Dormitory

Northeast USA

Final Report

Rendering Courtesy of WTW Architects

Cadell G. Calkins

Structural Option

Faculty Advisor: Dr. Richard A. Behr
April 30, 2012

Dormitory

Northeast USA

General

Function: Dormitory
Size: 92,389 sq. ft. Building A
Height: 57.75 feet
Stories: 4 above grade, 5 total
Construction: October 2010-January 2012
Cost: $26 Million for Buildings A & B and sitework
Delivery: Public/private partnership using a development team to deliver the project by guaranteed maximum price

Architecture

- Two traditional brick wings with an asphalt hip roof flank a central glass core with a flat roof
- Wings house suite style dormitory units with the core housing public and study spaces
- Strategic placement of interior windows on the ground floor allows for sunlight in interior rooms

MEP

- 34.5 kV 208Y/120V 3Φ 4 Wire feed for both buildings and a 2500 amp switchboard for Building A
- 250 kw Emergency Generator
- 54 geothermal wells for Building A using heat pumps
- 3 Energy Recovery Units in attic of Building A
- Fire protection by wet pipe sprinklers
- Uses Fluorescent, LED and Metal Halide Lighting

Project Team

Owner: Not Released
Architect: WTW Architects
Construction Manager: Massaro Corporation
MEP: H. F. Lenz Company
Structural: Taylor Structural Engineers, Inc.
Landscape: LaQuatra Bonci Associates
Developer: Allen & O'Hara Development Co. LLC

Structural

- Spread footings on rammed aggregate piers
- Lightweight wood construction with gypsum wall board and oriented strand board shear walls
- Concrete masonry unit core with Laminated Veneer Lumber joists on steel beams

Construction

- Building A to be completed before Building B
- Snow and bad weather caused up to a 3 week delay
- Retaining wall collapse caused delay


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http://www.engr.psu.edu/ae/thesis/portfolios/2012/CGC5037

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Executive Summary

The Dormitory, a four story tall dormitory in the Northeast United States with a walk-out basement is constructed mostly out of wood framing. Primarily, the Dormitory consists of an open web wood floor truss system supported by 2x6 wood stud walls. For lateral support, the primary system consists of either oriented strand board (OSB) or gypsum wall board (GWB) on wood studs.

As global warming continues to become a larger issue for the world, soon it will be a concern for a structural engineer in the form of increased wind and weather loads. This loading was then considered by creating a scenario in which the State College of Florida, Manatee-Sarasota facility wanted the Dormitory built on their campus.

Proper loads were first determined according to the Florida Building Code 2010, and ASCE 7-10, where applicable according to the code, resulting in a design wind speed of 150 mph. To withstand this load, two lateral system redesigns were considered. An oriented strand board shear wall design was completed on the wings resulting in only the walls between the suites to be changed from two layers of 5/8 inch gypsum wall board to 1 layer of 5/16 inch oriented strand board with 3 inch nail spacing at the edge. In addition, a steel braced frame design was undertaken. For a steel lateral system design, it was deemed logical to also design the gravity system using steel. This redesign made use of W8x31 columns at the four corners of each suite and a braced frame using 2 inch by 2 inch angles of differing thicknesses for the braces between each suite.

An electrical breadth study was completed on the new Dormitory where photovoltaic solar shingles were designed to partially remove the Dormitory from the grid and to provide backup power in the case of an emergency. In addition, a façade breadth was undertaken to design a new building enclosure system that would perform under the heat of the Florida sun and debris impacts during hurricanes.

Acknowledgements

Penn State Architectural Engineering Faculty

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Massaro Corporation

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WTW Architects

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Dormitory Owner

A special thank you to the Dormitory owner and its President who provided me the opportunity to use the Dormitory for my thesis.

Family and Friends

A heartfelt thank you goes out to all my family and friends. You have always stuck by me through the years and it is greatly appreciated.

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 site work 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 of well graded crushed rock. 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.

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.

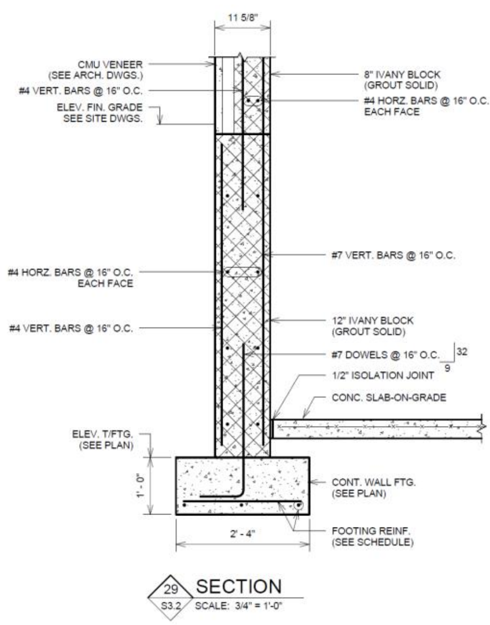

Figure 2: Typical Ivany Block Wall
Courtesy of WTW Architects
(Page S3.2)

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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 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 in. x 9.25 in. 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.

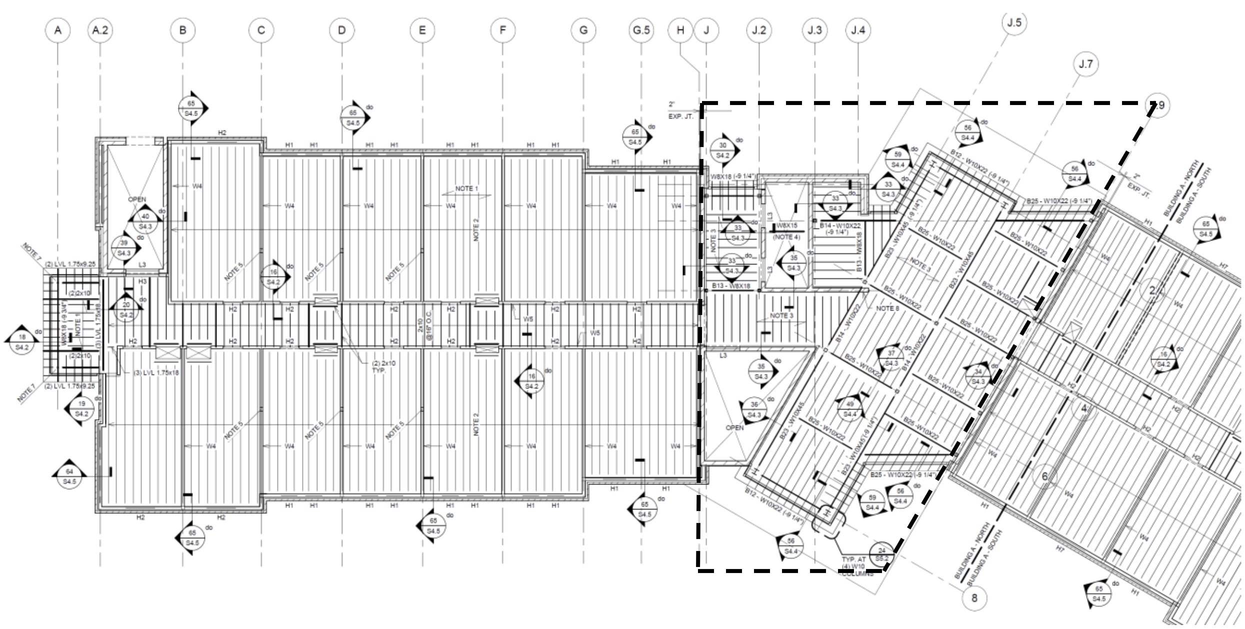

Figure 3: Typical North Wing Floor Plan Courtesy of WTW Architects (Page S1.3A)

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 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 in.
Interior Slabs	4000	0.45	Normal	¾ in.
Exterior Slabs	4000	0.40	Normal	¾ in.

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 in. dia.	N/A
Anchor Rods	ASTM F1554, ¾ in. 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.

Problem Statement

In this day and age, many problems face a structural engineer, including but not limited to earthquakes, terrorism, snow and other loads. One problem area that is not an apparent concern is the potential structural engineering effects of global warming. It is well known that global warming is a concern in the building industry, thus buildings are becoming more efficient and greener. However, global warming has the potential to become a design consideration for a structural engineer as well.

According to the National Wildlife Federation website, the maximum hurricane wind speeds are expected to increase 2 to 13 percent within this century. (Global Warming is Affecting Weather) As a hurricane is often the maximum wind event that a building is designed for, it is logical to assume that the maximum wind event will also increase 2 to 13 percent.

In regards to the Dormitory, which was originally designed for a wind speed of 90 mph (ASCE 7-05) or 115 mph (ASCE 7-10), at a maximum (+13%) this speed could increase to 102 mph or 130 mph, respectively, due to global warming. In able to better understand this increased wind load on a wood structure, the author of this proposal has created a scenario in which the State College of Florida, Manatee-Sarasota (SCF) would like to build their first on-campus housing and really liked the design of the Dormitory. In this area, the design wind speed is 150 mph.

Planned to be a potential haven for students during hurricanes or tornadoes caused by hurricanes, SCF has required that the Dormitory be capable of withstanding wind pressures due to hurricanes and tornadoes, and debris impacts on the façade. In addition, they also require that a foundation that will support the Dormitory on sandy soil.

In addition to these loads, the Dormitory will also need to be properly designed for gravity loads. In each suite, a floor live load of 40 pounds per square foot will need to be incorporated into the design as well as a 25.7 pound per square foot floor dead load. For public spaces, lobbies, corridors, and stairwells, a floor live load of 100 pounds per square foot will also need to be implemented into the design. Additionally, dead loads for the walls, concrete floor, and roof can be found in Table 7.

Table 7 – Dead Loads

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

Assuming that the same soil fill level and fill material will be used as the original Dormitory; a soil pressure of 45 pounds per square foot per foot of depth will need to be included in the design. This amounts to 0 pounds per square feet at the surface increasing linearly to 495 pounds per square foot at 11 feet deep. In Florida, it is generally assumed that wind loads will control the lateral system; however, seismic loads will need to be implemented in the design.

Proposed Solution

To meet the SCF's design requirements, the Dormitory's lateral system will need to be redesigned and a new foundation will also need to be designed. For the lateral system, it is foreseen that using a mostly gypsum wall board based system, like in the original Dormitory will be inadequate. In this case, it is best that two systems be considered for the wings:

- Oriented strand board (OSB) shear walls
- Steel braced frame shear walls

In addition, the lateral system in the core will also need to be redesigned and it is proposed to be two steel braced frame walls in the East-West direction and three steel braced frame walls in the North-South direction.

To counteract the moist conditions in a termite prone area, the Dormitory will also be completely redesigned as a steel building. This will amount to a redesign of the floor in steel joists and deck with a gravity system composed of mostly wide flange shapes. Furthermore, the lateral system will be designed using braced frames where applicable and moment frames elsewhere. Braced frames will be utilized in between dormitory rooms and moment frames will be used where large openings prevent an aesthetic use of braced frames.

To support the new steel structure, a new foundation system will need to be designed in able to properly support the Dormitory on Florida's sandy soil. This foundation is proposed to be a spread footing system similar to what is currently in place.

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Structural Depth

To move the Dormitory to Florida means the structure will see increased loads that it was not previously designed for. To determine these loads, ASCE 7-10 was utilized and full calculations can be found in Appendix A.

First, seismic loads were examined using a Site Class E consisting of soft soil and clay. The Dormitory was designed with a 1.0 importance factor and a Risk Category II. Table 8 contains a comparison of the S_{DS} and S_{D1} values for the Florida location and the Northeast location. Both situations lead to a Seismic Design Category A, with a base shear of 32.3 kips and an overturning moment of 850 foot-kips.

Table 8 – Seismic Design Values

Source	S_{DS}	S_{D1}
USGS Online (FL)	0.088g	0.065g
USGS Online (NE)	0.094g	0.060g

In comparison, the maximum wind loads were determined using a basic wind speed of 150 miles per hour in a partially enclosed structure. The directionality factor was taken as .85 and K_{ZT} was assumed to be 1.0. This wind yielded the maximum pressures in Figures 4 and 5 for the hip roof and flat roof, respectively. These pressures created an overturning moment in the E-W direction of 38,740 foot-kips and 10,230 foot-kips in the N-S direction, in the north wing. The north wing was determined to be the most sensitive to overturning as it has a smaller moment arm in the N-S direction than the south wing. For base shear, the Dormitory yielded 115 kips in the N-S direction and 729 kips in the E-W direction.

