ENGROWTH

Final Design Report

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Table of Contents

Table of Figures

Table of Tables

Technical Memorandum 1:
Executive Summary

1.1 Abstract

EnGrowth consists of four civil/environmental engineering students: Kendra Altena, Mitchell Feria, Bethany Goodrich, and Joel Smit. EnGrowth set out to design an expansion to Calvin College's existing Engineering Building. They worked closely with administration, faculty, and staff to develop a facility that will appropriately address the Engineering Department's growing space needs.

The current Engineering Building (EB) was built in 1998 to provide faculty offices, dedicated research space, a computer lab, and wood and metal shops. In addition, it was originally intended to accommodate a senior engineering class of approximately 50-60 students. However, recent enrollment trends and retention rates predict a senior class of 90-100 students within the next 20 years, as shown in Figure 1-1.

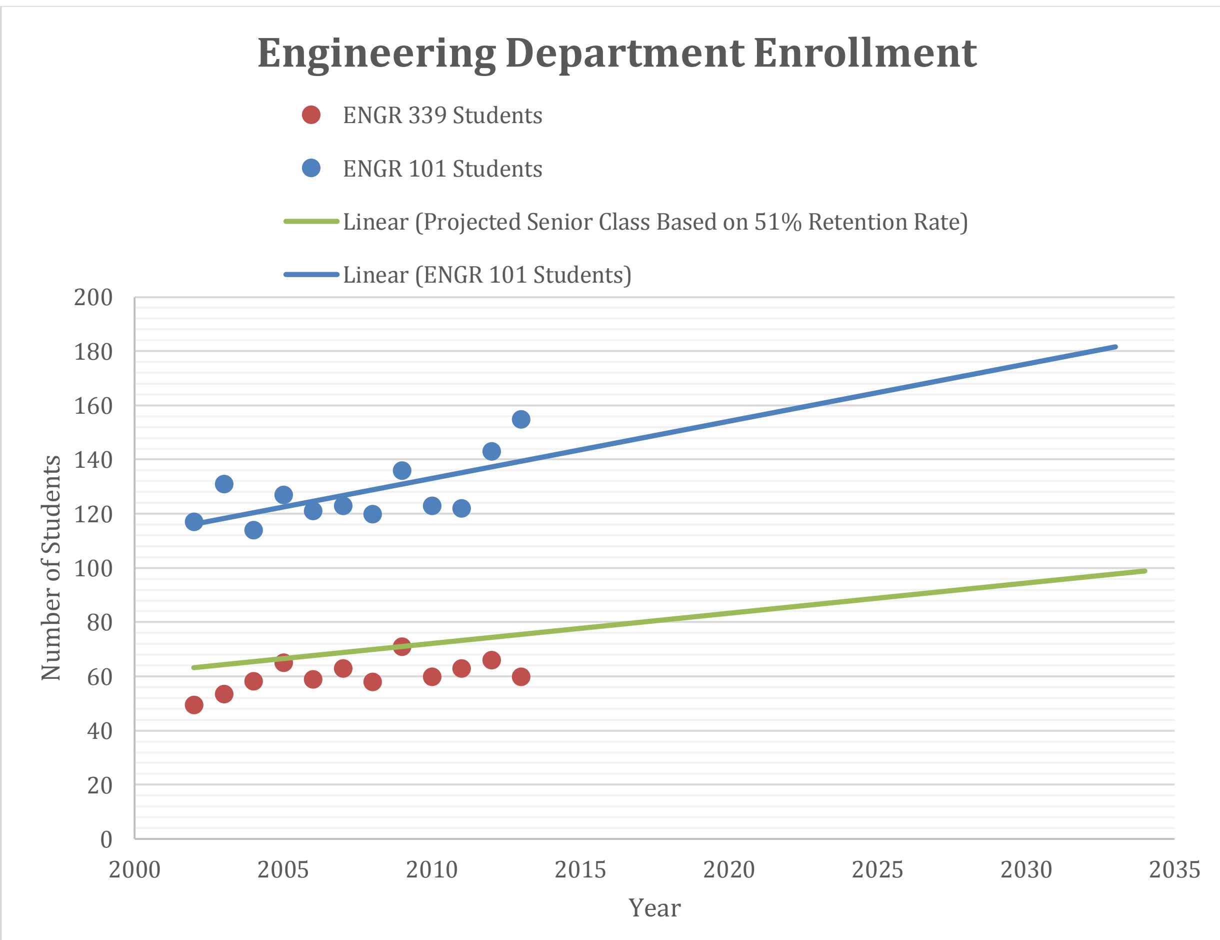

Figure 1-1 - Engineering Department Enrollment

As a team, EnGrowth sought to deliver a facility that most optimally accommodated the needs and desires of the Calvin Engineering Department. To remain consistent with this idea, the majority of the project's key components and parameters were defined and developed through a comprehensive interview process of the Engineering Department faculty and staff. These interviews strongly suggested a desire to better integrate the engineering program with the rest of campus, and this became a focal point for the design. The faculty also indicated that the most pressing space use needs were in the following areas: senior design projects, underclassmen projects, faculty research, faculty offices, upper level classrooms, and a chemistry/bio-engineering lab.

EnGrowth's final solution is the addition of a wing on the northeast corner of the existing Engineering Building. This expansion provides an additional 10,550 square feet of usable floor space. This area includes additional senior design project workspace, two classrooms, a chemistry lab, a welding room, and storage space. In addition to the expansion, minor renovations and re-allocation of space for the current building have been proposed to create dedicated spaces for faculty research and underclassmen projects. The floor plans for the existing and proposed facility are shown in Figure 1-2.

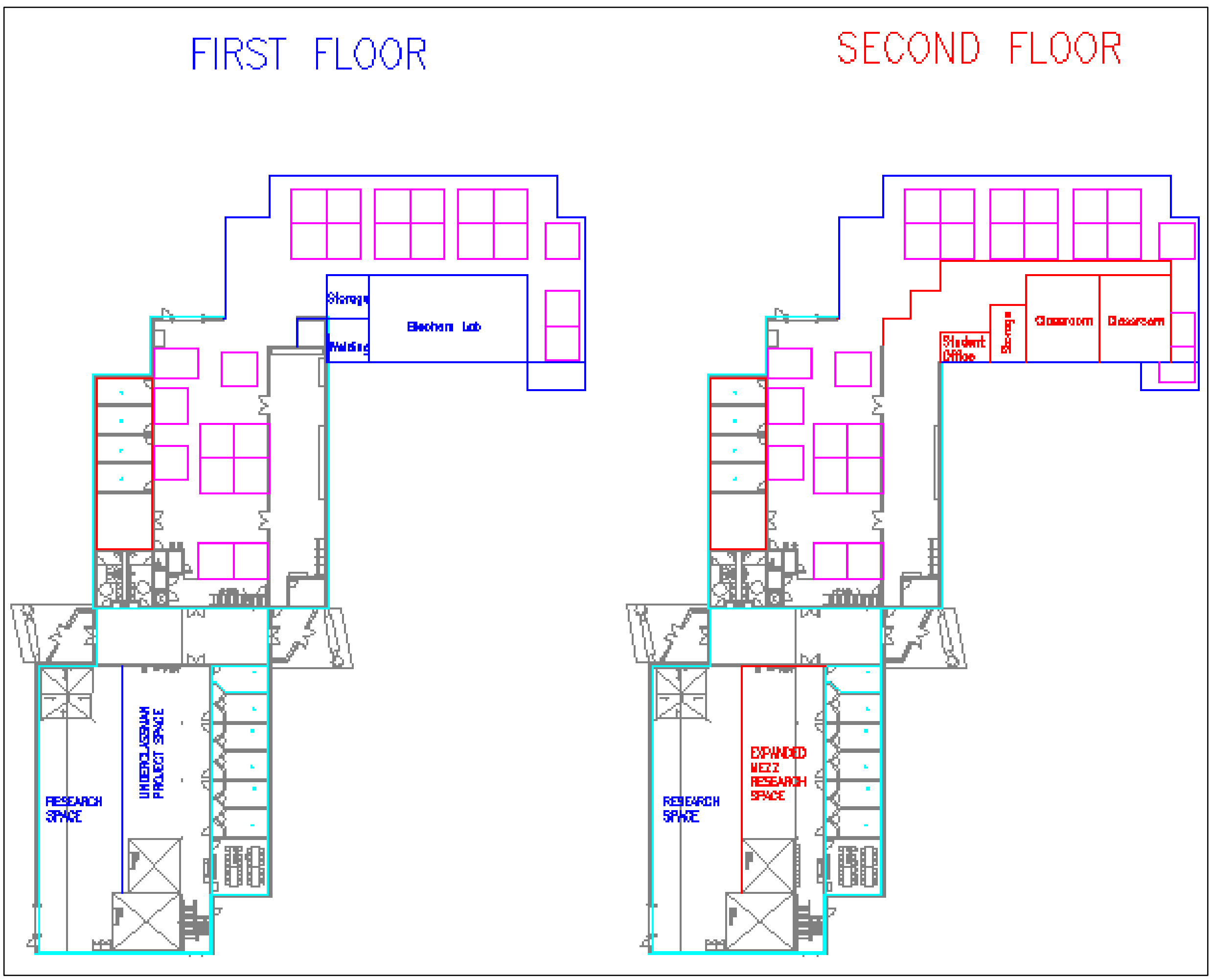

Figure 1-2 - Engineering Building Floorplans

The team desired to complete all civil engineering design work associated with the project, including site, architectural, and structural design, and has successfully developed a finalized set of civil, architectural, and structural plans for the proposal. These plans are included as supplemental information to this report.

The site design portion of this project consisted of two main elements. Due to the department's expressed desire to better incorporate the building with the rest of campus, EnGrowth's design necessitated the re-design of both Knollcrest Circle Drive as well as a number of parking lots along the west edge of Calvin's property line. Second, the proposed expansion required the re-routing and re-design a several utility lines, including storm sewer, sanitary sewer, cable, electrical, and watermain lines. EnGrowth's final site layout is shown in Figure 1-3.

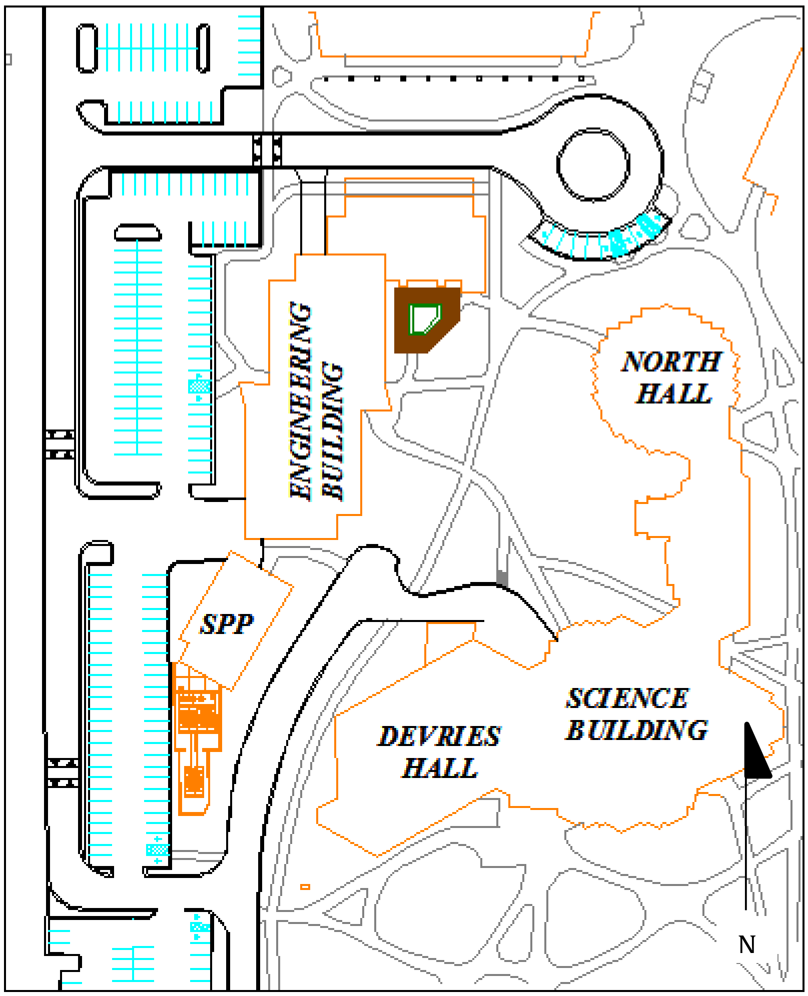

Figure 1-3 - Post-development Site Layout

The architectural design of the expansion was completed with two primary objectives. First, the expansion is designed to blend with the aesthetics of the rest of Calvin's campus. This was accomplished through incorporating the Frank Lloyd Wright inspired Prairie Style architecture that is prevalent throughout Calvin's campus, and also by utilizing the rustic and easily-recognizable "Calvin Brick." Second, EnGrowth wanted to give the building a more modern look to adequately demonstrate the innovation of the Engineering Department. This was done by exposing exterior structural members on the north side of the building as well as by the inclusion of large bay windows to showcase engineering projects to those passing by. These elements are depicted in Figure 1-4.

Figure 1-4 - Architectural Design of EB Expansion

The structural design of the facility was conducted in accordance with applicable design codes such as *ASCE 7-10: Minimum Design Loads for Buildings and Other Structures* and *ACI 318-05: Building Code Requirements for Structural Concrete and Commentary*. The design was completed using a number of software programs including STAADPRO and Ram Structural Systems, whose outputs were checked and verified with extensive hand calculations.

EnGrowth anticipates that the expansion to the Engineering Building will cost approximately $4.8 million. This includes all required design, material, and construction costs for the civil, architectural, and structural elements of the project. A detailed cost breakdown is provided in Appendix A.

Technical Memorandum 2: Background

2.1 History of Calvin College

Calvin College is a comprehensive liberal arts campus in Grand Rapids, Michigan. It was founded in 1876 by the Christian Reformed Church (CRC) and continues to be owned by the denomination. Calvin's reformed tradition is at the center of its actions. The school aims to transform students into agents of renewal and contributors in the redemptive work of Christ.

Currently, in 2014, Calvin enrolls approximately 4,300 undergraduate students participating in over 100 different major and minor programs. The *U.S. News & World Report* lists Calvin among the very best liberal arts colleges in the nation.[1]

2.2 Calvin Engineering Program

Calvin first began offering a fully Accreditation Board for Engineering and Technology (ABET) accredited engineering program in 1986 (retroactive to 1985). The program now consists of 19 full-time faculty and staff and over 400 students. According to Calvin Engineering's website, its mission is "to equip students to glorify God by meeting the needs of the world with responsible and caring engineering."[2] The department works to ensure that the values of the CRC and the emphasis on the liberal arts remain at the forefront of education, even in a very technical environment. Students who complete the four-year program receive a Bachelor of Science in Engineering Degree (B.S.E) with a concentration in one of four available engineering disciplines: Chemical Engineering, Civil & Environmental Engineering, Electrical & Computer Engineering, and Mechanical Engineering.

2.3 Existing Engineering Building

The current Engineering Building was built in 1998 with two wings: the Prince Engineering Design Center and the Vermeer Engineering Projects Center. The facility was

> Specifically arranged to facilitate students engaged in design activities related to various engineering projects, especially the capstone Senior Design course. This building provides space and equipment for all Calvin student engineers to do research, design models, and build and test prototypes.[3]

The Prince Engineering Design Center was designed for engineering offices and dedicated faculty-student research. The Vermeer Engineering Projects Center was designed with a large work area for approximately 40 to 50 senior design students and is equipped with a metal and wood workshop in close proximity to the design space.

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[1] "National Liberal Arts College Rankings." *US News & World Report*. N.p., n.d. Web. 15 Dec. 2013.
[2] "Engineering - Mission Statement." *Calvin College*. Calvin College, n.d. Web. 15 Dec. 2013.
[3] "About Us - Facilities." *Calvin College*. Calvin College, n.d. Web. 15 Dec. 2013.

2.4 Project Description

In recent years, enrollment in Calvin's Engineering Program has grown substantially. Enrollment in the program has increased by 20% since 2000 according to Calvin Enrollment (Day 10) Reports. There is concern that the current facilities utilized by the department will not sufficiently accommodate this rate of growth in the near future. In anticipation of a space shortage, Team EnGrowth has designed a multi-faceted approach to increase the available space in the Engineering Building (EB) and allow for continued growth.

The primary components of this project consist of full site development and structural design for an addition to the existing EB. EnGrowth examined four design alternatives for this expansion, and ultimately decided to expand the building to the northeast toward the center of campus. This expansion increases Senior Design Project space, and adds 10,550 square feet of usable floor space, two classrooms, a chemistry/chemical engineering lab, and a dedicated welding room to the existing Engineering Building.

The second aspect of the project is a re-allocation of space use within the existing facility. Many faculty members expressed the opinion that current space could be more efficiently utilized. EnGrowth sought to strategically plan space utilization to maximize the functionality of the building. This incorporates a remodel of the existing Prince Engineering Design Center (south bay), including the conversion of the south bay mezzanine into an enclosed second floor. This remodel provides an increase in faculty research space and a dedicated workspace for underclassmen projects. It also enables a possible future renovation to provide additional Engineering Department faculty offices.

2.5 Team Member Biographies

2.5.1 Kendra Altena

Kendra Altena is a senior at Calvin College in the Civil and Environmental Engineering Concentration. She was born and raised in Grand Rapids, Michigan, and graduated from Grand Rapids Christian High School. The most interesting part of this project for Kendra was the site plan and development aspect. She is very interested in the areas of hydraulic engineering and storm water management. Kendra has participated in several study abroad experiences in her time at Calvin. She spent a semester in the Netherlands, went on a 3-week interim trip to China, and went on a 3-week interim trip to Kenya. These abroad experiences have really increased her desire and life goal to work in the area of missions. After graduation Kendra will begin her career as an engineer at Prein & Newhof in Grand Rapids, MI. In addition to working professionally, she would like to get involved with missions and use her engineering degree to help people living in developing countries.

2.5.2 Mitchell Feria

Mitchell Feria is a senior engineering student at Calvin College, focusing in the Civil and Environmental concentration. He is from Aurora, Colorado, and will begin his professional career at GMB Architecture + Engineering in Holland, MI following graduation. Mitchell was intrigued to work on a project that was so intertwined with the non-technical aspects of the engineering industry; he was excited to see how the social, political, and even legal facets of a project impact the design. In addition, Mitchell was pleased to take part in a project that had such potential to give back to the Calvin Engineering Program, as he is very grateful for the opportunities that his education has provided him. Mitchell also has a passion for helping people in need and hopes to apply his engineering knowledge in a mission work capacity.

2.5.3 Bethany Goodrich

Bethany Goodrich is a senior Civil and Environmental Engineering student at Calvin College. She is from Albany, New York, and transferred to Calvin the Fall 2011 semester after completing two years of a Civil Engineering Technology program at a local community college. Study-abroad trips to Thailand/Cambodia and Kenya as well as upper-level engineering classes have confirmed her passion to develop innovative solutions for people in need. This project requires diverse structural analysis and design components which is in-line with Bethany's future career interests. Post Calvin, Bethany will work at URS Corporation as a Civil – Bridge Engineer. She intends to use engineering both domestically and internationally to help those in need.

2.5.4 Joel Smit

Joel Smit is a senior in the engineering program at Calvin College focusing in Civil and Environmental engineering. Joel will also graduate with a minor in Architecture. He is from Grand Rapids, Michigan, where he has lived all his life. The structural and architectural aspect of this expansion project was most interesting to Joel, because he is very passionate about aesthetically pleasing buildings that fulfill a specific need. He has always been fascinated with both the basic structures of buildings and the way engineering design can be used as an aesthetic architectural element. Following graduation, Joel will be pursuing his Master's in Structural Engineering at the University of Michigan. Eventually he would like to use his education along with a future professional license to work on high-rise building projects in large cities.

Technical Memorandum 3:
Architectural Design

3.1 Overview

The goal of the architectural design of the expansion is to enhance the design of the surrounding buildings and blend in well with the overall campus architectural plan. The architectural plans lay the foundation for the structural design of the expansion.

3.2 Prairie Style Architecture

Calvin College is known for its Prairie Style architecture. In 1957, William Beye Fyfe was commissioned to design the master plan for Calvin College. Fyfe was one of five of Frank Lloyd Wright's apprentices in 1932. He was a proud supporter of Wright's Prairie School of Architecture because of its clean lines and integration of the buildings with their landscapes[4].

His design of Calvin College clearly incorporates this style of architecture. The academic and residential buildings on Calvin's campus have very straight flat rooflines that mimic that of nature's horizon. Furthermore, the buildings use a very earthy type of brick. This type of brick alludes to the rustic feel of nature. In Fyfe's design, he develops a sense of home reminiscent of the Prairie Style. The staircases throughout Calvin's campus have bay windows in them, because the purpose was to place the viewer right next to nature. It also allows the viewers to decide how they should dress to go outside ahead of time, rather than at the door, like they would if they were in their own home.

3.3 Modernization of Existing Aesthetics and Façade

The intention of the Engineering Building's Expansion is to demonstrate new innovative technology in the design that relays a contemporary state-of-the-art engineering program to visitors. Therefore, the expansion incorporates new concepts that are grand in design and in stature. However, the design of the Engineering Building still fits in well with the surrounding buildings and does not look out of place. The use of existing brick, or "Calvin Brick," is important because it allows the building to reflect the materials of the surrounding brick buildings. The expansion includes many bay windows to provide natural light into the design area and classrooms to reduce the amount of artificial light necessary throughout the day. Bay windows also allow visitors who turn around in the round-about in front of the Spoelhof Fieldhouse Complex to gaze into the Engineering Building and observe the senior design projects that are representative of the Engineering Department. The exterior of the building facing the Fieldhouse Complex reveals the structural components of the building, which also helps demonstrate the innovation of the Engineering Department. Additionally, visitors can readily realize that it is the Engineering Building because structural members are representative of engineering as a whole. The Façade of the building facing toward the south incorporates more rustic materials and colors because the south side faces the park and walkway.

[4] Hamill, Sean D. "William Beye Fyfe, 90." *Chicago Tribune*. Chicago Tribune, 11 May 2001. Web. 15 Dec. 2013.

3.4 Architectural Floor Plans

3.4.1 First Floor Plan

The first floor of the Engineering Building is shown in Figure 3-1. The first floor of the existing research bay will now contain underclassmen project space. This is a desirable location for the project space, because it is separated from the seniors so the underclassmen are not intimidated by upperclassmen. Furthermore, the project space will be located on the existing 6” concrete floor slab that is designed for projects. A wall is added connecting the vibration chamber with the north wall to divide the project space from the professor’s research space.

