridor walls use ¾" oriented strand board and two layers of 5/8" gypsum wall board per wall. In comparison, the corridor walls take more direct shear while the exterior walls help with torsional shear.

In the East-West direction, the wings use similar shear walls as the North-South direction for the exterior walls. For the interior walls, the walls that separate the suites, the lateral forces are taken up by utilizing three layers of 5/8" gypsum wall board per wall. This creates a fairly even distribution of the lateral forces through the building.

For the Central Core, the lateral forces in each direction are taken by concrete masonry unit walls that surround the stairs and elevators and that line the walls where the core connects to the wings.

Materials Used

Materials listed in the tables below come from page S2.1, General Notes and Typical Details, of the structural drawings.

Concrete	f'_c (psi)	Max Water Cement Ratio	Weight	Max Aggregate Size
Foundations	3000	0.50	Normal	1 ½”
Interior Slabs	4000	0.45	Normal	¾”
Exterior Slabs	4000	0.40	Normal	¾”

Table 1

Mortar and Grout	Use	f'_c (psi)	Standard
Mortar	Above Grade	2100	ASTM C270, Type S
Mortar	Below Grade	2900	ASTM C270, Type M
Mortar	Ivany Block	2900	ASTM C270, Type M
Grout	All Masonry	3000	ASTM C476
Leveling Grout	Concrete	5000	CE-CRD-C621

Table 2

Masonry	f'_m (psi)	Standard
Hollow Units	1500	ASTM C90, Type N-1
Solid Units	1500	ASTM C145, Type N-1
Ivany Block	3000	ASTM C270, Type M

Table 3

Steel	Standard	Grade
Wide Flange Shapes	ASTM A992	50
Other shapes, plates, bars	ASTM A36	Typical
Steel HSS Shapes	ASTM A500	B
Steel Pipes	ASTM A53, Type E	B
Bolts	ASTM A325, Type N, ¾” dia.	N/A
Anchor Rods	ASTM F1554, ¾” dia.	36
Deformed Reinforcing Bars	ASTM A615	60
Welded Wire Fabric	ASTM A185	N/A
E70 Welding Electrode	AWS D1.1	N/A

Table 4

Wood Minimums	Grade	Fb (psi)	Fv (psi)	Fc (psi)	Ft (psi)	E (psi)
Spruce-Pine-Fir	#2	875	135	1150	450	1,400,000
Douglas Fir	#2	875	135	1150	450	1,400,000

Table 5

Wood Sheathing	APA Rated	Span Rating	Exposure
Floor	Yes	40/20	1
Roof	Yes	32/16	1
Wall	Yes	N/A	1

Table 6

Design Codes and Standards

According to Sheets S2.1 and LS0-1, Dormitory was designed according to:

- Pennsylvania Uniform Construction Code
    - (2009 International Building Code and other adopted ICC codes)
    - (American Society of Civil Engineers, ASCE 7-05)
- Building Code Requirements for Reinforced Concrete (ACI 318-08)
- Building Code Requirements for Masonry Structures (ACI 530-08)
- National Design Specification for Wood Construction 2005 (NDS-05)
- American Institute of Steel Construction (13^{th} Edition – 2005)
- Design Specifications for Metal Plate Connected Wood Trusses (TPI-85)

The same codes will be used for thesis with the following changes:

- ASCE 7-10 will be used in lieu of ASCE 7-05
- AISC 14^{th} Edition will be used in lieu of AISC 13^{th} Edition

Gravity Loads

Per the requirements of this report, gravity loads, including dead, live and snow loads, were assessed and checked against the loads listed on page S2.1 of the structural drawings. These loads had to be looked up, calculated, or assumed. After determining the loads, spot checks of certain members were done and those checks can be seen in Appendix A.

Dead Loads

A summary of the dead loads for Building A can be seen in Table 7 below and a more extensive list can be found in Appendix C, as part of the determination of building weight.

Material	Weight
Typical Brick Exterior Wall @ 10' tall	281 lb per linear foot of wall
Typical CMU Exterior Wall @ 10' tall	630 lb per linear foot of wall
Interior N-S Shear Wall @ 8.5' tall	84.75 lb per linear foot of wall
Interior E-W 2x6 Shear Wall @ 8.5' tall	79.05 lb per linear foot of wall
Interior E-W 2x4 Shear Wall @ 8.5' tall	84.49 lb per linear foot of wall
Precast Concrete Plank Floor	81 lb per square foot
Typical Sheathing on Wood Truss Floor	25.7 lb per square foot
Assumed Weight of Trussed Roof	16.4 lb per square foot of floor

Table 7

Live Loads

Table 8 below details what the structural drawings state as a design live load (page S2.1) and what is called for per ASCE 7-10. For equal comparison, the design load will be used for thesis computations.

Area	Design Load	ASCE 7-10 Load
Private Rooms and Corridors Serving Them	40 PSF	40 PSF
Public Rooms and Corridors Serving Them	100 PSF	100 PSF
Lobbies and Gathering Areas	100 PSF	100 PSF
Attic Mechanical Rooms	60 PSF	40 PSF*
Attic Catwalks and Access ways	60 PSF	40 PSF
Stairs and Landings	100 PSF	100 PSF

* Assumed 40 psf because the corridors (catwalks) serving these areas is 40 psf.
Table 8

Snow Loads

According to page S2.1 of the structural drawings, the design snow load for Building A is 30 psf, the same as the ground snow load. According to calculations performed using ASCE 7-10, the design roof snow load is actually permitted to be 18.9 psf. With this snow load, the roof live load per ASCE 7-10, 20 psf, would control the design. For design considerations, 30 psf will be used because that is what is used in the original design.

For snow drift calculations, only one area needed to be considered, the raised center section of the central glass core. Per the calculations, as can be seen in Appendix A, snow drift will only extend back 8 feet from the face of the glass and up 2 feet. This means that snow drift will only occur on the lower roofs of the central core. The hip roof did not need to be considered because the pitch of the snow drift (3:12) is less than the pitch of the roof (6:12), thus the snow drift doesn't need to be considered in the design for the hip roof.

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Built-Up Column Spot Check

Holding up the air handling unit in the attic of each wing, 6 built up (5) 2x6 columns are housed within the shear walls of each floor. The center column in the North Wing, on the West side was chosen for the spot check, see Figure 4 below. With determining the loads and assuming that the 60 psf of live load included the weight of the mechanical unit, it was determined that this column would only have to carry 4.8 kips of gravity loads. This load can be carried using a (2) 2x6 built-up column. For a (5) 2x6 built-up column, a 25.7 kip load can be carried. This large difference in loads is hard to tell where the discrepancy lies. It could be that the assumption of the 60 psf live load including the equipment weight is actually false, or assuming that the column could be treated as pinned at each end could also be false. Lastly, the most probable reason is that this column carries the least weight of the six, and for ease of construction, this column was built the same as the rest of the six columns. Please see Appendix A for the actual spot check calculations.

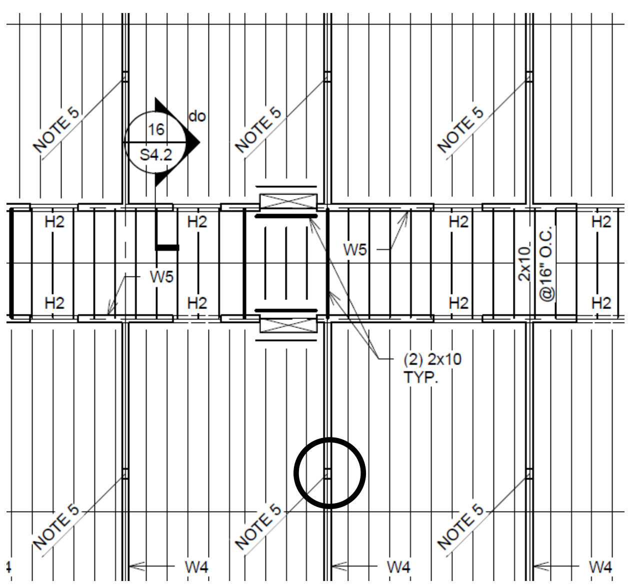

Figure 4: Spot Check Column Circled

Corridor Joist Spot Check

As can be seen by the figure above, each corridor consists of 2x10 floor joists spaced at 16" on center. After determining the weight the joist was carrying, 78 lb/ft total, assuming a private corridor, the load carried seemed way under capacity for the joist using LRFD. On a second calculation utilizing a public corridor assumption and ASD, a 2x10 was the smallest allowable size at 16" on center spacing. Since a 2x10 is the smallest size at 16" on center, it is reasonable to assume that the corridor is considered a public corridor. Please see Appendix A for calculations.

