Dormitory

Northeast USA

Technical Report 3


Rendering Courtesy of WTW Architects

Cadell G. Calkins

Structural Option

Faculty Advisor: Dr. Richard A. Behr

November 16, 2011

Table of Contents

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

Executive Summary

The following technical report evaluates the lateral force resistant system of Dormitory Building A located in Northeast USA. The plans were provided through the owner and WTW Architects. The wind and seismic loads from Technical Report 1 were used and updated for this report. Using ASCE 7-10 load combinations, 1.2D + 1.0W + L + 0.5S was determined to be the controlling combination due to wind creating much greater base shear and overturning moments. In unfactored form, the moment and base shear for seismic loads are 850 ft-kips and 32.27 kips, respectively, in comparison to a moment and base shear due to N-S wind of 2,997 ft-kips and 56.4 kips, respectively.

To check the lateral force resisting system for adequacy, hand calculations were used to check the OSB and GWB based shear walls for load and deflection. The OSB shear walls were well within the allowable strength and deflection limit of H/400, while the (2) ply 5/8" GWB shear walls neared the strength and deflection limit at the south end of the structure on the first floor.

An ETABS model was then developed to determine the adequacy of the GWB shear walls for deflection. Due to the hand calculation formula accounting for nail slip and elongation of wall anchorage, the formula yielded a deflection of 1.102 inches, compared to 0.157 inches that ETABS generated at the top of the structure.

The CMU shear walls of the central core were also checked for strength requirements and found that the wall was over designed for shear strength in the plane. In addition, diaphragm strength checks and stud wall shear strengths were conducted during the deflection checks.

In the end, the lateral force resisting system of the building is adequate for the loads provided.

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

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 over the central core.

Figure 1: Rendering Courtesy of WTW Architects

Structural Overview

Dormitory Building A rests on rammed aggregate piers at a depth of about 30 feet. 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. For lateral loads, oriented strand board and gypsum wall board provide the support needed for the wings, while concrete masonry units provide the support for the central core.

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

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

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.

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

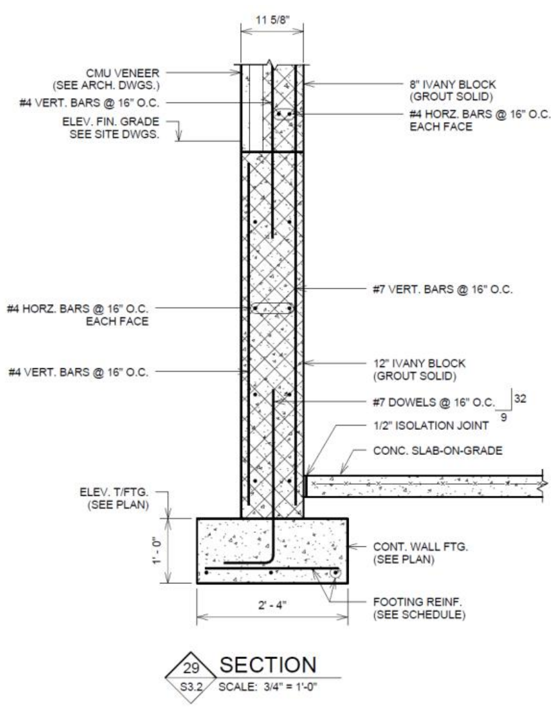


Figure 2: Typical Ivany Block Wall

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

Floor Construction

Considering the First Floor as part of the foundation, the Second through Fourth Floors are nearly identical. Each suite rests on 18 inch deep wood floor trusses spaced at 19.2 inches on center. On top of the trusses consists of ¾ in. of Gypcrete on top of ¼ in. sound mat all resting on ¾ in. 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 inches 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 in. on center topped with ¾ in. Gypcrete on top of ¾ in. plywood. For sound, 3.5 in. 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.

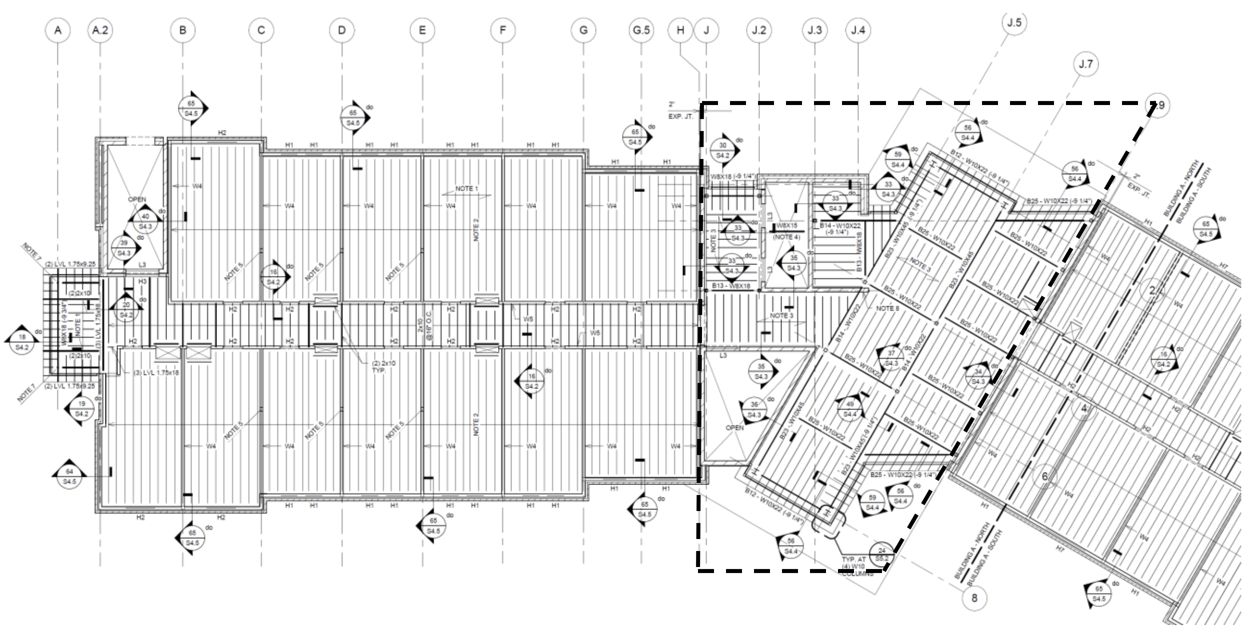


Figure 3: Typical South Wing Floor Plan

Lateral Systems

In regard to handling lateral forces, Building A is basically three 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 in. oriented strand board and 5/8 in. gypsum wall board per wall to resist the lateral forces, while the corridor walls use ¾ in. oriented strand board and two layers of 5/8 in. 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 in. gypsum wall board per wall. This creates a fairly even distribution of lateral forces throughout 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.

