Lake View Community Center Design Report

Submitted to
Mr. Scott Peterson
City Administrator, City of Lake View, IA

May 8, 2020

Prepared by:

Kevin Carpenter, Beau Kramer, Caleb Shudak, James Wood

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Executive Summary

This is IWFD Engineering's final report for the design of the new Lake View Community Center. IWFD Engineering is composed of team of high-level senior Civil Engineering students from the University of Iowa. The team is composed of members with expertise in Hydrology, Structural design and analysis, Architecture, and Management. IWFD Engineering has designed a Community Center that suits all the needs of the Lake View community. The Community Center will host a variety of entertainments and celebrations that will allow residents to attend events locally and stimulate the local economy.

The location of the site is to be south of Highway 175 and just north of the Cobblestone Inn & Suites. The wood-framed structure is 13,250 square feet with a main room of 8,000 square feet and includes a serving room, kitchen area with bar, fitness space, storage, and bathrooms. The exterior architecture includes native stone and the Lake View Blue in the design. The exterior also includes a fountain for picture purposes.

The parking lot is sized based on the capacity and contains 210 oversized spaces and is recommended to be paved in Hot Mix Asphalt (HMA) rather than brick pavers, gravel, or concrete as it provides the best strength and longevity relative to its cost and the cost of the other possibilities.

With the site being in close proximity to Black Hawk Lake, it was concluded that the stormwater management plan to contain and treat the runoff from the site. The solution includes green infrastructure to manage the stormwater to preserve and maintain the beauty of the lake area. A 530-foot bioswale designed to handle runoff from the site and future developed commercial land north of the project site is used. South of the parking lot, a 20,000 cubic-foot infiltration basin is used to handle the remaining runoff.

Following the analysis and design, IWFD Engineering performed a cost estimate. The estimate for the final design, including an HMA parking lot, is $1.9 million. Cost estimates for other parking lot surfaces and a detailed cost estimate can be viewed in the *Cost Estimate Section* of the report.

Table of Contents

Qualifications & Experience

Organization Name

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Organization & Team Descriptions

James Wood served as the External Project Manager for this project. His responsibilities included the organization and facilitation of client meetings, site visits, being the point of contact for the client, presentations and graphics, and client deliverables deadlines. During the design phase of the project he worked on the site plan, floor plan, 3D Model, Cost estimate, and the Poster/Presentation generation.

Caleb Shudak served as the Internal Project Manager for this project. The responsibilities associated with this role are the organization and facilitation of team meetings, meetings with our faculty advisor, preparation and distribution of meeting agendas, and organization and facilitation of team assignments. He oversaw the design and development of the green infrastructure, site plan, grading plan, foundations, and the plan set generation.

Kevin Carpenter served as the Report Production coordinator for this project. This role focuses on the coordination of all reports and preparation of graphics used within the report. This position had final editing decisions and choses style and format of the reports. He also oversaw and lead the design loading determination, report generation, and the shear and load-bearing wall design.

Beau Kramer served as the Technology Service coordinator for this project. The responsibilities associated with this role are to oversee and organize the group in the technological aspect, such as file organization and creation. Beau's responsibilities also included much of the architectural aspects of the project, such as the interior and exterior aesthetics. Beau lead the rendering generation, the design loading determination, the truss design, and assisted in the development of the final project items.

Design Services

Project Scope

IWFD Engineering is providing the preliminary design for a large venue used by the community to host large events and parties. The venue will be 13,250 square feet in order to host events for 450 persons. Of that total square footage, 8,000 square feet will be dedicated to a large main room to host larger events. The remaining will be dedicated to a serving room, storage, a kitchen area, and a recreational space. Included with our design of the community center is a parking lot that has 210 parking spaces. We also included green infrastructure to help manage stormwater while also helping to mitigate flooding issues that currently exist. This green infrastructure includes bioswales on the north and west edge of the site that accommodates the North portion of rainfall on the site as well as the future commercial lots and a infiltration basin on the south edge of the site that handles the water from a majority of the parking lot. Aesthetically, native stone or a similar material and the Lake View Blue will be incorporated into the design. The site includes a fountain on the east side of the building for photographic purposes.

Project Deliverables:

- Design of indoor venue for large group gatherings
- Design of parking lot design with possible permeable paving

- Design of an improved and green water runoff system

Work Plan

We have developed a plan to keep the project moving forward and to track progress of the project. We split the project into five phases: Design Proposal, Design and Concept Development, Design Analysis, Design Draft, and Final Design. Phase one includes the development of multiple solutions, drafting and finalizing our Proposal. Phase two includes narrowing design to one possibility and narrowing down the site layout, and structural and hydraulic possibilities. Phase three in composed of performing engineering analysis of chosen design option. Phase four includes beginning final design details and drafts of plan sets, renderings, reports and presentations. Phase five is producing final design details for faculty and clients. A visual timeline for the phases and tasks can be viewed in Appendix G under Gantt Chart.

Constraints, Challenges & Impacts

Constraints

The Community Center design was constrained by multiple factors. Beginning with the site, the lot size constrained the layout of the structure, parking lot and green space. The location of the parking lot is constrained by the hotel gravel lot south of the proposed site. The need for green infrastructure design presented a unique constraint on the possible ways IWFD Engineering was able to collect and drain the stormwater. The design of the structure itself was constrained by the given size requirements, wind loading, the interior layout, and the multi-use function of the interior. The aesthetic design was constrained by what materials and colors can be used to finish the structure as well as the cost of the structure.

Challenges

The design of the Community Center also had several engineering and design challenges for IWFD Engineering. The site location was a few hundred feet north of Black Hawk Lake and the area already has a drainage issue with runoff flowing over the road and directly into the lake. It is important to not increase this runoff amount with the increase of impermeable surfaces from our site development. With the current site elevations draining towards the lake, the stormwater management presented the largest challenge. Incorporation a floor plan that could serve the correct purpose for the new Community Center also presented a challenge. Presenting the Lake View community with an aesthetically pleasing Community Center design at a low cost was another challenge.

Societal Impacts

The Community Center will impact the city of Lake View in many positive ways. It will create a large venue in place of the old Lakewood Ballroom for members of the community to have gatherings for many important events. The building will also be a place to hold smaller gatherings and meetings for the community. The green infrastructure put in place reduces the runoff overflow and filters potential pollutants from the site runoff that enters Black Hawk Lake. The aesthetics of the building, the monuments outside, and green infrastructure create a beautiful breath of fresh air to Black Hawk Lake and the surrounding area that the community can utilize and be proud of.

Alternative Solutions That Were Considered

In the development of the final Community Center design, IWFD Engineering priced and considered three possible solutions based on the parking lot, which was viewed as the largest non-essential portion to price. The three possible solutions are a gravel parking lot, a lot composed entirely of permeable brick pavers, and a lot composed of asphalt (HMA).

The decision to have a gravel parking lot would result in the lowest overall cost for the project of $100,000. This is a common solution in rural areas that want low runoff from rainfall events. Due to the size needed for this parking lot the material choice has a large impact on the price of the project. A major downside of gravel is the finished surface, rough and loose, which makes walking more difficult depending on the time of year as well as troubles with snow removal in the winter pushing material around. A parking lot that still provides decreased runoff while giving a rigid surface is permeable pavers.

A permeable paver parking lot would result in the highest overall cost for the project of $700,000. This is a common solution in urban areas that want low runoff from rainfall events. Like the gravel in permeability, pavers are a high-end finish to a parking lot that substantially decreases the volume of water runoff. A solid finish like that of pavers without the decreased runoff is HMA.

The final parking lot type would be an HMA lot which would result in a mid-range cost for the project of $300,000. This is a common solution almost everywhere that is a cost-effective way to pave large areas but does not have any runoff upside. HMA allows for one solid service for travel like the finish of permeable pavers which allows for lower maintenance due to snow removal.

After considering all three parking lot surfaces IWFD Engineering decided to choose an HMA parking lot for the final design. This gave a mediocre dollar value for the cost estimate as well as designing for the worst case for runoff. As stated earlier the negative to this design is the increased runoff. With that in mind the infiltration basin and the bioswales were designed as if the entire parking lot and building were completely impervious. Therefore, any changes to the HMA parking lot to add infiltration are not necessary in order to have the green infrastructure function as intended.

Other alternative options considered were different floor plans and building layouts. After careful consideration by the Lake View community they decided to move forward with their original floor plan design. It was clear to IWFD Engineering that the Hometown pride committee had been discussing and reiterating this floor plan for a while, so the design was only ever changed minimally.

Final Design Details

Site Plan

In designing the site plan, the Community Center was placed in the Northeast corner of the site location to avoid rerouting runoff as shown in Figure 1 as the gold area. Placing the structure in the Northeast corner of the site allows for room for the placement of infiltration basins and bioswales to decrease the runoff heading to Black Hawk Lake due to the site. This also will ensure the structure will not be affected by water damage that water could cause. This provided IWFD Engineering with a large area to design and create a parking lot that can accommodate 450 people the Center is designed for which gave a minimum of 180 spaces needed, however the lot was over designed and 210 oversized spaces are shown.

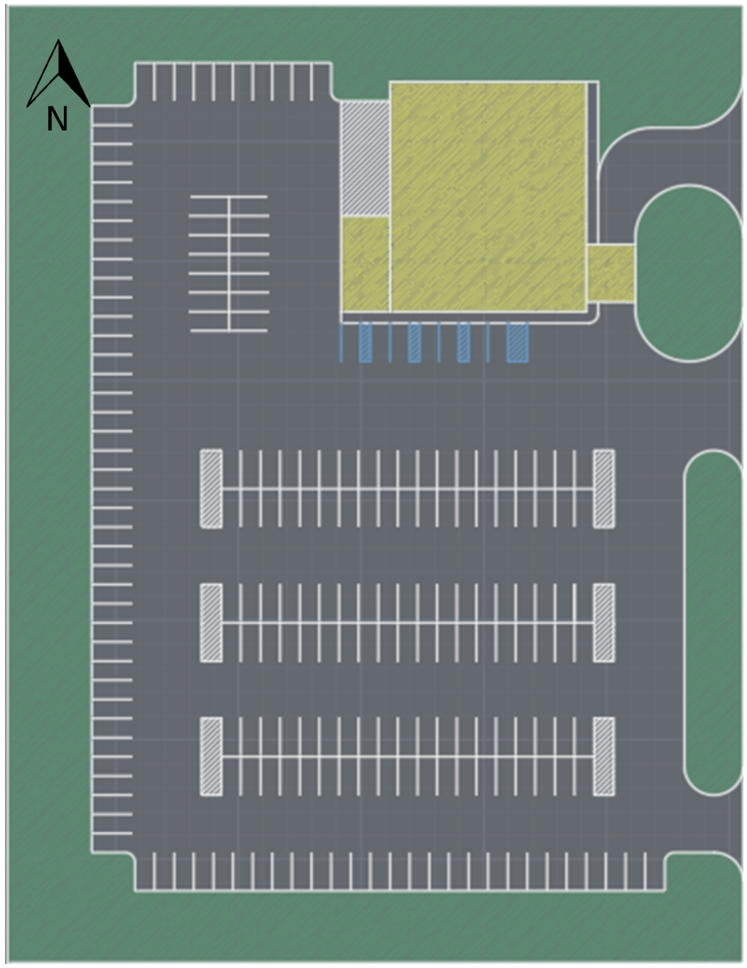

Figure 1. Post Construction Site Arrangement

Green Infrastructure

Following the design and layout of the site, a green and self-containing stormwater management solution was devised. The client had specifically requested the stormwater solution to contain green solutions in order to enhance the water quality before entering Blackhawk Lake. The self-containing solution was deemed important by IWFD Engineering in that it would reduce the runoff overflow onto Boulder Drive that came from the undeveloped site. This can be viewed in the pre-site analysis in Appendix D. The total flow was calculated using the Rational Method, outline in SUDAS Chapter 2B-4 and was 26.45 cfs before the site and 14.56 cfs post-development. This resulted in a 45% decrease in overflow runoff across Boulder Drive.

The green solution included a Bioswale and an Infiltration Basin. The Bioswale was design in accordance with the Iowa Department of Natural Resources Stormwater Management

Manual Chapter 5 – Section 5 (Bioswales) and Chapter 9 – Section 2 (Grass Swales). A Water Quality Volume (WQv) was the emphasis for both the Swale and Basin designs and is defined as the amount that includes in about 90% of storms for a given year. The calculations are outlined in Appendix E. The Swale is designed to accommodate a portion of the parking lot, the Center, and the post-development of the future commercial lots north of the Community Center site. The Infiltration Basin was design in accordance with the DNR Manual Chapter 5 – Section 3 (Infiltration Basins). The Bioswale pollutant removal capabilities can be viewed in Table 1 below.

Table 1. Pollutant removal capabilities of Bioswale

Pollutant removal efficiencies (%)								
Design	**Solids**	**Nutrients**		**Metals**			**Other**	
	TSS	TN	TP	Zn	Pb	Cu	FOG	COD**
200-ft swale	83	25*	29	63	67	46	75	25
100-ft swale	60	*	45	16	15	2	49	25

*Some swales (100-ft systems) show negligible or negative removal for TN
**Limited data
Sources: Barret et al., 1993; Schueler et al, 1991; Yu, 1993; and Yousef et al., 1985

With the Swale being over 500 feet in length, the pollutant removal capabilities are expected to exceed the expectations outlined above. The Infiltration basin pollutant removal capabilities can be seen in Table 2 below. It has also been designed to drain in under 24-hours in accordance with DNR requirements in order to be able to handle rainy seasons.

Table 2. Pollutant removal capabilities of Infiltration Basin

Pollutant	**Removal rate %**
Sediment	90%
Total P	60-70%
Total N	55-60%
Metals	85-90%
Bacteria	90%

Source: US EPA, 1983; Stahre and Urbonas, 1990; ASCE, 2001

Community Center Design

The Lake View Community Center was designed to be a large 100 ft by 120ft event space for the Lake View community. The community center has a large main room for the events, restrooms, a possible wellness room, a serving room, a kitchen and bar, a garage, and a space for a temporary stage. The structure was designed to be typical wood truss construction due to the lumberyard in town. The entire design was created with the idea to keep costs as low as possible for the community which is what led to minimal exterior windows and simple room shapes and overall building design. The view of the exterior of the building is shown below for a reference in Figure 2. Another view of the large main room is shown below in Figure 3 as well for a reference of the size of the room.

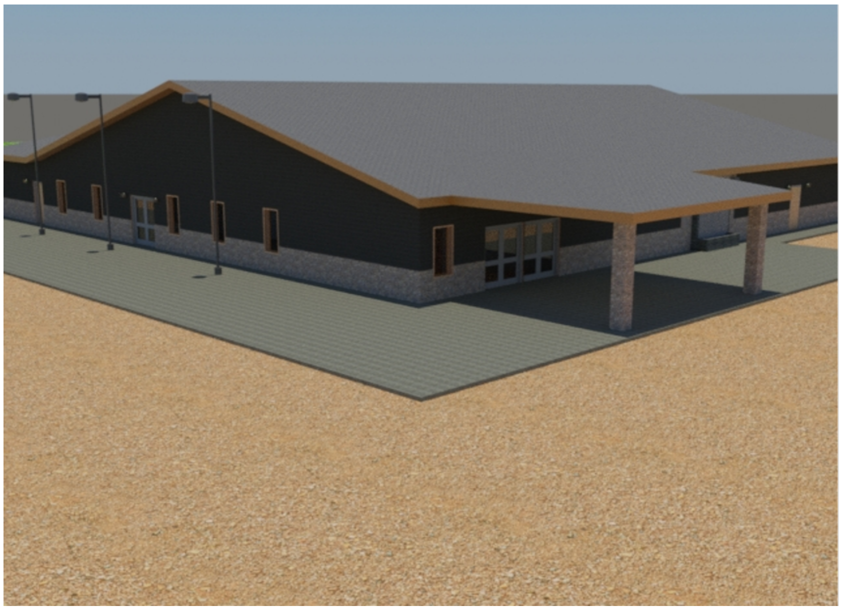

Figure 2. Exterior rendering of the southeast corner of the building

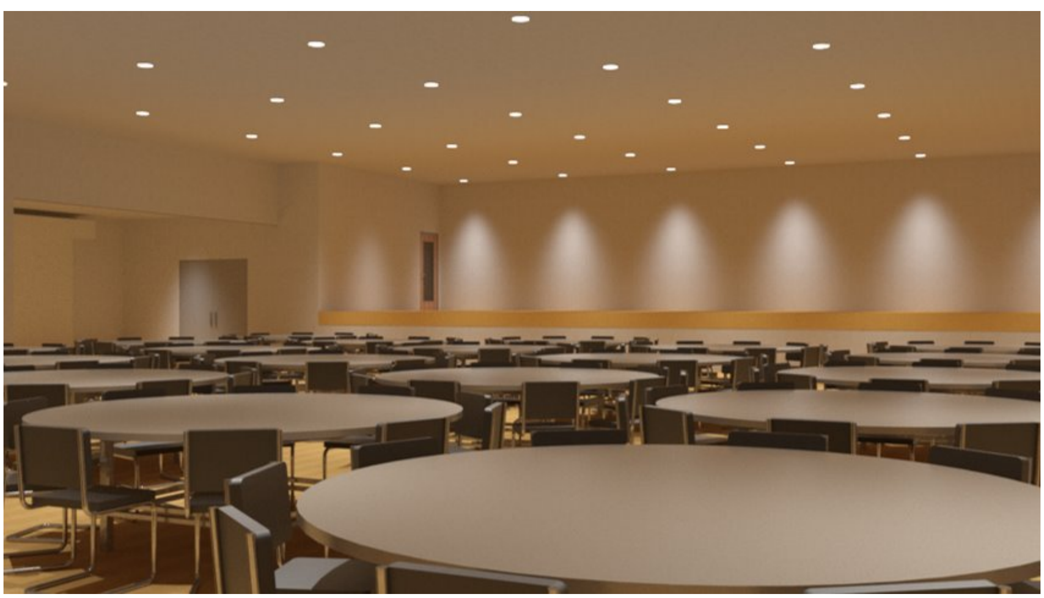

Figure 3. Interior rendering of the main room as if entered through the main entrance

Truss

The beginning of the Community Center structural analysis began by determining the wind, snow, and dead load the structure would be designed for. The loads were determined in accordance with ASCE 7-10. The live loads were from chapter 7, the dead loads were calculated using a dead load calculator on excel and the wind loads were from chapter 28. For the design loading calculations, we used the ASCE7-10 manual. Chapter 7 was used in determining the snow load on the truss (i.e. live load). The dead load was determined by the weight of materials used in constructing the trusses and the envelope method. Chapter 28 was used for determining the lateral and uplift loading caused by wind.

In designing the roof truss system, we decided to span the building with three trusses. Upon analysis we found that one large truss spanning the entire 100 feet was not feasible. Therefore, we chose to put one large Howe truss spanning 66 feet across the center of the main building and added mono trusses on each side of the Howe trusses to span 17 feet across the exterior rooms. The details and calculations of the trusses we decided to use can be viewed in Appendix B. The members of the Howe truss had to be enlarged from normal residential size. The top and bottom chord are to be constructed with 2x8 dimensional lumber while the interior members are to be made from 2x6 dimensional lumber. We were able to use normal residential sized mono trusses for the exterior rooms which are made of 2x6 dimensional lumber on the top and bottom chords and 2x4 dimensional lumber for the interior members.

Wall

For the design of the shear walls we used *Design of Wood Structures ASD.* Chapter 10 on shear walls was used to determine the lateral forces acting on the exterior walls of the building. The *Wood Frame Construction Manual* was used to determine the vertical forces the bearing and exterior walls can withstand. Shear walls are different than other load bearing or interior walls in that they have braced panels to withstand horizontal wind and seismic forces. The self-weight of the walls was calculated based on material weights and then included in the vertical force acting downward.