Central Core Beam Spot Check

Seen in Figure 5 below, Beam B25 (W10x22) was the third spot check. The beam reaction schedule states that the beam will have a dead load reaction of 5 kips, live load reaction of 10 kips, and a total of 15 kips. These values were not obtained during the spot check. The check assumed a 100 psf live load and pinned ends which resulted in a factored moment of 45.55 kip-ft. If the joists and the nailer plate provide lateral bracing for the beam, then the most economical beam is a W10x12. However, if the beam is not braced laterally, then the best beam is a W8x21, or in W10 shapes a W10x22. This matches up with the designed beam without considering connections.

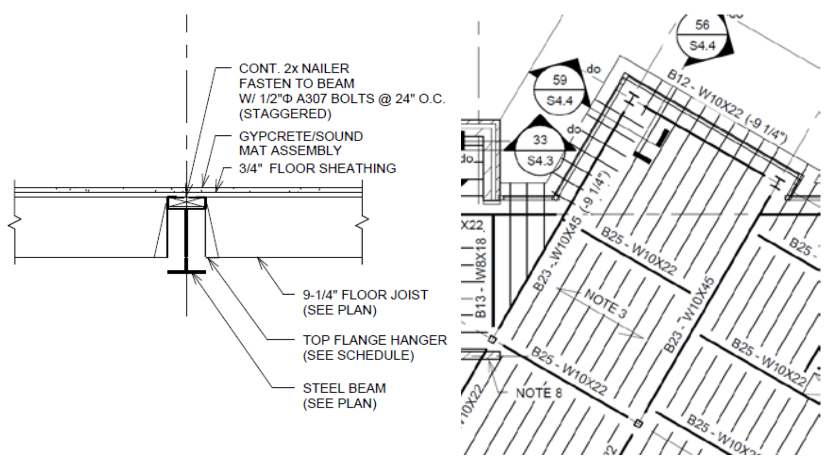

Figure 5: Beam B25 Plan and Detail

Lateral Loads

Lateral loads for Building A consist of wind loads, soil loads, and seismic loads according to ASCE 7-10. Further design considerations will be needed to design for the wind force against a wall, the roof uplift and soil pressures against the basement walls.

Wind Loads

Because Building A is less than 60 feet tall, (60 feet tall was assumed in lieu of 58 feet) the Main Wind Force Resisting System (Envelope Procedure) can be used for wind load analysis. According to ASCE 7-10, the basic wind speed for Northeast USA is 105 mph, however the structural drawings state a 90 mph design wind speed. Also, because the building is located on the lower half of a hill, the terrain factor does not come into play.

In Figures 6 and 7 below, notice that for the MWFRS (Envelope Procedure) the windward pressure is considered constant throughout the height of the building. This is a special case that is only true for the Envelope Procedure for a building this high. Please see Appendix B for wind load calculations.

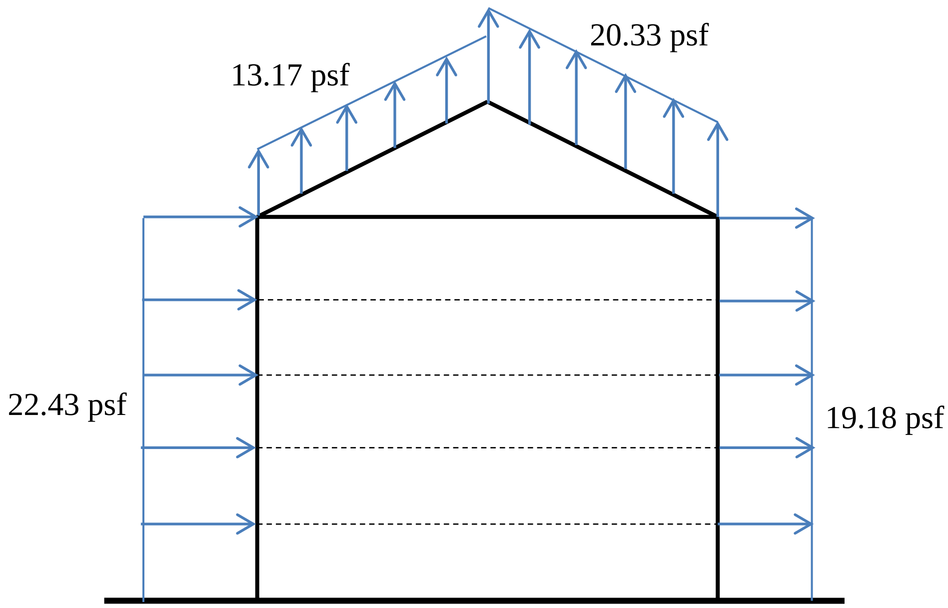

Figure 6: N-S and E-W wind pressures for the hip roofs of the wings

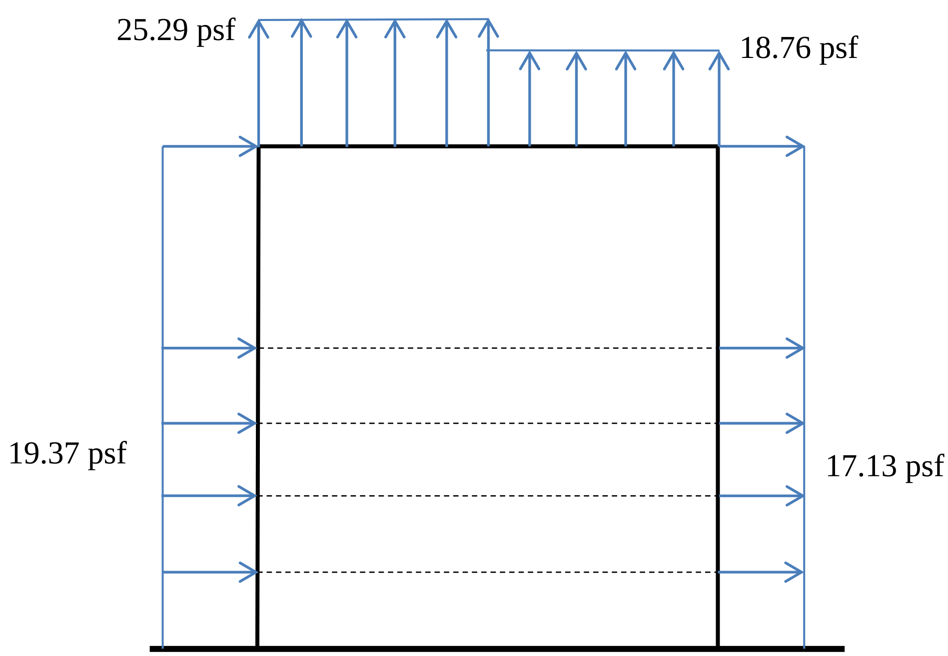

Figure 7: N-S and E-W wind pressures for the flat roof of the central core

Soil Loads

Due to part of Building A being underground, some soil load calculations were required. At 11 feet beneath the surface, with the worst case fill of SM soils, a pressure of 45 psf per foot of depth was required to be counteracted. At 11 feet, the water table does not come into play, so hydrostatic pressure on the slab and basement walls was not an issue. At the base of the basement wall, a pressure of 495 psf was to be counteracted. Please see Appendix B for soil load calculations.

Seismic Loads

Located in Site Class C, per the geotechnical report, Building A was seismically analyzed using the Equivalent Lateral Force Procedure. Per Table 9, every source found a different value for S_{DS} and S_{D1}.

Source	S_{DS}	S_{D1}
ASCE 7-10	0.10g	0.052g
USGS Online	0.094g	0.060g
S2.1 Drawing	0.119g	0.058g
Geotechnical Report	0.121g	0.059g

Table 9

However, because the values for S_{DS} are below 0.167g and S_{D1} are below 0.067g, Building A falls into the Seismic Design Category A. This amounts to Cs being 0.01.

The building weight was calculated in Appendix C assuming that the parts of the building that needed to be restrained from movement included the hollow core concrete planks and above. This resulted in a net weight of the building at 3227 kips with a base shear of 32.27 kips and an overturning moment of 830 ft-kips. According to the design, the base shear is 30 kips. The difference could come from an overestimate of weight or an improper assumption that the hollow core concrete planks are part of the building weight.

Figure 8 below shows the vertical distribution of seismic forces at each level. Please see Appendix C for building weight calculations and seismic load calculations.