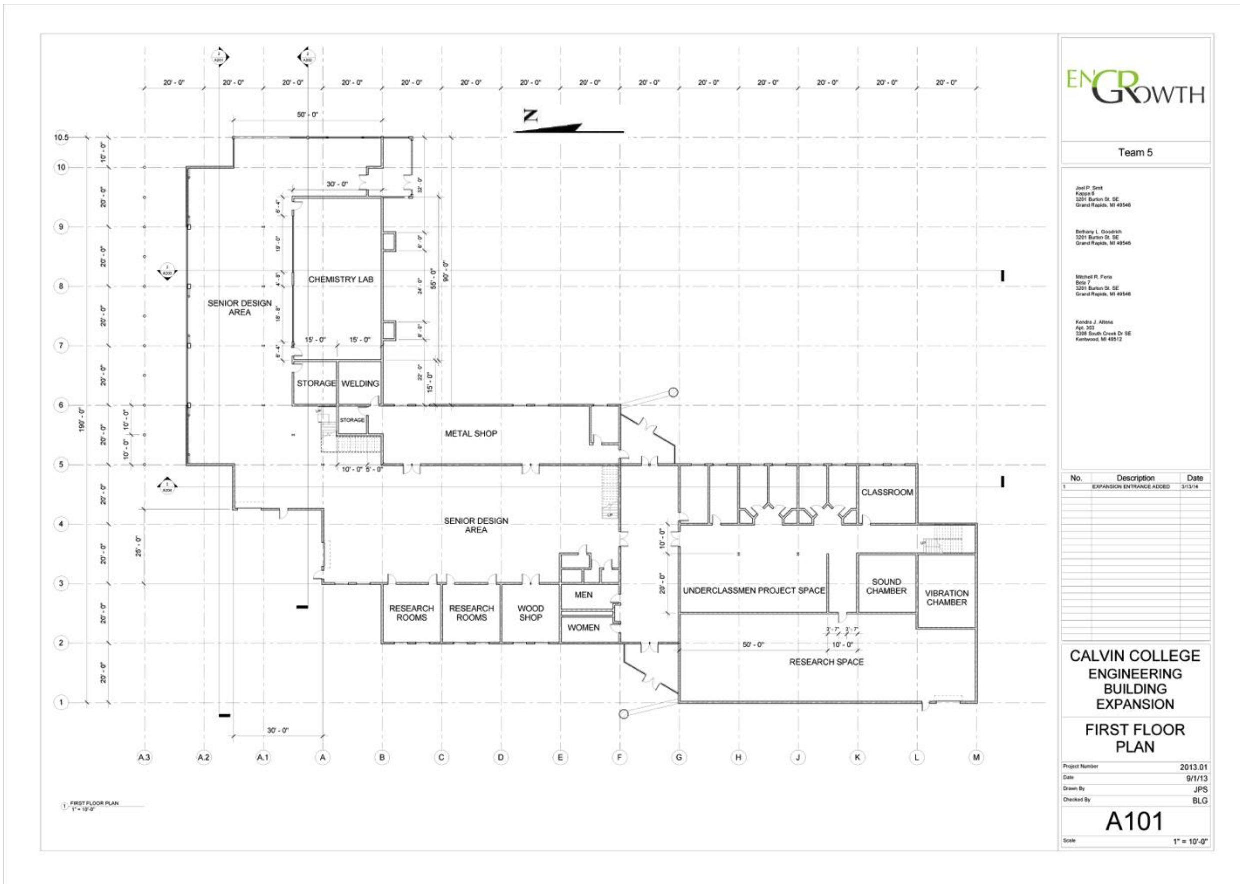


Figure 3-1 - First Floor Plan

The first floor of the expansion includes two storage spaces, one for the metal shop and one for the project bays, a welding room for the metal shop, a chemistry and biomedical laboratory, and more senior design project space. One of the problems with the current welding location is that it is divided from the rest of the shop by an ultraviolet curtain. Although the curtain reduces the amount of light from the arc welder, there is still a possibility that a student might catch a glimpse of the light through a gap in the curtain. In the expansion, the weld room for the metal is placed around the corner to eliminate this problem. There is also a door to the weld room to ensure a student does not accidently come near the arc welder when it is in operation. The Biomedical and Chemical Engineering laboratory is placed on the first floor because it is more desirable to have the laboratory directly on the concrete slab on grade to reduce vibrations. If the laboratory were to be on the second floor, the beams would all have to be reinforced to eliminate any vibrations. This not only requires additional engineering analysis, but also requires more costly construction. Placing the laboratory on the first floor eliminates these unnecessary costs.

The senior design space is open and connected to the existing design space. This allows for unity between the teams. The teams will not have to be separated from each other or spread out in different wings or floors of the Engineering Building. In doing so, they will have the ability to easily visit other teams and get their advice or help on their project if it incorporates an aspect another team has expertise in. This will facilitate a more collaborative learning environment.

In addition, an entrance is added on the southeast corner of the expansion. This is to allow traffic to easily enter the expansion. Students will be able to walk next to the senior design space and see the projects on their way to class. Furthermore, it allows better passage and access to the Engineering Building allowing for a more efficient use of the space.

3.4.2 Second Floor Plan

The second floor of the Engineering Building is shown in Figure 3-2. The existing research bay is expanded to incorporate more professor research space. The space will be divided into research offices for each professor to have a private space if it is critical to have a controlled atmosphere. The current storage space across from the new research space will be turned into additional offices for professors.

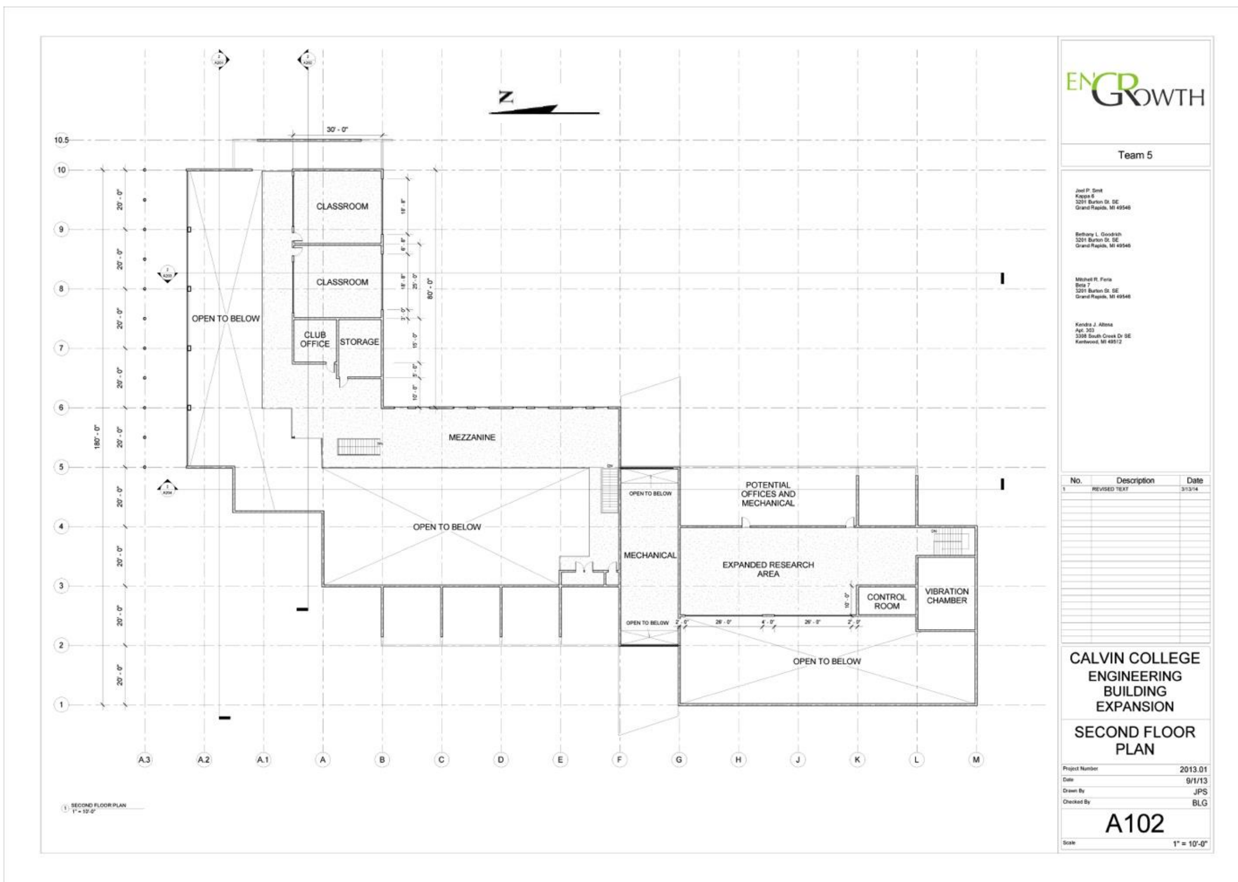


Figure 3-2 - Second Floor Plan

The second floor of the expansion consists of a storage closet, a student club office, and two classrooms. The current mezzanine in the project bay will be expanded and extend into the expansion. This allows for a better transition from one bay into another. The mezzanine will also transition into a mezzanine walkway that gives access to the two classrooms. This walkway will allow students who have classes in the Engineering Building to look at the senior design projects as they walk to and from class. The students will be able to see the progress the teams are making and can also see the practical aspects and uses of topics they are currently learning in class.

The north façade of the expansion facing the aquatic center is shown in Figure 3-3, the senior design project bay in the expansion is shown in Figure 3-4, and Figure 3-5 shows the view of the expansion from the mezzanine hallway. Figure 3-6 shows the additional entrance on the southeast corner of the expansion.

Figure 3-3 - North Façade of the Expansion

Figure 3-4 - New Senior Design Project Space

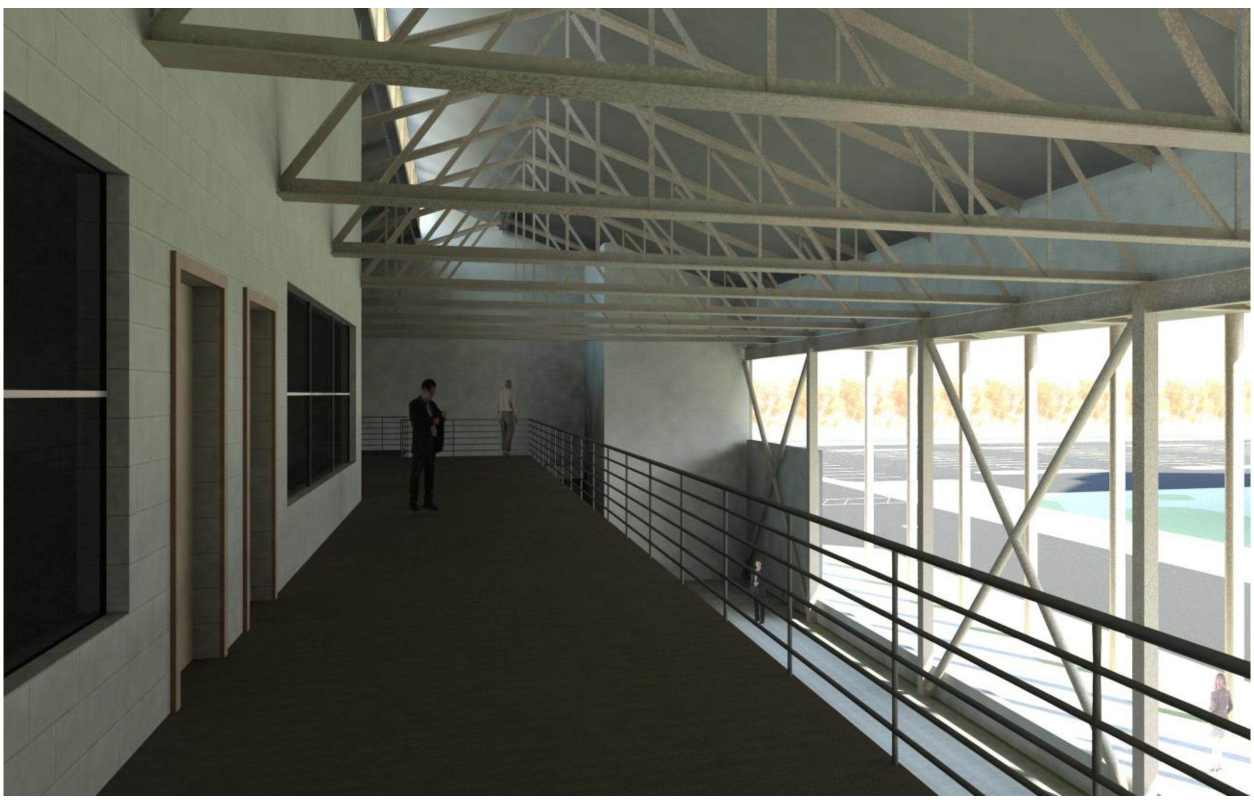


Figure 3-5 - Expanded Project Space from the Mezzanine Hallway

Figure 3-6 - East Corner of the Expansion

Technical Memorandum 4: Structural Design

4.1 Overview

The structural design for the expansion and the renovation included identification and calculation of the loads acting on the building, steel design for the framing of the roof truss system, steel design of the mezzanine floor beams, steel design for the north steel cross bracing shear wall, masonry design for the concrete masonry walls, concrete foundation design, connection detailing for the trusses and walls, and structural drafting of the plans.

4.2 Load Calculations

4.2.1 Load Overview

The different types of loads acting on the expansion and the renovation to the Engineering Building were determined to be dead loads and live loads on the roof, mezzanine, and ground floor; snow loads on the roof structure; and wind loads and seismic loads on the walls. The *Minimum Design Loads for Buildings and Other Structures*[5] was used to calculate these load values.

Table 4-1 shows the load values for the different types. These are the entire load values used for the sizing of members and walls.

Table 4-1 - Load Calculation Values

Location	Dead Load (psf)	Type	Live Load (psf)	Snow Load (psf)	Snow Drift (psf)	MWFRS (psf)	Components & Cladding (psf)	Seismic Shear (kips)
Roof	22.0	Mechanical	40	22.1	26.8	N/A	N/A	N/A
Mezzanine	100.0	Stairs	100	N/A	N/A	N/A	N/A	N/A
		Hallway	80					
		Classroom	40					
		Computer Room	100					
Ground Floor	N/A	Project Space	80	N/A	N/A	N/A	N/A	N/A
		Laboratory	150					
Exterior Walls	N/A	N/A	N/A	N/A	N/A	16.5	19.8	17.2

4.2.2 Dead Loads

The dead loads are the permanent loads acting axially on the expansion. Table 4-2 shows the dead load values for the roof and mezzanine. The dead load value for the mezzanine is 82.32 pounds per square foot (psf), but a value of 100 psf was used for it as a safety factor.

[5] *Minimum Design Loads for Buildings and Other Structures*, 2010 Edition, ASCE Standard 7-10

Table 4-2 - Dead Loads for Expansion

	Type	Description	Load (psf)
Roof	Decking	18 Gage Metal Decking	3
	Framing	Steel Joists and Girders	3
	Roofing	Shingles	6.5
	Sprinklers	Sprinklers	2
	Insulation	(6) Styrofoam per 1 inch Thick	1.5
	Ceiling	Suspended Acoustical Tile	1.8
	Miscellaneous	Miscellaneous	4.2
		Total	22

	Type	Description	Load (psf)
Mezzanine	Decking	Vulcraft B18 Decking	2.82
	Framing	Carpet and Pad	1
	Concrete	Concrete Regular per 1 inch (5)	62.5
	Framing	Steel Joists and Girders	10
	Ceiling	Suspended Acoustical Tile	2
	Sprinklers	Sprinklers	2
	Mechanical	Mechanical and Electric	2
		Total	82.32

4.2.3 Live Loads

The live load values are temporary loads acting on the structure rather than permanent loads. Table 4-3 shows the live load values used for the roof, mezzanine, and ground floor. These values were taken from Table 4-1 in ASCE 7-10.

Table 4-3 - Live Loads for Expansion

	Type of Use	Load (psf)
Roof	Mechanical Ductwork	40

	Type of Use	Load (psf)
Mezzanine and Floor	Office	50
	Computer Room	100
	Stairs	100
	Hallways	80
	Storage	20
	Laboratories	150
	Classrooms	40

The mezzanine was designed for 100 psf for the computer rooms rather than the 40 psf used for the classrooms because it allows for adaptations of the mezzanine in the future. This ensures the mezzanine will have the structural integrity necessary to accommodate any future changes in use that would result in more weight being applied than what is currently present. Although this results in slightly larger wide flange beams on the mezzanine, the benefits of having an adaptable space outweighs the added cost of larger beams.

4.2.4 Snow Loads

The snow loads were designed according to chapter 7 of ASCE 7-10. The expansion is a fully exposed, terrain category B building in risk category II. The following equations were used to calculate the snow load, Ps, acting on the roof of the expansion.

$$P_f = 0.7C_eC_tI_sP_g$$

$$P_s = C_sP_f$$

Where,

C_t = 0.9 (Fully exposed, terrain category B)

I_s = 1.00 (Risk category II)

C_t = 1.00 (Thermal factor)

C_s = 1.0 (Roof slope factor)

P_g = 35 psf (Flat roof snow load in Kent County, Michigan)

The snow load acting axially on the roof of the expansion was calculated to be 22.05 psf.

The snowdrift force was calculated because there is a clerestory in the roof that has resulted in the two slopes of the roof joining at different heights. The values for the snowdrift were incorporated exactly into the truss design to ensure the trusses were designed with accurate snow loads. Figure 4-1 shows a diagram of the snow loads acting on the building and Figure 4-2 shows how the snow loads are distributed.

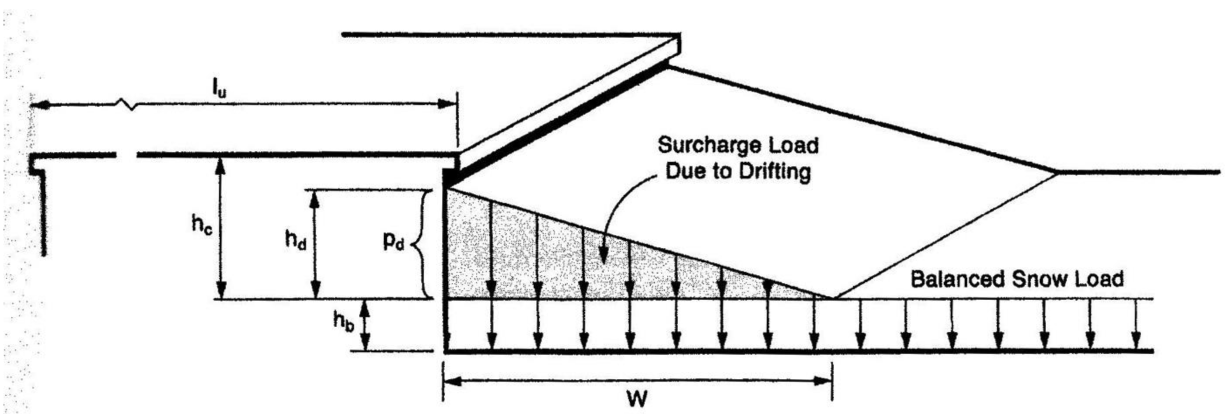


Figure 4-1 - Snow Drift Load Diagram[6]

[6] Figure 7-8 Configuration of Snow Drifts on Lower Roofs

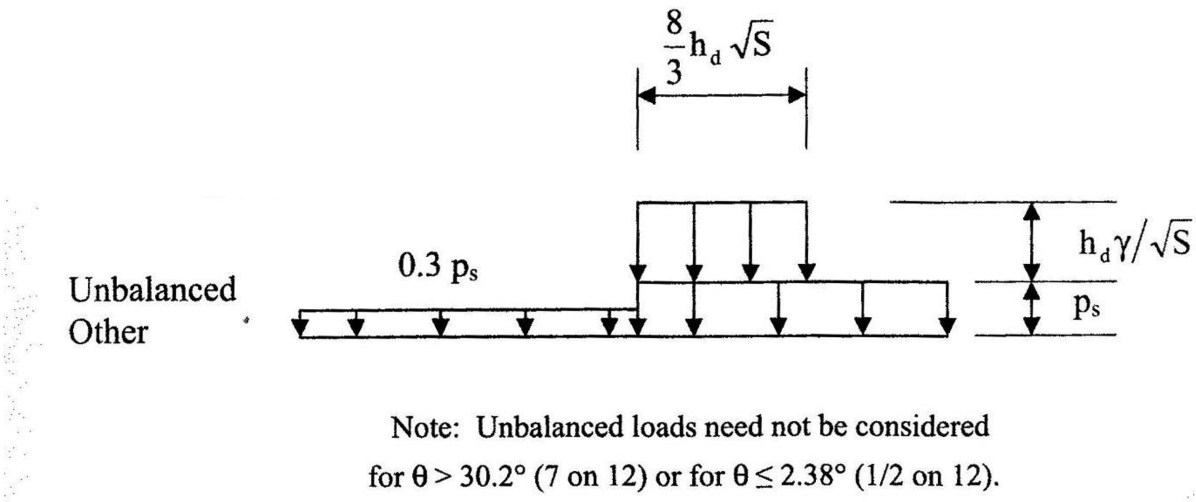


Figure 4-2 - Unbalanced Roof Snow Loads[7]

The two loads used for the modeling and framing of members were Ps and $h_d\gamma/\sqrt{S}$. The 0.3Ps was not used for modeling because it is smaller than Ps. Ps was used for the entire roof as a safety factor. The values and calculations for $h_d\gamma/\sqrt{S}$ and the distance of the snowdrift, $8/3h_d\sqrt{S}$, are shown below.

$$\gamma = 0.13P_g + 14 = 0.13(35psf) + 14 = 18.55\ pcf$$

$$h_d\gamma\sqrt{S} = (25\ feet)(18.55pcf)\sqrt{3} = 26.77\ psf$$

$$\frac{8}{3}h_d\sqrt{S} = \frac{8}{3}(25feet)\sqrt{3} = 11.55\ feet$$

Where,

P_g = 35 psf

H_d = 2.5 feet (Height of clerestory)

S = 3 (Roof slope run for a rise of one)

P_s = 22.05 psf

However, when the snowdrift was modeled in STAADPRO, the distance of the snowdrift was set as 10 feet because the last one-foot of distance is very close to the roof snow load Ps.

4.2.5 Wind Loads

Two different types of wind loads were calculated to ensure an accurate representation of the wind loads. The Mean Wind-Force Resisting System (MWFRS) method in Chapter 28 was used to calculate the in-plane wind load shear force on the walls of the expansion. The Components and Cladding method in Chapter 30 was used to calculate the out of plane wind load forces acting on the walls. The values for the wind forces are located in Table 4-1. The velocity pressure at 25 feet was

[7] Figure 7-5 Balanced and Unbalanced Snow Loads for Hip and Gable Roofs

calculated, and then the internal and external pressure coefficients, GC_p, were factored in to provide the wind load force. The following equations were used in these calculations.

$$q_z = 0.00256 K_z K K_{zt} K_d V^2 = 19.14\, psf$$

$$P = q_z\left(GC_p \pm GC_{pi}\right) = 9.57 psf, 16.46 psf$$

Where,

K_z = 0.665 (Velocity pressure exposure)

K_{zt} = 1 (Topography factor, exposure category B)

K_d = 0.85 (Wind directionality factor for MWFRS)

V = 115 mph (Basic wind speed at 35 feet above ground)

G = 0.85 (Rigid buildings)

C_p = 0.8 (Windward wall)

GC_{pi} = ±0.18 (For enclosed buildings)

The components and cladding method of Low-Rise Buildings, Simplified was used, and the values were taken from Table 30.7-2 in ASCE 7-10. The expansion was categorized as a risk category II; with a basic wind speed, V=115 mph; in exposure category B; with a topographic factor, K_{zt} = 1; a roof angle of 1/3; and an adjustment factor λ=1 for a 35 foot mean roof height. Figure 4-3 shows the different wind zones and Table 4-4 shows the corresponding wind load forces for the zones.

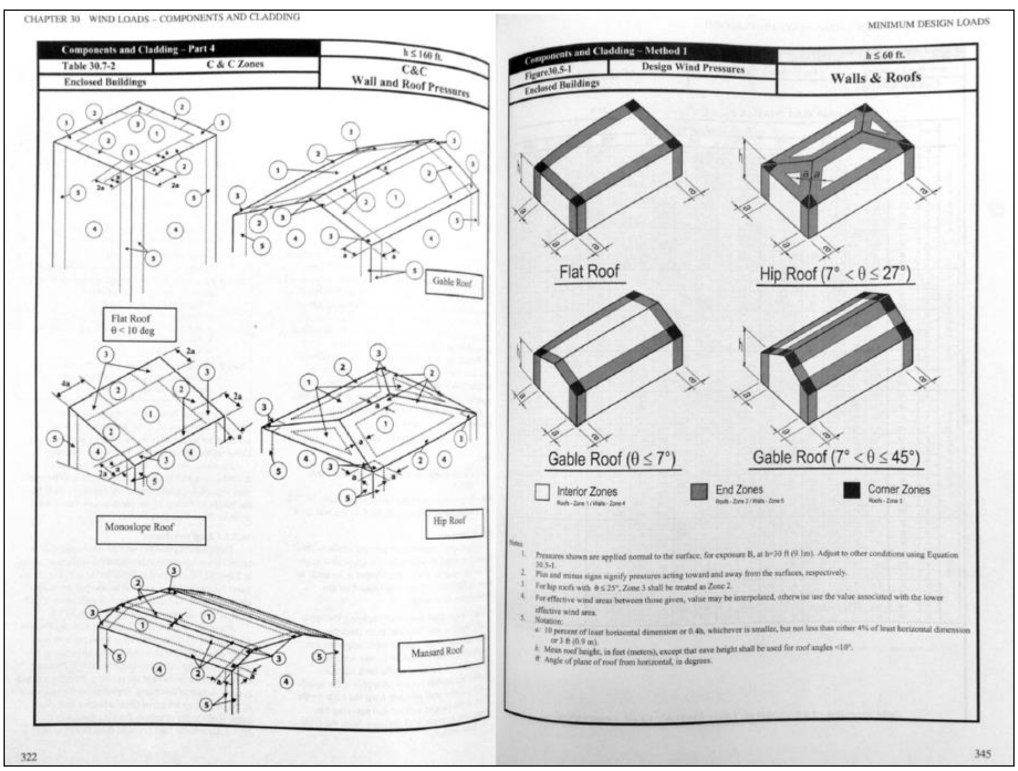


Figure 4-3 - Zone Diagram

Table 4-4 - Components and Cladding Wind Load Zones and Forces

Windward (Pnet30)		Leeward (Pnet30)	
Zone	Wind Force (psf)	Zone	Wind Force (psf)
1	19.8	1	19.8
2	19.8	2	19.8
3	19.8	3	19.8
4	17.7	4	17.7
5	17.7	5	17.7

4.2.6 Seismic Loads

The seismic loads acting on the building were calculated according to the Equivalent Lateral Force Procedure in Chapter 12 of ASCE 7-10. Although seismic loads rarely control the lateral load cases in Michigan, EnGrowth took the loads into consideration to prove it would not control the lateral load cases. The value for the seismic shear is found in Table 4-1.