In the design of the exterior walls we used 2x8 lumber due to the large length each wall would be spanning, as well as the large vertical and lateral forces the wall would undergo. These exterior walls will have sheathing plywood paneling designed to withstand the horizontal wind loads. Using these resources, and the designated lumber size, every wall was calculated to be able to withstand the vertical loads acting on it. However, the south facing wall with the garage door in the original design could not handle the lateral load acting on it. This caused us to move that wall flush with the original South wall of the building and move the garage door to the West side of the garage. In the design of the interior load bearing walls 2x6 lumber was used because these walls only experience vertical forces they did not need to be as thick. Support beams were also designed to span the remainder of the building that the interior load bearing walls do not extend to. Based on the resources used the two bearing walls and beams were found to be adequate to support the vertical loads acting on them. The wall drawings can be viewed within the project plan set and the corresponding calculations can be found in Appendix B. Other interior walls were sized either using 2x6 or 2x4 lumber depending on the lengths they span.

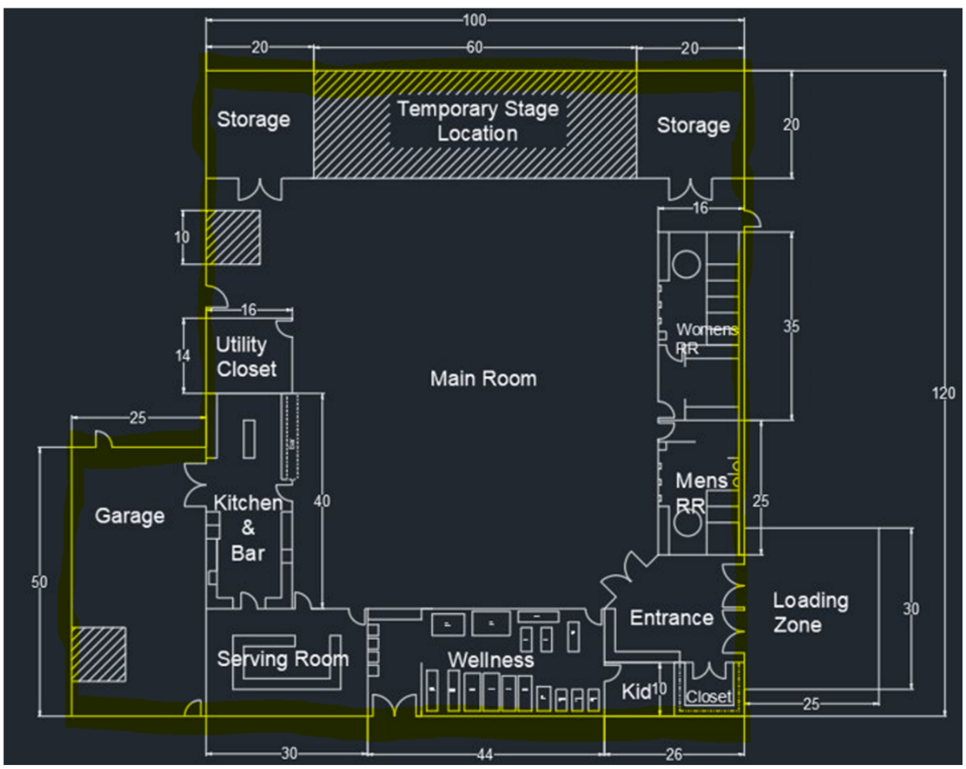

Figure 4. Floor Plan showing highlighted exterior shear walls

Foundation

Following the determination of loads and the visible structure design, the foundation was analyzed and designed. The foundations were designed in accordance with Foundation Analysis and Design by Joseph E. Bowles, P.E., S.E., the 2015 International Building Code (IBC), and ACI 318. A crucial piece to safe and effective design is accurate soil properties. The soil properties were determined from a Soil Survey of Sac County, Iowa done by the Unites States Department of Agriculture Soil Conservation Services in 1979 and from current Iowa DNR Geospatial Data for the site location. The results of these finding can be viewed in Appendix C. The foundation results were as follows. In accordance with the IBC Chapter 18, Figure 1809-2, the frost penetration for the region is about 60 inches. The foundations are to be placed at 72 inches to avoid the freeze-thaw effects of frost. Following the analysis, which can be viewed in Appendix C, the exterior foundation dimensions is a 6'-6" eccentrically loaded strip footing, except for the west and north portion of the west garage. The north strip footing is a 3'-6" eccentrically loaded footing and the west strip footing is a 4'-0" eccentrically loaded footing. An eccentric footing means the stem is offset of the center to combat the rotational effects of the wind. The interior strip footings are 2'-0" wide. For structural integrity, one #5 rebar is required every 12 inches with no shear reinforcements. Further detailing can be viewed in the project plan set.

Engineer's Cost Estimate

The final design cost estimate of the Lake View Community Center was roughly 1.8 million dollars. The breakdown of this cost is shown in Table 2 below and was found using RSMeans data. This value is consistent with the square footage area estimate that RSMeans also provides. For the estimates a nearby town was given to the online calculator as a reference point for costs in northwest Iowa. In understanding that construction in a small community works differently compared to a large city the costs were broken down in a way that will allow Lake View to better estimate their true building cost.

In addition to the final design cost estimate the different parking lot cost estimates are also shown in Table 4. This table shows where the weight of the different costs can come from and how the final number was reached. With that being said the final project cost could range anywhere from 1.7 million to 2.3 million depending on the parking lot and finishes.

Table 3 and 4. The Final Design Cost Estimate and the Alternative Costs

Final Design Cost Estimate Summary

Item	Material	Installation	Total
Concrete	$160,500	$164,700	$325,300
Wood Framing and Structure	$246,600	$215,300	$461,800
Mechanical	$186,600	$104,500	$247,500
Plumbing & Electrical	$98,000	$110,000	$207,900
Finishes	$55,500	$49,600	$105,000
Parking Lot	$220,000	$80,000	$300,000
Green Design	$42,200	$24,800	$67,000
Subtotal	$1,009,000	$749,000	$1,715,000
Engineering (2.5%)			$43,000
Contingency (10%)			$172,000
Total			$1,900,000

Parking Lot Options

Item	Material	Installation	Total
Gravel Lot	$80,000	$20,000	$100,000
HMA	$220,000	$80,000	$300,000
Permeable Pavers	$350,000	$350,000	$700,000

Overall, this project came in at a reasonable final dollar value, a more detailed cost estimate showing how we got to the final number is shown in Appendix F. In that table there are two sets of values, the first set is the raw data from the RSMeans online website, the second is the adjusted values for 2020 using a time value of money (TVM) analysis. These values were found using the average rate of inflation for construction costs over the past ten years which was about 3%.

APPENDICES

Appendix A – Design Loads

Appendix B – Shear Wall & Truss Calculations

Appendix C – Foundation Calculations

Appendix D – Runoff Calculations

Appendix E – Bioswale & Infiltration Basin Calculations

Appendix F – Detailed Cost Estimate

Appendix G – Gantt Chart

Appendix H – References

Appendix A – Design Loads

Wind Speed (mph)	Classification	Exposure Category	Ridge Height (ft)	Eave Height (ft)	Building Width (ft)	Building Length (ft)	Roof Type	Topo Factor, Kzt	Direct. Factor, Kd	Enclosed?	Hurricane Region
115	II	C	28.667	12	100	120	Gable	1	0.85	Y	N

Roof Angle q	Mean Roof Ht (ft)	Is h <= 60' ?	Is h <= Lesser of L or B?	(+)GCpi Coef	(-)GCpi Coef
18.44	20.33	Y	Y	0.18	-0.18

a	zg	Kh	I	qh
9.5	900	0.91	1	26.04

Wall and Roof End Zone Widths

a	2a
8.13	16.27

MWFRS Wind Load for Transverse

	GCpf	p (w/+GCpi)	p (w/-GCpi)
Zone 1	0.52	8.76	18.13
Zone 2	-0.69	-22.65	-13.28
Zone 3	-0.47	-16.89	-7.51
Zone 4	-0.42	-15.50	-6.13
Zone 5	-0.45	-16.41	-7.03
Zone 6	-0.45	-16.41	-7.03
Zone 1E	0.78	15.63	25.00
Zone 2E	-1.07	-32.55	-23.18
Zone 3E	-0.67	-22.22	-12.85
Zone 4E	-0.62	-20.78	-11.41

MWFRS Wind Load for Longitudinal

	GCpf	p (w/+GCpi)	p (w/-GCpi)
Zone 1	0.40	5.73	15.10
Zone 2	-0.69	-22.65	-13.28
Zone 3	-0.37	-14.32	-4.95
Zone 4	-0.29	-12.24	-2.86
Zone 5	-0.45	-16.41	-7.03
Zone 6	-0.45	-16.41	-7.03
Zone 1E	0.61	11.20	20.57
Zone 2E	-1.07	-32.55	-23.18
Zone 3E	-0.53	-18.49	-9.11
Zone 4E	-0.43	-15.88	-6.51

Zones 2/2E Dist. (ft)
30

MWFRS Wind Load for Transverse, Torsional Case

	p (w/-GCpi)	p (w/-GCpi)
Zone 1T	2.19	4.53
Zone 2T	-5.66	-3.32
Zone 3T	-4.22	-1.88
Zone 3T	-3.88	-1.53

MWFRS Wind Load for Transverse, Torsional Case

	p (w/-GCpi)	p (w/-GCpi)
Zone 1T	1.43	3.77
Zone 2T	-5.66	-3.32
Zone 3T	-3.58	-1.24
Zone 3T	-3.06	-0.71

For Transverse, Longitudinal, and Torsional Cases:

Zone 1 is windward wall for interior zone.
Zone 2 is windward roof for interior zone.
Zone 3 is leeward roof for interior zone.
Zone 4 is leeward wall for interior zone.
Zones 5 and 6 are sidewalls.
Zone 1T is windward wall for torsional case
Zone 3T is leeward roof for torsional case

Zone 1E is windward wall for end zone.
Zone 2E is windward roof for end zone.
Zone 3E is leeward roof for end zone.
Zone 4E is leeward wall for end zone.
Zone 2T is windward roof for torsional case.
Zone 4T is leeward wall for torsional case.

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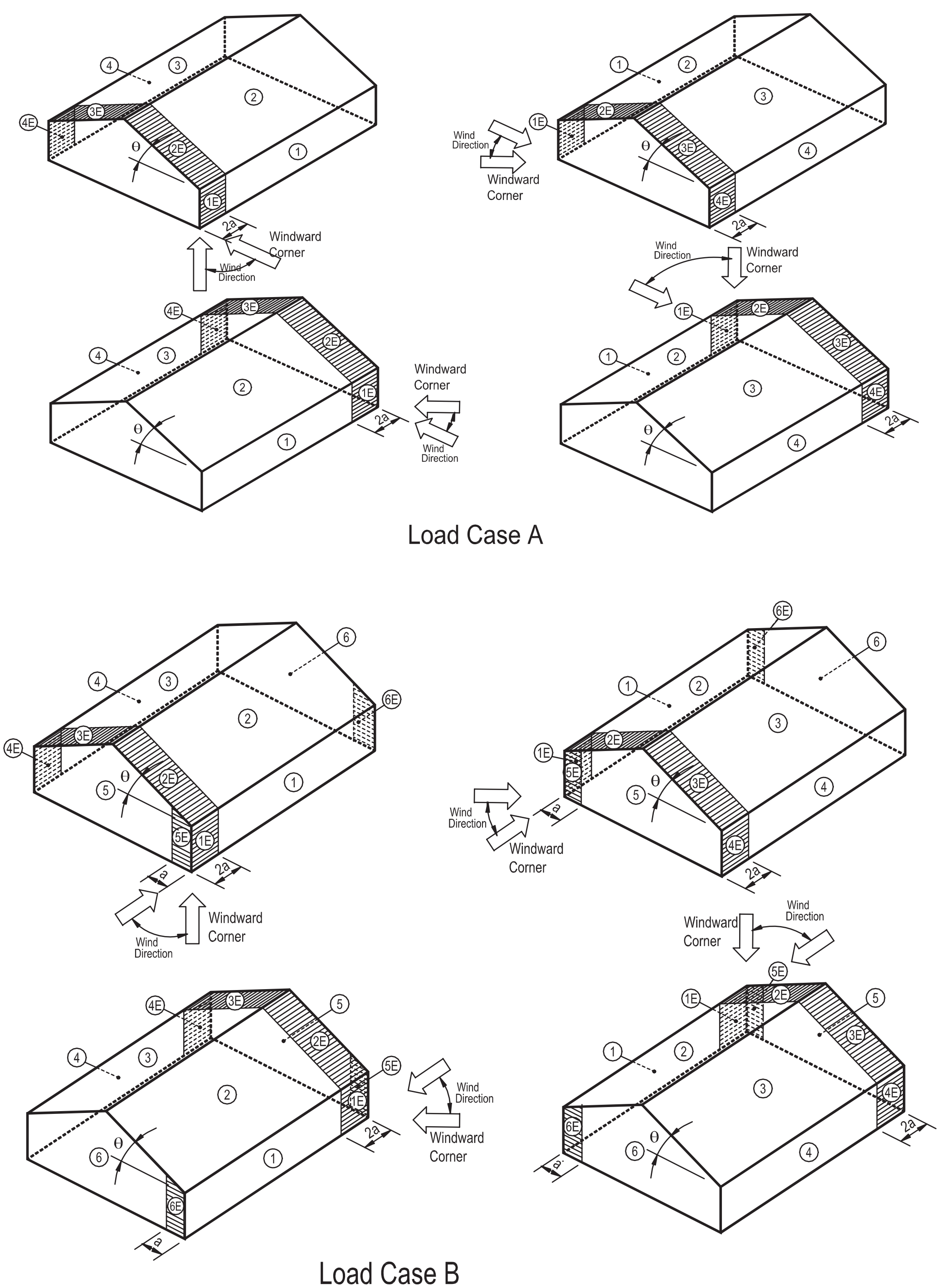
Main Wind Force Resisting System – Part 1		$h \leq 60$ ft.
Figure 28.4-1	**External Pressure Coefficients (GC_{pf})**	**Low-rise Walls & Roofs**
Enclosed, Partially Enclosed Buildings		



Basic Load Cases

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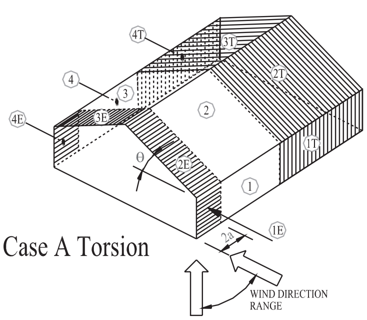
Main Wind Force Resisting System – Part 1		$h \leq 60$ ft.
Figure 28.4-1 (cont.)	External Pressure Coefficients (GC_{pf})	Low-rise Walls & Roofs
Enclosed, Partially Enclosed Buildings		

Roof Angle θ (degrees)	LOAD CASE A							
	Building Surface							
	1	2	3	4	1E	2E	3E	4E
0-5	0.40	-0.69	-0.37	-0.29	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	0.80	-1.07	-0.69	-0.64
30-45	0.56	0.21	-0.43	-0.37	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	0.69	0.69	-0.48	-0.48

Roof Angle θ (degrees)	LOAD CASE B											
	Building Surface											
	1	2	3	4	5	6	1E	2E	3E	4E	5E	6E
0-90	-0.45	-0.69	-0.37	-0.45	0.40	-0.29	-0.48	-1.07	-0.53	-0.48	0.61	-0.43

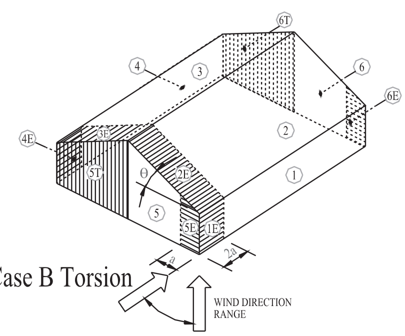
Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. For values of θ other than those shown, linear interpolation is permitted.
3. The building must be designed for all wind directions using the 8 loading patterns shown. The load patterns are applied to each building corner in turn as the Windward Corner.
4. Combinations of external and internal pressures (see Table 26.11-1) shall be evaluated as required to obtain the most severe loadings.
5. For the torsional load cases shown below, the pressures in zones designated with a "T" (1T, 2T, 3T, 4T, 5T, 6T) shall be 25% of the full design wind pressures (zones 1, 2, 3, 4, 5, 6).
   Exception: One story buildings with h less than or equal to 30 ft (9.1m), buildings two stories or less framed with light frame construction, and buildings two stories or less designed with flexible diaphragms need not be designed for the torsional load cases.
   Torsional loading shall apply to all eight basic load patterns using the figures below applied at each Windward Corner.
6. For purposes of designing a building's MWFRS, the total horizontal shear shall not be less than that determined by neglecting the wind forces on the roof.
   Exception: This provision does not apply to buildings using moment frames for the MWRFS.
7. For flat roofs, use θ = 0° and locate the zone 2/3 and zone 2E/3E boundary at the mid-width of the building.
8. For Load Case A, the roof pressure coefficient *(GC_{pf})*, when negative in Zone 2 and 2E, shall be applied in Zone 2/2E for a distance from the edge of roof equal to 0.5 times the horizontal dimension of the building measured perpendicular to the ridge line or 2.5 times the eave height at the windward wall, whichever is less; the remainder of Zone 2/2E extending to the ridge line shall use the pressure coefficient *(GC_{pf})* for Zone 3/3E.
9. Notation:
   a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
   h: Mean roof height, in feet (meters), except that eave height shall be used for θ ≤ 10°.
   θ: Angle of plane of roof from horizontal, in degrees.



