Stormwater Clarifying Basin Design for the University of Florida

AQUA MACHINA

Prepared for:
The University of Florida
Alachua County, Florida

Submitted to:
Water Environment Federation's Technical Exhibit and Conference
2024 Student Design Competition

Prepared by:
University of Florida
Capstone Design Team Aqua Machina
Maya Patel, Bianca Piñeros, Rita Hippe, Morgan Meuleman,
Olivia Bergmann, Ross Nygard, Katherine Palmer, & Anthony Trow

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Abstract

Aqua Machina's scope encompasses design of a clarification system receiving loads from a representative paved urban land use; a stormwater management condition found commonly across Florida and the USA. Considering Florida's environmental resources and the 2024 Florida Clean Waterways Act, designs are based on load reduction for nutrients (total nitrogen, TN; total phosphorous, TP) and particulate matter (PM). Four clarifier design alternatives are examined: (A1) Regulatory Presumptive Guidance, (A2) No Baffles, (A3) Baffled, and (A4) Baffles Optimized with Artificial Intelligence (AI). Alternatives were developed using a database of rainfall-runoff from an impervious University of Florida (the client) catchment. Using a unit operations approach and AI (machine learning algorithms), the team examined clarifier designs to minimize resource expenditures while achieving load reduction requirements. This project supports the client's AI initiatives and 2020-2030 Campus Master Plan stormwater goals, while addressing public outreach and education. With AI, the project enhances the potential for optimizing stormwater treatment. An additional extensibility study is presented to demonstrate portability of our design to different environmental conditions. This consisted of two additional design alternatives of (XA1) Underground Baffled Basin and (XA2) Permeable Pavement for a similar paved urban land use of similar geometrics in New Orleans, Louisiana.

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Mitch Griffin, PE, Jacobs Engineering	Joe Aufmuth, UF Geospatial Information Librarian
Sandra St Clair, UF ESSIE Admin	Alexander Miller, PE, HDR
Gordy, Mochi, Miso, and Miss Lady; Our Team Mascots of Feline Descent	

Thank you!

PROJECT TEAM

AQUA MACHINA

Rita Hippe Water/Wastewater Engineer I at Hazen and Sawyer
Team Lead

Project Responsibilities:
Primary project and team manager, assisted with programming, design computations, and report editing

Relevant Experience:
Water engineering consulting experience, specifically in pilot studies for water treatment facilities and septic-to-sewer projects

Katherine Palmer Civil Analyst at Kimley-Horn & Working towards a M.S. in Water Systems
Economic Analyst

Project Responsibilities:
Primary economic feasibility and cost estimation specialist, assisted with project and team management

Relevant Experience:
Wastewater engineering consulting experience, specifically in modeling, septic-to-sewer projects, lift stations, and pipelines

Ross Nygard Environmental Engineering Sciences, 4th year
Computation Lead

Project Responsibilities:
Primary programmer and design computations expert, assisted with technical report editing

Relevant Experience:
Machine learning model development in R and MATLAB, hydrologic modelling in HEC-HMS

Olivia Bergmann Water/Wastewater Designer at HDR
Visual Report Curator

Project Responsibilities:
Primary author of visual report, assisted with GIS data collection, analysis, and the extensibility study

Relevant Experience:
2 years experience water engineering, 1 year experience data analysis, modeling and machine learning in R

PROJECT TEAM

AQUA MACHINA

Maya Patel Environmental Engineering Sciences, Grad Student
Technical Writing Lead

Project Responsibilities:
Primary author of technical report, assisted with visual report delivery and client communications

Relevant Experience:
3+ years water/wastewater consulting, UF ASCE Concrete Canoe visual design captain & chapter president

Anthony Trow Associate Staff Professional at SCS Engineers
Geographic Information Systems Specialist

Project Responsibilities:
Primary GIS data analyst, assisted with information acquisition, communications, design and calculations

Relevant Experience:
4 years experience QA/QC and management, 3 years experience programming in MATLAB and R

Bianca Pineros Environmental Engineering Sciences, 5th year
Information Acquisition Officer

Project Responsibilities:
Primary point of contact for stakeholders and other relevant parties, assisted with regulatory and legal analyses, and graphics

Relevant Experience:
Water resource engineering consulting experience, Superfund and EPA community involvement coordinating, UF SHPE secretary

Morgan Meuleman Water/Wastewater Engineer I at Mead & Hunt
Legal & Regulatory Specialist

Project Responsibilities:
Primary analyst of regulatory compliance and legal resource materials, assisted with report editing and economic analyses

Relevant Experience:
Water/Wastewater engineering consulting experience, specifically septi-to-sewer projects and I&I studies

Introduction

Project Background

The Campus Master Plan has served as the principle guiding document for land development on the University of Florida (UF) main campus and 13 additional UF properties since its first issue, the 1995-2000 Master Plan, was completed. The General Infrastructure section of the 2020-2030 UF Campus Master Plan outlines the goals, objectives, and policies which will be used to guide the design of construction and maintenance projects for stormwater and other utility systems on campus. Enhancing water quality and drainage conditions through the maintenance of an economical and sustainable stormwater system is the primary stormwater goal of the University (UF, 2020). Objectives 1.3 and 1.5, which address sedimentation, water quality, and community involvement aspects of stormwater infrastructure, align the implementation of a stormwater clarification pond with the long-term development plans of the University (UF, 2020).

Project Location

The paved surface parking area serving the Reitz Union Student Center is on UF campus in Gainesville, Florida. Comprised of 2,000 acres and over 900 buildings, UF makes up almost 5% of the total area of the City of Gainesville, Florida (UF, 2024a). Alachua county, of which Gainesville is the largest city and county seat, is one of 18 counties which are covered in whole or in part by the St. Johns River Water Management District (SJRWMD) (City of Gainesville, 2024; SJRWMD, 2024a). Figure 1 highlights that the project location is within the Lake Alice watershed and the proximity to the waterbody.

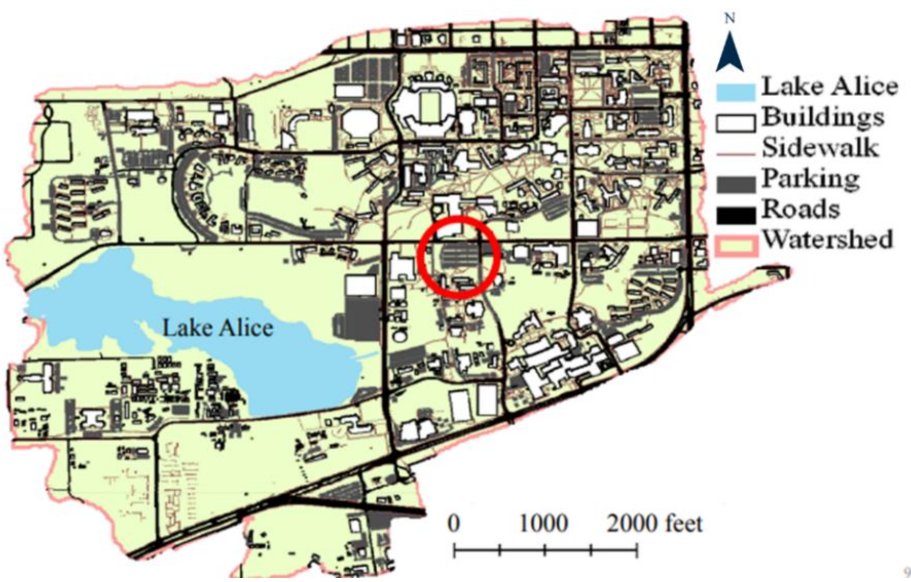

Figure 1: Vicinity Map Locating Study Site (Kertesz et al, 2009).

Problem Statement

More than 60% of the UF main campus lies within the Lake Alice watershed, named after the most prominent waterbody and recreational crown jewel of the campus. Lake Alice provides aesthetic and cultural services to the community with its popular trails and serene scenery, that serves as a habitat for diverse local wildlife, and provides significant groundwater recharge. (UF, 2016a, 2016c). Lake Alice receives nitrogen (N), phosphorous (P), and particulate matter (PM) loads from the impervious and vegetated surfaces throughout the watershed. The high level of urbanization present within the Lake Alice watershed and historically limited stormwater treatment infrastructure guidance for the UF campus pose a risk to the water chemistry of the lake. Algal blooms and flooding occur, however, there is a degree of improvement as the stormwater infrastructure on campus is updated. Stormwater treatment will improve water chemistry, reduce risks associated with high peak flows, flooding, and provide an opportunity for increased community education and engagement. The paved surface parking serving the Reitz Union is the selected project site to implement a stormwater treatment basin design.

Project Constraints

The basin location and dimensions are geometrically constrained within the limited grass-covered area directly adjacent to the paved surface parking without requiring a reduction in the available parking area. The majority of construction should be completed during the summer academic period (mid-May through early August), when foot-traffic on and around the site is significantly reduced in comparison to the Fall and Spring semesters. The client has indicated that disturbance, demolition, or realignment of the access road to the Phelps Lab should be avoided if possible. Additionally, the client requests that the surface parking remain operational at 100% capacity during construction.

Project Objectives

The objectives of the proposed design project align with the 2020-2030 UF Campus Master Plan Stormwater objectives. The primary objective is to improve and manage the chemistry (loads) and stormwater hydrograph discharges generated from the paved surface parking and transported to Lake Alice (UF, 2020). The design of a stormwater clarification basin which is optimized to balance cost and treatment efficiency is proposed to achieve this goal. The secondary objective is to provide community outreach and education opportunities throughout the construction process and as an aspect of the final product (UF, 2020). This goal will be achieved through regular community involvement opportunities during the project timeline and the inclusion of informational materials such as placards and signage in the completed project site.

Existing Conditions

Site Evaluation and Design Constraints

The site location is the Reitz Union Parking Area and the grass-covered strip to its west. This site location shown in Figure 2 is owned by UF and is the micro-watershed contributing to the stormwater the project aims to treat prior to conveyance downstream. The surface parking area is a total of 3.23 acres and consists of approximately 76% impervious area and approximately 24% pervious area (Kertesz et al., 2009). Refer to Table 1 for more quantitative data pertaining to the project site. Because UF is a state university, the institution's ownership of this land is an extension of the State of Florida, making the campus public land.

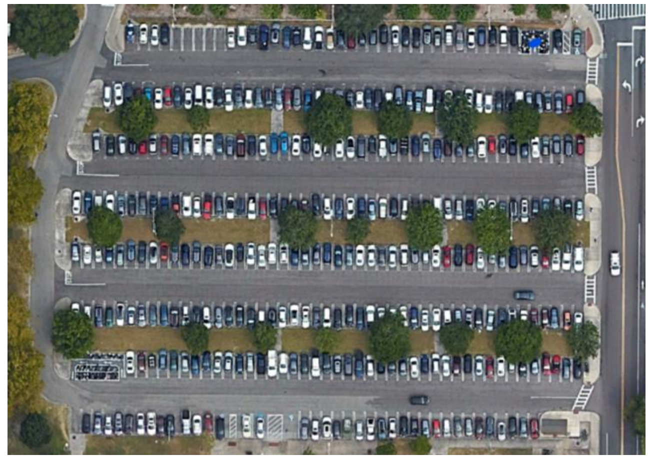

Figure 2: Watershed contributing stormwater this project aims to treat (Florida Marine Research Institute, n.d.).

Table 1: Existing Site Conditions (ESRI; Kertesz et al., 2009; United States Department of Agriculture [USDA], 1986).

Site Conditions	
Watershed Area	3.23 acres
Total % Impervious	75.61%
Time to Concentration	0.86 mins (Top) 0.99 mins (Bottom)
Curve Number (CN)	39
East - West Slope	3%
North - South Slope	1.5%
Traffic	700 vehicles/day
Mean Volatile Particulate Matter (PM) Fraction in Runoff	48% (30% - 61%)
Runoff pH	6.4 - 8.6

The parking area is in the heart of UF campus, bordering roads that provide access to the University Welcome Center and several academic buildings. Due to the site's central location, it is classified as a high-traffic area.

For this project, stormwater control concerns are limited to stormwater produced within the paved parking area. The elevated vegetated islands that drain to the paved parking area surface are a major cause of the higher nutrient loading on this site. This biogenic material is mobilized and transported to the paved surface parking area during storm events. All stormwater within the parking area is transported through existing Florida Department of Transportation (FDOT) catch basins and conveyance pipes that drain to Lake Alice without treatment. This is representative of stormwater collected from nearly 77% of UF's over 400 storm drains (UF, 2016b). See Appendix A: Site Specifics for a more detailed description of existing site conditions.

Pre-Design Hydrology Analysis

Incorporating pertinent geographic datasets such as elevation, soil, hydrologic, land cover, and pavement cover layers within the Lake Alice watershed were utilized to examine the hydrologic response of the watershed (see Appendix A: Site Specifics). The elevation of Lake Alice is approximately 78 feet above sea level, while the surface parking area is approximately 127 feet above sea level (ESRI; USDA, 2023a, 2023b). The elevation gradient is from the project location, down towards Lake Alice, which serves as the drainage basin of the Lake Alice watershed. Topographical data and related hydrologic system attributes were determined using data from the Florida Geographic Data Library (FGDL, 2008) and the Geo-Facilities Planning and Information Research Center.

Within the surface parking area, the pavement is sloped towards the proposed basin project location, providing the necessary hydraulic gradient to drain by gravity (Kertesz et al., 2009) to the proposed basin location. Appendix A: Site Specifics shows the topography of the surface parking area which will contribute stormwater to the proposed basin project site, which was determined using shapefiles and conducting spatial analysis within AutoCAD to model the process flow (AutoDesk).

Legal and Regulatory

To determine the treatment goals of this project, research of applicable regulations was conducted, as shown below in Table 2. The design parameters are based on the designation and associated treatment requirements for the Lake Alice waterbody maintained by UF.

Table 2: Applicable Stormwater Regulations (Code of Federal Regulations 1983; Ellard, 2015; Environmental Protection Agency [EPA], 2023a, 2023b, 2023c, 2024; Florida Department of Environmental Protection [FDEP], 2022, 2023b, 2024a, 2024c, 2024d; Olexa et al.,

Regulation and Agency	Description
EPA Clean Water Act (CWA) of 1972	Regulates the discharge of constituents into U.S. waterways.
EPA National Pollutant Discharge Elimination System (NPDES) of 1972	Permit program created by Clean Water Act that regulates pollution of U.S. water bodies by point sources.
EPA NPDES Stormwater of 1987	Set precedent on treating effluent from Municipal Separate Storm Sewer Systems (MS4s), a non-point source. Released in two phases.
EPA NPDES Stormwater Phase 1 of 1990	The first phase addresses stormwater from “medium” and “large” MS4s.
EPA NPDES Stormwater Phase 2 of 1999	The second phase addresses stormwater from “small” MS4s, which includes the City of Gainesville.
FDEP Clean Waterways Act (CWA) of 2024	Provided an update to stormwater design and operation regulations aimed at minimizing the impact of known sources of nutrient discharges.

FDEP Florida Watershed Restoration Act (FWRA) of 1999	Specifies how the TMDL program should be implemented in Florida.
FDEP Total Maximum Daily Loads (TMDLs) of 1992	Maximum amount of a given constituent that a surface water can accept while still meeting water quality standards.
FDEP Basin Management Action Plan (BMAP) of 1999	The plan set forth to meet the TMDLs for an impaired water body through current and future projects with the assistance of local input.
FDEP Florida Impaired Waters Rule (IWR) of 2001	Establishes a methodology to identify waters that will be included on the State's Clean Water Act Section 303(d) list of impaired waters and therefore requires the development of a TMDL.

All applicable rules and regulations pertaining to the project were reviewed and compiled by the Aqua Machina team to ensure the outgoing water quality in each design alternative would meet the corresponding regulation. The final net improvement criteria determined for our site was an 80% load reduction in influent constituents, from FDEP's Clean Waterways Act of 2024 (S.B. 7040, 2024). Reference Appendix B: Regulatory and Risk for a timeline with more detailed information about each of the identified regulations and a chart that demonstrates the process, which was implemented to identify the classification of Lake Alice, and thus which criteria were necessary to follow in this project's design.

Design Alternatives

Design Theory

Partitioning of nutrients and chemicals to particulate matter (PM), specifically nitrogen (N) and phosphorus (P) in this case, determined treatment design methodology selection for the development of the proposed designs, as nutrient loading reduction is the primary project goal. For this site, a significant fraction of the nutrient loadings in the stormwater are partitioned to PM, whether as the source biogenic matter or as inorganic detritus and grit from the surface parking area. Clarification through particle settling, the application of sedimentation using Newton's Law of Settling, was a primary design consideration in the development of the proposed alternatives. Particle size and specific gravity influence the settling behavior of particulate matter, in addition to flow characteristics such as surface overflow rate and to a lesser extent temperature (FDOT, 2021). The design alternatives (A1) Regulatory Presumptive Guidance, (A2) No Baffles, (A3) Baffled, and (A4) Baffled: AI Optimized, were all developed

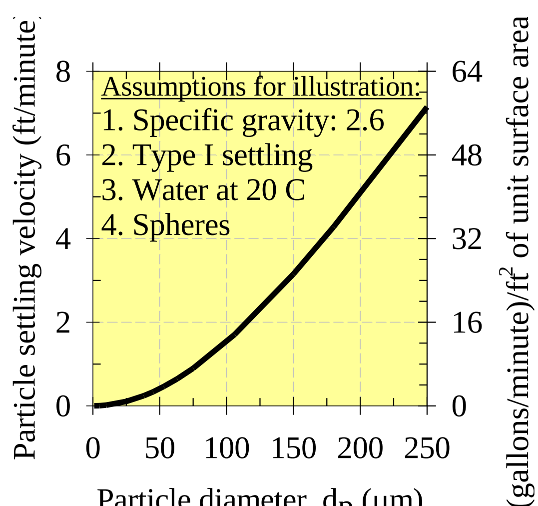

Figure 3: Settling Velocities of PM.

based on the principles of particle mechanics applied to clarifier sizing to optimize particle and nutrient removal. Figure 3 shows the settling velocity distribution of PM. The generated settling velocity curve is shown for illustration and will vary based on specific gravity, water temperature and particle shape.

An additional portion of total nitrogen (TN) load reduction was accounted for by a filtering mechanism. The removal efficiency increases with a decrease in media diameter but at a cost of increased head loss (Liu et al., 2010). The head loss associated with the addition of these filters was calculated using the Ergun Equation (Equation 14) (each filter cartridge is designed with a height of 1.85 feet and a diameter of 1.5 feet and was determined to not have a significant effect on the overall design. Another parameter considered was the filter surface loading rate (SLR), which was used to design the sizing and number of filters used for each design (Liu et al., 2010). Reference Appendix D: Design for further filter design specifications.

For design storm calculations, the use of both National Resources Conservation Service (NRCS) design storms and site-specific historical storm event data, were utilized. Hydrograph analysis, particularly with the Soil Conservation Service Hydrograph Type 2 assumption, provides insights into historical storm behavior versus design storm parameters (see Appendix C: Design Storm Development for more details). Lumped Level Pool routing methodology was used for basin design and storage considerations, to manage peak flows and needed storage volume of the basin. Analysis performed using the historical site-specific storm data, including watershed characteristics, facilitated the calculation of peak flow using the Rational Method equation (Equation 2).

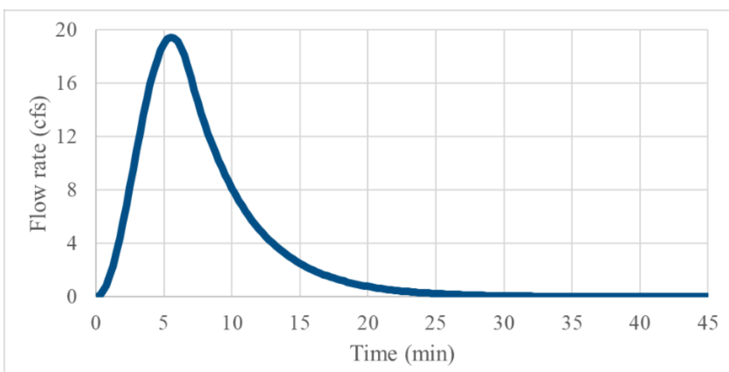

Figure 4: Design storm direct runoff hydrograph.

These historical storms are not a signature of a 25-year storm generally accepted by SJRWMD, however Water Management Districts will allow the mean annual storm to be implemented for load reduction design. Therefore, the Atlas 14 tool, created by the National Ocean and Atmospheric Administration (NOAA) (Perica, et al., 2013), was used to identify the intensity for a 25-year storm according to Equation 1. The 25-year, 10-minute storm was chosen, due primarily to the rapid conveyance time (time of concentration) of the surface parking area which was generally less than 10 minutes for the historical events. The design storm resulted in the hydrograph shown in Figure 4. The design storm was used for minimum storage requirements in the settling zone, while the historical storms ultimately mainly guided load reduction calculations.