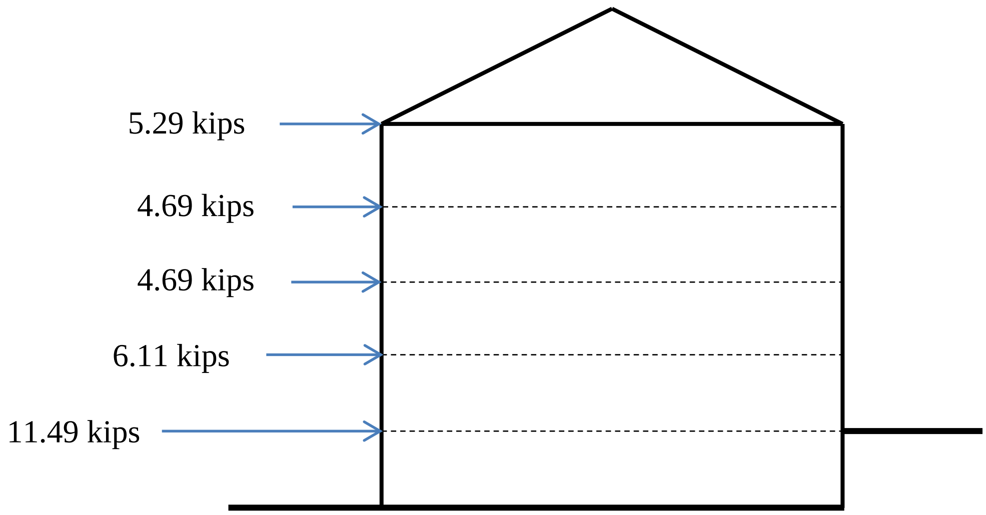

Figure 8: Seismic Story Shear Loads

Conclusion

Per the requirements of this assignment, the existing structural conditions of Dormitory Building A were examined. Each structural system was examined for the loads it had to carry and spot checks of certain load carrying members were examined.

Gravity loads were determined through the use of ASCE 7-10 and found to reasonably match the design loads, except for the mechanical attic. The dead loads had to be assumed in some cases utilizing a few typical wall types and an assumed weight per square foot for the roofing structure.

Lateral loads were also determined using ASCE 7-10, of which wind loads were determined by use of the Main Wind Force Resisting System (Envelope Procedure). This method resulted in an equal wind pressure distribution on the windward face of the building. Seismic loads were determined using the Equivalent Lateral Force procedure which resulted in a Seismic Design Category A where the base shear was equivalent to 1% of the building weight. Some differences were found in lateral loads between design and thesis, but these can be explained with the assumptions used and the different codes utilized.

Spot checks were performed on a corridor wood joist, central core steel beam and a column supporting the attic mechanical space. After revising the assumptions, the corridor wood joist specified in the plans was the most economical for a public corridor loading. Likewise, for a public loading, the steel beam proved to be the most economical if the design adheres to W10 shapes for the core. Lastly, the column supporting the attic mechanical space did not appear to be the most economical solution. It appeared to be way over designed and this over design could be due to a lax assumption on the weight of the mechanical equipment.

Overall, Dormitory Building A seems properly designed for the loads stated within the drawings.

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Appendix A – Gravity Load Calculations

Snow Loads | Tech 1 | Page 1 of 1

Assume
- Unbalanced Snow Load not considered due to roof pitch of 6:12 < 7:12
- Asphalt is not a slippery surface

$p_f = 0.7 C_e C_t I_s p_g \cdot (C_s)$

C_e = (Surface Roughness B) 0.9 (Fully Exposed)

$C_t = 1.0$

$I = 1.0$ (Snow Importance Factor = 1.0 per ASCE)

$p_g = 30$ psf

$p_f = 0.7 \cdot (.9)(1.0)(1.0)(30) = \boxed{18.9 \text{ psf}}$ < Design = 30 psf

Leeward Drift

h_d from 20' upper roof length = 1.5'

Windward Drift

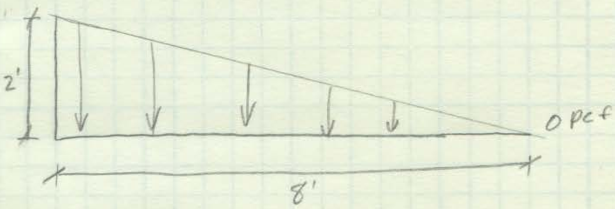
h_d from 60' (approximate from slope roof and flat roof) = 2.7' · 3/4 = 2'

Use 2' for h_d < h_c

Drift width = $4 \cdot h_d$ = 8'

$\gamma = 0.13(30) + 14 = 17.9$ pcf < 30 pcf Good

$h_d \gamma = 17.9 \cdot 2 = 35.8$ pcf



$h_b = p_s / \gamma = 18.9 / 17.9 = 1.06'$

Snow will only drift on the lower flat roofs.

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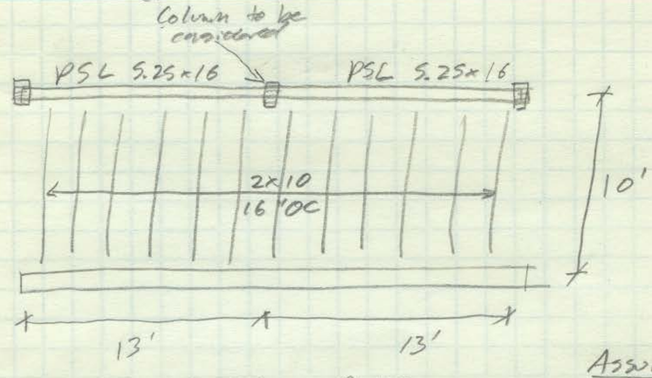
Gravity Check (5) 2x6 Column | Tech 1 | Page 1 of 3



Mechanical Attic Platform
Partial Framing

Height = 10'

Columns run from foundation to attic, but are discontinuous at each floor

Assume
- Attic Live Load (60 psf) takes into account equipment weight
- Nailing is adequate
- Effective Length in both directions = 10'

Load From Materials Chart:

Sheathing = 2.2 psf
MEP = 4 psf
2x10@16" = 2.175 psf
8.4 psf · 13' · 5' = 546 lb + 26.3 · 13' = 888 lb

Live Load = 60 psf (not reducing due to assumption of equipment weight)
60 · 13 · 5 = 3900 lb

ASD ⇒ 3900 + 888 = 4788 lb ⇒ 4800 lb = 4.8 kips D+L

$F_c^* = C_D \cdot C_F \cdot (F_b) = 1.0 \cdot (1400) \cdot 1.1 = 1540 \text{ psi}$

$F_{cE} = \dfrac{.822 \cdot E'_{min}}{(l_e/d)^2}$ $\quad \dfrac{l_e}{d} = \dfrac{10 \cdot 12}{5.5} = 22 < 50$ Good

$\dfrac{.822 \cdot .58\text{E}6}{22^2} = 1192$ $\quad K_f = 1.0$, $c = .8$

$$C_p = 1.0\left[\frac{1+\left(\frac{1192}{1540}\right)}{2\cdot(.8)} - \sqrt{\left[\frac{1+\left(\frac{1192}{1540}\right)}{2\cdot(.8)}\right]^2 - \frac{1192/1540}{0.8}}\;\right] = 0.5673$$

$F_c = .5673 \cdot 1540 = 873.7 \text{ psi}$

$P = F_c \cdot A = 873.7 \cdot (1.5 \cdot 5.5) \cdot 5 = 36.0$ kips > 4.8k Good

$\dfrac{4.5 \cdot 1000}{(873.7)(1\frac{1}{2} \cdot 5.5)} = 0.62$ ⇒ 1 plies ⇒ Causes K_f & F_{cE} to change

Gravity Check (5) 2x6 Column | Tech 1 | Page 2 of 3

Try 4 plies ⇒ F_{CE} & K_f equal to 5 plies

$F_c = 873.7$ psi

$P = 873.7 \cdot 4 \cdot (1.5 \cdot 5.5) = 28.8k > 4.8k$ Good

Try 1 ply

$\frac{l_e}{d} = \frac{10 \cdot 12}{1.5} = 80 > 50$ No Good

Try 2 plies

$\frac{l_e}{d} = \frac{10 \cdot 12}{3} = 40 < 50$ Good

$F_{ce} = \frac{.822 \cdot .58E6}{40^2} = 298$ $\frac{F_{ce}}{F_c^*} = \frac{298}{1540} = .1935$

$C_p = .6\left[\frac{1 + .1935}{2 \cdot .8} \sqrt{\left(\frac{1.1935}{1.6}\right)^2 - \frac{.1935}{.8}}\right] = .4945$

$F_c = .4945 \cdot 1540 = 761$ psi

$P = 761 \cdot 2 \cdot 1.5 \cdot 5.5 = 12.556$ kips $> 4.8k$ Good

$\boxed{\text{2 plies works for my assumptions}}$

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Gravity Check (5) 2x6 Column | Tech 1 | Page 3 of 3

For SPF

$F_c^* = 1.0 \cdot 1150 \cdot 1.1 = 1265$ psi

$F_{cE} = \dfrac{.822 \cdot .5156}{22^2} = 866$

$$C_p = 1.0\left[\frac{1+\frac{866}{1265}}{1.6} \sqrt{\left(\frac{1+\frac{866}{1265}}{1.6}\right)^2 - \frac{866/1265}{.8}}\right] = .529$$