Case A Torsion

Transverse Direction



Case B Torsion

Longitudinal Direction

Torsional Load Cases

Appendix B – Shear Wall & Truss Calculations

Scissor Truss Design

$Live := 100\ psf$

$Ct := 1 \quad Ce := 1 \quad Is := 1.1 \quad Pg := 35\ psf \quad Cs := 1$

$Pf := 0.7 \cdot Ce \cdot Ct \cdot Is \cdot Pg = 26.95\ psf$

$Ps := Cs \cdot Pf = 26.95\ psf$

$Lu := 33 \quad Pg := 35 \quad y := 18.55\ pcf$

$hd := \sqrt{Is} \cdot \left(\left(0.43 \cdot \sqrt[3]{Lu} \cdot \sqrt[4]{Pg + 10} \right) - 1.5 \right) = 2.173 \qquad hd := 2.173\ ft$

$s := 2$

$$UnbalancedSnowLoad := Ps + \frac{hd \cdot y}{\sqrt{s}} = 55.453\ psf$$

$BalancedSnowLoad := Ps = 26.95\ psf$

$Deadtotal := 8.05\ psf$

$DeadHorizontal := 9\ psf$

Insert item # for each material in the highlighted cells		Total Dead Load (psf)		Total Horizontal Dead Load (psf)
Total dead load will be displayed in cell C2		8.05		9.00
		Roof Rise	6	13.41640786
		Roof Run	12	
Item #	Roof Rafters	Dead Load Value	Item Number	Dead Load (psf)
1	2x4 @ 12" O.C.	1.65	5	1.9
2	2x4 @ 16" O.C.	1.3		
3	2x4 @ 24" O.C	0.82		
4	2x6 @ 12" O.C.	2.54		
5	2x6 @ 16" O.C.	1.9		
6	2x6 @ 24" O.C	1.27		
7	2x8 @ 12" O.C.	3.39		
8	2x8 @ 16" O.C.	2.54		
9	2x8 @ 24" O.C	1.7		
10	2x10 @ 12" O.C.	4.29		
11	2x10 @ 16" O.C.	3.21		
12	2x10 @ 24" O.C	2.15		
13	Wood Trusses (2x4 T & B Chords) @ 24" O.C.	2.5		
14	None	0		
Item #	Roof Covering Materials	Dead Load Value	Item Number	Dead Load (psf)

Non-Commercial Use Only

Wind Loads

Non-Commercial Use Only

Mono Truss Design

$Live := 100\ psf$

$Ct := 1 \quad Ce := 1 \quad Is := 1.1 \quad Pg := 35\ psf \quad Cs := 1$

$Pf := 0.7 \cdot Ce \cdot Ct \cdot Is \cdot Pg = 26.95\ psf$

$Ps := Cs \cdot Pf = 26.95\ psf$

$Lu := 17 \quad Pg := 35 \quad y := 18.55\ pcf$

$hd := \sqrt{Is} \cdot \left(\left(0.43 \cdot \sqrt[3]{Lu} \cdot \sqrt[4]{Pg + 10}\right) - 1.5\right) = 1.43 \quad hd := 1.43\ ft$

$s := 2$

$UnbalancedSnowLoad := Ps + \dfrac{hd \cdot y}{\sqrt{s}} = 45.707\ psf$

$BalancedSnowLoad := Ps = 26.95\ psf$

Wind Loads

Non-Commercial Use Only

Perforated Shear Wall Design

$q := 23.18\ psf$ $h := 12\ ft$ $L1 := 100\ ft$ $L2 := 120\ ft$

$Wu := q \cdot h = 278.16\ plf$ $Rb1 := Wu \cdot L1 \cdot \frac{1}{2} = 13.908\ kip$ $Ra1 := Rb1$

$Vs1 := \frac{Ra1}{L1} = 139.08\ plf$

$Rb2 := Wu \cdot L2 \cdot \frac{1}{2} = 16.69\ kip$ $Ra2 := Rb2$

$Vs2 := \frac{Ra2}{L2} = 139.08\ plf$

Nail spacing 3 inches with 8 diameter nail size

$v := 490\ plf$

South Wall

$h := 12\ ft$ $ho := 8\ ft$ $bo := 23\ ft$ $b := 125\ ft + 6\ in$

$sq := ((125\ ft + 6\ in) \cdot 12\ ft) + 400\ ft^2 = (1.906 \cdot 10^3)\ ft^2$

$F := sq \cdot q = 44.181\ kip$ $\frac{ho}{h} = 0.667$ $bfh := b - bo = 102.5\ ft$ $\frac{bfh}{b} = 0.817$

Based on Co table

$Co := 0.88$

$Vperforated1 := Co \cdot v \cdot bfh = 44.198\ kip$ $Vperforated1 > F = 1$

Vertical load

$Truss := 55.76\ psf$ $SWtruss := 1.1\ kip + \frac{15}{2}\ kip$ $Beam := 663.5\ plf$

$Roof := \frac{(Truss \cdot b \cdot (8\ in))}{2} = 2.333\ kip$ $Cvertical := Beam \cdot b = 83.269\ kip$

$Rt := Roof + SWtruss = 10.933\ kip$ $Cvertical > Rt = 1$

Total force into foundation

$Sw := 5.12763\ kip$ $TF := Rt + Sw = 16.06\ kip$

Non-Commercial Use Only

Perforated Shear Wall Design

$q := 23.18\ psf$ $h := 12\ ft$ $L1 := 100\ ft$ $L2 := 120\ ft$

$Wu := q \cdot h = 278.16\ plf$ $Rb1 := Wu \cdot L1 \cdot \frac{1}{2} = 13.908\ kip$ $Ra1 := Rb1$

$Vs1 := \frac{Ra1}{L1} = 139.08\ plf$

$Rb2 := Wu \cdot L2 \cdot \frac{1}{2} = 16.69\ kip$ $Ra2 := Rb2$

$Vs2 := \frac{Ra2}{L2} = 139.08\ plf$

Nail spacing 3 inches with 8 diameter nail size

$v := 490\ plf$

South Wall

$h := 12\ ft$ $ho := 8\ ft$ $bo := 23\ ft$ $b := 125\ ft + 6\ in$

$sq := ((125\ ft + 6\ in) \cdot 12\ ft) + 400\ ft^2 = (1.906 \cdot 10^3)\ ft^2$

$F := sq \cdot q = 44.181\ kip$ $\frac{ho}{h} = 0.667$ $bfh := b - bo = 102.5\ ft$ $\frac{bfh}{b} = 0.817$

Based on Co table

$Co := 0.88$

$Vperforated1 := Co \cdot v \cdot bfh = 44.198\ kip$ $Vperforated1 > F = 1$

Vertical load

$Truss := 55.76\ psf$ $SWtruss := 1.1\ kip + \frac{15}{2}\ kip$ $Beam := 663.5\ plf$

$Roof := \frac{(Truss \cdot b \cdot (8\ in))}{2} = 2.333\ kip$ $Cvertical := Beam \cdot b = 83.269\ kip$

$Rt := Roof + SWtruss = 10.933\ kip$ $Cvertical > Rt = 1$

Total force into foundation

$Sw := 5.12763\ kip$ $TF := Rt + Sw = 16.06\ kip$

Non-Commercial Use Only

East Wall

$h := 12\ ft$ $ho := 8\ ft$ $bo := 20\ ft + 18\ in$ $b := 120\ ft$

$sq := 120\ ft \cdot 12\ ft = (1.44 \cdot 10^3)\ ft^2$ $\frac{ho}{h} = 0.667$ $bfh := b - bo = 98.5\ ft$ $\frac{bfh}{b} = 0.821$

$F := sq \cdot q = 33.379\ kip$

Based on Co table

$Co := 0.84$

$Vperforated2 := Co \cdot v \cdot bfh = 40.543\ kip$ $Vperforated2 > F = 1$

Vertical load

$Truss := 55.76\ psf$ $SWtruss := \frac{15}{2}\ kip$ $Beam := 663.5\ plf$

$Roof := \frac{(Truss \cdot b \cdot (17\ ft + 6\ in))}{2} = 58.548\ kip$ $Cvertical := Beam \cdot b = 79.62\ kip$

$Rt := Roof + SWtruss = 66.048\ kip$ $Cvertical > Rt = 1$

Total force into foundation

$Sw := 4.901\ kip$

$TF := Rt + Sw = 70.949\ kip$

North Wall

$sq := (100\ ft \cdot 12\ ft) + 400\ ft^2 = (1.6 \cdot 10^3)\ ft^2$ $h := 12\ ft$ $ho := 0\ ft$ $bo := 0\ ft$ $b := 100\ ft$

$F := sq \cdot q = 37.088\ kip$ $\frac{ho}{h} = 0$ $bfh := b - bo = 100\ ft$ $\frac{bfh}{b} = 1$

Based on Co table

$Co := 1$

$Vperforated3 := Co \cdot v \cdot bfh = 49\ kip$ $Vperforated3 > F = 1$

Vertical load

$Truss := 55.76\ psf$ $SWtruss := 1.1\ kip$ $Beam := 663.5\ plf$

$Roof := \frac{(Truss \cdot b \cdot (8\ in))}{2} = 1.859\ kip$ $Cvertical := Beam \cdot b = 66.35\ kip$

$Rt := Roof + SWtruss = 2.959\ kip$ $Cvertical > Rt = 1$

Total force into foundation

$Sw := 4.58\ kip$ $TF := Rt + Sw = 7.539\ kip$

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Back West Wall

$sq := 69.5\ ft \cdot 12\ ft = 834\ ft^2$ $h := 12\ ft$ $ho := 10\ ft$ $bo := 13\ ft + 8\ in$ $b := 69\ ft + 6\ in$

$F := sq \cdot q = 19.332\ kip$ $\frac{ho}{h} = 0.833$ $bfh := b - bo = 55.833\ ft$ $\frac{bfh}{b} = 0.803$

Based on Co table

$Co := 0.75$

$Vperforated4 := Co \cdot v \cdot bfh = 20.519\ kip$ $Vperforated4 > F = 1$

Vertical load

$Truss := 55.76\ psf$ $SWtruss := \frac{15}{2}\ kip$ $Beam := 663.5\ plf$

$Roof := \frac{(Truss \cdot b \cdot (17\ ft + 6\ in))}{2} = 33.909\ kip$ $Cvertical := Beam \cdot b = 46.113\ kip$

$Rt := Roof + SWtruss = 41.409\ kip$ $Cvertical > Rt = 1$

Total force into foundation

$Sw := 2.458\ kip$ $TF := Rt + Sw = 43.867\ kip$

Inner West Wall

$sq := (50\ ft + 6\ in) \cdot 12\ ft = 606\ ft^2$ $h := 12\ ft$ $ho := 8\ ft$ $bo := 7\ ft + 4\ in$ $b := 50\ ft + 6\ in$

$F := sq \cdot q = 14.047\ kip$ $\frac{ho}{h} = 0.667$ $bfh := b - bo = 43.167\ ft$ $\frac{bfh}{b} = 0.855$

Based on Co table

$Co := 0.87$

$Vperforated5 := Co \cdot v \cdot bfh = 18.402\ kip$ $Vperforated5 > F = 1$

Vertical load

$Truss := 55.76\ psf$ $SWtruss := \frac{15}{2}\ kip$ $Beam := 663.5\ plf$

$Roof := \frac{(Truss \cdot b \cdot (17\ ft + 6\ in))}{2} = 24.639\ kip$ $Cvertical := Beam \cdot b = 33.507\ kip$

$Rt := Roof + SWtruss = 32.139\ kip$ $Cvertical > Rt = 1$

Total force into foundation

$Sw := 2.12285\ kip$ $TF := Rt + Sw = 34.262\ kip$

Non-Commercial Use Only

Side West Wall

$h := 12\ ft$ $ho := 10\ ft$ $bo := 10\ ft$ $b := 50\ ft + 6\ in$

$sq := (50\ ft + 6\ in) \cdot 12\ ft = 606\ ft^2$

$\frac{ho}{h} = 0.833$ $bfh := b - bo = 40.5\ ft$ $\frac{bfh}{b} = 0.802$

$F := sq \cdot q = 14.047\ kip$

Based on Co table

$Co := 0.77$

$Vperforated6 := Co \cdot v \cdot bfh = 15.281\ kip$ $Vperforated6 > F = 1$

Vertical load

$Truss := 55.76\ psf$ $SWtruss := \frac{15}{2}\ kip$ $Beam := 663.5\ plf$

$Roof := \frac{(Truss \cdot b \cdot (17\ ft + 6\ in))}{2} = 24.639\ kip$ $Cvertical := Beam \cdot b = 33.507\ kip$

$Rt := Roof + SWtruss = 32.139\ kip$ $Cvertical > Rt = 1$

Total force into foundation

$Sw := 1.7226\ kip$ $TF := Rt + Sw = 33.862\ kip$

Side North Wall

$sq := (25\ ft + 6\ in) \cdot 12\ ft = 306\ ft^2$ $h := 12\ ft$ $ho := 8\ ft$ $bo := 7\ ft + 4\ in$ $b := 25\ ft + 6\ in$

$F := sq \cdot q = 7.093\ kip$ $\frac{ho}{h} = 0.667$ $bfh := b - bo = 18.167\ ft$ $\frac{bfh}{b} = 0.712$

Based on Co table

$Co := 1$

$Vperforated8 := Co \cdot v \cdot bfh = 8.902\ kip$ $Vperforated8 > F = 1$

Vertical load

$Truss := 55.76\ psf$ $SWtruss := \frac{15}{2}\ kip$ $Beam := 663.5\ plf$

$Roof := \frac{(Truss \cdot b \cdot (12\ ft))}{2} = 8.531\ kip$ $Cvertical := Beam \cdot b = 16.919\ kip$

$Rt := Roof + SWtruss = 16.031\ kip$ $Cvertical > Rt = 1$

Total force into foundation

$Sw := 0.988625\ kip$ $TF := Rt + Sw = 17.02\ kip$

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Bearing Wall Design

Left bearing wall

$b := 54\ ft + 1\ in$ $\quad l := (44\ ft + 8\ in) + (19\ ft + 2\ in)$

Vertical load

$Truss := 55.76\ psf$ $\quad SWtruss := \frac{15}{2}\ kip + \frac{34.8}{2}\ kip$ $\quad Beam := 2075\ plf$

$Roof := \frac{(Truss \cdot 120\ ft \cdot 65\ ft)}{2} = 217.464\ kip$ $\quad CverticalW := Beam \cdot b = 112.223\ kip$

$Rt := Roof + SWtruss = 242.364\ kip$ $\quad CverticalB := Beam \cdot l = 132.454\ kip$

$CverticalT := CverticalB + CverticalW = 244.677\ kip$ $\quad CverticalT > Rt = 1$

Total force into foundation

$Sw := 2.54554\ kip$ $\quad TF := Rt + Sw = 244.91\ kip$

Right bearing wall

$b := 60\ ft + 6\ in$ $\quad l := (31\ ft + 8\ in) + (25\ ft + 8\ in)$

Vertical load

$Truss := 55.76\ psf$ $\quad SWtruss := \frac{15}{2}\ kip + \frac{34.8}{2}\ kip$ $\quad Beam := 2075\ plf$

$Roof := \frac{(Truss \cdot 120\ ft \cdot 65\ ft)}{2} = 217.464\ kip$ $\quad CverticalW := Beam \cdot b = 125.538\ kip$

$Rt := Roof + SWtruss = 242.364\ kip$ $\quad CverticalB := Beam \cdot l = 118.967\ kip$

$CverticalT := CverticalB + CverticalW = 244.504\ kip$ $\quad CverticalT > Rt = 1$

Total force into foundation

$Sw := 2.6796\ kip$ $\quad TF := Rt + Sw = 245.044\ kip$

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Wind Speed (mph)	Classification	Exposure Category	Ridge Height (ft)	Eave Height (ft)	Building Width (ft)	Building Length (ft)	Roof Type	Topo Factor, Kzt	Direct. Factor, Kd	Enclosed?	Hurricane Region
115	II	C	28.667	12	100	120	Gable	1	0.85	Y	N

Roof Angle q	Mean Roof Ht (ft)	Is h <= 60' ?	Is h <= Lesser of L or B?	(+)GCpi Coef	(-)GCpi Coef
18.44	20.33	Y	Y	0.18	-0.18

a	zg	Kh	I	qh
9.5	900	0.91	1	26.04

Wall and Roof End Zone Widths

a	2a
8.13	16.27

MWFRS Wind Load for Transverse

	GCpf	p (w/+GCpi)	p (w/-GCpi)
Zone 1	0.52	8.76	18.13
Zone 2	-0.69	-22.65	-13.28
Zone 3	-0.47	-16.89	-7.51
Zone 4	-0.42	-15.50	-6.13
Zone 5	-0.45	-16.41	-7.03
Zone 6	-0.45	-16.41	-7.03
Zone 1E	0.78	15.63	25.00
Zone 2E	-1.07	-32.55	-23.18
Zone 3E	-0.67	-22.22	-12.85
Zone 4E	-0.62	-20.78	-11.41

MWFRS Wind Load for Longitudinal

	GCpf	p (w/+GCpi)	p (w/-GCpi)
Zone 1	0.40	5.73	15.10
Zone 2	-0.69	-22.65	-13.28
Zone 3	-0.37	-14.32	-4.95
Zone 4	-0.29	-12.24	-2.86
Zone 5	-0.45	-16.41	-7.03
Zone 6	-0.45	-16.41	-7.03
Zone 1E	0.61	11.20	20.57
Zone 2E	-1.07	-32.55	-23.18
Zone 3E	-0.53	-18.49	-9.11
Zone 4E	-0.43	-15.88	-6.51

Zones 2/2E Dist. (ft)
30

MWFRS Wind Load for Transverse, Torsional Case

	p (w/-GCpi)	(w/-GCpi)
Zone 1T	2.19	4.53
Zone 2T	-5.66	-3.32
Zone 3T	-4.22	-1.88
Zone 3T	-3.88	-1.53

MWFRS Wind Load for Transverse, Torsional Case

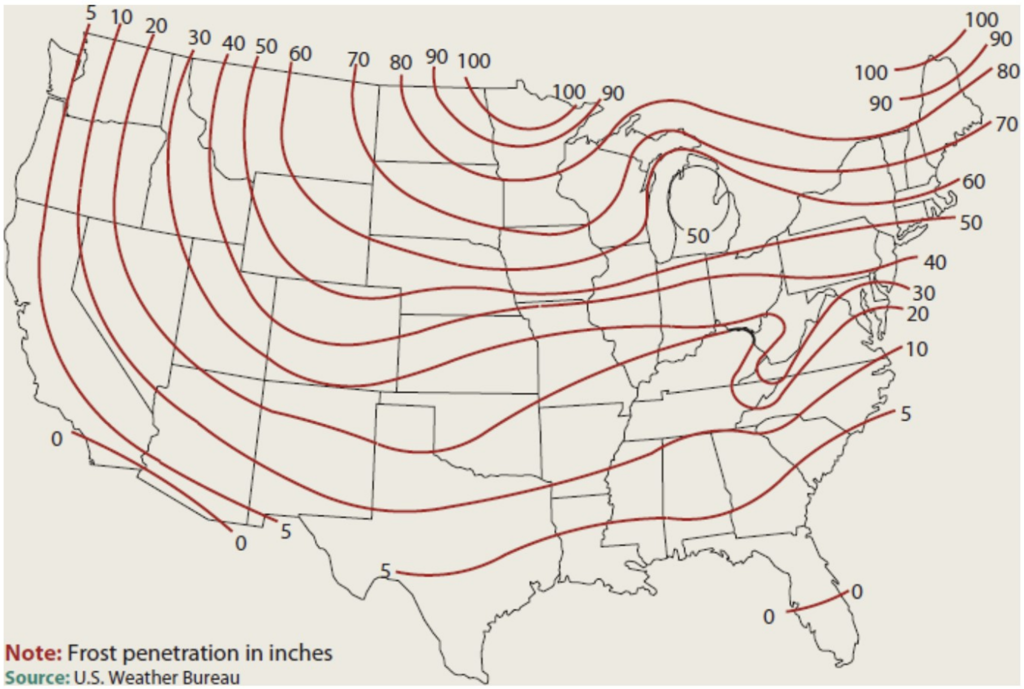
	p (w/-GCpi)	(w/-GCpi)
Zone 1T	1.43	3.77
Zone 2T	-5.66	-3.32
Zone 3T	-3.58	-1.24
Zone 3T	-3.06	-0.71

Appendix C – Foundation Calculations

Soil/Foundation Information:

Frost Penetration Depth:

Figure 1809-2 Frost penetration depths.



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Soil Data: (From U.S. Department of Agriculture Soil Conservation Services)

USDA United States Department of Agriculture Natural Resources Conservation Service

Web Soil Survey

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Area of Interest (AOI) | Soil Map | Soil Data Explorer | Download Soils Data | Shopping Cart (Free)

Printable Version | Add to Shopping Cart

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Map Unit Legend

Sac County, Iowa

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
2s088	Clarion loam, Bemis moraine, 2 to 6 percent slopes	2.7	58.8%
2tsj4	Nicollet loam, 1 to 3 percent slopes	1.5	33.0%
2tsj5	Webster clay loam, Bemis moraine, 0 to 2 percent slopes	0.3	6.3%
2tsjg	Clarion loam, Bemis moraine, 6 to 10 percent slopes, moderately eroded	0.1	1.9%
Totals for Area of Interest		**4.6**	**100.0%**

Soil Map

Scale (not to scale)

Warning: Soil Map may not be valid at this scale.