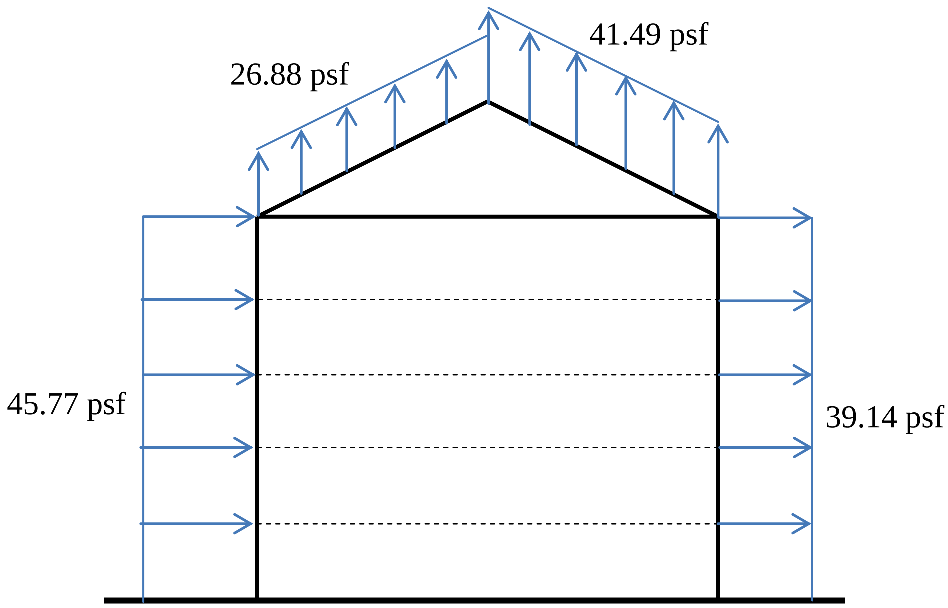

Figure 4: N-S and E-W maximum wind pressures for the hip roofs of the wings

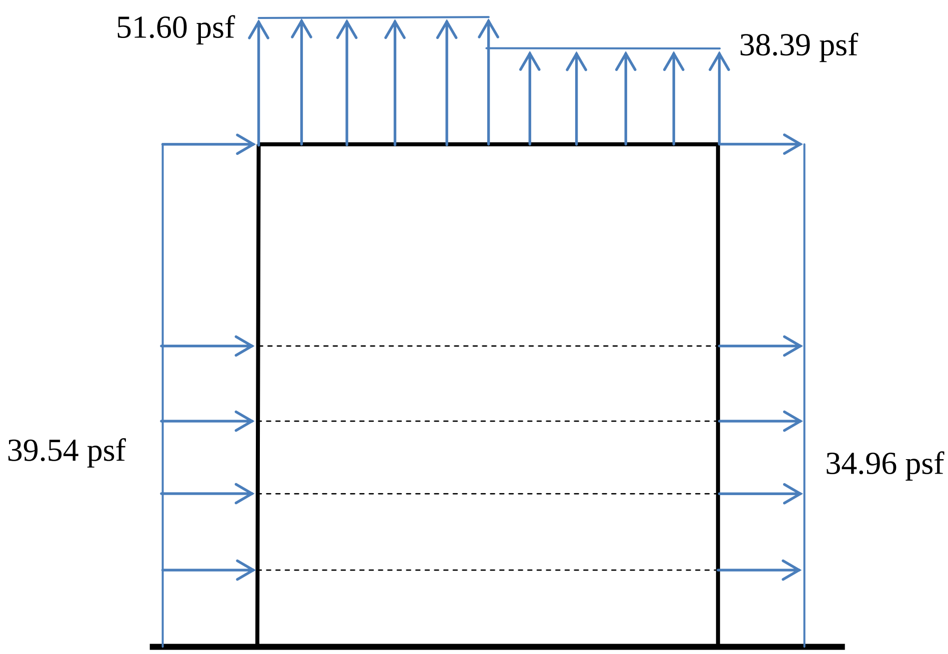

Figure 5: N-S and E-W maximum wind pressures for the flat roof of the central core

Based on overturning moment and base shear in all directions, wind loads control over seismic loads in the new design. To properly resist the overturning moment, the Dormitory created a resisting moment due to self weight of 96,810 foot-kips in the E-W direction and 613,000 foot-kips in the N-S direction.

To design for the increased wind loads, two design considerations were undertaken. First, a system similar to the current system was designed using oriented strand board shear walls. In addition, a second system was also designed in which a steel braced frame system was utilized. For the second system, a full gravity system redesign in steel was also completed for cost comparisons between a system that is prone to termite and decay damage (wood), and a system that is better suited to handle decay (steel). Lastly, a foundation check was completed to ensure that the current foundation system would work with the new location.

Oriented Strand Board Shear Wall Lateral System

To utilize the Dormitory in Florida, a new lateral system was needed for the increased wind loads. First, a lateral system using oriented strand board similar to what was built in the Northeast USA was designed with the calculations in Appendix B. For the design, an inter-story drift limit of h/400 was chosen and wind pressures as calculated in Appendix A were designed for, as wind controlled over seismic.

To begin the design, it was determined from the wind load overturning moment calculations that the north wing encountered larger loads in the north-south direction due to it being shorter than the south wing. For construction ease, it is assumed that whatever is designed in the north wing will be utilized in the south wing and thus, only the north wing design was calculated. In addition, due to the use of a wooden floor system, a flexible diaphragm assumption was made, resulting in all shear walls relying on a tributary width for load distribution.

In the north-south direction, the original design of ¾ inch OSB on the corridor walls and ½ inch OSB on the exterior walls was checked for the increased wind load and it was determined that the shear capacity of the OSB at the ground floor level was sufficient. The ½ inch OSB had a shear capacity of 536 pounds per foot, greater than the wind load shear of 165 pounds per foot. Similarly, the ¾ inch OSB had a shear capacity of 760 pounds per foot, which was greater than required shear due to wind of 262.5 pounds per foot. Next, the east-west direction was checked and it also found that the ½ inch OSB on the exterior walls was sufficient for the shear due to wind of 170 pounds per foot. However, the original use of GWB on the walls between adjacent suite rooms was not adequate and an OSB replacement needed designed. Deflection checks were then carried out and found that the original OSB walls were acceptable in deflection as their inter-story drifts were around 0.17 inches, below the h/400 limit of 0.30 inches.

To replace the GWB shear walls, it was determined that a system needed to be capable of withstanding 475 pounds per foot of shear over 44 feet. The original system could only provide 400 pounds per foot capacity. To accomplish this, it was determined according to the National Design Specification for Wood Construction 2005 that 5/16 inch OSB with 6d nails at 4 inch panel edge spacing was adequate with a capacity of 604 pounds per foot. Nevertheless, when the deflection check was completed, the 5/16 inch OSB shear walls exceeded the 0.30 inches for the h/400 limit with a drift of 0.326 inches. By increasing the panel edge spacing to 3 inches, the deflection due to edge panel spacing reduced from 0.264 inches to 0.198 inches, bringing the total inter-story deflection down to 0.260 inches, under the h/400 limit.

To finish out the design utilizing a system similar to the original, the toe nails that attached the shear walls to the diaphragm needed to be checked. This design was carried out on the 475 pound per foot of shear, equivalent to 633 pounds per 16 inches. Utilizing 16d common nails in Douglas Fir with 1.5 inch lumber, each nail was capable of providing 141 pounds of shear resistance. This amounted to a minimum of 5 nails required every 16 inches. Lastly, the diaphragm was checked for shear capacity and deflection. The original design called for ¾ inch OSB with 8d common nails at a 6 inch panel edge spacing, which provided a shear resistance of 505 pound per linear foot, according to the NDS 2005. At a 10 foot floor to floor height, the 505 pounds per linear foot load can be compared to the wind load by converting to 50.5 pounds per square foot (psf). The diaphragm capacity of 50.5 psf exceeds the design pressure of 39.14 psf, thus the diaphragm is strong enough to support the wind load. For the deflection check, the deflection of the diaphragm calculated to be 0.037 inches, which when combined with the shear wall deflection of 0.26 inches, resulted in a net deflection of 0.297 inches, below the h/400 limit of 0.30 inches.

Steel Gravity System

For comparison, a second system needed to be designed that better manages the threats of termites and decay. For this, a preliminary steel gravity system was first considered and designed, with a steel lateral system to be designed in later calculations and combined with the steel gravity system design. The calculations for the steel gravity system can be found in Appendix C. The loads for the design are 100 pounds per square foot for corridor and public space and 40 pounds per square foot for the rooms.

First, the design was devised to be a steel deck with concrete topping on open web steel joists on wide flange beams and columns. For the rooms, it was assumed that the joists would be spaced less than 5 feet apart which allowed for the use of 1.5C24 Vulcraft deck with 2 inch normal weight concrete topping with 3 span condition. Using a 40 pound per square foot live load and 53.2 pounds per square foot dead load, combine to give a total load of 93.2 pounds per square foot. At 5 feet span, the deck can support and l/240 load of 121 pounds per square foot. Similarly, the corridors were assumed that they would span less than 4 feet, and even with the increased corridor load to 153.2 pounds per square foot total load, 1.5C24 Vulcraft deck with 2 inch normal weight concrete topping was sufficient with 3 span conditions.

Next, the joists for the corridor were designed as both wide flange members and open web steel joists. For a spacing of 4 feet on center and a span of 7.5 feet, the ASD loads of 613 pounds per foot total load far exceeded the capacity of a Vulcraft 10K1 steel joist at 550 pounds per foot. For this spacing, a wide flange shape was then designed for a moment of 6.33 kip-feet and a shear of 3.4 kips, and found that a W8x10 was adequate at carrying the load; however it exceeded the compact limit. To remedy this situation, and to find a shape that better matched the seat height on the joist, a W4x13 was chosen as it surpassed the Zx and Ix requirements of 1.7 inches^4 and 9.48 inches^4, respectively. For the corridors, Vulcraft joist substitutes were checked at a conservative 8 feet, however they proved to be impractical due to the long span and heavy loads. Lastly, a 3 foot spacing was checked for joists and it was determined that a Vulcraft 8K1 open web steel joist would work with a total load of 460 pounds per foot.

For the rooms, the joists would span a maximum length of 27 feet and have to support a total factored load of 640 pounds per foot with 5 feet center to center joist spacing. To keep an equivalent ceiling to floor depth, as was previously design, it was imperative to keep the thickness of joist, deck, and concrete at 20 inches deep or less. With a 3 inch tall deck and concrete cross section, a 17 inch deep or less steel joist must be used. Through this requirement, a Vulcraft 16K9 open web steel joist was the most economical alternative being able to support 658 pounds per foot, greater than 640 pounds per foot.

In addition, at the wall lines, the weight from the wall increased the total load on the joist to 723 pounds per foot, assuming the walls that run parallel to the joists do not transfer any gravity loads. To keep the 17 inch deep requirement, an intermediate column would need to be placed in the wall, which resulted in a 10K1 joist. However, as the member at the wall line was planned to be part of the lateral system, it was then designed to be a wide-flange beam. This resulted in a W12x16 which had a design capacity of 75.4 kip-feet, greater than the design moment of 52 kip-feet. Considering an intermediate column, like the joists, the requirement for a wide flange beams drops to a W10x12 having an Ix of 53.8 inches^4, greater than the required Ix of 12.9 inches^4 and a moment capacity of 46.9 kip-feet, greater than the required 13 kip-feet.

For the columns, it was determined in Appendix C that an interior column located adjacent to the corridor would experience an unreduced load of 45.9 kips and an exterior column would experience a load of 41.1 kips. For a conservative first try and because these columns will need to be designed for wind loads as part of the lateral system, a load of 46 kips is assumed for interior and exterior columns in the wings. Using a pinned-pinned, and braced at each floor assumption, an unbraced length of 10 feet was used and a W8x31 column was found to be sufficient for all the columns in the wings on each floor. A W8x31 has a compressive strength at 10 feet length of 317 kips, greater than the design load of 108 kips.

In the core, the original columns were steel and the design can be reused in the Florida design. To check the columns, beam B25, a random beam in the core, was checked and according to the plans, it had dead load reactions of 5 kips at each end. Assuming this load is a uniformly distributed load and pinned supports, with a tributary width of 12.66 feet, results in the floor weighing 59.2 pounds per square feet (dead load). This exceeds the new steel floor design of 53.2 pounds per square feet (dead load). As such, the current beams and columns in the core are adequate since the original floor was a higher dead load than the new floor.

Steel Braced Frame Lateral System

To continue with a steel framed building, a braced frame system for lateral loads was designed for the loads given in Appendix A and a drift limit of h/400. In the east-west direction, a braced frame would be placed in each wall that separated adjacent suite rooms, the same walls that used a gypsum wall board shear wall in the original design. For the north-south direction, shear walls will be placed in the corridor walls where the braces don't interfere with the current layout of doors.

To start, the member sections found from the steel gravity system design was used for the initial design of the braced frame system. Using STAAD and a flexible diaphragm assumption, a typical braced frame was modeled with an 11 foot ground floor to floor height and 10 foot floor to floor heights for the rest of the structure except the top floor with 8 foot floor to floor height. To simplify the design and ease the construction, it was assumed that all the braced frames in the east-west direction would use the same design. For this assumption, a 22 foot room length, or a 51 foot building length was utilized for design. After the original design was completed, a 27 foot room length, or a 61 foot building length was checked, and found that the forces in each member were less than the forces determined for the 22 foot room length. STAAD outputs for the east-west braced frames can be seen in Appendix D, and the braced frame configuration can be seen in Figure 6.

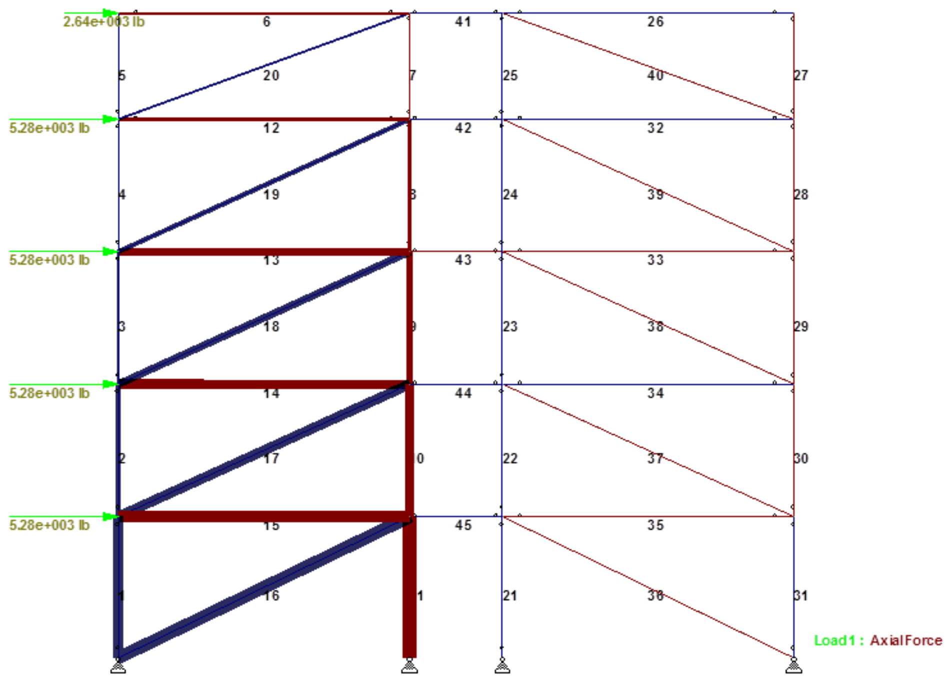

Figure 6: Typical East-West Braced Frame

Once the initial design was completed in STAAD and the loads on each member were determined, each member was designed for the controlling load case. Calculations for each member number can also be seen in Appendix D. For the columns in tension, using a load combination of 0.9D + 1.0W, dead load being a compressive force and wind load causing a tension force, it was determined that none of the columns experience tension due to wind, thus gravity loads with a load combination of 1.2D + 1.6L control. In other words, the columns won't experience tension and thus the footings won't need to be designed for uplift. For the columns that experience compression due to wind, a load combination of 1.2D + 1.6L + 1.0W was utilized for column design. Note, this load combination exceeds the ASCE7-10 load combinations of 1.2D +1.6L and 1.2D + 1.0L +0.5W, but the loads for each column are still under the design strength each column can provide, and the oversized columns will provide easier constructability. For the beams at each floor level, a combined bending and compression loading was determined to control the design with a load combination of 1.2D + 1.0W + 1.0L. The increased compression made the original gravity design inadequate and a W12x35 was found to be the lightest shape acceptable on all floors but the roof level where a W12x30 was adequate. Lastly, the braces were designed as 2 inch by 2 inch angles of differing thicknesses with the lowest brace needing ¼ inch thickness, the second brace needing 3/16 inch thickness, and the rest needing 1/8 inch thickness.