Treatment Chemistry

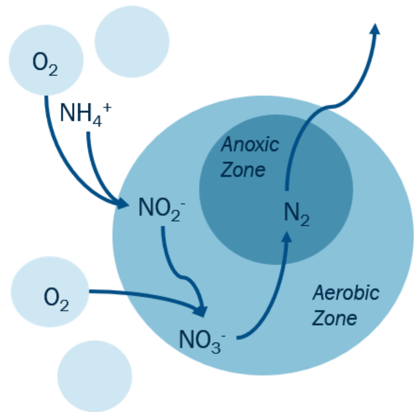

Figure 5: Nitrification and Denitrification of Sludge (Hazard et al., 2018).

For improved nitrogen removal, each of the proposed alternatives includes a 2-foot depth sludge collection zone. This 2-foot deep collection zone at the bottom of each basin design alternative allows for the accumulation of settled PM without compromising the basin's treatment volume storage or producing an insufficient residence time for settling. This sludge zone will go anoxic within 48 hours (Liu et al., 2010) as the PM accumulates and the organic matter, consortium of micro-organisms and nutrients facilitate denitrification to further reduce the nitrogen (as nitrate) that has partitioned into the water column from TN partitioned to PM. Figure 5 demonstrates this process. Several gravity-driven media radial cartridge filters (RCF) approximately 1.8 feet in height and 1.5 feet in diameter will also provide separation of PM-bound N and P. Each RCF will contain an aluminum oxide coated media (AOCM) crushed to a diameter of 1 mm. The aluminum oxide coating functions as additional texture to increase the surface area, which promotes the capturing of finer PM and the N and P bound with this PM (Ordonez et al., 2020).

Phosphorus removal efficiency in alternatives A3 and A4, in addition to removal via PM settling, is supported through the implementation of carbonated recycled concrete (CRC) within the gabion baskets used to construct the clarifier baffle walls. Crushed CRC, based on adsorption equilibrium isotherm findings, provides ample surface area and the proper alkalinity for phosphorous deposition and adsorption (Wu, 2013). Thus, implementing gabion basket baffles over standard concrete baffles provides an added nutrient reduction mechanism. The AOCM contained in the RCFs provide an additional phosphorus adsorption capacity, further increasing the overall phosphorus removal efficiency.

Design Alternatives

The following sections offer a detailed overview of the design of each alternative, covering aspects such as location, footprint, and the design rationale. Additionally, each alternative's specific design components and treatment methodology will be thoroughly discussed.

Table 3: Proposed Design Alternatives

ID	Name	Description
A1	Presumptive Guidance	Clarifying basin sized based on presumptive guidance methods used in practice as required by rulemaking in the F.A.C. Ch. 62-330 (2024) as directed by Florida Clean Waterways Act (Chapter 2020-150, Laws of Florida).
A2	No Baffles	Clarifying basin sized with fundamental physical principles and surface overflow rate (SOR).

A3	Baffled	Clarifying basin sized with fundamental physical principles in conjunction with baffles to reduce footprint area and improve treatment through improved hydrodynamics (reduce dead zones, short-circuiting)
A4	Baffled: AI Optimized	Clarifying basin sized with an AI Neural Network algorithm which optimizes basin geometrics, baffles and basin costs

Table 4: Performance Characteristics of Each Desian Alternative.

ID	L (ft)	W (ft)	H (ft)	A (ft^2)	B	Load Reduction Goal (80%)
A1	550	55	8	30,250	-	No
A2	150	15	8	2,250	-	Yes
A3	140	14	8	1,690	5	Yes
A4	140	14	8	1,690	3	Yes

Each design will be loaded by stormwater as conveyed from the surface parking area catchment, using a concrete pipe with a diameter of 18-inches, a length of 50 feet, a slope of 6%, and a Manning's roughness of 0.0013 (For design details see Appendix D: Design). This influent location is identical for all design alternatives and will be located in the center of the north wall of the basin. Table 4 summarizes the basin geometric design and performance characteristics of each alternative.

Basin length, ***L***, is the distance measured from the basin wall containing the inlet pipe to the basin wall containing the effluent orifices. Basin width, ***W***, is the distance measured from side to side of the basin. Basin height, ***H***, includes the 2-ft. sludge collection zone and 1-ft. of freeboard, which was established based on conventional design practices and the need to create an anoxic sludge zone (Liu et al., 2010). Basin area, ***A*¸** is the footprint area of the basin and is equal to the product of the length and width internal dimensions (inside to inside of perimeter walls). Number of baffles, ***B***, represents the number of gabion baffles. For specific details regarding construction material, more detailed cost estimates are provided in Appendix E: Alternative Development and Design. The column indicated by "Reduction Goal (80%)" reflects whether all targeted constituents meet the 2024 CWA regulatory load reduction target of 80%.

Alternative 1: Presumptive Guidance

Design Overview

Design Alternative 1 (A1) was developed using the methodology for application in Florida for the reduction of nutrient loads in stormwater to equal to or less than that of predeveloped levels (Harper and Baker, 2003). The load-based methodology, which implements site-specific hydrologic characteristics, was applied for the estimation of pre- and post-development annual loadings. This estimation considered site characteristics such as area of land in the watershed, the type of land usage, number of different land use categories, annual site rainfall and the runoff "C" value. This design requires a 14-day residence time to meet presumptive guidance requirements, and as such, has much greater storage considerations to be made compared to the other alternatives

(Li, H., 2021, 2022). As shown in Figure 6, the basin is centered laterally within the available ground adjacent to the surface parking area, to utilize the hydraulic gradient created by the existing topography.

This alternative follows the current presumptive guidance practice of stormwater management for Florida for the design of stormwater basins. Because of this, design consultants, owners and contractors are most familiar with the design and construction of this type of basin (Li, H., 2021, 2022).

Figure 6: Alternative 1 footprint (ESRI).

Design Composition

A1 is a 550x55-foot basin. This meets the recommended length-to-width ratio of 10:1, which provides nominal reduction in short circuiting. The basin footprint is 30,250 square feet, and with landscaping and fencing, the area is 36,400 square feet. The maximum basin storage (not including freeboard) is 212,000 cubic feet. As outlets, this basin has two one-inch (nominal) orifices with the invert of each orifice located directly above the 2-ft. sludge zone. These orifice diameters are designed to achieve the required 14-day residence time for biological removal of nitrogen as TN (SJRWMD, 1993, 2024b). This design has annular screens to prevent the orifices from being clogged with debris but does not include filters as they are not required with presumptive guidance.

Alternative 2: No Baffles

Design Overview

Design Alternative 2 (A2) is sized using direct application of particulate settling velocity and Newton's Law of settling for discrete particle settling (Type I). This was determined using a particle size distribution (PSD) collected at the project location from a previous study (see Appendix D: Design). This method differs from A1 by sizing the basin around PM settling behavior and flow through the basin, resulting in a more accurate load reduction determination (load reduction is no longer presumptive) which results in a significantly reduced basin footprint.

Figure 7: Alternative 2 footprint (ESRI).

The initial basin and outlet orifice dimensions were sized using Reynold's Transport Theorem which simplifies to the standard level-pool routing method. The Type I particle settling velocities for each particle diameter were calculated with the site-specific PSD, which informed the mass fraction of PM that was able to settle out within the basin at each time step, from which PM load reduction was determined. With the knowledge of partitioning of

P and N to PM, these load reductions were also determined based on PM load reduction. This process was performed iteratively to determine the basin dimensions which would achieve desired space and load reduction requirements (see Appendix D: Design for a detailed description of this process).

Design Composition

A2 is a 150x15-foot basin, which also meets the recommended 10:1 length to width ratio for clarifying basins. The total basin depth of 8-ft. includes 1 foot of freeboard, a 2-foot sludge zone, and 5 feet dedicated to the sedimentation process. The total footprint area of this design is 2,250 square feet, and the maximum basin storage capacity is 15,750 cubic feet, excluding 1 foot of freeboard. This basin features two 1.25-inch diameter orifices directly above the 2-foot sludge zone for redundancy, as well as annular screens around the orifice openings to prevent clogging. This design, and all subsequent alternatives, required 6 filters to reduce the SLR to achieve higher performance. This design ultimately meets the constituent load reduction goal of 80% for PM, TP and TN.

Alternative 3: Baffled

Design Overview

For Design Alternative 3 (A3), gabion baskets filled with 75 to 100-mm diameter CRC that functioned as baffles were introduced to reduce basin footprint, while maintaining load reduction goals. Baffles reduce dead zones and short-circuiting within clarifying basins while also functioning as horizontal biological and physical filters, but there reaches a point in which additional baffles provide diminishing return on load reduction and return on investment. This design used 5 transverse baffles, which studies have demonstrated to yield improved hydraulic efficiency within a rectangular basin of similar geometrics (Wilson & Venayagamoorthy, 2010). Additionally, the baffles increase energy and flow dissipation, as they are comprised of approximately 35% porosity (Jalil et al., 2019).

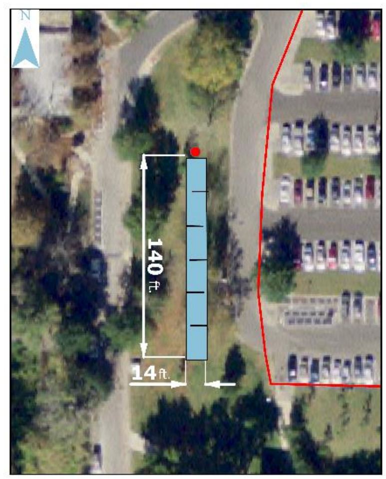

Figure 8: Alternative 3 footprint with baffles shown (ESRI).

PM load reduction was calculated in a similar way to the previous alternative, by using the site-specific PSD and particle settling mechanics. The PM load reduction with the addition of five baffles was implemented based on existing research to provide a PM load reduction for this alternative (FDOT, 2021). The CRC media used within the gabion baskets provides phosphorous adsorption, biological uptake and filtration for TP loads. The hydraulic conductivity of the permeable CRC-filled gabions provides tortuous filtration, and high surface area for adsorption, chemical precipitation of orthophosphate and biological uptake, further improving the overall load reduction of this basin design; at a lower footprint than the previous alternatives.

Design Composition

A3 is designed with a length of 140 feet and a width of 14 feet, still meeting the recommended 10:1 length to width ratio. The total footprint area of this design is 1,690 square feet, and the maximum basin storage capacity is 11,830 cubic feet, excluding 1 foot of freeboard. This basin features two 1.25-inch diameter orifices directly above the 2-foot sludge zone for redundancy, as well as annular screens to prevent clogging. The profile of A3 mirrors that of A2, with identical sludge zone, designated sedimentation zone, and freeboard dimensions as A2. This design required 6 filters to reduce the SLR and achieve higher nutrient reduction. The addition of 5 baffles, constructed with gabion baskets, further improved the overall removal efficiency. See Appendix E: Alternative Development and Design for further design details and design calculations. In function, the actual load reduction efficiency will be higher than the calculated efficiency, as the baffles bring improved volumetric utilization by reducing short-circuiting and dead zones. This design ultimately meets the constituent load reduction goal of 80% for all three constituents of concern.

Alternative 4: Baffles – AI Optimized

Design Overview

A4, machine learning (ML) was introduced as a tool to optimize cost and load reduction. This design aligns with AI initiatives through UF's supercomputing resources (HiPerGator) to provide more optimal solutions, in this case design optimization. This also provides an engineering and scientific workforce with the tools to advance and implement AI to more optimally solve problems in their disciplines (UF, 2023a).

Figure 9: Alternative 4 footprint with baffles shown (ESRI).

Linear regression (LN), which is a simple machine learning model, and artificial neural network (ANN) models were used to predict cost of construction and load reduction by this alternative design. The models were trained with data generated from the iterative processes used to develop A2 and A3, so that there was a dataset with various basin areas, number of baffles, and their associated costs and load reduction values. Using a root-mean-squared error (RMSE) metric, LN was determined to provide a better predicted cost, and ANN was determined to provide a better predicted load reduction.

Analysis of these models yielded the basin dimensions and baffle number that balances the goals of required load reduction at a reduced overall cost based on the highest sum of weighted normalized scores, with the final parameters of 140 feet by 14 feet, and 3 baffles, as shown in Figure 9. Comparison with manually calculated values compared favorably with the results of these models in optimizing basin design for cost-effectiveness and load reduction, providing compliance with the Florida 2024 Clean Waterways Act load reduction while also meeting the client's needs. Refer to Appendix E: Alternative Development and Design for more information on the design process.

Design Composition

A4 is a 140x14-foot basin, which has the same footprint and profile as A3, but meets target load reductions with two less baffles (only three). ML predicted that the optimal number of gabion baffles is three, as the 80% constituent load reduction goals are still met with only three baffles, which results in a lower cost and easier maintenance. This is supported by the asymptotic behavior between the number of baffles and PM settling efficiency (Wilson and Venayagamoorthy, 2010). Similar to A2 and A3, this design required 6 filters to reduce the SLR and achieve higher load reduction. As with A3, the actual efficiency will be higher than the calculated efficiency due to the improved hydraulic efficiency provided by the baffles. This design ultimately meets the constituent load reduction goal of 80% for all three constituents of concern as shown below in Table 5, which is compared to the rest of the alternatives.

Table 5: Removal efficiencies for each alternative, as well as the mechanisms behind constituent removal.

Alternative ID	Constituent	Removal Efficiency	Sedimentation & Anaerobic Digestion	Filters (finer PM)	Filters (adsorption)
A1	PM	99.0%	✓		
	TN	99.0%	✓		
	TP	77.0%	✓		
A2	PM	99.1%	✓	✓	
	TN	80.0%	✓	✓	
	TP	90.0%	✓	✓	✓
A3 + Gabions	PM	99.4%	✓	✓	
	TN	80.2%	✓	✓	
	TP	91.0%	✓	✓	✓
A4 + Gabions	PM	99.4%	✓	✓	
	TN	80.2%	✓	✓	
	TP	91.0%	✓	✓	✓

Water Quality

Water Quality Volumes

The volume of stormwater which is collected and treated by the basin is equal to 9,042 cubic feet and remains consistent for each of the alternatives as the catchment area remains the same. This volume was calculated using a scale-modified 25-year, 10-minute NRCS design storm for the southeast region. The scaling of this storm was performed to align with the timescale of storm events contributing stormwater more accurately to the micro-catchment, as the study area is significantly smaller than the basins with which NRCS storm models most accurately represent. The intensity pattern of the storm, or the shape of the curve, was fit to a dataset describing a storm event which took place over a 45-minute period, instead of 24-hours. More detail regarding the creation of the design storm can be found in Appendix C: Design Storm Development.

Constituent Load Reduction

Table 6: Site Constituent Loading Concentrations (FDOT, 2021).

Constituent	Median Site Value
TN	4.52 (mg/L)
TP	3.23 (mg/L)
PM Diameter	0.102 (mm)

As described in the Legal and Regulatory section, the nutrient reduction goal for the project was set at 80% to meet the requirements of the 2024 Florida Clean Waterways Act. Biogenic material generated from the raised vegetated islands in the surface parking area are the main source of PM. An analysis of the existing site data and the use of Equation 15 identified the particulate fraction of TN (primarily as biogenic material: grass clippings, leaves, organic detritus) as approximately 0.40. As a result, load management of PM through sedimentation and filtration were significant for nutrient load control. A combination of four mechanisms (sedimentation, filtration, anaerobic denitrification, and biological aerobic uptake), was utilized to reach the 80% nutrient reduction goal of the project. The sedimentation mechanism separated settleable and sediment-size PM (25-75 and >75 µm), which also separates PM-bound nutrients. The filtration mechanisms (gabion baffles and radial granular media filters) separate suspended PM (< 25 µm) while also providing a chemical precipitation/adsorption mechanism for total dissolved phosphorus (TDP) to the CRC of the gabions or the AOCM. Anaerobic denitrification, in the 2-foot sludge collection zone, targeted denitrification of total dissolved nitrogen (primarily nitrate). Finally in alternatives A3 and A4, aerobic uptake was facilitated by the microorganisms on the CRC gabion baffles. The implementation of gabion baffle walls also facilitated an 80% load reduction goal through the reduction of dead zones, the decrease in short circuiting, more volumetric utilization, the increase of horizontal filtration, and adsorption of TDP. This same design with the combination of these mechanisms provides extensibility to more industrialized urban areas such as New Orleans where the transport and fate of metals is also of concern (Bhada et al., 2009).

Cost Comparison of Alternatives

Materials and Installation

For each of the four alternatives, a cost estimate was prepared using the unit costs from the 2015 RS Means database (RS Means, 2015). The costs were adjusted to account for inflation between January 2015 and February 2024. An average inflation rate of 2.06 from the U.S. Bureau of Labor Statistics was utilized to prorate the RS Means values to present day. The final costs were adjusted to match regional costs based on RS Means regional conversions.

The construction of each basin design alternative was based on the use of identical building materials and met UF's building specifications. The structural walls and floor of the basin consisted of 8-inch-thick concrete with epoxy-coated #4 rebar at 8 inches on-center in both directions. The required earthwork for each alternative included excavation, grading, selective tree removal, grubbing and stump removal, and compaction/densification. The overall cost of earthwork was a function of the basin design alternative footprint. Additionally, due to UF's building codes and landscape requirements each alternative required the installation of a fence and at least three-foot-tall bushes surrounding the site. The fence requirement for this project is a six foot-tall, galvanized steel wire fence. Native plants are a requirement for landscaping. The University's landscaping master plan provided a list of pre-approved plants that could be utilized on the site. The team's recommendation is that Holly bushes should be planted along the perimeter of a basin design alternative location.

The alternatives also had some features that varied affecting the overall cost of the project. Alternatives A1 and A2 required demolition due to the site constraints. A1 required the demolition of a sidewalk and another adjacent paved parking area at the southern portion of the site. A2 also included the removal of the sidewalk to ensure the construction crew had enough working area to construct the project. Additionally, A2, A3, and A4 required the installation of radial cartridge filters within the basins. These filters are constructed using a PVC cartridge and the required media is made of crushed concrete with an aluminum oxide coating to facilitate TDP (primarily as orthophosphate) adsorption. The construction of these filters requires the contractor to reuse the concrete (sidewalks) from the demolition crush the concrete to a nominal one-mm diameter and allow the crushed media to carbonate by exposure to the atmosphere for 30 days. These construction requirements were considered and included in the overall unit cost of the filters. A3 and A4 also require the installation of gabion baffles. These baffles required the use CRC as a substrate which was also generated from the recycling of concrete pavement and sidewalk. The cost of these baffles was determined from previous installation of CRC gabion baffles at the Naples, FL airport as part of a basin retrofit (FDOT, 2016).

One challenge was determining the land valuation for the site since UF is on public land. The overall land value was determined by contacting UF's Facility Services. Facility Services provided the valuation for our given parcel based on an Alachua County Property Appraiser (Alachua County, 2023). The land valuation was determined to be $100,000 per acre on UF's campus. A detailed land valuation can be found in Appendix F: Comparison of Alternatives.

Table 7 summarizes the estimated costs, including 1 year of annual operation and maintenance, of each of the four design alternatives along with the alternative ID, name, and footprint area. Total costs were rounded to the nearest thousand for each alternative. Detailed cost estimates for each alternative, including specified materials and construction fees, can be found in Appendix E:

Table 7: Estimated Costs of Design Alternatives

Alternative ID	Name	**Footprint Area (sf)**	**Cost**
A1	Presumptive Guidance	30,250	$742,000
A2	No Baffles	4,000	$168,000
A3	Baffled	3,600	$159,000
A4	Baffled: AI Optimized	3,600	$158,000

Alternative A1 costs significantly more than the other three alternatives. The difference in cost can be attributed to A1's larger footprint. The additional area increases earthwork, concrete, landscaping, and fencing costs. Significantly, the additional area requirement adds additional demolition costs since the required basin footprint conflicts with the sidewalk, walls and parking area behind UF's Mechanical and Aerospace Engineering building. The costs of the other three alternatives are relatively similar with Alternative 2 being the second most expensive alternative and Alternative 4 the least expensive. While the range of costs for the last three alternatives is not significant, ranging from 168,000 to 158,000, Alternative 4 does provide additional cost benefits, the other three alternatives do not. The basin's reduced footprint reduced the costs of earthwork, concrete, landscaping, and fencing. These reductions offset the additional cost of adding gabion baffles to the system; that improved intra-basin hydrodynamics and load reduction. The CRC gabion basket substrate also improves the basins phosphorous adsorption capacity. Alternative 4 provides the most cost-effective design that meets Florida's 2024 Clean Waterways Act regulations (Fla. Admin. Code Ann. ch. 62-330 (2024)).