In all cases, wind loads will be applied to the brick or metal panel or glass façade and directed to the floor diaphragms above and below the exterior walls by the flexure of the exterior stud walls. The floor diaphragms would then transfer the load to the shear walls as described above, which continue down to concrete planks. The planks are assumed to be a rigid diaphragm that transfers the shear to soil it sits on top of.

For seismic loadings, the mass of each section is concentric with the center of rigidity. The seismic loads start at the center of mass in each diaphragm at each floor level and this load is then transferred to the shear walls as described above for wind.

Materials Used

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

Table 1 – Concrete Specifications

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 2 – Mortar and Grout Specifications

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 3 – Masonry Specifications

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 4 – Steel Specifications

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 5 – Wood Minimum Specifications

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 6 – Wood Sheathing Specifications

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

Design Codes and Standards

According to Sheets S2.1 and LS0-1, the 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 (13th 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 14th Edition will be used in lieu of AISC 13th Edition

These changes in code were made because these are the newest editions of the codes.

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 4, 5 and 6 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. The figures show the worst case for each direction of wind pressure. Please see Appendix A for wind load calculations.

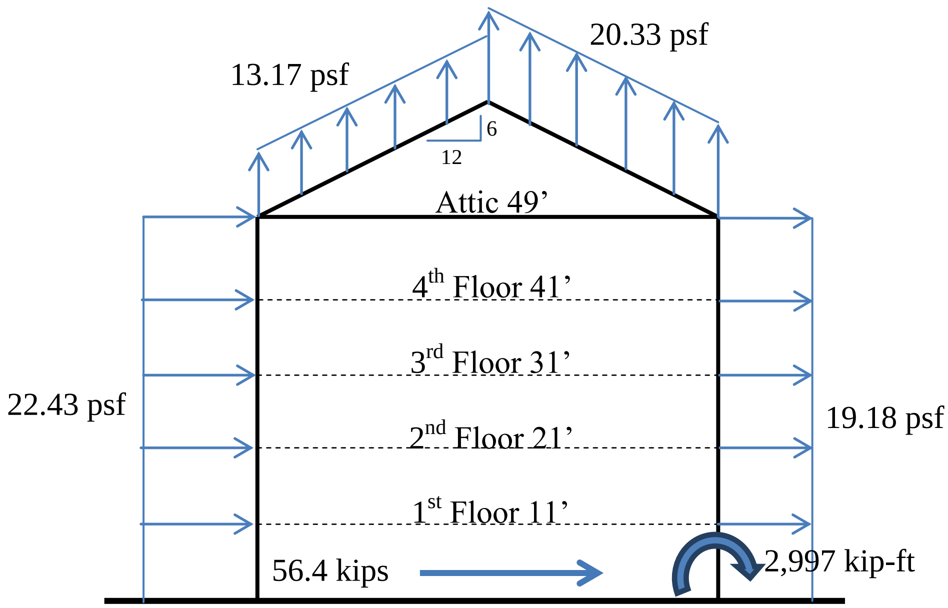


Figure 4: N-S maximum wind pressures for the hip roofs of the wings
The shear and moment are for the entire structure.

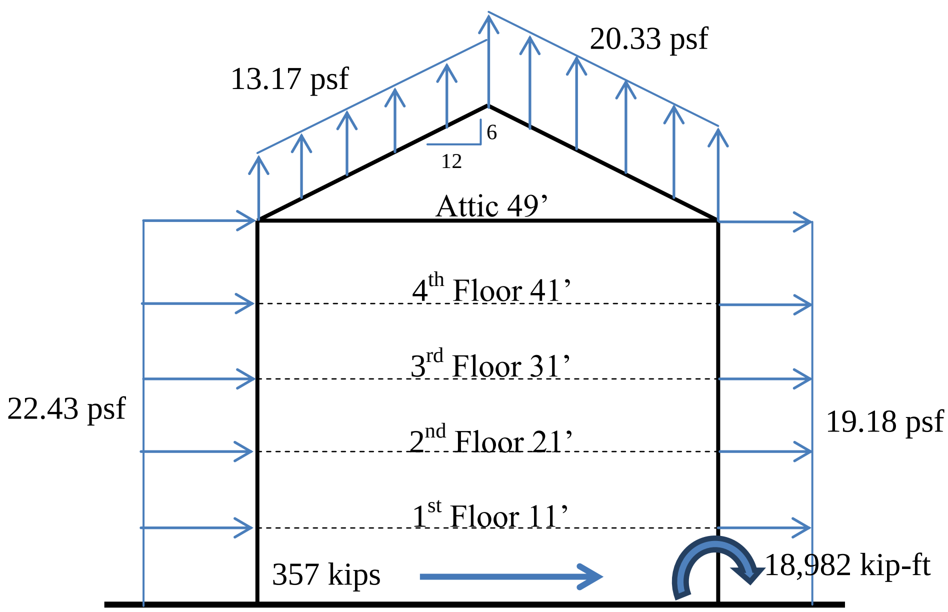


Figure 5: E-W maximum wind pressures for the hip roofs of the wings
The shear and moment are for the entire structure.