The building was based on a site class D, because the soil properties under the expansion were not known. The structure was also categorized as a seismic design category B. The seismic base shear, V, and the seismic response coefficient were calculated using the following equations.

$$C_s = \frac{S_{ds}}{(\frac{R}{I_e})} = 0.0352$$

$$V = C_s W$$

Where,

S_{ds} = 0.0704

I_e = 1.00 (Importance factor)

R = 2 (Response modification factor)

W = 489 kips (Effective weight of the building)

4.3 Roof Framing

The roof framing of the expansion consists of 4 WT beams and 6 different trusses (Figure 4-4).

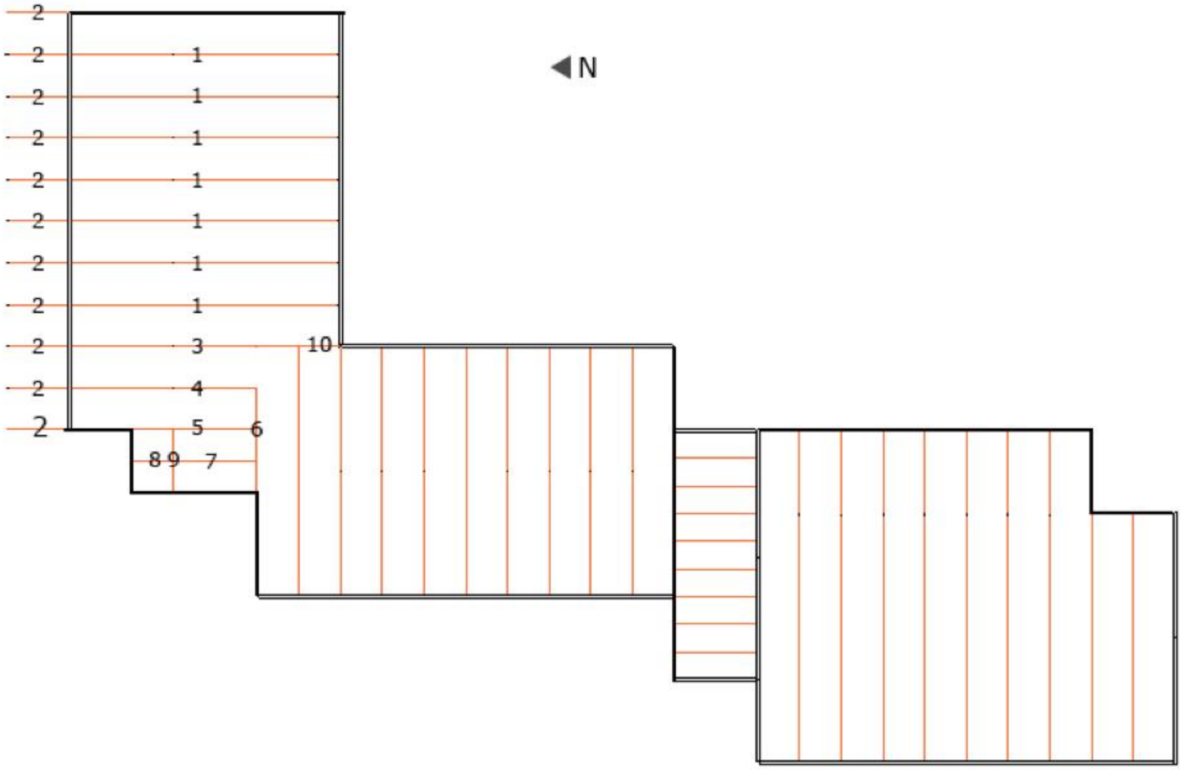


Figure 4-4 - Roof Framing of the Expansion with Beams and Trusses Labeled

The 4 WT beams were hand designed using the AISC Steel Construction Manual, Fourteenth Edition. Table 4-5 shows the optimized sizes for each of the beams, and the calculation procedure for beam 10 can be found in Appendix B.

Table 4-5 - Beam Designations

Beam	Zx, req'd	Member	Zx Actual
7	13.6	WT6x60	16.2
8	3.4	WT6x32.5	7.5
9	15.3	WT6x60	16.2
10	3.4	WT6x32.5	7.5

The Zx value for the beams were designed to be slightly higher than the calculated Zx value to ensure the beams had enough strength to support the applied loads. Although there were beams that had Zx values closer to the calculated values, larger beams with higher Zx values were chosen because they are standard beams sizes that are less expensive to manufacture and purchase.

Originally the main truss for the expansion was going to extend 80 feet with 15 feet outside of the bay. However, when the truss was modeled in STAADPRO, the design of member sizes for the outside of the truss remained as WT22x145. These beams weigh 145 pounds per linear foot and are 22 inches in height. Not only are these beams very expensive to obtain, they are significantly larger than the WT6x32.5 members making up the outside of the current trusses in the engineering building. EnGrowth did many hand verifications to determine if these optimized members from

STAADPRO were accurate and concluded that they were. Uniformity and a low construction cost is very important to EnGrowth, so the team split the truss into two different trusses: a 15 foot exterior truss (Truss 2) from the north wall to the columns on the exterior, and a 65 foot truss (Truss 1) spanning the interior of the expansion. This allowed EnGrowth to design the truss with WT6x32.5 members on the outside and 2L3-1/2x3x1/4 for the web members. This is a much more inexpensive truss, and it mimics the current truss in the Engineering Building. Hand calculations were done on the members of the truss to verify that the members are designed correctly. Figure 4-5 shows the profile for the main 65-foot truss.

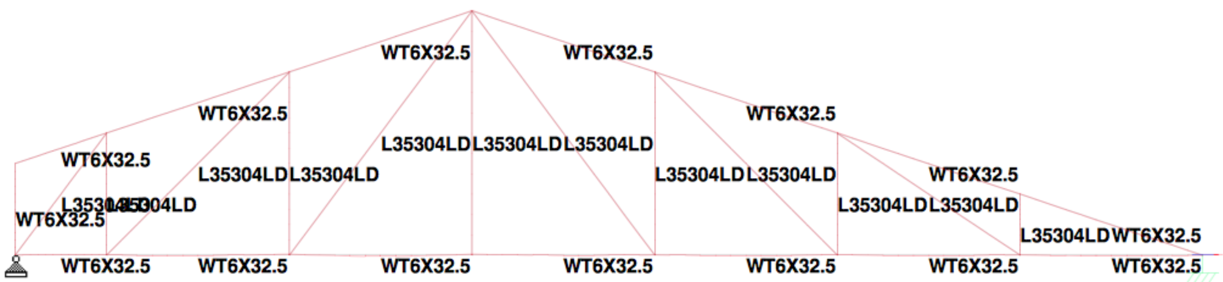


Figure 4-5 - 65-Foot Truss Profile

The north wall of the current engineering bay was removed because of the expansion. As a result, a smaller, 25-foot truss, truss 6, was designed to replace the wall being removed. The profile for truss 6 is shown in Figure 4-6. This truss spans from the wall on grid A to a column that supports both truss 6 and truss 4.

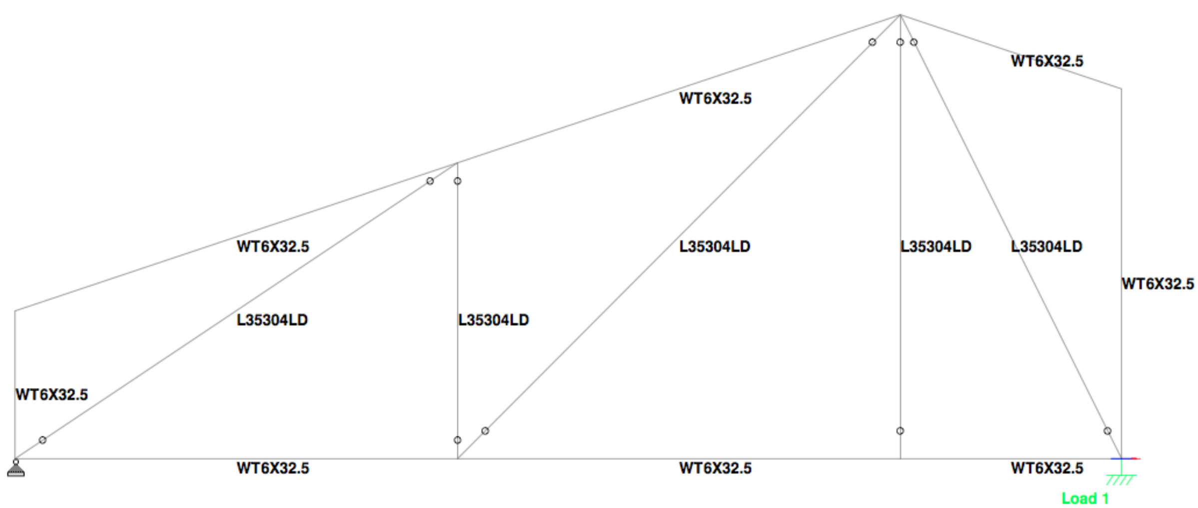


Figure 4-6 - Truss 6 Profile with Members

Furthermore, truss 5 and beams 7, 8, and 9 were designed to keep the roof slope consistent with that of the rest of the expansion. Truss 5 is a 30-foot truss that has beam 9 spanning into it. The reaction from beam 9 onto truss 5 was obtained from hand calculations in the beam design and then placed in the STAADPRO model for truss 5. The profile and truss members for truss 5 are shown in Figure 4-7.

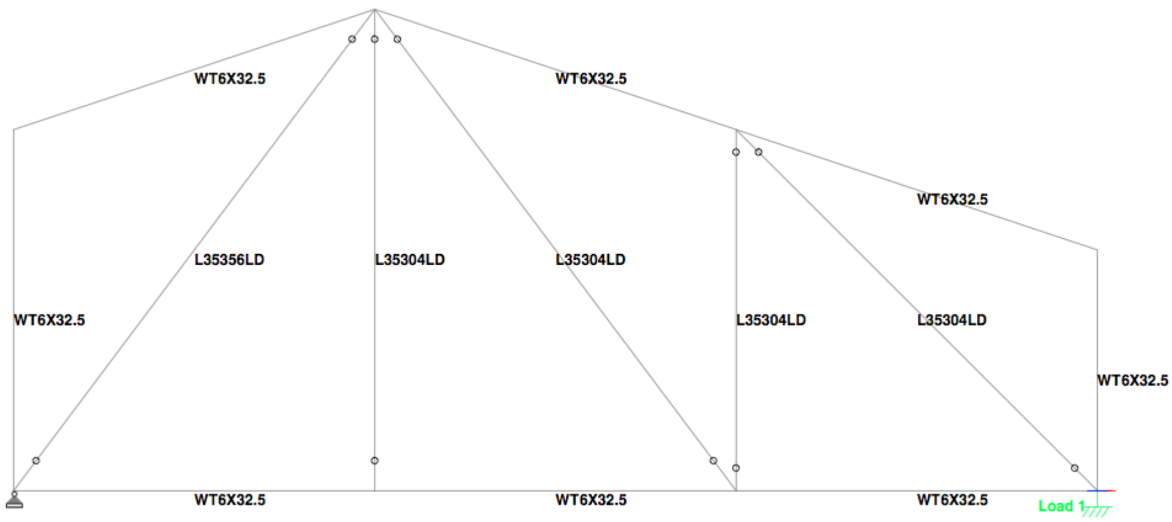


Figure 4-7 - Truss 5 Profile with Members

This truss spans from the wall to truss 6. The reaction acting on truss 6 was obtained from STAADPRO for truss 5 and was modeled in the STAADPRO design for truss 6. Although the reaction from truss 5 was already factored, EnGrowth still used it because having a larger load ensures the truss will support any load that could be placed upon it.

Trusses 3 and 4 are the same shape as truss 1, but are shorter. This results in their irregular shapes. These two trusses were modeled in STAADPRO with the same members as truss 1. Since the two trusses have the same profile as truss 1 but are only shorter, both trusses were able to be designed with the same members. The profiles for trusses 3 and 4 are shown in Figures 4-8 and 4-9.

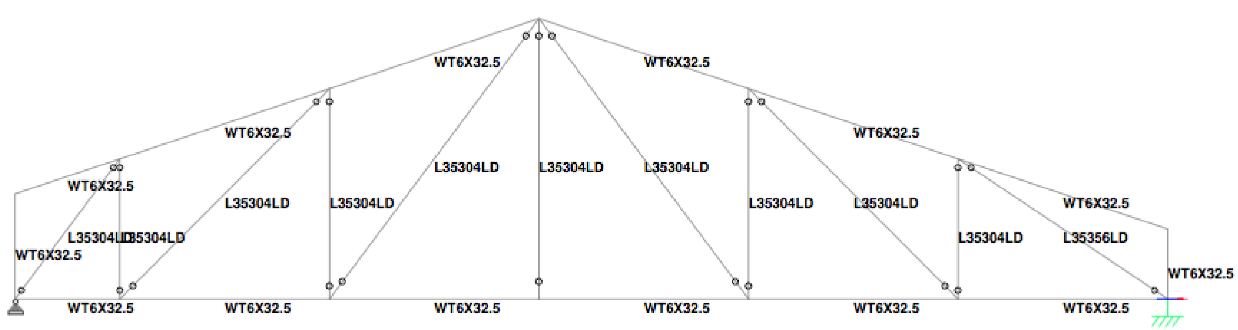


Figure 4-8 - Truss 3 Profile with Members

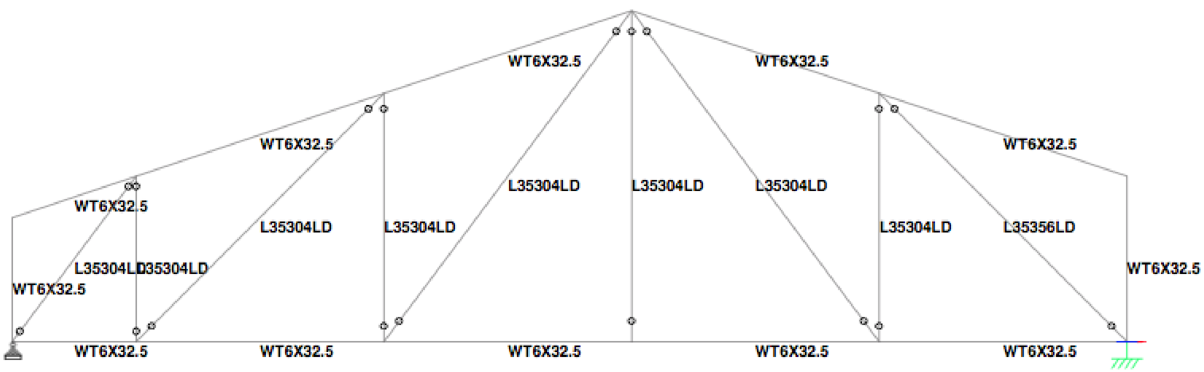


Figure 4-9 - Truss 4 Profile with Members

EnGrowth completed hand calculations for these trusses to verify that the STAADPRO outputs were accurate. The hand calculation for the 30-foot truss is shown in Appendix C to demonstrate the method used.

4.4 Mezzanine Floor Design

The steel design of the mezzanine floor was designed using a program called RAM Structural System. Figure 4-10 shows the designed floor system of the expansion along with the current mezzanine and Figure 4-11 shows the beams with their corresponding numbers.

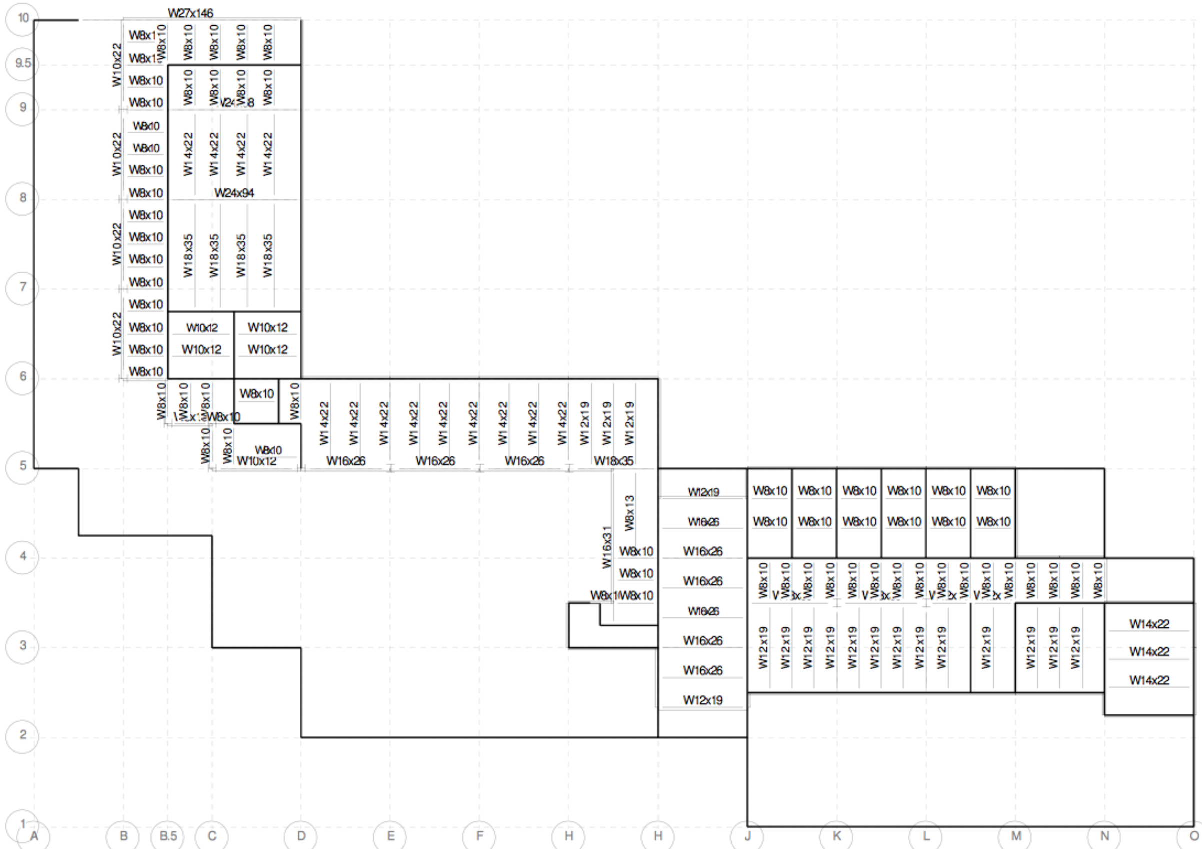


Figure 4-10 - Mezzanine Floor Beam Framing

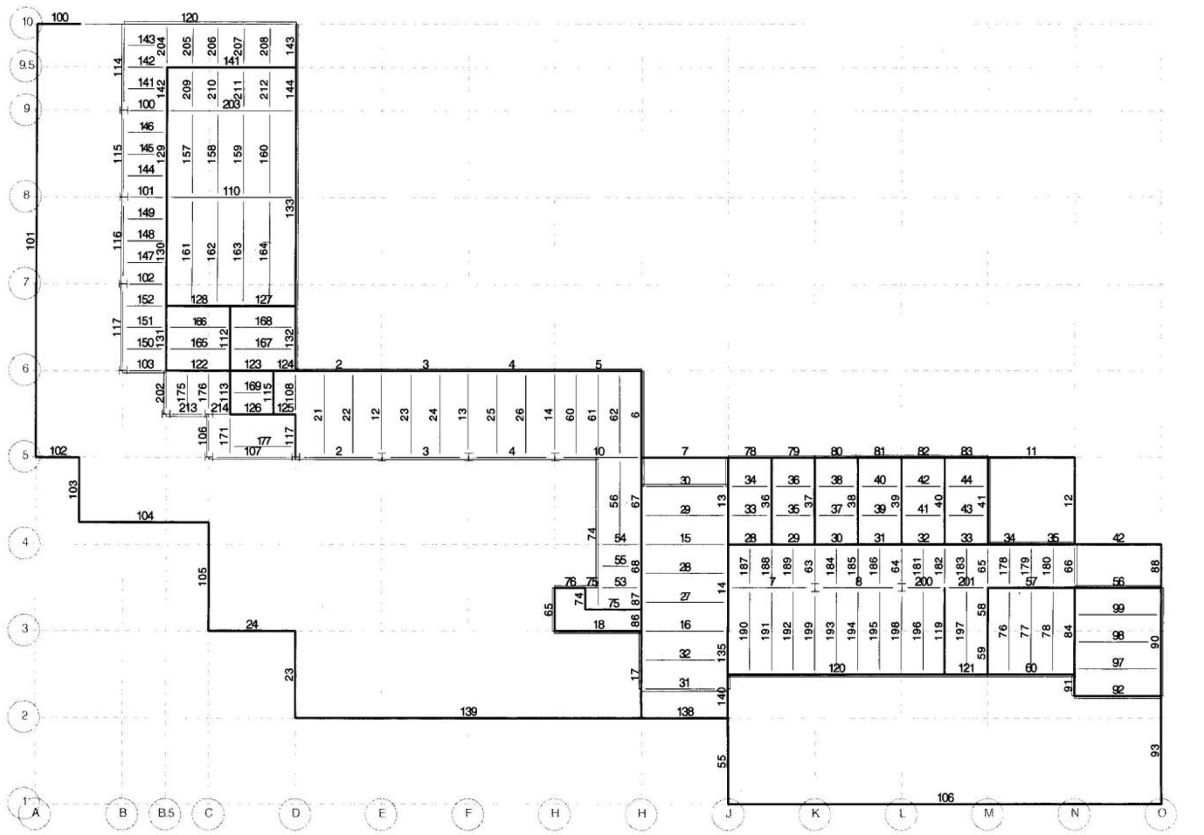


Figure 4-11 - Mezzanine Floor Beam Numbers

Some of the beams were designed by RAM Structural System with a camber in the middle to compensate for a deflection greater than 1 inch in the beam. EnGrowth assessed these beams and decided it was more desirable and cost efficient to design these beams with a larger depth to help reduce the deflection. Having a larger depth also allows for more utilities to run throughout the ceiling in between the beams.

Beam 164 was designed by RAM Structural System as a W16x26 beam with a 1-inch camber in the middle. Using the calculations found in Appendix D, a W18x35 with a deflection of 0.734 inches. This beam is only 9 pounds per foot heavier than the original 16x26 beam which not significant when compared to the tributary load of 500 pounds per foot on the mezzanine and beam.

These other beams with cambers were replaced by larger beams and input into RAM Structural System which verified the hand calculations and the beam choices. Hand calculations were also performed to verify the program correctly designed the rest of the beams as well.

4.5 Front Wall Beam and Column Design

The north steel cross bracing shear wall was modeled in STAADPRO. The members were optimized using the program. The members for the wall are shown in Figure 4-12.