For the north-south direction a similar design scheme to the east-west direction was implemented. It was determined that for the columns that saw an increased compression load, this load did not change the W8x31 column size. In addition, on the beams at each floor level, the combined loading determined that a W12x35 was still the best wide flange section. Lastly, for the angle braces, a 2 inch by 2 inch angle was also used with a 5/16 inch thickness at the lowest brace, the second brace required a thickness of 3/16 inch, and the rest needing 1/8 inch thickness. Figure 7 shows the typical north-south braced frame.

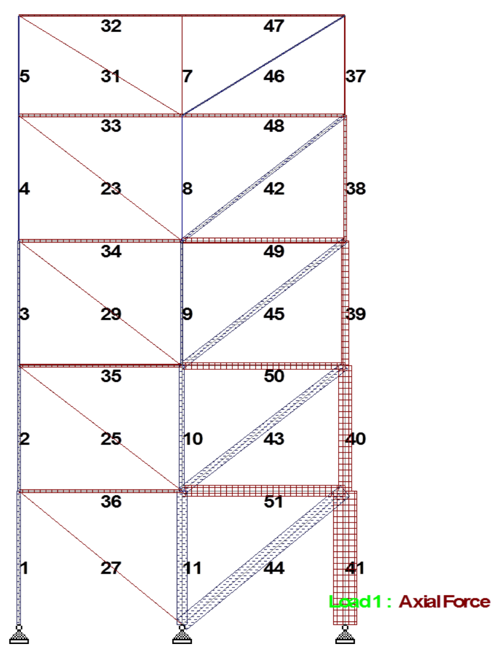

Figure 7: Typical North-South Braced Frame

Comparison

After calculations were complete and both designs finalized, a cost and drift comparison was completed between each system. For the drift comparison, Table 8 describes each system and the drift at the roof level. For the OSB shear walls, drift calculations can be found in Appendix B and for the brace frame, drift was computed using ETABS and the output can be seen in Appendix F.

Table 8 – Drift Comparisons

System	Drift at Roof (inches)	h/400 (inches)
OSB Shear Wall E-W	1.30	1.47
Steel Braced Frame E-W	0.32	1.47
OSB Shear Wall N-S	0.86	1.47
Steel Braced Frame N-S	0.75	1.47

For cost comparison, RS Means Light Commercial Cost Data 2010 was utilized and detailed calculations can be seen in Appendix H. Table 9 shows the cost of each entire system broken down by the gravity system and the walls. The values show that the OSB shear wall system with a wood gravity system cost slightly more than the equivalent braced frame shear wall design with a steel gravity system. This is caused by the wood truss floor being more expensive than an equivalent steel joist floor system.

Table 9 – Cost Comparisons

System	Gravity	Walls	Total
OSB Shear Wall	$1,228,000	$288,000	$1.516 million
Steel Braced Frame	$1,072,000	$261,000	$1.333 million

In addition to cost, durability within Florida should also be compared. With an OSB shear wall system, the entire wooden structure is susceptible to termite damage, decay and rot in the hot and humid climate, all of which can significantly impede the structural performance. On the contrary, the steel building is better suited to resist the effects of the Florida climate.

With the proper paint, even in the high humidity, the steel system will keep its strength and not diminish like wood.

Foundation Consideration

Upon obtaining a geotechnical report near the SCF's location, a typical spread footing was designed for the sandy soil. The geotechnical report was prepared by Ground Down Engineering and found on The Miller Alliance's website (Ground Down Engineering, Inc. 2005). Utilizing a 2 foot by 2 foot concrete pier and 3,000 psi concrete, the footing was first designed for punching shear, as punching shear would control for shear in a square footing with concentric load. The punching shear calculation yielded a required depth of reinforcement of 6.689 inches and a total thickness of 11 inches. For bearing, the width of the footing was determined to be 8 feet wide. Next, the flexure was calculated, resulting in a reinforcement requirement of (12) #5's spaced evenly. Finally, the pier was designed and it was found that (4) #8's were required for bearing capacity. Calculations can be found in Appendix I and a reinforcement diagram can be seen in Figure 8.

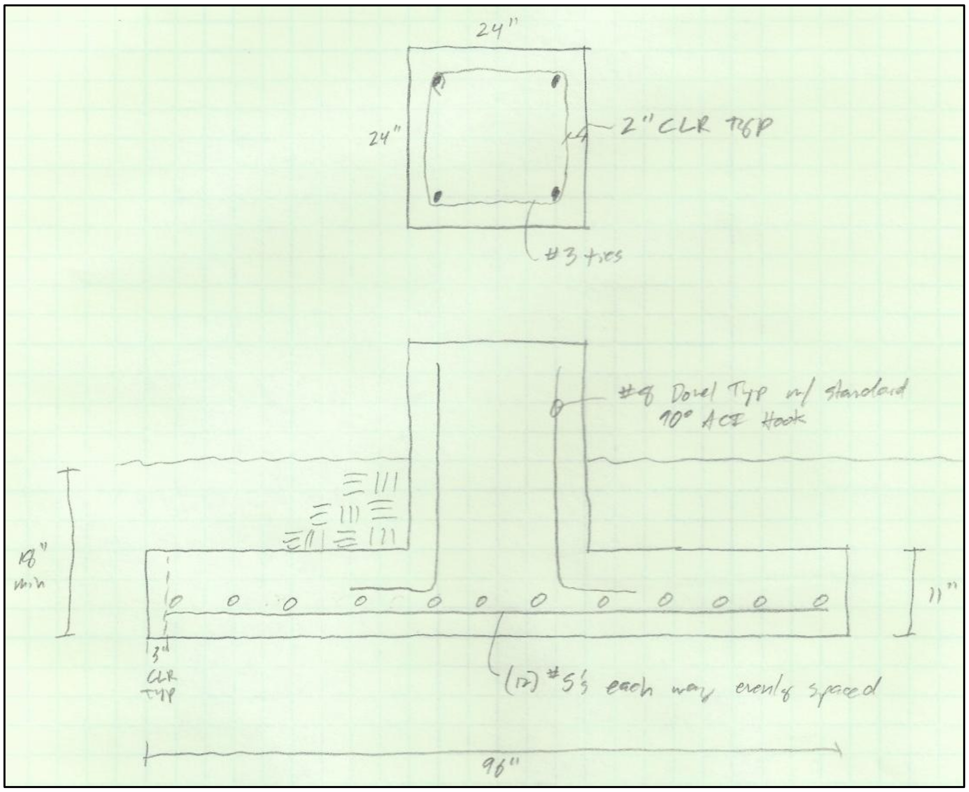

Figure 8: Typical Spread Footing Reinforcement Diagram

In comparison to the current system, the Florida system uses wider foundations in able to not overburden the soil, 8 feet compared to 5 feet. If an 8 foot wide footing proved to be two wide, a rammed aggregate pier system similar to the original building can be implemented. According to a brochure by Geopier, rammed aggregate piers can be used in Florida soils and were used on a project in St. Petersburg, FL. (Geopier Foundation Company) St. Petersburg, FL is within 30 miles of the proposed SCF site, also on the Gulf of Mexico, which one can assume means that the soils at both locations are very similar and thus a rammed aggregate pier system can be used at both locations. This could reduce the footing width back down to 5 feet.

MAE Material Incorporation – Steel Brace Connections

The information gained in AE 534, Steel Connections, was used to design a typical braced frame connection in the east-west direction. Seen in Figure 9 with calculations in Appendix G, the braced frame connection had to be designed for relatively small loads. The angle, as previously designed lateral system calculations, would be attached to a ¼ inch plate. Using a force of 26.4 kips; a 4 inch wide plate yielded a gross area of 1 square inch, greater than the required 0.82 square inches. Next, the weld between the angle and plate was sized by starting with a 1/8 inch weld at 5 inches long. With a C of 3.7 from Table 8-4 in AISC Steel Manual 14th Edition, the Dmin was determined to be 1.9 16ths of an inch, rounded up to 1/8 inch. Continuing with weld design, the weld between the plate and beam was then designed using Table 8-4 and an inclination of 15 degrees, conservative for the actual 24.44 degrees. This yielded two 9 inch long 1/8 inch fillet welds, as the minimum weld size controlled. Lastly, the beam would be attached to the column with a ¼ inch shear tab, 3/16 inch weld, and two ¾ inch A325 bolts. The 2 bolts and shear tab were able to carry 24.5 kips, whereas the required load was 8.7 kips. Due to the configuration and steep angle (24.44°) of the brace, the edge of the plate would need to be placed 4.92 inches from the face of the column for the forces to align at the assumed joint of all three elements. For other brace connections, it is possible that the smaller welds could be used, however, for constructability and to avoid mistakes, all the braces would use the same weld sizes.

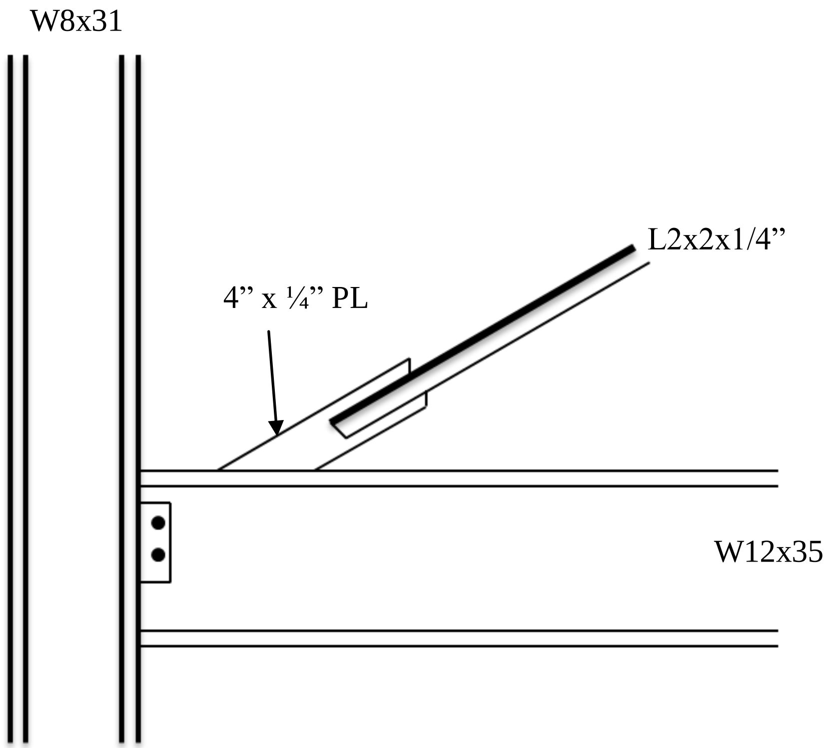

Figure 9: Typical Braced Frame Connection

In addition, information learned in AE 537, Building Performance Failures and Forensic Techniques, and AE 542, Building Enclosure Science and Design, was applied to properly design and detail the façade for impact, pressure and heat resistance, with a discussion found in Breadth Topic 1. Lastly, skills gained in AE 597A, Computer Modeling, were utilized to model the steel braced frame shear walls in both STAAD in the E-W and N-S directions. In addition, a 3-D model in ETABS was created and analyzed using AE597A knowledge to ensure the drift was within the prescribed limits. The STAAD models and outputs can be seen on previous pages as well in Appendices D and E and the ETABS model in Appendix F and Figure 10.

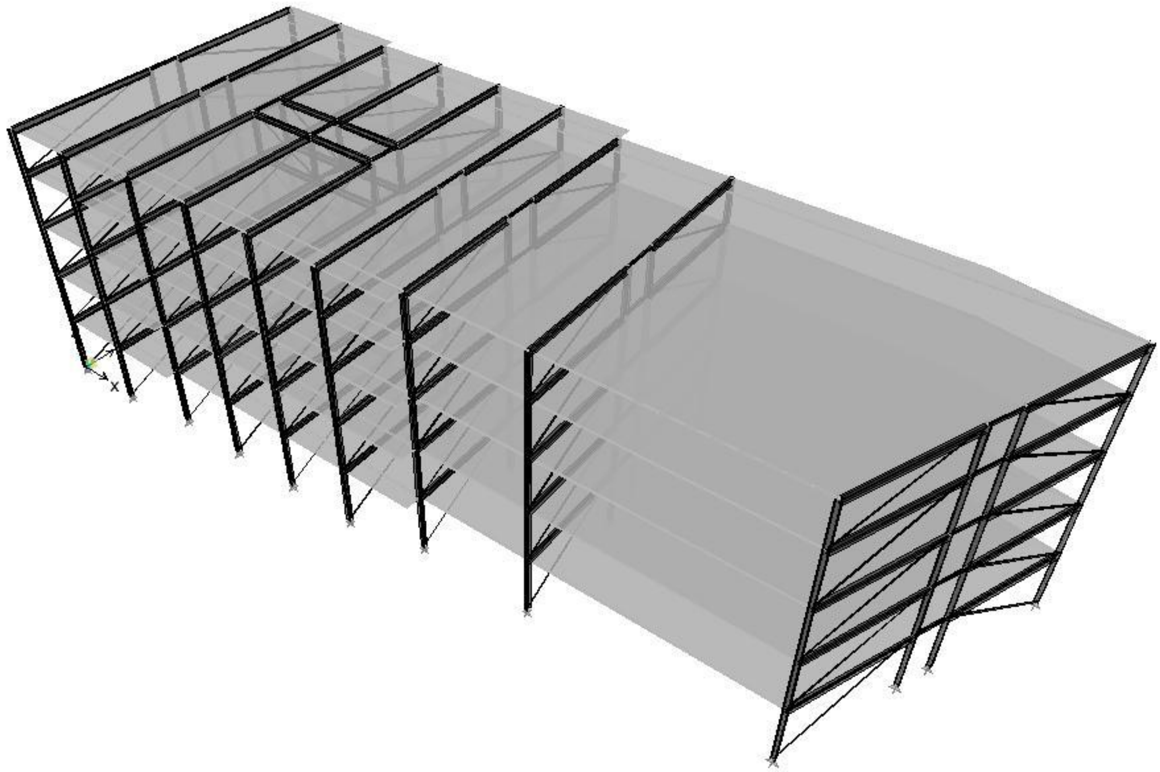

Figure 10: ETABS North Wing 3-D Model

Breadth Topics

Breadth 1: Façade Design

As part of the SCF requirements, a façade designed to resist impacts and the large pressures seen during a hurricane is required. This will include finding a façade system well suited to impacts. Also, the new façade will need to be properly designed and detailed for heat loads and waterproofing.