Operation and Maintenance

Proper operation and maintenance procedures are important to ensure the clarifying basin continues to operate as designed after installation. Annual removal of accumulated sediments in the bottom of the basin should be performed as the system's primary standard maintenance. Inspections should take place to identify any indications of residue build-up in adjoining pipes, impeded flow, basin sediment flushing, gabion basket damage, erosion of the soil surrounding the site, basin exterior walls, and interior structures. If these issues were to occur, basin performance would be reduced, and the clarifier's performance may no longer meet the desired output water quality of the client. Structural issues and erosion of surrounding soils, if left unmanaged, can become a health and safety risk to people and the environment.

UF should perform inspection and maintenance of the conveyance pipes, including removal of debris and sediment, in congruence with its standard practices. During the first year of operation, particularly the first rainy season ranging from late May to early October, the basin may require more attentive maintenance as proper biological denitrification conditions will still be developing and basin output performances may be variable during this period.

Recommended Alternative

Alternative Evaluation

The decision matrix shown in Table 8Table 8: Evaluation of Alternatives based on Peak Reduction, Community Impact, Space, Nutrient Loading, and Cost on the following page was used as the primary tool in evaluating the alternatives for selection. Based on the priorities of the client, applicable regulations, and consideration of potential future needs, the project team identified six criteria for evaluation. These included: peak flow reduction (considering both reduction of time to peak and reduction of peak outflow), sustainability, space, nutrient loading (consisting of removal efficiency for both TP and TN), community impact, and cost. The evaluation criteria were each assigned weighting factors, listed under each column label and within the *Weight* row. The performance of each alternative within each evaluation criteria category is scored from 1 to 4, with 1 representing the best, or most desirable, score. See Appendix F: Comparison of Alternatives for a detailed breakdown of the decision matrix shown below in Table 8.

Table 8: Evaluation of Alternatives based on Peak Reduction, Community Impact, Space, Nutrient Loading, and Cost.

	Peak Reduction	Sustainability	Community Impact	Space	Nutrient Loading	Cost	Weighted Score	Overall Rank
Weight	0.0667	0.0667	0.1333	0.2000	0.2667	0.2667	-	-
A1	1	4	4	4	3	4	3.40	4
A2	2	2	3	2	1	2	1.87	3
A3	2	1	2	1	1	1	1.20	2
A4	2	1	1	1	1	1	1.07	1

Selected Alternative

Using the evaluation criteria provided in Table 8, Aqua Machina recommends the selection of A4, the baffled, AI optimized basin. This option achieves the required nutrient load reduction of 80% in addition to taking up the least amount of footprint area, implementing community benefit in design, meeting a minimized cost, and reducing peak outflow and time to peak characteristics of the micro-watershed site, as well as aligning with the client's AI initiatives.

Extensibility Study

Project Expansion and Funding

To further improve the stormwater conditions of the site, the infiltration rate of the vegetated raised parking islands within the parking area could be improved by converting them to rain gardens, this collects the stormwater rather than allow it to runoff onto the impervious pavement. Also, the basin recommended for this project could be retrofitted with additional treatment phases as needed in the future, such as oil-separating or ultrafine particle removal train components. Were the University to acquire additional funding through the Environmental Protection Agency (EPA) and Florida Department of Environmental Protection (FDEP), the same design could be implemented at additional paved surface parking areas on campus, further improving the water quality being discharged into Lake Alice.

FDEP provides a variety of Water Restoration Funding opportunities for local governments and eligible entities, some of which would be applicable to this project's expansion. EPA recently announced $41 million in grants for stormwater management projects. See Appendix G:

Extensibility for several available grants that would aid in expanding the application of this technology to appropriate locations (FDEP, 2024b).

Project Extensibility

Recognizing the significance of paved parking areas in stormwater management is crucial for implementing targeted mitigation measures, effective stormwater management planning, and the extensibility of our project. Strategies such as Green Stormwater Infrastructure (GSI) and various configurations of stormwater retention basins can help mitigate the adverse effects of impervious surfaces across the nation, promoting sustainable stormwater management practices and safeguarding water quality in the surrounding environment (Obropta and Monaco, 2017).

A4, which we propose be implemented in areas similar to Gainesville, FL, requires modifications in order to provide portability to regions with flat topography and high water table conditions, such as New Orleans, LA. As a result, we examined two alternatives that are not gravity-driven that can be applied to such areas while still applying the same basic design process utilized for our Florida-based design. These include an alternative that is a modification of A4 and an alternative that implements a GSI design. This section assesses the plausibility of these two alternatives through proposed designs at a paved surface parking of similar geometrics to the Florida project site. Within New Orleans, the selected site for this study is a 3-acre paved surface parking that services a shopping center along the Mississippi River.

Extensibility Alternative 1: Underground Baffled Basin

Design Overview

Extensibility Alternative 1 (XA1) is a modification to our recommended basin design in Florida. The lack of elevation change present in New Orleans presented the main challenge. To solve this issue, we propose placing a baffled basin below the project site by converting our A4 design to an underground vault. New Orleans sits at an average of 3 feet above sea level, with the selected project site at approximately 10 feet above sea level. It was necessary to consider the high-water table that the underground basin would experience and consider buoyant forces in our modifications, to then counteract them with concrete pilings. The effluent is then pumped to the nearest waterbody, in this case, the Mississippi River.

Design Composition

The XA1 basin design is identical to that of A4, with the addition of 14 friction piles, 40 feet in length with a diameter of 2 feet, along the centerline of the basin to counteract the buoyant forces. As this basin will be placed underground, the footprint area only consists of the necessary access manholes for inspections and maintenance. The maximum basin storage capacity is 11,830 cubic feet, excluding 1 foot of freeboard. The basin features an inlet pipe connecting to the site's existing stormwater catchment, an 11x3.5x.5-foot energy dissipation wall, and two 2-inch effluent orifices with .25-inch annular screens to prevent clogging directly above the 2-foot sludge zone.

Additionally, two .5-hp, 10-gpm pumps with variable frequency drives (VFD) will be installed to pump the discharge along the 250-foot pipe connecting to the Mississippi River. The top of the basin will feature four 36-inch diameter vented manholes. Similarly to A4, this design required 6 filters to reduce the SLR and achieve higher performance. The addition of 3 baffles, constructed with gabion baskets, improved the overall removal efficiency further. In function, removal efficiency will be higher due to the improved volumetric utilization that the baffles provide in the calculations. To prevent excess sludge accumulation, regular inspections and maintenance are required. Reference Appendix G: Extensibility for further design information.

Extensibility Alternative 2: Permeable Pavement

Design Overview

For Extensibility Alternative 2 (XA2) we selected permeable pavement as the most appropriate GSI design. Decreasing the impervious percentage of our proposed project site will help alleviate stormwater control issues. We propose implementing a system of interlocking permeable pavers in low-traffic areas, such as the parking stalls. These pavers feature a unique locking system that eliminates the need for joint filler, promoting higher infiltration rates. Impervious asphalt will still be utilized in the driving areas of the surface parking, but such areas will be slightly sloped to direct stormwater towards the pervious pavers, aiding in stormwater infiltration for the entire paved surface parking. Additionally, the asphalt will also act as the edge restraint to ensure the pavers remain securely in place. The proposed permeable pavement design complies with the City of New Orleans' Stormwater Management Code and aligns with requirements to retain or detain and filter the initial 1.25 inches of stormwater from each rainfall event (City of New Orleans, Municipal Code § Sec 26-15.121.7.a.4).

Design Composition

The total area of parking stalls within our proposed project site is 64,700 square feet, which would require 57,869 permeable pavers sized at 11.75x13.70x4.5 inches. The first layer of this design is a compacted subgrade of at least 24 inches, (City of New Orleans, Municipal Code § Sec 26-15.121.7.a.4). Placed on top of the subgrade will be a M200 Woven Monofilament geotextile. The last layer before the pavers is an even and compacted washed stone base of at least 6 inches. With this preparation the pavers can be directly placed on the stone base in the correct orientation and with a quarter-inch joint width between each paver. The proposed system is designed to handle significant rainfall events, with the capacity to store runoff from a 10-year storm event. Regular maintenance would be required to prevent clogging of the permeable surface and maintain the structural integrity of the subsurface layers. Design details can be found in Appendix G: Extensibility.

Extensibility Alternative Analysis

Implementing effective stormwater management strategies for paved parking areas is essential for mitigating the negative impacts of impervious surfaces and promoting sustainable water management practices. The two proposed alternatives for New Orleans—Extensibility Alternative 1 (XA1) and Extensibility Alternative 2 (XA2)—demonstrate how tailored design solutions can address the unique challenges of regions with flat topography and high water tables, such as New Orleans.

Both alternatives offer viable solutions for stormwater management in New Orleans, but each has some trade-offs. XA1 mirrors our Gainesville-based basin, providing a robust, high-capacity system that leverages advanced engineering to manage stormwater in a challenging environment, though it requires significant construction and maintenance efforts that can come with a high cost. XA2, on the other hand, offers a more straightforward, sustainable approach with easier maintenance, though it may not provide the same level of stormwater storage capacity as XA1, and also comes with a high cost from needing to demolish the whole parking area (see Appendix G: Extensibility for cost breakdowns). Additionally, the Gainesville-based basin was designed for nitrogen and phosphorus load reduction, which are not necessarily the constituents of concern in other areas that these designs could be implemented. Ultimately, the choice between these alternatives will depend on the specific priorities and constraints of the project, including budget, maintenance capabilities, and long-term sustainability goals.

Education and Outreach

University Curriculum Integration

The location of the clarifying basin on UF campus and the proximity of our proposed New Orleans project to other universities and tourist areas creates an ideal opportunity for students and members of the community to learn more about the environmental implications of stormwater management. Courses centered around environmental engineering, stormwater management, and water chemistry would benefit especially. Appendix H: Education Program lists relevant UF courses and Tulane courses that could take field trips to the project sites to see direct applications of their coursework.

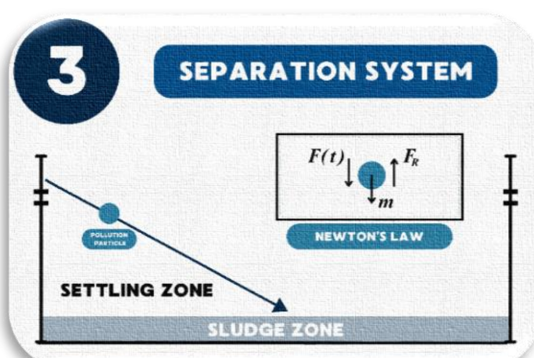

Figure 10: Example Signage - Separation System Diagram.

Signage

The strategic placement of informational signage about the basic purpose and principles of the proposed designs, the relationship between stormwater management and the ecosystem health of surrounding waterbodies, and key processes such as denitrification and phosphorous adsorption add an aspect of public education and outreach to the project site which integrate with the public more effectively. These proposed locations provide visibility in high-traffic areas in which students, staff, professors, tourists, and community members have access to this signage.

The example sign shown in Figure 10. demonstrates the separation of particles through settling and highlights the application of Newton's Law to this process. The local community can be invited to learn about stormwater and its effects on local water quality. Our project aims to create an inclusive environment to encourage Gainesville residents to visit the university campus and get more invested in the management of their local water bodies as a community-wide initiative.

Conclusion

The project encompasses four basin alternatives including (A1) Presumptive Guidance, (A2) No Baffles, (A3) Baffled, and (A4) AI Optimized Baffled with an extensibility study that contains two additional alternatives of (XA1) Underground Baffled Basin and (XA2) Permeable Pavement. A1 adopts established methodologies for Central and South Florida, offering a solution designed around residence time within the basin. A2 leverages site-specific particulate settling behaviors and Newton's Law to achieve a balance between efficiency, cost-effectiveness, and space utilization. A3 further refines the design by integrating gabion baskets as baffles, enhancing particle settling and nutrient treatment functionality while minimizing site disturbance. For A4, machine learning was introduced to predict the basin configuration that would yield the optimal cost, sizing, and treatment efficiency. Together, these alternatives underscore the importance of precision, adaptability, and innovation in addressing complex environmental challenges.

Using the evaluation criteria, Aqua Machina recommends the selection of A4, as a comprehensive and innovative approach to stormwater management to address environmental challenges by meeting nutrient reduction regulatory standards. This additionally aligns with UF AI initiatives to build supercomputing resources in AI to tackle real-world problems and developing and advancing AI in the workforce (UF, 2023a). For areas with different geographical conditions, such as those with flat topography and high water tables, we recommend the use of underground basin systems or permeable pavement initiatives. This comprehensive approach ensures that Aqua Machina's stormwater management strategies are not only innovative and effective, but also adaptable to diverse environmental conditions, setting a benchmark for future projects and contributing to sustainable urban development.

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Appendices

Appendix A: Site Specifics

A.1: Reitz Parking Lot

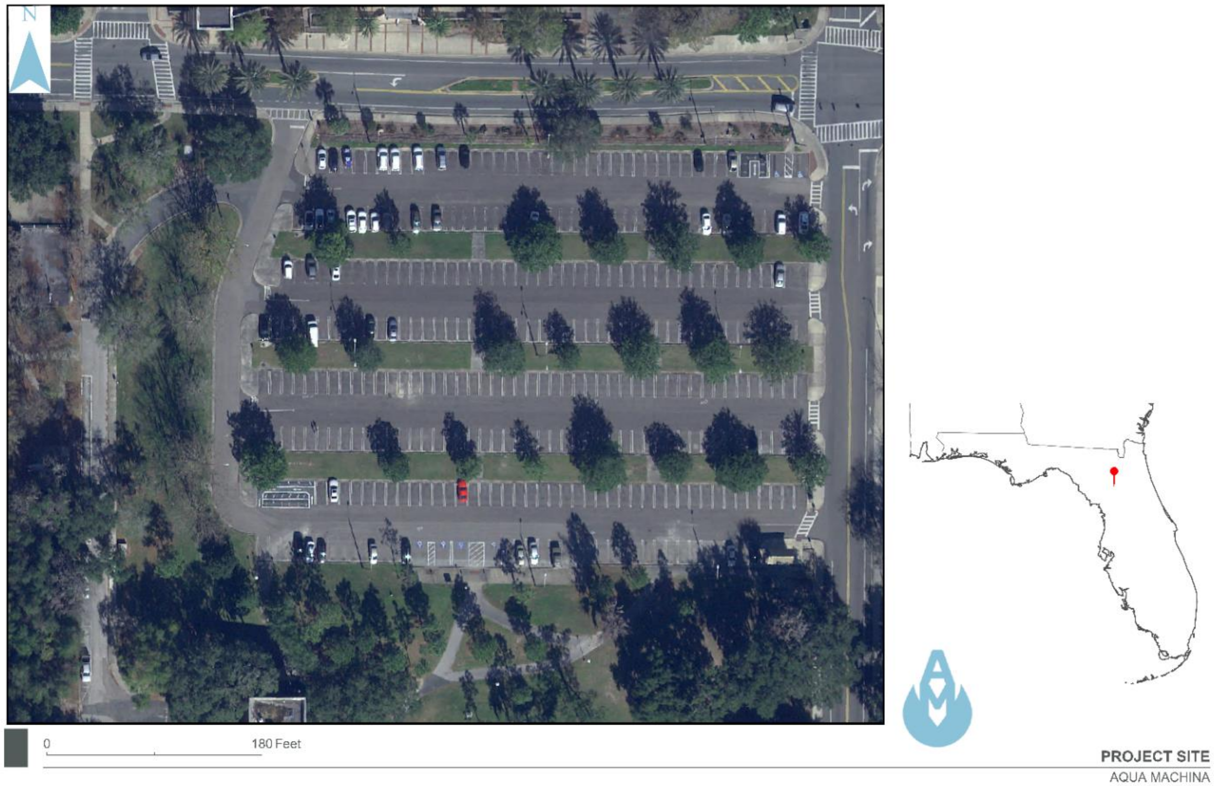
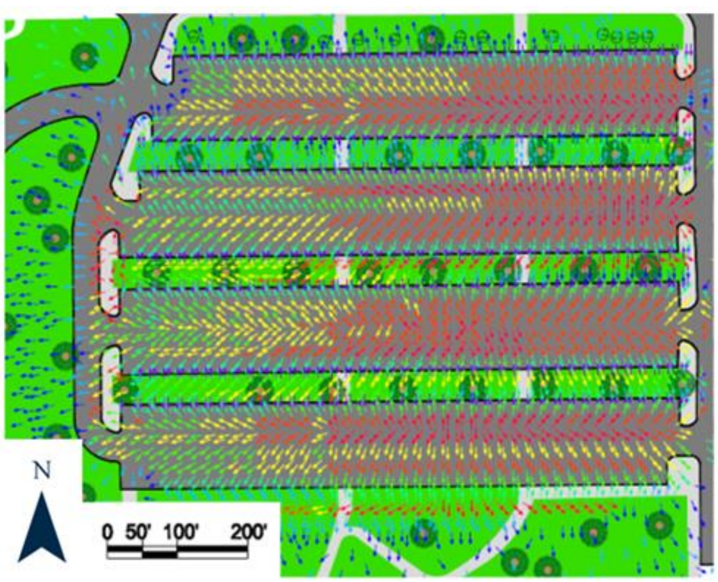


Figure 11: Site Plan View (Florida Marine Research Institute, n.d.)



Minimum Slope	Maximum Slope	Color
0%	1.58%	
1.58%	2.42%	
2.42%	3.29%	
3.29%	4.30%	
4.30%	5.93%	
5.93%	12.25%	
12.25%	90.03%	

Figure 12: Existing Site Slope Conditions (FDOT, 2021); ESRI; Kertesz et al., 2009; USDA, 1986)

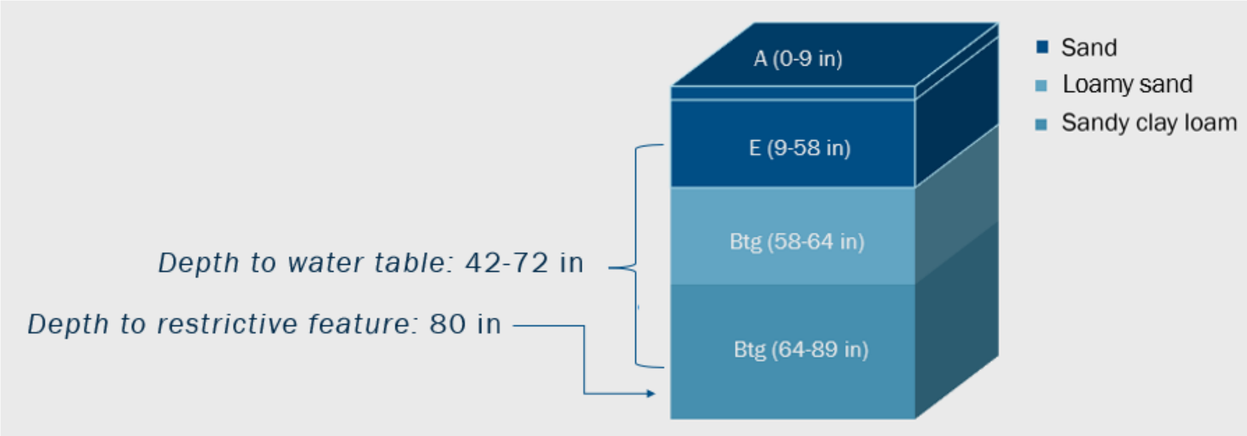


Figure 13: Existing Soil Parameters at Site (USDA, 2023a, 2023b)

Soil characteristics on the project site, shown above in Figure 13, were compiled as part of the preliminary site selection process. There were no unusual or concerning findings for this site.