You have zoomed in beyond the scale at which the soil map for this area is intended to be used. Mapping of soils is done at a particular scale. The soil surveys that comprise your AOI were mapped at 1:15,800. The design of map units and the level of detail shown in the resulting soil map are dependent on that map scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

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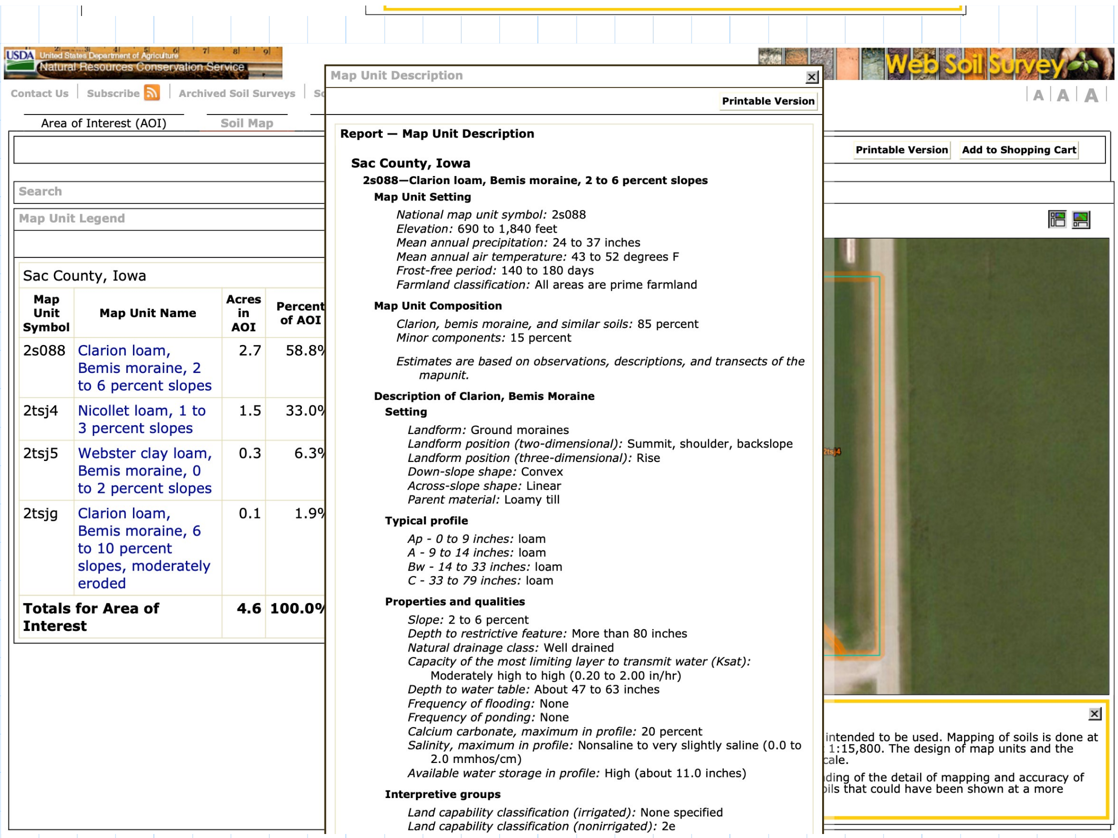

Sac County, Iowa

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
2s088	Clarion loam, Bemis moraine, 2 to 6 percent slopes	2.7	58.8%
2tsj4	Nicollet loam, 1 to 3 percent slopes	1.5	33.0%
2tsj5	Webster clay loam, Bemis moraine, 0 to 2 percent slopes	0.3	6.3%
2tsjg	Clarion loam, Bemis moraine, 6 to 10 percent slopes, moderately eroded	0.1	1.9%
Totals for Area of Interest		**4.6**	**100.0%**

Map Unit Description

Printable Version

Report — Map Unit Description

Sac County, Iowa

2s088—Clarion loam, Bemis moraine, 2 to 6 percent slopes

Map Unit Setting

National map unit symbol: 2s088
Elevation: 690 to 1,840 feet
Mean annual precipitation: 24 to 37 inches
Mean annual air temperature: 43 to 52 degrees F
Frost-free period: 140 to 180 days
Farmland classification: All areas are prime farmland

Map Unit Composition

Clarion, bemis moraine, and similar soils: 85 percent
Minor components: 15 percent

Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Clarion, Bemis Moraine

Setting

Landform: Ground moraines
Landform position (two-dimensional): Summit, shoulder, backslope
Landform position (three-dimensional): Rise
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Loamy till

Typical profile

Ap - 0 to 9 inches: loam
A - 9 to 14 inches: loam
Bw - 14 to 33 inches: loam
C - 33 to 79 inches: loam

Properties and qualities

Slope: 2 to 6 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.20 to 2.00 in/hr)
Depth to water table: About 47 to 63 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 20 percent
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Available water storage in profile: High (about 11.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 2e

Soil Survey Data:

108 SOIL SURVEY

TABLE 10.—*Engineering properties*

Map symbol and soil name	Depth	USDA texture	Classification: Unified	Classification: AASHTO
	In			
135, 135B: Coland	0–42	Clay loam	OL, CL, CH, MH	A–6, A–7
	42–60	Loam, sandy loam, sandy clay loam	CL, SC	A–4, A–6
138B, 138C, 138C2, 138D2: Clarion	0–16	Loam	CL, CL–ML	A–4, A–6
	16–32	Loam, clay loam	CL, CL–ML	A–4, A–6
	32–60	Loam, sandy loam	CL, CL–ML	A–4, A–6

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USCS Soil-class	Description	Cohesion (kPa)	Friction angle (°)
GW	well-graded gravel, fine to coarse gravel	0	40
GP	poorly graded gravel	0	38
GM	silty gravel	0	36
GC	clayey gravel	0	34
GM-GL	silty gravel	0	35
GC-CL	clayey gravel with many fines	3	29
SW	well-graded sand, fine to coarse sand	0	38
SP	poorly graded sand	0	36
SM	silty sand	0	34
SC	clayey sand	0	32
SM-SL	silty sand with many fines	0	34
SC-CL	clayey sand with many fines	5	28
ML	silt	0	33
CL	clay of low plasticity, lean clay	20	27
CH	clay of high plasticity, fat clay	25	22
OL	organic silt, organic clay	10	25
OH	organic clay, organic silt	10	22
MH	silt of high plasticity, elastic silt	5	24

Unified Soil Classification System (USCS)

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Foundation Design

Lake View Community Center

Exterior Continuous Footing:

Loads:

$P_1 := 71\ kip$ $V_1 := 44.2\ kip$ $M_1 := V_1 \cdot (4\ ft) = 176.8\ kip \cdot ft$

Soil Parameters: $LL_{imp} := 100\ psf$

$c' := 0\ psf$ $c_u := 10\ kPa = 1.45\ psi$ $\phi' := 30\ deg$ $E_s := 50\ MPa = (1.044 \cdot 10^6)\ psf$

$\gamma_{soil} := 130\ pcf$ $\gamma_{sat} := 135\ pcf$ $\gamma_{fill} := 120\ pcf$ $\gamma_w := 62.4\ pcf$ $\mu_s := 0.45$

Concrete Parameters: $GWT := 14\ ft$

$\gamma_c := 150\ pcf$

Footing Parameters:

Footing Depth:

$D_{min} := 60\ in = 5\ ft$ $D_f := 5\ ft + 6\ in$

Footing Thickness:

$t_f := 12\ in$

Fill Depth:

$D_{fill} := D_f - t_f = 4.5\ ft$

Slab Thickness:

$t_{slab} := 8\ in$

Factors of Safety:

$FS_q := 3$ $FS_v := 2$ $FS_T := 2$

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Foundation Design
Lake View Community Center

Bearing Capacity Analysis: (Vesic's Equations) - Exterior Footings

$B := 6\ ft + 6\ in$ $\quad L := 120\ ft$

$A_f := B \cdot L = 780\ ft^2$ $\quad w_f := t_f \cdot B \cdot \gamma_c = 0.975\ \frac{kip}{ft}$

$e := \frac{M_1 - (P_1 \cdot 1.5\ ft)}{P_1} = 0.99\ ft \quad \leq \quad \frac{B}{6} = 1.083\ ft$

Surcharge Load:

$q_s := D_f \cdot \gamma_{fill} + \gamma_c \cdot 6\ in + 100\ psf = 835\ psf$

Groundwater Effects:

$GWT = 14\ ft$

$D_f + B = 12\ ft$

Because GWT is greater than Df+B, there are no groundwater effects on the net bearing pressure.

Bearing Capacity Factors:

$$N_q := e^{\pi \cdot \tan(\phi')} \cdot \tan\left(45\ deg + \frac{\phi'}{2}\right)^2 = 18.401$$

$$N_c := \frac{(N_q - 1)}{\tan(\phi')} = 30.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi') = 22.402$$

Modification Factors:

Shape Factors:

$B' := B - 2 \cdot e = 4.52\ ft$ $\quad L' := L - 2 \cdot e = 118.02\ ft$

$s_c := 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right) = 1.023$ $\quad s_\gamma := 1 - \left(0.4 \cdot \left(\frac{B'}{L'}\right)\right) = 0.985$

$s_q := 1 + \left(\frac{B'}{L'}\right) \cdot \tan(\phi') = 1.022$

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Depth Factors:

$$k := \left(\frac{D_f}{B}\right) = 0.846$$

$$d_c := 1 + 0.4 \cdot k = 1.338$$

$$d_q := 1 + 2 \cdot k \cdot \tan(\phi') \cdot (1 - \sin(\phi'))^2 = 1.244$$

$$d_\gamma := 1$$

Load Inclination Factors:

$$m_l := \frac{2 + \frac{L}{B}}{1 + \frac{L}{B}} = 1.051 \qquad m_b := \frac{2 + \frac{B}{L}}{1 + \frac{B}{L}} = 1.949 \qquad m := \sqrt{m_l^2 + m_b^2} = 2.214$$

$$c' := c_u = 1.45 \ psi$$

$$i_c := 1 - \frac{m \cdot V_1}{A_f \cdot c' \cdot N_c} = 0.98$$

$$i_q := \left(1 - \frac{V_1}{P_1 + \frac{A_f \cdot c'}{\tan(\phi')}}\right)^m = 0.744$$

$$i_\gamma := \left(1 - \frac{V_1}{P_1 + \frac{A_f \cdot c'}{\tan(\phi')}}\right)^{m+1} = 0.651$$

Base Inclination Factors:

$$b_c := 1 \qquad b_q := 1 \qquad b_\gamma := 1$$

Ground Inclination Factors:

$$g_c := 1 \qquad g_q := 1 \qquad g_\gamma := 1$$

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Net Bearing Pressure:

$$q_n := c' \cdot N_c \cdot (s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c) + q_s \cdot N_q \cdot (s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q) + 0.5 \cdot B' \cdot \gamma_{soil} \cdot N_\gamma \cdot (s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma)$$

$$q_n = 188.893\ psi$$

$$Re := 1 - \sqrt{\frac{e}{ft}} = 0.005 \qquad q_{n'} := q_n \cdot (1 - Re) = 187.959\ psi$$

Allowable Bearing Pressure:

$$q_a := \frac{q_{n'}}{FS_q} = 9.022\ ksf$$

Actual Bearing Pressure:

$$q := \frac{P_1}{A_f} + \gamma_{fill} \cdot D_{fill} + \gamma_c \cdot 6\ in + 100\ psf + \gamma_c \cdot B = 12.368\ psi$$

$$w_f := B \cdot L \cdot t_f \cdot \gamma_c = 117\ kip$$

$$q_{min} := \frac{P_1 + w_f}{B \cdot L} \cdot \left(1 - \frac{6 \cdot e}{B}\right) = 20.734\ psf$$

$$q_{max} := \frac{P_1 + w_f}{B \cdot L} \cdot \left(1 + \frac{6 \cdot e}{B}\right) = 461.317\ psf$$

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Sliding Analysis:

$A_f := B \cdot L = 780\ ft^2$ $\qquad V_1 = 44.2\ kip$

Assume:

$FS_v := 2$ $\qquad c' := c_u$ $\qquad w_f := \gamma_c \cdot t_f \cdot A_f = 117\ kip$

Calculations:

Neglect Active Pressure: *Cohesive Soil

$V_{slide} := V_1 = 44.2\ kip$

$F_{max} := \left(P_1 + w_f\right) \cdot \tan\left(\phi'\right) + 0.5 \cdot c' \cdot A_f = 189.995\ kip$

$P_p := 0.5 \cdot \tan\left(45\ deg + \frac{\phi'}{2}\right)^2 \cdot \gamma_{fill} \cdot B \cdot D_f^{\ 2} = 35.393\ kip$

$V_n := F_{max} + 0.5 \cdot P_p = 207.691\ kip$

$$FS_v := \frac{V_n}{V_{slide}} = 4.699$$

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Settlement Analysis:

$H := 5 \cdot B = 32.5\ ft$

$A_f := B \cdot L = 780\ ft^2$

Bowles Method:

$\alpha := 4$ $\quad B' := \frac{B}{2}$ $\quad L' := \frac{L}{2}$ $\quad M := \frac{L'}{B'} = 18.462$ $\quad N := \frac{H}{B'} = 10$

$$I_1 := \frac{1}{\pi} \cdot \left(\left(M \cdot \ln\left(\frac{\left(1+\sqrt{M^2+1}\right) \cdot \sqrt{M^2+N^2}}{M \cdot \left(1+\sqrt{M^2+N^2+1}\right)} \right) \right) + \ln\left(\frac{\left(M+\sqrt{M^2+1}\right) \cdot \sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}} \right) \right) = 0.752$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \left(\operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2+N^2+1}} \right) \right) = 0.139$$

$$I_s := I_1 + \left(\frac{1 - 2 \cdot \mu_s}{1 - \mu_s} \right) \cdot I_2 = 0.777$$

$I_f := 1$

Final Settlement Calculations: $\qquad \delta_E \le 0.5\ in$

$$q_{net} := q - \gamma_{fill} \cdot D_f = 7.785\ psi$$

$$\delta_{flex} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1 - {\mu_s}^2\right)}{E_s} \right) \cdot B' = 0.104\ in$$

$$\delta_{rigid4} := 0.93 \cdot \delta_{flex} = 0.097\ in$$

Because the rigid settlement is less than 0.5 in. the design works.

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Interior Continuous Footing:

Loads:

$P_1 := 246\ kip$ $LL_{imp} := 100\ psf$

Soil Parameters:

$c' := psf$ $c_u := 10\ kPa = 1.45\ psi$ $\phi' := 30\ deg$ $E_s := 50\ MPa = \left(1.044 \cdot 10^6\right)\ psf$

$\gamma_{soil} := 130\ pcf$ $\gamma_{sat} := 135\ pcf$ $\gamma_{fill} := 120\ pcf$ $\gamma_w := 62.4\ pcf$ $\mu_s := 0.45$

Concrete Parameters: $GWT := 14\ ft$

$\gamma_c := 150\ pcf$

Footing Parameters:

Footing Depth:

$D_{min} := 60\ in = 5\ ft$ $D_f := 5\ ft + 6\ in$

Footing Thickness:

$t_f := 12\ in$

Fill Depth:

$D_{fill} := D_f - t_f = 4.5\ ft$

Slab Thickness:

$t_{slab} := 8\ in$

Factors of Safety:

$FS_q := 3$ $FS_v := 2$ $FS_T := 2$

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Bearing Capacity Analysis: (Vesic's Equations) - Exterior Footings

$B := 2\ ft + 0\ in$ $\qquad L := 120\ ft$

$A_f := B \cdot L = 240\ ft^2$ $\qquad w_f := t_f \cdot B \cdot \gamma_c = 0.3\ \frac{kip}{ft}$

Surcharge Load:

$q_s := D_f \cdot \gamma_{fill} + \gamma_c \cdot 6\ in + 20\ psf = 755\ psf$

Groundwater Effects:

$GWT = 14\ ft$

$D_f + B = 7.5\ ft$

Because GWT is greater than Df+B, there are no groundwater effects on the net bearing pressure.

Bearing Capacity Factors:

$$N_q := e^{\pi \cdot \tan(\phi')} \cdot \tan\left(45\ deg + \frac{\phi'}{2}\right)^2 = 18.401$$

$$N_c := \frac{(N_q - 1)}{\tan(\phi')} = 30.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi') = 22.402$$

Modification Factors:

Shape Factors:

$$s_c := 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right) = 1.01 \qquad s_\gamma := 1 - \left(0.4 \cdot \left(\frac{B}{L}\right)\right) = 0.993$$

$$s_q := 1 + \left(\frac{B}{L}\right) \cdot \tan(\phi') = 1.01$$

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Depth Factors:

$$k := \text{atan}\left(\frac{D_f}{B}\right) = 1.222$$

$$d_c := 1 + 0.4 \cdot k = 1.489$$

$$d_q := 1 + 2 \cdot k \cdot \tan(\phi') \cdot (1 - \sin(\phi'))^2 = 1.353$$

$$d_\gamma := 1$$

Load Inclination Factors:

$$m_l := \frac{2 + \frac{L}{B}}{1 + \frac{L}{B}} = 1.016 \qquad m_b := \frac{2 + \frac{B}{L}}{1 + \frac{B}{L}} = 1.984 \qquad m := \sqrt{{m_l}^2 + {m_b}^2} = 2.229$$

$$i_c := 1 \qquad i_q := 1 \qquad i_\gamma := 1$$

Base Inclination Factors:

$$b_c := 1 \qquad b_q := 1 \qquad b_\gamma := 1$$

Ground Inclination Factors:

$$g_c := 1 \qquad g_q := 1 \qquad g_\gamma := 1$$

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Net Bearing Pressure:

$$q_n := c' \cdot N_c \cdot (s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c) + q_s \cdot N_q \cdot (s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q) + 0.5 \cdot B \cdot \gamma_{soil} \cdot N_\gamma \cdot (s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma)$$

$$q_n = 152.173 \ psi$$

Allowable Bearing Pressure:

$$q_a := \frac{q_n}{FS_q} = 50.724 \ psi$$

Actual Bearing Pressure:

$$q := \frac{P_1}{A_f} + \gamma_{fill} \cdot D_{fill} + \gamma_c \cdot 6 \ in + 100 \ psf + \gamma_c \cdot B = 14.167 \ psi$$

$$\gamma_{soil} \cdot D_f = 4.965 \ psi$$

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Sliding Analysis:

$A_f := B \cdot L = 240\ ft^2$ $\quad$ $V_1 = ?\ kip$

Assume:

$FS_v := 2$ $\quad$ $c' := c_u$ $\quad$ $w_f := \gamma_c \cdot t_f \cdot A_f = 36\ kip$

Calculations:

Neglect Active Pressure: *Cohesive Soil

$V_{slide} := V_1 = ?\ kip$

$F_{max} := \left(P_1 + w_f\right) \cdot \tan\left(\phi'\right) + 0.5 \cdot c' \cdot A_f = 187.875\ kip$

$P_p := 0.5 \cdot \tan\left(45\ deg + \frac{\phi'}{2}\right)^2 \cdot \gamma_{fill} \cdot B \cdot D_f^{\ 2} = 10.89\ kip$

$V_n := F_{max} + 0.5 \cdot P_p = 193.32\ kip$

$$FS_v := \frac{V_n}{V_{slide}} = ?$$

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Settlement Analysis:

$H := 5 \cdot B = 10\ ft$

$A_f := B \cdot L = 240\ ft^2$

Bowles Method:

$\alpha := 4 \qquad B' := \frac{B}{2} \qquad L' := \frac{L}{2} \qquad M := \frac{L'}{B'} = 60 \qquad N := \frac{H}{B'} = 10$

$$I_1 := \frac{1}{\pi} \cdot \left(\left(M \cdot \ln\left(\frac{\left(1+\sqrt{M^2+1}\right) \cdot \sqrt{M^2+N^2}}{M \cdot \left(1+\sqrt{M^2+N^2+1}\right)} \right) \right) + \ln\left(\frac{\left(M+\sqrt{M^2+1}\right) \cdot \sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}} \right) \right) = 0.737$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \left(\operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2+N^2+1}} \right) \right) = 0.156$$

$$I_s := I_1 + \left(\frac{1-2 \cdot \mu_s}{1-\mu_s} \right) \cdot I_2 = 0.765$$

$I_f := 1$

Final Settlement Calculations: $\delta_E \le 0.5\ in$

$q_{net} := q - \gamma_{fill} \cdot D_f = 9.583\ psi$

$$\delta_{flex} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1-{\mu_s}^2\right)}{E_s} \right) \cdot B' = 0.039\ in$$

$\delta_{rigid4} := 0.93 \cdot \delta_{flex} = 0.036\ in$

Because the rigid settlement is less than 0.5 in. the design works.