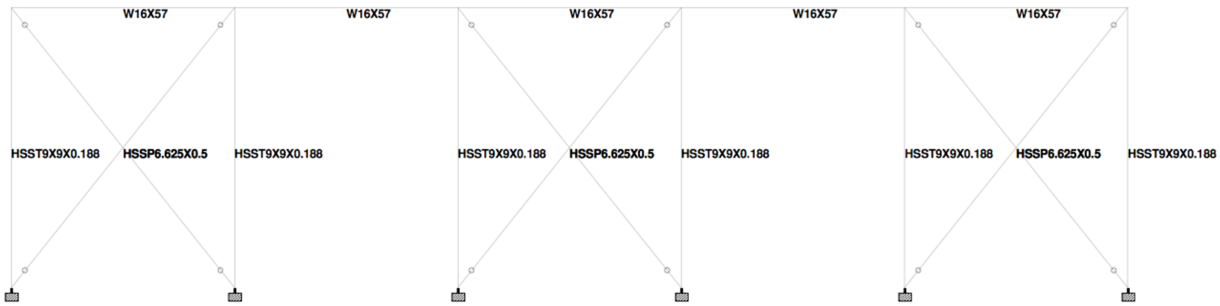


Figure 4-12 - Steel Shear Wall Frame with Member Sizes

Hand calculations were done to design the beams at the top of the shear wall and the vertical columns. However, these were only hand designed with axial loads. When the wind load was taken into account in the STAADPRO model, these beams increased in size to accommodate the horizontal forces. The columns were designed as square HSS pipes to allow the beams to easily tie into the columns. The cross bracing was designed as round HSS pipes for aesthetic purposes.

The front columns on the exterior of the expansion were designed as round HSS pipes. The factored reactions acting on these columns from the small, 25-foot truss were 28.8 kips. These columns were designed as HSS10x0.188 which has a capacity of 151 kips. These columns are quite capable of supporting the trusses acting on them.

4.6 Shear Wall Design

Charlie Raabe, a structural engineer at URS, assisted with the masonry design for the concrete shear wall. The design was done using a program called Structural Masonry Design System. The in-plane case was done with both wind loads and seismic loads. However, the wind loads controlled the design. The four masonry shear walls were designed with 8-inch concrete masonry units with #5 reinforcing bars 48 inches on center for in-plane loading. The out of plane case was analyzed for the smallest masonry shear wall, 15 feet, and it was calculated that #6 reinforcing bars at 48 inches on center were necessary. EnGrowth decided to use #6 bars at 48 inches on center for all of the walls, because the #5 and #6 bars are only different in size by 1/8 inch and are hard to tell apart in the field. The same bars are to be used throughout the site to eliminate #5 bars being mistakenly used in walls where the out of plane forces require it. Furthermore, since the worst case scenario was the 15 foot wall and the rest of the walls are greater in length, #6 bars will provide enough strength to handle the out of plane shear loads. The out of plane results showing the use of #6 bars can be located in Appendix E.

4.7 Interior Masonry Design

The interior masonry walls were also designed using Structural Masonry Design System. These were designed using #5 steel reinforcing bars located 48 inches on center. The walls were designed using the loads acting on the mezzanine. These walls are over designed for the loads. Figure 4-13 shows the interaction diagram of the walls. The points lie well inside the curved lines demonstrating they are not near the capacity of the walls.

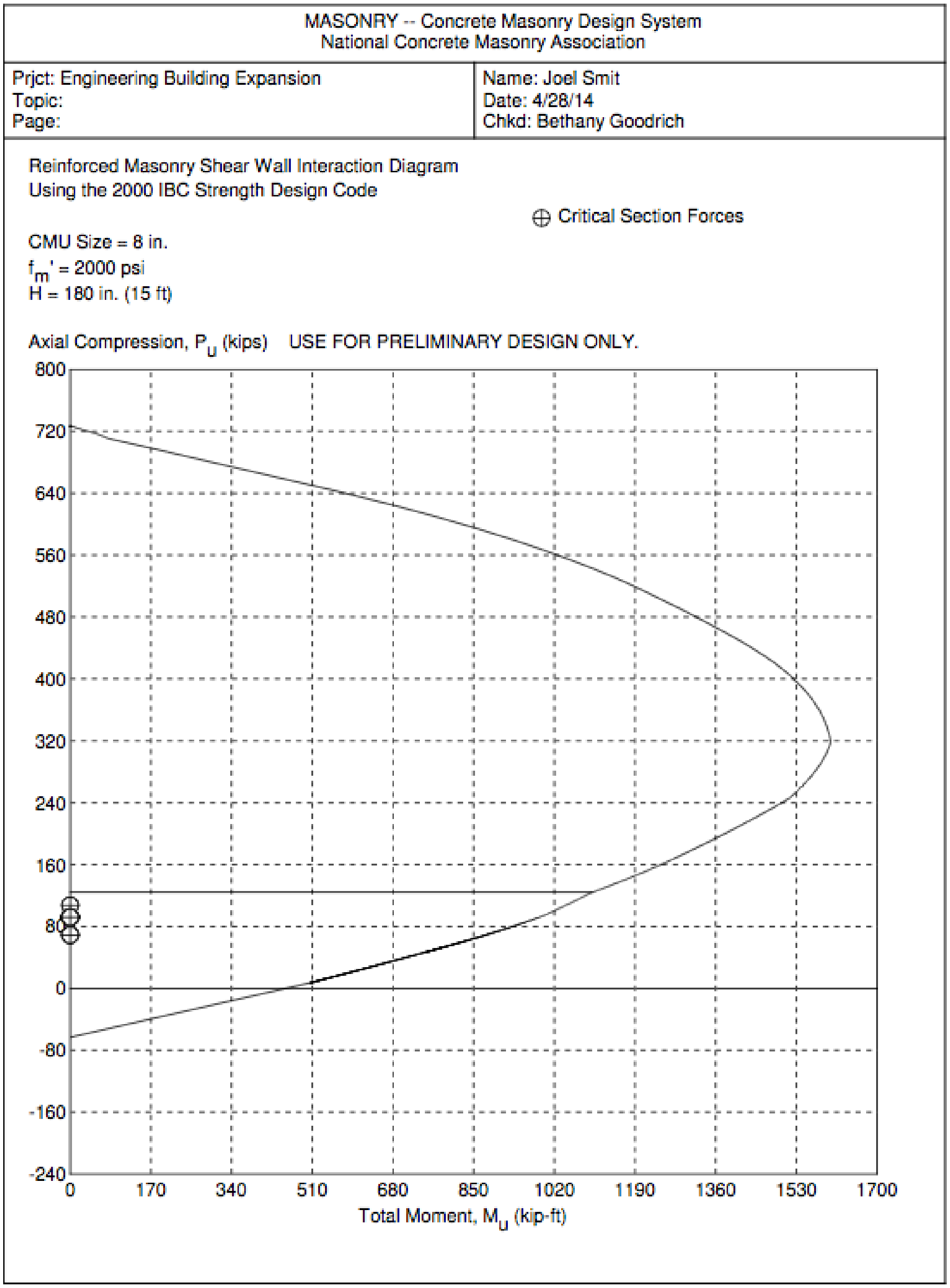


Figure 4-13 - Interior Wall Interaction Diagram

4.8 Foundation Design

The foundations were designed following procedures outlined in Chapter 15 of *Reinforced Concrete* by James K. Wight and James G. MacGregor. These procedures also referenced design specification sections from the *Building Code Requirements for Structural Concrete and Commentary* (ACI 318-05).

4.8.1 Continuous Footing Design:

A T-shape design was used for the wall footing design. A foundation wall, equal in width to the above grade wall, extends 3 feet below grade. Attached to the foundation wall is a 1-foot thick footing spanning at least 6 inches on both sides of the foundation wall, with the bottom of the footing at 4 feet below grade. Figure 4-14 shows an example of the continuous wall footing designs.

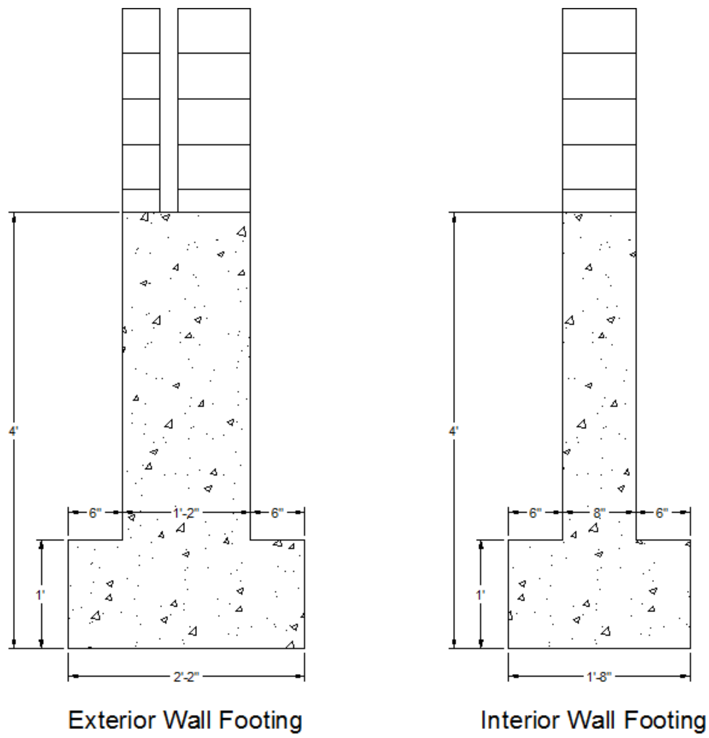


Figure 4-14 - Continuous Wall Footing Design

To simplify the continuous footing designs, openings in the above grade walls were ignored and large point loads, greater than 5 kips and not spanning a continuous wall length, were assumed to be transmitted to the soil by a column to a column footing.

To prevent soil settling, factored loads from the roof, mezzanine, walls, columns, and footings were used. These loads were not allowed to exceed the soil bearing capacity of 3500 psf distributed under the width of the footing. If the wall load exceeded the soil bearing capacity, the footing width was increased by 1-foot increments: 6 inches on each side of the foundation wall.

Next, rebar was designed to resist shear, flexural, and temperature and shrinkage forces. Detailed calculations for continuous foundation designs are found in Appendix F.

Table 4-6 summarizes the footing designs determined to adequately resist shear, bending, and temperature and shrinkage forces. These designs are detailed in the master plan set on drawing

S201.[8] Tensile reinforcing was not necessary for two of the footings because the footings only experienced compressive forces.

Table 4-6 - Footing Designs

Location	Dimensions	Temperature & Shrinkage Reinforcing	Tensile Reinforcing
Exterior	2'-2"x1'-0"	2 - #5	-
Exterior	3'-2"x1'-0"	3 - #5	#4 @ 9" O.C.
Interior	2'-8"x1'-0"	2 - #5	-

4.8.2 Column Footing Design:

The concrete column footing thickness was set at 12 inches throughout the expansion. The square size dimensions of the footing, l, were determined by dividing the axial load, carried through the columns, P_u, by the soil bearing capacity of 3500 pounds per square-foot as shown in the following equation.

$$l = \sqrt{\frac{P_u}{w_{soil}}}$$

Next, column base plates were designed by adding 6 inches, 3 inches on each side, to the depth of the column. The thickness of the base plate was determined by dividing in the applied column load, P_u, by the base plate yield strength, Fy, and dimensions, B and N, as noted in the equation below. Figure 4-15 annotates the variables used in the base plate design equation. Table 4-7 summarizes the various column base plates utilized in the structural design.

$$t = m\sqrt{\frac{(2P_u)}{0.90F_y(BN)}}$$

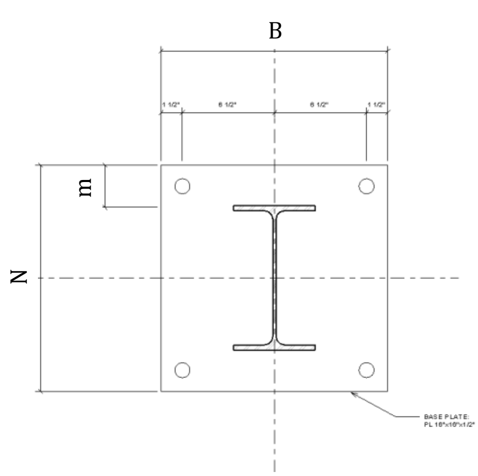


Figure 4-15 – Typical Steel Base Plate Design

8http://www.calvin.edu/academic/engineering/2013-14-team5/Resources/Engineering_Building_Expansion_Plan_Set.pdf

Table 4-7 - Base Plate Designs

Base Plate Schedule	Dimensions (NxBxT)	Column Type
BP-1	16"x16"x1/2"	W10x33
BP-2	15"x15"x1/2"	HSS9x9x1/8
BP-3	10"x10"x3/4"	HSS4x4x1/4
BP-4	16"x16"x1/4"	HSS10x0.188
BP-5	22"x22"x1/2"	W16x67

These base plates are detailed in the plan set on drawing S202. [9]

A footing schedule for the columns was developed with the appropriate steel reinforcing according to ACI 318-05. Similar to the continuous footings, steel reinforcing for the columns footings was designed to resist shear, bending, and temperature and shrinkage forces. To be consistent with the interior masonry walls throughout the expansion, #5 bars were used in most footing designs. Detailed column footing design calculations can be referenced in Appendix G. The final column footing designs are shown in Table 4-8 and detailed on drawing S201[10] in the final plan set.

Table 4-8 - Column Footing Designs

Footing Schedule	Dimensions (LxWxT)	Rebar Both Directions (Quantity - Bar Size - Spacing)	Service Load Capacity (kips)	Quantity
F-2.5	2'-6"x2'-6"x1'-0"	6 - #3 bars @ 4" O.C.	22	13
F-4.0	4'-0"x4'-0"x1'-0"	4 - #5 bars @ 12" O.C.	56	12
F-4.5	4'-6"x4'-6"x1'-0"	4 - #5 bars @ 16" O.C.	71	3
F-5.0	5'-0"x5'-0"x1'-0"	5 - #5 bars @ 12" O.C.	88	3
F-5.5	5'-6"x5'-6"x1'-0"	7 - #5 bars @ 10" O.C.	106	6

4.8.3 Design Summary

Sheet S101 of the final plan set shows the final layout of the foundations within the expansion to the Engineering Building.[11] Exterior footings extend 4 feet below grade to prevent frost from creeping into the soil below the building. Columns are connected to footings using a base plate and four anchor bolts. The location of the base plates are noted on drawing S102.

[9] http://www.calvin.edu/academic/engineering/2013-14-team5/Resources/Engineering_Building_Expansion_Plan_Set.pdf

[10] http://www.calvin.edu/academic/engineering/2013-14-team5/Resources/Engineering_Building_Expansion_Plan_Set.pdf

[11] http://www.calvin.edu/academic/engineering/2013-14-team5/Resources/Engineering_Building_Expansion_Plan_Set.pdf

Technical Memorandum 5: Roadway and Parking Lot Design

5.1 Site Development:

Figure 5-1 shows a comparison of the existing site and the new site proposed by EnGrowth. Due to an expressed desire from the Engineering Department to better integrate the building with the rest of campus, the design expands the Engineering Building towards North Hall, and re-routes Knollcrest Circle Drive to the west edge of the property, as shown. With this design, all academic buildings on Calvin's west campus are enclosed within Knollcrest Circle. Not only does this concept address campus integration, but EnGrowth also believes it improves the safety of the site by decreasing the intersection of vehicles on Knollcrest Circle and pedestrians crossing the road to access the Engineering Building, North Hall, Science Building, or DeVries Hall from the existing parking lots.

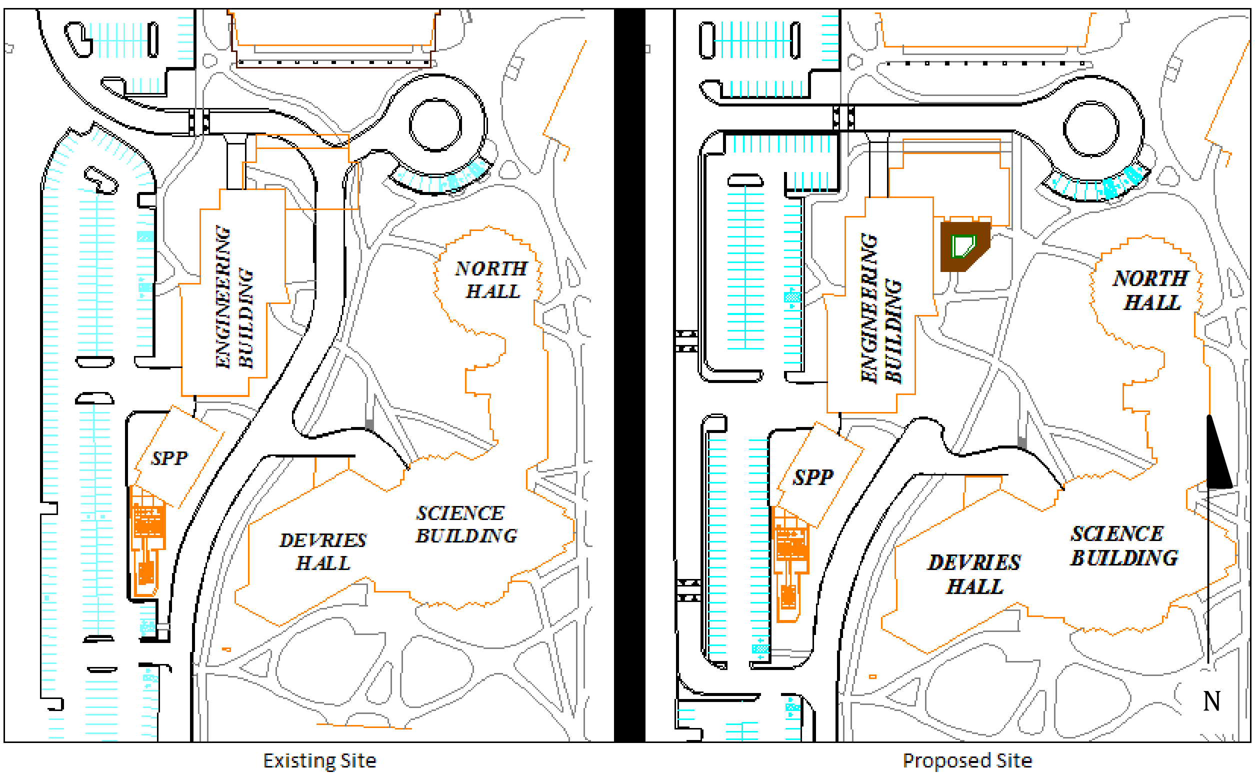


Figure 5-1 - Existing Site vs. Proposed Site

5.1.1 Roadway Design

As shown in Figure 5-1, Knollcrest Circle has been re-routed from the east side to the west side of the Engineering Building. The portion of the road immediately to the east of the power plant will remain intact for exclusive use as an access road to the loading dock adjacent to DeVries Hall/Science Building. The new section of roadway, through the existing parking lot, mimics the design of the rest of Knollcrest Circle. It is 24 feet wide (gutter pan to gutter pan), crowned with 2% slopes, and includes 2-foot rolled curbs. It consists of a 12 inch Class II sub-base, 6 inches of crushed concrete, and 3 inches of asphalt. A typical pavement cross-section is provided in Figure 5-2. Further site layout details are available on Sheet C103 on the team website[12] and in the attached drawing plan set. The turning radii for the new road have been sized to accommodate the AASHTO design vehicle classification of an Intercity Bus (BUS-40). This design vehicle most closely resembles the charter buses that often traverse the roadway carrying sports teams to and from the Spoelhof Fieldhouse Complex.

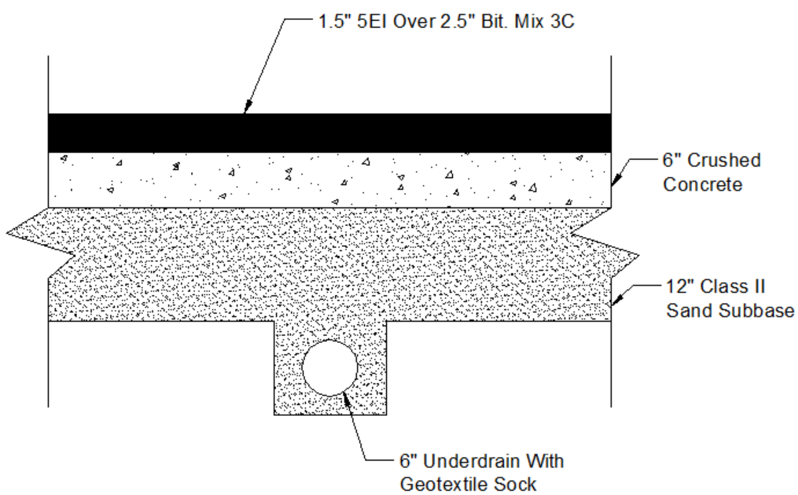


Figure 5-2 - Pavement Detail (Roadway & Parking Lots)

5.1.2 Parking Lot Design & Layout

The parking lots to the west of the Engineering Building needed to be redesigned due to the re-routing of Knollcrest Circle through these lots. The new parking lot will be designed to current Calvin College parking lot and roadway standards, shown in Figure 5-2. The existing area of study contains 222 parking spaces. It was discovered that these parking lots were not designed to current City of Grand Rapids Zoning Ordinance requirements. The new layouts are designed to adhere to this code; parking spaces are designed with a minimum width of 8.5 feet and minimum depth of 18 feet, while the parking lot aisle maintains a width of at least 22 feet. These new size requirements, in addition to the spaces lost due to the road relocation, restrict the new lots to 165 parking spaces. This loss of 57 parking spaces is a significant problem, as a feasible design must actually *increase* parking availability due to an increase in building square footage (Grand Rapids Zoning Ordinance). EnGrowth has proposed possible options to mitigate the issue, though the detailed planning and design of these options has been deemed outside of the team's design scope, so as not to distract from the primary design components.

[12] http://www.calvin.edu/academic/engineering/2013-14-team5/Resources/Engineering_Building_Expansion_Plan_Set.pdf

5.1.2.1 Parking Mitigation Option 1: Land Acquisition

There are no suitable places on campus to add a parking lot. However, if Calvin desires to fully satisfy all the necessary parking requirements outlined in the Grand Rapids Zoning Ordinance, one option is to acquire additional land and construct a parking lot or structure there. A number of residential lots exist just to the south of Calvin's baseball field and to the west of the Spoelhof Fieldhouse Complex, some of which are already owned by Calvin. This area was mentioned to EnGrowth as a potential location for a future parking structure should Calvin purchase a few more of the lots. The location for this option is shown in Figure 5-3. The proposed structure is shown in blue, Calvin-owned property is outlined in orange, and the houses further west outlined in white as possible land acquisition locations.

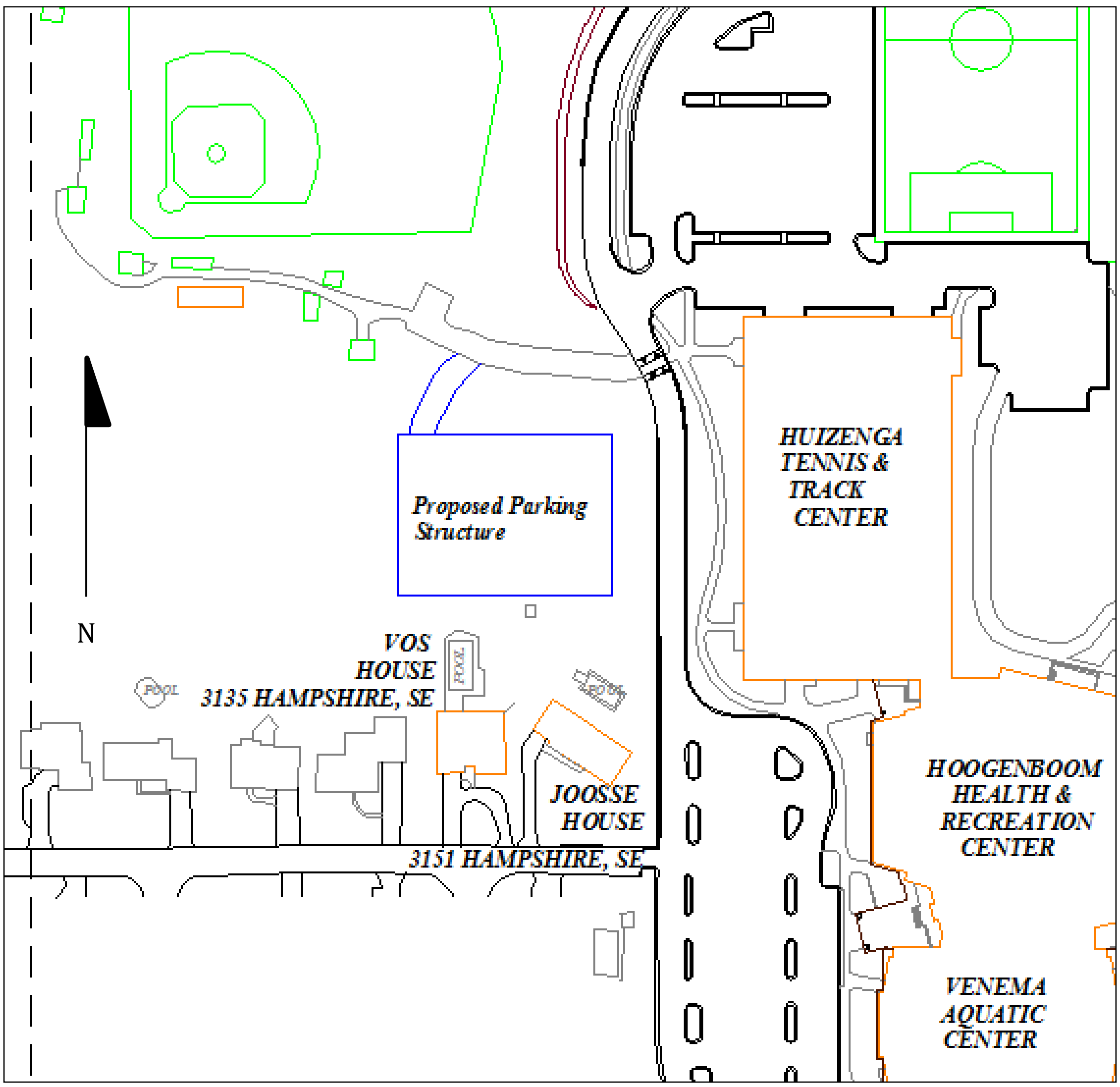


Figure 5-3 - Possible Parking Structure Location

5.1.2.2 Parking Mitigation Option #2: Policy Change

An alternate option that would allow greater parking availability for faculty, staff, and commuter students would be to prohibit freshman, and perhaps even sophomores, from having cars on campus. This practice is common at many other colleges and universities. According to Calvin's Campus Safety office, there have been 693 parking permits given to freshman and sophomores. It is not possible to determine specifically how many freshmen park a car on campus, but this option would significantly decrease campus parking requirements.