A.2: Campus Located in Gainesville

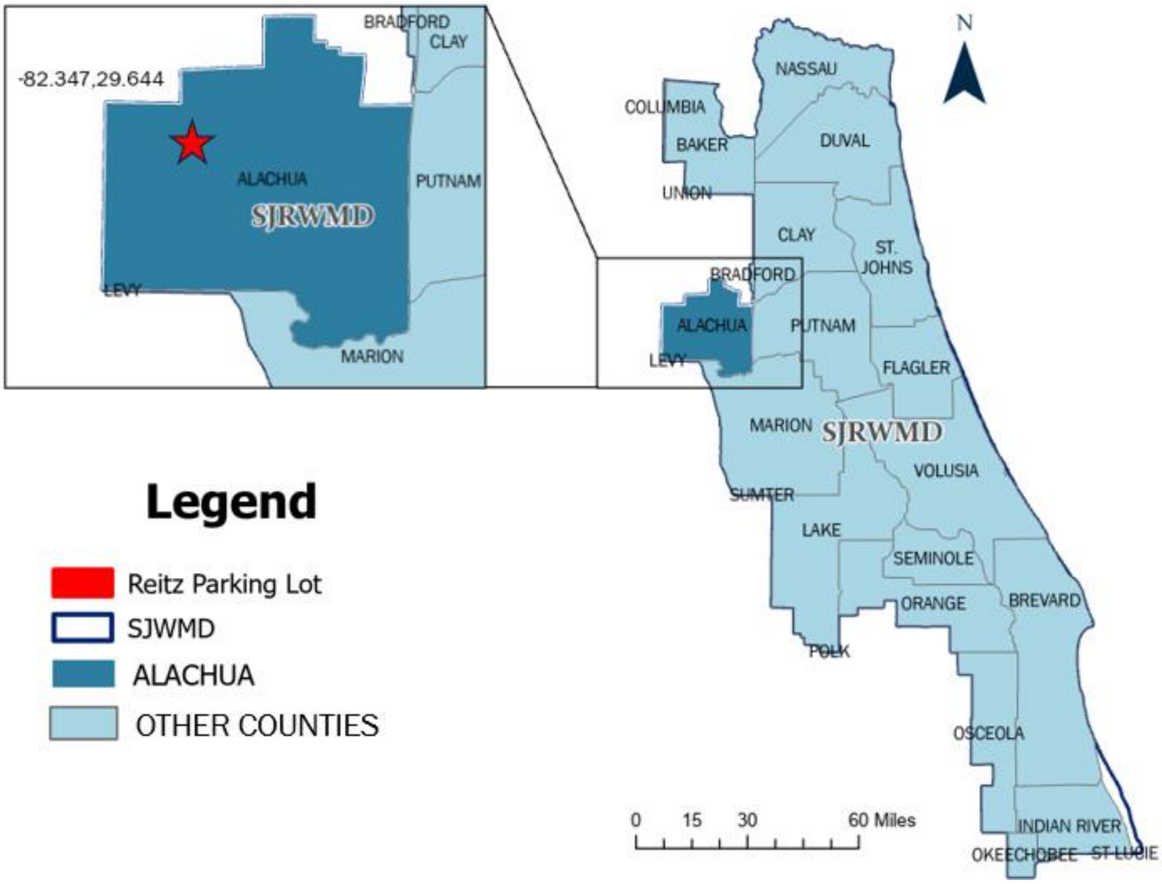


Figure 14: St. John's River Water Management District, Highlighting Alachua County (ESRI; SJRWMD, 2022)

The University of Florida's main campus is located in the City of Gainesville, identified on this map by the red star. Gainesville is part of Alachua County, which is one of 18 counties fully or partially located within the St. John's River Water Management District.

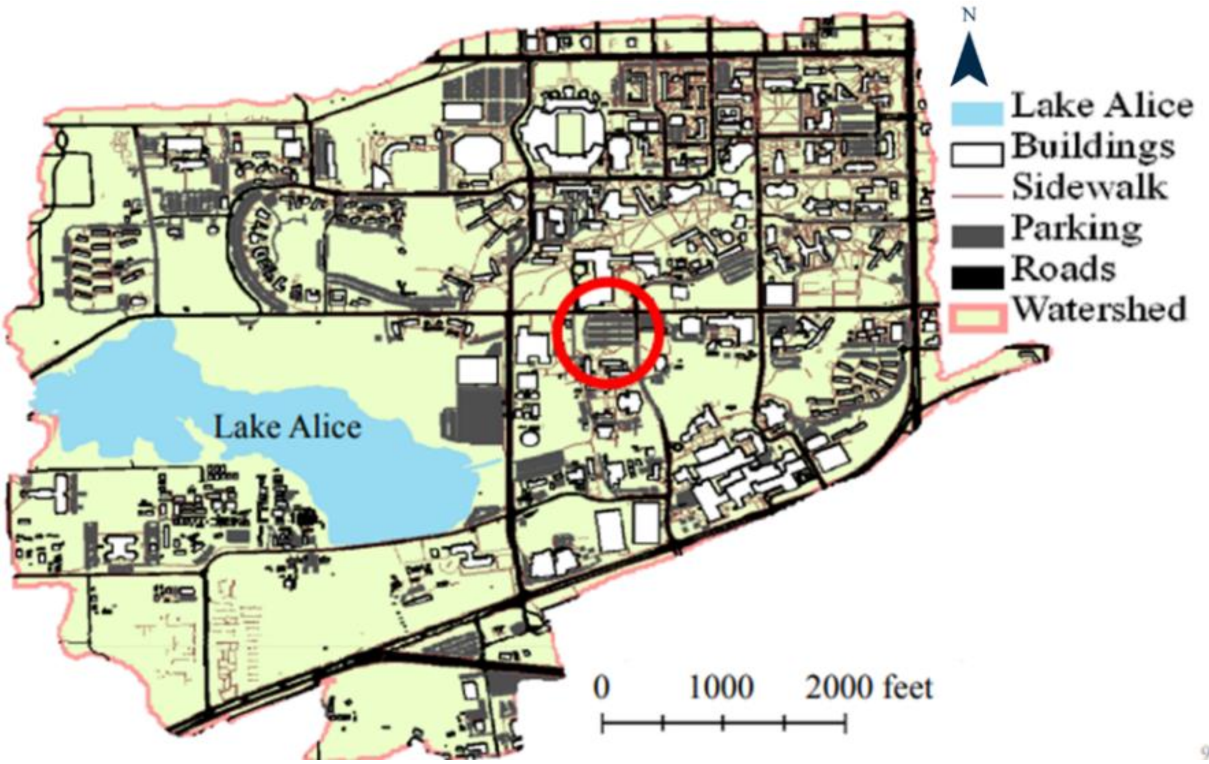


Figure 15: Vicinity Map Locating Study Site (Kertesz et al., 2009)

The University of Florida's campus comprises of 2,000 acres and over 900 buildings. The university's campus makes up almost 5% of the total area of Gainesville, Florida.

A.3 Watershed Information

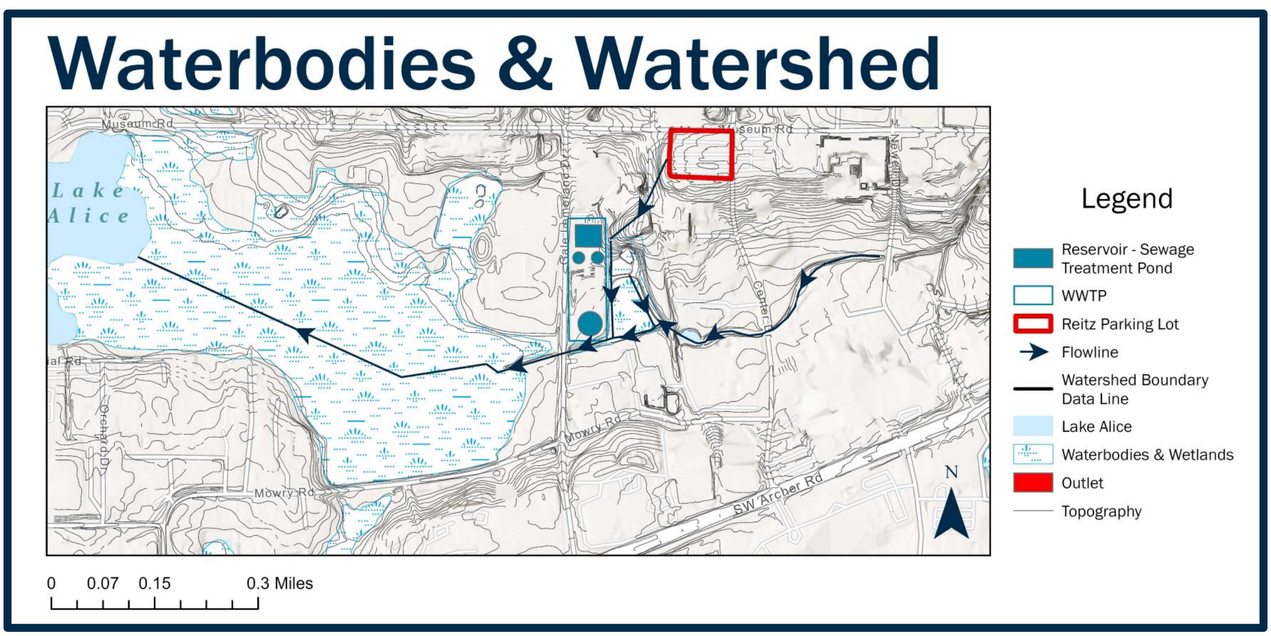


Figure 16: Waterbodies and Watershed Information

The watershed topography features a steep slope from the southwest corner of the site to lake Alice and the Lake Alice floodplain.

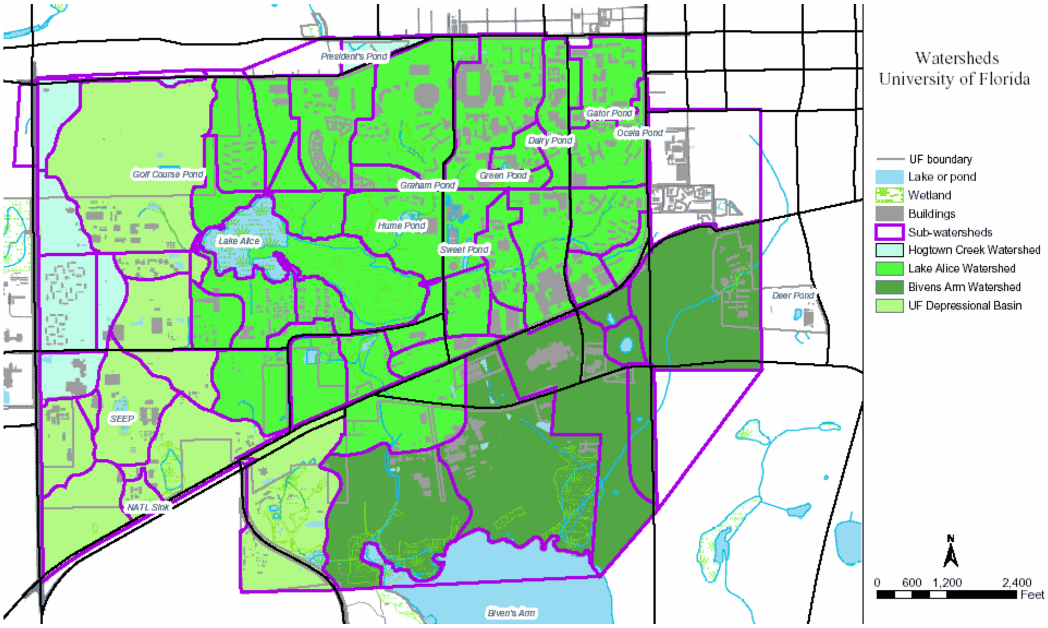


Figure 17: Lake Alice Watershed

More than 60% of the UF campus lies within the Lake Alice watershed. The high level of urbanization and historically limited stormwater treatment infrastructure pose a risk to the water quality of the lake.

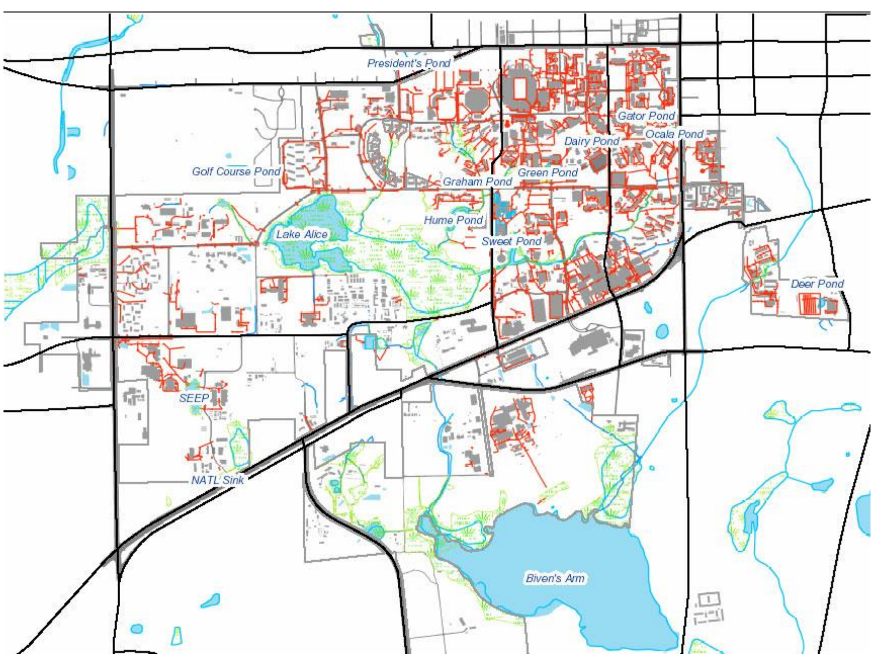


Figure 18: Drainage System on the University of Florida's Main Campus (UF, 2016b)

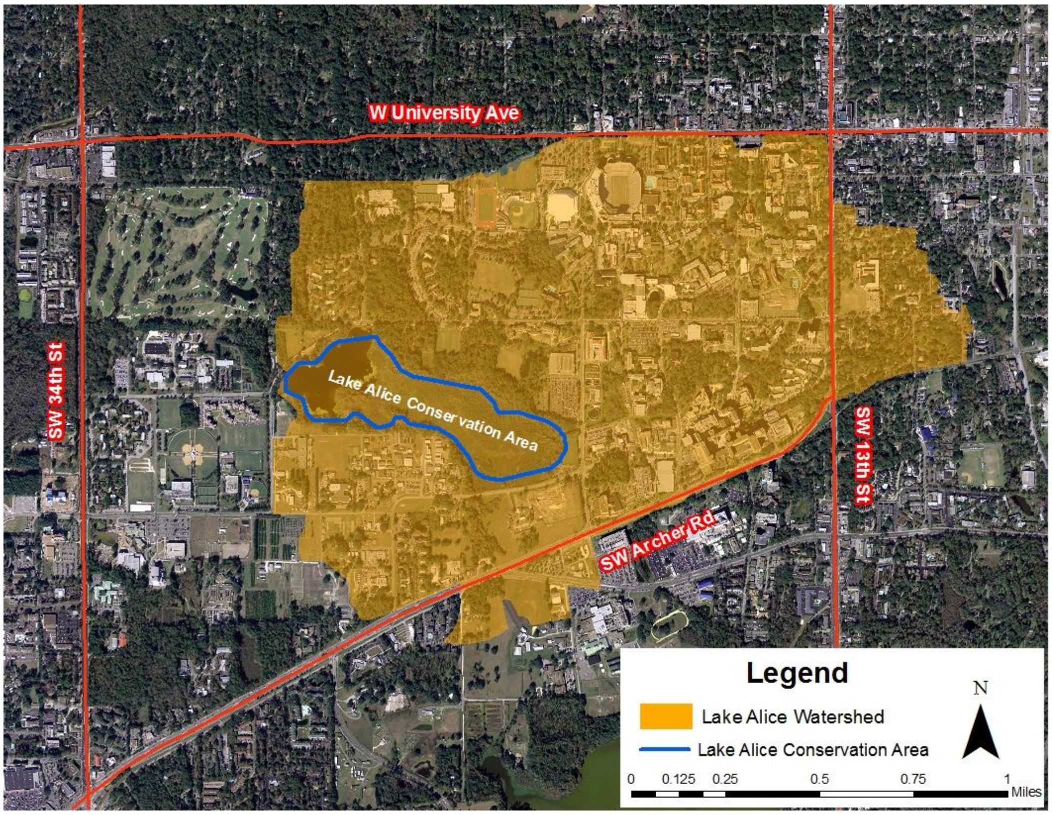


Figure 19: Lake Alice Watershed Area (UF, 2016b)

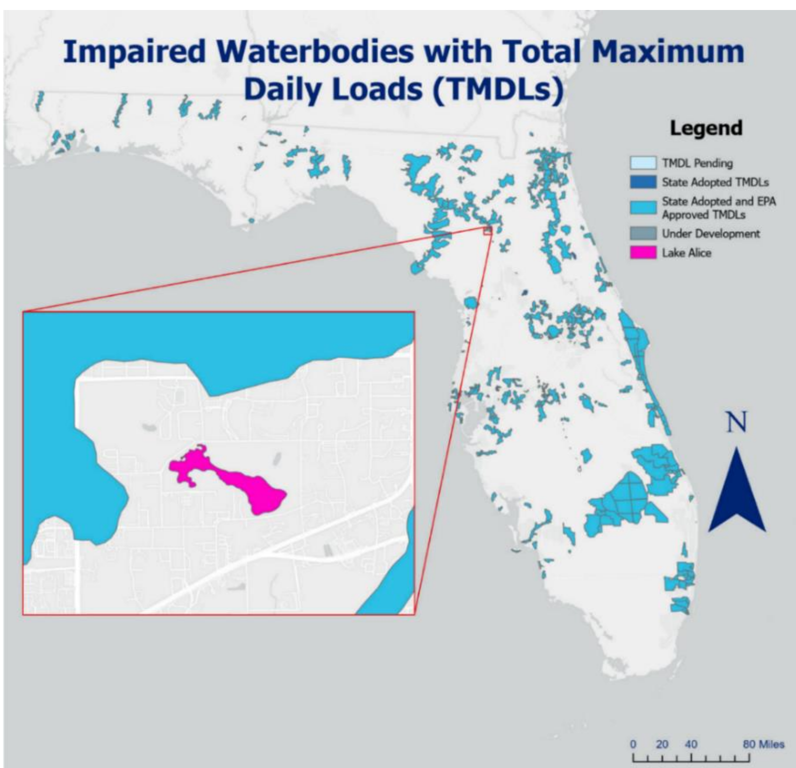


Figure 20: Impaired Waterbodies in Florida with Total Maximum Daily Loads (ESRI; FDEP, 2023)

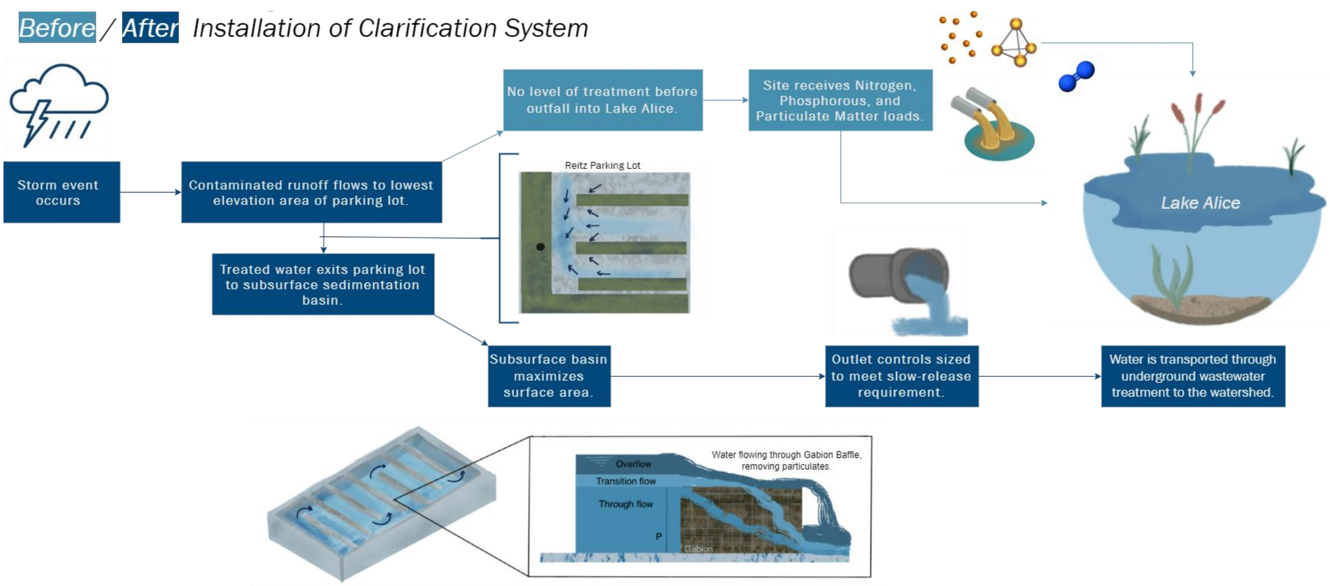


Figure 21: Process Flow Diagram

Appendix B: Regulatory and Risk

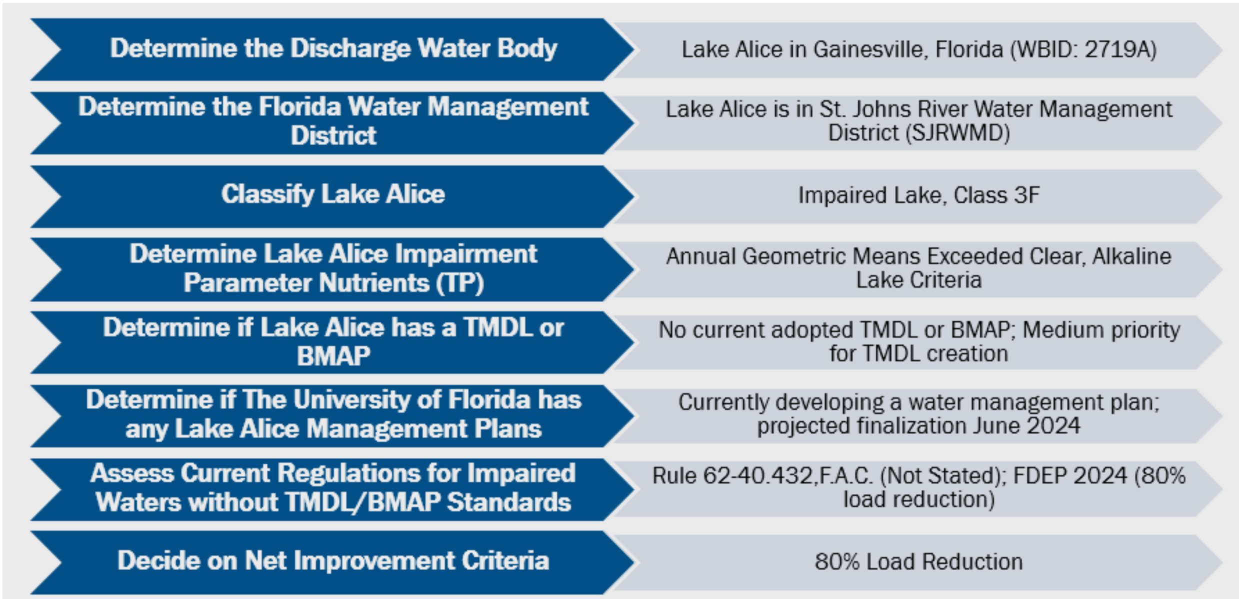


Figure 22: Process Used to Determine the Water Body Categorization of Lake Alice (UF, 2016c; SJRWMD, 2022; FDEP, 2023; FDEP, 2022; UF)

Figure 22 demonstrates the process undertaken to determine the load reduction goals of the project based out outfall waterbody requirements, note that the waterbody is classified as impaired, exceeding clear alkaline lake criteria, but does not have an adopted TMDL, BMAP, or campus specific plan- though one is currently in development. Based on their research and the project parameters, the regulatory team selected an 80% load reduction.