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North Garage Exterior Continuous Footing:

Loads:

$P_1 := 18\ kip$ $V_1 := 7.1\ kip$ $M_1 := V_1 \cdot (4\ ft) = 28.4\ kip \cdot ft$

Soil Parameters: $LL_{imp} := 100\ psf$

$c' := 0\ psf$ $c_u := 10\ kPa = 1.45\ psi$ $\phi' := 30\ deg$ $E_s := 50\ MPa = (1.044 \cdot 10^6)\ psf$

$\gamma_{soil} := 130\ pcf$ $\gamma_{sat} := 135\ pcf$ $\gamma_{fill} := 120\ pcf$ $\gamma_w := 62.4\ pcf$ $\mu_s := 0.45$

Concrete Parameters: $GWT := 14\ ft$

$\gamma_c := 150\ pcf$

Footing Parameters:

Footing Depth:

$D_{min} := 60\ in = 5\ ft$ $D_f := 5\ ft + 6\ in$

Footing Thickness:

$t_f := 12\ in$

Fill Depth:

$D_{fill} := D_f - t_f = 4.5\ ft$

Slab Thickness:

$t_{slab} := 8\ in$

Factors of Safety:

$FS_q := 3$ $FS_v := 2$ $FS_T := 2$

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Bearing Capacity Analysis: (Vesic's Equations) - Exterior Footings

$B := 3\ ft + 6\ in$ $\qquad L := 25\ ft$

$A_f := B \cdot L = 87.5\ ft^2$ $\qquad w_f := t_f \cdot B \cdot \gamma_c = 0.525\ \dfrac{kip}{ft}$

$e := \dfrac{M_1 - (P_1 \cdot 1\ ft)}{P_1} = 0.578\ ft$ $\quad \le \quad$ $\dfrac{B}{6} = 0.583\ ft$

Surcharge Load:

$q_s := D_f \cdot \gamma_{fill} + \gamma_c \cdot 6\ in + 100\ psf = 835\ psf$

Groundwater Effects:

$GWT = 14\ ft$

$D_f + B = 9\ ft$

Because GWT is greater than Df+B, there are no groundwater effects on the net bearing pressure.

Bearing Capacity Factors:

$$N_q := e^{\pi \cdot \tan(\phi')} \cdot \tan\left(45\ deg + \frac{\phi'}{2}\right)^2 = 18.401$$

$$N_c := \frac{(N_q - 1)}{\tan(\phi')} = 30.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi') = 22.402$$

Modification Factors:

Shape Factors:

$B' := B - 2 \cdot e = 2.344\ ft$ $\qquad L' := L - 2 \cdot e = 23.844\ ft$

$s_c := 1 + \left(\dfrac{B'}{L'}\right) \cdot \left(\dfrac{N_q}{N_c}\right) = 1.06$ $\qquad s_\gamma := 1 - \left(0.4 \cdot \left(\dfrac{B'}{L'}\right)\right) = 0.961$

$$s_q := 1 + \left(\frac{B'}{L'}\right) \cdot \tan(\phi') = 1.057$$

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Depth Factors:

$$k := \left(\frac{D_f}{B}\right) = 1.571$$

$$d_c := 1 + 0.4 \cdot k = 1.629$$

$$d_q := 1 + 2 \cdot k \cdot \tan(\phi') \cdot (1 - \sin(\phi'))^2 = 1.454$$

$$d_\gamma := 1$$

Load Inclination Factors:

$$m_l := \frac{2 + \frac{L}{B}}{1 + \frac{L}{B}} = 1.123 \qquad m_b := \frac{2 + \frac{B}{L}}{1 + \frac{B}{L}} = 1.877 \qquad m := \sqrt{{m_l}^2 + {m_b}^2} = 2.187$$

$$c' := c_u = 1.45 \; psi$$

$$i_c := 1 - \frac{m \cdot V_1}{A_f \cdot c' \cdot N_c} = 0.972$$

$$i_q := \left(1 - \frac{V_1}{P_1 + \frac{A_f \cdot c'}{\tan(\phi')}}\right)^m = 0.714$$

$$i_\gamma := \left(1 - \frac{V_1}{P_1 + \frac{A_f \cdot c'}{\tan(\phi')}}\right)^{m+1} = 0.612$$

Base Inclination Factors:

$$b_c := 1 \qquad b_q := 1 \qquad b_\gamma := 1$$

Ground Inclination Factors:

$$g_c := 1 \qquad g_q := 1 \qquad g_\gamma := 1$$

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Net Bearing Pressure:

$$q_n := c' \cdot N_c \cdot (s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c) + q_s \cdot N_q \cdot (s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q) + 0.5 \cdot B' \cdot \gamma_{soil} \cdot N_\gamma \cdot (s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma)$$

$$q_n = 204.218\ psi$$

$$Re := 1 - \sqrt{\frac{e}{ft}} = 0.24 \qquad q_{n'} := q_n \cdot (1 - Re) = 155.229\ psi$$

Allowable Bearing Pressure:

$$q_a := \frac{q_{n'}}{FS_q} = 51.743\ psi$$

Actual Bearing Pressure:

$$q := \frac{P_1}{A_f} + \gamma_{fill} \cdot D_{fill} + \gamma_c \cdot 6\ in + 100\ psf + \gamma_c \cdot B = 10.04\ psi$$

$$w_f := B \cdot L \cdot t_f \cdot \gamma_c = 13.125\ kip$$

$$q_{min} := \frac{P_1 + w_f}{B \cdot L} \cdot \left(1 - \frac{6 \cdot e}{B}\right) = 3.388\ psf$$

$$q_{max} := \frac{P_1 + w_f}{B \cdot L} \cdot \left(1 + \frac{6 \cdot e}{B}\right) = 708.041\ psf$$

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Sliding Analysis:

$A_f := B \cdot L = 87.5\ ft^2$ $\quad V_1 = 7.1\ kip$

Assume:

$FS_v := 2$ $\quad c' := c_u$ $\quad w_f := \gamma_c \cdot t_f \cdot A_f = 13.125\ kip$

Calculations:

Neglect Active Pressure: *Cohesive Soil

$V_{slide} := V_1 = 7.1\ kip$

$F_{max} := (P_1 + w_f) \cdot \tan(\phi') + 0.5 \cdot c' \cdot A_f = 27.107\ kip$

$P_p := 0.5 \cdot \tan\left(45\ deg + \frac{\phi'}{2}\right)^2 \cdot \gamma_{fill} \cdot B \cdot D_f^{\,2} = 19.058\ kip$

$V_n := F_{max} + 0.5 \cdot P_p = 36.636\ kip$

$$FS_v := \frac{V_n}{V_{slide}} = 5.16$$

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Settlement Analysis:

$H := 5 \cdot B = 17.5\ ft$

$A_f := B \cdot L = 87.5\ ft^2$

Bowles Method:

$\alpha := 4 \qquad B' := \frac{B}{2} \qquad L' := \frac{L}{2} \qquad M := \frac{L'}{B'} = 7.143 \qquad N := \frac{H}{B'} = 10$

$$I_1 := \frac{1}{\pi} \cdot \left(\left(M \cdot \ln\left(\frac{\left(1+\sqrt{M^2+1}\right) \cdot \sqrt{M^2+N^2}}{M \cdot \left(1+\sqrt{M^2+N^2+1}\right)} \right) \right) + \ln\left(\frac{\left(M+\sqrt{M^2+1}\right) \cdot \sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}} \right) \right) = 0.77$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \left(\operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2+N^2+1}} \right) \right) = 0.092$$

$$I_s := I_1 + \left(\frac{1 - 2 \cdot \mu_s}{1 - \mu_s} \right) \cdot I_2 = 0.787$$

$I_f := 1$

Final Settlement Calculations: $\delta_E \le 0.5\ in$

$q_{net} := q - \gamma_{fill} \cdot D_f = 5.456\ psi$

$$\delta_{flex} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1 - {\mu_s}^2\right)}{E_s} \right) \cdot B' = 0.04\ in$$

$\delta_{rigid4} := 0.93 \cdot \delta_{flex} = 0.037\ in$

Because the rigid settlement is less than 0.5 in. the design works.

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West Garage Exterior Continuous Footing:

Loads:

$P_1 := 35\ kip$ $V_1 := 14.25\ kip$ $M_1 := V_1 \cdot (4\ ft) = 57\ kip \cdot ft$

Soil Parameters: $LL_{imp} := 100\ psf$

$c' := 0\ psf$ $c_u := 10\ kPa = 1.45\ psi$ $\phi' := 30\ deg$ $E_s := 50\ MPa = \left(1.044 \cdot 10^6\right)\ psf$

$\gamma_{soil} := 130\ pcf$ $\gamma_{sat} := 135\ pcf$ $\gamma_{fill} := 120\ pcf$ $\gamma_w := 62.4\ pcf$ $\mu_s := 0.45$

Concrete Parameters: $GWT := 14\ ft$

$\gamma_c := 150\ pcf$

Footing Parameters:

Footing Depth:

$D_{min} := 60\ in = 5\ ft$ $D_f := 5\ ft + 6\ in$

Footing Thickness:

$t_f := 12\ in$

Fill Depth:

$D_{fill} := D_f - t_f = 4.5\ ft$

Slab Thickness:

$t_{slab} := 8\ in$

Factors of Safety:

$FS_q := 3$ $FS_v := 2$ $FS_T := 2$

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Bearing Capacity Analysis: (Vesic's Equations) - Exterior Footings

$B := 4\ ft + 0\ in$ $\qquad L := 50\ ft$

$A_f := B \cdot L = 200\ ft^2$ $\qquad w_f := t_f \cdot B \cdot \gamma_c = 0.6\ \frac{kip}{ft}$

$e := \frac{M_1 - (P_1 \cdot 1\ ft)}{P_1} = 0.629\ ft$ $\quad \le \quad$ $\frac{B}{6} = 0.667\ ft$

Surcharge Load:

$q_s := D_f \cdot \gamma_{fill} + \gamma_c \cdot 6\ in + 100\ psf = 835\ psf$

Groundwater Effects:

$GWT = 14\ ft$

$D_f + B = 9.5\ ft$

Because GWT is greater than Df+B, there are no groundwater effects on the net bearing pressure.

Bearing Capacity Factors:

$$N_q := e^{\pi \cdot \tan(\phi')} \cdot \tan\left(45\ deg + \frac{\phi'}{2}\right)^2 = 18.401$$

$$N_c := \frac{(N_q - 1)}{\tan(\phi')} = 30.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi') = 22.402$$

Modification Factors:

Shape Factors:

$B' := B - 2 \cdot e = 2.743\ ft$ $\qquad L' := L - 2 \cdot e = 48.743\ ft$

$s_c := 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right) = 1.034$ $\qquad s_\gamma := 1 - \left(0.4 \cdot \left(\frac{B'}{L'}\right)\right) = 0.977$

$$s_q := 1 + \left(\frac{B'}{L'}\right) \cdot \tan(\phi') = 1.032$$

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Depth Factors:

$$k := \left(\frac{D_f}{B}\right) = 1.375$$

$$d_c := 1 + 0.4 \cdot k = 1.55$$

$$d_q := 1 + 2 \cdot k \cdot \tan(\phi') \cdot (1 - \sin(\phi'))^2 = 1.397$$

$$d_\gamma := 1$$

Load Inclination Factors:

$$m_l := \frac{2 + \frac{L}{B}}{1 + \frac{L}{B}} = 1.074 \qquad m_b := \frac{2 + \frac{B}{L}}{1 + \frac{B}{L}} = 1.926 \qquad m := \sqrt{m_l^2 + m_b^2} = 2.205$$

$$c' := c_u = 1.45\ psi$$

$$i_c := 1 - \frac{m \cdot V_1}{A_f \cdot c' \cdot N_c} = 0.975$$

$$i_q := \left(1 - \frac{V_1}{P_1 + \frac{A_f \cdot c'}{\tan(\phi')}}\right)^m = 0.73$$

$$i_\gamma := \left(1 - \frac{V_1}{P_1 + \frac{A_f \cdot c'}{\tan(\phi')}}\right)^{m+1} = 0.634$$

Base Inclination Factors:

$$b_c := 1 \qquad b_q := 1 \qquad b_\gamma := 1$$

Ground Inclination Factors:

$$g_c := 1 \qquad g_q := 1 \qquad g_\gamma := 1$$

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Net Bearing Pressure:

$$q_n := c' \cdot N_c \cdot \left(s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c\right) + q_s \cdot N_q \cdot \left(s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q\right) + 0.5 \cdot B' \cdot \gamma_{soil} \cdot N_\gamma \cdot \left(s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma\right)$$

$$q_n = 197.927\ psi$$

$$Re := 1 - \sqrt{\frac{e}{ft}} = 0.207 \qquad q_{n'} := q_n \cdot (1 - Re) = 156.922\ psi$$

Allowable Bearing Pressure:

$$q_a := \frac{q_{n'}}{FS_q} = 52.307\ psi$$

Actual Bearing Pressure:

$$q := \frac{P_1}{A_f} + \gamma_{fill} \cdot D_{fill} + \gamma_c \cdot 6\ in + 100\ psf + \gamma_c \cdot B = 10.347\ psi$$

$$w_f := B \cdot L \cdot t_f \cdot \gamma_c = 30\ kip$$

$$q_{min} := \frac{P_1 + w_f}{B \cdot L} \cdot \left(1 - \frac{6 \cdot e}{B}\right) = 18.571\ psf$$

$$q_{max} := \frac{P_1 + w_f}{B \cdot L} \cdot \left(1 + \frac{6 \cdot e}{B}\right) = 631.429\ psf$$

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Sliding Analysis:

$A_f := B \cdot L = 200\ ft^2$ $V_1 = 14.25\ kip$

Assume:

$FS_v := 2$ $c' := c_u$ $w_f := \gamma_c \cdot t_f \cdot A_f = 30\ kip$

Calculations:

Neglect Active Pressure: *Cohesive Soil

$V_{slide} := V_1 = 14.25\ kip$

$F_{max} := \left(P_1 + w_f\right) \cdot \tan\left(\phi'\right) + 0.5 \cdot c' \cdot A_f = 58.413\ kip$

$P_p := 0.5 \cdot \tan\left(45\ deg + \frac{\phi'}{2}\right)^2 \cdot \gamma_{fill} \cdot B \cdot {D_f}^2 = 21.78\ kip$

$V_n := F_{max} + 0.5 \cdot P_p = 69.303\ kip$

$$FS_v := \frac{V_n}{V_{slide}} = 4.863$$

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Settlement Analysis:

$$H := 5 \cdot B = 20\ ft$$

$$A_f := B \cdot L = 200\ ft^2$$

Bowles Method:

$$\alpha := 4 \qquad B' := \frac{B}{2} \qquad L' := \frac{L}{2} \qquad M := \frac{L'}{B'} = 12.5 \qquad N := \frac{H}{B'} = 10$$

$$I_1 := \frac{1}{\pi} \cdot \left(\left(M \cdot \ln\left(\frac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)} \right) \right) + \ln\left(\frac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}} \right) \right) = 0.762$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \left(\operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2 + N^2 + 1}} \right) \right) = 0.124$$

$$I_s := I_1 + \left(\frac{1 - 2 \cdot \mu_s}{1 - \mu_s} \right) \cdot I_2 = 0.785$$

$$I_f := 1$$

Final Settlement Calculations: $\delta_E \le 0.5\ in$

$$q_{net} := q - \gamma_{fill} \cdot D_f = 5.764\ psi$$

$$\delta_{flex} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1 - {\mu_s}^2\right)}{E_s} \right) \cdot B' = 0.048\ in$$

$$\delta_{rigid4} := 0.93 \cdot \delta_{flex} = 0.044\ in$$

Because the rigid settlement is less than 0.5 in. the design works.

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Spread Footings:

Loads:

$P_1 := 10.2\ kip$ $\quad T := 5.65\ kip$ $\quad LL_{imp} := 100\ psf$

Soil Parameters:

$c' := 0\ psf$ $\quad c_u := 10\ kPa = 1.45\ psi$ $\quad \phi' := 30\ deg$ $\quad E_s := 50\ MPa = \left(1.044 \cdot 10^6\right)\ psf$

$\gamma_{soil} := 130\ pcf$ $\quad \gamma_{sat} := 135\ pcf$ $\quad \gamma_{fill} := 120\ pcf$ $\quad \gamma_w := 62.4\ pcf$ $\quad \mu_s := 0.45$

Concrete Parameters:

$GWT := 14\ ft$

$\gamma_c := 150\ pcf$

Footing Parameters:

Footing Depth:

$D_{min} := 60\ in = 5\ ft$ $\quad D_f := 5\ ft + 6\ in$

Footing Thickness:

$t_f := 12\ in$

Fill Depth:

$D_{fill} := D_f - t_f = 4.5\ ft$

Slab Thickness:

$t_{slab} := 8\ in$

Factors of Safety:

$FS_q := 3$ $\quad FS_v := 2$ $\quad FS_T := 2$

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Bearing Capacity Analysis: (Vesic's Equations) - Exterior Footings

$B := 3\ ft + 0\ in$ $\qquad L := B$

$A_f := B \cdot L = 9\ ft^2$ $\qquad w_f := t_f \cdot B \cdot \gamma_c = 0.45\ \dfrac{kip}{ft}$

Surcharge Load:

$q_s := D_f \cdot \gamma_{fill} + \gamma_c \cdot 6\ in + 100\ psf = 835\ psf$

Groundwater Effects:

$GWT = 14\ ft$

$D_f + B = 8.5\ ft$

Because GWT is greater than Df+B, there are no groundwater effects on the net bearing pressure.