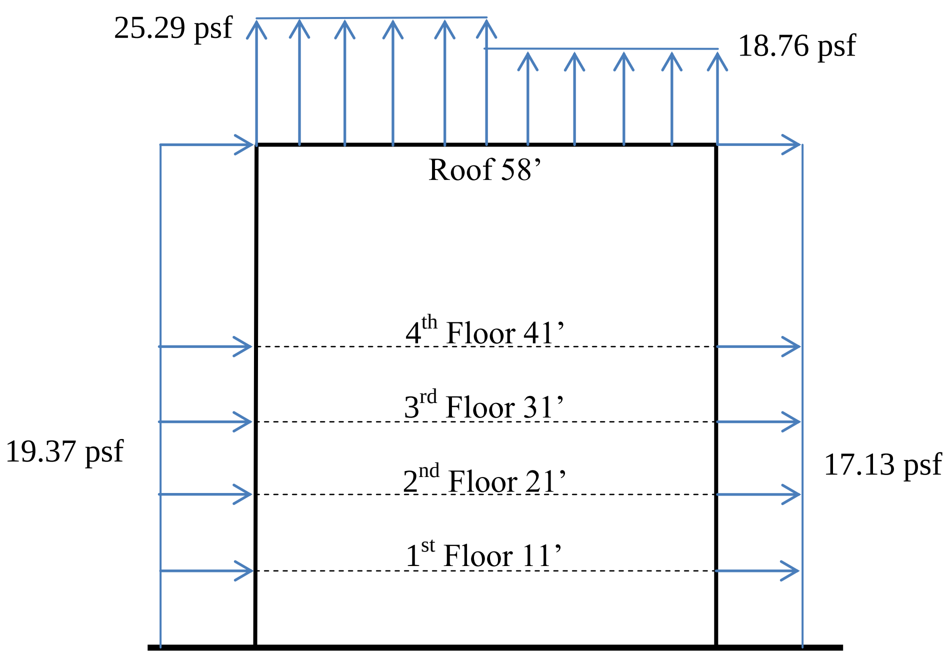


Figure 6: N-S and E-W max wind pressures for the flat roof of the central core

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

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 7

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 850 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 7 below shows the vertical distribution of seismic forces at each level. Please see Appendix B for building weight calculations and seismic load calculations.

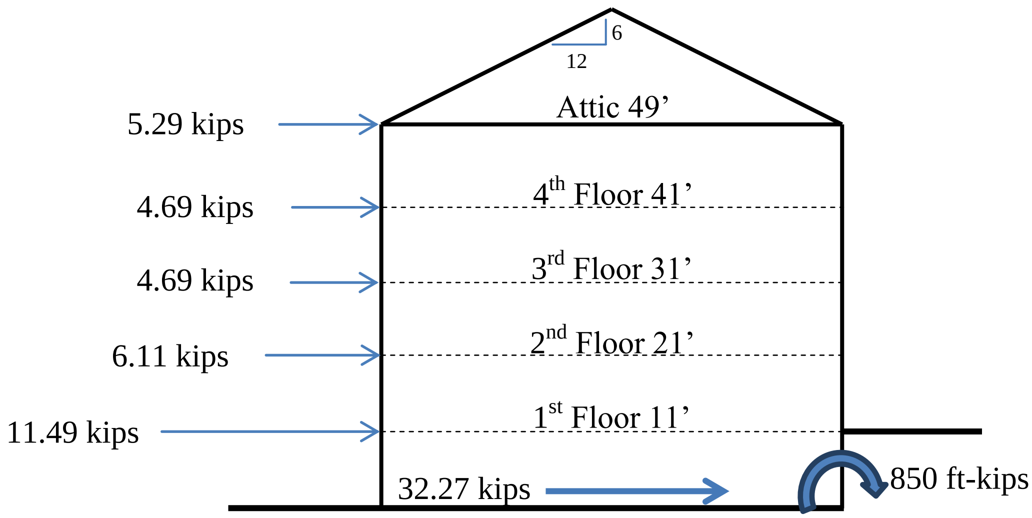


Figure 7: Seismic Story Shear Loads

Summary of Loads

Due to large shear and moments from wind loads, wind load will control over seismic loads throughout the entire structure. For the computation of each load case, please see Appendix B for seismic and Appendix A for wind.

Due to the lateral force resisting system being composed of wood floors, a flexible diaphragm assumption can be used, meaning that the load in each shear wall is dependent on the tributary area of the shear wall.

As an entire system, the overturning moment must be resisted by the weight of the building. The worst case scenario of about 19,000 kip-ft of moment, must be resisted by weight of 633.3 kips centered in the building. Above the first floor, the building has a weight of 2078 kips, well above the required weight. With the building being able to stop the overturning moment by its own weight, the foundations are not affected.

Note that for a true lateral system analysis, the basement walls would have to be looked at for soil loads. However, at this time, my knowledge and abilities lack in the analysis of basement walls. This will need to be looked at in detail at a later time.

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

Lateral System Analysis

In determining the efficiency of the lateral system, hand calculations were done first to determine the strength and deflection of the wood and CMU elements and a computer model was then developed to check the hand calculations.

For the analysis, the building was broken into its three parts, the North Wing, South Wing, and Core. The South Wing was not looked at extensively due to the North Wing controlling the designs. The assumption of a flexible diaphragm was used throughout, which resulted in each shear wall taking a tributary area of load. This assumption also meant that torsion due to wind would be taken by the shear walls that the wind load was applied to. This means that the diaphragm flexes instead of the building twisting under the load.

Stud Shear Walls and Diaphragm

In floors 1 through 4, the shear walls in the wings are composed of gypsum and OSB. For analysis, the worst case of each was identified on the First Floor and checked against strength and deflection (h/400). The results are in Table 8.

Table 8 – Shear Wall Deflection and Shear Load check

Element	Allowable Deflection (in)	Actual Deflection (in)	Allowable Unit Shear (lbs/ft)	Actual Unit Shear (lbs/ft)
½" OSB	0.30	0.113	335	80.6
¾" OSB	0.30	0.0973	475	129
(2) 5/8" GWB	0.30	0.271	250	233

In addition, some of the studs also have to resist flexure from wind because they are an exterior wall.