$F_c = .529 \cdot 1265 = 669$ psi

$P = F_c A = 669 \cdot 5.5 \cdot 1.5 \cdot 5 = 27.6\ k > 4.8\ k$ Good

---

F_{cE} @ 11' height $= 728$

$C_p = .575$

$F_c = 623 \Rightarrow P = 25.7\ k > 4.8$ Good

Gravity Check B25 | Tech 1 | Page 1 of 1

Typical Beam
W10x22

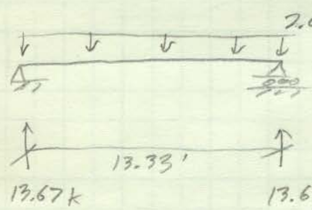
length = 13'4" = 13.33'

Sheathing = 2.2 psf
Gypcrete (3/4) = 7.2 psf
Carpet = 2 psf
MEP = 4 psf
Ceiling = 1 psf
16.4 psf
+ 3.525
19.9 psf ⇒ 20 psf

LVL 1.75x9.25 @ 16"OC = 4.7 · 12/16 =
= 3.525

Tributary Area = 11' wide

20 · 11 = 220 plf + 22 = 242 plf D



Live = 100 psf · 11' = 1100 plf

1.2D + 1.6L = 1.2(242) + 1.6(1100)
= 2050

L_b = 2'0" per Section 37/34.3

Assume Laterally Braced

$M = \frac{wl^2}{8} = 45.55$ k-ft

$\Delta_L = \frac{5wL^4}{384EI} = \frac{5 \cdot 1100 \cdot 13.33^4 \cdot 1728}{384 \cdot 29E6 \cdot I} = \frac{l}{360} = \frac{13.33 \cdot 12}{360}$

$I = 60.64$ in^4 ← Controls

$\Delta_{L+D} = \frac{5wL^4}{384EI} = \frac{5 \cdot 1342 \cdot 13.33^4}{384 \cdot 29E6 \cdot I} = \frac{l}{240} = I = 49.32$ in^4

⇒ W12x14 = 88.6 > 60.64 Good, does not meet max uniform load tables

$\phi M_n = \phi F_y Z_x = 45.55 = .9 \cdot 50 \cdot Z_x$ $Z_x = 1.012$

Max Uniform Load: 2.05 k/ft

Using Table 3-10: M = 45.55, L_b = 2' ⇒ W10x12

If the unbraced length is 13'4", then either:

	M	V	I
W8x21	51 k-ft	62.1 k	75.3
or			
W10x22	62 k-ft	73.4 k	118
	Good	Good	Good
Need	45.55	13.67	60.64

Both beams work, but if sticking with W10 shapes then

W10x22 as per the design.

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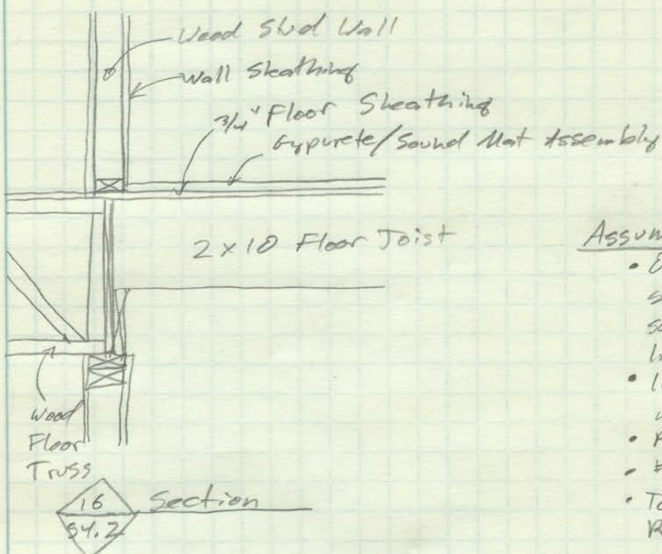
Gravity Check → | Tech 1 | Page 1 of 3

Corridor 2x10 Typical Floor



Assume
- Only loads on 2x10 are selfweight, 3/4" sheathing, sound mat assembly and live load.
- 16" spacing as is typical in wood construction.
- Pinned Connections
- #2 Douglas Fir (North)
- Too small Area for Load Reduction

Span = 7'2" = 7.1667'

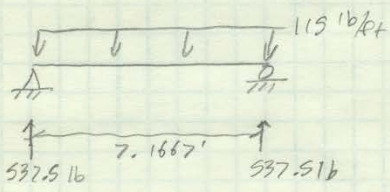
Load from Materials Chart:

Sheathing = 2.2 psf
Gypcrete (3/4") = 7.2 psf
Carpet = 2 psf
MEP = 4 psf
Acoustic Ceiling = 1 psf
16.2 psf @ 16" oc = 16.2 · 16/12 = 21.6 lb/ft + SW

Selfweight (SW) Assumed as a 2x10 Douglas Fir @ 30 lb/ft³ = 2.9 lb/ft

Total Dead = 21.6 + 2.9 = 24.5 lb/ft
Live = 40 psf @ 16"oc = 53.33 lb/ft (private corridor)

LRFD
W = 1.2D + 1.6L = 1.2(24.5) + 1.6(53.33) = 114.7 = 115 lb/ft
24.5 + 53.33 = 78 lb/ft



Maximum Shear = 537.5 lb @ supports

Maximum Moment = 738.3 lb-ft @ center $\left(\frac{wL^2}{8}\right)$

Gravity Check → | Tech 1 | Page 2 of 3

Corridor 2x10 Typical Floor

Bending

$$f_b = \frac{6M}{bd^2} = \frac{6 \cdot 738.3}{1.5 \cdot 9.25^2} \cdot 1728 = 59642 \text{ lb/ft}^2 = 414.2 \text{ psi}$$

$$F_b' = F_b \cdot K_F \cdot \phi_b \cdot \lambda \cdot C_r \cdot C_F \cdot C_L$$

$$= 850 \text{ psi} \cdot (2.16/\phi_b) \cdot (\phi_b) \cdot (0.8) \cdot (1.15) \cdot (1.1) \cdot (1.0) = 1858 \text{ psi}$$

$$1858 > 414.2 \quad \text{Good}$$

Try a 2x8 assuming same dead load

$$F_b' = 850 \cdot 2.16 \cdot .8 \cdot 1.15 \cdot 1.2 \cdot 1.0 = 2027 \text{ psi}$$

$$f_b = \frac{6 \cdot 738.3}{1.5 \cdot 7.25^2} \cdot \frac{1728}{144} = 674.2 \text{ psi}$$

ok

Try a 2x6 assume same dead load

$$F_b' = 850 \cdot 2.16 \cdot .8 \cdot 1.15 \cdot 1.3 \cdot 1.0 = 2196 \text{ psi}$$

$$f_b = \frac{6 \cdot 738.3}{1.5 \cdot 5.5^2} \cdot 12 = 1172$$

Good

Shear 2x10

$$f_v = \frac{3V}{2db} = \frac{3 \cdot 537.5}{2 \cdot 1.5 \cdot 9.25} = 58.1 \text{ lb/in}^2$$

Good

$$F_v = F_v' \cdot K_F \cdot \phi_v \cdot \lambda = 180 \cdot 2.16 \cdot .8 = 311 \text{ lb/in}^2$$

Try a 2x6

$$f_v = \frac{3 \cdot 537.5}{2 \cdot 1.5 \cdot 5.5} = 97.7 \text{ lb/in}^2$$

Good

$$F_v = 180 \cdot 2.16 \cdot .8 = 311 \text{ lb/in}^2$$

Deflection

$$2\times10 \Rightarrow I = \frac{bd^3}{12} = \frac{1.5 \cdot 9.25^3}{12} = 99$$

$$\Delta = \frac{5wL^4}{384EI} = \cancel{\frac{\ell}{360}} = \frac{5 \cdot 78 \cdot 7.1667^4 \cdot 1728}{384 \cdot 1.656 \cdot 99} = .029 \text{ in}$$

should be 53, but if it works now, 53 will work & it will also meet $\frac{\ell}{240}$ requirement

ok

$$\frac{\ell}{360} = \frac{7.1667 \cdot 12}{360} = .239 \text{ in}$$

Try 2x6

$$I = \frac{1.5 \cdot 5.5^3}{12} = 20.8$$

ok

$$\Delta = \frac{5 \cdot 78 \cdot 7.1667^4 \cdot 1728}{384 \cdot 1.656 \cdot 20.8} = .139 \text{ in}$$

$\boxed{2\times6 \text{ works}}$

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Gravity Check → | Tech 1 | Page 3 of 3

Corridor 2x10

Assume Live Load = 100 psf = 133 plf

ASD

$W = D + L = 133 + 24.5 = 157.5 \Rightarrow 160$ plf

Max Shear = 573 k

Max Moment = 1027 k

$f_b = \frac{6M}{bd^2} = \frac{6 \cdot 1027}{1.5 \cdot 9.25^2} \cdot \frac{1728}{12 in} = 576$ psi