Bearing Capacity Factors:

$$N_q := e^{\pi \cdot \tan(\phi')} \cdot \tan\left(45\ deg + \frac{\phi'}{2}\right)^2 = 18.401$$

$$N_c := \frac{(N_q - 1)}{\tan(\phi')} = 30.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi') = 22.402$$

Modification Factors:

Shape Factors:

$$s_c := 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right) = 1.611 \qquad s_\gamma := 1 - \left(0.4 \cdot \left(\frac{B}{L}\right)\right) = 0.6$$

$$s_q := 1 + \left(\frac{B}{L}\right) \cdot \tan(\phi') = 1.577$$

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Depth Factors:

$$k := \operatorname{atan}\left(\frac{D_f}{B}\right) = 1.071$$

$$d_c := 1 + 0.4 \cdot k = 1.429$$

$$d_q := 1 + 2 \cdot k \cdot \tan(\phi') \cdot (1 - \sin(\phi'))^2 = 1.309$$

$$d_\gamma := 1$$

Load Inclination Factors:

$$m_l := \frac{2 + \frac{L}{B}}{1 + \frac{L}{B}} = 1.5 \qquad m_b := \frac{2 + \frac{B}{L}}{1 + \frac{B}{L}} = 1.5 \qquad m := \sqrt{{m_l}^2 + {m_b}^2} = 2.121$$

$i_c := 1$ $\quad i_q := 1$ $\quad i_\gamma := 1$

Base Inclination Factors:

$b_c := 1$ $\quad b_q := 1$ $\quad b_\gamma := 1$

Ground Inclination Factors:

$g_c := 1$ $\quad g_q := 1$ $\quad g_\gamma := 1$

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Net Bearing Pressure:

$$q_n := c' \cdot N_c \cdot (s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c) + q_s \cdot N_q \cdot (s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q) + 0.5 \cdot B \cdot \gamma_{soil} \cdot N_\gamma \cdot (s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma)$$

$$q_n = 238.564 \ psi$$

Allowable Bearing Pressure:

$$q_a := \frac{q_n}{FS_q} = 79.521 \ psi$$

Actual Bearing Pressure:

$$q := \frac{P_1}{A_f} + \gamma_{fill} \cdot D_{fill} + \gamma_c \cdot 6 \ in + 100 \ psf + \gamma_c \cdot B = 15.961 \ psi$$

$$w_f := B \cdot L \cdot t_f \cdot \gamma_c = 1.35 \ kip$$

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Uplift Analysis:

$A_f := B \cdot L = 9\ ft^2$

Assume:

$w_f = 1.35\ kip$ $FS_t := 2$

$T = 5.65\ kip$

$p := 2 \cdot B + 2 \cdot L = 12\ ft$ $w_f := B \cdot L \cdot t_f \cdot \gamma_c = 1.35\ kip$

$s_u := c_u$

$\dfrac{s_u \cdot p \cdot D_f + w_f}{FS_t} = 7.567\ kip$ $>$ $T = 5.65\ kip$

Settlement Analysis:

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$H := 5 \cdot B = 15 \; ft$

$A_f := B \cdot L = 9 \; ft^2$

Bowles Method:

$\alpha := 4 \qquad B' := \frac{B}{2} \qquad L' := \frac{L}{2} \qquad M := \frac{L'}{B'} = 1 \qquad N := \frac{H}{B'} = 10$

$$I_1 := \frac{1}{\pi} \cdot \left(\left(M \cdot \ln \left(\frac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)} \right) \right) + \ln \left(\frac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}} \right) \right) = 0.498$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \left(\operatorname{atan} \left(\frac{M}{N \cdot \sqrt{M^2 + N^2 + 1}} \right) \right) = 0.016$$

$$I_s := I_1 + \left(\frac{1 - 2 \cdot \mu_s}{1 - \mu_s} \right) \cdot I_2 = 0.501$$

$I_f := 1$

Final Settlement Calculations: $\delta_E \le 0.5 \; in$

$$q_{net} := q - \gamma_{fill} \cdot D_f = 11.377 \; psi$$

$$\delta_{flex} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1 - {\mu_s}^2\right)}{E_s} \right) \cdot B' = 0.045 \; in$$

$$\delta_{rigid4} := 0.93 \cdot \delta_{flex} = 0.042 \; in$$

Because the rigid settlement is less than 0.5 in. the design works.

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Concrete/Rebar Design:

Section Properties:

$E_s := 29000\ ksi$ $w_c := 150\ pcf$ $t_f := 1\ ft$ $A_{b5} := 0.44\ in^2$

$f_{c'} := 4000\ psi$ $f_y := 60\ ksi$ $d_{b5} := 0.625\ in$

$E_c := 33 \cdot (150)^{1.5} \cdot \sqrt{4000}\ psi = 3834.254\ ksi$ $cover := 3\ in$

$\beta := 0.85$

Analysis: $n := 1$

Singly Rein. Rectangular Section:

$b := 12\ in$ $d := t_f - cover = 9\ in$ $A_s := A_{b5} \cdot n = 0.44\ in^2$

$$y_c := \frac{\sqrt{n \cdot A_s} \cdot \sqrt{n \cdot A_s + 2 \cdot b \cdot d} - n \cdot A_s}{b} = 0.777\ in$$

$$I := n \cdot A_s \cdot (d - y_c)^2 + \frac{1}{12} \cdot b \cdot y_c^3 + b \cdot y_c \cdot \left(\frac{y_c}{2}\right)^2 = 31.628\ in^4$$

$$A_{stensioncontrol} := \frac{0.85 \cdot f_{c'} \cdot b \cdot \beta}{f_y} \cdot \left(\frac{3 \cdot d}{8}\right) = 1.951\ in^2$$

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f_{c'} \cdot b} = 0.647\ in$$

$$\phi M_n := 0.9 \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 17.179\ kip \cdot ft$$

Shear Check:

$\lambda := 1$ $\rho_w := \frac{A_s}{b \cdot d} = 0.004$

$$\phi V_c := 0.9 \cdot min\left(5 \cdot \lambda \cdot \sqrt{f_{c'} \cdot psi}, 8 \cdot \lambda \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{f_{c'} \cdot psi}\right) \cdot b \cdot d = 7.855\ kip$$

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Interior Footing:

$$w := 14.167\ psi \cdot 1\ ft = 2040.048\ \frac{lbf}{ft}$$

Shear - Moment EQ:

$$V(x) := w \cdot x$$

$$M(x) := \frac{w \cdot x^2}{2}$$

$$M(0.5\ ft) = 0.255\ kip \cdot ft$$

$$V(0.5\ ft) = 1.02\ kip$$

Spread Footing:

$$w := 15.961\ psi \cdot 1\ ft = 2298.384\ \frac{lbf}{ft}$$

Shear - Moment EQ:

$$V(x) := w \cdot x$$

$$M(x) := \frac{w \cdot x^2}{2}$$

$$M(0.5\ ft) = 0.287\ kip \cdot ft$$

$$V(0.5\ ft) = 1.149\ kip$$

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Exterior Footings:

$$q_{min} := 20.734\ \frac{lbf}{ft} \qquad q_{max} := 461.317\ \frac{lbf}{ft} \qquad B := 6\ ft + 6\ in$$

$$slope := \frac{q_{max} - q_{min}}{B} = 67.782\ psf$$

$$q := q_{max} - (4\ ft + 3\ in) \cdot slope = 173.244\ \frac{lbf}{ft}$$

$$w_1 := q \qquad w_2 := (q_{max} - q) = 288.074\ \frac{lbf}{ft} \qquad x := 4\ ft + 3\ in$$

$$M_{max} := w_1 \cdot \frac{x^2}{2} + 0.5 \cdot w_2 \cdot x \cdot \left(\frac{2}{3}\ x\right) = 3.299\ kip \cdot ft$$

$$V_{max} := w_1 \cdot x + 0.5 \cdot x \cdot w_2 = 1.348\ kip$$

North Garage Footings:

$$q_{min} := 3.388\ \frac{lbf}{ft} \qquad q_{max} := 708.04\ \frac{lbf}{ft} \qquad B := 3\ ft + 6\ in$$

$$slope := \frac{q_{max} - q_{min}}{B} = 201.329\ psf \qquad x := 2\ ft + 3\ in$$

$$q := q_{max} - (x) \cdot slope = 255.049\ \frac{lbf}{ft}$$

$$w_1 := q \qquad w_2 := (q_{max} - q) = 452.991\ \frac{lbf}{ft}$$

$$M_{max} := w_1 \cdot \frac{x^2}{2} + 0.5 \cdot w_2 \cdot x \cdot \left(\frac{2}{3}\ x\right) = 1.41\ kip \cdot ft$$

$$V_{max} := w_1 \cdot x + 0.5 \cdot x \cdot w_2 = 1.083\ kip$$

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West Garage Footings:

$$q_{min} := 18.571 \frac{lbf}{ft} \qquad q_{max} := 631.43 \frac{lbf}{ft} \qquad B := 4\ ft + 4\ in$$

$$slope := \frac{q_{max} - q_{min}}{B} = 141.429\ psf \qquad x := 2\ ft + 6\ in$$

$$q := q_{max} - (x) \cdot slope = 277.858 \frac{lbf}{ft}$$

$$w_1 := q \qquad w_2 := \left(q_{max} - q\right) = 353.573 \frac{lbf}{ft}$$

$$M_{max} := w_1 \cdot \frac{x^2}{2} + 0.5 \cdot w_2 \cdot x \cdot \left(\frac{2}{3}\ x\right) = 1.605\ kip \cdot ft$$

$$V_{max} := w_1 \cdot x + 0.5 \cdot x \cdot w_2 = 1.137\ kip$$

Rebar design provides adequate flexural strength for all footings and the concrete provides adequate strength for shear forces.

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Appendix D – Runoff Calculations

Runoff Analysis

Pre-Construction

Catchment Area One:

Time of Concentration: (based on NRCS Lag Method - SUDAS Section 2B-3-D)

$A_1 := 259153\ ft^2 = 5.949\ acre$ $\quad A_1 := 5.949$

$l := 927.7\ ft$ $\quad l := 927.7$

$CN := 79$ (Table 2B - 4.03 Soils Group C)

$$S := \frac{1000}{CN} - 10 = 2.658$$

$$Y := \frac{10\ ft}{l} = 0.011\ ft \qquad Y := 1.1\%$$

$$t_c := \frac{100 \cdot l^{0.8} \cdot (S+1)^{0.7}}{1900 \cdot Y^{0.5}} = 294.283$$

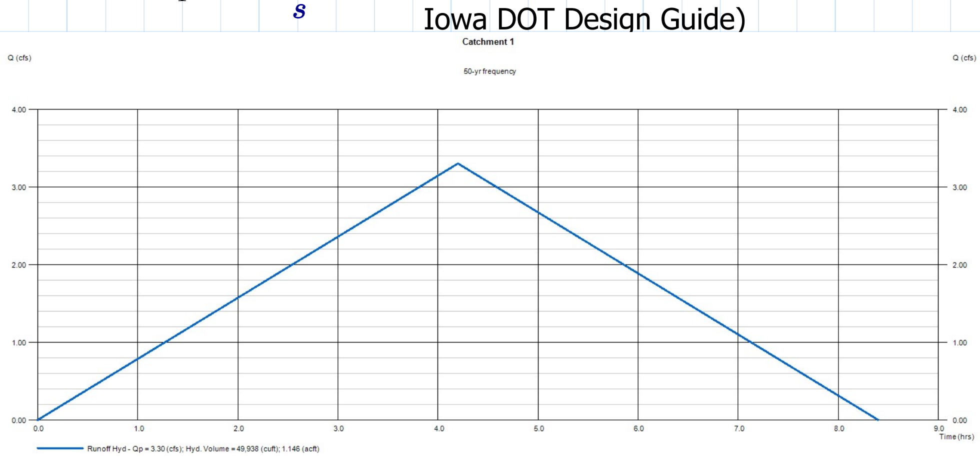
$$t_c := 294\ min = 4.9\ hr$$

Flowrate from Civil 3D:

$C_{runoff} := 0.6$ Table 2B-4.01 (SUDAS Design Manual)

$A_{impermeable} := 54175\ ft^2 = 1.244\ acre$ $\quad C_{imper} := 0.97$

$Q_1 := 3.3\ \frac{ft^3}{s}$ based on 50-year return Period (Per Iowa DOT Design Guide)



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Runoff Analysis

Pre-Construction

NRCS Velocity Method:

(1.) Sheet Flow:

$n := 0.24$ (Dense Grass - SUDAS Table 2B-3.01)

$P_2 := 3.01$ (2-year, 24 hour rainfall (in) - Table 2B-2.04 SUDAS)

$l := 50$ (Initial flow of rainfall - no more than 100 ft) - units (ft)

$S := \frac{2}{l} = 0.04$ (Slope - ft/ft)

$$T_s := \frac{0.007 \cdot (n \cdot l)^{0.8}}{P_2^{\,0.5} \cdot Y^{0.4}} = 0.179$$

$$T_s := 0.179\ hr = 10.74\ min$$

(2.) Shallow Concentrated flow:

$l := 350\ ft$

$s := 7\ \frac{ft}{l} = 0.02$

$V_c := 6.962 \cdot (s)^{0.5}\ \frac{ft}{sec} = 0.985\ \frac{ft}{s}$ (ft/s) - Table 2B - 3.01 (Short - Grass Prairie)

$$T_{sc} := \frac{l}{V_c} = 5.925\ min$$

(3.) Open Channel Flow:

$n := 0.03$ (Manning Coeff. - Natural Stream #1)

Assumptions for 'r':

$d := 0.2\ ft$ $w := 0.2\ ft$ (Triangular Channel)

$a := 0.5\ w \cdot d = 0.02\ ft^2$ $P := 2 \cdot \left(\sqrt{(0.5\ w)^2 + d^2}\right) = 0.447\ ft$

$r := \frac{a}{P} = 0.045\ ft$ $r := 0.045$

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Runoff Analysis

Pre-Construction

$$l := 927\ ft - 350\ ft = 577\ ft$$

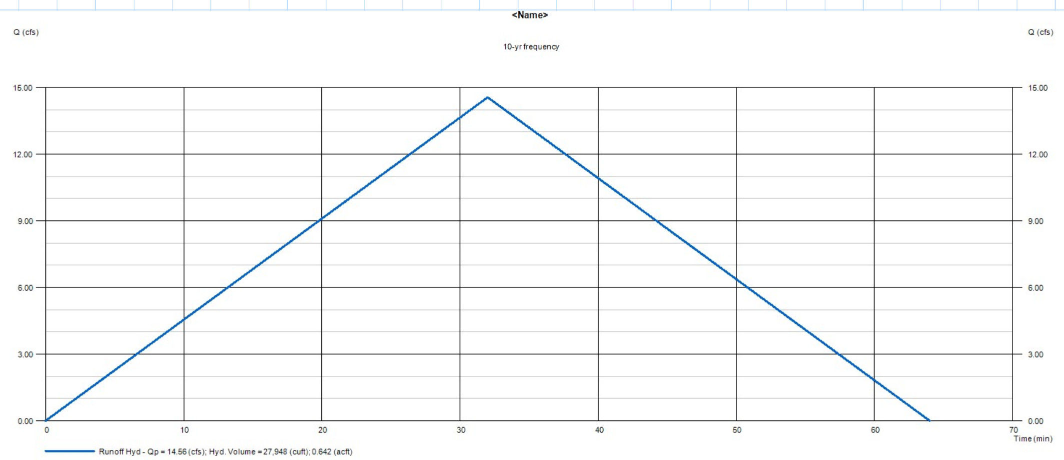
$$s := 6\ \frac{ft}{l} = 0.01$$

$$V_{oc} := \frac{1.49 \cdot r^{\frac{2}{3}} \cdot s^{0.5}}{n} = 0.641 \qquad V_{oc} := 0.641\ \frac{ft}{sec} = 0.641\ \frac{ft}{s}$$

$$T_o := \frac{l}{V_{oc}} = 15.003\ min$$

Time of Concentration:

$$T_c := T_s + T_{sc} + T_o = 31.667\ min$$



$$Q_{10y1} := 14.56\ \frac{ft^3}{sec} = 14.56\ \frac{1}{s} \cdot ft^3 \qquad V := 27948\ ft^3 = 0.642\ acre \cdot ft$$

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Runoff Analysis

Pre-Construction

Catchment Area Two:

Time of Concentration:

(based on NRCS Lag Method - SUDAS Section 2B-3-D)

$A_2 := 79643\ ft^2 = 1.828\ acre$ $\qquad A_2 := 1.83$

$l := 512.5\ ft$ $\qquad l := 512.5$

$CN := 79$ (Table 2B - 4.03 Soils Group C)

$$S := \frac{1000}{CN} - 10 = 2.658$$

$Y := \frac{12\ ft}{l} = 0.023\ ft$ $\qquad Y := 2.3\%$

$$t_c := \frac{100 \cdot l^{0.8} \cdot (S+1)^{0.7}}{1900 \cdot Y^{0.5}} = 126.598$$

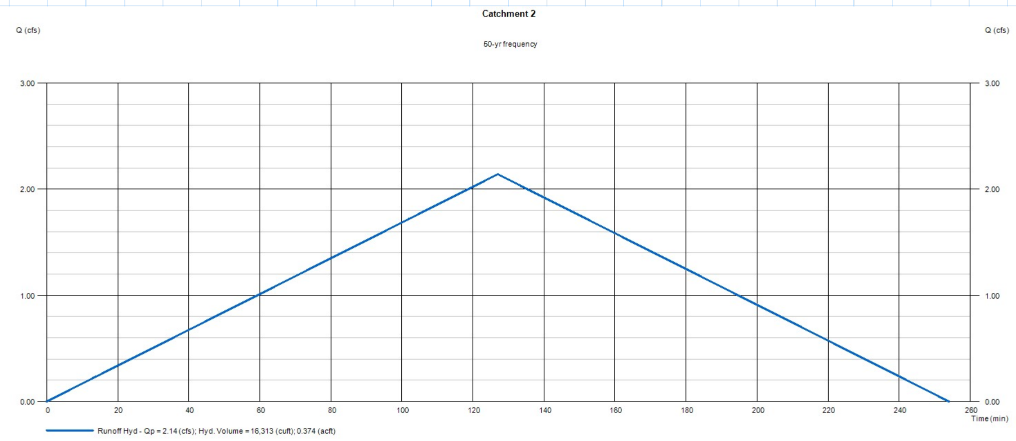
$$t_c := 126.6\ min = 2.11\ hr$$

Flowrate from Civil 3D:

$C_{runoff} := 0.6$ Table 2B-4.01 (SUDAS Design Manual)

$A_{impermeable} := 0\ ft^2 = 0\ acre$ $\qquad C_{imper} := 0.97$

$Q_2 := 2.14\ \frac{ft^3}{s}$ based on 50-year return Period (Per Iowa DOT Design Guide)



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Runoff Analysis

Pre-Construction

NRCS Velocity Method:

(1.) Sheet Flow:

$n := 0.24$ (Dense Grass - SUDAS Table 2B-3.01)

$P_2 := 3.01$ (2-year, 24 hour rainfall (in) - Table 2B-2.04 SUDAS)

$l := 50$ (Initial flow of rainfall - no more than 100 ft) - units (ft)

$S := \frac{2}{l} = 0.04$ (Slope - ft/ft)

$$T_s := \frac{0.007 \cdot (n \cdot l)^{0.8}}{{P_2}^{0.5} \cdot Y^{0.4}} = 0.133$$

$$T_s := 0.179\ hr = 10.74\ min$$

(2.) Shallow Concentrated flow:

$l := 100\ ft$

$s := 7\ \frac{ft}{l} = 0.07$

$V_c := 6.962 \cdot (s)^{0.5}\ \frac{ft}{sec} = 1.842\ \frac{ft}{s}$ (ft/s) - Table 2B - 3.01 (Short - Grass Prairie)

$$T_{sc} := \frac{l}{V_c} = 0.905\ min$$

(3.) Open Channel Flow:

$n := 0.03$ (Manning Coeff. - Natural Stream #1)

Assumptions for 'r':

$d := 0.2\ ft$ $w := 0.2\ ft$ (Triangular Channel)

$a := 0.5\ w \cdot d = 0.02\ ft^2$ $P := 2 \cdot \left(\sqrt{(0.5\ w)^2 + d^2}\right) = 0.447\ ft$