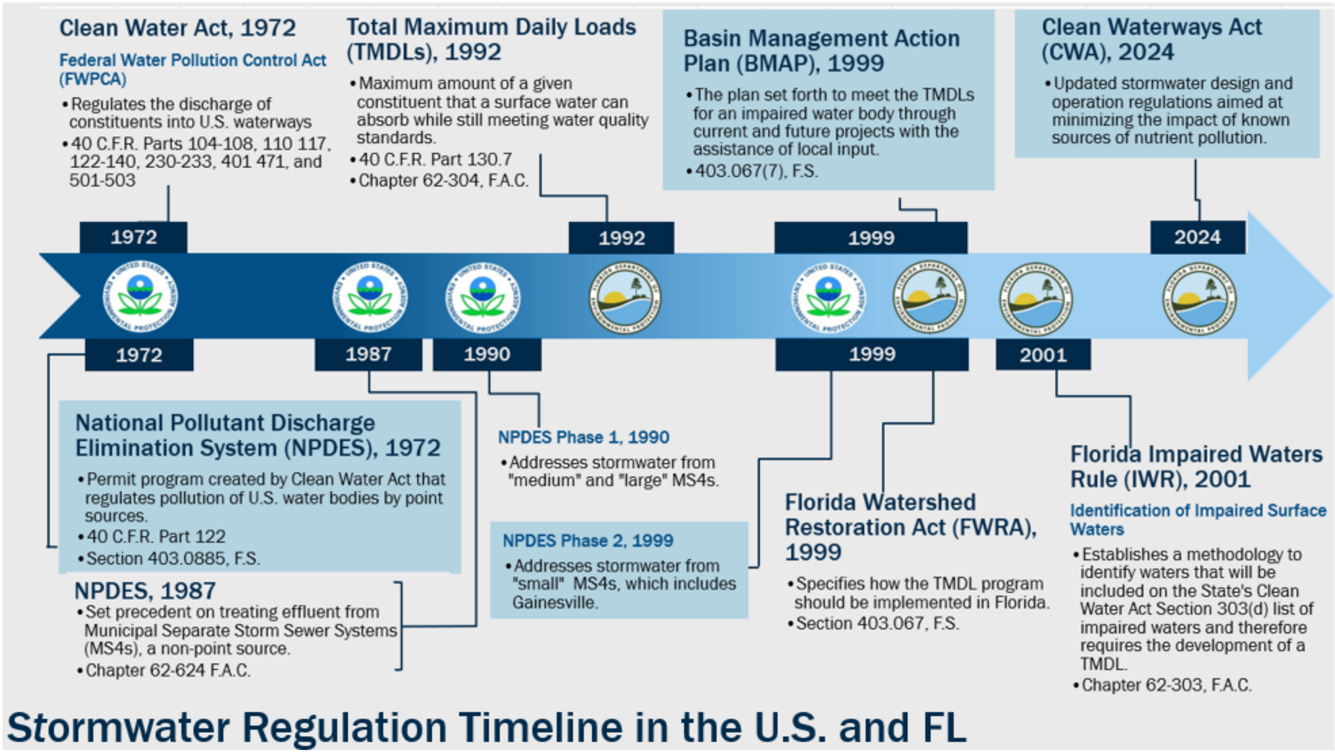


Figure 23: Detailed Stormwater Regulation Timeline (EPA, 2023b; CFR, 1983; FDEP, 2024a; Ellard, 2015; EPA, 2023c; EPA, 2024; FDEP, 2022; Olexa et al., 2021; FDEP, 2024d; FDEP, 2024c; EPA, 2023b)

Figure 23 identifies the key regulations which were researched by our legal and regulatory team. The 1972 Clean Water Act NPDES permit program, 1990 development of BMAPs and 1990 CWA NPDES Phase 2 addressing small water bodies (including Lake Alice) are highlighted here for their importance. The Stormwater treatment requirements under the Clean Waterways Act were of particular interest, as the FDEP implementation of these changes is ongoing.

Appendix C: Design Storm Development

C.1: Volume Determination

C.1.1: Historical Modeled Hydrographs

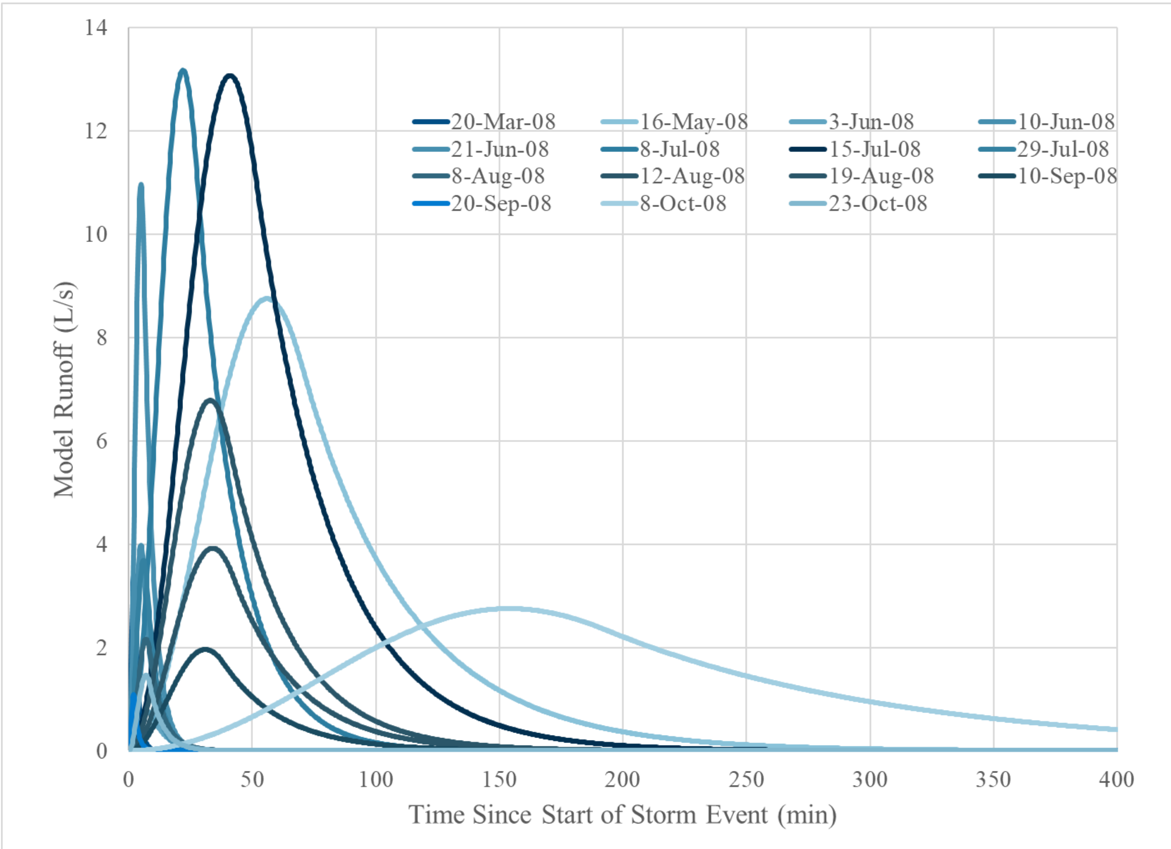


Figure 24: Event-based hydrographs (FDOT, 2021)

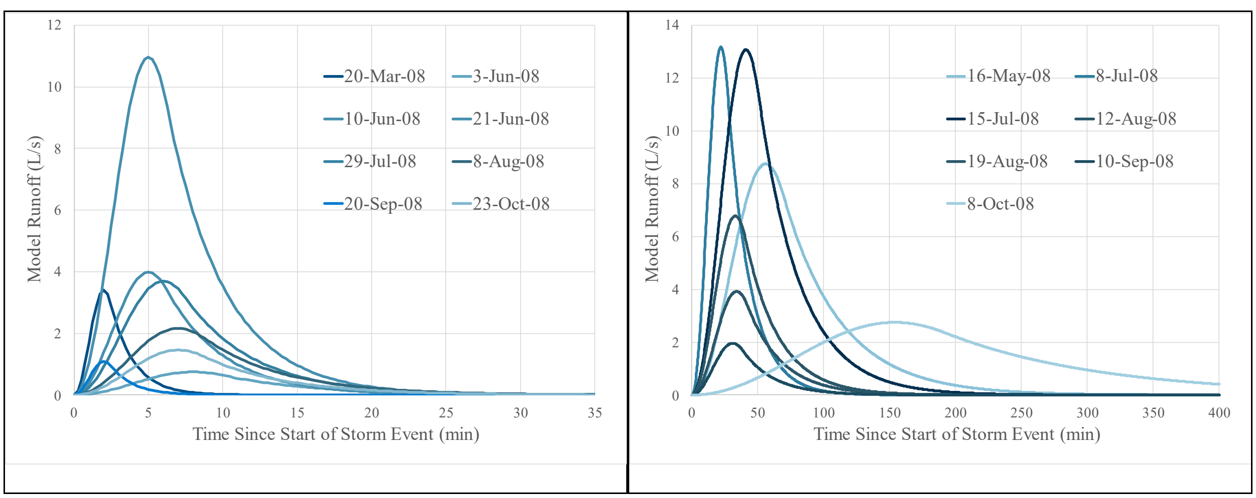


Figure 25: Event-based hydrographs (FDOT, 2021), separated by storms with flow rates that were less than 0.005 L/s at 60 minutes (left) and those that were greater than or equal to 0.005 L/s at 60 minutes (right)

In the process of hydrological analysis and stormwater management, several key methodologies and data sources are employed. The integration of NRCS design storms alongside site-specific historical storm event data ensures a comprehensive understanding of the hydrological dynamics such as peak flow. Hydrograph analysis, utilizing the SCS hydrograph Type 2 assumption, offers valuable insights into the temporal distribution of stormwater. Additionally, the adoption of a Lumped Level Pool Routing methodology aids in guiding storage considerations, facilitating informed decisions regarding the management of water resources within the studied system. These combined approaches contribute to a thorough and effective assessment of hydrological processes and inform the development of robust stormwater management strategies.

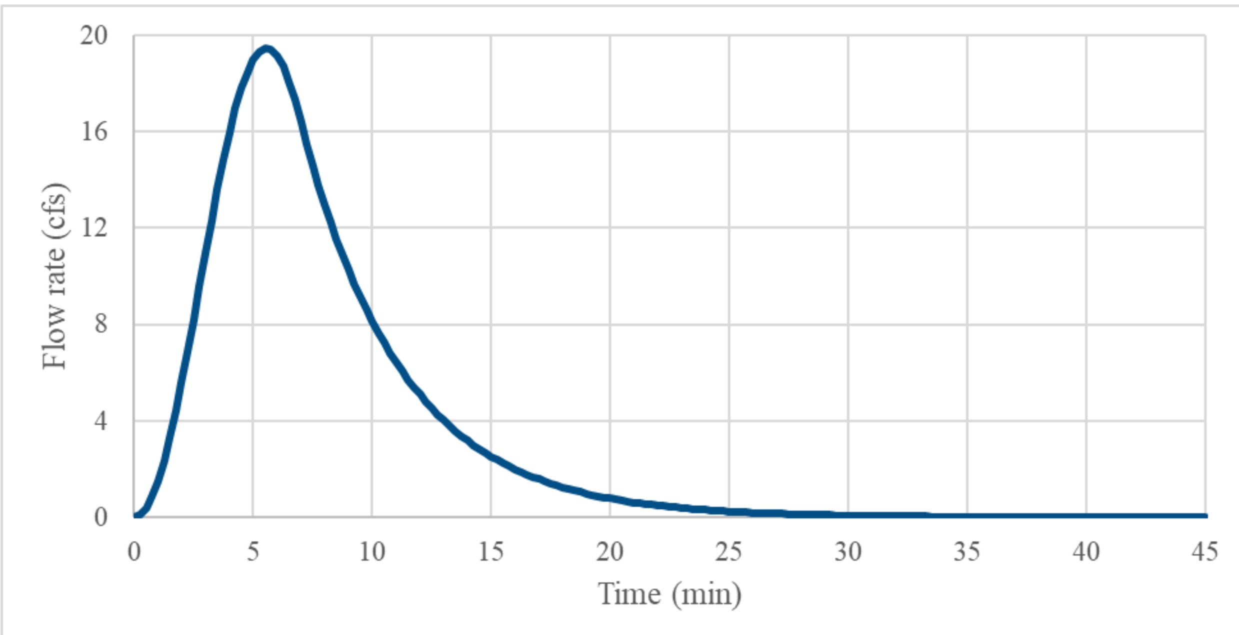


Figure 26: Inflow Hydrograph for Model Storm

C.1.2: Model and Design Storms

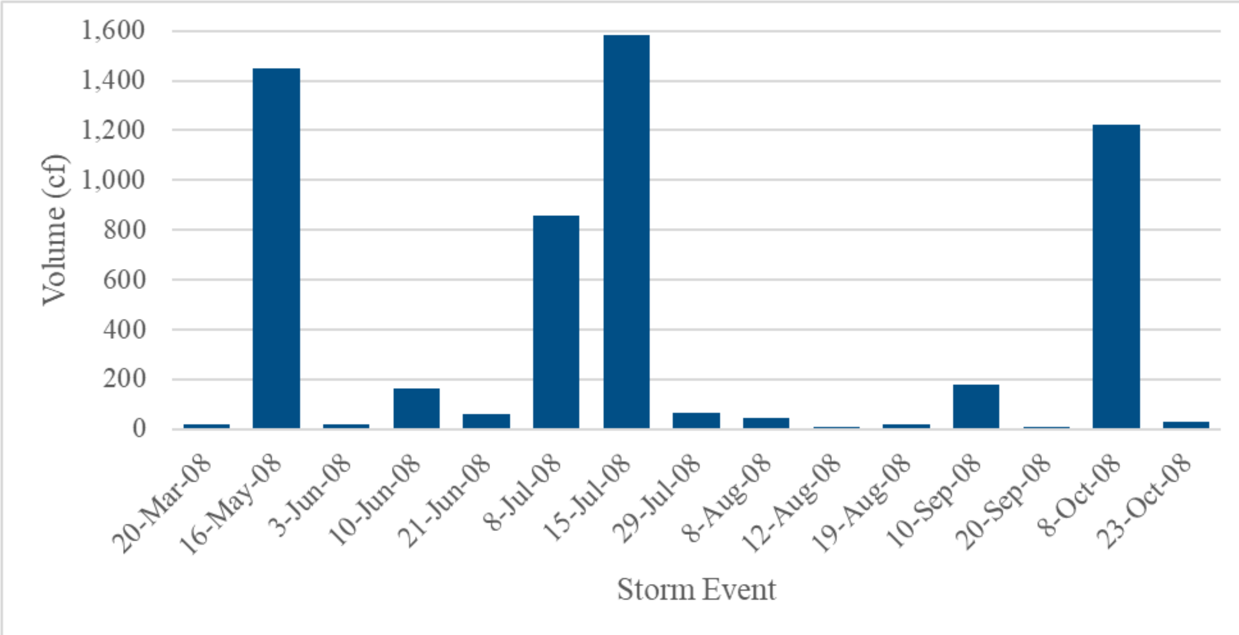


Figure 27: Runoff Volume for Modeled Historical Storm Events

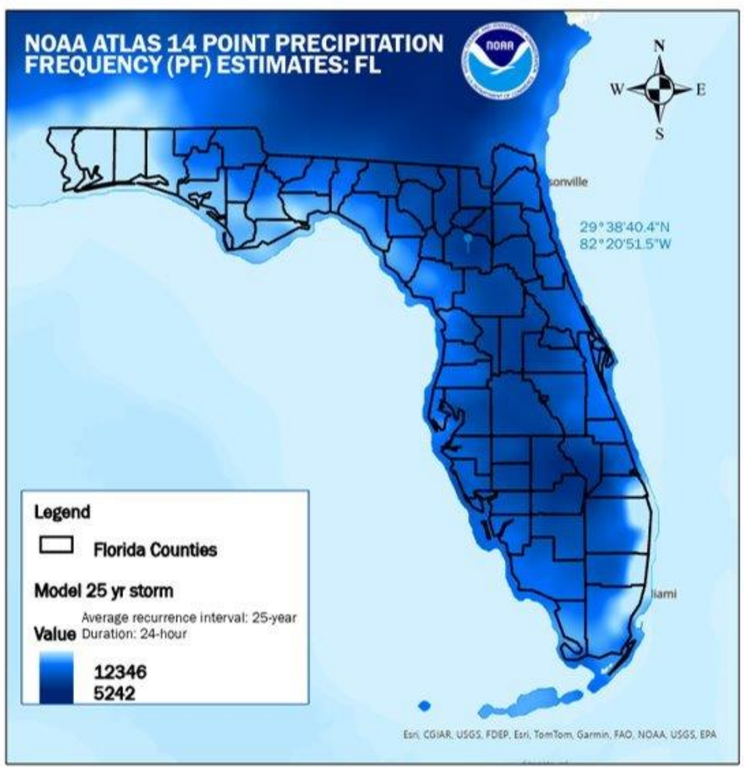


Figure 28: NOAA ATLAS 14 Point Precipitation Frequency Estimates in Florida based on a recurrence interval of 25 years and a storm duration of 1/6 hours. The depth was 1.3 inches. Data from NOAA Atlas 14 (NOAA, 2017)

Table 9: Design Storm Calculations

Average Intensity	
Duration (min) = 10 **Depth (inches) = 1.3**	*Equation 1* $I_{avg} = \frac{duration}{depth} = \frac{1.30\ in}{10\ minutes\ \times \frac{1\ hr}{60\ minutes}} \approx 7.8\ \frac{in}{hr}$
Peak Flow	
$\boldsymbol{I_{avg} = 7.8\ \frac{in}{hr}}$ **A (acres) = 3.3** **K = 1.008** **C = 0.75**	*Equation 2* $Q_p = KCI_{avg}A = 1.008 \times 0.75 \times 3.3 \times 7.8 = 19.46\ cfs$
Sheet Flow Travel Time	
Manning's n = 0.011 **L (ft) = 4792.9** **S (ft/ft) = 0.032** $\boldsymbol{P_2}$ **(in) = 4.16**	*Equation 3* $T_c = \frac{0.007(nL)^{0.08}}{(P_2)^{0.5} s^{0.4}} = 0.85602\ min$
Time to peak	
$\boldsymbol{T_c = 0.85602\ min}$	*Equation 4* $L = 0.6T_c = 0.513\ min$ *Centroid of excessive rainfall is at halfway point of duration, assuming constant rainfall
Artificial Hydrograph	
$\boldsymbol{L}$ **(min)** $\boldsymbol{= 0.513}$ $\boldsymbol{Q_p(cfs) = 19.46}$	*Equation 5* $Q = \begin{cases} \frac{Q_p}{2}\left[1 - cos\left(\frac{\pi t}{t_p}\right)\right], 0 \le t \le 1.25t_p \\ 4.34Q_p e^{-\frac{1.3t}{t_p}}, \quad 1.25t_p < t \end{cases}$ *See Hydrograph (Figure 26) with values of Q plotted against time below
Volume of runoff (V_{RO})	
t (min) = 0:45	*Equation 6* $V_{RO} = \int Qdt$ *Equation 7* $V_{RO} \approx \frac{\sum_{t=0}^{45} Q}{0.25\ min \times \frac{60\ sec}{1\ min}} = 9{,}042.23\ ft^3$ *Q < 0.01 cfs at 41.75 minutes

Appendix D: Design

D.1: Design Goals

The goals of this project cover four main areas: water quality, client expenditures, align with the current initiatives of the University, and community involvement.