For the new façade, many systems were researched before determining a probable candidate. A rain screen cladding system that uses wall-cladding panels was chosen for its ability to dissipate heat from the sun and excess water during high rain events. This type of system is prevalent in Europe and thus finding a manufacturer in the United States was a controlling factor in the design. American Fiber Cement (AFC) Corporation's Textura wall cladding system was chosen on the basis of being manufactured in the United States and the only product that AFC produces that is capable of withstanding impacts during hurricanes.

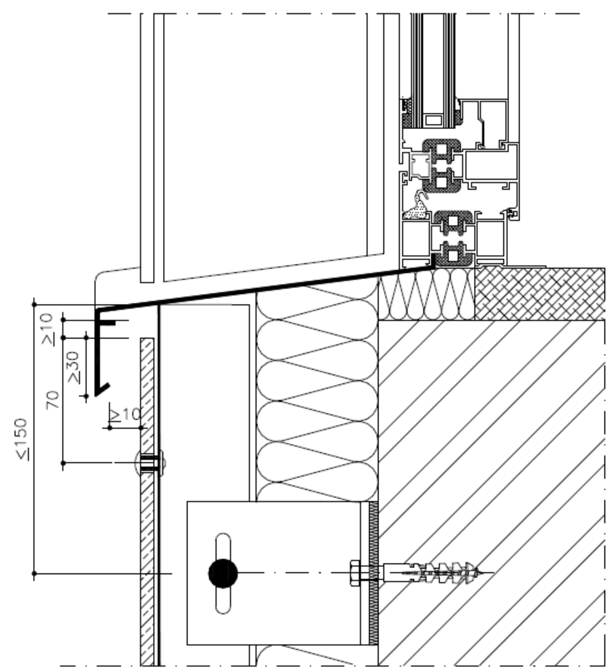

Figure 11: Rain screen section at window sill (image property of American Fiber Cement)

Once a system was chosen, the system was evaluated for its performance in Sarasota, FL. Assuming each layer thickness from Figure 11, the R-value of the system was determined using H.A.M., resulting in an R-value of 14.86. In addition, a condensation analysis also completed using H.A.M. determined that no condensation occurred in the wall with a vapor resistance of 21.751. Printouts from the two analyses can be viewed in Appendix J with a page of the AFC brochure describing the rain cladding system that is used in the Textura product line.

For impact resistance, it is this author's recommendation, in lines with the manufacturer's recommendation that the specification be written with the following requirement. When referring to the exterior cladding, "this product has been designed and tested to comply with the Requirements of the Florida Building Code 2010 edition including High Velocity Hurricane Zone (HVHZ), TAS 202 and TAS 203" (Rain Screen Cladding)

For glazing design under wind pressure and impacts, a "sacrificial ply" design will be implemented. A sample of a "sacrificial ply" design can be seen in Figure 12 where the outer ply is allowed to break while keeping the interior ply intact. With a maximum window size of 5 feet square, or 25 square feet, the glazing must be designed for a wind pressure of 50 pounds per square feet conservatively. According to the simplified procedure used by Minor and Norville in Figure 13, a 5/16 inch laminated glass ply is strong enough to support the necessary wind load. Working this into the sacrificial ply using notes from AE542, any sacrificial ply thickness can be used, however, to make the thinnest possible glazing unit, a 1/8 inch laminated until will be utilized for the sacrificial ply. Lastly, in the insulated glass unit, an air space of 1/16 inch should be used.

An accurate cost comparison of the new façade is unable to be completed based on the idea that the Textura product line and rain cladding in general is very proprietary. In addition, since all research points to overseas production, even though it is an American company, large shipping and import costs will add to the total cost. Lastly, with such a proprietary product, a higher installation cost could be encountered. In comparison, with all the extra costs that a rain cladding system could endure, a brick façade similar to the original is most likely more cost effective.

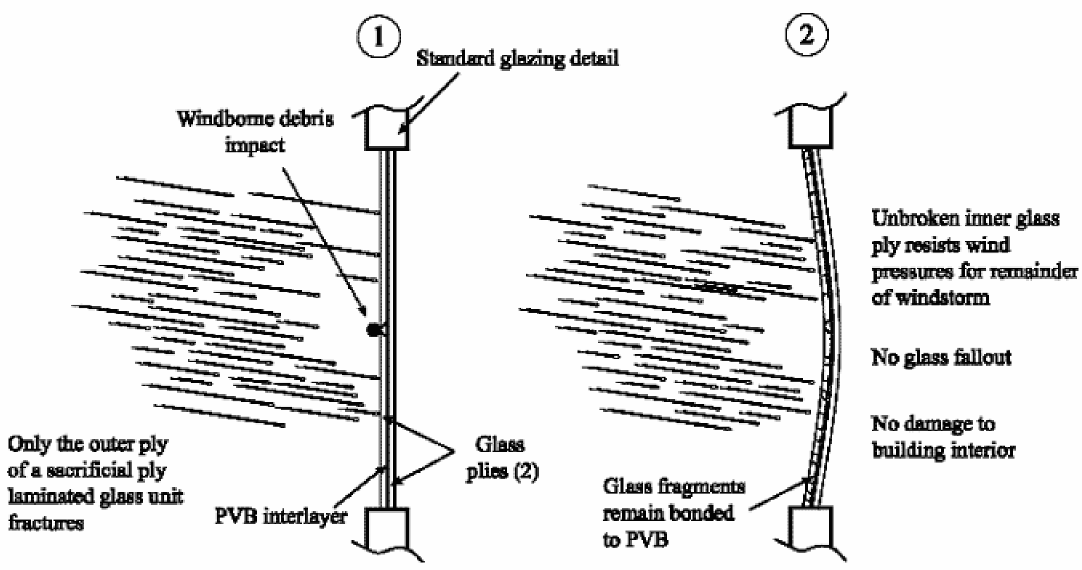

Figure 12: "Sacrificial Ply" Example (image acquired from Penn State University)

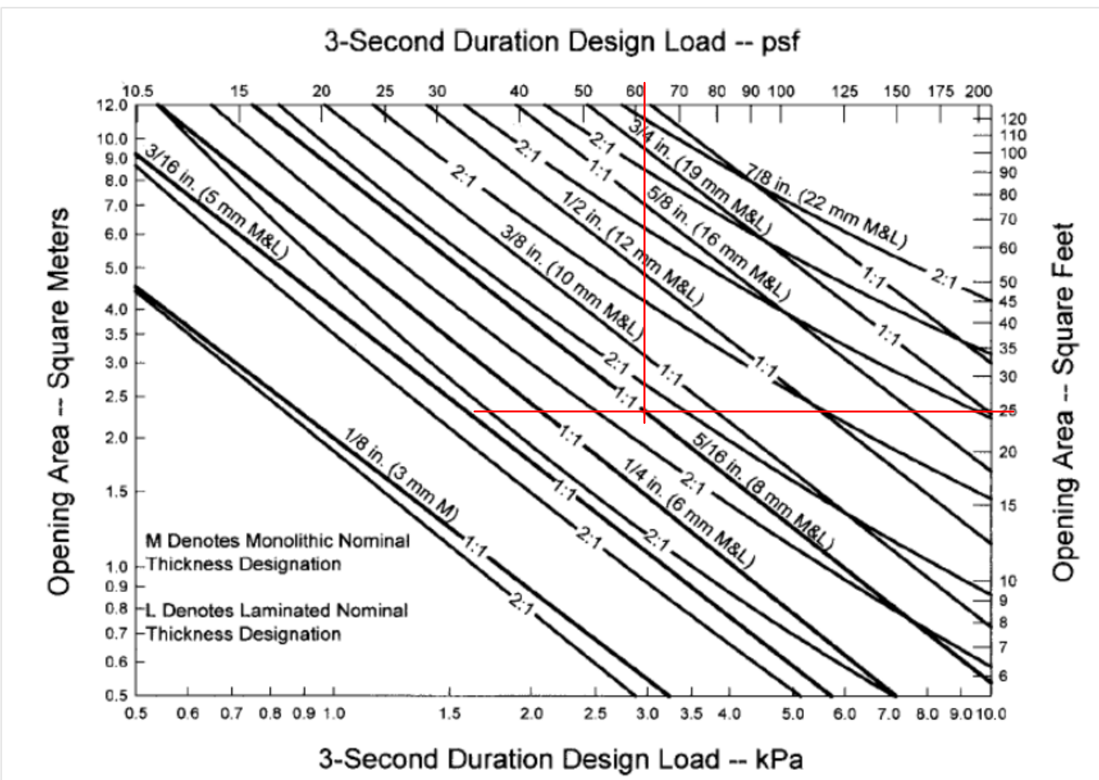

Figure 13: Simplified Glass Design Chart (acquired from Minor and Norville)

Breadth 2: Electrical – Solar Panel Design

As this proposal started with a discussion on global warming, it is imperative that global warming be considered for a breadth topic. To consider global warming, it is this author's desire to design a solar panel system that can partially take the Dormitory off of the grid. To match the SCF's desire to make this a haven for students during a storm, the solar panel system will also double as a backup system should the Dormitory lose power. For this, a proper DC battery system will need to be implemented to power the essential systems of the Dormitory.

For a photovoltaic system, many designs were considered including traditional photovoltaic modules and photovoltaic films. The design that was eventually chosen is a type of a building integrated photovoltaic system, where the photovoltaics act as part of the building enclosure. This was chosen due to the very light weight of the shingles, and the ease of installation, as well as the short payback period and 20 year warranty. Within the last few years, Dow Chemical Company recently invented a new solar shingle which replaces a traditional asphalt shingle. The solar shingle uses "thin-film cells of copper indium gallium diselenide (CIGS), a photovoltaic material that typically is more efficient at turning sunlight into electricity than traditional polysilicon cells" (DOW POWERHOUSE Solar Shingle).

At a payback period of about 10 years according to DOW, half the service life of the solar shingles, and an install time similar to regular shingles, solar shingles are a competitive approach for solar panel design. According to a separate website, a typical home installation of solar shingles would cost about $15,000 or $8,000 more than a traditional asphalt roof. With current users seeing a savings of about $800 a year, it takes 10 years to pay back the initial cost (Lynch-Morin 2011). While DOW's solar shingles are just entering the market, they come with a 20 year warranty as well as certification that they are fire and uplift resistant. As a replacement for asphalt shingles, these certifications are a requirement (Lynch-Morin 2011). In Figures 14 and 15, a typical solar shingle installation can be seen where the shingles are installed very similar to asphalt shingles.

Figure 14: Solar Shingle installation using regular roofing nails (image property of DOW Chemical Company)

Figure 15: Solar Shingle installation using traditional asphalt shingle pattern (image property of DOW Chemical Company)

For the solar shingle system to double as a backup system should the Dormitory ever lose power, a battery backup system would need to be designed. As batteries are most efficient storing DC power, a battery backup system would need to be installed before the inverter. However, if the Dormitory is designed as a safe haven during a hurricane, the battery backup must be designed to handle emergency lighting and medical refrigeration. All in all, a solar shingle system will be an efficient way and cost effective way to remove the Dormitory from the grid.

Conclusions

As global warming is already a design consideration to other engineering disciplines, it will soon be a consideration of the structural engineer. By creating a situation in which a building, previously designed for wind loads of 115 miles per hour, in the Northeast USA, is moved to Florida, where wind loads are increased to 150 miles per hour, larger wind loads due to global warming can be examined. Through two careful redesigns, the effects of the larger loads on the Dormitory were considered.

An oriented strand board shear wall system, similar to the original, proved to be a viable system for the new loads due to the fact that only the walls between the suites were changed from (2) 5/8 inch layers of gypsum wall board to one 5/16 inch layer of oriented strand board. To check a different strategy, a complete redesign of the Dormitory in steel revealed that it too was a reasonable strategy. However, a steel design cost less than the wood design by approximately 12%, but would also not rot and decay like wood.

Lastly, by moving the building to a new climate, a façade breadth was completed in order to provide a building enclosure that allowed for impact resistance and heat mitigation in the Florida climate. These requirements gave way to a rain screen system which when properly detailed can dissipate heat well and provide impact resistance. For glazing, a “sacrificial ply” insulating glass unit, needs a 5/16 inch inner ply for wind pressure and a 1/8 inch outer ply to protect against impacts. In addition, in the new climate, solar power can be utilized and thus an electrical breadth was completed. This study revealed a low cost option in the form of DOW POWERHOUSE solar shingles that can take the Dormitory partially off the grid and provide battery backup in the case of an emergency.

It is this author’s recommendation to use a steel gravity system with steel braced frame shear walls. In the Florida climate, termites, decay, and high moisture all come together to debilitate a structure. By going with the cheaper steel system, the lifetime of the Dormitory can be extended with the use of steel instead of wood.