$r := \frac{a}{P} = 0.045\ ft$ $r := 0.045$

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Runoff Analysis

Pre-Construction

$$l := 512.5\ ft - 100\ ft = 412.5\ ft$$

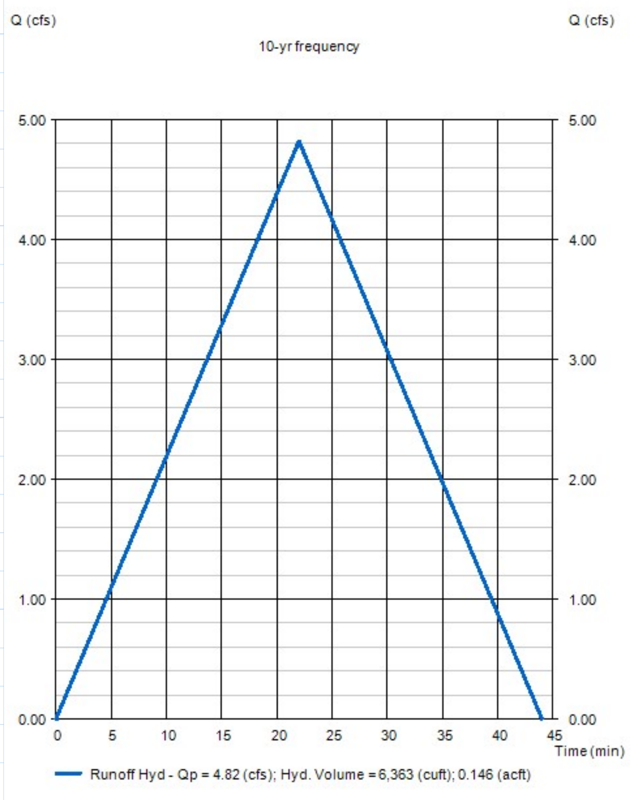
$$s := 6\ \frac{ft}{l} = 0.015$$

$$V_{oc} := \frac{1.49 \cdot r^{\frac{2}{3}} \cdot s^{0.5}}{n} = 0.758 \qquad V_{oc} := 0.641\ \frac{ft}{sec} = 0.641\ \frac{ft}{s}$$

$$T_o := \frac{l}{V_{oc}} = 10.725\ min$$

Time of Concentration:

$$T_c := T_s + T_{sc} + T_o = 22.37\ min$$



$$Q_{10y2} := 4.82\ \frac{ft^3}{sec} = 4.82\ \frac{1}{s} \cdot ft^3$$

$$V := 6363\ ft^3 = 0.146\ acre \cdot ft$$

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Runoff Analysis

Pre-Construction

Catchment Area Three:

Time of Concentration: (based on NRCS Lag Method - SUDAS Section 2B-3-D)

$A_3 := 137400\ ft^2 = 3.154\ acre \qquad A_3 := 3.154$

$l := 825\ ft \qquad l := 825$

$CN := 79$ (Table 2B - 4.03 Soils Group C)

$$S := \frac{1000}{CN} - 10 = 2.658$$

$$Y := \frac{12\ ft}{l} = 0.015\ ft \qquad Y := 2.3\%$$

$$t_c := \frac{100 \cdot l^{0.8} \cdot (S+1)^{0.7}}{1900 \cdot Y^{0.5}} = 185.283$$

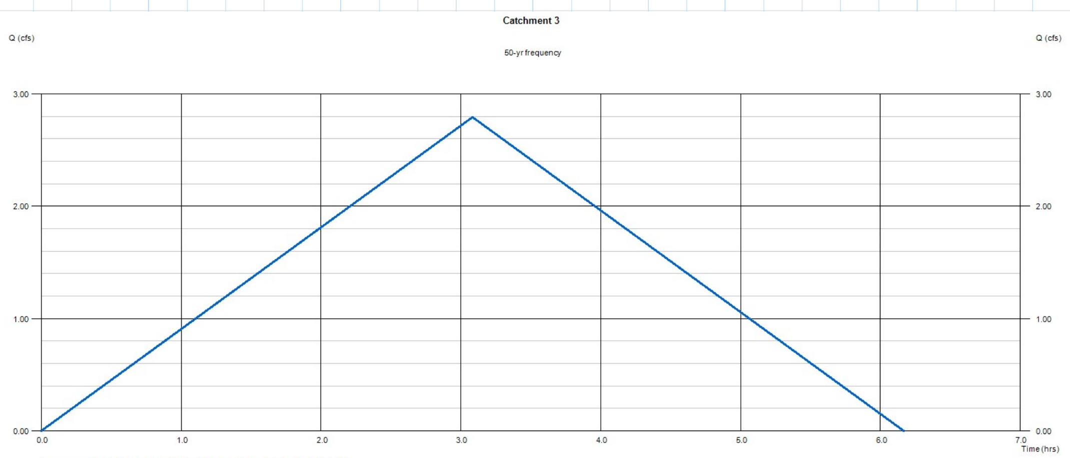
$$t_c := 185.3\ min = 3.088\ hr$$

Flowrate from Civil 3D:

$C_{runoff} := 0.6$ Table 2B-4.01 (SUDAS Design Manual)

$A_{impermeable} := 0\ ft^2 = 0\ acre \qquad C_{imper} := 0.97$

$Q_3 := 2.14\ \frac{ft^3}{s}$ based on 50-year return Period (Per Iowa DOT Design Guide)



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Runoff Analysis

Pre-Construction

NRCS Velocity Method:

(1.) Sheet Flow:

$n := 0.24$ (Dense Grass - SUDAS Table 2B-3.01)

$P_2 := 3.01$ (2-year, 24 hour rainfall (in) - Table 2B-2.04 SUDAS)

$l := 50$ (Initial flow of rainfall - no more than 100 ft) - units (ft)

$S := \frac{2}{l} = 0.04$ (Slope - ft/ft)

$$T_s := \frac{0.007 \cdot (n \cdot l)^{0.8}}{P_2^{0.5} \cdot Y^{0.4}} = 0.133$$

$$T_s := 0.179\ hr = 10.74\ min$$

(2.) Shallow Concentrated flow:

$l := 225\ ft$

$s := 6\ \frac{ft}{l} = 0.027$

$V_c := 6.962 \cdot (s)^{0.5}\ \frac{ft}{sec} = 1.137\ \frac{ft}{s}$ (ft/s) - Table 2B - 3.01 (Short - Grass Prairie)

$$T_{sc} := \frac{l}{V_c} = 3.298\ min$$

(3.) Open Channel Flow:

$n := 0.03$ (Manning Coeff. - Natural Stream #1)

Assumptions for 'r':

$d := 0.2\ ft$ $w := 0.2\ ft$ (Triangular Channel)

$a := 0.5\ w \cdot d = 0.02\ ft^2$ $P := 2 \cdot \left(\sqrt{(0.5\ w)^2 + d^2}\right) = 0.447\ ft$

$r := \frac{a}{P} = 0.045\ ft$ $r := 0.045$

Non-Commercial Use Only

Runoff Analysis

Pre-Construction

$l := 825\ ft - 225\ ft = 600\ ft$

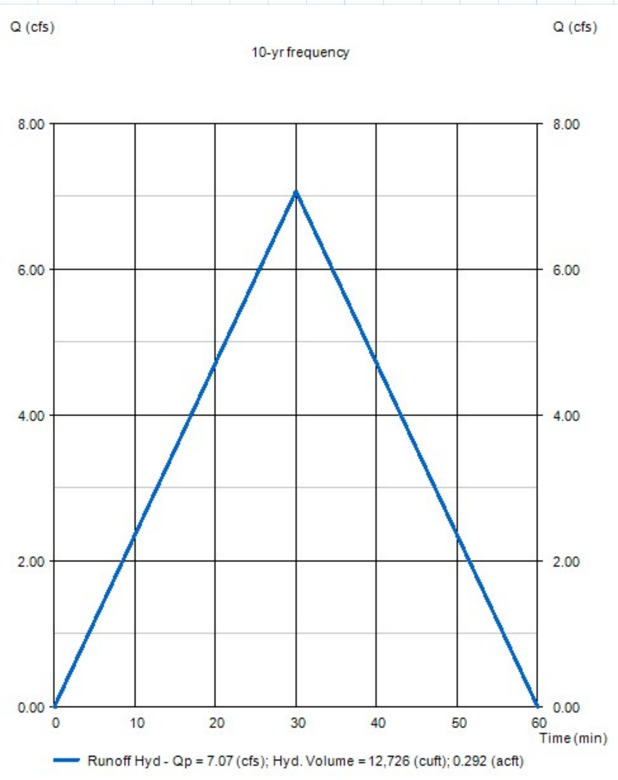
$s := 6\ \frac{ft}{l} = 0.01$

$V_{oc} := \frac{1.49 \cdot r^{\frac{2}{3}} \cdot s^{0.5}}{n} = 0.628$ $V_{oc} := 0.641\ \frac{ft}{sec} = 0.641\ \frac{ft}{s}$

$T_o := \frac{l}{V_{oc}} = 15.601\ min$

Time of Concentration:

$T_c := T_s + T_{sc} + T_o = 29.639\ min$



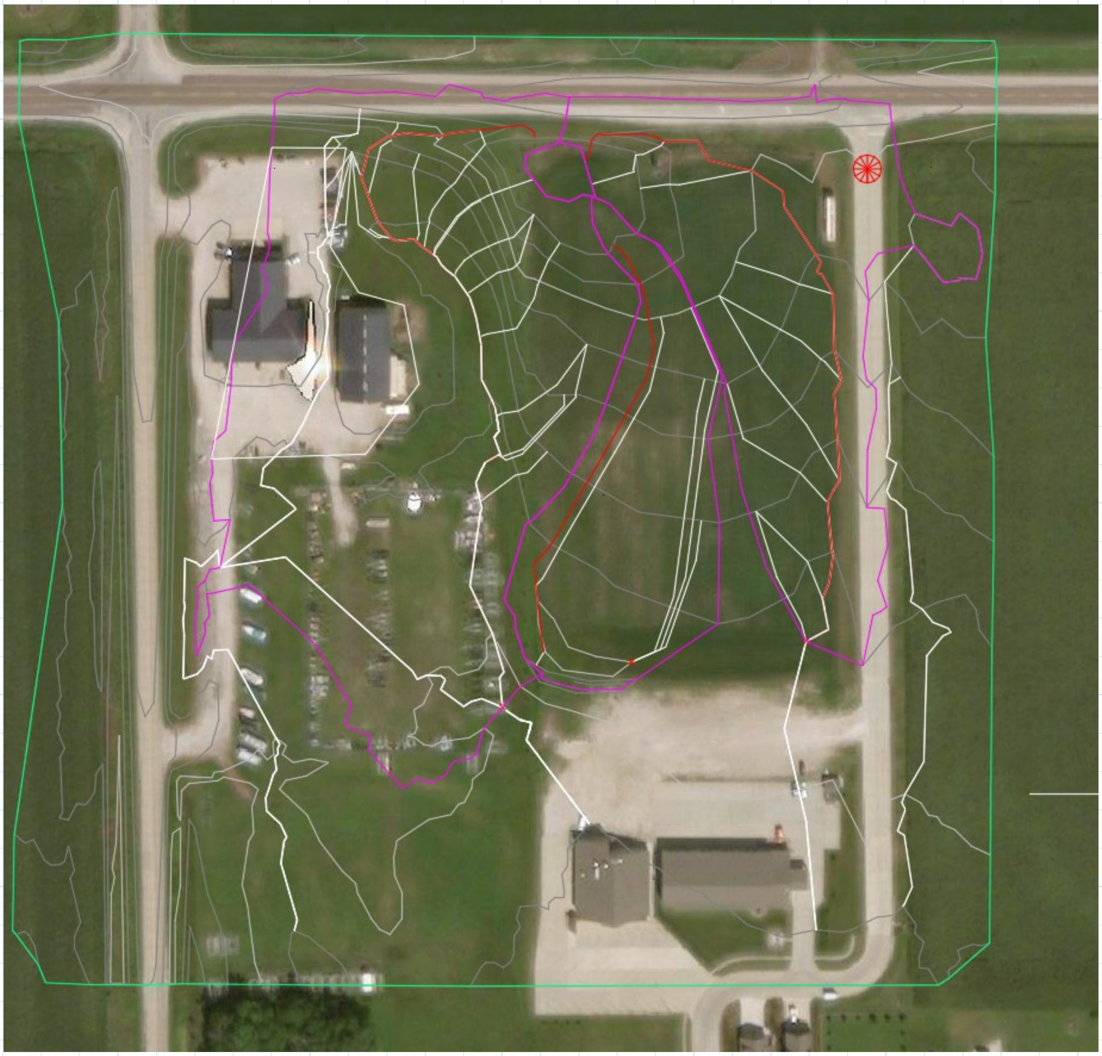
$Q_{10y3} := 7.07\ \frac{ft^3}{sec} = 7.07\ \frac{1}{s} \cdot ft^3$

$V := 12726\ ft^3 = 0.292\ acre \cdot ft$

Non-Commercial Use Only

Runoff Analysis

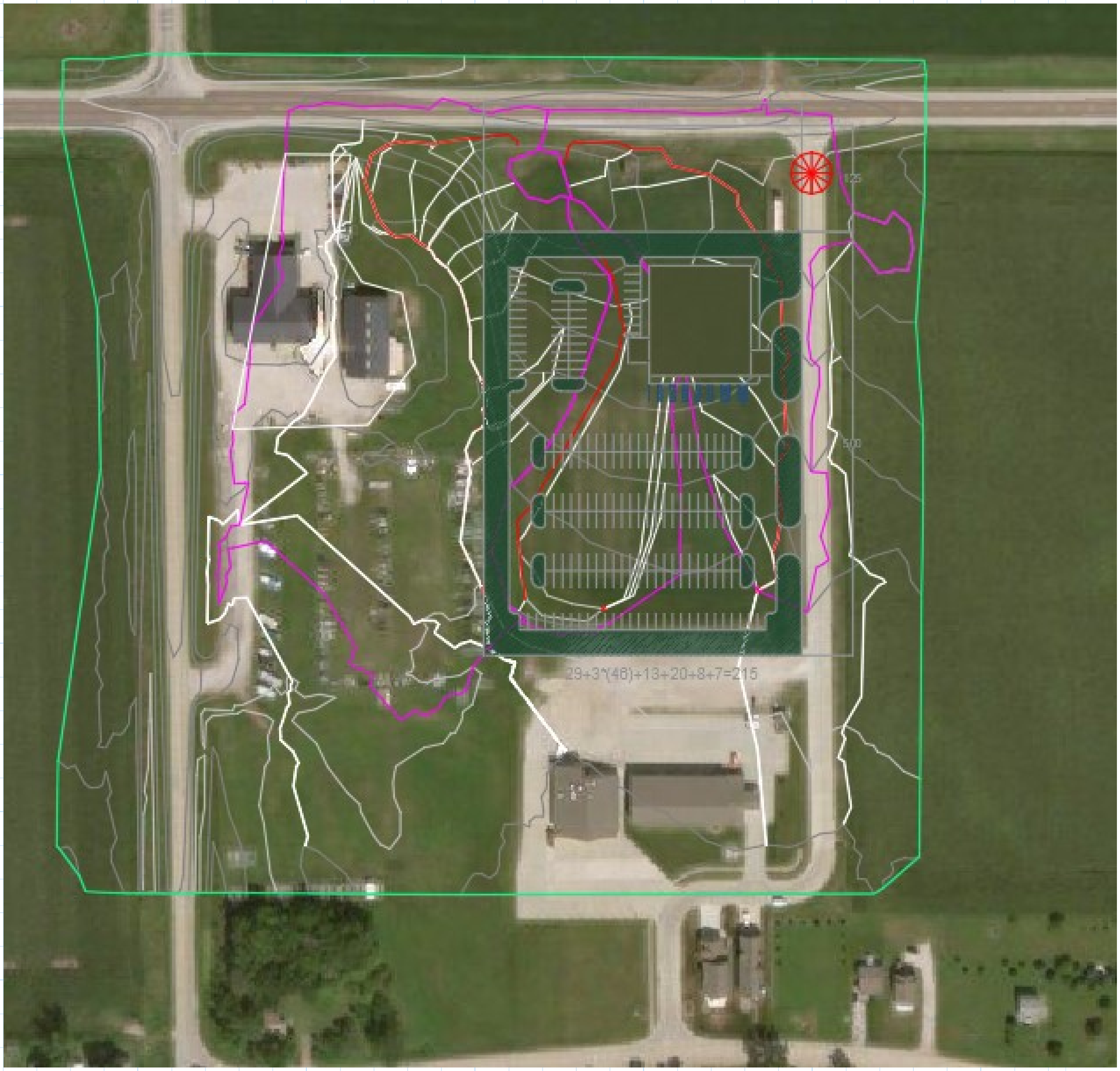
Pre-Construction



Non-Commercial Use Only

Runoff Analysis

Pre-Construction



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Post-Construction:

Eliminates Catchment Area 2 and 3 with self-containing site plan.

Before Site:

$$Q_{total} := Q_{10y1} + Q_{10y2} + Q_{10y3} = 26.45 \ \frac{ft^3}{s}$$

After Site:

$$Q_{total} := Q_{10y1} = 14.56 \ \frac{ft^3}{s}$$

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Appendix E – Bioswale & Infiltration Basin Calculations

Bioswale/Retention Cell Design

Compute the WQv Peak Runoff Rate:

$Depth := 1.25\ in$ (per the IaDNR)

$t_d := 24\ hr$ $CN := 96$ $C := 0.96$

$i_{wq} := \frac{Depth}{24\ hr} = 0.052\ \frac{in}{hr}$ $S := \frac{1000}{CN} - 10 = 0.417$ $S := 0.417\ in$

The required Water Quality Volume.

Drainage Areas: (Civil 3D)

Area (Lot One):

$A_1 := 19760\ ft^2 = 0.454\ acre$

Area (Building):

$A_B := 13500\ ft^2 = 0.31\ acre$

Area (Lot Two):

$A_2 := 45825\ ft^2 = 1.052\ acre$

Area (Lot Three):

$A_3 := 45375\ ft^2 = 1.042\ acre$

Area (North Lot):

$A_{North} := 36405\ ft^2 = 0.836\ acre$

WQv Storm Volume:

Volumetric Runoff Coeff.:

$R_v := 0.05 + 0.009 \cdot (95) = 0.905$ $CN := 98$

Water Quality Volume:

$WQ := R_v \cdot 1.25\ in = 1.131\ in$ $A_m := A_1 + A_B + A_{North} = 1.599\ acre$

$WQ_v := WQ \cdot A_m = 6567.378\ ft^3$ Volume for given storm event.

Volume of Swale must be greater than this volume.

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Peak Runoff for other Key Rainfall Events (24 hour Storm):
Based on Rational Method Analysis.