Technical Memorandum 6: Utility Design

6.1 Utility Relocation and Design

One of the main components of the site plan design is the re-routing and design of the existing utilities. Currently all of the utilities follow the general path of the existing Knollcrest Circle Drive. As a result, they will pass under the proposed location of the expansion for the Engineering Building (Figure 6-1).

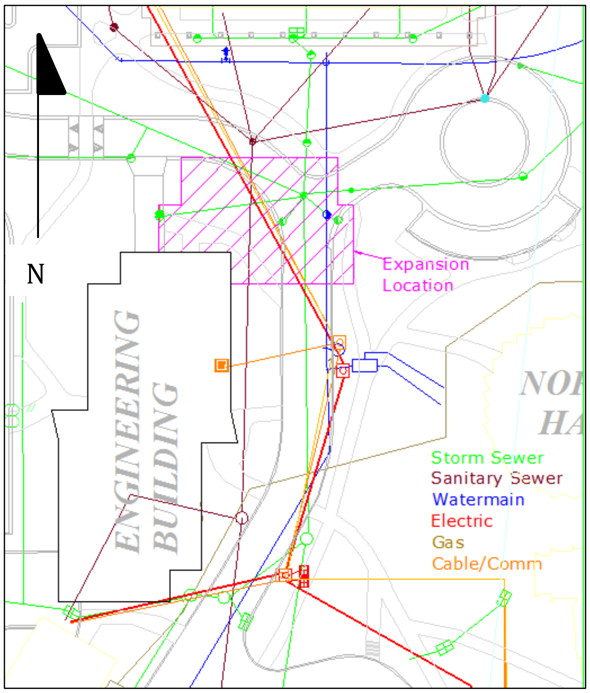


Figure 6-1 - Existing Utilities in Conflict with Proposed Expansion

As seen in Figure 6-1, the gas main does not present any conflict with the proposed expansion and will need to be avoided during the construction process, specifically in the removal of the existing road. The relocation of the water main, electric, cable, and communication utilities is simple because they can be rerouted around the building expansion and re-laid at their existing depths.

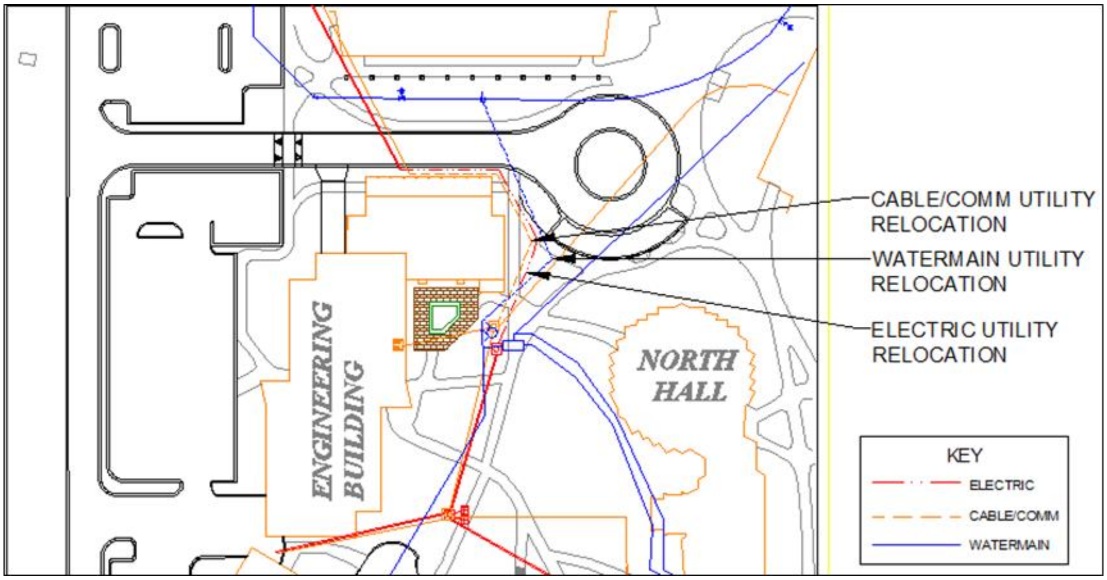


Figure 6-2 - Relocation of Watermain, Electric, and Cable Utilities

The relocation of the storm and sanitary sewers is more complex because they depend on gravity for drainage. Figure 6-3 shows the proposed relocation for the sanitary sewer.

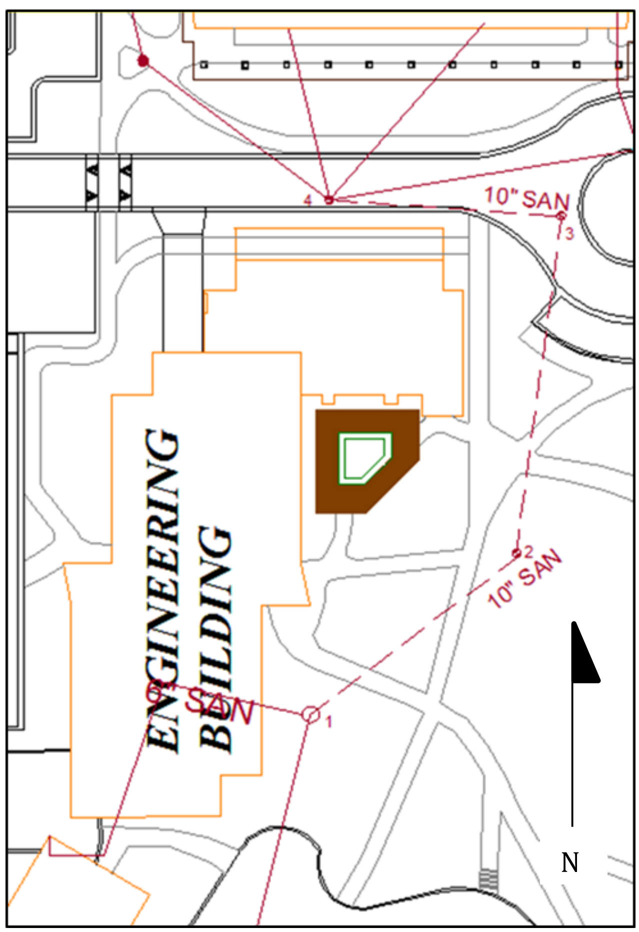


Figure 6-3 - Proposed Location of Sanitary Sewer

The diameters of the proposed additional sanitary pipes will be 10 inches (the same diameter as the pipe to be removed). Table 6-1 shows the proposed invert elevations as well as the resulting slopes of the additional pipes. For a 10-inch diameter pipe, the minimum slope is 0.0025, which is met for all three proposed pipe lengths.

Table 6-1 - Sanitary Sewer Slope Analysis

Manhole	Invert Elev. [ft]	DS Pipe length [ft]	Slope
4 (EX)	760.41	110	0.003
3 (PROP)	760.08	160	0.003
2 (PROP)	759.60	125	0.003
1 (EX)	759.22		

There is an increase of approximately 4500 square feet of impervious surface area on the site. Due to this increase, storm water runoff management is a key aspect of the site design. The additional runoff that is generated (plus some of the existing runoff from the site) will be handled by the use of a rain garden, or bioretention system (See Technical Memorandum 7). In addition, the three pipes (labeled in pink in Figure 6-4) that are being added will be designed to handle the necessary runoff from Calvin's campus.

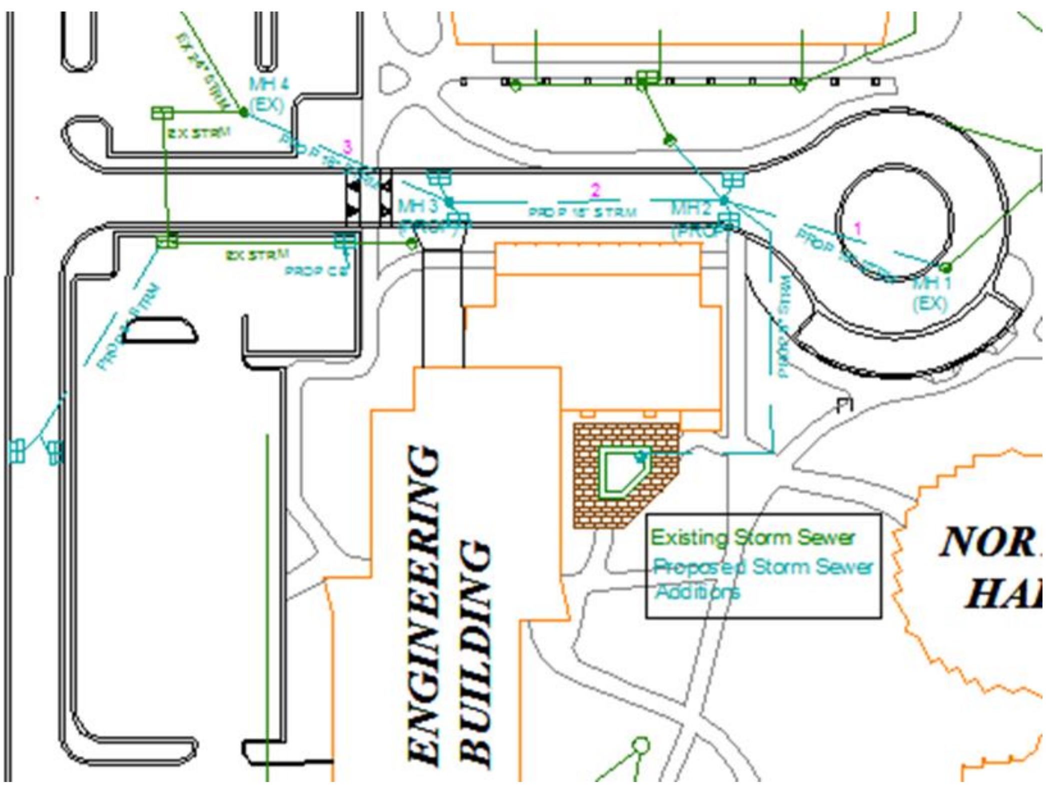


Figure 6-4 - Proposed Storm Sewer

An analysis of the existing storm sewer system was done to design the new pipes and manholes. The rational method for storm water conveyance was used to determine whether the existing system can handle the runoff from the proposed site as well as to determine the invert elevations of manholes 2 and 3 and the diameters of pipes 1, 2, and 3 (in pink). The additional pipes were designed to handle a 10-yr storm. The Microsoft Excel file that was used for this process can be found on the team's website.[13] The rational equation used as the basis for this design can be seen below.

$$Q = CIA$$

Where,

Q = Flow Rate into System (cfs)

C = Average Rational Coefficient over Total Drainage Area

I = Rainfall Intensity

A = Area of Watershed (acre)

A topographic map of Calvin's campus was used to conduct a watershed delineation to determine what area of Calvin's campus drains into the retention pond near Lake Drive via the system that passes by the Engineering Building, Spoelhof Fieldhouse Complex, and baseball field. Figure 6-5 shows the area of Calvin's campus using this path determined by the watershed delineation.

[13] http://www.calvin.edu/academic/engineering/2013-14-team5/Home.html

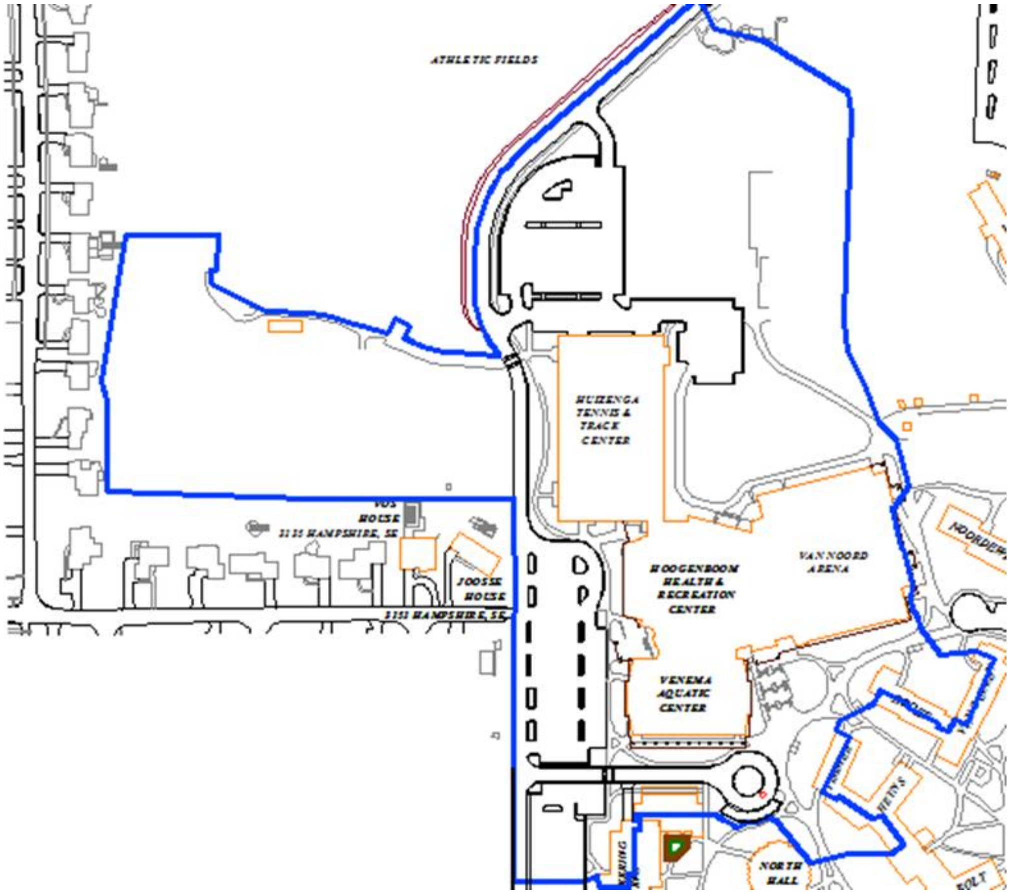


Figure 6-5 - Stormwater Contributing Area

This analysis was challenging as the invert elevations and pipe diameters in the system upstream of the manhole located at the cul-de-sac intersection were unknown. For this reason, three different areas were analyzed as individual “watersheds”, which can be seen in Figure 6-6.

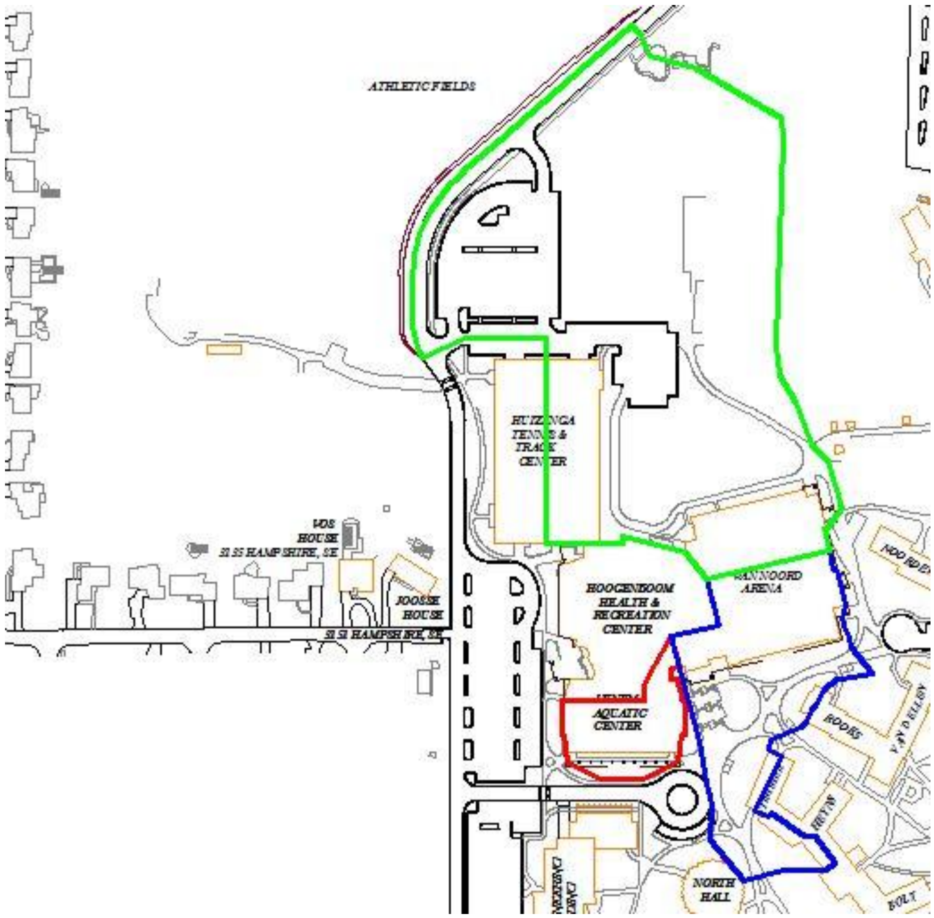


Figure 6-6 - The Three "Upstream Watersheds"

Flow rates from the three different watersheds into different manholes in the existing system were determined using the rational method. See Appendix H for tables outlining this process. Table 6-2 shows the flow rates determined for various inlets into the system.

Table 6-2 - "Individual Watershed" Flow Rates

Flow Rates Added from "Individual Watersheds"	
Q into MH 1 [cfs]	5.65
Q into MH 2 [cfs]	1.98
Q into MH 2 (from RG) [cfs]	0.28
Q into MH 9 [cfs]	16.27

These flow rates were then input into the master storm sewer design spreadsheet. The use of the rational method required the determination of totals for the different land use types (grass/landscaped, roof, and concrete). Figure 6-7 shows the area determination for each land use.

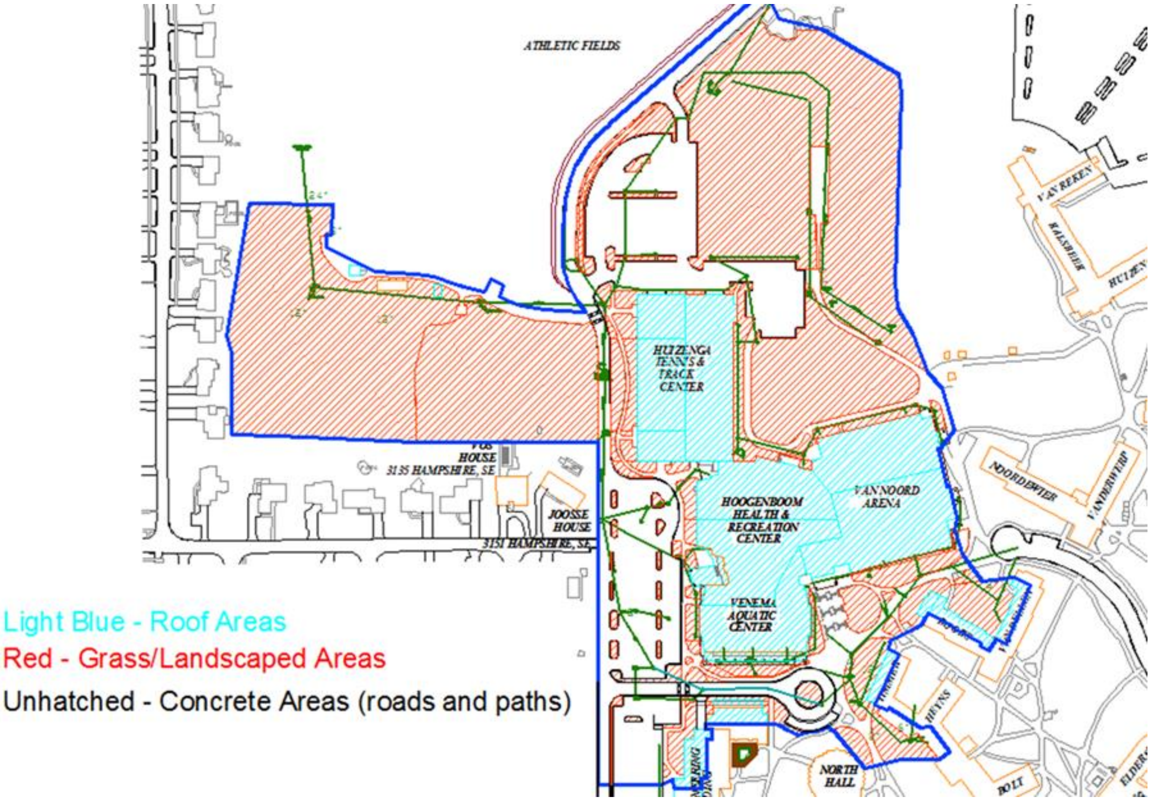


Figure 6-7 - Drainage Area Determination by Type

For the design of the new pipes, the area outlined in black in Figure 6-7 was used as the upstream drainage area. Using an iterative process of setting the pipe diameters and checking whether they had the capacity to carry the amount of flow necessary, it was determined that the new pipes had to be 18"-diameter pipes. Tables 6-3 and 6-4 show an outline of this process for the three pipes being designed. In Table 6-4 the "Capacity" column must have a greater value than the "Q" column to ensure the pipe is big enough.

Table 6-3 - Pipe Information for New Pipes

From Node					Pipe				To Node				
MH #	Ground Elevation [ft]	Invert Elevation [ft]	Crown Elevatoin [ft]	Cover [ft]	Diameter [in]	Length [ft]	Minimum Slope	Slope	MH #	Ground Elevation [ft]	Invert Elevation [ft]	Crown Elevation [ft]	Cover [ft]
1	779	769.19	770.69	8.31	18	113	0.0028	0.003	2	774	768.85	770.35	3.65
2	774	768.85	770.35	3.65	18	131	0.0028	0.003	3	776	768.46	769.96	6.04
3	776	768.46	769.96	6.04	18	105.5	0.0028	0.003	4	775.8	768.14	769.64	6.16

Table 6-4 - Preliminary Hydraulic Analysis for New Pipes

MH #	Dist to next DS MH	New CA	Accum CA	Time [min]	Rainfall Intensity [in/hr]	Q [cfs]	Slope	K (Q/s^.5)	Minimum Size [in]	K	Capacity [cfs]	Q/Qo	V/Vo	V [ft/s]	Travel time [min]
1	113					5.65	0.003	103	18	105.1	5.76	0.98	1.14	3.71	0.51
2	131	0.51	0.51	20.51	3.02	3.81	0.003	70	18	105.1	5.76	0.66	1.07	3.47	0.63
3	105.5	0.16	0.67	21.14	2.98	2.01	0.003	37	18	105.1	5.76	0.35	0.91	2.96	0.59

See Appendix I for tables showing process for the existing system (from circle drive to retention pond).