The strength of the diaphragm was also checked. The deflection of an 18.66' section of diaphragm was found to be .0184" and the allowable unit shear was found to be 50.5 psf, higher than the wind load of 19.18 psf.

For drift checks, the worst case wall is assumed and the 18.66' of diaphragm is also assumed. This results in a story drift of 0.289 inches. The accepted allowable drift to prevent façade damage is h/400, or 0.300 inches. Similarly, if the total drift is computed as the sum of all story drifts plus the diaphragm drift, the total drift comes out to be 1.102 inches. If the same factor of h/400 is checked, this now becomes 1.140 inches. One thing to note is that this drift only occurs in one spot on the building, where the 18.66 feet spacing between shear walls is located on the South Wing.

In addition, for the shear walls to transfer the shear between each floor, an adequate nailing scheme must be utilized. The existing system of three 16d common nails at 16 inches on center must resist a shear of 233 pounds per foot, or an equivalent shear of 311 pounds per nailing group. The allowable shear load of a 16d common nail, as calculated in Appendix C, is 141 pounds per nail, or 423 pounds per nailing group.

CMU Shear Walls

In a very basic strength analysis, CMU shear walls were checked using information from the SEAOC Design Manual. Through the calculations, it was determined that, for the computed shear value at the first floor, that the shear strength of the wall is overdesigned. The needed shear capacity of the walls was determined to be 6.2 kips, while the shear walls provide 62.7 kips of resistance. The difference in capacity is most likely due to the idea that the shear wall also carries gravity loads and snow loads.

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

Computer Analysis

ETABS was used to compute the drift of a GWB based shear wall. To do this, the Modulus of Elasticity and Shear Modulus were obtained from information collected in a Forest Products Laboratory Research Paper. For the document, please see the references on the CPEP site. This yielded a Modulus of Elasticity of 2.45 $x10^5$ lb/in^2 and a Shear Modulus of 1.05 $x10^5$ lb/in^2.

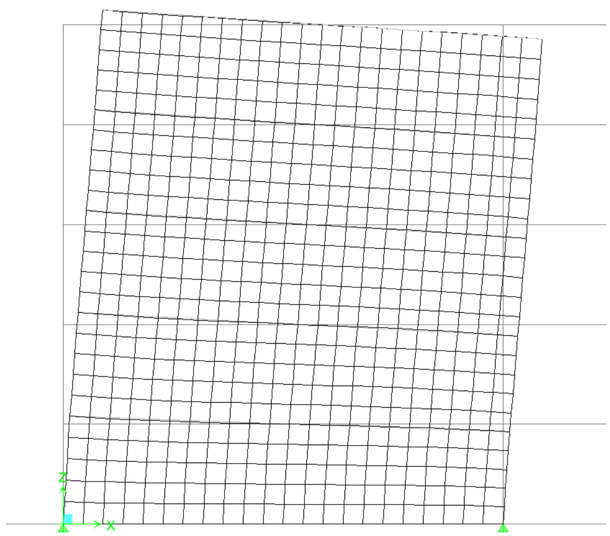


Figure 8: Deflection in ETABS due to wind

For the analysis, a 44 foot long wall on 4 stories, with a thickness of 1.25", or the equivalent of (2) 5/8" layers, yielded a total drift of 0.157 inches, to be compared with 1.102 inches by hand calculations. The deflected shape can be seen in Figure 8. With these two numbers being considerably different, it is believed that ETABS does not view the assembly correctly. Comparing the hand formula to the inputs needed for ETABS, two major differences arise. First, in the bending portion of the formula, the modulus of elasticity and area needed are the modulus and area of the end posts. ETABS does not realize that these are the only items taking the bending loads. ETABS believes that the gypsum wall board is able to take its own bending loads, when in fact it does not. Also, the formula uses an apparent shear wall shear stiffness. This number comes from the NDS tables.

ETABS relies on the actual shear stiffness of the wall, which it calculates itself. Lastly, ETABS cannot account for the vertical elongation of the wall anchorage system in a suitable manner. It can use springs, however, it views the springs as acting over the entire area of the wall. All in all, ETABS over estimates the shear stiffness of the wall, creating a lower drift.

Conclusion

After an extensive study of the lateral force resisting system of the Dormitory, it is evident that the lateral system is sufficient for the wind and seismic loads.

The gypsum wall board shear walls consisting of two plies of 5/8" GWB are adequate for a worst case scenario of 233 pounds per foot on the first floor level and with this load, the nailing scheme of three 16d common nails at 16 inches on center is also sufficient. In addition, the OSB shear walls were also adequate and perhaps over designed assuming that the entire length of wall and door headers aid in transferring the shear.

For the CMU shear wall, it appears that those too are over designed for the required shear. This over design is most likely due to the walls having to carry gravity and snow loads in addition to lateral loads.

When comparing the hand calculations to a computer analysis of the GWB wall system, it was evident that ETABS was unable to accurately represent all the nail slip and elongation of the anchorage system. This resulted in a significantly underestimated deflection.

In the end, the lateral force resisting system of CMU shear walls in the core and OSB and GWB shear walls in the wings, with a wood, flexible diaphragm, is adequate for the wind loads and seismic loads for this project.

Appendix A – Wind and Soil Load Calculations

Basic Wind Speed=	105 mph
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

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

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)

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

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

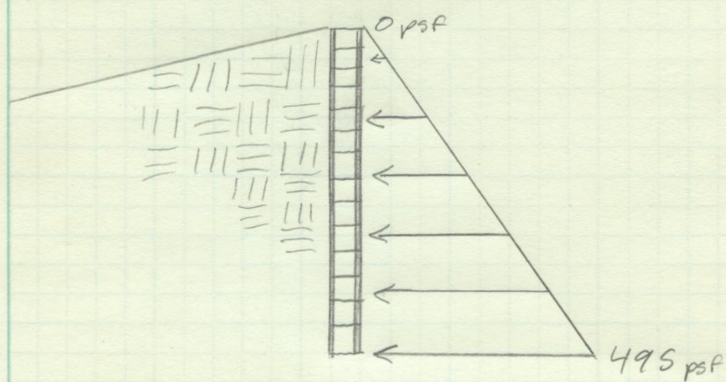
Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