~~Per previous calculations, 1858 > 576 Good~~

$f_v = \frac{3V}{2db} = \frac{3 \cdot 573}{2 \cdot 1.5 \cdot 9.25} = 61.9$ psi

$F_b' = 850 \cdot 1.0 \cdot 1.15 \cdot 1.1 \cdot 1.0 = \boxed{1075 > 576 \text{ Good}}$

Try 2x6 $F_b' = 850 \cdot 1.0 \cdot 1.15 \cdot 1.3 = 1271$

$f_b = \frac{6 \cdot 1027 \cdot 12}{1.5 \cdot 5.5^2} = 1629 > 576$ No

$F_v' = 180 \cdot 1.0 = 180 > \boxed{61.9 \text{ Good}}$

Deflection

$\Delta_L = \frac{5 \cdot 133 \cdot 7.1667^4 \cdot 1728}{384 \cdot 1.6E6 \cdot 99} = .050$ in $\frac{l}{360} = \frac{7.1667 \cdot 12}{360} = .239$ Good

$\Delta_T = \frac{5 \cdot 160 \cdot 7.1667^4 \cdot 1728}{384 \cdot 1.6E6 \cdot 99} = 0.060$ in $\frac{l}{240} = \frac{7.1667 \cdot 12}{240} = .358$ Good

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Appendix B – Wind and Soil Load Calculations

Basic Wind Speed=	105
K_d=	0.85
K_{zt}=	1.0

Hip Roof (26.6°)

	K_z	q_z	Gc_{pi} (+)	Gc_{pi} (-)	Gc_{pf}	**P (psf)**	
1	0.85	20.39184	0.55	-0.55	0.5498	0.00	22.43
2	0.85	20.39184	0.55	-0.55	-0.096	-13.17	9.26
3	0.85	20.39184	0.55	-0.55	-0.447	-20.33	2.10
4	0.85	20.39184	0.55	-0.55	-0.3904	-19.18	3.25
1E	0.85	20.39184	0.55	-0.55	0.7274	3.62	26.05
2E	0.85	20.39184	0.55	-0.55	-0.1856	-15.00	7.43
3E	0.85	20.39184	0.55	-0.55	-0.5844	-23.13	-0.70
4E	0.85	20.39184	0.55	-0.55	-0.5344	-22.11	0.32

Flat Roof

	K_z	q_z	Gc_{pi} (+)	Gc_{pi} (-)	Gc_{pf}	**P (psf)**	
1	0.85	20.39184	0.55	-0.55	0.4	-3.06	19.37
2	0.85	20.39184	0.55	-0.55	-0.69	-25.29	-2.85
3	0.85	20.39184	0.55	-0.55	-0.37	-18.76	3.67
4	0.85	20.39184	0.55	-0.55	-0.29	-17.13	5.30
1E	0.85	20.39184	0.55	-0.55	0.61	1.22	23.65
2E	0.85	20.39184	0.55	-0.55	-1.07	-33.03	-10.60
3E	0.85	20.39184	0.55	-0.55	-0.53	-22.02	0.41
4E	0.85	20.39184	0.55	-0.55	-0.43	-19.98	2.45

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Wind Load | Tech 1 | Page 1 of 1

Use MWFRS (Envelope Procedure)
Low Rise Building: Height = 58' < 60' Assume 60'

Importance Factor: (Risk Category = II) 1.0

Basic Wind Speed = 105 mph [90 mph design]

$K_d = 0.85$

Exposure Category = ~~C~~ B [B design]

$K_{zt} = 1.0$ (lower half of hill)

Enclosure Classification: Partially Enclosed

$GC_{pi} = \pm 0.55$

K_z =	K_z	q_z
0 – 15 ft	.85	20.39
20	.90	21.59
25	.94	22.55
30	.98	23.51
40	1.04	24.95
50	1.09	26.15
60	1.13	27.11

See Spreadsheet

Flat Roof

Surface	Case A
1	0.40
2	–.69
3	–.37
4	–.29
1E	0.61
2E	–1.07
3E	–.53
4E	–.43

$p = q_h[GC_{pf} - GC_{pi}]$ lb/ft²

$q_h = q_z$ at mean height $h = z$

~~$K_z = 1.13$~~ ~~$q_z = 27.11$~~
$GC_{pi} = \pm 0.55$ $GC_{pf} = 0.4$
(location 1)

~~$p = 27.11[0.40 - (0.55)] =$~~ $\boxed{-4.07 \text{ psf}}$

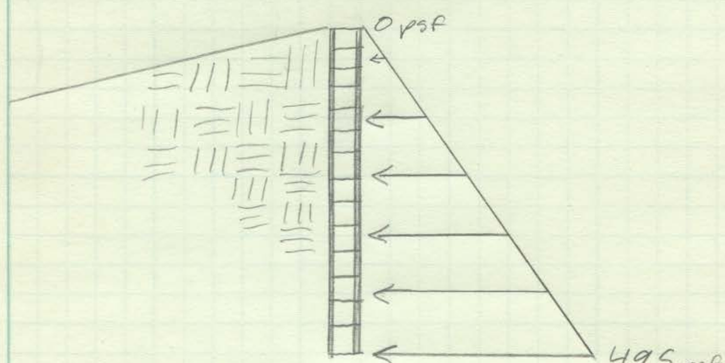
~~$p = 27.11[0.40 - (-0.55)] =$~~ $\boxed{25.75 \text{ psf}}$

Soil Forces | Tech 1 | Page 1 of 1

According to the geotechnical report, fill soils (GW, GP, GM, SW, SP or SM) can be used behind basement walls.

Per ASCE 7-10, the worst condition (SM) yields a design lateral load of 45 psf per foot of depth

At 11'0" of basement depth: 45 · 11' = 495 psf @11'



Appendix C – Seismic Load Calculations

Materials and Weights:			
Floor		**Weight (psf)**	**Weight Source**
	3/4" Plywood Flooring	2.2	APA
	1" Gyp-Crete Underlay	9.6	Maxxon
	3/4" Gyp-Crete Underlay	7.2	Maxxon
	1/4" Sound Mat	0.14	Maxxon
	2x10 Floor Joists per Spec @ 16" OC (Assume 30 lb/ft^3)	2.175	NDS
	1.75x9.25 LVL 1.9E Joists @ 16" OC (4.7 lb per foot)	3.525	Trus Joist
	3.5" Batt Insulation	0.14	BoiseCascade
	5/8" GWB Ceiling (assume 0.1 psf for resilient channel)	2.4	Georgia Pacific
	Carpet	2	Assumed
	MEP and Miscellaneous	4	Assumed
	Acoustical Ceiling	1	AISC
	Precast Concrete Planks	56	Flexicore
	2" Concrete Topping	25	ACI 318-08
Floor Truss		**Weight (lb)**	**Weight Source**
	A1 Floor Truss	164.1	Montgomery
	A1G Floor Truss	197.3	Montgomery
	A1P Floor Truss	175.1	Montgomery
	A1PG Floor Truss	207.1	Montgomery
	A1X Floor Truss	175.1	Montgomery
	A1XG Floor Truss	207.1	Montgomery
	A2 Floor Truss	154.7	Montgomery
	B1 Floor Truss	147.2	Montgomery
	B1G Floor Truss	182.8	Montgomery
	B1P Floor Truss	158.2	Montgomery
	B1X Floor Truss	158.2	Montgomery
	B1Z Floor Truss	180.4	Montgomery
	B2 Floor Truss	127.6	Montgomery
	B5 Floor Truss	26.9	Montgomery
	C1 Floor truss	194.1	Montgomery
	C1G Floor Truss	275.6	Montgomery
	C1X Floor Truss	282.9	Montgomery

	C2 Floor Truss	177.8	Montgomery
	C3 Floor Truss	193	Montgomery
	C4 Floor Truss	57.5	Montgomery
	C5 Floor Truss	97	Montgomery
	C6 Floor Truss	38.7	Montgomery
	C7 Floor Truss	280.6	Montgomery
	C8 Floor Truss	47.3	Montgomery
	C9 Floor Truss	143.8	Montgomery
	C10 Floor Truss	16.6	Montgomery
	D1 Floor Truss	131.2	Montgomery
	D2 Floor Truss	129.5	Montgomery
	E1 Floor Truss	127.6	Montgomery
	E1G Floor Truss	197.1	Montgomery
	E1X Floor Truss	137.1	Montgomery
	E1XG Floor Truss	204.6	Montgomery
	E2 Floor Truss	99.1	Montgomery
	E3 Floor Truss	16.9	Montgomery
	H1 Floor Truss	52.8	Montgomery
	H2 Floor Truss	51.3	Montgomery
	H3 Floor Truss	61.6	Montgomery
	H4 Floor Truss	63.9	Montgomery
	H5 Floor Truss	34.5	Montgomery
	H6 Floor Truss	16.8	Montgomery
	H7 Floor Truss	23.7	Montgomery
	J1 Floor Truss	19.3	Montgomery
	J2 Floor Truss	21.8	Montgomery
	J3 Floor Truss	34.1	Montgomery
	J4 Floor Truss	35.9	Montgomery
	J5 Floor Truss	21.1	Montgomery
	J6 Floor Truss	20.4	Montgomery