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Appendix A – Load Determination

Seismic
Load Determination

Site Class → E, soft soil, clay

Risk Category = II

Importance Category = 1.0

Equivalent lateral force procedure

$V = C_S W$

$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$ $\quad S_{DS} = \frac{2}{3} S_{MS}$ $\quad S_{MS} = F_a S_S$

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

According to USGS online

$S_S = .053g$ $\quad S_{MS} = .132g$ $\quad S_{DS} = .088g < .167g$ Table 11.6-1

$S_1 = .028g$ $\quad S_{M1} = .098g$ $\quad S_{D1} = .065g < .067g$ Table 11.6-2

∴ Seismic Design Category = A

Seismic force Resisting System: GWB on wood studs (light frame)

$R = 2.5$ $\quad C_S = .01$ per section 1.4

Base Shear = $W \cdot .01$ (Equation 1.4-1)

$= 3227 \cdot .01 = \boxed{32.27\ k}$ design = 30 k

Overturning Moment = $11.49k \cdot 11' + 6.11 \cdot 21' + 4.69 \cdot 31' + 4.69 \cdot 41' + 5.29 \cdot 49'$

$= \boxed{850\ k\text{-}ft}$

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Basic Wind Speed (mph)=	150
K_d=	0.85
K_{zt}=	1.0

Hip Roof (26.6°)

	K_z	q_z	Gc_{pi} (+)	Gc_{pi} (-)	Gc_{pf}	**p**	
1	0.85	41.616	0.55	-0.55	0.5498	-0.01	**45.77**
2	0.85	41.616	0.55	-0.55	-0.096	**-26.88**	18.89
3	0.85	41.616	0.55	-0.55	-0.447	**-41.49**	4.29
4	0.85	41.616	0.55	-0.55	-0.3904	**-39.14**	6.64
1E	0.85	41.616	0.55	-0.55	0.7274	7.38	53.16
2E	0.85	41.616	0.55	-0.55	-0.1856	-30.61	15.16
3E	0.85	41.616	0.55	-0.55	-0.5844	-47.21	-1.43
4E	0.85	41.616	0.55	-0.55	-0.5344	-45.13	0.65

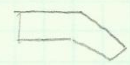
Flat Roof

	K_z	q_z	Gc_{pi} (+)	Gc_{pi} (-)	Gc_{pf}	**p**	
1	0.85	41.616	0.55	-0.55	0.4	-6.24	**39.54**
2	0.85	41.616	0.55	-0.55	-0.69	**-51.60**	-5.83
3	0.85	41.616	0.55	-0.55	-0.37	**-38.29**	7.49
4	0.85	41.616	0.55	-0.55	-0.29	**-34.96**	10.82
1E	0.85	41.616	0.55	-0.55	0.61	2.50	48.27
2E	0.85	41.616	0.55	-0.55	-1.07	-67.42	-21.64
3E	0.85	41.616	0.55	-0.55	-0.53	-44.95	0.83
4E	0.85	41.616	0.55	-0.55	-0.43	-40.78	4.99

Wind
Load Determination

For base shear of entire Structure, assume face dimension is:

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


← N

Overturning = (39.14)(49')(49/2) + 26.98 psf (30')(45') + 41.49 psf (30')(15')

= 101946 lb-ft/ft

= 101946 · 380/1,000 = $\boxed{38{,}740 \text{ k-ft}}$ >> 850 k-ft seismic

wind controls in E-W

For N-S direction, N-wing, least

Overturning = (39.14)(49)(49/2) + 26.98 (30)(130) + 41.49 (30)(15)

= 170500 lb-ft/ft

= 170500 · 60'/1000 = $\boxed{10{,}230 \text{ k-ft}}$ >> 850 k-ft seismic

wind controls in N-S

Base Shear in N-S = 39.14 · 60 · 49/1000 = $\boxed{115 \text{ k}}$ >> 32.27 k seismic

wind controls

Base Shear in E-W = 39.14 · 390 · 49/1000 = $\boxed{729 \text{ k}}$ >> 32.27 k seismic

wind controls

Moment Resistance due to Self-Weight

E-W: 3227 k · 30' = 96,810 k-ft > 38,740 k-ft ∴ ok

N-S: 3227 k · 380/2 = 613,000 k-ft > 10,230 k-ft ∴ ok

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Appendix B – OSB Redesign

Lateral Design in Wings, GWB & OSB

North Wing
- Assume Flexible Diaphragm

N-S (worst case)

1/2" OSB takes $(27/2)(33)(39.14) = 17437$ lbs shear $= 17.4\text{ k} = 164.5$ lb/ft over 106'

3/4" OSB takes $(27/2 + 8/2)(33)(39.14) = 22603$ lbs shear $= 22.6\text{k} = 262.8$ lb/ft

Shear Capacity
1/2" OSB $= 670 \cdot .8 = 536$ lb/ft > 164.5 lb/ft ∴ ok
3/4" OSB $= 950 \cdot .8 = 760$ lb/ft > 262.8 lb/ft ∴ ok

E-W (worst case)

1/2" OSB takes $\left(\frac{13.66}{2}\right)(33)(39.14) = 8.82\text{ k}$ Over 52' $= 170$ lb/ft < 335 ∴ ok
(2) 5/8" GWB takes $\left(\frac{19.33}{2} + 13/2\right)(33)(39.14) = 20.9\text{ k}$ Over 44' $= 474.5$ lb/ft

Shear Check
(2) 5/8" GWB $= 500 \cdot .8 = 400$ lb/ft < 474.5 Not ok

∴ $\frac{474.5}{.8} = 593$ ∴ Use 5/16" OSB w/ 6d nails at 4" Panel Edge Spacing

$= 755 \cdot .8 = 604 > 474.5$ ∴ ok

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

Deflection Checks

1/4" assumed, not a controlling factor

1/2" OSB $= \Delta = \frac{8 \cdot 170 \cdot 10^3}{1.6E6 \cdot 10.5 \cdot 52} + \frac{170 \cdot 10}{1000 \cdot 15} + \frac{10 \cdot (.25)}{52} =$

$= .001557 + .11333 + .04807 = .163$ in
$h/400 = 10 \cdot 12/400 = .3" > .163"$ ∴ ok

3/4" OSB $= \Delta = \frac{8 \cdot 262.8 \cdot 10^3}{1.6E6 \cdot 16.5 \cdot 96} + \frac{262.8 \cdot 10}{1000 \cdot 19} + \frac{10(.25)}{96}$

$= .000925 + .138316 + .02907 = .168$ in $< h/400 = .3$
∴ ok

5/16" OSB $= \Delta = \frac{8 \cdot 474.5 \cdot 10^3}{1.6E6 \cdot 10.5 \cdot 44} + \frac{474.5 \cdot 10}{1000 \cdot 18} + \frac{10(.25)}{44}$

$= .005135 + .263611 + .0568 = .326 > h/400 = .3$ ∴ No

increase to 3" fastener edge spacing $\frac{474.5 \cdot 10}{1000 \cdot 24} = .1977 \Rightarrow .260 < .3$ ∴ ok

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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 $\left(\frac{18.66}{2} + \frac{18.66}{2}\right)(33)(37.14) = 22641$ over 51' = 444 lb/ft

North wing is worst case

Use 5/16" OSB with 3" fastener edge spacing

Check Toe Nails

Worst case shear = 474.5 lb/ft = 633 lb/16"

16d common nails in Douglas Fir w/ 1½" lumber = 141 lb/nail

$\frac{633}{141} = 4.5 \approx \boxed{5 \text{ nails per } 16"}$

Diaphragm Check

Try original: 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 > 39.14 psf wind ∴ ok

Deflection

$$\delta = \frac{5 \cdot 391.4 \cdot 44^3}{8 \cdot 1.6E6 \cdot 18.66 \cdot (1\frac{1}{4} \cdot 18)} + \frac{.25 \cdot 391.4 \cdot 44}{1000 \cdot 670} + 0$$

$$= .03102 + .006426 = .037$$

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

Appendix C – Steel Gravity Design

Gravity Design - Deck

Loads
Gypcrete (3/4") = 7.2 psf
Carpet = 2 psf
MEP = 4 psf
Ceiling = 1 psf
14.2 psf

Rooms
Span ≤ 5'

Use 1.5C Vulcraft Deck
Try t = 2.00" ⇒ 3.5" slab thickness Weight = 37 psf

37 + 14.2 + 2 = 53.2 psf = dead
(deck)

SDI check
3 span = 7'2" > 5' ∴ ok

Live = 40 psf

Total = 93.2 psf

3 span yields f'_b = 142 > 93.2 ok
$\ell/240$ = 121 > 93.2 ok
$\ell/180$ = 161 > 53.2 ok

For Rooms: use 1.5C24 Vulcraft Deck with 2" NW concrete spanning 5' or less.

Corridor
Span ≤ 2.5'

SDI Check
3 span = 7'2" > 4' ∴ ok

Try same deck as rooms

Dead = 53.2 psf
Live = 100 psf
Total = 153.2 psf

3 span yields f_b = 220 > 153.2 ok
$\ell/240$ = 236 > 153.2 ok
$\ell/180$ = 314 > 53.2 ok

For Corridor: use 1.5 C24 Vulcraft Deck with 2" NW concrete spanning 4' or less

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Gravity Design - Joists

Wings - Corridor

Loads

	@ 4'	@ 3'
Dead = 53.2 psf	213 plf	160 plf
Live = 100 psf	400 plf	300 plf
Total = 153.2 psf	613 plf	460 plf

613 plf > 550 plf for a 10K1 ∴ More economical to use beams

Span = 7.5'

LRFD

$w_u = 1.2(.213) + 1.6(.4) = .8956 = .9\ k/ft$

$M_u = \frac{.9 \cdot 7.5^2}{8} = \boxed{6.33\ k\text{-}ft}$ $\qquad V_u = \frac{wl}{2} = \frac{.9 \cdot 7.5}{2} = 3.4\ k$

$\Delta_{max} = L/360 = 7 \cdot 12/360 = .233\ in$

conservative

$I_{x\,req} = \frac{5wl^4}{384 E \Delta_{max}} = \frac{5 \cdot .9 \cdot 7.5^4 \cdot 12^3}{384 \cdot 29000 \cdot .233} = 9.48\ in^4$

From Table 3-3, $I_x = 30.8 > 9.5$ ∴ ok, use W8x10

Check: $\phi M_n = 37.9 > 6.33$ ∴ ok
$\phi V_n = 40.2 > 3.4$ ∴ ok

Check for better section: $M_n = M_p = F_y Z_x$ Fully Braced
$\phi M_n = \quad = 50ksi \cdot Z_x \cdot .9$

$\phi M_n > M_u \qquad Z_{x\,min} = \frac{6.33 \cdot 12}{.9 \cdot 50} = 1.7\ in^3$

Try W4x13 $\quad Z_x = 6.28 > 1.7$ & $I_x = 11.3 > 9.48$ ∴ ok
does not exceed compact limit.

Joist Substitutes: Span ≥ 8' impractical due to heavy loads

Check 3' Spacing

LRFD $w_u = 1.2(.16) + 1.6(.3) = .672 = 672\ plf < 825$ for 8K1 ∴ ok

$W_L = 1.6 \cdot .3 = .48 = 480 < 550\ plf$ ∴ ok

$\boxed{\text{Use 8K1 spaced at 3' OC}}$

Gravity Designs - Joists

Rooms - Wings

LRFD
Loads @ 5'

Dead: 53.2 psf 266 plf
Live = 40 psf 200 plf
Total factored = 128 psf 640 plf

Span ≤ 27'
Start in Joist Guide table with total = 466 ⇒ 24K6 @ 503

For equivalent floor to floor & floor to ceiling heights, limit depth to 20" - 3" = 17" deep

Use 16K9 = 658 > 640 ∴ ok
246 > 200 ∴ ok

Use 16K9 @ 5' spacing or less

Room - Walls - Wings

Loads = Loads above + walls

Steel Stud = 1 psf AISC
Drywall (3) 5 psf AISC
Misc = 1 psf
7 psf @ 10' tall = 70 plf

Dead = 266 + 70 = 336 plf
Total = 336 · 1.2 + 200 · 1.6 = 723 plf ∴ too much for 16" depth

Use an additional column within walls ∴ Span ≤ 14'

Use 10K1 at walls with intermediate column

Consider Beams

$M_u = \frac{.723 \cdot 24^2}{8} = 52$ k-ft $V_u = \frac{.723 \cdot 24}{2} = 8.7 < 79.2$ ∴ ok

$\Delta_{max} = \ell/360 = 24 \cdot 12/360 = .8"$
$I_{x\,req} = \frac{5 \cdot .32 \cdot 24^4 \cdot 12^3}{384 \cdot 29000 \cdot .8} = 103$ in⁴ = 103 ∴ ok

From table 3-2, $\boxed{W12 \times 16}$ $\phi M_n = 75.4 > 52$ ∴ ok

If intermediate columns: $M_u = \frac{.723 \cdot 12^2}{8} = 13$ k-ft < 46.9 ok
$I_x = 12.9$ in⁴ < 53.8 ok

Use $\boxed{W10 \times 12}$

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Gravity Design - Columns

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Rooms - Wings

Column at exterior wall: Tributary width = $27.66/2 = 13.83'$, length = $27/2 = 13.5'$ > Area = 186.7 ft²

$K_{LL} = 2$

373.4 < 400
No reduction

LRFD

Load: D = 53.2 psf
L = 40 psf
T_f = 128 psf

$128 \cdot 186.7 = 23{,}900 \text{ lb} = \boxed{23.9 \text{ kips}}$

$23.9k + \frac{723}{1000}_{\text{wall}} \cdot 27/2 = 33.7 \text{ kips} + 27.66/2 \cdot 54 \cdot 10'/1000 = \boxed{41.1 \text{ kips}}$

Exterior Wall:
Gypsum: 3 psf
Steel Stud: 1 psf
Brick: 40 psf
≈ 45 psf · 1.2 = 54 psf

Wings - Corridor

$$23.9k + \frac{27.66/2}{1000} \cdot \left[1.2 \cdot 53.2 + 1.6 \cdot 100\right] \cdot 3.5' + \frac{723}{1000} \cdot 27/2 + \frac{27.66}{2} \cdot 10 \cdot 10'/1000$$

$= \boxed{45.9 \text{ kips}}$

Interior Wall:
Gypsum: 6 psf
Studs: 1 psf
≈ 8 psf · 1.2 ≈ 10 psf

Column Design: Use 46 kips for both design cases, per floor

K = 1.0 pinned - pinned
L = 31' ground to 3rd floor, braced y-y at floors
L = 31' 3rd Floor to roof, braced y-y at floors

$KL_y = 31$

Required Strength: Roof = 16.5 psf · 1.2 + 30 psf · 1.6 = 68 psf
* Assume redesigned roof in steel weighs less than original design plus snow.
= 68 psf · 235 ft² = 16 k
4th Floor = 46 k
3rd Floor = 46 k
$\boxed{108 k}$

2nd Floor = 46 k
1st Floor = 46 k
Ground = 46 k
$\boxed{138k} + 108k = \boxed{246k}$

Gravity Design - Column

For 108k, try a W8 x 31 ϕP_n = 317 @ 10'

r_x/r_y = 1.72 $KL = \frac{31}{1.72} = 18'$ ϕP_n = 178 @ 18' > 108 ok

* Assume also braced in x-x

For 246k, try a W8 x 31 ϕP_n = 317 @ 10' > 246k ∴ ok

* Assume also braced in x-x

Corridor - Check: Beam B25, from tech 1, Dead Load reactions of 5 kips
Core

Assume continuous load: $\frac{5.2}{13.33}$ = .75 k/ft = 750 lb/ft

750/12.66 = 59.2 lb/ft²

New Floor = 53.2 psf < 59.2 psf ∴ OK

For Core: use beams & columns as designed for original design.