1 year Storm:

$i_1 := 2.66 \frac{in}{hr}$ $d_1 := 0.66\ in$

$Q1_1 := C \cdot i_1 \cdot A_1 = 1.168 \frac{ft^3}{s}$ $Q2_1 := C \cdot i_1 \cdot A_2 = 2.709 \frac{ft^3}{s}$

$Q3_1 := C \cdot i_1 \cdot A_3 = 2.682 \frac{ft^3}{s}$ $Q_{B1} := C \cdot i_1 \cdot A_B = 0.798 \frac{ft^3}{s}$

$Q_{N1} := C \cdot i_1 \cdot A_{North} = 2.152 \frac{ft^3}{s}$

2 year Storm:

$d_2 := 0.78\ in$ $i_2 := 3.14 \frac{in}{hr}$

$Q1_2 := C \cdot i_2 \cdot A_1 = 1.379 \frac{ft^3}{s}$ $Q2_2 := C \cdot i_2 \cdot A_2 = 3.198 \frac{ft^3}{s}$

$Q3_2 := C \cdot i_2 \cdot A_3 = 3.166 \frac{ft^3}{s}$ $Q_{B2} := C \cdot i_2 \cdot A_B = 0.942 \frac{ft^3}{s}$

$Q_{N2} := C \cdot i_2 \cdot A_{North} = 2.54 \frac{ft^3}{s}$

5 year Storm: $i_5 := 3.96 \frac{in}{hr}$ $d_5 := 0.99\ in$

$Q1_5 := C \cdot i_5 \cdot A_1 = 1.739 \frac{ft^3}{s}$ $Q2_5 := C \cdot i_5 \cdot A_2 = 4.033 \frac{ft^3}{s}$

$Q3_5 := C \cdot i_5 \cdot A_3 = 3.993 \frac{ft^3}{s}$ $Q_{B5} := C \cdot i_5 \cdot A_B = 1.188 \frac{ft^3}{s}$

$Q_{N5} := C \cdot i_5 \cdot A_{North} = 3.204 \frac{ft^3}{s}$

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10 year Storm: $i_{10} := 4.69\ \frac{in}{hr}$ $d_{10} := 1.17\ in$

$Q1_{10} := C \cdot i_{10} \cdot A_1 = 2.059\ \frac{ft^3}{s}$ $Q2_{10} := C \cdot i_{10} \cdot A_2 = 4.776\ \frac{ft^3}{s}$

$Q3_{10} := C \cdot i_{10} \cdot A_3 = 4.729\ \frac{ft^3}{s}$ $Q_{B10} := C \cdot i_{10} \cdot A_B = 1.407\ \frac{ft^3}{s}$

$$Q_{N10} := C \cdot i_{10} \cdot A_{North} = 3.794\ \frac{ft^3}{s}$$

25 year Storm: $i_{25} := 5.74\ \frac{in}{hr}$ $d_{25} := 1.43\ in$

$Q1_{25} := C \cdot i_{25} \cdot A_1 = 2.52\ \frac{ft^3}{s}$ $Q2_{25} := C \cdot i_{25} \cdot A_2 = 5.845\ \frac{ft^3}{s}$

$Q3_{25} := C \cdot i_{10} \cdot A_3 = 4.729\ \frac{ft^3}{s}$ $Q_{B25} := C \cdot i_{25} \cdot A_B = 1.722\ \frac{ft^3}{s}$

$$Q_{N25} := C \cdot i_{25} \cdot A_{North} = 4.644\ \frac{ft^3}{s}$$

100 year Storm: $i_{100} := 7.46\ \frac{in}{hr}$ $d_{100} := 1.86\ in$

$Q1_{100} := C \cdot i_{100} \cdot A_1 = 3.276\ \frac{ft^3}{s}$ $Q2_{100} := C \cdot i_{100} \cdot A_2 = 7.597\ \frac{ft^3}{s}$

$Q3_{100} := C \cdot i_{100} \cdot A_3 = 7.522\ \frac{ft^3}{s}$ $Q_{B100} := C \cdot i_{100} \cdot A_B = 2.238\ \frac{ft^3}{s}$

$$Q_{N100} := C \cdot i_{100} \cdot A_{North} = 6.035\ \frac{ft^3}{s}$$

System is Online. (BioSwale Design)

*Water is going directly from drainage area to swale.

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Location and Size Pretreatment Practices:

Grass Swale. Optimum Flow Velocity if <= 2.0 fps.

Channel Protection Volume:

$$A_m := A_B + A_1 + A_{North} = 0.002\ mi^2$$

$Q := 1.40\ in$ $\quad q_i := 2.95\ \frac{ft^3}{s}$ $\quad q_u := 1632\ \frac{ft^3}{s \cdot mi^2 \cdot in}$

$$q_{oqi} := 0.02$$

$$V_{svr} := 0.683 - 1.43\ (q_{oqi}) + 1.64 \cdot (q_{oqi})^2 - 0.804 \cdot (q_{oqi})^3 = 0.655$$

$$C_{pv} := V_{svr} \cdot Q \cdot A_m = 5323.97\ ft^3$$

Overbank Flood Protection Volume:

$$A_m := A_B + A_1 + A_{North} = 0.002\ mi^2$$

$Q := 4.071\ in$ $\quad q_i := 6.59\ \frac{ft^3}{s}$ $\quad q_u := 3641\ \frac{ft^3}{s \cdot mi^2 \cdot in}$

$$q_{oqi} := 0.02$$

$$V_{svr} := 0.683 - 1.43\ (q_{oqi}) + 1.64 \cdot (q_{oqi})^2 - 0.804 \cdot (q_{oqi})^3 = 0.655$$

$$O_{fpv} := V_{svr} \cdot Q \cdot A_m = 15481.344\ ft^3$$

If Swale Volume is larger than the above volume, this means it was be able to handle larger storms and these storms will not affect its structural integrity.

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Pretreatment Volume:

$$V := 0.1\ in \cdot A_m = 580.542\ ft^3$$

Forebay Volume. A possible pretreatment solution if deemed necessary. A forebay ponds water before it enters the Swale, hence a pretreatment measure.

Swale Dimensions:

$L := 530\ ft$ $\qquad slope := \frac{8\ ft}{L} = 1.509\%$ $\qquad d_{avg} := 12\ in$

$a := 0.01\ ft$

$$T_1 := 2\ ft \cdot \sqrt{\frac{d_{avg}}{a}} = 20\ ft$$

$$A_{flow1} := \frac{2}{3} \cdot T_1 \cdot d_{avg} = 13.333\ ft^2$$

$$V_{water} := A_{flow1} \cdot L = 7067\ ft^3$$

Flow volume based on a 12 inch depth of water, must be greater than the WQv volume in accordance with DNR Stormwater Manual.

Number of Check Dams:

$d_{max} := 18\ in$

$$spacing := \frac{d_{max}}{slope} = 99.375\ ft$$

Place every 100 ft

$$\frac{L}{100\ ft} = 5.3$$

6 Required. And will slow flow of water, allowing for more infiltration. Infiltration and velocity calculations to follow.

2-year and 25 year Velocity Check:

$n := 0.05$ $\qquad W := 24.5$ $\qquad S := 0.015$ $\qquad Q_2 := 3.43$ $\qquad Q_{25} := 6.59$

$$D_2 := \left(\frac{Q_2 \cdot n}{1.49 \cdot S^{0.5} \cdot W}\right)^{\frac{3}{5}} = 0.141$$

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$$V := \frac{Q_2}{W \cdot D_2} = 0.99 \qquad V_2 := 0.99 \ \frac{ft}{s}$$

$$D_{25} := \left(\frac{Q_{25} \cdot n}{1.49 \cdot S^{0.5} \cdot W} \right)^{\frac{3}{5}} = 0.209$$

$$V := \frac{Q_{25}}{W \cdot D_{25}} = 1.286 \qquad V_{25} := 1.286 \ \frac{ft}{s}$$

$$D_{25} := 0.209 \ ft$$

$$D_{swale} := d_{max} + 0.5 \ ft + D_{25} = 2.209 \ ft$$ above the 12 in WQv flow.

$$T := 2 \ ft \cdot \sqrt{\frac{D_{swale} - D_{25}}{a}} = 28.284 \ ft$$

$$A_{flow} := \frac{2}{3} \cdot T \cdot \left(D_{swale} - D_{25} \right) = 37.712 \ ft^2$$

$$V_{swale} := L \cdot A_{flow} = 19987.552 \ ft^3$$ Total Swale Volume.

Checking Infiltration Rate/Time:

$$k := 1.0 \ \frac{in}{hr}$$

$$Q_{inf} := k \cdot L \cdot T_1 = 0.245 \ \frac{ft^3}{s}$$

$$time := \frac{A_{flow1} \cdot L}{Q_{inf}} = 8 \ hr$$

Less than the required 24 hours for WQv event.

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Retention Cell Design: (Infiltration Basin)

$A_{lot2} := A_2 = 1.052\ acre$ $A_{lot3} := A_3 = 1.042\ acre$

WQv Calculations:

Will only take runoff from the lower section of site (majority of parking lot)

$R_v := 0.96$

$WQ := R_v \cdot 1.25\ in$ $A_i := A_{lot2} + A_{lot3} = 2.094\ acre$

$WQ_v := WQ \cdot A_i = 9120\ ft^3$

Other Rainfall Events: (NRCS TR-55 Per Iowa DNR)

$Q_1 := 5.34\ \frac{ft^3}{s}$ 1 year

$Q_2 := 6.19\ \frac{ft^3}{s}$ 2 year

$Q_5 := 7.82\ \frac{ft^3}{s}$ 5 year

$Q_{10} := 9.40\ \frac{ft^3}{s}$ 10 year

$Q_{25} := 11.91\ \frac{ft^3}{s}$ 25 year

Again, Online System

$S := 0.25\ in$ based on Sand type

Table C2-S1- 4: Soil specific recharge factors

Hydrologic Soil Group	Average Annual Recharge Volume (in/yr)	Soil Specific Recharge Factor (S)
A-Sandy	18	0.51
B-Silty	12	0.34
C-Clayey	6	0.17
D-Clayey	3	0.08

$R_{ev} := A_i \cdot S \cdot R_v = 1824\ ft^3$

S chosen based on C and D soil groups (Clarion Loam)

Max. Allowable depth:

$f := 1.02\ \frac{in}{hr}$ $T_p := 72\ hr$

$d_{max} := f \cdot T_p = 6.12\ ft$

$d_{max} := 4\ ft$

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Channel Protection Volume:

$Q := 1.870\ in \qquad q_i := 2.68\ \frac{ft^3}{s} \qquad q_u := 1632\ \frac{ft^3}{s \cdot mi^2 \cdot in}$

$q_{oqi} := 0.02$

$V_{svr} := 0.683 - 1.43\ (q_{oqi}) + 1.64 \cdot (q_{oqi})^2 - 0.804 \cdot (q_{oqi})^3 = 0.655$

$C_{pv} := V_{svr} \cdot Q \cdot A_i = 9309.564\ ft^3$

Overbank Flood Protection Volume:

$Q := 5.04\ in \qquad q_i := 11.91\ \frac{ft^3}{s} \qquad q_u := 3641\ \frac{ft^3}{s \cdot mi^2 \cdot in}$

$q_{oqi} := 0.02$

$V_{svr} := 0.683 - 1.43\ (q_{oqi}) + 1.64 \cdot (q_{oqi})^2 - 0.804 \cdot (q_{oqi})^3 = 0.655$

$O_{fpv} := V_{svr} \cdot Q \cdot A_i = 25091.019\ ft^3$

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Assuming Trapezoidal Basin:

$L := 375\ ft$ $W := 37.5\ ft$ $m := 3$ $\frac{L}{W} = 10$

$L_b := L - 2 \cdot m \cdot d_{max} = 351\ ft$

$W_b := W - 2 \cdot m \cdot d_{max} = 13.5\ ft$

Dimensions based on remaining area of site.

$$V := \frac{(L \cdot W + L_b \cdot W_b) \cdot d_{max}}{2} = 37602\ ft^3$$

Total Volume of Basin.

$d_b := 24\ in$

$$V := \frac{(L \cdot W + L_b \cdot W_b) \cdot d_b}{2} = 18801\ ft^3$$

$$T_p := \frac{d_b}{f} = 23.529\ hr$$

Under the 48 hour infiltration requirement for 24 inch depth. 24 inches is substantially larger than the WQv volume, meaning the WQv event will infiltrate in less than a day.

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Appendix F – Detailed Cost Estimate

RSMeans Cost Data (TVM)

Category	Item	2011			2020 (3% Const Cost Inflation)		
		Material	Installation	Total	Material	Installation	Total
Concrete	Strip Footing	$20,500	$18,500	$39,000	$26,700	$24,100	$50,900
	Foundation Wall	$27,000	$45,500	$72,500	$35,200	$59,400	$94,600
	6" Slab on grade (reinforced)	$75,600	$62,200	$137,800	$98,600	$81,200	$179,800
Wood Framing and Structure	Exterior Walls (inc. Siding and drywall)	$32,000	$39,000	$71,000	$41,800	$50,900	$92,600
	Interior Walls (inc. drywall)	$10,500	$24,000	$34,500	$13,700	$31,300	$45,000
	Windows	$2,000	$1,000	$3,000	$2,600	$1,300	$3,900
	Doors	$49,000	$15,000	$64,000	$63,900	$19,600	$83,500
	Drywall Ceiling	$11,000	$37,000	$48,000	$14,400	$48,300	$62,600
	Wood Trusses	$66,000	$31,000	$97,000	$86,100	$40,400	$126,600
	Roofing Material (Asphalt Shingles)	$18,500	$18,000	$36,500	$24,100	$23,500	$47,600
Mechanical	70 Ton A/C Unit	$85,000	$53,000	$138,000	$110,900	$69,200	$180,100
	Heating Unit	$58,000	$51,500	$109,500	$75,700	$67,200	$142,900
Plumbing & Electrical	9 Water Closets (3 Men, 6 Womens)	$7,000	$5,500	$12,500	$9,100	$7,200	$16,300
	2 Urinals	$2,500	$1,500	$4,000	$3,300	$2,000	$5,200
	6 Lavatories (3 & 3)	$3,600	$3,300	$6,900	$4,700	$4,300	$9,000
	Kitchen Sink/Counters	$2,000	$1,000	$3,000	$2,600	$1,300	$3,900
	Light Fixtures	$60,000	$73,000	$133,000	$78,300	$95,200	$173,500
Finishes	Painting	$4,500	$14,000	$18,500	$5,900	$18,300	$24,100
	Wood floor	$38,000	$24,000	$62,000	$49,600	$31,300	$80,900
Green Design	Seeded Areas	$30,000	$18,000	$48,000	$39,100	$23,500	$62,600
	Subdrainage Piping	$2,400	$1,000	$3,400	$3,100	$1,300	$4,400
	Total	$610,000	$540,000	$1,140,000	$790,000	$700,000	$1,490,000

Appendix G – Gantt Chart

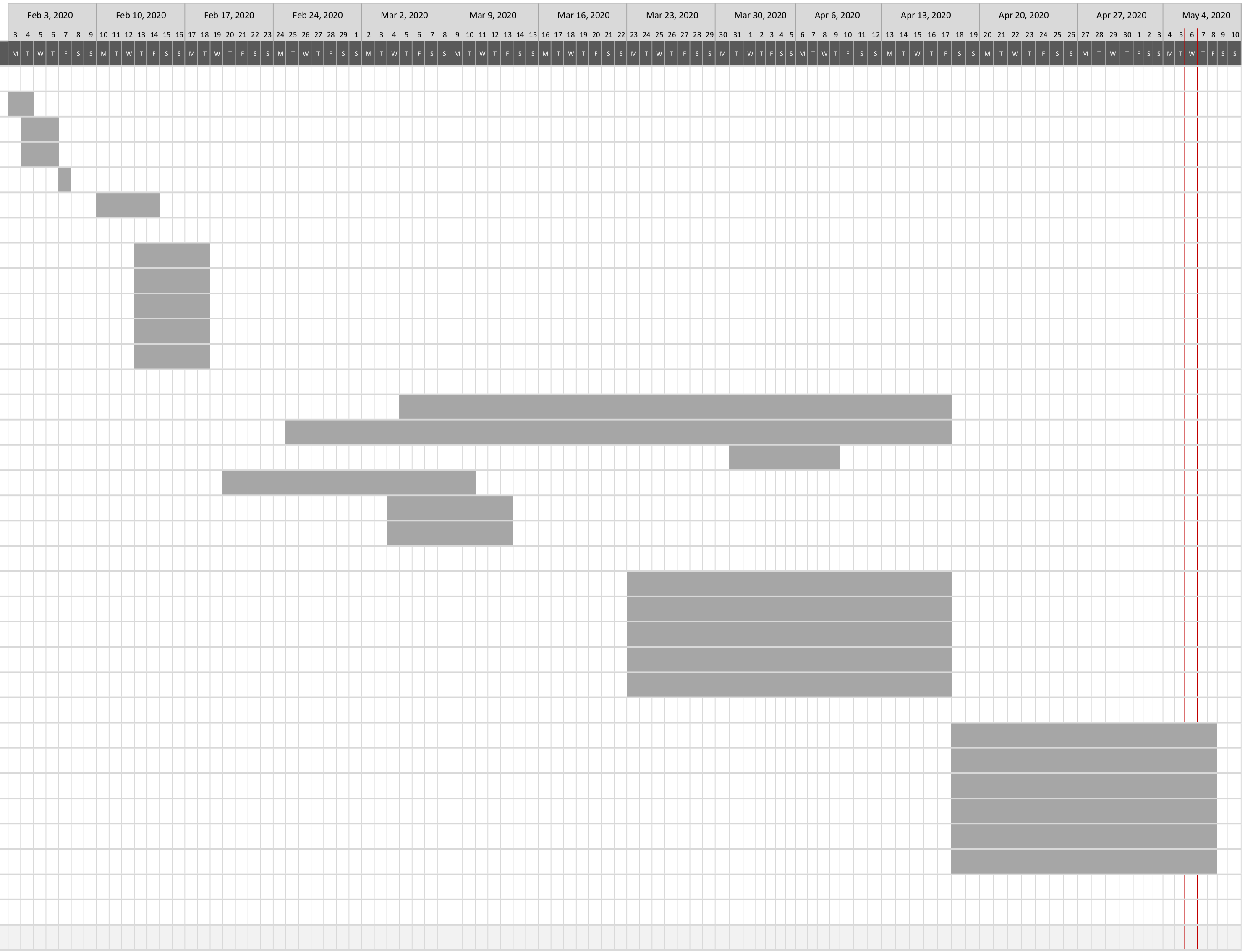
LAKE VIEW COMMUNITY CENTER

IWFD Engineering

Project Start: Mon, 2/3/2020

Display Week: 1

TASK	ASSIGNED TO	PROGRESS	START	END
Phase 1: Design Proposal				
Alternate Solutions		100%	2/3/20	2/4/20
Proposal Report Draft		100%	2/4/20	2/6/20
Proposal Presentation Draft		100%	2/4/20	2/6/20
Proposal Report/Presentation		100%	2/7/20	2/7/20
Proposal Presentation		100%	2/10/20	2/14/20
Phase 2: Design Development				
Site Arrangement		100%	2/13/20	2/18/20
Foundations		100%	2/13/20	2/18/20
Structural Options		100%	2/13/20	2/18/20
Parking Lot Development		100%	2/13/20	2/18/20
Hydrology Options		100%	2/13/20	2/18/20
Phase 3: Design Analysis				
Structural System Analysis		100%	3/5/20	4/17/20
Roof Analysis		100%	2/25/20	4/17/20
Foundations Analysis		100%	3/31/20	4/9/20
Site Design		100%	2/20/20	3/10/20
Runoff Analysis		100%	3/4/20	3/13/20
Pipe/Open Channel Flow Analysis		100%	3/4/20	3/13/20
Phase 4: Design Draft				
3D Rendering Generation		100%	3/23/20	4/17/20
Plan Generation		100%	3/23/20	4/17/20
Report Generation		100%	3/23/20	4/17/20
Presentation Creation		100%	3/23/20	4/17/20
Poster Creation		100%	3/23/20	4/17/20
Phase 5: Final Design				
3D Rendering Finalization		100%	4/18/20	5/8/20
Final Plan Set		100%	4/18/20	5/8/20
Final Calculations		100%	4/18/20	5/8/20
Final Report		100%	4/18/20	5/8/20
Presentation Creation		100%	4/18/20	5/8/20
Poster Creation		100%	4/18/20	5/8/20
Insert new rows ABOVE this one				



Appendix H – References

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