Steel Beams		Weight (lb/ft)	Length (ft)	Weight (lb)	Weight Source
	W8x18	18	16	288	AISC
	W8x15	15	8	120	AISC
	W10x30	30		0	AISC
	W12x87	87		0	AISC
	W10x45	45		0	AISC
	W10x26	26		0	AISC
	MC12x45	45		0	AISC
	B01 W12x45	45		0	AISC
	B02 W10x45	45		0	AISC
	B03 Not on Building A				
	B04 W10x33	33		0	AISC
	B05 W21x68	68		0	AISC
	B06 W10x33	33		0	AISC
	B07 W10x33	33		0	AISC
	B08 W10x45	45		0	AISC
	B09 W10x33	33		0	AISC
	B10 W21x83	83		0	AISC
	B11 W10x30	30		0	AISC
	B12 W10x22	22	13.33	293.26	AISC
	B13 W8x18	18	9	162	AISC
	B14 W10x22	22	12.66	278.52	AISC
	B15 W14x43	43		0	AISC
	B16 W14x132	132		0	AISC
	B17 W14x174	174		0	AISC
	B18 W14x193	193		0	AISC
	B19 W14x22	22		0	AISC
	B20 W14x53	53		0	AISC
	B21 W14x132	132		0	AISC
	B22 W14x145	145		0	AISC
	B23 W10x45	45	22	990	AISC
	B24 Not On Building A				
	B25 W10x22	22	13.33	293.26	AISC

Wood Beams		Weight (lb)	Weight Source
	5.25x18 PSL 2.0E	29.5	Trus Joist
	5.25x16 PSL 2.0E	26.3	Trus Joist
	5.25x14 PSL 2.0E	15.3	Trus Joist
	1.75x9.25 LVL 1.9E	4.7	Trus Joist
	1.75x18 LVL 1.9E	9.2	Trus Joist
	2x10 per Spec (Assume 30 lb/ft^3)	2.9	NDS
Columns		**Weight (lb)**	**Weight Source**
	3.5" Dia. Schedule 40 Pipe Column	9.12	AISC
	(5) 2x6 Wood Stud Post (Assume 30 lb/ft^3)	8.6	NDS
Stud Walls		**Weight (psf)**	**Weight Source**
	2x4 Wood Studs @16" OC (Assume 35 lb/ft^3)*	0.975	NDS
	2x4 Wood Studs @24" OC (Assume 35 lb/ft^3)*	0.65	NDS
	2x6 Wood Studs @16" OC (Assume 35 lb/ft^3)*	1.5	NDS
	3" Batt Insulation	0.12	BoiseCascade
	2x4 Steel Studs @ 24" OC	1.5	AISC
	5/8" GWB (assume 0.1 psf for resilient channel)	2.4	Georgia Pacific
	3/4" OSB	2.55	Georgia Pacific
	MEP and Miscellaneous	1	Assumed
	* 35 lb/ft3 was assumed to take into account headers		
Wall Types		**Weight (plf)**	**Weight Source**
	WL @ 8.5 ft tall	84.745	Plans
	WM @ 8.5 ft tall	79.05	Plans
	WN @ 8.5 ft tall	84.49	Plans

Exterior Walls		Weight (psf)	Weight Source
	Brick	22.5	Glen Gery
	4" Ground Face CMU	28	Masonry Advisory
	Sheathing	2.2	APA
	2x4 Wood Studs @16" OC (Assume 35 lb/ft^3)*	0.975	NDS
	5/8" GWB Ceiling	2.4	Georgia Pacific
	8" CMU	34	Masonry Advisory
	Brick Wall Assembly @10 ft tall	281	Plans
	CMU Wall Assembly @10 ft tall	630	Plans
Roof		**Weight (lb)**	**Weight Source**
	Trusses: 2' OC Assume All Trusses are Similar in density		
	Sample Truss: 275 lb @23' Long = 12.0 lb/ft		
	=6.0 lb/ft^2 of floor area	6	
	R-30 Blown in Insulation	1	Insulation Dr.
	5/8" GWB	2.4	Georgia Pacific
	3/4" Wood Sheathing*	3	AISC
	Asphalt Shingles*	3	AISC
	Membrane*	1	AISC
	*Roof Area assumed to equal 1.2 x Floor Area		
	including slope, dormers and overhangs		
	Total Roof=	16.4	

Weight per Floor:					
	Member	Count	Length (ft)	Weight (lb)	Shear Force
Roof - 4th Floor		Area=	20627	sqft	
	Roof Structure			338282.8	
	Same walls as floor below			190759.75	
				529042.55	
			Total:	**529.0 kips**	5.29 kips
4th Floor - 3rd Floor		Area=	20627	sqft	
	C1	10		1941	
	A1P	3		525.3	
	A1	136		22317.6	
	A1G	18		3551.4	
	A2	30		4641	
	A1PG	3		621.3	
	A1X	4		700.4	
	A1XG	2		414.2	
	B1	137		20166.4	
	B1G	4		731.2	
	B1P	1		158.2	
	B1X	1		158.2	
	B1Z	2		360.8	
	B2	4		510.4	
	PSL 5.25x18	4	23	1407.6	
	PSL 5.25x14	4	10	612	
	PSL 5.25x14	2	17	520.2	
	C5	2		194	
	C4	1		57.5	
	C3	2		386	
	C1	24		4658.4	
	C1G	3		826.8	
	C1X	2		565.8	
	C2	12		2133.6	
	LVL 1.75x9.25	39	10	1833	
	LVL 1.75x9.25	45	12	2538	

	LVL 1.75x9.25	20	6	564	
	LVL 1.75x9.25	47	9	1988.1	
	E1	10		1276	
	E1X	3		411.3	
	E1G	3		591.3	
	E1XG	1		197.1	
	D1	11		1443.2	
	D2	13		1683.5	
	W8x18	2		32	
	W8x15	1		8	
	B12	2		586.52	
	B13	2		324	
	B14	3		835.56	
	B23	2		1980	
	B25	11		3225.86	
	Interior Walls				
	WL		200	16949	
	WM		450	35572.5	
	WN		1200	101388	
	Brick Exterior Walls		810	227610	
				469196.24	
			Total:	**469.2 kips**	4.69 kips
3rd Floor - 2nd Floor		**Area=**	**20627 sqft**		
	Assumed same as above floor			**469.2 kips**	4.69 kips
2nd Floor - 1st Floor		**Area=**	**20424 sqft**		
	Same as 3rd floor with different exterior walls				
	Floor Total Without Exterior Walls			355391.24	
	CMU Exterior Walls		810	255150	
				610541.24	
			Total:	**610.5 kips**	6.11 kips

2nd Floor - 1st Floor	**Area=**	**10084 sqft**		
	Concrete Planks		816804	
	Walls (use half of first floor walls)		332104.75	
			1148908.75	
			1148.9 kips	11.49 kips
Total Weight:			**3226.9 kips**	**32.27 kips**

Seismic Loading | Tech 1 | Page 1 of 1

Site Class → C per Geotechnical Report

Risk Category = II

Importance Factor = 1.0

Equivelant Lateral Force Procedure

$V = C_s W$

$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$

$S_{DS} = 2/3\, S_{MS}$ $\quad$ $S_{MS} = F_a S_s$

$S_{D1} = 2/3\, S_{M1}$ $\quad$ $S_{M1} = F_v S_1$

$S_s = .15g$ $\quad$ $F_a = 1.0$ $\quad$ $S_{MS} = .15g$ $\quad$ $\cdot 2/3 = S_{DS} = .10g$

$S_1 = .06g$ $\quad$ $F_v = 1.3$ $\quad$ $S_{M1} = .079g$ $\quad$ $\cdot 2/3 = S_{D1} = .052g$

According to USGS Online:

$S_s = 0.117g$ $\quad$ $S_{MS} = 0.14g$ $\quad$ $S_{DS} = 0.094g < 0.167g$

$S_1 = 0.053g$ $\quad$ $S_{M1} = 0.09g$ $\quad$ $S_{D1} = 0.060g < 0.067g$

Seismic Design Category = A

Seismic Force Resisting System: Masonry Shear Walls and GWB on wood studs (light frame)