D.2: Constant Parameters

Table 10: Constant parameters used in all alternative calculations

Constant Parameters		
PM Particle Density	162.313	lb/ft^3
Dynamic Viscosity of Water*	0.000633	lb/ft-s
Gravitational Acceleration	32.17	ft/s^2
Specific Weight of Water	62.4	lb/ft^3

*Dynamic viscosity was calculated using recorded temperature of 23.5°C at site from previous studies (FDOT, 2021)

D.3: Design Calculations

D.3.1: PM Sedimentation and Separation Efficiency

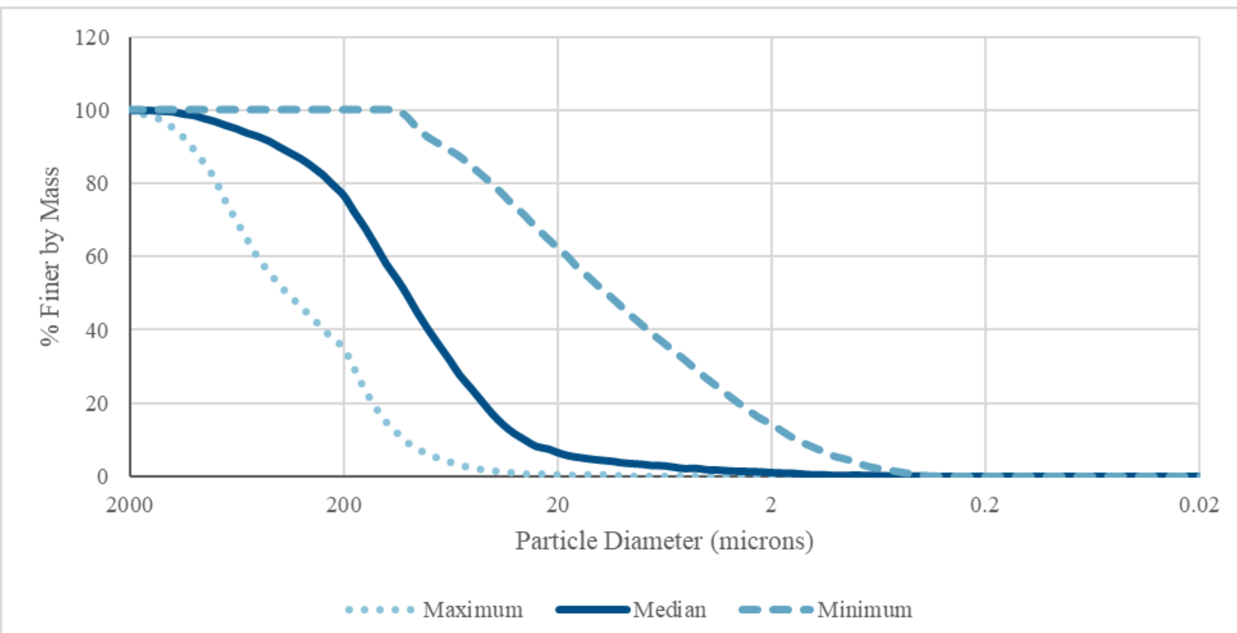


Figure 29: Particle size distribution (PSD) for the site, which was a heterodispersed sandy silt (SM) gradation with d_{50} = 102 µm and $\rho_s = 2.6\ g/cm^3$ (FDOT, 2021). This PSD is a typical one for the Rietz Union parking lot

The data in Figure 29 shows that most nutrients are bound to particles, with a heterogeneous distribution across particle sizes spanning three orders of magnitude. This variability is pivotal for our analysis, which considers factors beyond just particle size, making it multivariate rather than univariate. The specific gravity of the particles is measured at 2.56 g/cm^3, indicating their density.

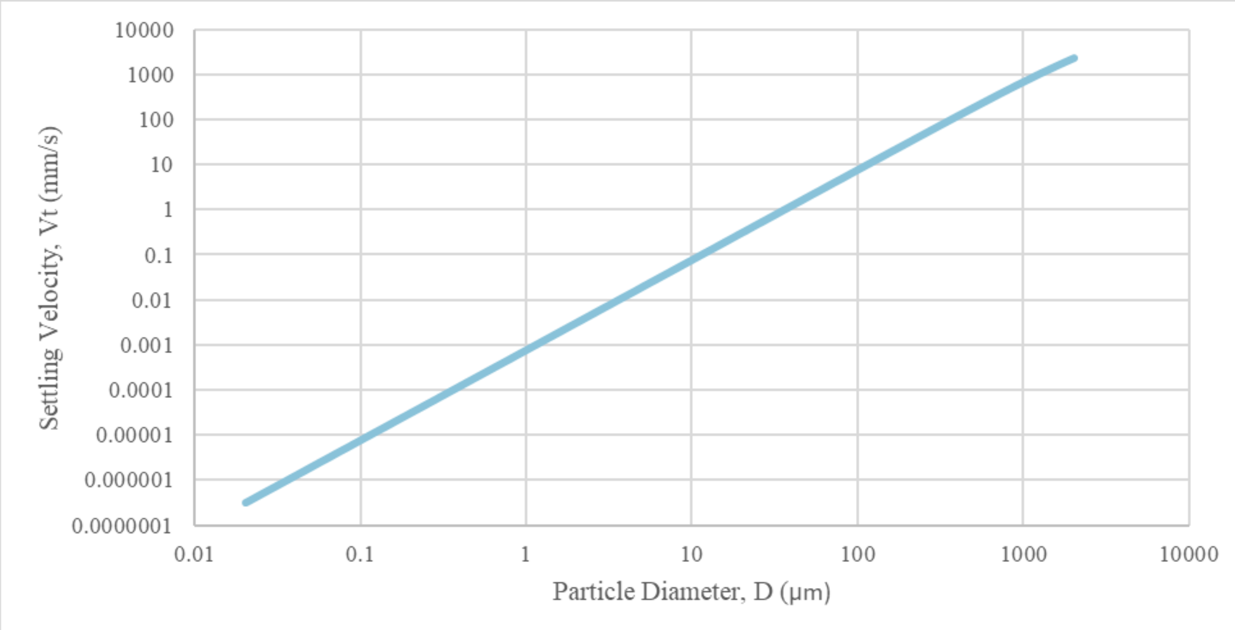


Figure 30: Settling velocities of PM using the median PSD (FDOT, 2021)

$$v_t = \left[\frac{4}{3}\frac{g(\rho_s - \rho)d}{C_d\rho}\right]^{\frac{1}{2}}$$

$$V_c = \frac{Q}{A_s}$$

$$(PM)_i \text{ separated} = (1 - X_c) + \int_0^{X_c} \frac{V_{si}}{V_c}\,dx$$

To achieve the goal of reducing nutrient-associated particulate matter (PM), clarification through particle settling is employed, utilizing Newton's Law of Settling to elucidate the process. Factors such as particle size, specific gravity, and turbulence significantly influence settling behavior, guiding the design of treatment alternatives. By considering particle mechanics, design alternatives for clarifier sizing are tailored to effectively address nutrient removal objectives.

D.3.2: Inlet Pipe Sizing

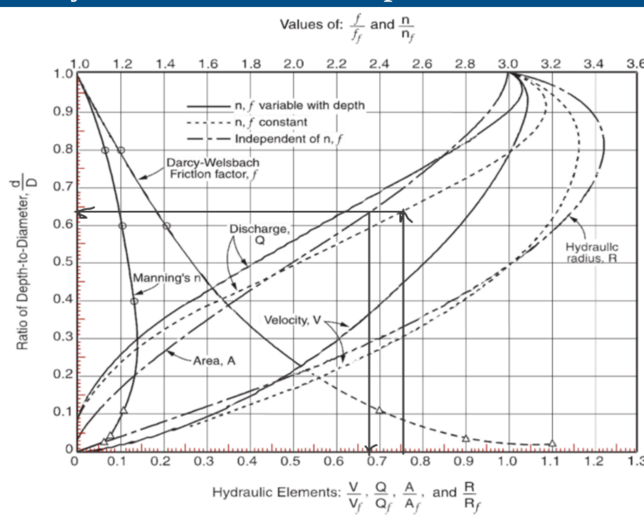
This pipe will be used for every design. It is centered at the entrance-side of the basin.

Table 11: Calculations for minimum pipe diameter

Pipe Diameter	
$\boldsymbol{Qpeak}$=19.46 $\boldsymbol{cfs}$	*Equation 8* $D = 1.333\left[\frac{nQ}{S^{\frac{1}{2}}}\right]^{\frac{3}{8}} = 1.349 \text{ ft} = 16.19069 \text{ in}$
$\boldsymbol{S}$=6%	
$\boldsymbol{n}$=0.013	

Because this would require a custom diameter pipe, a final size of 18" was selected as the inlet pipe because 18" is the next largest commercially available size.

Table 12: Calculating the values needed to check travel time for the water through the pipe

Full Pipe Flow Rate	
$\boldsymbol{n}$=0.013	*Equation 9* $Q_f = \frac{0.463}{n} D^{\frac{8}{3}} S^{\frac{1}{2}} = 25.721 \text{ cfs}$
$\boldsymbol{D}$=1.5 $\boldsymbol{ft}$	
$\boldsymbol{S}$=0.06	
Using the Hydraulic-Elements Graph	
$\boldsymbol{Qpeak/Qf}$=0.756 **(calculated)**	 *Figure 31: The hydraulic-elements graph with added lines from the values to the left (Eko Prasetyo, Fitriani, and Susanti, 2020)*
$\boldsymbol{d/D}$=0.642	
$\boldsymbol{A/Af}$=0.686	
Estimate Travel Time through Pipe	
$\boldsymbol{D}$=18 $\boldsymbol{in}$=1.5 $\boldsymbol{ft}$	*Equation 10* $A_f = \frac{\pi}{4} D^2 = 1.767\ sf$

A/Af=0.686	*Equation 11* $$A = \frac{A}{A_f} \times A_f = 1.212\ sf$$
$Qpeak$=19.46 cfs	*Equation 12* $$v_{peak} = \frac{Q_{peak}}{A} = 16.053 \frac{ft}{s}$$
A=1.212 sf	
L=50 ft	*Equation 13* $$T = \frac{L}{v} = 1.212\ s$$

Because travel time through the inlet pipe is just over one second, it can be ignored for this analysis.

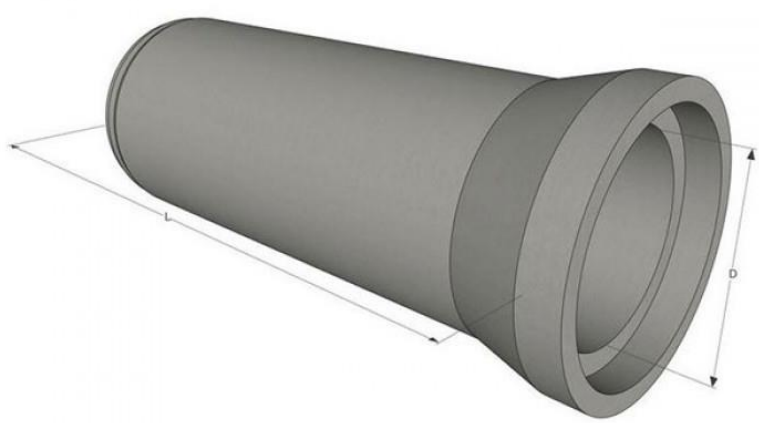


Figure 32: Pipe Design

D.3.3: Nutrient Reduction Strategy

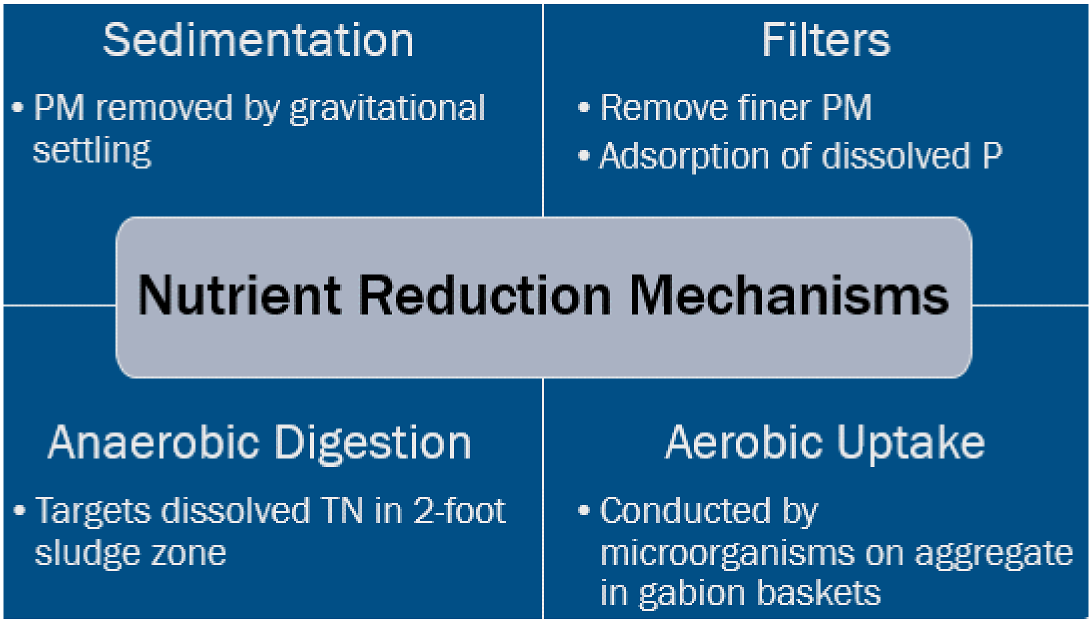


Figure 33: Nutrient Reduction Mechanisms

For our nutrient reduction strategy, we implement a serial combination of 4 mechanisms including sedimentation followed by filters, along with anerobic digestion and aerobic uptake.

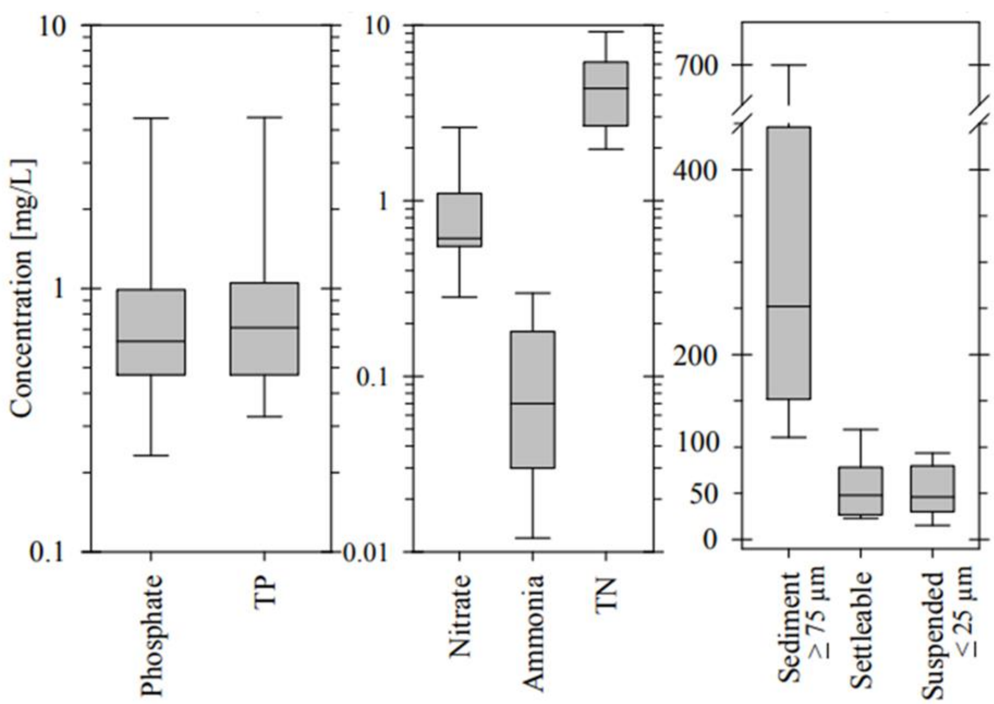


Figure 34: Nutrient (P, N) and Particulate Matter Concentration Distribution Found on Site (Kertesz et al., 2009)

The data collected from the site in Figure 34 reveal specific characteristics regarding orthophosphate, nitrogen, ammonia, and PSD (Particle Size Distribution). Orthophosphate and nitrogen are present in the expected proportions. Ammonia levels are noted to be low, which aligns with expectations as high concentrations of ammonia can be toxic, especially in the form of ammonium. The PSD analysis indicates that particles larger than 75 microns dominate the mass, with smaller particles being less prevalent, measuring below 50 microns. To assess the representativeness of the site, questions regarding its typicality and the adequacy of the values arise. The site is described as a vegetated watershed with a source area where biogenic material accumulates on impervious pavement, which is a common scenario. Additionally, elevated vegetated islands contribute to the inability to retain nitrogen, phosphorus, and particulate matter.

Table 13: Site Pollutant Loading Concentrations (FDOT, 2021)

Site Constituent Loading Concentrations	
Constituent	**Median Site Value**
Total Nitrogen (TN)	4.52 (mg/L)
Total Phosphorus (TP)	3.23 (mg/L)
PM D_{50}	0.102 (mm)

D.3.3.1: Filters

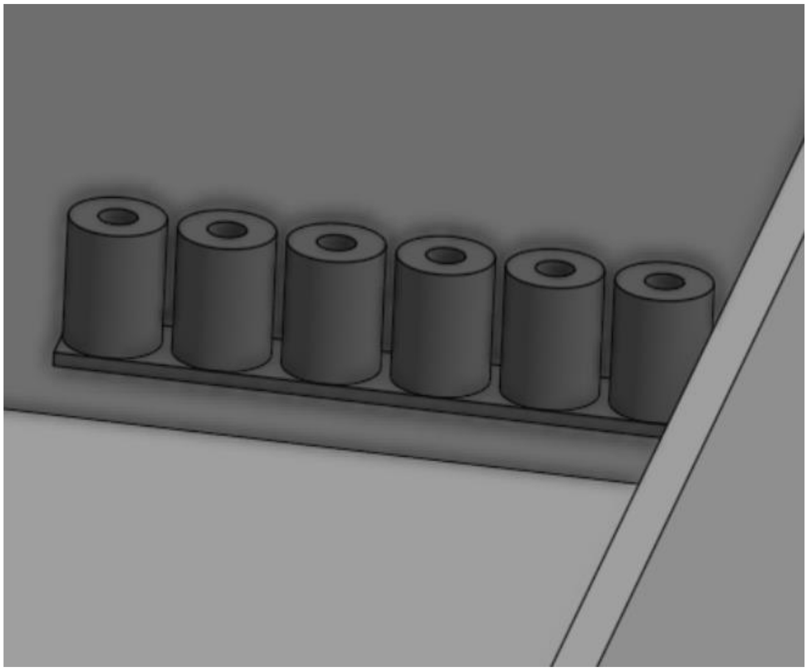


Figure 35: Filter Design Recommendation

Each filter cartridge is designed with a height of 1.85 feet and a diameter of 1.5 ft.

Equation 14: Ergun Equation

$$\frac{\Delta H}{L} = \frac{k_o (L_e/L)^2 \mu}{\rho g} \frac{(1-\eta_m)^2}{{\eta_m}^3} \left(\frac{a}{v}\right)^2 V + k_2 \frac{1-\eta_m}{{\eta_m}^3} \left(\frac{a}{v}\right) \frac{V^2}{g}$$

ΔH/L = head loss per unit depth of media bed (mm/mm); k0 = shape factor; μ = fluid viscosity (N-s/m^2); ρ = fluid density (g/cm^3); g = acceleration of gravity (9.81 m/s^2); ηm = bed macro porosity (the pore volume between the packed media); a/v = media surface area per unit of media volume (m^{-1}); V = superficial velocity (m/s); k2 = dimensionless Ergun constant (k2) (0.5 for angular porous media); Φ = sphericity factor (<1); d_p = measure of media granular diameter (μm); L = depth of media bed in the direction of radial flow; and Le = length of actual flow pathway through the depth of media bed (Sansalone et al., 2009).