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: $45 \cdot 11' = 495$ psf @ 11'



Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

Appendix B – 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**

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

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

$V = C_s W$

$S_{DS} = \frac{2}{3} S_{MS}$ $S_{MS} = F_a S_s$

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

$S_{D1} = \frac{2}{3} S_{M1}$ $S_{M1} = F_v S_1$

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

$S_1 = .06g$ $F_v = 1.3$ $S_{M1} = .078g$ $\cdot \frac{2}{3} = S_{D1} = .052g$

According to USGS Online:

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

$S_1 = 0.053g$ $S_{M1} = 0.089g$ $S_{D1} = 0.060g < 0.067g$

Seismic Design Category = A

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

$R = 2$ for Masonry $C_s = 0.01$ per Section 1.4

$R = 2.5$ for light frame

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

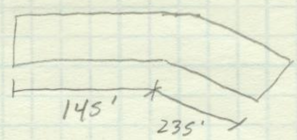
$= 2078 \cdot 0.01 = \boxed{20.78 \text{ Kips}}$ design = 30 kips

Distribution Calculated in Spreadsheet.

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

Appendix C – Stud Wall and Diaphragm Check

Wind Load | Base Shear & Overturning



For Base Shear of entire structure, assume the face dimension is

145' + 235' = 380' in the E-W directions

~~Base Shear~~ Overturning = $(22.43\,psf + 19.18\,psf)(49')(49/2) + 13.17\,psf\,(30')(45') +$

$(20.33\,psf)(30')(15') = 76\,880$ lb-ft/ft

$76\,880 \cdot 380/1000 = \boxed{29{,}215 \text{ k-ft}} \gg 80$ k-ft Seismic

Wind controls in E-W

In N-S directions: (N-wing), Least

Overturning $(22.43 + 19.18)(49)(49/2) + 13.17(30)(130) + (20.33)(30)(15)$

$= 110464$ lb-ft/ft

$110464 \cdot (60')/1000 = \boxed{6{,}628 \text{ k-ft}} \gg 850$ k-ft

wind controls in N-S

Base Shear in N-S = $\dfrac{(22.43 + 19.18)(60)(49)}{1000} = \boxed{122 \text{ k}} \gg 32.3$ k

wind controls

Base Shear in E-W wind controls by observation.

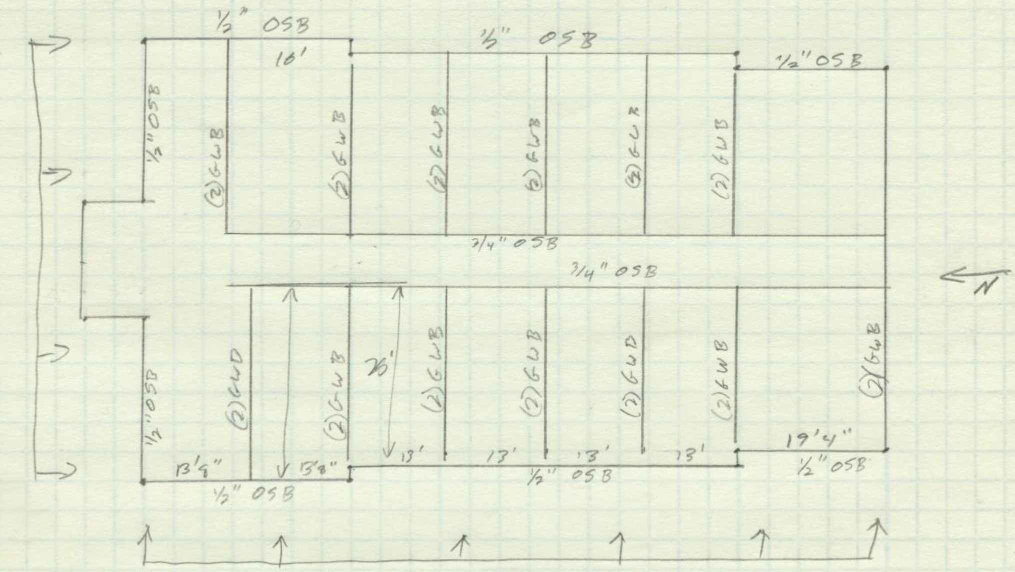
ASCE 7-05 Comparison

$p_s = \lambda K_{zt} I\, p_{s30} = (1.22) \cdot 1.0 \cdot (1.0) \cdot$

21.9	A	= 26.7	psf	
3.5	B	= 4.3	psf	22.43 – 3.25
15.9	C	= 19.4	psf	compare to ~~22.43 + 19.18~~
3.5	D	= 4.3	psf	~~Calc is wrong~~

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

North Wing
Typical Floor 1st → 4th Design Shear at 1st Floor



Max = 19.18 psf
Height = 33'

Assume Flexible Diaphragm

N-S (worst case)

1/2" OSB takes $(27/2)(33)(19.18) = 8544$ lbs Shear $= 8.54$ k $= 80.57$ lb/ft
Over 106'

3/4" OSB takes $(27/2 + 8/2)(33)(19.18) = 11077$ lbs Shear $= 11.08$ k $= 128.8$ lb/ft
Over 86'

E-W (worst case)

1/2" OSB takes $\left(\frac{13.66}{2}\right)(33)(19.18) = 4.32$ k Over 52' $= 83.1$ lb/ft

(2) 5/8" GWB takes $\left(\frac{19.83}{2} + \frac{13}{2}\right)(33)(19.18) = 10.23$k Over 44' $= 233$ lb/ft

For Torsional Wind Loads $(M_T = .75(P_{wy} + P_{Lw})B_y e_y)$, because of flexible diaphragm assumption, the above worst, case designs still control

Check

1/2" OSB $= \Delta = \frac{8 \cdot 83.1 \cdot 10^3}{1.6E6 \cdot 10.5 \cdot 52} + \frac{83.1 \cdot 10}{1000 \cdot 15} + \frac{10 \cdot (.25)}{52}$