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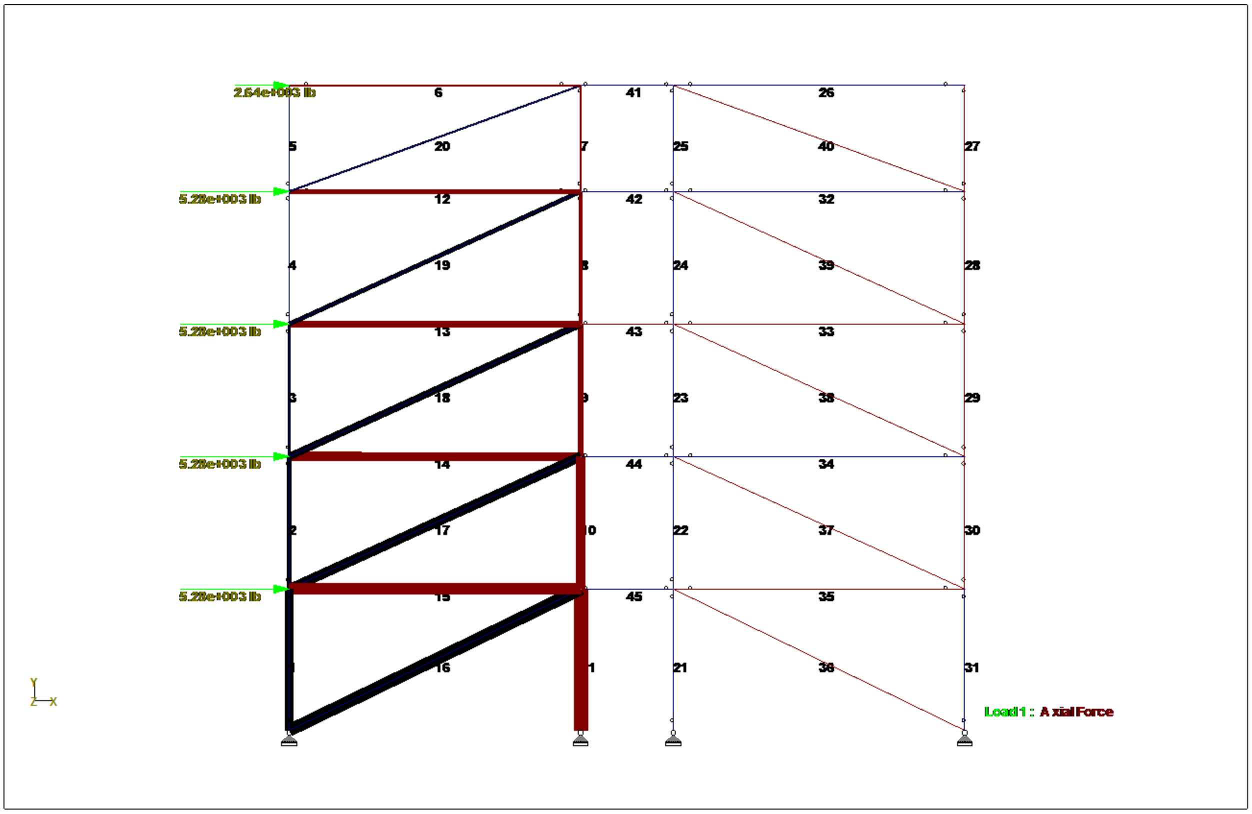
Appendix D – East-West Braced Frame Design



Typical Braced Frame in the East-West Direction, showing beam numbers and the loads and member calculations on the following pages are designated by the beam numbers

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Beam End Forces for Typical Braced Frame in East-West Direction

Beam	Node	L/C	Axial Fx (lb)
1	1	1:LOAD CASE 1	-19E 3
	2	1:LOAD CASE 1	19E 3
2	2	1:LOAD CASE 1	-10.6E 3
	3	1:LOAD CASE 1	10.6E 3
3	3	1:LOAD CASE 1	-4.56E 3
	4	1:LOAD CASE 1	4.56E 3
4	4	1:LOAD CASE 1	-960.763
	5	1:LOAD CASE 1	960.763
5	5	1:LOAD CASE 1	-0.084
	6	1:LOAD CASE 1	0.084
6	6	1:LOAD CASE 1	2.64E 3
	12	1:LOAD CASE 1	-2.64E 3
7	12	1:LOAD CASE 1	960.728
	11	1:LOAD CASE 1	-960.728
8	11	1:LOAD CASE 1	4.56E 3
	10	1:LOAD CASE 1	-4.56E 3
9	10	1:LOAD CASE 1	10.6E 3
	9	1:LOAD CASE 1	-10.6E 3
10	9	1:LOAD CASE 1	19E 3
	8	1:LOAD CASE 1	-19E 3
11	8	1:LOAD CASE 1	30.5E 3
	7	1:LOAD CASE 1	-30.5E 3
12	5	1:LOAD CASE 1	7.93E 3
	11	1:LOAD CASE 1	-7.93E 3
13	4	1:LOAD CASE 1	13.2E 3
	10	1:LOAD CASE 1	-13.2E 3
14	3	1:LOAD CASE 1	18.5E 3
	9	1:LOAD CASE 1	-18.5E 3
15	2	1:LOAD CASE 1	23.8E 3
	8	1:LOAD CASE 1	-23.8E 3
16	1	1:LOAD CASE 1	-26.4E 3
	8	1:LOAD CASE 1	26.4E 3
17	2	1:LOAD CASE 1	-20.3E 3
	9	1:LOAD CASE 1	20.3E 3
18	3	1:LOAD CASE 1	-14.5E 3
	10	1:LOAD CASE 1	14.5E 3
19	4	1:LOAD CASE 1	-8.71E 3
	11	1:LOAD CASE 1	8.71E 3
20	5	1:LOAD CASE 1	-2.81E 3
	12	1:LOAD CASE 1	2.81E 3

*All other members are zero force members.

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Braced Frame Members

Under Wind Load (member #)

1) $-19k\ W \cdot 1.0 + .9(41.1 - 40 \cdot 1.6 \cdot \frac{96.7}{1000}) = 7.23k$ – Not Worst Case

2), 3), 4), 5) Not Worst Case (tension caused by wind, doesn't overcome dead load)

6) $1.2D + 1.0W + L$ = combined loading: W12x16 $p = \cancel{\frac{1}{21 \times 10^{-3}}}$

W12x26 $p = 21 \times 10^{-3} \cdot 2.64k = .055 < .2$

$\frac{1}{2} \cdot 21 \times 10^{-3} \cdot 2.64 + \frac{9}{8}(20.8 \times 10^{-3} \cdot 52) = 1.24 > 1.0$ No

$\boxed{W12x30}$ $pP_r = 18.1 \times 10^{-3} \cdot 2.64 \leq .2$ $\frac{1}{2} \cdot 18.1 \times 10^{-3} \cdot 2.64 + \frac{9}{8}(16.3 \times 10^{-3} \cdot 52) = .977 < 1.0$ ok

7) $.96k + 45.9 \cong 47k < 178k$ ok

8) $4.6k + 16k + 46k = 66.6 < 178k$ ok

9) $10.6k + 16k + 46k + 46k = 118.6 < 178k$ ok

10) $19k + 16k + 46k \cdot 3 = 173 < 178k$ ok

11) $30.5k + 16k + 46k \cdot 4 = 230.5 < 317k$ ok

12) Try W12x30 $pP_r = 18.1 \times 10^{-3} \cdot 7.93 = .144 < .2$

$\frac{1}{2} \cdot 18.1 \times 10^{-3} \cdot 7.93 + \frac{9}{8}(16.3 \times 10^{-3} \cdot 52) = 1.02 \geq 1.0$ No

$\boxed{W12x35} = \frac{1}{2} \cdot 15 \times 10^{-3} \cdot 7.93 + \frac{9}{8}(12.2 \times 10^{-3} \cdot 52) = .77 \leq 1.0$ ok

13) $\boxed{W12x35}$ $pP_r = 15 \times 10^{-3} \cdot 13.2 = .198 < .2$

$\frac{1}{2} \cdot 15 \times 10^{-3} \cdot 13.2 + \frac{9}{8}(12.2 \times 10^{-3} \cdot 52) = .81 < 1.0$ ok

14) $\boxed{W12x35}$ $pP_r = 15 \times 10^{-3} \cdot 18.5 = .277 > .2$

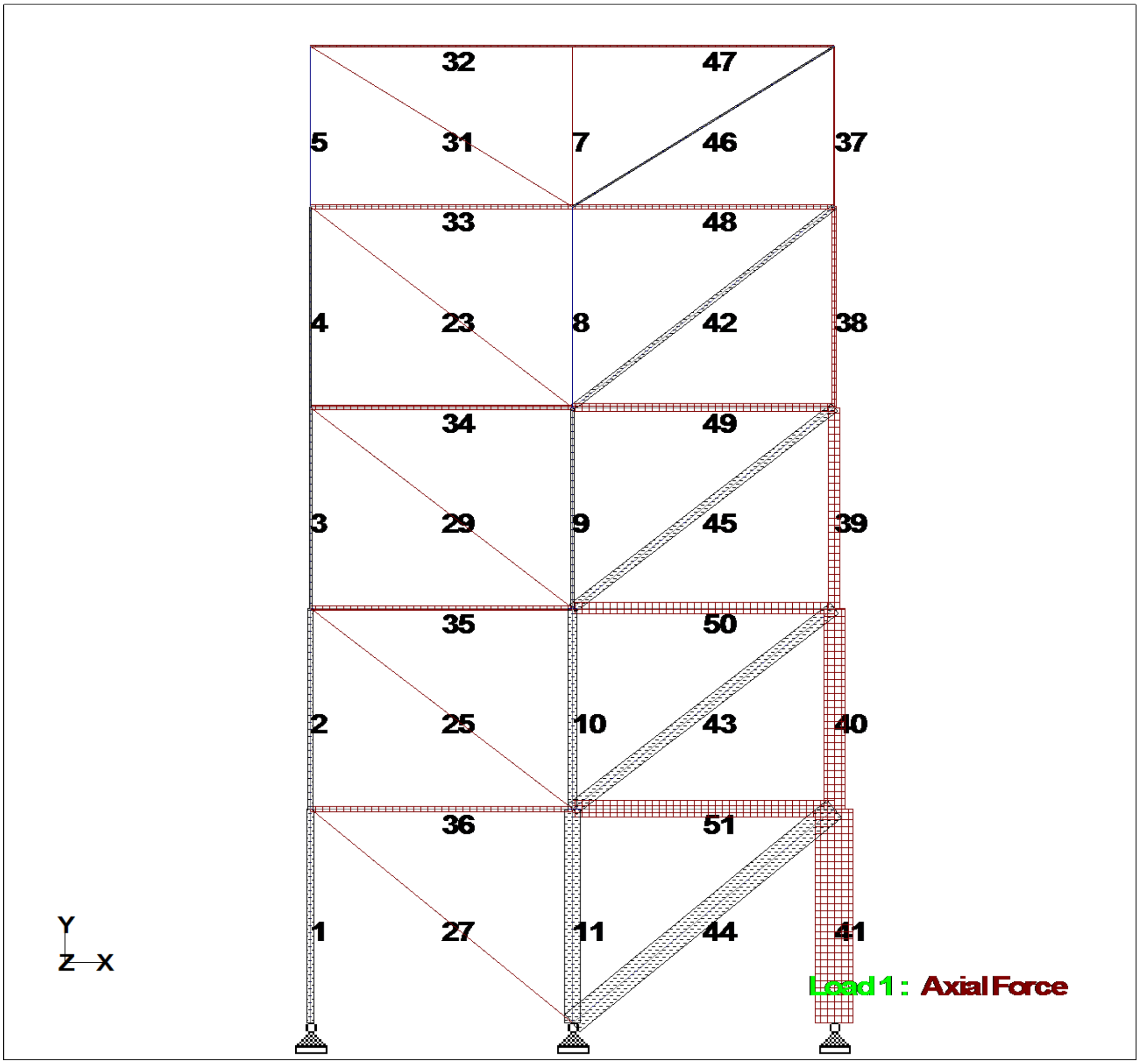
$15 \times 10^{-3} \cdot 18.5 + (12.2 \times 10^{-3} \cdot 52) = .911 < 1.0$ ok