Equation 15: Particulate fraction of TN

$$Particulate\ Fraction\ of\ TN = \frac{(778\ + 298\ + 2{,}018\,)\ ^{\mu g}/_{L}}{4{,}523\ ^{\mu g}/_{L}} = 0.40$$

Used event-based concentrations from Table 14 to determine the fraction of TN that is bound to particulates.

Equation 16: First-Order Kinetics

$$[C] = \{[C]_0\}e^{-k(t)}$$

$[C]_0$ = Influent concentration of Nitrate = 3.81 mg/L; k_0 = 0.0102; R^2 = 0.97; t = clarifier detention (hours)

This equation was utilized to calculate the effluent concentration, which was then compared to the influent concentration to determine the removal efficiency of anaerobic denitrification in the sludge zone of the clarifier (Kertesz et al., 2009).

Table 14: Event-Based Concentrations of Suspended, Settleable, Sediment, Dissolved Nitrogen, TN, and Rainfall Expressed as Event-Mean Concentration (Zhang and Sansalone, 2014)

2008 events	Sediment N (ug/L)	Settleable N (ug/L)	Suspended N (ug/L)	Dissolved N (ug/L)	Total N (ug/L)	Rainfall TN (ug/L)
Median	778	298	716	2,018	4,523	575
Mean	1,161	383	788	2,386	4,717	646
Std Dev	1,414	311	550	1,301	2,631	143

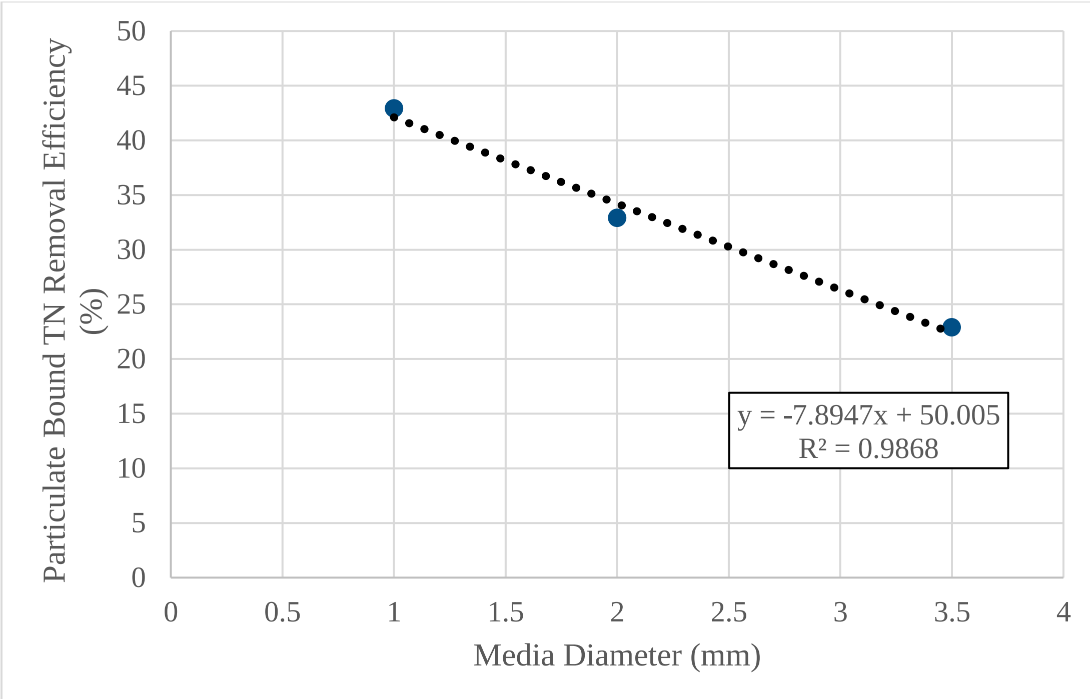


Figure 36: The Relationship between Removal Efficiency Progression and Filter Media Diameter (Liu et al., 2010)

D.3.3.2: Anaerobic Digestion

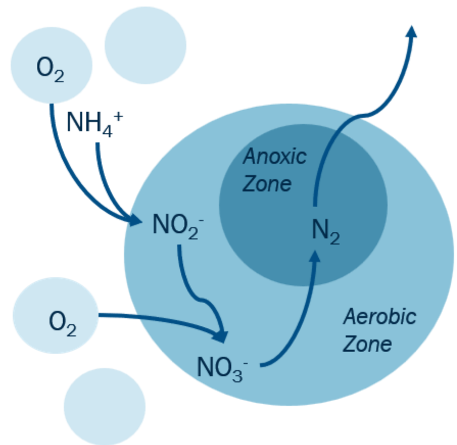


Figure 37: Denitrification Process (Hazard et al., 2018)

Each proposed alternative includes a 2-foot-deep sludge and anaerobic zone which allows settled particles to accumulate without compromising treatment volume. Moisture in the zone creates an anoxic environment, aiding denitrification.

Table 15: Nitrification and denitrification chemical equations

Nitrification	
Ammonia Oxidation	$NH_3 + O_2 \rightarrow NO_2^- + H_2O + 2H^+$
Nitrate Oxidation	$NO_2^- + O_2 \rightarrow NO_3^-$
Denitrification	
	$2NO_3^- \rightarrow 2NO_2^- \rightarrow 2NO \rightarrow N_2O \rightarrow N_2 + O_2$

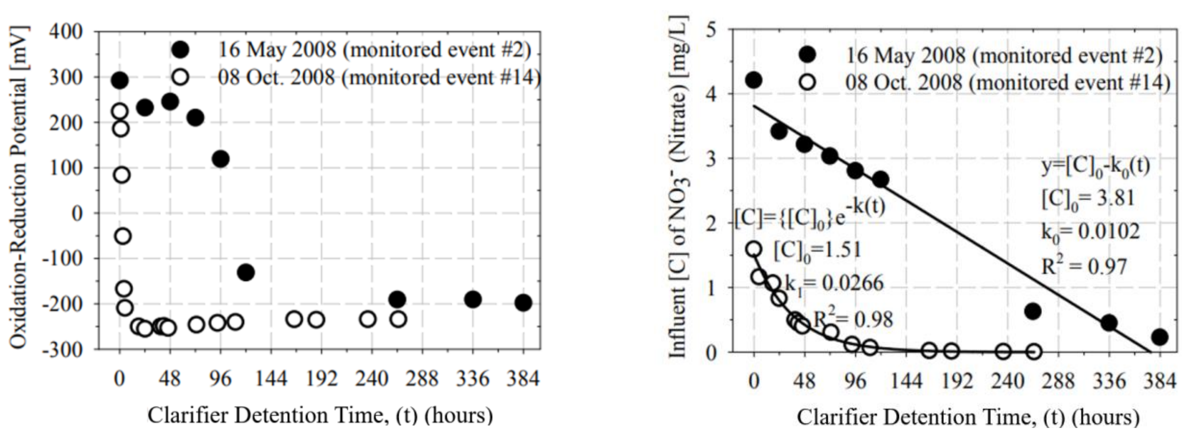


Figure 38: Redox as a Function of Runoff Detention in Subsurface Filtration (BMPs Contain High Microbial Activity for Electron Transfers) (Kertesz et al., 2009)

D.3.3.3: Aerobic Uptake

Baffles at basin scale:

- Volumetric utilization
- Tortuosity increased
- Plug flow increased
- Residence time distribution improved

Baffles at gabion scale:

- Hydraulically conductive
- Tortuous effective porosity
- PM filtration
- Horizontal trickling filter

Baffles at CRC media scale:

- Higher surface area, pH substrate
- C-S-H, $Ca(OH)_2$, CSA substrate
- Adsorption, chemical precipitation
- Mass transfer of phosphorus to CRC

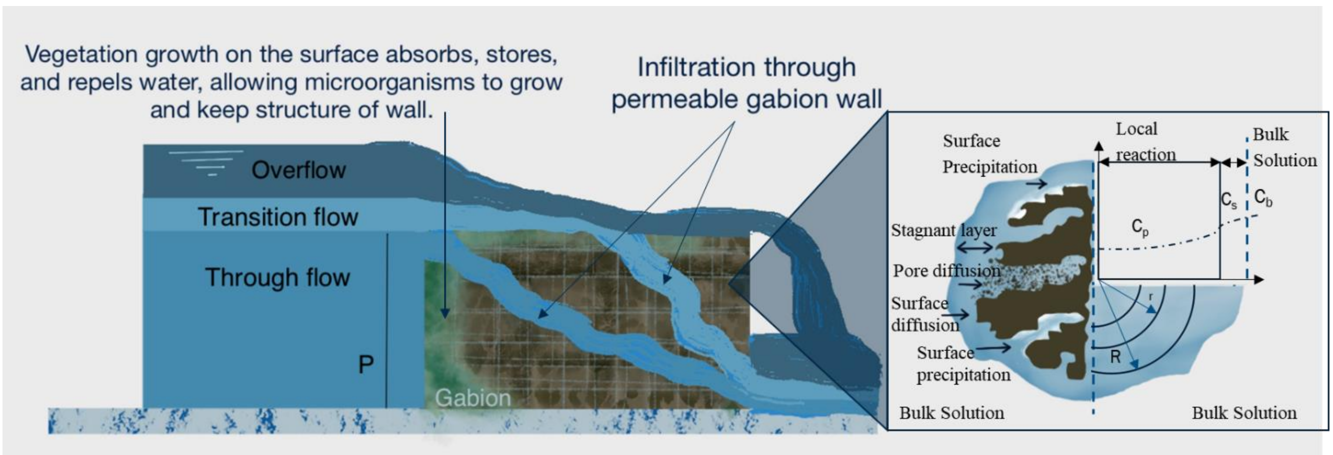


Figure 39: Gabion Wall Mechanisms

Appendix E: Alternative Development and Design

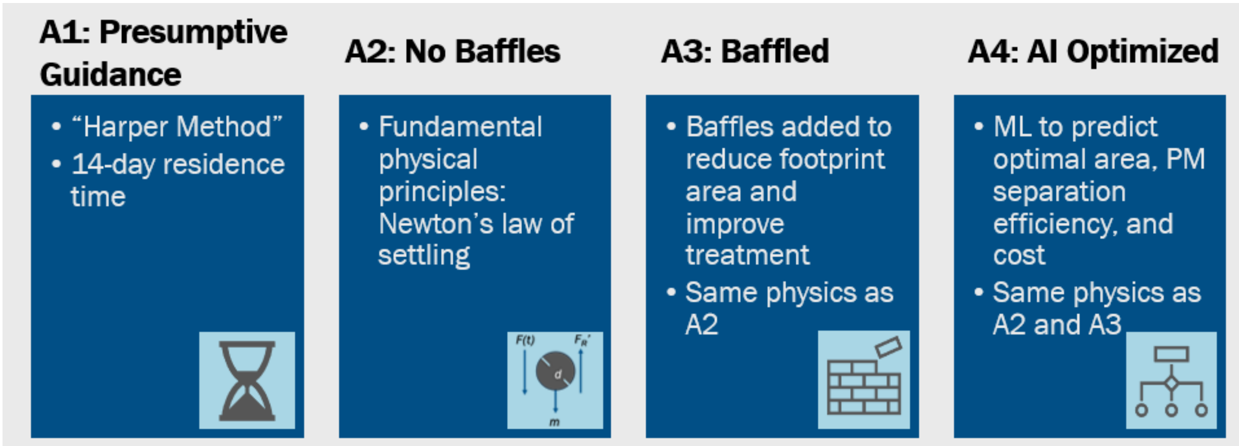


Figure 40: Proposed Geometric Design Options

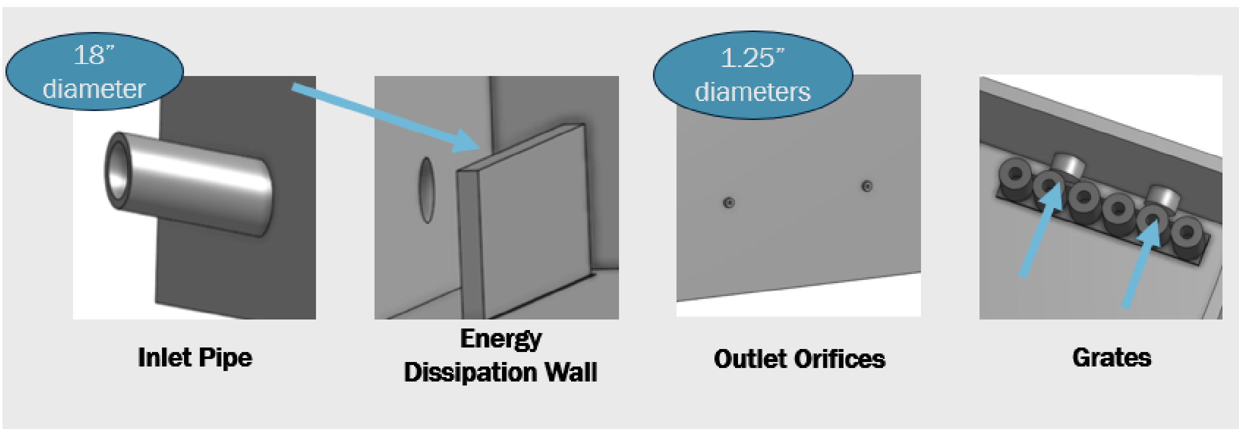


Figure 41: Constant Design Features Across Alternatives

E.1: Alternative 1: Presumptive Guidance

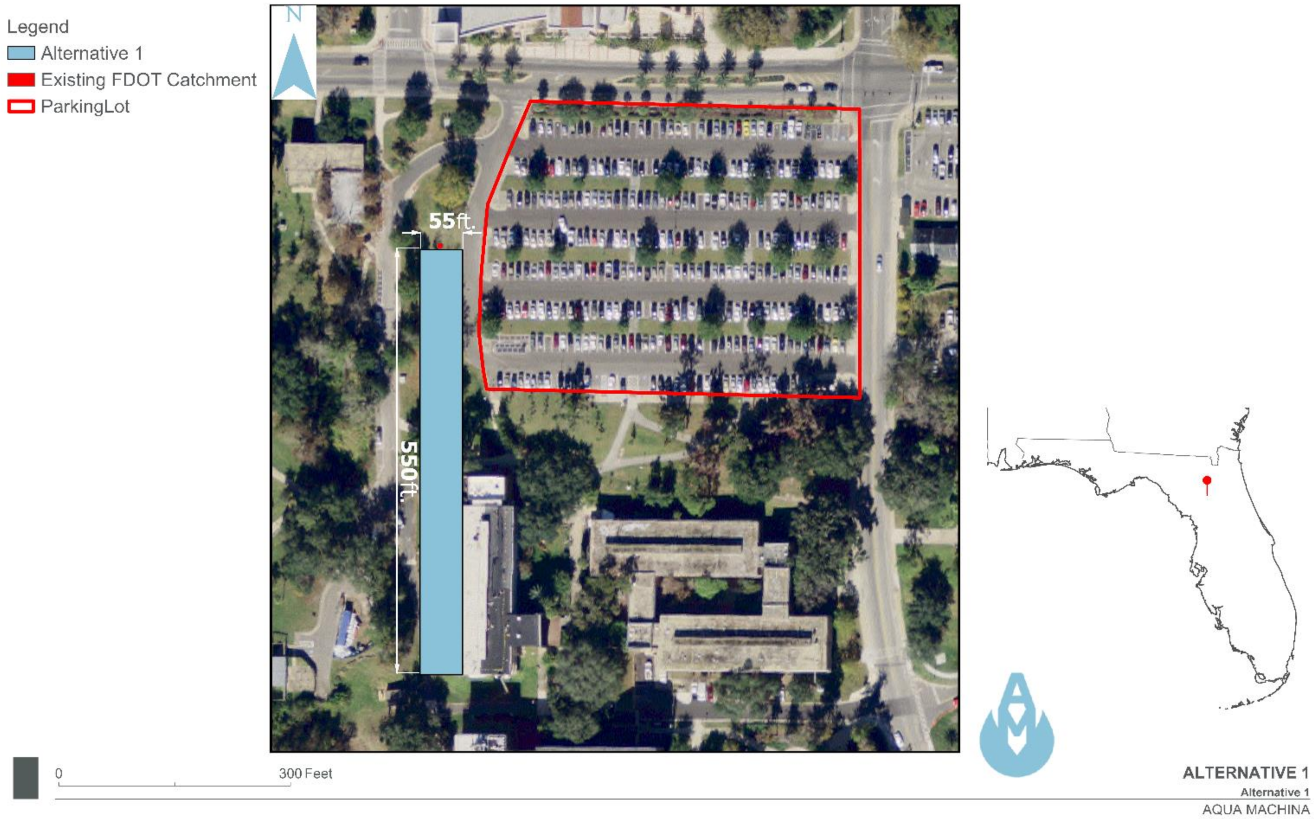


Figure 42: Alternative 1 Footprint with Details (ESRI)

Table 16: Alternative 1 PM, TN, and TP Removal Efficiencies

	Removal Efficiency	Sedimentation and Anaerobic Digestion	Filters (finer PM)	Filters (adsorption)
PM	99.0%	✓		
TN	99.0%	✓		
TP	77.0%	✓		

Table 17: Alternative 1 Design Summary

ID	L (ft)	W (ft)	H (ft)	A (ft^2)	# of Baffles	Reduction Goal (80%)
A1	550	55	8	30,250	-	No

E.1.1: Design Parameters

*Table 18: Parameters used in Alternative 1 design. *Total basin storage includes the total available water storage, which does not include the 1 foot of freeboard, but considers the volume of the wall for energy dissipation, and the filters*

Fixed Dimensions		
Orifice Diameter (2 orifices at 2 ft height):	0.37	in
Depth	8	ft
L:W Ratio	10:1	
Minimum Dimensions to Meet Storage Requirements		
Storage Required from Inflow	150,000	cf
Surface Area	30,000	sf
Length	548	ft
Width	54.8	ft
Design Dimensions		
Length	550	ft
Width	55.0	ft
Surface Area	30,250	sf
Wall for Energy Dissipation		
Length	0.50	ft
Width	44.0	ft
Height	1.50	ft
Total Basin Storage*	150,200	cf
Design Metrics		
Max stage	2.35	ft
Time to outflow peak	37.4	mins
Peak outflow rate	0.0116	cfs
Residence time	14.1	days
Particle removal efficiency through settling	99.0	%

Table 19: Presumptive Guidance Calculations (Initial Volume)

Calculations			
	Value	**Units**	**Notes**
Target Removal Efficiency Minimum	80%		
Watershed Area	3.3	acres	
Watershed Area	144,000	sf	
Impervious% of watershed	75.6%		
DCIA% of Impervious	100%		(DCIA = Directly Connected Impervious Area; assuming 100% because it is a parking lot)
CN for impervious area	98		
CN for non-impervious area	39		
DCIA% of Watershed	75.6%		
nDCIA%	0.00%		
nDCIA CN	39		
Dry retention depth	1	in	(Found in Table B.4 of Harper and Baker, 2003)
Dry retention depth	0.0833	ft	
Actual efficiency	81.0%		(Found in Table B.4 of Harper and Baker, 2003)
Dry Retention volume	11,950	cf	
Required volume	12,021.7	cf	(Adding volume from filter and energy dissipator, which cannot be used by the water)
Pond depth	5	ft	
Surface area	2404	sf	
L:W	10		
W =	15.5	ft	
L =	155	ft	

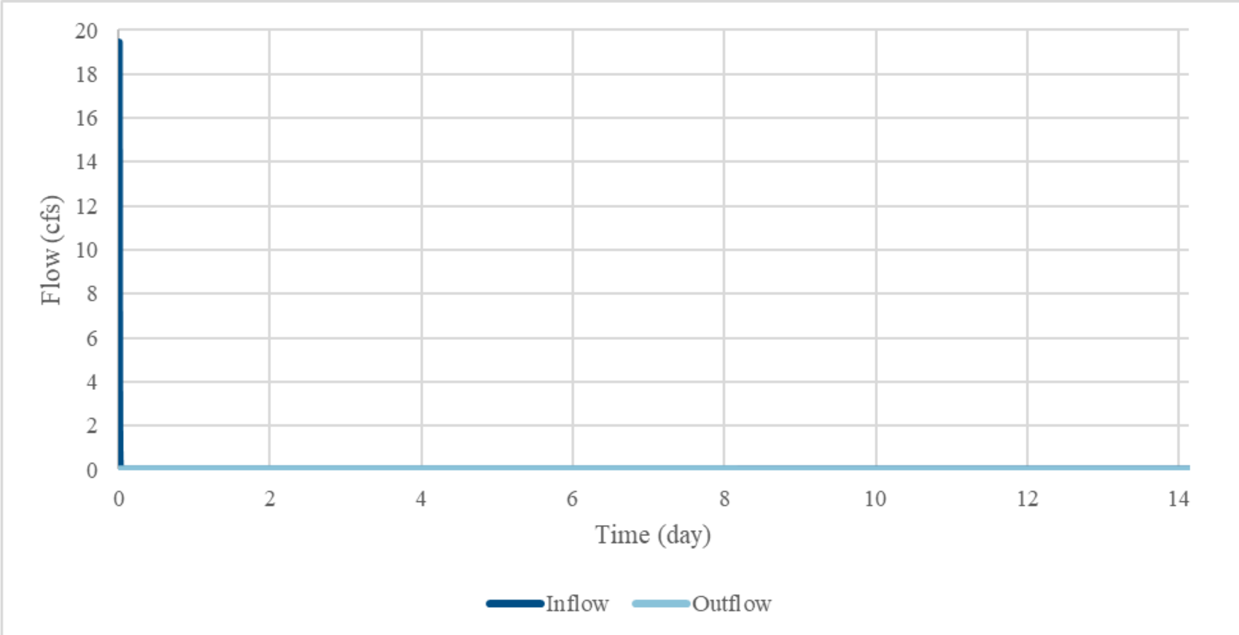


Figure 43: Inflow/Outflow hydrograph for Alternative 1, using the design storm described in C.1.2: Model and Design Storms. The figure below shows a zoomed in portion where the intersection of these lines is.