$= .000479 + .0554 + .0568$

$= .113$ in

$h/400 = 10 \cdot 12/400 = .3$" ok

North Wing

Check cont 1/4" assumed for elongation, not a controlling factor

$3/4"\ OSB = \frac{8 \cdot 128.8^3 \cdot 10^3}{1.6E6 \cdot 16.5 \cdot 96} + \frac{128.8 \cdot 10}{1000 \cdot 19} + \frac{10(.25)}{86}$

$= .000454 + .06779 + .0291$

$= .0973 < .3$ ok for h/400

Shear Capacity $1/2"\ OSB = 670/2 = 335$ lb/ft ok
$3/4"\ OSB = 950/2 = 475$ lb/ft ok

Check GWB

$5/4"\ (2\ ply) = \frac{4 \cdot 233 \cdot 10^3}{1.6E6 \cdot 10.5 \cdot 44} + \frac{233 \cdot 10}{1000 \cdot 11} + \frac{10(.25)}{44}$

$= .00252 + .2118 + .0568$

$= .271 \leq .3$ ok

Shear Capacity $= 500/2 = 250$ lb ok

South Wing

In N-S direction, the North Wing is the worst case because the walls are shorter in that direction.

E-W
OSB same as North Wing
(2) 5/8" GWB takes $\left(\frac{18.66}{2} + \frac{18.66}{2}\right)(33)(19.18) = 11811$ over $51' = 231.6$ lb/ft

The North Wing is a worst case.

Check Toe Nails

16d (3) @ 16" OC worst case shear = 233 lb/ft = 311 lb/16"

= 103.6 lb/nail shear

16 d common nails in Douglas fir w/ 1½" member = 141 lb/nail

$\boxed{141 > 103.6\ OK}$

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

Diaphragm Check

3/4" OSB w/ 8d common nail @ 6" edge spacing unblocked

$V_w = 505$ plf @ 10' floor to floor height

$V_w = 50.5$ psf > 19.18 psf wind $\therefore$ ok

Deflection

$$\delta = \frac{5 \cdot 192 \cdot 44^3}{8 \cdot 1.6E6 \cdot 18.66 \cdot (1\tfrac{1}{4} \cdot 18)} + \frac{.25 \cdot 192 \cdot 44}{1000 \cdot 670} + 0$$

$$= .0152 + .003152 = .0184 \text{ in}$$

$$L/360 = 18.66 \cdot 12/360 = .622 \quad \text{ok}$$

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

Appendix D – CMU Shear Wall Check

CMU Shear Wall Check

8" CMU Grout Solid w/ #5's @8"OC

Assume Flexible Diaphragm, since the wood diaphragm is much less stiff then the CMU walls.

Wall Length = 8'

$$\text{Load} = \left(\frac{19.33}{2}\right)(33)(19.18) = 6117 = 6.117\text{ k}$$

$$\frac{M}{V_d} = \frac{152\text{ k-ft}}{6.117(8)} = 3.106$$

$C_d = 1.8$

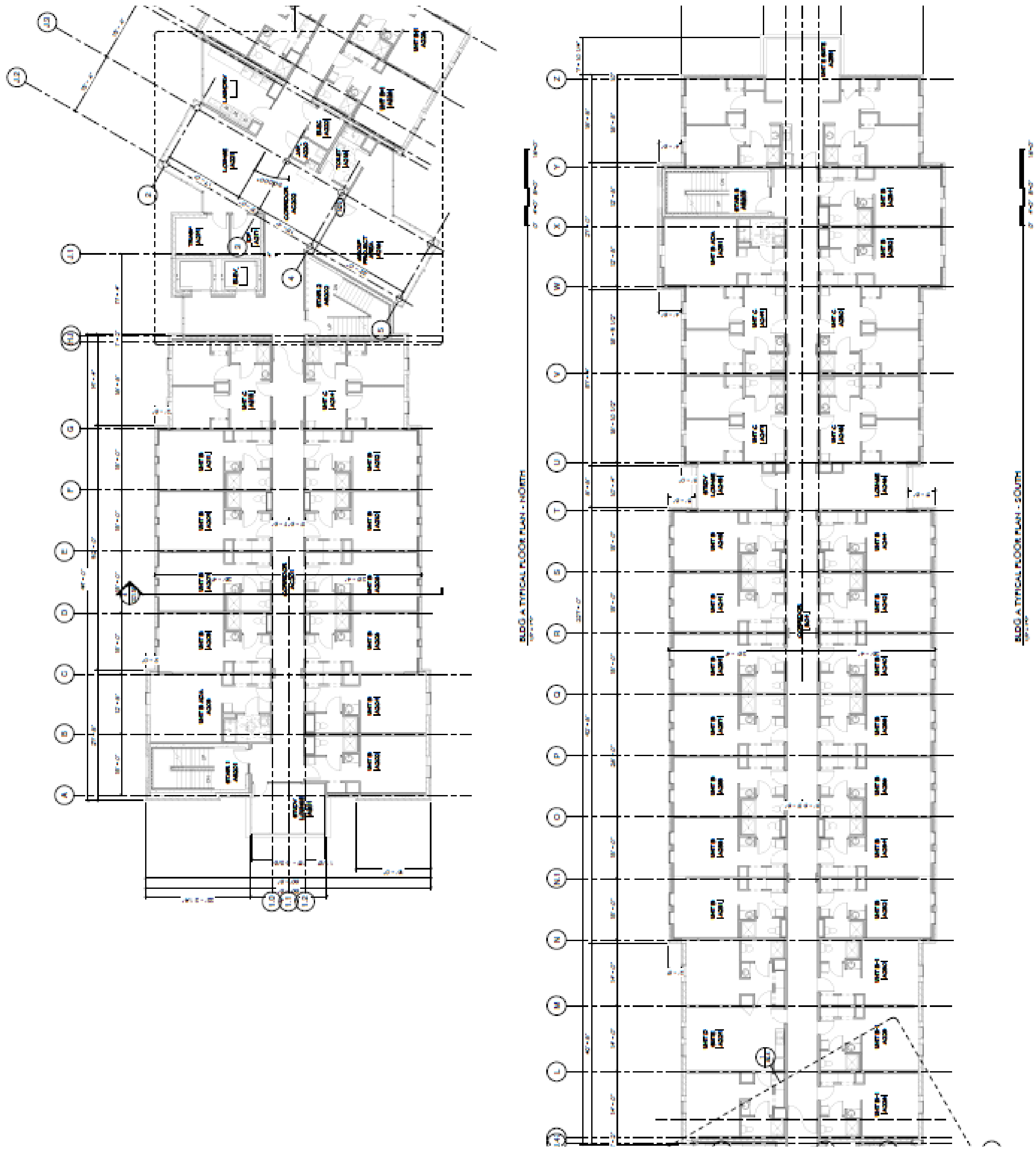
For $\phi = .6$ $\quad \phi V_s = (.6)(7.625)(96)\left[\left(\frac{.20^{\#}}{7.625 \cdot 16}\right)\right](60{,}000) = 43.2\text{ k}$