R = 2 for Masonry
R = 2.5 for light frame

$C_s = 0.01$ per Section 1.4

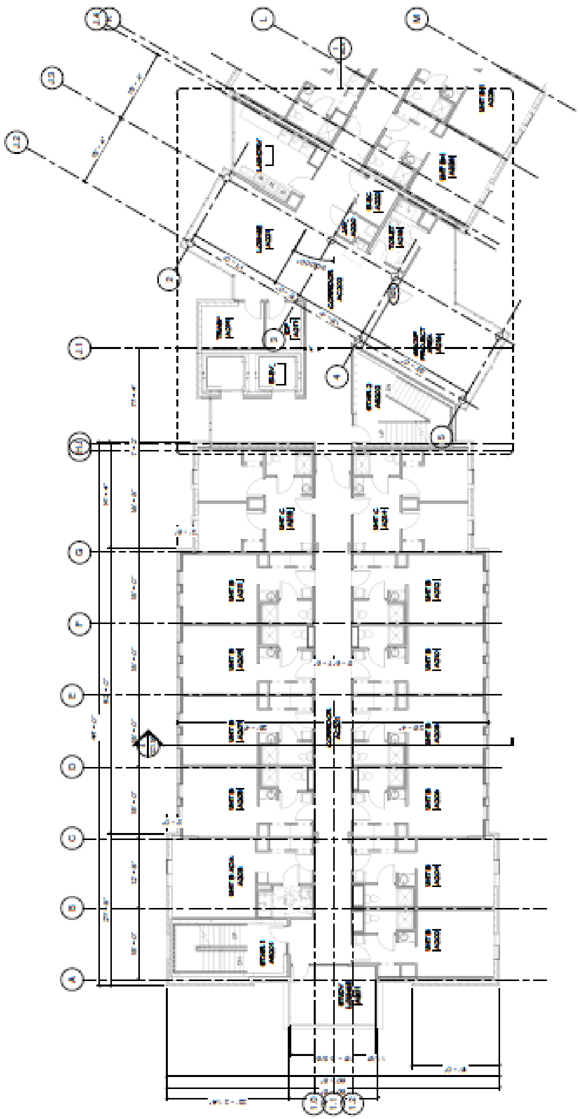
Base Shear = $W \cdot 0.01$ (Equation 1.4-1)

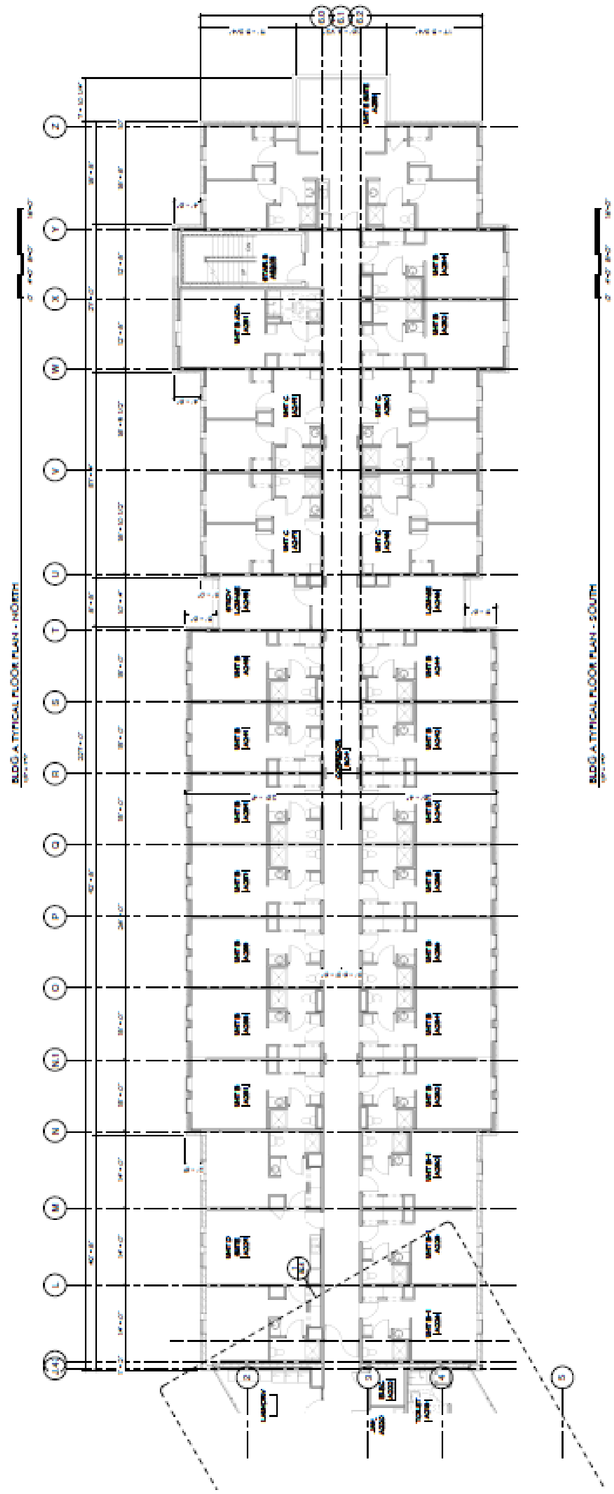
$= 2078 \cdot 0.01 = \boxed{20.78 \text{ Kips}}$ design = 30 kips

Distribution Calculated in Spreadsheet.