15) $\boxed{W12x35}$ $pP_r = 15 \times 10^{-3} \cdot 23.8 \geq .2$

$15 \times 10^{-3} \cdot 23.8 + (12.2 \times 10^{-3} \cdot 52) = .991 \leq 1.0$ ok

16) Angles · $W = 26.4k$ $\phi P_n = 30.6$ L2x2x1/4
17) $W = 20.3k$ $\phi P_n = 23.4$ L2x2x3/16
18) $W = 14.5k$ $\phi P_n = 15.9$ L2x2x1/8
19) $W = 8.7k$ $\phi P_n = 15.9$ L2x2x1/8
20) $W = 2.8k$ $\phi P_n = 15.9$ L2x2x1/8

Appendix E–North-South Braced Frame Design



Typical Braced Frame in the North-South Direction, showing beam numbers and the loads and member calculations on the following pages are designated by the beam numbers

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Beam End Forces for Typical Braced Frame in North-South Direction

Beam	Node	L/C	Axial Fx (kip)
1	1	1:LOAD CASE 1	-10.369
	2	1:LOAD CASE 1	10.369
2	2	1:LOAD CASE 1	-6.889
	3	1:LOAD CASE 1	6.889
3	3	1:LOAD CASE 1	-4.012
	4	1:LOAD CASE 1	4.012
4	4	1:LOAD CASE 1	-1.897
	5	1:LOAD CASE 1	1.897
5	5	1:LOAD CASE 1	-0.622
	6	1:LOAD CASE 1	0.622
7	12	1:LOAD CASE 1	0.495
	11	1:LOAD CASE 1	-0.495
8	11	1:LOAD CASE 1	-0.074
	10	1:LOAD CASE 1	0.074
9	10	1:LOAD CASE 1	-4.524
	9	1:LOAD CASE 1	4.524
10	9	1:LOAD CASE 1	-12.742
	8	1:LOAD CASE 1	12.742
11	8	1:LOAD CASE 1	-24.633
	7	1:LOAD CASE 1	24.633
23	10	1:LOAD CASE 1	0.000
	5	1:LOAD CASE 1	0.000
25	8	1:LOAD CASE 1	0.000
	3	1:LOAD CASE 1	0.000
27	2	1:LOAD CASE 1	0.000
	7	1:LOAD CASE 1	0.000
29	4	1:LOAD CASE 1	0.000
	9	1:LOAD CASE 1	0.000
31	6	1:LOAD CASE 1	0.000
	11	1:LOAD CASE 1	0.000
32	6	1:LOAD CASE 1	2.808
	12	1:LOAD CASE 1	-2.808
33	11	1:LOAD CASE 1	6.289
	5	1:LOAD CASE 1	-6.289
34	4	1:LOAD CASE 1	6.296
	10	1:LOAD CASE 1	-6.296
35	9	1:LOAD CASE 1	6.461
	3	1:LOAD CASE 1	-6.461
36	2	1:LOAD CASE 1	7.554
	8	1:LOAD CASE 1	-7.554
37	13	1:LOAD CASE 1	1.296
	14	1:LOAD CASE 1	-1.296
38	14	1:LOAD CASE 1	7.003
	15	1:LOAD CASE 1	-7.003
39	15	1:LOAD CASE 1	17.225
	16	1:LOAD CASE 1	-17.225
40	16	1:LOAD CASE 1	31.894
	17	1:LOAD CASE 1	-31.894
41	17	1:LOAD CASE 1	56.550
	18	1:LOAD CASE 1	-56.550
42	14	1:LOAD CASE 1	-8.253
	10	1:LOAD CASE 1	8.253
43	16	1:LOAD CASE 1	-20.114

	8	1:LOAD CASE 1	20.114
44	7	1:LOAD CASE 1	-33.969
	17	1:LOAD CASE 1	33.969
45	9	1:LOAD CASE 1	-14.252
	15	1:LOAD CASE 1	14.252
46	11	1:LOAD CASE 1	-2.230
	13	1:LOAD CASE 1	2.230
47	12	1:LOAD CASE 1	1.950
	13	1:LOAD CASE 1	-1.950
48	14	1:LOAD CASE 1	7.322
	11	1:LOAD CASE 1	-7.322
49	10	1:LOAD CASE 1	11.832
	15	1:LOAD CASE 1	-11.832
50	16	1:LOAD CASE 1	16.598
	9	1:LOAD CASE 1	-16.598
51	8	1:LOAD CASE 1	25.454
	17	1:LOAD CASE 1	-25.454

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Braced Frame Members

N-S, corridors

37) 1.3 + 16k = 17.3k

38) 7.0 + 16k + 46k·1 = 69k

39) 17.2 + 16k + 46k·2 = 125.2k

40) 31.9 + 16k + 46k·3 = 185.9k

all less than 317k, ok as designed $\boxed{W8 \times 31}$

41) 56.55 + 16k + 46k·4 = 257k

51) $\rho P_r = 5.15\times10^{-3} \cdot 25.5k \cdot .2 = .13 <$ $\boxed{W12 \times 35}$

$\frac{1}{2} 5.15\times10^{-3} \cdot 25.5 \left(\frac{\cancel{12.2}}{6.5} \times 10^{-3} \cdot 75 \right) = \cancel{1.3} > 1.0$ ~~No~~
$= .55 > 1.0$ ok

50, 49, 48, 47, ok by previous calc.

44)	Angles	w = 33.9	ϕP_n = 37.6	L 2x2x 3/16
43)		w = 20.1	ϕP_n = 23.4	L 2x2x 3/16
45)		w = 14.3	ϕP_n = 15.9	L 2x2x 1/8
42)		w = 6.3	ϕP_n = 15.9	L 2x2x 1/8
46)		w = 2.2	ϕP_n = 15.9	L 2x2x 1/8

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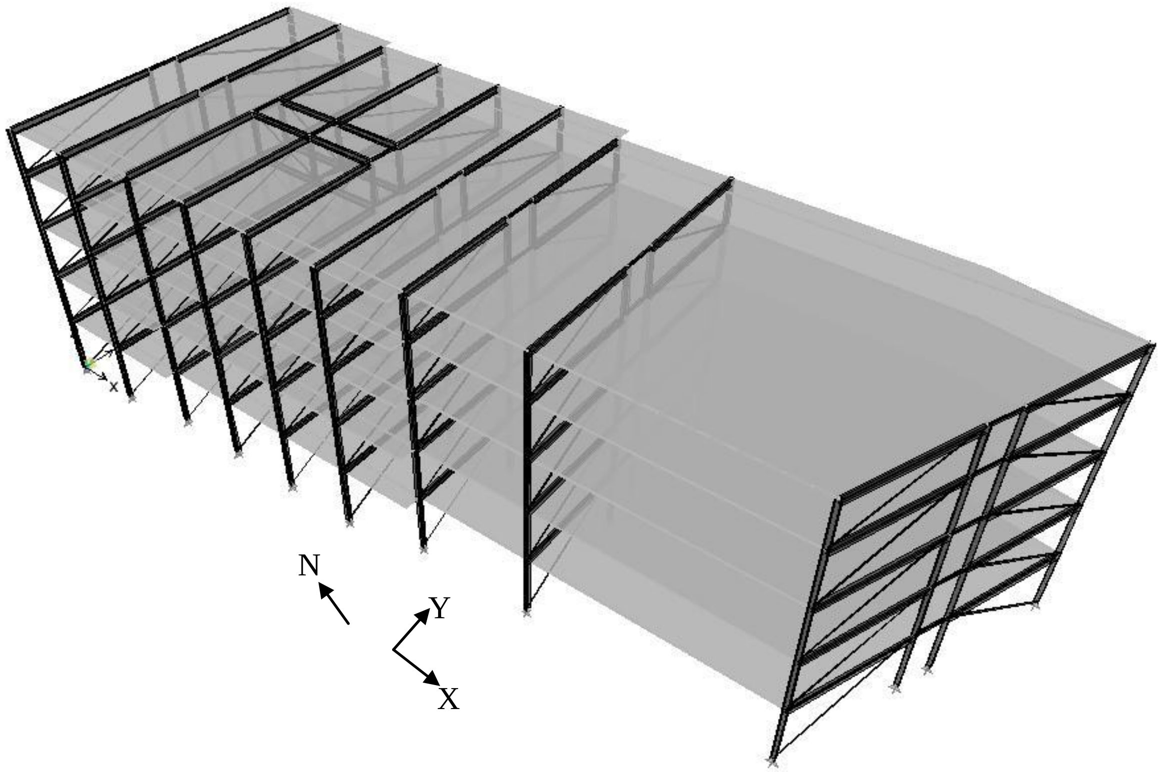
Appendix F – ETABS Output



ETABS Displacement in E-W direction

STORY	DISP-X (in)	DISP-Y (in)	DRIFT-X (in)	DRIFT-Y (in)
ROOF	-0.323689	0.309974	0.000274	0.000228
STORY4	-0.297428	0.288053	0.000409	0.000360
STORY3	-0.248358	0.244817	0.000545	0.000519
STORY2	-0.182917	0.182577	0.000671	0.000673
STORY1	-0.102429	0.101825	0.000776	0.000771

ETABS Displacement in N-S direction

STORY	DISP-X (in)	DISP-Y (in)	DRIFT-X (in)	DRIFT-Y (in)
ROOF	0.751618	0.056716	0.000640	0.000047
STORY4	0.690201	0.052186	0.000958	0.000071
STORY3	0.575240	0.043632	0.001263	0.000095
STORY2	0.423727	0.032223	0.001541	0.000117
STORY1	0.238794	0.018167	0.001809	0.000138

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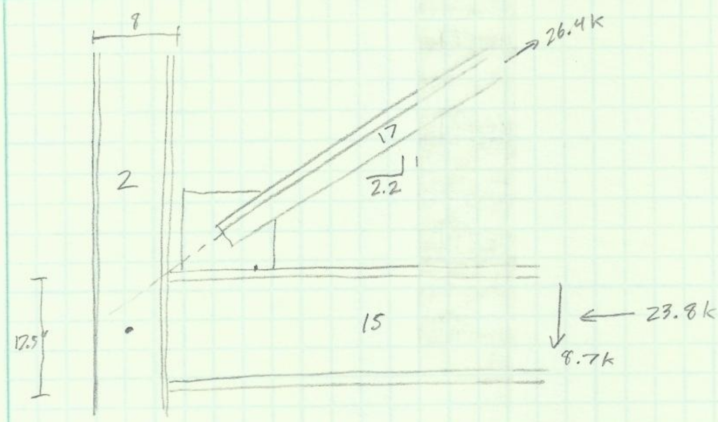
Appendix G – Braced Frame Connection

Brace Design Connection



Angle

$W = 26.4k$, $\phi P_n = 30.6k$, use $\angle 2 \times 2 \times 1/4$

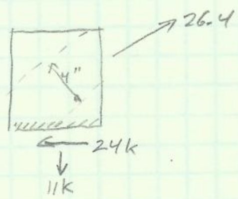
least of yielding & rupture

Slenderness

$12 \cdot 24.2 / .391 = 743 > 300$ (r_z)

Not a requirement, but will consider abiding by this in design.

Plate



$\phi P_n = .9 F_y A_g$

$= .9(36) \cdot A_g \geq 26.4$

$A_g = .815 \text{ in}^2$

$\phi P_n = .75 F_u A_e$

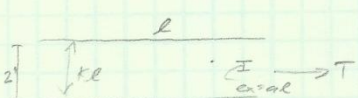
$= .75 \cdot 58 \cdot A_e \geq 26.4$

$A_e = .607 \text{ in}^2$

assume all welds, thus A_g controls

Use 4" wide plate A36 × 1/4" deep $A_g = 1 \text{ in}^2$

Weld to Angle



try $\boxed{1/8\text{" weld}, \ l = 5\text{"}}$

$k = .4$

$e_x = 1 - .609 = .291$

$a = .06^*$

$C = 3.7$ via table 8-4

$D_{min} = \frac{26.4}{.75 \cdot 3.7 \cdot 1.0 \cdot 5"} = 1.9$ or $2/16$ in weld. Assumption works

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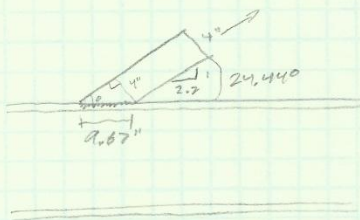
Weld to Beam

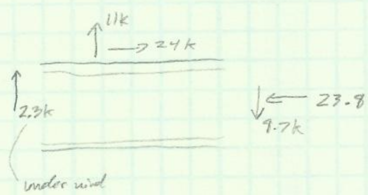


$l = 9"$, $K = 0$
$e_x = al = 0$, $a = 0$

Use table 8-4 with 15° angle

$D_{min} = \frac{26.4}{.75 \cdot 1.0 \cdot 3.96 \cdot 9} = .987$ 16ths, use 1/16th weld

Beam to Column



Gravity load controls without wind 9.7k

Try 2 bolts with 1/4" plate $\phi R_n = 24.5k > R_u = 9.7k$

Block Shear, Shear yielding, & Shear rupture will not control for uncoped section

Bolt Bearing Beam web table 10-1 = 263k .52 > 9.7k

$t_{weld} = 5/8 \cdot 1/4 = .16 \Rightarrow 3/16"$ weld each side

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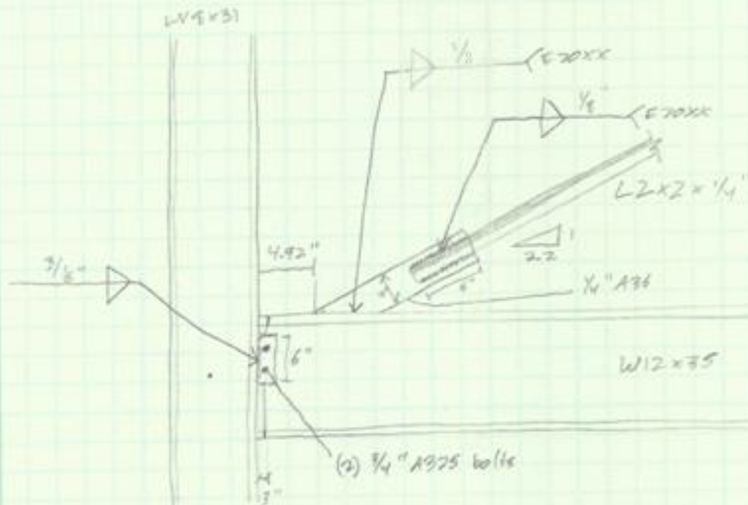
Faculty Advisor: Dr. Richard A. Behr