$$V_m = C_d A_{nv} \sqrt{f'_m} = (1.80)(7.625)(96)\sqrt{2500\text{ psi}} = 65.9\text{ k}$$

$$\phi V_n = .6(65.9) + 43.2 = 82.7 > 6.117\text{ k} \quad \text{ok}$$

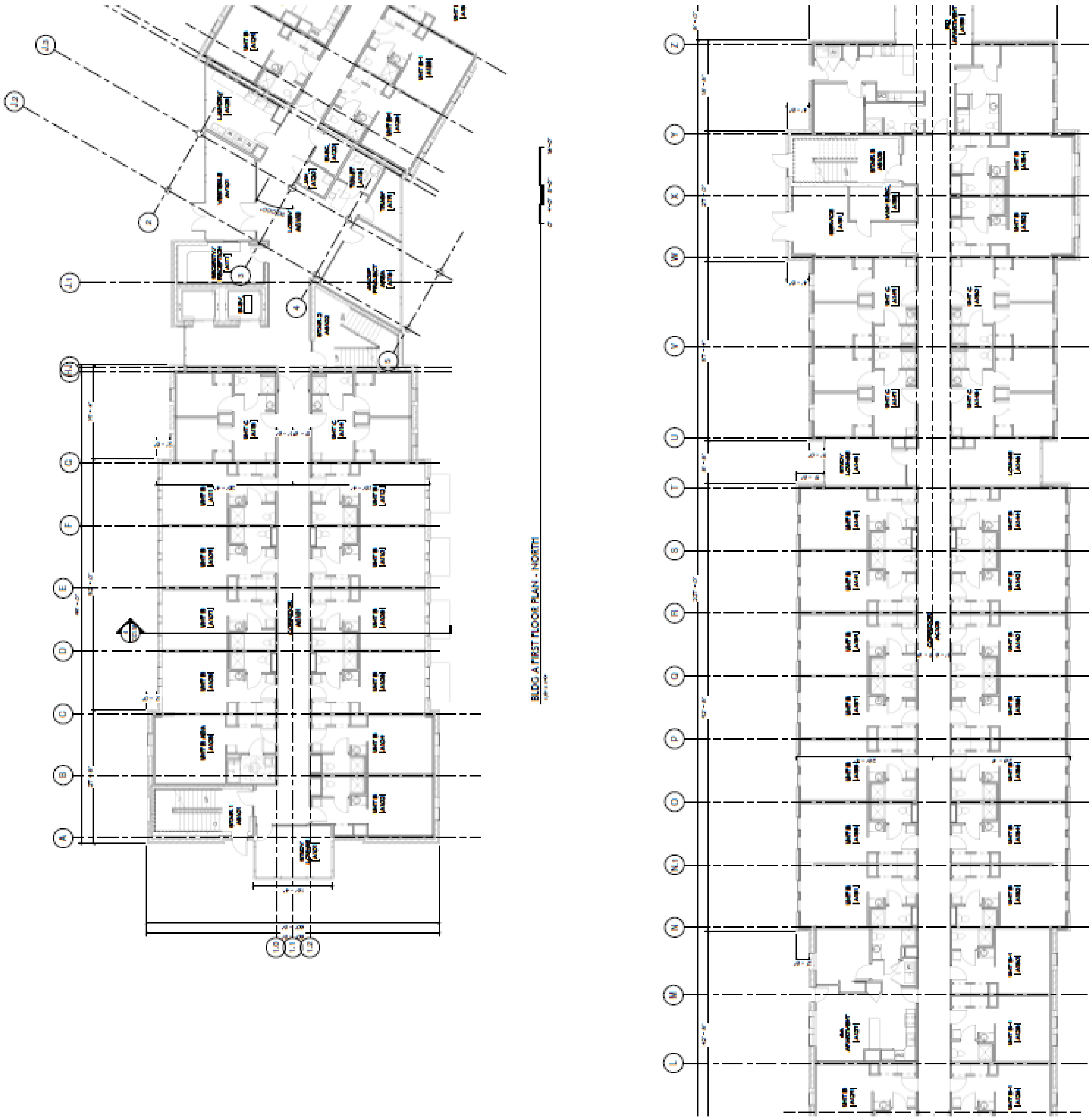
Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