Technical Memorandum 7: Rain Garden and Park Design

7.1 Environmental Sustainability

Team EnGrowth believes that they are called to be environmental stewards of God's creation. As Christians first, and beyond that as Christian engineers, it is largely their responsibility to be at the forefront of reversing the trend of environmental degradation. They must not only be aware of their call to stewardship, but also that as engineers they have significant influence over processes that, if handled improperly, can lead to environmental degradation. This combination puts them in a unique position, one that should not be taken lightly. With this philosophy, Team EnGrowth has been intentional in ensuring that their design incorporates sustainable solutions and properly addresses any environmental concerns that may be related to the project.

7.2 Low Impact Development

The primary area in which Team EnGrowth has incorporated sustainable solutions to the Engineering Building expansion project is through the implementation of a set of land development strategies known as Low Impact Development (LID). The aim of Low Impact Development is to handle a site's storm water management in an environmentally sustainable way. Storm water has traditionally been managed with conveyance techniques: the removal of storm water runoff from a site as quickly as possible. In recent years, it has been discovered that many of these conveyance practices can lead to the subsidence of natural freshwater aquifers; conveyance often causes fresh water supply to exit its natural watershed, which interrupts the Earth's natural hydrologic cycle. LID aims to reverse this trend by promoting greater amounts of infiltration on a site, providing aquifer recharge as well as reducing the amount of pollutants in groundwater.

7.3 Rain Garden (Bioretention System) Design

The primary Low Impact Development technique that Team EnGrowth has implemented is a rain garden. Rain gardens resemble traditional gardens, but employ carefully layered soils and native plants to remove pollutants, especially heavy metals, such as Pb, Cu, and Zn, from storm water runoff through a combination of physical, chemical, and biological methods.[14] Additionally, they serve as small-scale infiltration basins to promote aquifer recharge.

Though a capacity analysis confirmed that the existing storm sewer system could handle any additional runoff created by our development, our team decided to divert the runoff from ~18,000 square feet of the site to a rain garden. This option proved much more consistent with EnGrowth's dedication to environmental stewardship. The proposed bioretention system is situated in the

[14] "LID Urban Design Tools - Bioretention." *LID Urban Design Tools - Bioretention*. N.p., n.d. Web. 15 Dec. 2013.

building bend created by the proposed expansion as shown in Figure 7-1. A profile of the rain garden is shown in Figure 7-2.

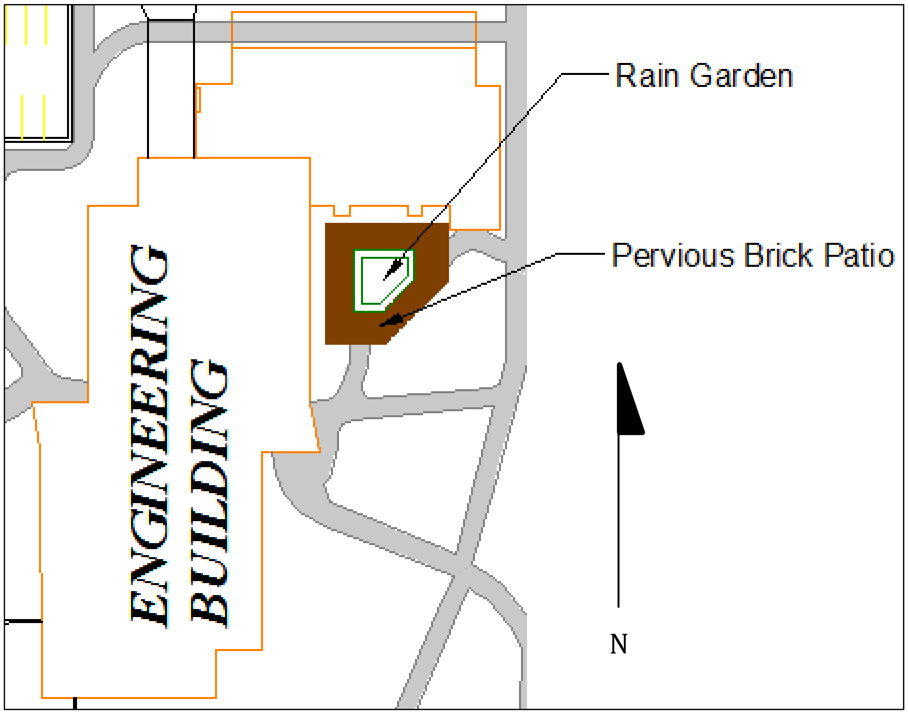


Figure 7-1 - Proposed Rain Garden Location

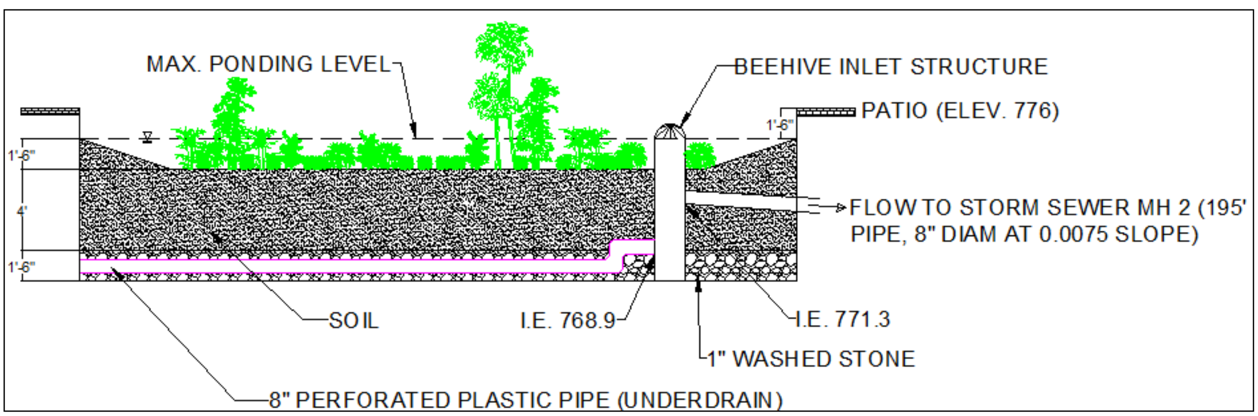


Figure 7-2 - Rain Garden Profile Drawing

The majority of the rain garden influent will come from the roofs of the existing north bay of the building and the expansion wing. In addition to its functional purposes, the garden will be able to be used for civil/environmental class demonstrations, consistent with the original design and intent of the Engineering Building. The rain garden was designed to the specifications provided in the Southeast Michigan Council of Governments (SEMCOG) "Low Impact Development Manual for Michigan," and is sized to accommodate its designated watershed for a 2-year, 24 hour storm. Using this design storm and the drainage area shown in Figure 7-3, the SCS Curve Number method computed a required storage volume of approximately 1,850 cubic feet. EnGrowth's design provides 1,915 cubic feet of volume.

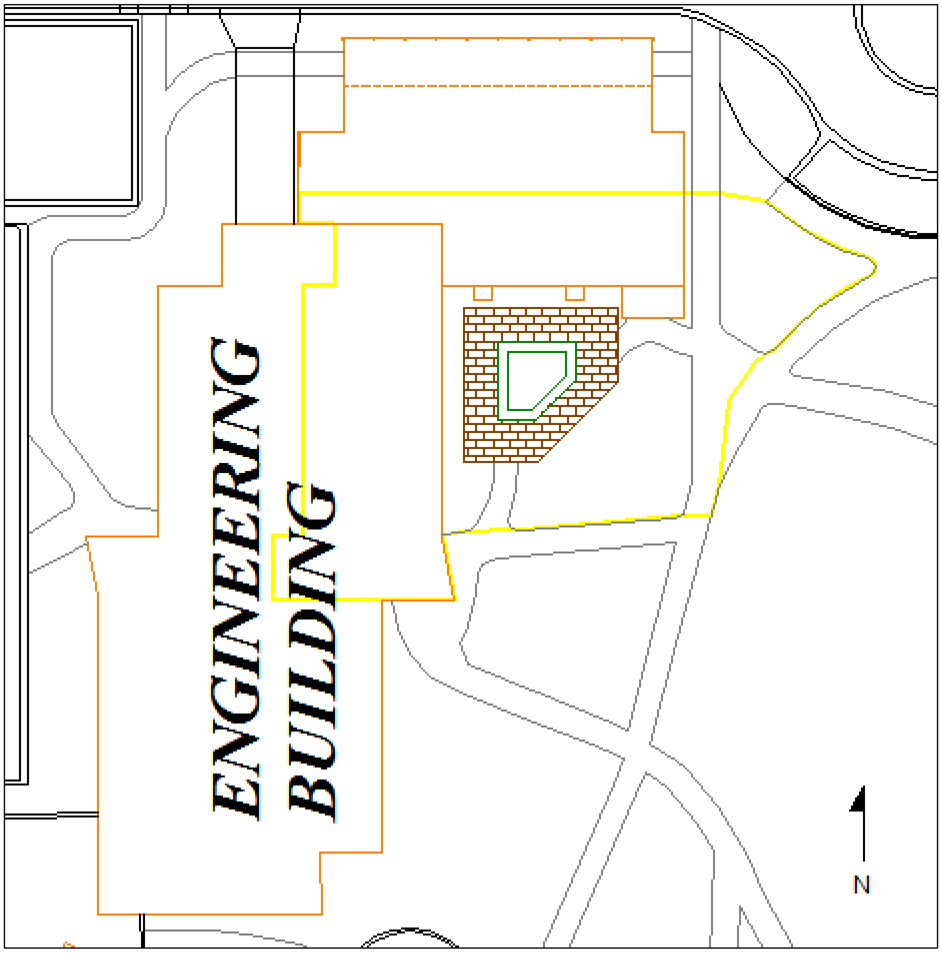


Figure 7-3 - Rain Garden Contributing Drainage Area (Outlined in Yellow)

Full design calculations, as well as filter soil and plant specifications for the garden are provided in the Appendix J. As Calvin's campus is located in a region of very clayey soils, an underdrain to the storm sewer system was designed for the rain garden system to accommodate excess water that cannot infiltrate into the ground.

EnGrowth's proposed rain garden will be located within a courtyard/patio area between the Engineering Building and Science Building as a place for students to gather together to study, converse, and engage in recreational activities (refer to Technical Memorandum 6: Design Alternatives). EnGrowth believes this idea further accomplishes the goal of integrating the Engineering Building with the rest of campus by creating a more inviting and inclusive atmosphere around the facility. To adhere to the team's commitment to environmental stewardship, this patio has been designed with pervious brick pavers.

Technical Memorandum 8: Appendices

8.1 Appendix A

PROJECT COST BREAKDOWN

EnGrowth's Projected Project Cost

Expansion Project Initial Construction Cost Estimate

	DESIGN WORK	LABOR HOURS	HOURLY WAGE ($)	LABOR COST ($)
DESIGN COST ESTIMATE	Architectural Design Work	100	$ 80.00	$8,000
	Structural Engineering Design Work	300	$ 120.00	$36,000
	Site Engineering Design Work	250	$ 90.00	$22,500
	Subtotal	**650**		**$66,500**
	LINE ITEMS	**Square-Footage**	**Unit Cost ($/ft^2)**	**Cost**
Construction Cost Estimate	EB Expansion	10,550	$ 290.00	$3,059,500
	EB Rennovation	3,000	$ 290.00	$870,000
	Site Civil Cost	0	$ -	$355,000
	Subtotal			**$4,284,500**

Subtotals		**$4,351,000**
Risk (Contingency)	10%	$435,100
Total Project Cost Estimate (Design + Construction)		**$4,786,100**
Total Project Cost Estimate		**$4,800,000**

Building Cost Breakdown

Autocad Drawings - First Semester

Location	Area (sq-ft)
Project	9733.3
Research	8200.6
*Total	17933.9

* Use 17900 for cost estimate

Floor Plans - Second Semester

Location	Area (sq-ft)
South Bay - 1st floor	7600
South Bay - Mezzanine	4000
Hall 1st Floor	1200
Hall 2nd Floor	1200
North Bay - First Floor	7600
North Bay - Mezzanine	2500
Total	24100

RS MEANS SQUARE FOOT COST (2014):

1999 - Cost	2014 - Cost	Percent Increase
$ 100.90	$ 186.70	85%

Cost Pro-Rated for Expansion & Renovation

Bid Value 1998	2014 Cost (RSMeans % Increase)	Building Sq-Ft	Unit Cost
$ 2,700,000	$ 4,995,000	24100	$ 207.26
$ 2,700,000	$ 4,995,000	17900	$ 279.05

*Use $290.00 for Estimate to account for chemistry lab & glass curtian wall

Site Cost Breakdown

	SOURCE	PAY ITEM	DESCRIPTION	UNIT	EST QUANTITY	UNIT PRICE	AMOUNT
Removals	COW	1170	Remove Ex Watermain	Ft	182	$ 8.54	$ 1,554.28
	MDOT	2030011	Dr Structure, Rem	Ea	5	$ 223.44	$ 1,117.22
	MDOT	2030015	Sewer, Rem, Less than 24" (San and Strm)	Ft	684	$ 7.45	$ 5,093.34
	MDOT	2030016	Sewer, Rem, 24" to 48" (Strm)	Ft	380	$ 9.90	$ 3,762.30
	MDOT	2040020	Curb and Gutter, Rem	FT	3100	$ 5.84	$ 18,118.88
	MDOT	2040050	Pavt, Rem	Syd	2311	$ 4.45	$ 10,287.22
	MDOT	2040055	Sidewalk, Rem	Syd	378	$ 6.95	$ 2,624.50
Improvements	COW	3180	Sanitary Sewer 10"	Ft	395	$ 73.20	$ 28,914.00
	COW	3275	Sanitary Sewer Drainage Structure STD 4' Dia	Ea	2	$ 1,952.00	$ 3,904.00
	MDOT	3020016	Aggregate Base, 6 inch (crushed concrete)	Syd	2311	$ 5.30	$ 12,258.13
	MDOT	3020022	Aggregate Base, 9 inch	Syd	756	$ 7.58	$ 5,728.32
	MDOT	4020002	Sewer, Cl A, 8 inch, Tr Det A	Ft	310	$ 30.95	$ 9,594.62
	MDOT	4020006	Sewer, Cl A, 18 inch, Tr Det A	Ft	350	$ 34.85	$ 12,197.64
	MDOT	4030010	Drainage Structure Cover (San and Strm)	Ea	4	$ 408.36	$ 1,633.42
	MDOT	4030200	Dr Structure, 24 inch dia (CB for Strm and RG)	Ea	8	$ 874.04	$ 6,992.29
	MDOT	4030210	Dr Structure, 48 inch dia (MH)	Ea	2	$ 1,338.48	$ 2,676.96
	MDOT	4040043	Underdrain, Pipe, Open-Graded, 6"	Ft	570	$ 4.28	$ 2,442.34
	MDOT	4040063	Underdrain, Subbase, 6"	Ft	570	$ 3.30	$ 1,879.18
	MDOT	5010031	HMA, 3C (1.5")	Ton	191	$ 59.55	$ 11,354.28
	MDOT	5010034	HMA, 36A (1.5")	Ton	62	$ 66.15	$ 4,123.62
	MDOT	5010056	HMA, 5E1 (1.5")	Ton	191	$ 65.75	$ 12,536.10
	URS Proj		Bit. Mix 11L (1.5")	Ton	62	$ 43.70	$ 2,723.97
	MDOT	8020015	Curb and Gutter, Conc	Ft	3640	$ 15.60	$ 56,784.00
	MDOT	8230052	Gate Valve and Box, 8 inch	Ea	1	$ 1,282.21	$ 1,282.21
	MDOT	8230156	Water Main, DI, 8 inch	Ft	210	$ 67.60	$ 14,196.00
	URS (Leo)		tapping sleeve and valve	Ea	1	$ 10,000.00	$ 10,000.00
	SEMCOG		Rain Garden (per vol of storage)	Cf	1915	$ 7.00	$ 13,405.00
			Pervious Bricks	SF	8	$ 621.50	$ 5,027.94
	KV Landscaping Inc		Sand Subbase	CF	1240	$ 0.40	$ 496.00
			Items Total				$ 262,707.76
	URS (Leo)		Contingency (30%)	LS			$ 78,812.33
	URS (Leo)		Mobilization (5%)	LS			$ 13,135.39
	TOTAL EST CONSTR COST						$ 354,655.47
						Use	$ 355,000.00

Adjustment Factors

Source	Year	Factor
MDOT	2012	1.04
COW	2003	1.22

multiplied unit costs by 2% per year

8.2 Appendix B

ROOF BEAM TYING 55-FOOT TRUSS TO SOUTH WALL

CALVIN Engineering

Name:

Class:

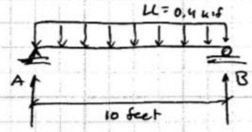
Date:

Beam tying 55-foot truss to south wall

$SL = 0.221$ klf

$DL = 0.22$ klf

$LL = 0.4$ klf



The decking is perpendicular to the beam

Assume full lateral bracing of compression flange $L_b = 0$ feet

Load Case $1.2DL + 1.6LL + 0.5SL = 1.2(0.22\text{ klf}) + 1.6(0.4\text{ klf}) + 0.5(0.221\text{ klf}) = 1.015\text{ klf}$

Reactions $R_A = R_B = \frac{1}{2}(1.015\text{ klf})(10\text{ feet}) = 5.07^k$

$$M_u = \frac{w\ell^2}{8} = \frac{(1.015\text{ klf})(10\text{ feet})^2}{8} = 12.75\text{ ft-kips}$$

Deflection $w = 0.22 + 0.4 + 0.221 = 0.841$ klf

WT6x32.5

E=29000 ksi

I=20.6 in⁴

$$\Delta max = \frac{5w\ell^4}{384EI} = \frac{5(0.841/12)(10\times12)^4}{384(29000\text{ ksi})(20.6\text{ in}^4)} = 0.317\text{ in}$$

$$Z_{req} = \frac{M_u}{\phi F_y} = \frac{(12.75\text{ ft-kips})(12\frac{in}{ft})}{0.9(50\text{ ksi})} = 3.40\text{ in}^3$$

$\boxed{\text{Choose a WT6x32.5}}$ $Z_x = 7.5\text{ in}^3$ AISC Table 1-8

8.3 Appendix C

STAADPRO TRUSS HAND CHECK – 30 FOOT TRUSS

CALVIN
Engineering

Name: ____________
Class: ____________
Date: ____________

Truss Member Checks

30 ft Truss, Vertical Diaphram, Beam 12, 2L3½x3x¼ (T)

Staad Results
P_u = 5.091 KIPS
L = 9.995 ft

Axial Tension (AISC 5-8)

2L3½x3x¼ Ag = 3.16 in² Yielding = ϕP_N = 102 KIPS ← controls
Ae = 2.37 in² Rupture = ϕP_N = 103 KIPS

$$\text{Ratio} = \frac{P_U}{\phi P_N} = \frac{5.091^{\text{KIPS}}}{102^{\text{KIPS}}} = 0.0499 = 0.050 \checkmark$$

Staad Ratio = 0.05 ✓

Staad Results ok

∴ Staad Models accurate @ design

8.4 Appendix D

RAM STRUCTURAL SYSTEMS HAND CHECKS

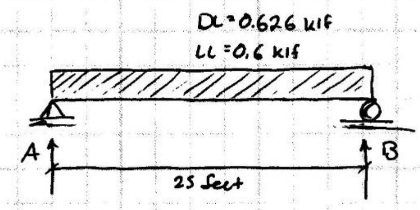
CALVIN Engineering

Name: ______________________

Class: ______________________

Date: ______________________

Beam 164 (Computer Room)



DL = 100 psf
LL = 100 psf
D_B = 0.026 klf

Tributary Width $= \frac{30\text{ feet}}{5\text{ beams}} = 6$ feet/beam

DL = (100 psf)(6 feet/beam) = 0.6 klf ✓
LL = (100 psf)(6) = 0.6 klf

Load Case 1.2DL+1.6LL = 1.2(0.626 klf) + 1.6(0.6) = 1.71 klf

Moment

$M_{max} = \frac{w\ell^2}{8} = \frac{(1.71\text{ klf})(25\text{ feet})^2}{8} = 133.59$ ft-kips ✓

Reactions

$R_A = R_B = \frac{1}{2}(1.71\text{ klf})(25\text{ feet}) = 21.38^k$ ✓

Deflection w = 0.626 + 0.6 = 1.226 klf

W16
E = 29000 ksi
I = 301 in⁴

$\Delta_{max} = \frac{5w\ell^4}{384EI} = \frac{5\left(\frac{1.226}{12}\right)(25\times12)^4}{384(29000)(301)} = 1.23$ in Too much deflection with W16

$Z_x = \frac{M_u}{\phi F_y} = \frac{(133.59\text{ ft-kips})(12)}{0.9\,(50\text{ ksi})} = 35.62\text{ in}^3$ Can use W12×26, $Z_x = 37.2\text{ in}^3$

Instead of cambering the beam to offset this deflection, try a W18.

CALVIN Engineering

Name:
Class:
Date:

W18×35 $D_B = 0.035$ klf

$$\Delta max = \frac{5 w \ell^4}{384 EI} = \frac{5\left(\frac{1.235}{12}\right)(25\times12)^4}{384(29000)(510)} = 0.734 \text{ in}$$

$$\Delta allow = \frac{L}{240} = \frac{25\times12}{240} = 1.25 \text{ in}$$

$E = 29000$ ksi
$I = 510 \text{ in}^4$

$$W = \overbrace{(0.6+0.035)}^{DL} + \overbrace{0.6}^{LL} = 1.235$$

A W18×35 has a Z_x of 66.5 in^3, this is higher than the necessary Z_x of 35.62 in^3

Using a W18×35 for the 25 ft beam span is more desireable than cambering a W16×26 because it does not need to be cambered and it provides more storage space in the ceiling for mechanical and pipes above the chemistry lab.

W12×26 deflection analysis

$$\Delta max = \frac{5\left(\frac{1.226}{12}\right)(25\times12)^4}{384(29000)(204)} = 1.821 \text{ in}$$ <u>too much deflection</u>

$I = 204 \text{ in}^4$
$D_B = 0.026$ klf
$W = 0.6+0.026+0.6 = 1.226$ klf

W18×35 $(Z_x = 66.5 \text{ in}^3)$

load case $= 1.2(0.635) + 1.6(0.6) = 1.722$ klf

$$\text{Moment} = \frac{(1.722)(25)^2}{8} = 134.53 \text{ ft-kips}$$

$$Z_x = \frac{(134.53 \text{ ft-kips})(12 \frac{in}{ft})}{0.9(50 \text{ ksi})} = 35.88 \text{ in}^3$$

$66.5 \text{ in}^3 \geq 35.88 \text{ in}^3$ ✓

Reactions

$$R_A = R_B = \tfrac{1}{2}(1.722 \text{ klf})(25') = 21.53^k$$

Our Z_x allows us to use this beam.

$\boxed{\text{W18×35}}$

8.5 Appendix E

STRUCTURAL MASONRY DESIGN SYSTEM FOR SHEAR WALLS

MASONRY DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY
The National Concrete Masonry Association Brick Industry Association Masonry 6.0
Western States Clay Product Association International Code Council (Release 6.0.1)

Prjct: Calvin Engineering addition	Name: Joel Smit
Topic: wall 2 out of plane	Date: 4-4-14
Page:	Chkd: Bethany Goodrich

Design of a Reinforced Masonry Wall with Out-of-Plane Loads
Using the 2012 IBC Strength Design Code

Material and Construction Data
- 8 in. CMU, Partial grout, running bond
- Wall Weight = 41.03 psf
- Type S Masonry cement / Air-entrained PCL Mortar, Coarse Grout
- CMU Density = 115.0 pcf
- f'_m = 2,000 psi (Specified)
- $E_m = 900f'_m$ = 1,800,000 psi

Wall Design Details
- Thickness = 7.625 in.
- Height = 296.0 in. (Simply Supported Wall, Effective height = H)
- x = 3.813 in.
- #6 Bars, Fy = 60,000
- Reinforcement Spacing = 48.00 in. On-Center
- Effective Width = 48.00 in.