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Appendix H – Cost Comparison

Cost Comparison

Wood gravity w/ OSB shear walls

	Mat	Install	Total	
Precast Plank – 3100 –	4.05	3.68	11.73	per sf
Wood Floor – 3000 –	10.20	5.73	34.70	per sf
Floors total =	93.55	20.87	115.83	per sf ground
SF = 20,627				
Total =	0.7972 m	.4304 m	1.2276m	
Partitions – interior 1850	1.75	2.90	4.12	
(not brick) – exterior 1300	.64	1.75	2.39	

Interior (44 · 10' · 25') · 3 + 44 · 9' · 25' · 1 = 41,800 sqft
Exterior (900 · (10 · 3 + 8 + 11) = 39,200 sqft

Total Cost = 98,084 189,820 = $287,904

Steel Gravity with Braced Frames

Steel deck on joists – 3700 – 9.65 4.34 12.99
SF = 20,627
total = .714 m .3581 m $1.072 m

Partitions 5200 .89 2.33 3.22
Total = 91,000 sqft
= 77,090 184,730 $260,820

Summary

Total Costs

Wood Shear Walls: $287,904
Steel Interior walls: $260,820

Wood Gravity System: 1.224 million
Steel Gravity System: 1.072 million

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Appendix I – Foundation Calculations

According to the plans, typical footing supports a factored 150 kips

According to the Geotechnical report, soil capacity = 2.5 k
2'x2' pier
Use 3,000 psi concrete

$q_a = \frac{P}{A} = 2.5 = \frac{150}{B^2}$ $B = 7.75 = 8'$

$q = \frac{150}{8^2} = 2.344$ ksf $= \boxed{16.276 \text{ psi}}$

square footing with concentric load, punching shear controls

$V_c \leq \phi(2 + 4/\beta)\sqrt{f'_c} = .75(2 + 4/1)\sqrt{3000} = 246$ psi
$\phi 4 \sqrt{f'_c} = .75(4)\sqrt{3000} = \boxed{164 \text{ psi}}$

$d^2\left(164 + \frac{16.276}{4}\right) + d\left(164 + \frac{16.276}{2}\right) \cdot 24 = \frac{16.276}{4}\left((8 \cdot 12)^2 - 24^2\right)$

$d = 6.689''$ $h = d + \frac{1}{2}$ bar + cover $= 6.69 + .625 + 3 = 10.3'' = 11''$

$b_o = 2(b+d) + 2(c+d) = 2(24 + 6.689) + 2(24 + 6.689) = 122.756$

$V_c = .75\left(\left(\frac{40 \cdot 6.689}{122.756}\right) + 2\right)\sqrt{3000} = 171.7$ psi   Not Controlling

Flexure

$l = \frac{8-2}{2} = 3'$

$M_u = \frac{ql^2}{2} = \frac{2.344 \cdot 3^2}{2} = 10.55$ k-ft

$a = 1.96 A_s$

$\phi M_n = \phi A_s f_y (d - a/2)$

$12 \cdot 10.55 = .9 (A_s)(60)\left(6.689 - \frac{1.96 A_s}{2}\right)$

$A_s \geq .371$ in²/ft   Min spacing #5 @ 10" $A_s = .374$ in²/ft

$\rho_{min} = .0018$ $\rho = \frac{A_s}{bh} = \frac{.374}{12 \cdot 10} = .00317$ ∴ ok

$c = \frac{a}{\beta_1} = \frac{1.96(.374)}{.85} = \cancel{.06756''}\ .862''$

$\varepsilon_s = \frac{.003(d-c)}{c} = \frac{.003(6.689 - .862)}{.862} = .0203$ in/in $> .005$ in/in $\phi = .9$ ok

$s = 10''$oc $\leq 3(10) = 30$ ok   Use 12 #5's
≤ 18 ok

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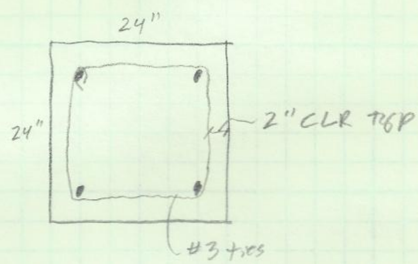
Bearing

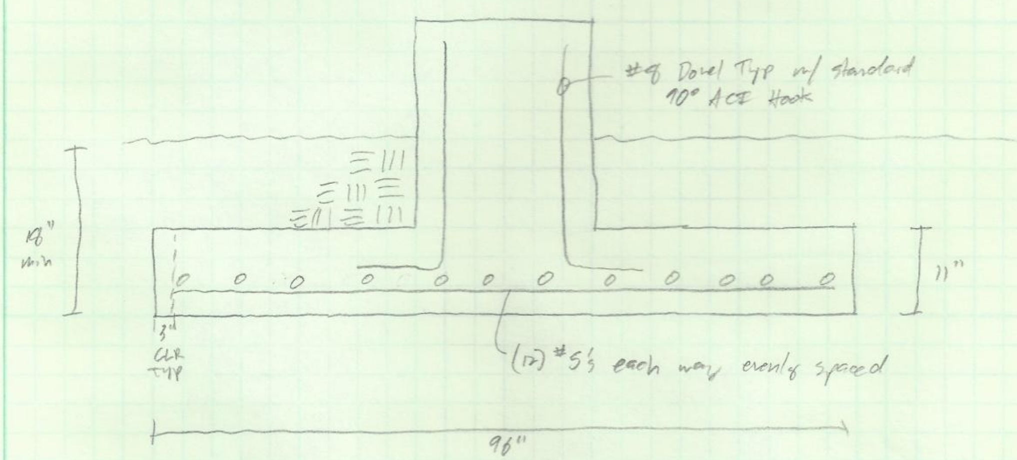
$B_n = .85 f'_c A_1$

$\phi B_n = .65(.85)(3)(24 \times 24) = 955\text{ k} > P_u = 150$ ok

$A_{min\ prov} = .005(24 \times 24) = 2.88\text{ in}^2$ 4 corners & 4 sides ⇒ .36 in²
4 corners = .72 in²

Use (4) #8's $A_s = 3.16\text{ in}^2$



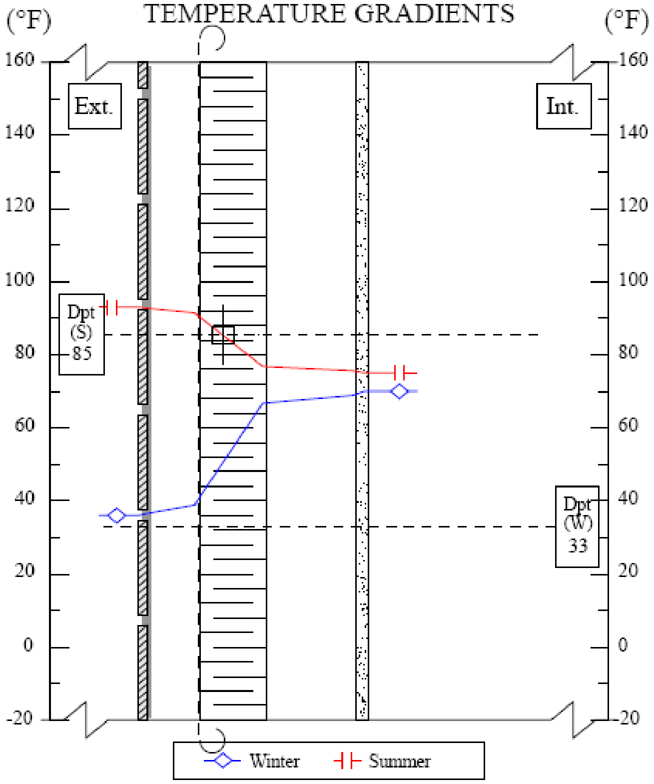


Appendix J – Façade Breadth

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Dormitory
Number	
City	Tampa, FL
Date	4/3/2012
Analysis by:	Cadell Calkins
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	70	36	75	93
RH (%)	25	75	50	77
DPT (°F)	33	29	56	85

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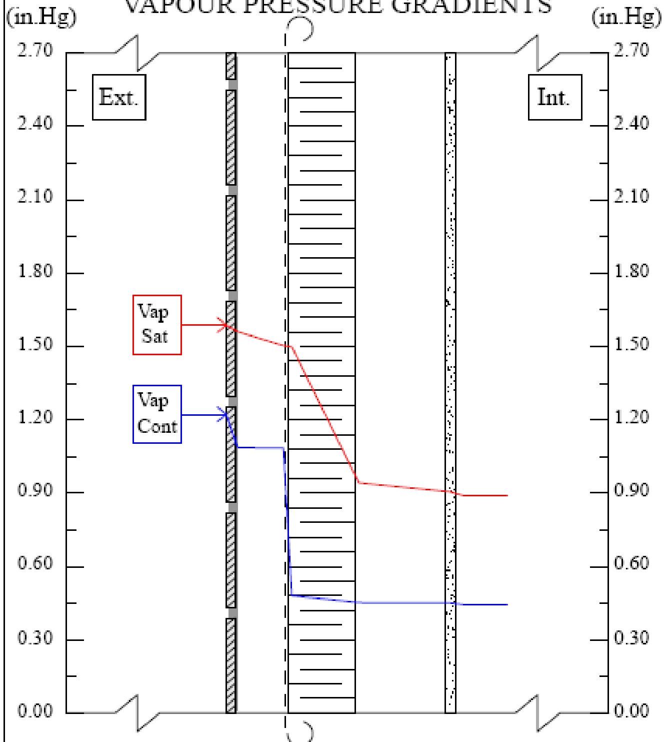
	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	tile, facing, 1/2 in.	No Recor...	Generic...	0.50	0.46	36.7	92.6
2	cavity, 2 in.	No Recor...	Generic...	2.00	0.98	38.9	91.4
3	poly film, (6mil)	No Recor...	Generic...	0.01	0.12	39.2	91.3
4	rigid ins.,(expand.), 3 in.	No Recor...	Generic...	3.00	11.86	66.7	76.8
5	cavity, 4 in.	No Recor...	Generic...	4.00	0.98	68.9	75.6
6	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	70.0	75.0
	Total or (Layer 0)			10.01	14.86	(35.6)	(93.2)

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



** NO CONDENSATION **

PROJECT

Name	Dormitory
Number	
City	Tampa, FL
Date	4/3/2012
Analysis by:	Cadell Calkins
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	—	—	75	93
RH (%)	—	—	50	77
DPT (°F)	—	—	56	85

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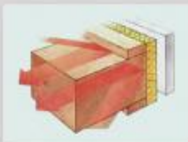
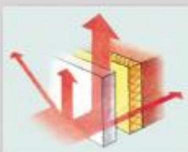
	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCont (in.Hg)
1	tile, facing, 1/2 in.	No Recor...	Generic...	3.814	92.4	1.536	1.069
2	cavity, 2 in.	No Recor...	Generic...	0.016	91.3	1.480	1.068
3	poly film, (6mil)	No Recor...	Generic...	16.827	91.1	1.474	0.473
4	rigid ins.,(expand.), 3 in.	No Recor...	Generic...	0.773	76.7	0.928	0.446
5	cavity, 4 in.	No Recor...	Generic...	0.033	75.6	0.892	0.445
6	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.197	75.0	0.876	0.438
7							
8							
9							
10							
11							
12							
	TOTAL or (Layer 0)			21.751	(93.0)	(1.563)	(1.203)

Rain screen cladding



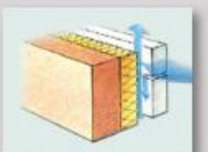
Preventing thermal bridges

As the insulating material is on the outside of the structural wall, it can easily be mounted without interruptions caused by floor slabs. In this way, any thermal bridges that occur at each floor slab can be prevented. These thermal bridges are also the cause of surface condensation that may result in fungus growth.



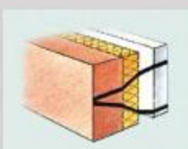
Dissipating heat from the sun

The ventilated rainscreen cladding system has a cooling effect when temperatures outside are high. Most of the sun's rays are reflected away from the building. Heat passing through the exterior wall panel is partially dissipated by the ventilating effect of the space between the exterior cladding panel and the structural wall. Any residual heat managing to penetrate buildings is very minor.



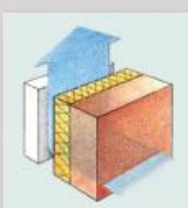
Rainscreen

Architectural wall-cladding panels act as a rain-screen on the outside of the building and keep the structural wall absolutely dry. The air space connected to the outside air evacuates water and humidity that might have penetrated behind the wall-cladding panels through its horizontal or vertical joints. This water will never reach the load bearing wall and/or the thermal insulation.



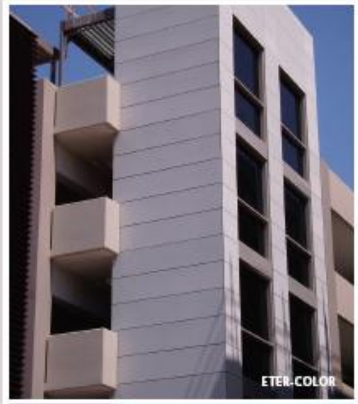
Protecting the basic structure and load-bearing wall against temperature variations

In view of the fact that the insulation material is applied to the outside of the building, changes in temperature are very minor compared with those found in conventional constructions where insulation is applied on the interior. This principle works in summer and winter in both hot and cold climates.



Prevention of internal condensation

Insulation material can be applied to the outside of the structural wall because it is protected effectively by the architectural exterior wall panel. Because of differences in vapour pressure and temperature passing through the wall, condensation has been shown to occur close to the ventilated area and not in the structural wall itself. As a result, the ventilating effect is easily sufficient to dry out the thermal insulating material.



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