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

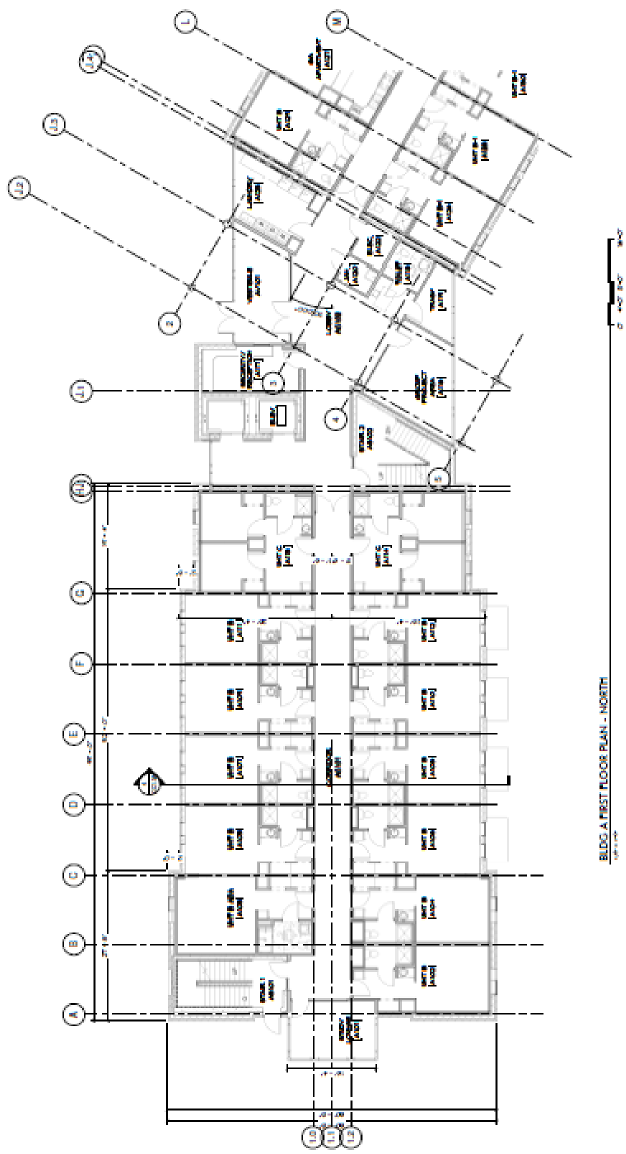
Appendix D – Typical Floor Plans

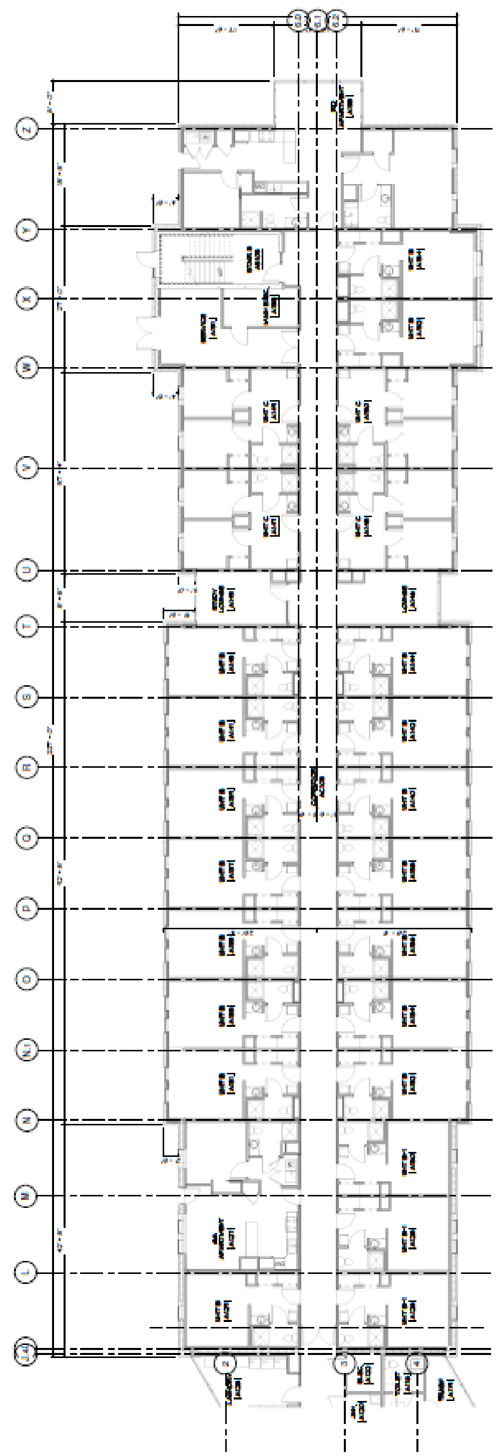




Typical Floor Plan
Courtesy of WTW Architects

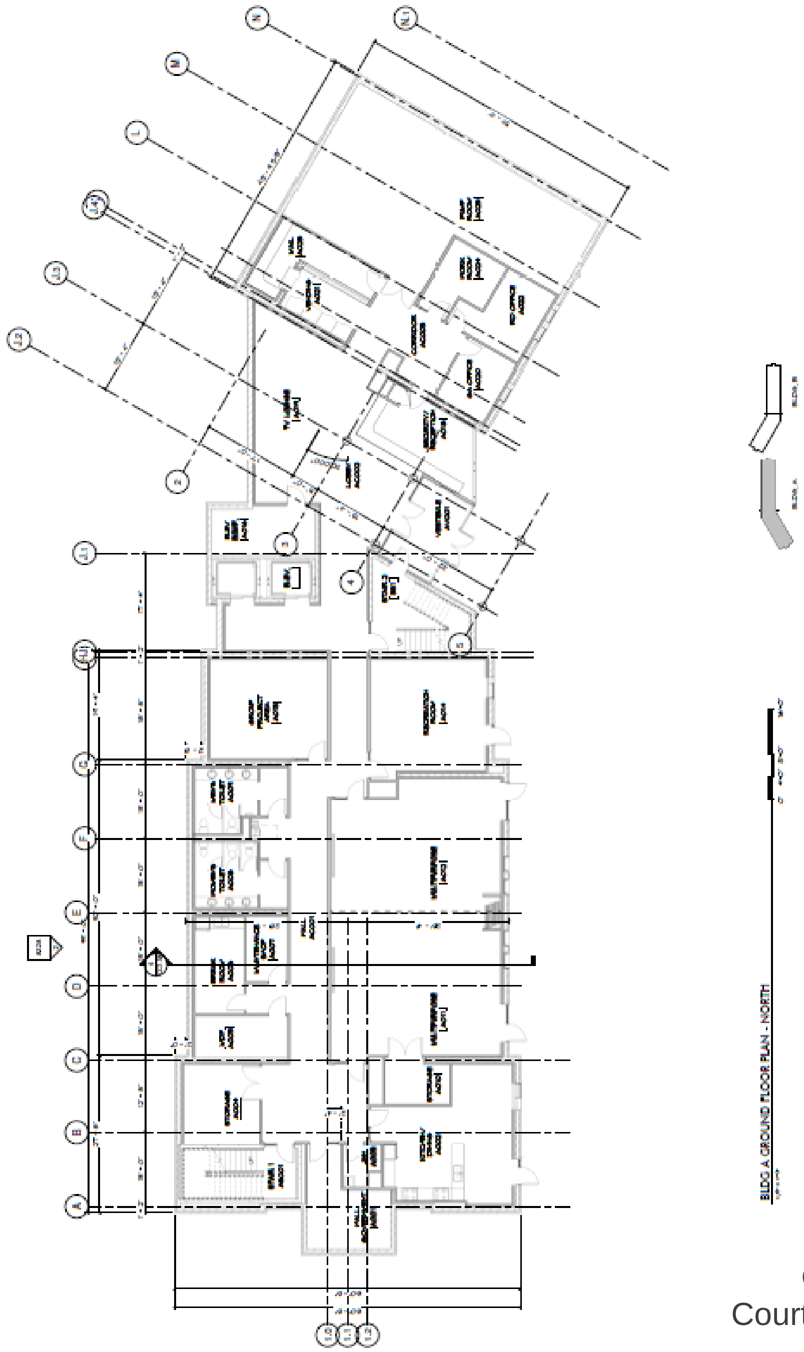
Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr





First Floor Plan
Courtesy of WTW Architects

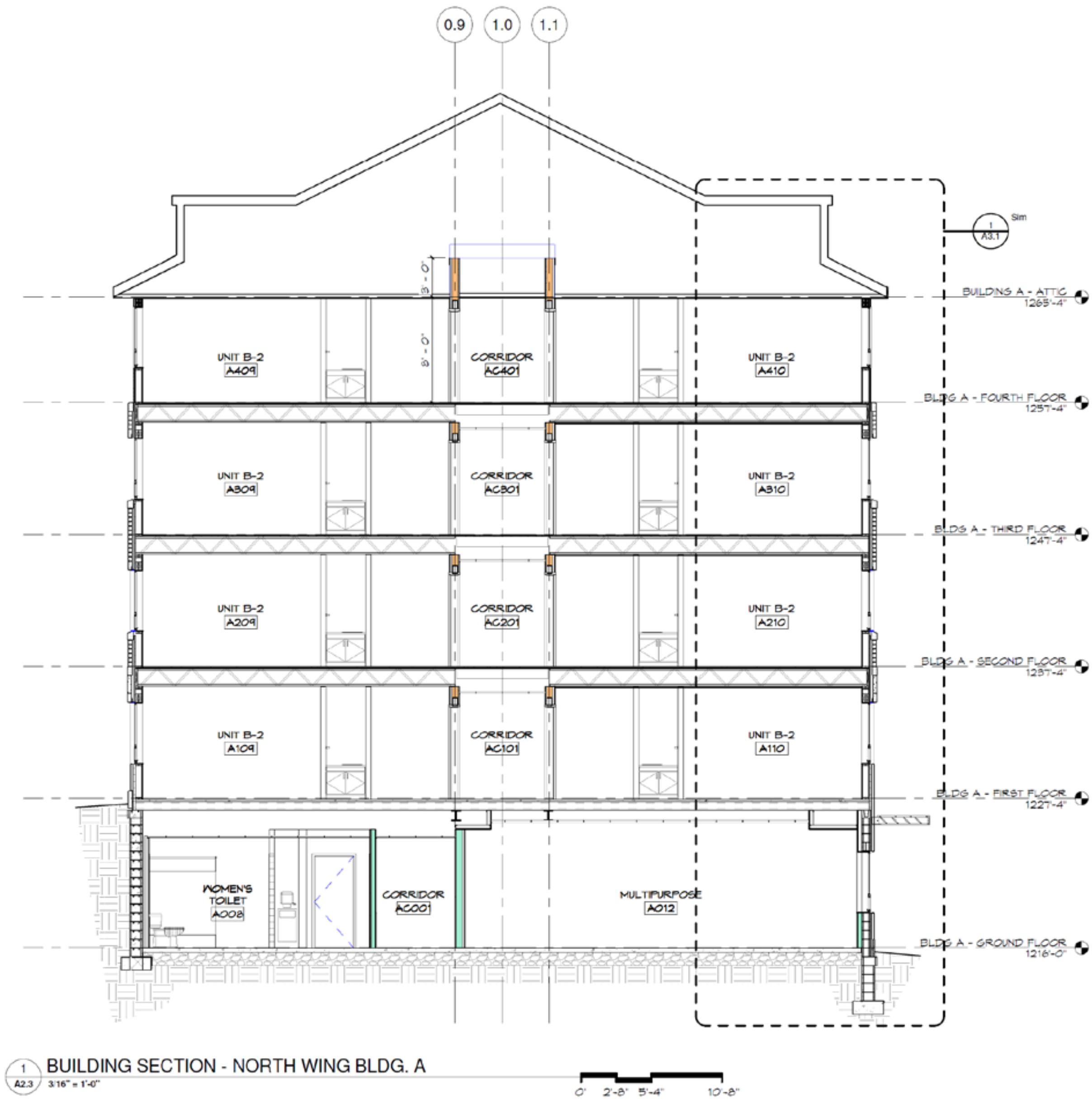
Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr



Ground Floor Plan
Courtesy of WTW Architects

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

Appendix E – Building Section



Building Section
Courtesy of WTW Architects