Appendix E – Typical Floor Plans

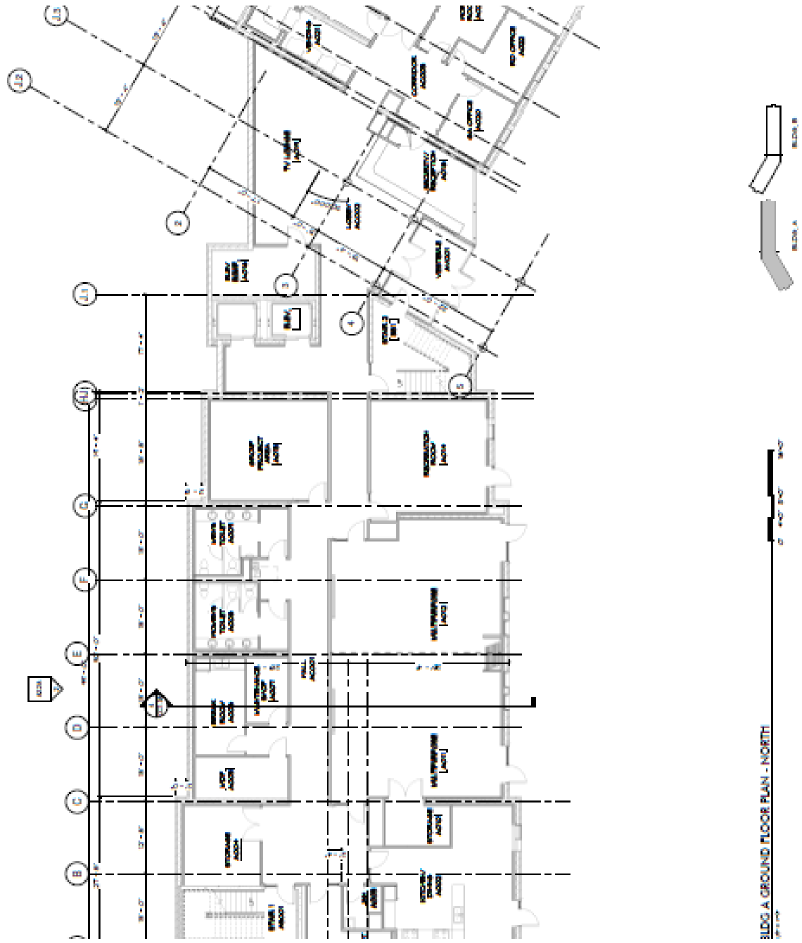


Typical Floor Plan
Courtesy of WTW Architects

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr



First Floor Plan
Courtesy of WTW Architects

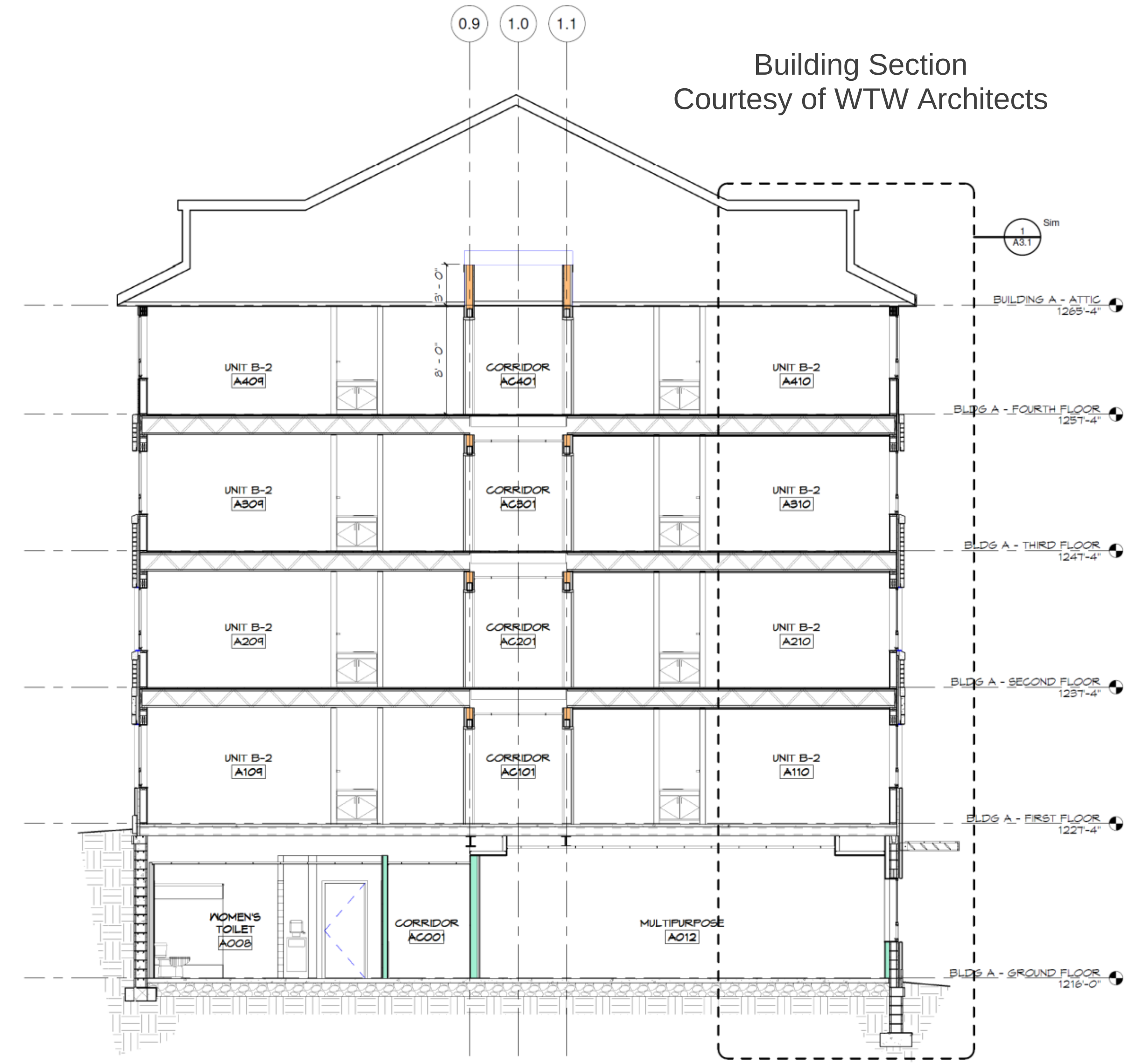


Ground Floor Plan
Courtesy of WTW Architects

Cadell Calkins
Faculty Advisor: Dr. Richard A. Behr

Appendix F – Building Section

Building Section
Courtesy of WTW Architects



1 A2.3 BUILDING SECTION - NORTH WING BLDG. A
3/16" = 1'-0"