Wall Design Section Properties
- A_o = 40.64 in^2 per foot width
- S_o = 87.09 in^3 per foot width
- I_o = 332.0 in^4 per foot width
- r_o = 2.858 in

Wall Average Section Properties
- A_{avg} = 49.62 in^2 per foot width
- I_{avg} = 351.7 in^4 per foot width
- r_{avg} = 2.661 in

Wall Support: Simply Supported Wall

Specified Load Components

Load	P (lb)	e (in)	W1(psf)	W2 (psf)	L (lb/ft)	h1 (in)	h2 (in)
Dead	220	0	0	0	0	0	24
Live	0	0	0	0	0	0	24
Soil	0	0	0	0	0	0	24
Fluid	0	0	0	0	0	0	24
Wind	0	0	20	20	0	0	296
Seismic	0	0	0	0	0	0	24
Roof	0	0	0	0	0	0	24
Rain	0	0	0	0	0	0	24
Snow	220	0	0	0	0	0	24

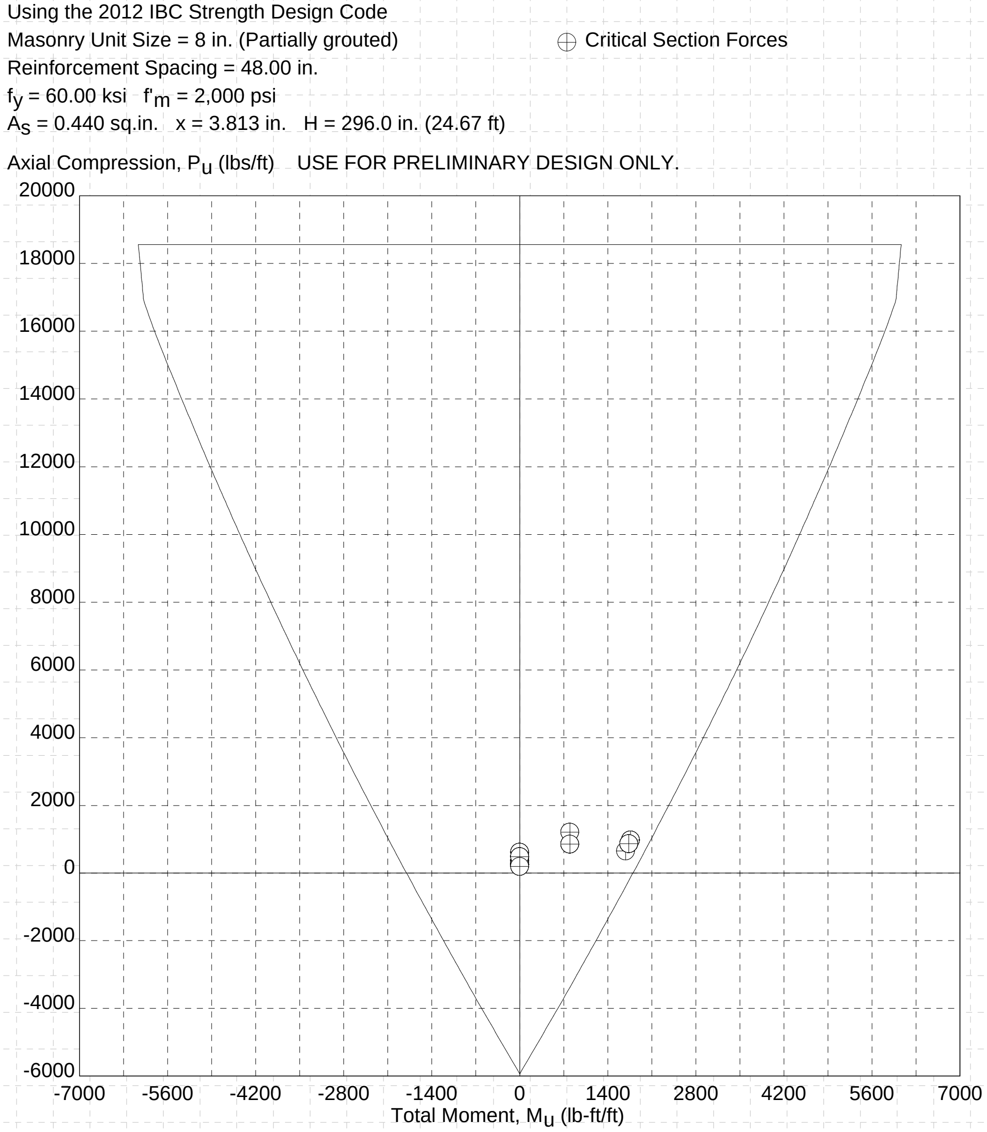
MASONRY — DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY

The National Concrete Masonry Association | Brick Industry Association | Masonry 6.0
Western States Clay Product Association | International Code Council | (Release 6.0.1)

Prjct: Calvin Engineering addition	Name: Joel Smit
Topic: wall 2 out of plane	Date: 4-4-14
Page:	Chkd: Bethany Goodrich

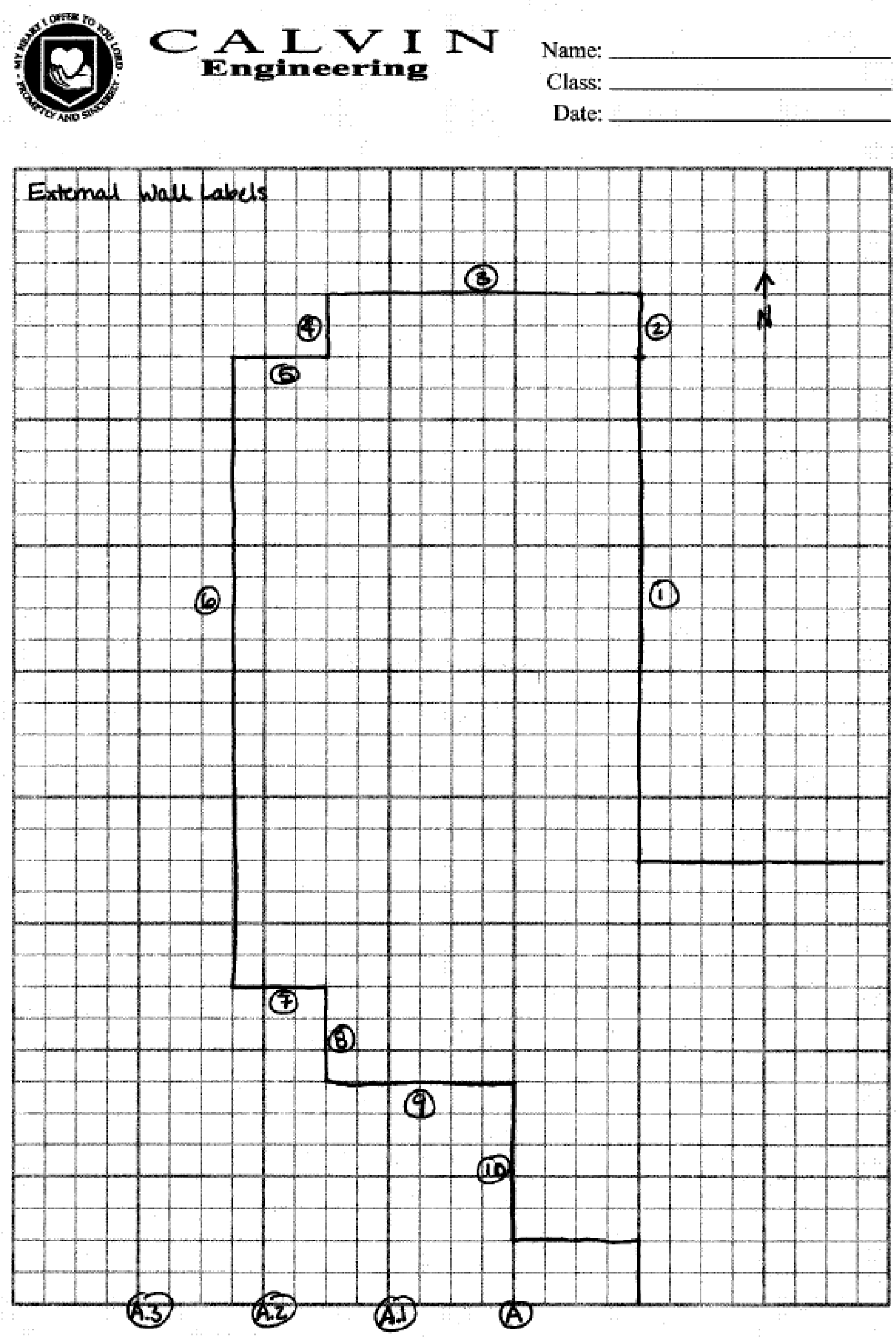
Reinforced Wall Interaction Diagram
Using the 2012 IBC Strength Design Code
Masonry Unit Size = 8 in. (Partially grouted)
Reinforcement Spacing = 48.00 in.
f_y = 60.00 ksi f'_m = 2,000 psi
A_s = 0.440 sq.in. x = 3.813 in. H = 296.0 in. (24.67 ft)

Axial Compression, P_u (lbs/ft) USE FOR PRELIMINARY DESIGN ONLY.



8.6 Appendix F

CONTINUOUS WALL FOUNDATION HAND CALCULATIONS

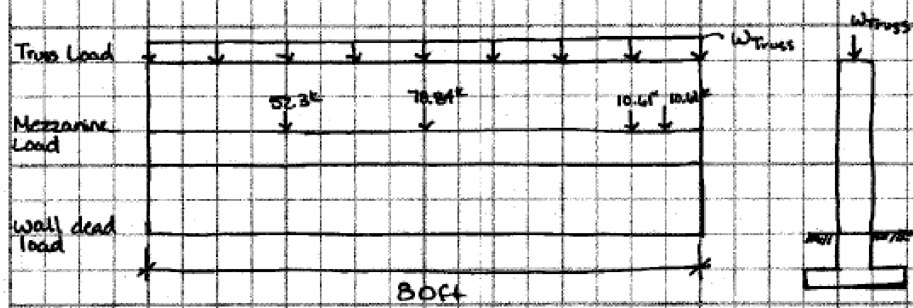


CALVIN Engineering

Name: ______
Class: ______
Date: ______

Wall 1 (Exterior)

80ft Wall



Total Wall Load on Soil

- Truss Load

$$w_{truss} = \frac{P_u(\#\text{ of trusses})}{L} = \frac{(36.5^k)(9\text{ trusses})}{80ft} = 4.106^{klf}$$

- Mezzanine Load

  * assume large point loads taken by columns

$$w_{mezz} = \frac{P_u(\#\text{ of beams})}{L} = \frac{10.61^k(2\text{ beams})}{15ft} = 1.42^{klf}$$

- Concrete ~~Masonry~~ Wall Load

  CMU = 45.0 psf

  Exterior Brick = 115pcf * Assume Brick is entire wall height

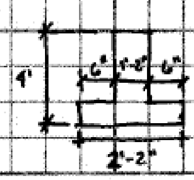
$$w_{CMU} = \frac{45.0\text{ psf} * 25ft}{1000\#/kip} = 1.125^{klf}$$

$$w_{Brick} = \frac{115pcf * 4\text{in brick} * 25ft}{1000\#/kip * 12in/ft} = 0.958^{klf}$$

$$w_{wall} = 1.4 * (w_{CMU} + w_{Brick}) = 1.4(1.125^{klf} + 0.958^{klf}) = 2.92^{klf}$$

- Concrete footing Load

  CONCRETE = 145pcf



$$w_{conc} = \frac{1.4 * 1.167ft * 4ft * 145pcf}{1000\#/kip} = 0.948^{klf}$$

CALVIN Engineering

Name:
Class:
Date:

<u>Wall 1 (Exterior)</u>

Total Wall Load on Soil

$W_1 = W_{TRUSS} + W_{wall_1} + W_{conc} = 4.106^{klf} + 2.92^{klf} + 0.948^{klf}$
$W_1 = 7.97\ klf$

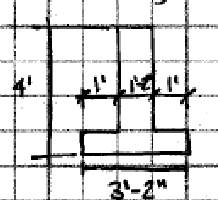
Soil Bearing Capacity
$S_B = 3500\ psf$
$W_{soil} = S_B * \text{Footing Length} = 3500\,psf * 2.167\,ft = 7.58^{klf}$

Footing Length ok if $W_{TOTAL\ wall} < W_{soil}$.

$7.91^{klf} > 7.58^{klf}$ ∴ Footing No Good!

Increase Footing by 6" on each side.

New Footing



New $S_B = 3500\ psf$
$W_{soil} = \dfrac{3500\,psf * 3.167\,ft}{1000\,\#/kip} = 11.08^{klf}$

$W_1 = 7.97^{klf} < W_{soil} = 11.08^{klf}$ ∴ footing ok. ✓

IF add mezzanine Loads:

$W_{78.84}$ → assume 5 points over 80 ft span
$W_{78.84} = \dfrac{78.84^k(5)}{80\,ft} = 4.93^{klf}$

$W_{52.3}$ → assume 5 points over 80 ft span
$W_{52.3} = \dfrac{52.3(5)}{80\,ft} = 3.27^{klf}$

Total Wall Load:

$W_T = 7.97^{klf} + 4.93^{klf} = 12.9^{klf}$

$W_T = 7.97^{klf} + 3.27^{klf} = 11.2^{klf}$

Both > W_{soil} → use column footing.

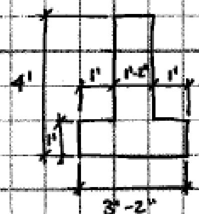
CALVIN Engineering

Name: ____________

Class: ____________

Date: ____________

Exterior Footing Reinforcement Design $(L = 3'\text{-}2")$



$b_w = 1'\text{-}2" = 14"$ $\quad q_a = 3600\ psf$

$t_f = 12"$ $\quad q_{nu} = 4900\ psf$

$b_f = 3'\text{-}2" = 38"$

• Check shear

$d_{shear} = t_f - 3"\ cover - \frac{1}{2}(\text{assume } 1"\ bar\ \phi) = 12" - 3" - 0.5" = 8.5"$

$d_{trib\ shear} = \frac{b_f}{2} - \frac{b_w}{2} - d_{shear} = \frac{38"}{2} - \frac{14"}{2} - 8.5\ in = 3.5"$

$V_u = d_{trib} * t_f * q_u = \frac{(3.5")(12")(4.9\ ksf)}{(12\ in/ft)^2} = 1.43\ kip$

$\phi V_c = \phi 2\sqrt{f'_c}\ b_w d_{shear} = \frac{0.75(2)\sqrt{3000\ psi}(14")(8.5")}{1000\#/kip} = 9.78\ kip$

Shear reinforcing not req'd if $V_u < \phi V_c$ $\therefore$ No shear reinforcing

• Steel Design for Flexure

*critical Section for moment @ face of concrete wall

$d_{trib\ moment} = \frac{b_f}{2} - \frac{b_w}{2} = \frac{38"}{2} - \frac{14"}{2} = 12"$

$M_u = \frac{q_u d_{trib}^2}{2} = \frac{4.9\ ksf\,(12\ in/12\ in/ft)^2}{2} = 2.45\ k\text{-}ft$

$M_u \le \phi M_n = \phi A_s f_y jd \quad \therefore \quad A_s = \frac{M_u}{\phi f_y jd} = \frac{2.45\ k\text{-}ft\,(12\ in/ft)}{0.90(60\ ksi)(0.95)(8.5\ in)} = 0.067\ in^2$

→ ACI 10.5.4 & 7.12.2.1 $\quad A_{s,min} = 0.0018\,bh = 0.0018(12\ in)(12\ in) = 0.26\ in^2$

(area of temp & shrinkage)

$\boxed{\text{Choose } \#5 \text{ @ } 12"\ \text{O.C.} \quad A_s = 0.31\ in^2/ft}$

$$\ell_d = \frac{\ell_d}{d_b} \times \frac{\psi_e}{\lambda} \times d_b, \text{ but not less than 12 in.}^a$$

Bar No.	$f'_c = 3000$ psi Bottom Bar	$f'_c = 3000$ psi Top Bar	$f'_c = 3750$ psi Bottom Bar	$f'_c = 3750$ psi Top Bar	$f'_c = 4000$ psi Bottom Bar	$f'_c = 4000$ psi Top Bar	$f'_c = 5000$ psi Bottom Bar	$f'_c = 5000$ psi Top Bar	$f'_c = 6000$ psi Bottom Bar	$f'_c = 6000$ psi Top Bar
Case 1: Clear spacing of bars being developed or spliced not less than d_b, clear cover not less than d_b, and stirrups or ties not less than the ACI Code minimum, throughout ℓ_d										
or										
Case 2: Clear spacing of bars being developed or spliced not less than $2d_b$ and clear cover not less than d_b.										
f_y = 60,000 psi, uncoated bars, normal-weight concrete										
3 to 6	43.8	57.0	39.2	50.9	37.9	49.3	33.9	44.1	31.0	40.3
7 to 18	54.8	71.2	49.0	63.7	47.4	61.7	42.4	55.2	38.7	50.3
f_y = 40,000 psi, uncoated bars, normal-weight concrete										
3 to 6	29.2	38.0	26.1	34.0	25.3	32.9	22.6	29.4	20.7	26.9
Other Cases										
f_y = 60,000 psi, uncoated bars, normal-weight concrete										
3 to 6	65.7	85.4	58.8	76.4	56.9	74.0	50.9	66.2	46.5	60.5
7 to 18	82.2	106.8	73.5	95.6	71.1	92.6	63.6	82.8	58.1	75.5
f_y = 40,000 psi, uncoated bars, normal-weight concrete										
3 to 6	43.8	57.0	39.2	51.0	38.0	49.4	33.9	44.1	31.1	40.4

[a] ψ_e, coating factor; λ, lightweight-concrete factor.

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Name:
Class:
Date:

Exterior Footing Reinforcement Design ($L = 3'\text{-}2''$)

- Check j-assumption

$$a = \frac{A_{s,act} f_y}{\phi f'_c b_w} = \frac{(0.31 in^2)(60^{ksi})}{0.85(3^{ksi})(12 in)} = 0.61 in \quad \therefore \text{tension controlled}$$

$\phi = 0.90$

a = tension controlled if $< 2/3\, d_t$ $\quad d_{shear} = 8.5''$

$\phi M_n = \phi A_{s,act} f_y (d - a/2) = 0.90(0.31 in^2)(60^{ksi})(8.5 in - 0.61 in/2) = 11.43$ k-ft

$M_u < \phi M_n \therefore$ ok ✓ $\boxed{\#5 \text{ bar @ } 12 \text{ in o.c. ok}}$

- Check development
  - Table A-6 → $\ell_d = 43.8\, d_b = 43.8(5 in/8) = 27.375$ in
  - distance from point of max bar stress to end of bar

    $\ell_{d\,max}$ = d_{rebeam} − 3" cover = 12 in − 3 in = 9 in

    $\ell_{d\,max} < \ell_d \therefore$ hook bar

    180° hook → $\ell_{dh\,\#5} = 9.59$ in > 9 in ∴ No good

    ↳ Table A-6

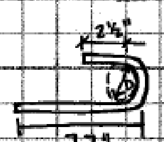
USE #4 @ 9 in o.c. $A_s = 0.27 in^2$

$$a = \frac{(0.27 in^2)(60 ksi)}{0.85(3^{ksi})(12 in)} = 0.53 in \quad \phi = 0.90$$

$\phi M_n = 0.90(0.27 in^2)(60 ksi)(8.5 in - \frac{0.53}{2} in) / 12 in/ft = 10.0^{k\text{-}ft}$

$M_u < \phi M_n \therefore \boxed{\#4 \text{ @ } 9'' \text{ oc. ok}}$

- Check development USE Table A-8
  - Table $\ell_d = 43.8\, d_b = 43.8(0.5 in) = 21.9$ in (A-6)
  - $\ell_{d\,max} = 9$ in (A-8)
  - $\ell_{d\,max} < \ell_d \therefore$ hook bar
  - 180° hook → $\ell_{dh\,\#4} = 11 * 0.7 = 7.7$ in



$D = 6 d_b = 6(0.5) = 3''$

CALVIN
Engineering

Name: ______________
Class: ______________
Date: ______________

Exterior Footing Reinforcement Design (L = 3'-2")

• longitudinal steel

$A_{s,min} = 0.0018bh = 0.0018(38in)(12in) = 0.82 \text{ in}^2$

$\boxed{\text{USE \#5 bar } A_s = 0.93 \text{ in}^2}$ 3 #5 bars

• connection b/t wall & footing

ACI 15.8.2.2 → reinforcing equivalent to vertical walls must extend into footing ∴ USE #6 bar ... reference shear wall designs.

Footing Design * add 2 #5 bars at top & bottom of footing wall for soil discontinuities.

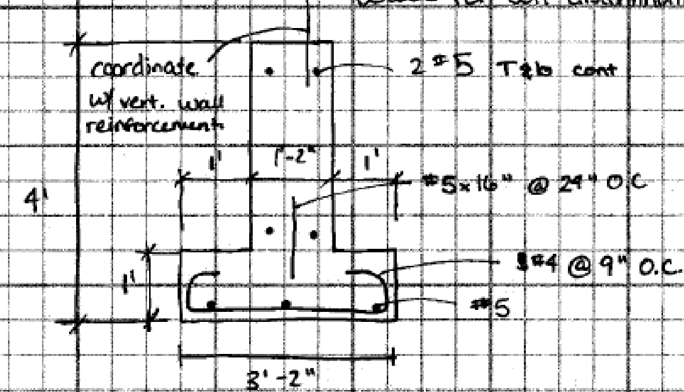


TABLE A-7M Basic Compression Development Length, ℓ_{dc} (mm)[a]

$\ell_{dc} = \ell_{dc} \times$ (Factors in ACI Code Section 12.3.3)

Bar No.	f'_c (MPa) 20	25	30	35	40
	$f_y = 420$ (MPa)				
10	235	210	192	177	168
13	305	273	249	231	216
16	376	336	307	284	266
19	446	399	364	337	315
22	517	462	422	390	365
25	587	525	479	444	415
29	681	609	556	515	481
32	751	672	613	568	531
36	845	756	690	639	598
43	1010	903	824	763	714
57	1338	1197	1093	1012	946
	$f_y = 300$ (MPa)				
10	200	200	200	200	200
15	252	225	205	200	200
20	335	300	274	254	240

[a]Lengths may be reduced if excess reinforcement is anchored or if the splice is enclosed in a spiral. See ACI Code Section 12.3.3. Reduced length shall not be less than 200 mm.

TABLE A-8 Basic Development Lengths for Hooked Bars, ℓ_{dh} (in.)

$\ell_{dh} = \ell_{dh} \times$ (Factors in ACI Code Section 12.5.3)[a]

Normal-weight concrete, $f_y = 60{,}000$ psi

Standard 90° or 180° Hooks

Bar No.	f'_c (psi) 3000	4000	5000	6000
3	8.2	7.1	6.4	5.8
→4	→11	9.5	8.5	7.8
→5	→13.7	11.9	10.6	9.7
6	16.4	14.2	12.7	11.6
7	19.2	16.6	14.9	13.6
8	22	19	17	15.5
9	25	21	19	17.5
10	28	24	22	20
11	31	27	24	22
14	37	32	29	26
18	49	43	38	35

[a]ℓ_{dh} is defined in Fig. 8-12a. The development length of a hook, ℓ_{dh}, is the product of ℓ_{hb} from this table and factors relating to bar yield strength, cover, presence of stirrups, and type of concrete, given in ACI Code Section 12.5.3. The resulting length, ℓ_{dh}, shall not be less than the larger of eight bar diameters or 6 in.

8.7 Appendix G

COLUMN FOOTING DESIGN SAMPLE CALCULATIONS

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Name:

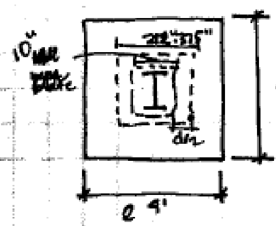
Class:

Date:

Footing Design F4.0

$q = 3.5$ ksf

$q_N = 4.9$ ksf



Check thickness for 2-way shear

#5 bar diameter

$d = 12\text{in} - 3\text{in} - 0.625\text{in} = 8.375\text{in}$

$V_u = 4.9\text{ksf}\left[\ldots - (2.375/12)^2\right] = 4.9\text{ksf}\left[(2')^2 - (18.375/12)^2\right] = 8.11\text{ kip}$

Length of critical perimeter $= \underline{b_o} = 4(10" + 8.375\text{in}) = 73.5$ in

→ ϕV_c smallest of:

$\beta_c = 1$ (ratio of long side to short side)

$V_c = \left(2 + \frac{4}{\beta_c}\right)\lambda\sqrt{f'c}\, b_o d = \left(2 + \frac{4}{1}\right)(1.0)\sqrt{3000\text{psi}}\ 73.5\text{in}\,(8.375\text{in})$

6 > 4 ∴ Not govern $V_c =$

$V_c = \frac{\alpha_s d}{b_o} + 2 = \frac{40(8.375\text{in})}{73.5\text{in}} = 4.56$ 4.56 > 4 ∴ USE 4 controls

$V_c = 4\lambda\sqrt{f'c}\, b_o d = 4(1.0)\sqrt{3000\text{psi}}\,(73.5\text{in})(8.375\text{in}) / 1000\#/\text{kip} = 134.9\text{ kip}$

$\phi V_c = 0.75(134.9\text{ k}) = 101.2\text{ kip}$

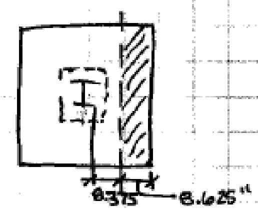
$\phi V_c > V_u$ ∴ footing design ok. (thick enough)

* footing okay if HSS 4x4 $\phi V_c = 79.1$ kip

CALVIN Engineering

Name: ______
Class: ______
Date: ______

Footing Design F4.0



Check one-way shear

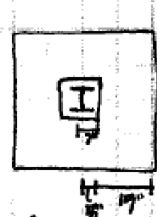
$d = 8.375\text{in}$

$V_u = 4.9\text{ksf}\left(4\text{ft} \cdot \frac{8.625"}{12}\right) = 14.1\text{kip}$

$\phi V_c = \phi 2\lambda\sqrt{f'c}\, b_w d = 0.75(2)(1.0)\sqrt{3000\text{psi}}\,(48\text{in})(8.375\text{in}) = 33.03^{\text{kip}}$

$\therefore$ ok in one-way shear

Flexural Reinforcement



$M_u = 4.9\text{ksf}\left[4' \cdot \frac{(19"/12)^2}{2}\right] = 24.6$ ft-kip

*assume $j = 0.95$ & $\phi = 0.90$

$A_s = \frac{24.6\,^{\text{ft-kip}} \times 12\text{in/ft}}{0.90 \cdot 60\text{ksi} \cdot 0.95 \cdot 8.375\text{in}} = 0.69\text{in}^2$

min $A_s = 0.0018bh = 0.0018(48")(12\text{in}) = 1.04\text{in}^2 \rightarrow$ governs
max spacing = 18in

Try $\boxed{4\ \#5 \text{ bars each way},\ A_s = 1.24\text{in}^2}$

$a = \frac{A_s f_y}{\beta f'c\, b} = \frac{1.24\text{in}^2(60\text{ksi})}{0.85(3\text{ksi})(48\text{in})} = 0.607"$ $\therefore$ tension controlled & $\phi = 0.90$

$\phi M_n = \phi A_s f_y(d - a/2) = 0.90(1.24\text{in}^2)(60\text{ksi})\left(\frac{8.375\text{in} - 0.607/2}{12\text{in/ft}}\right) = 45.0^{\text{k-ft}}$

$\phi M_n > M_u \therefore$ ok.

Check development

$\ell d = 43.8\, d_b = 43.8(0.625\text{in}) = 27.375\text{in}$

footing $\rightarrow$ 17" $- 3"_{\text{cover}} =$ 14" $\ngtr \ell d \therefore$ No Good Hook bars

CALVIN Engineering

Name: ______

Class: ______

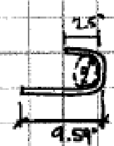
Date: ______

Footing Design F4.0

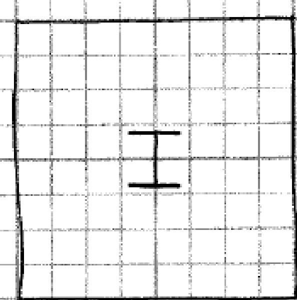
Hook bars

ℓ_{dh} #5 = 13.7(0.7) = 9.59"

ℓ_{df} = 14" ∴ hook ok



$D = 6d_b = 6(0.625in) = 3.75in$



4 #5 both directions

TABLE A-2 Areas of Multiples of Reinforcing Bars (in.^{2})

Number of bars	Bar number								
	#3	#4	#5	#6	#7	#8	#9	#10	#11
1	0.11	0.20	0.31	0.44	0.60	0.79	1.00	1.27	1.56
2	0.22	0.40	0.62	0.88	1.20	1.58	2.00	2.54	3.12
3	0.33	0.60	0.93	1.32	1.80	2.37	3.00	3.81	4.68
4	0.44	0.80	1.24	1.76	2.40	3.16	4.00	5.08	6.24
5	0.55	1.00	1.55	2.20	3.00	3.93	5.00	6.35	7.80
6	0.66	1.20	1.86	2.64	3.60	4.74	6.00	7.62	9.36
7	0.77	1.40	2.17	3.08	4.20	5.53	7.00	8.89	10.9
8	0.88	1.60	2.48	3.52	4.80	6.32	8.00	10.2	12.5
9	0.99	1.80	2.79	3.96	5.40	7.11	9.00	11.4	14.0
10	1.10	2.00	3.10	4.40	6.00	7.90	10.0	12.7	15.6
11	1.21	2.20	3.41	4.84	6.60	8.69	11.0	14.0	17.2
12	1.32	2.40	3.72	5.28	7.20	9.48	12.0	15.2	18.7
13	1.43	2.60	4.03	5.72	7.80	10.3	13.0	16.5	20.3
14	1.54	2.80	4.34	6.16	8.40	11.1	14.0	17.8	21.8
15	1.65	3.00	4.65	6.60	9.00	11.8	15.0	19.0	23.4
16	1.76	3.20	4.96	7.04	9.60	12.6	16.0	20.3	25.0
17	1.87	3.40	5.27	7.48	10.2	13.4	17.0	21.6	26.5
18	1.98	3.60	5.58	7.92	10.8	14.2	18.0	22.9	28.1
19	2.09	3.80	5.89	8.36	11.4	15.0	19.0	24.1	29.6
20	2.20	4.00	6.20	8.80	12.0	15.8	20.0	25.4	31.2

TABLE A-3 Minimum Required Beam Widths (in.)

Number of bars in one layer	Bar number							
	#3 and #4	#5	#6	#7	#8	#9	#10	#11
2	6.0	6.0	6.5	6.5	7.0	7.5	8.0	8.0
3	7.5	8.0	8.0	8.5	9.0	9.5	10.5	11.0
4	9.0	9.5	10.0	10.5	11.0	12.0	13.0	14.0
5	10.5	11.0	11.5	12.5	13.0	14.0	15.5	16.5
6	12.0	12.5	13.5	14.0	15.0	16.5	18.0	19.5
7	13.5	14.5	15.0	16.0	17.0	18.5	20.5	22.5
8	15.0	16.0	17.0	18.0	19.0	21.0	23.0	25.0
9	16.5	17.5	18.5	20.0	21.0	23.0	25.5	28.0
10	18.0	19.0	20.5	21.5	23.0	25.5	28.0	31.0

Note: Tabulated values based on No. 3 stirrups, minimum clear distance of 1 in., and a 1½-in. cover.

8.8 Appendix H

FLOW DETERMINATION FOR "INDIVIDUAL WATERSHEDS"

East Watershed (Upstream of MH #1)

East WS (Blue)	Area [sf]	Area [acre]	Fraction	Rational C	Average C	Length of Longest Reach [ft]	t_c	Rainfall Intensity (from MDOT IDF)	Q [cfs]
Total Upstream Area (upstream of circle drive)	119632	2.75			**0.67**	574	20 [min]	3.09	5.65
Area of Grass	34620								
RVD WS	5838		0.34	0.35					
Total	40458								
Area of Roof	50675								
RVD WS	3566		0.45	0.83					
Total	54241								
Area Concrete path	24933		0.21	0.82					

Van Noord Arena Watershed

North WS (VA and surr) (Red)	Area [sf]	Area [acre]	Fraction	Rational C	Average C	Length of Longest Reach [ft]	t_c	Rainfall Intensity (from MDOT IDF)	Q [cfs]
Total Upstream Area	34150	0.78			**0.77**	300	18 [min]	3.29	1.98
Area of Grass	4530		0.13	0.35					
Area of Roof	29620		0.87	0.83					

Watershed North of Huizenga Track and Tennis Center

North of HT&T (Green)	Area [sf]	Area [acre]	Fraction	Rational C	Average C	Length of Longest Reach [ft]	t_c	Rainfall Intensity (from MDOT IDF)	Q [cfs]
Total Upstream Area	493394	11			**0.54**	1374	26.5 [min]	2.65	16.27
Area of Grass	293232		0.59	0.35					
Area of Roof	62372		0.13	0.83					
Area of Concrete	137790		0.28	0.82					

Rain Garden Watershed

Rain Garden Watershed	Area [sf]	Area [acre]	Fraction	Rational C	Average C	Length of Longest Reach [ft]	t_c	Rainfall Intensity (from MDOT IDF)	Q [cfs]
Total Upstream Area	5700	0.13			**0.65**	200	16.7 [min]	3.38	0.29
Area of Grass	2171		0.38	0.35					
Area of Roof	2736		0.48	0.83					
Area of Concrete	793		0.14	0.82					

8.9 Appendix I

MASTER STORM SEWER DESIGN SPREADSHEET TABLES

Manhole Drainage Areas

MH #	Total Area [sf]	Roof Area [sf]	Grass Area [sf]	Pavement Area [sf]
1	area determined in indiv. watershed flow determination			
2	31490	5494	7020	18976
3	9174	0	1009	8165
4	55664	5710	9352	40602
5	85148	34955	7369	42824
6	54867	29196	6270	19401
7	11176	6168	2242	2766
8	33341	18359	8441	6541
9	area determined in indiv. watershed flow determination			
10	123242	8615	97282	17345
11	163498	800	156426	6272

Initial Manhole Data

MH	Ground Elev [ft]	DA [ft^2]	Road Area [ft^2]	DS Pipe Length [ft]
1	779.00			113
2	774.00	31490	18976	131
3	776.00	9174	8165	105.5
4	775.80	55664	40602	125
5	776.67	85148	42824	200
6	778.33	54867	19401	176
7	777.10	11176	2766	120
8	776.00	33341	6541	138
9	778.10	0	0	248
10	759.10	123242	17345	338
11	756.00	163498	6272	278
12 (discharge to pond)	752.88			

Drainage Area Information

MH and Drainage area information					C Asphalt		C Roof		C Lawn		
					0.81		0.83		0.35		
MH	Ground Elev [ft]	DA [ft^2]	DA [ac]	Road Area [ft^2]	CA Roads [ft^2]	Total Roof area [ft^2]	CA Roof [ft^2]	Lawn Area [ft^2]	CA Lawn [ft^2]	Average C	Basin Ave. CA [ac]
1	779.00										
2	774.00	31490	0.72	18976	15371	5494	4560	7020	2457	0.71	0.51
3	776.00	9174	0.21	8165	6614	0	0	1009	353	0.76	0.16
4	775.80	55664	1.28	40602	32888	5710	4739	9352	3273	0.73	0.94
5	776.67	85148	1.95	42824	34687	34955	29013	7369	2579	0.78	1.52
6	778.33	54867	1.26	19401	15715	29196	24233	6270	2195	0.77	0.97
7	777.10	11176	0.26	2766	2240	6168	5119	2242	785	0.73	0.19
8	776.00	33341	0.77	6541	5298	18359	15238	8441	2954	0.70	0.54
9	778.10	0	0.00	0	0	0	0	0	0	0.00	0.00
10	759.10	123242	2.83	17345	14049	8615	7150	97282	34049	0.45	1.27
11	756.00	163498	3.75	6272	5080	800	664	156426	54749	0.37	1.39
12 (discharge to pond)	752.88	0	0.00	0	0		0	0	0	0.00	0.00

Pipe Information

From Node					**Pipe**				**To Node**				
MH #	Ground Elevation [ft]	Invert Elevation [ft]	Crown Elevation [ft]	Cover [ft]	Diameter [in]	Length [ft]	Minimum Slope	Slope	MH #	Ground Elevation [ft]	Invert Elevation [ft]	Crown Elevation [ft]	Cover [ft]
1	779.00	769.19	770.69	8.31	18	113	0.0028	0.0030	2	774.00	768.85	770.35	3.65
2	774.00	768.85	770.35	3.65	18	131	0.0028	0.0030	3	776.00	768.46	769.96	6.04
3	776.00	768.46	769.96	6.04	18	105.5	0.0028	0.0030	4	775.80	768.14	769.64	6.16
4	775.80	767.96	769.96	5.84	24	125	0.0017	0.0020	5	776.67	767.71	769.71	6.96
5	776.67	767.71	769.71	6.96	24	200	0.0017	0.0020	6	778.33	767.31	769.31	9.02
6	778.33	766.81	769.31	9.02	30	176	0.0015	0.0020	7	777.10	766.46	768.96	8.14
7	777.10	766.46	768.96	8.14	30	120	0.0015	0.0020	8	776.00	766.22	768.72	7.28
8	776.00	766.22	768.72	7.28	30	138	0.0015	0.0020	9	778.10	765.94	768.44	9.66
9	778.10	765.94	767.94	10.16	24	248	0.0017	0.0451	10	759.10	754.76	756.76	2.34
10	759.10	754.75	756.25	2.85	18	338	0.0028	0.0055	11	756.00	752.90	754.40	1.60
11	756.00	752.90	755.40	0.60	30	278	0.0015	0.0023	12	755.50	752.27	754.77	0.73
12 (discharge to pond)	752.88	752.27	752.27	0.61									
					yellow cells determined by design spreadsheet								

Preliminary Hydraulic Analysis

MH #	Dist to next DS MH	New CA	Accum CA	Time [min]	Rainfall Intensity [in/hr]	Q [cfs]	Slope	K (Q/s^.5)	Minimum Size [in]	K	Capacity [cfs]	Q/Qo	V/Vo	V [ft/s]	Travel time [min]
1	113					5.65	0.0030	103	18	105.1	5.76	0.98	1.14	3.71	0.51
2	131	0.51	0.51	20.51	3.02	3.81	0.0030	70	18	105.1	5.76	0.66	1.07	3.47	0.63
3	105.5	0.16	0.67	21.14	2.98	2.01	0.0030	37	18	105.1	5.76	0.35	0.91	2.96	0.59
4	125	1.52	2.20	21.73	2.94	6.45	0.0020	144	24	226.2	10.12	0.64	1.06	3.41	0.61
5	200	0.94	3.13	22.34	2.89	9.06	0.0020	203	24	226.2	10.12	0.90	1.13	3.64	0.92
6	176	0.97	4.10	23.26	2.83	11.61	0.0020	260	30	410.1	18.34	0.63	1.06	3.95	0.74
7	120	0.19	4.29	24.00	2.79	11.97	0.0020	268	30	410.1	18.34	0.65	1.06	3.97	0.50
8	138	0.54	4.83	24.50	2.75	13.28	0.0020	297	30	410.1	18.34	0.72	1.09	4.07	0.57
9	248	0.00	4.83	25.07	2.72	29.40	0.0451	138	24	226.2	48.04	0.61	1.05	16.05	0.26
10	338	1.27	6.10	25.33	2.71	16.52	0.0055	223	24	226.2	16.73	0.99	1.14	6.07	0.93
11	278	1.39	7.49	26.25	2.66	19.91	0.0023	419	36	666.6	31.69	0.63	1.05	4.72	0.98
12 (discharge to pond)	0	0.00	7.49	27.24	2.6	19.47									
								changed blue cells to use 18 in pipe instead of 15 in							

8.10 Appendix J

RAIN GARDEN CALCULATIONS AND DETAILS

Rain Garden Calculations:

Calc. By: Mitchell Feria
Checked By: Kendra Altena

Required Storage Volume:

Watershed Area:

$A_{roof} := 8375ft^2$ Total Roof Area

$A_{pavement} := 1673ft^2$ Total Paved Area

$A_{grass} := 7383ft^2$ Total Grassy Area

$A_{tot} := A_{roof} + A_{pavement} + A_{grass} = 17431\,ft^2$ Total Area

Curve Numbers: Soil Type C

$CN_{roof} := 98$

$CN_{pavement} := 98$

$CN_{grass} := 74$

$$CN_{overall} := \frac{A_{roof}}{A_{tot}} \cdot CN_{roof} + \frac{A_{pavement}}{A_{tot}} \cdot CN_{pavement} + \frac{A_{grass}}{A_{tot}} \cdot CN_{grass}$$

$CN_{overall} = 87.835$

Therefore, let CN = 88.

$CN := 88$ Overall Curve Number

Runoff Volume:

$S := \frac{1000in}{CN} - 10in = 1.364 \cdot in$ Potential Maximum Retention

$Ia := 0.2 \cdot S = 0.273 \cdot in$ Initial Abstraction

$P := 2.37in$ Depth of Precipitation: 2-yr, 24hr storm

$Pe := \frac{(P - Ia)^2}{P - Ia + S} = 1.271 \cdot in$ Precipitation Excess

$\boxed{V_{runoff} := Pe \cdot A_{tot} = 1846.123 \cdot ft^3}$ Total Runoff Volume

Calc. By: Mitchell Feria
Checked By: Kendra Altena

Provided Storage Volume:

$A_{top} := 540ft^2$ Rain Garden Surface Area; Top

$A_{pond\_bott} := 298ft^2$ Surface Area at Bottom of Ponding (3:1 side slope)

$$A_{pond} := \frac{A_{top} + A_{pond\_bott}}{2} = 419\,ft^2$$

Ponding Volume:

$h_f = 18 \cdot in$ Ponding Depth

$\boxed{V_{pond} := A_{pond} \cdot h_f = 628.5 \cdot ft^3}$ Ponding Volume

Filter Bed Volume:

$d_f := 48in$ Filter Bed Depth (Soil Depth)

$\eta_f := 0.417$ Soil Porosity

$\boxed{V_{soil} := A_{top} \cdot d_f \cdot \eta_f = 900.72 \cdot ft^3}$ Filter Bed Volume

Storage Bed Volume:

$d_s := 18in$ Storage Bed Depth (Aggregate Depth)

$\eta_s := 0.476$ Porosity (1" Washed Stone)

$\boxed{V_{bed} := A_{top} \cdot d_s \cdot \eta_s = 385.56 \cdot ft^3}$

$V_{tot} := V_{pond} + V_{soil} + V_{bed}$

$\boxed{V_{tot} = 1914.78 \cdot ft^3}$ Total Storage Provided

Demand to Capacity Ratio:

$$\boxed{r := \frac{V_{runoff}}{V_{tot}} = 0.964}$$

Underdrain Calculations:

Calc. By: Kendra Altena
Checked By: Mitchell Feria

Headloss Through Filter Bed:

$v_{soil} := 2 \cdot \frac{in}{hr}$

Infiltration Rate Through Soil

$K_{soil} := 0.001 \cdot \frac{cm}{s} = 1.417 \cdot \frac{in}{hr}$

$L_{soil} := d_f = 48 \cdot in$

Assumption: Table 4.1 page 84 (Fund. of Geotech. Engineering) Assumed middle of course sand range (1.0-0.01)

$$\boxed{h_{Lsoil} := \frac{(v_{soil} \cdot L_{soil})}{K_{soil}} = 5.644\,ft}$$

Headloss Through Storage Bed:

$v_{stone} := 2 \cdot \frac{in}{hr}$

Assumption of infiltration/flow through stone

$K_{stone} := 75 \cdot \frac{cm}{s}$

Assumption: Table 4.1 page 84 (Fund. of Geotech. Engineering) Assumed middle of clean gravel range (100-1)

$L_{stone} := d_s = 18 \cdot in$

$$\boxed{h_{Lstone} := \frac{(v_{stone} \cdot L_{stone})}{K_{stone}} = 3.387 \times 10^{-4} \cdot in}$$

Headloss Through Pipe:

$v_{avg} := \frac{(v_{soil} + v_{stone})}{2} = 2 \cdot \frac{in}{hr}$

$Q := v_{avg} \cdot A_{top} = 0.025 \cdot \frac{ft^3}{s}$

$f := 0.082$

Friction factor for 200-mm (8-inch) corrugated plastic pipe *http://cedb.asce.org/cgi/WWWdisplay.cgi?8636*

$D_{pipe} := 8 \cdot in = 0.667\,ft$

$L_{pipe} := 25 \cdot ft$

Calc. By: Kendra Altena
Checked By: Mitchell Feria

$$k := \frac{\left(8 \cdot f \cdot L_{pipe}\right)}{\left(\pi^2 \cdot g \cdot D_{pipe}^{\ 5}\right)} = 0.392 \frac{s^2}{ft^5}$$

Darcy-Weisbach equation for hydraulic resistance

$$p := 2$$

$$\boxed{h_{Lpipe} := k \cdot Q^2 = 2.451 \times 10^{-4} ft}$$

Total Head Loss:

$$\boxed{h_{tot} := h_{Lsoil} + h_{Lstone} + h_{Lpipe} = 5.645 ft}$$

Flow to Existing System (MH 2):

Proposed Rain Garden can handle 2-yr storm.

Existing Stormwater System Designed for 10-yr storm.

$$P_{2yr} := 2.37 \cdot in$$

"Computing Flood Discharge for Small Ungaged Watersheds" -Table 3.1

$$P_{10yr} := 3.52 \cdot in$$

$$Frac_p := \frac{P_{2yr}}{P_{10yr}} = 0.673$$

$$Frac_{ToEx} := 1 - Frac_p$$

$$\boxed{Frac_{ToEx} = 0.327}$$

Fraction of watershed runoff entering storm sewer

Flow From Rain Garden Watershed to Storm Sewer:

Calc. By: Kendra Altena
Checked By: Mitchell Feria

$Area_{pavement} := A_{pavement} \cdot Frac_{ToEx} = 547\,ft^2$

$Area_{grass} := A_{grass} \cdot Frac_{ToEx} = 2412\,ft^2$

$Area_{roof} := A_{roof} \cdot Frac_{ToEx} = 2736\,ft^2$

$Area_{TotRG} := Area_{pavement} + Area_{grass} + Area_{roof}$

$\boxed{Area_{TotRG} = 0.131 \cdot acre}$ Watershed Area to Storm Sewer

Rational Method "C" Value:

$C_{roof} := 0.83$

$C_{grass} := 0.35$

$C_{pavement} := 0.82$

$$C_{avg} := C_{roof} \cdot \frac{Area_{roof}}{Area_{TotRG}} + C_{grass} \cdot \frac{Area_{grass}}{Area_{TotRG}} + C_{pavement} \cdot \frac{Area_{pavement}}{Area_{TotRG}} = 0.626$$

Therefore, let C = 0.63

$C_{overall} := 0.63$

Additional Flow to Storm Sewer:

$L_{reach} := 180 \cdot ft$ Pipe Length

$t_c := 15 \cdot min + \left(\frac{L_{reach}}{2 \cdot \frac{ft}{s}} \right) = 16.5 \cdot min$ Time of Concentration

$I_{rain} := 3.39 \cdot \frac{in}{hr}$ Rainfall Intensity

$Q_{RG} := C_{overall} \cdot 3.39 \cdot 0.131 = 0.28$

Therefore, excess flow from rain garden to storm sewer = 0.28 cfs

Calc. By: Mitchell Feria
Checked By: Kendra Altena

Cost Analysis:

$V_{tot} = 1.915 \times 10^{3} \cdot ft^{3}$ Total Storage Volume

$unit_{price} := 7 \frac{\$}{ft^{3}}$ Cost of Construction: "*Low Impact Development Manual for Michigan*"

$Cost_{total} := V_{tot} \cdot unit_{price}$

$\boxed{Cost_{total} = 13403.46 \$}$

FILTER SOIL SPECIFICATIONS:

50% 2NS (#4) WASHED SAND
20% FINELY GROUND HARDWOOD BARK
10% LEAF COMPOST
10% MICHIGAN PEAT MOSS
10% SANDY LOAM TOPSOIL

SUGGESTED PLANT SELECTIONS:

PLANTS:
BAPTISIA AUSTRALIS
NEW ENGLAND ASTER
LOBELIA
AGASTACHE FOENICULUM
RUDBECKIA
CHELONE
IRIS VERSICOLOR
MONARDA
JOE PYE WEED

GRASSES:
SCHIZACHYRIUM SCOPARIUM
PANICUM VIRGATUM
CAREX PENSYLVANICA