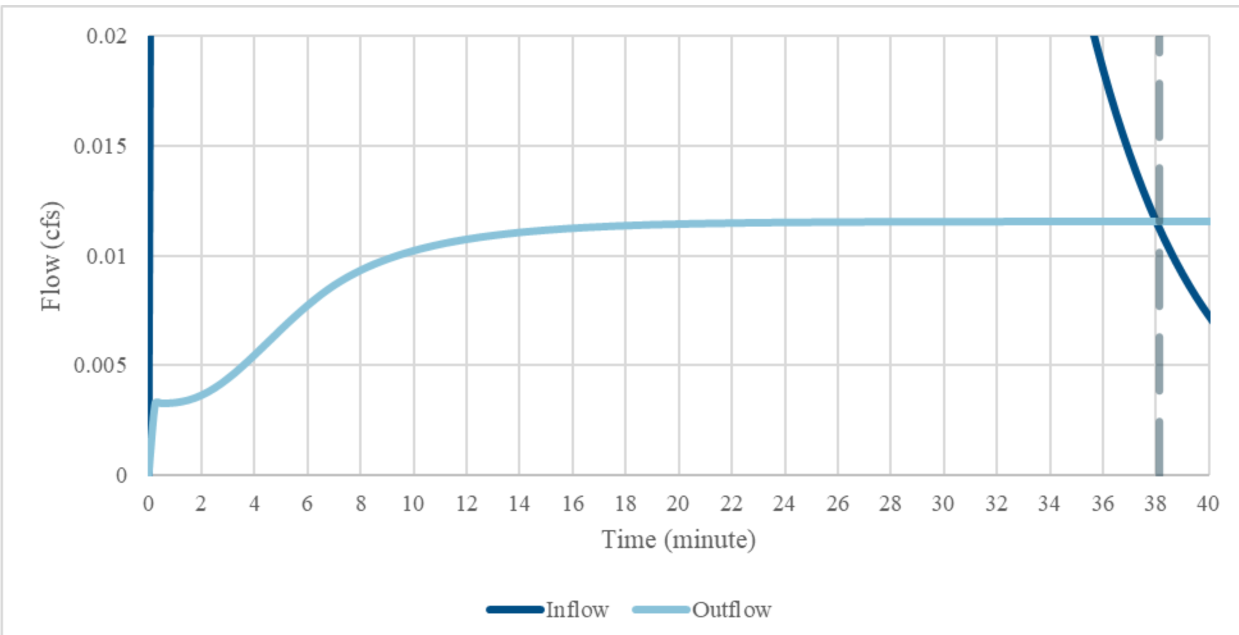


Figure 44: A zoomed in portion of the inflow/outflow hydrograph above. It shows the first 40 minutes of the storm, and up to 0.02 cfs. A dashed line was added at the time to peak outflow, at 37.25 minutes.

E.1.2: Design Method

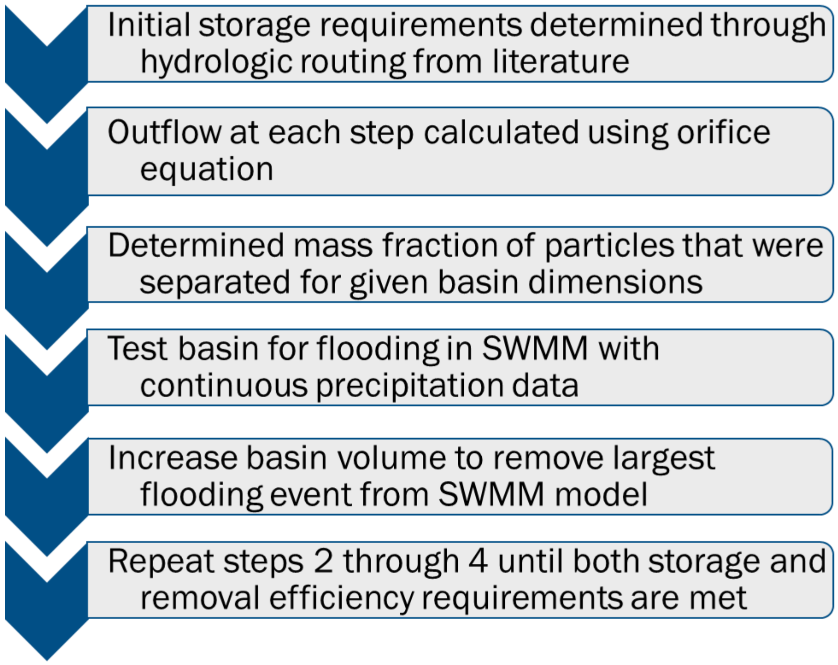


Figure 45: Presumptive Guidance Design Process

Because the watershed in question is a parking lot, 100% of the impervious area is also directly connected impervious area (DCIA). As such, it comprised 75.61% of the watershed.

Using the composite curve number and this DCIA percentage, a retention depth of 1 inch is found by referencing Table B.4 in Evaluation of Alternative Stormwater Regulations for Southwest Florida (Harper and Baker, 2003).

Multiplying that depth by the watershed area gives the total clarifier volume needed.

Equation 17

$$V_{storage} = A_{WS} \times d_{table} = 11{,}979\ cf$$

Using the predetermined settling zone depth of 5 feet, the surface area was found.

Equation 18

$$SA = \frac{V_{clarifier}}{h_{settling\ zone}} = 2{,}397.673\ sf$$

Using a predetermined L:W ratio of 10:1, the length and width could be found.

Equation 19

$$w_{clarifier} = \sqrt{\frac{SA}{10}} = 15.478\ ft$$

Equation 20

$$l_{clarifier} = 10w = 154.78\ ft$$

With an initial width, the volume can be recalculated by finding the volume of the clarifier that would be occupied by the energy dissipator. The dissipator is 6 inches thick and will be tall enough to cover the entire height of the inlet pipe, making it 3.5 feet tall. Only 1.5 feet of that will be within the settling zone and modify the needed volume. The width of the dissipator is 3 feet less than the width of the clarifier, with 1.5 feet on either side.

Equation 21

$$V_{dissipator} = 0.5\ ft \times 1.5\ ft \times \left(w_{clarifier} - 3\ ft\right) = 9.363\ cf$$

This volume is added to the needed storage found in Equation 1717 which gives a minimum volume of 11988.36 cubic feet. From there, the final surface area, length, and width are found.

Equation 22

$$SA_{SWMM0} = \frac{V_{SWMM0}}{5\ ft} = 2{,}397.67\ sf$$

Equation 23

$$w_{SWMM0} = \sqrt{\frac{SA_{SWMM0}}{10}} = 15.484\ ft$$

Equation 24

$$l_{SWMM0} = 154.844\ ft$$

A SWMM model was then generated based on the area of the parking lot. The parameters held constant across different simulations are shown below in Table 20. Historical precipitation data in Gainesville, FL from NOAA's NCDC datasets was used for a continuous simulation using the Kinetic Wave method (NOAA National Climatic Data Center, 2001). Any parameter used by SWMM but not mentioned was left in its default setting.

Table 20: The constant parameters in the SWMM model. Any parameters not stated were held default. The ones that changed were the length and width of the basin, and the maximum depth of the conduits connecting the basin to the outlets

Parameter	Value	Units
Subcatchment Lot 1		
Area	3.3	ac
Width	165	ft
% Slope	2.7[+]	%
Manning's n for impervious area	0.01	
Manning's n for pervious area	0.1	
Junction J1		
Invert El.	115	ft
Conduit C1		
Inlet Node	J1	
Outlet Node	Basin1	
Shape	Circular	
Max. Depth	1.5	ft
Length	50	ft
Roughness	0.01	
Inlet Offset	0	ft
Outlet Offset	2	ft
Storage Unit Basin1		
Invert El.	105	ft
Initial Depth	2	ft
Surcharge depth	1	ft
Evap. Factor	1	
Daily Evaporation (Constant)	0.169	in/day
Storage Shape	Pyramidal	
Side Wall Slope	0	
Conduits C2 and C3		
Inlet Node	Basin1	
Outlet Node	Out1	
Shape	Circular	
Length	1	ft
Inlet Offset	2	ft
Outlet Offset	0	ft
Outfalls Out1 and Out2		
Invert El.	105.93	ft

[+] While 2.7% was not the value used to calculate concentration time, it is the average slope across the entire parking lot.

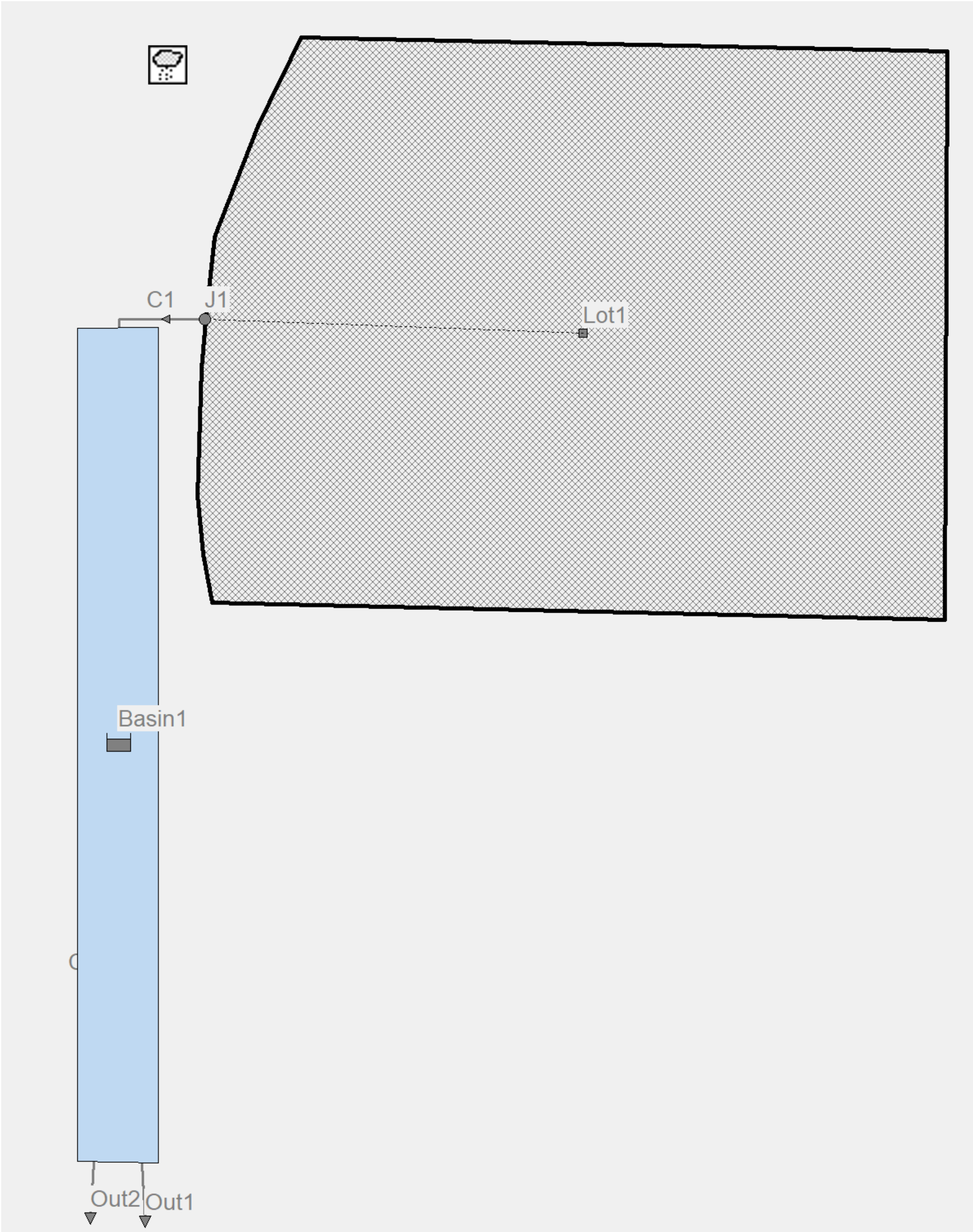


Figure 46: Layout of the SWMM model, showing the basin, parking lot, and all junctions. The size of the basin on the map reflects the final size of the basin.

Multiple simulations were conducted, each time checking to see if Basin 1 experienced any flooding. If it did, the largest individual flood event was found. The volume of flooded water was calculated based on the flooding rate provided by SWMM at each hourly time step. That volume was then added to V_{SWMM0} and any previous iterations' volumes. The orifice diameter for the basin was then adjusted until the design storm had a residence time of 14 days with the new volume. Ultimately, this process was repeated for a total of 6 different basin dimensions, including both the initial found above, and the final. The flood volume calculations are shown below in Table 21 through Table 25. An example of a flooding event in the SWMM model is shown in Figure 47: The largest flooding event, ranging from an hour before precipitation began to 24 hours later. This is the largest flooding event for the first set of dimensions, 155 ft x 15.5 ft x 5 ft with two 0.37 inch orifices.

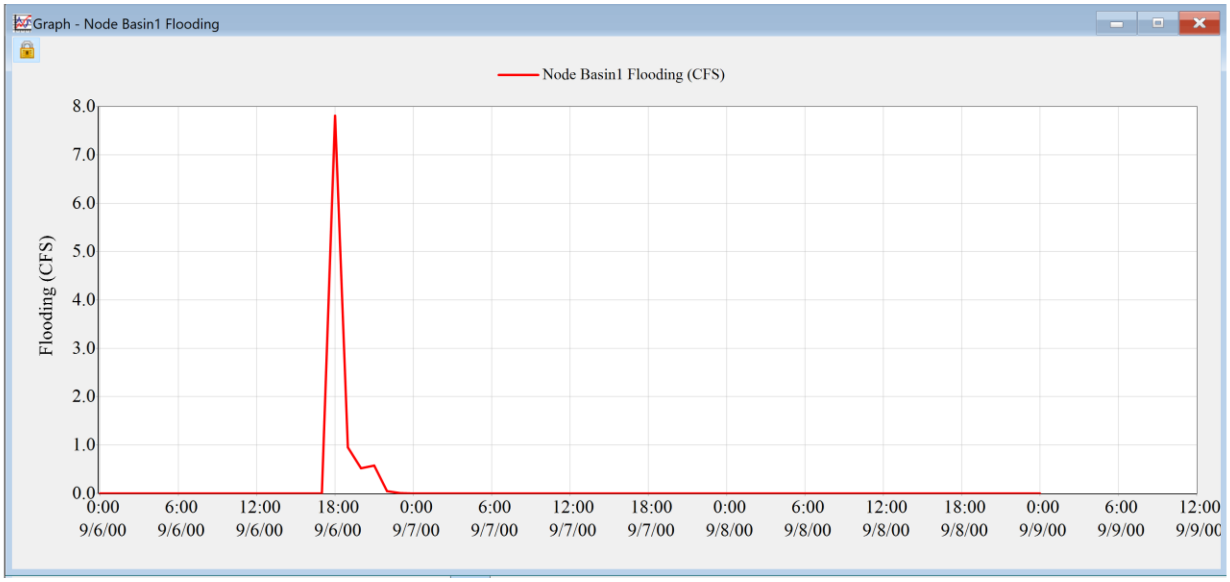


Figure 47: The largest flooding event, ranging from an hour before precipitation began to 24 hours later. This is the largest flooding event for the first set of dimensions, 155 ft x 15.5 ft x 5 ft with two 0.37 inch orifices

Table 21: The largest flooding event from the first dimensions for A1

BASIN SIZE: 155 FT X 15.5 FT X 5 FT		
Time	Flooding (cfs)	Cumulative flood vol (cf)
9/6/2000 16:00	0	---
9/6/2000 17:00	0	0
9/6/2000 18:00	7.81	28116
9/6/2000 19:00	0.96	31572
9/6/2000 20:00	0.52	33444
9/6/2000 21:00	0.58	35532
9/6/2000 22:00	0.05	35712
9/6/2000 23:00	0.01	35748
9/7/2000 0:00	0	35748
9/7/2000 1:00	0	35748
9/7/2000 2:00	0	35748
9/7/2000 3:00	0	35748

Table 22: The largest flooding event from the second dimensions for A1

BASIN SIZE: 301 FT X 30.1 FT X 5 FT		
Time	Flooding (cfs)	Cumulative flood vol (cf)
7/31/2013 15:00	0	-----
7/31/2013 16:00	0	0
7/31/2013 17:00	6.65	23940
7/31/2013 18:00	3.88	37908
7/31/2013 19:00	0.24	38772
7/31/2013 20:00	0.1	39132
7/31/2013 21:00	0	39132
7/31/2013 22:00	0	39132
7/31/2013 23:00	0	39132
8/1/2013 0:00	0	39132
8/1/2013 1:00	0	39132
8/1/2013 2:00	0	39132

Table 23: The largest flooding event from the third dimensions for AI

BASIN SIZE: 417 ft X 41.7 ft X 5 ft		
Time	Flooding (cfs)	Cumulative Flood Vol (cf)
6/24/2012 22:00	0	---
6/24/2012 23:00	0	0
6/25/2012 0:00	0	0
6/25/2012 1:00	0	0
6/25/2012 2:00	4.68	16848
6/25/2012 3:00	0.24	17712
6/25/2012 4:00	0.28	18720
6/25/2012 5:00	0.53	20628
6/25/2012 6:00	0.86	23724
6/25/2012 7:00	0.01	23760
6/25/2012 8:00	0	23760
6/25/2012 9:00	0	23760

Table 24: The largest flooding event from the fourth dimensions for A1

BASIN SIZE: 470 FT X 47 FT X 5 FT		
Time	Flooding (cfs)	Cumulative Flood Vol (cf)
9/7/2004 3:00	0	---
9/7/2004 4:00	0	0
9/7/2004 5:00	0	0
9/7/2004 6:00	0	0
9/7/2004 7:00	0.22	792
9/7/2004 8:00	0.82	3744
9/7/2004 9:00	2.44	12528
9/7/2004 10:00	0.72	15120
9/7/2004 11:00	0	15120
9/7/2004 12:00	0.09	15444
9/7/2004 13:00	0	15444
9/7/2004 14:00	0	15444
9/7/2004 15:00	0	15444
9/7/2004 16:00	0	15444
9/7/2004 17:00	0	15444
9/7/2004 18:00	0	15444
9/7/2004 19:00	0	15444
9/7/2004 20:00	0	15444
9/7/2004 21:00	1.7	21564
9/7/2004 22:00	0.02	21636
9/7/2004 23:00	0	21636
9/8/2004 0:00	0	21636
9/8/2004 1:00	0	21636
9/8/2004 2:00	0	21636
9/8/2004 3:00	0	21636

Table 25: The largest flooding event from the fifth dimensions for AI

BASIN SIZE: 515 FT X 51.5 FT X 5 FT		
Time	Flooding (cfs)	Cumulative flood vol (cf)
9/7/2004 3:00	0	---
9/7/2004 4:00	0	0
9/7/2004 5:00	0	0
9/7/2004 6:00	0	0
9/7/2004 7:00	0	0
9/7/2004 8:00	0	0
9/7/2004 9:00	2.44	8784
9/7/2004 10:00	0.72	11376
9/7/2004 11:00	0	11376
9/7/2004 12:00	0.09	11700
9/7/2004 13:00	0	11700
9/7/2004 14:00	0	11700
9/7/2004 15:00	0	11700
9/7/2004 16:00	0	11700
9/7/2004 17:00	0	11700
9/7/2004 18:00	0	11700
9/7/2004 19:00	0	11700
9/7/2004 20:00	0	11700
9/7/2004 21:00	1.7	17820
9/7/2004 22:00	0.02	17892
9/7/2004 23:00	0	17892
9/8/2004 0:00	0	17892
9/8/2004 1:00	0	17892
9/8/2004 2:00	0	17892
9/8/2004 3:00	0	17892

E.1.3: Drawings

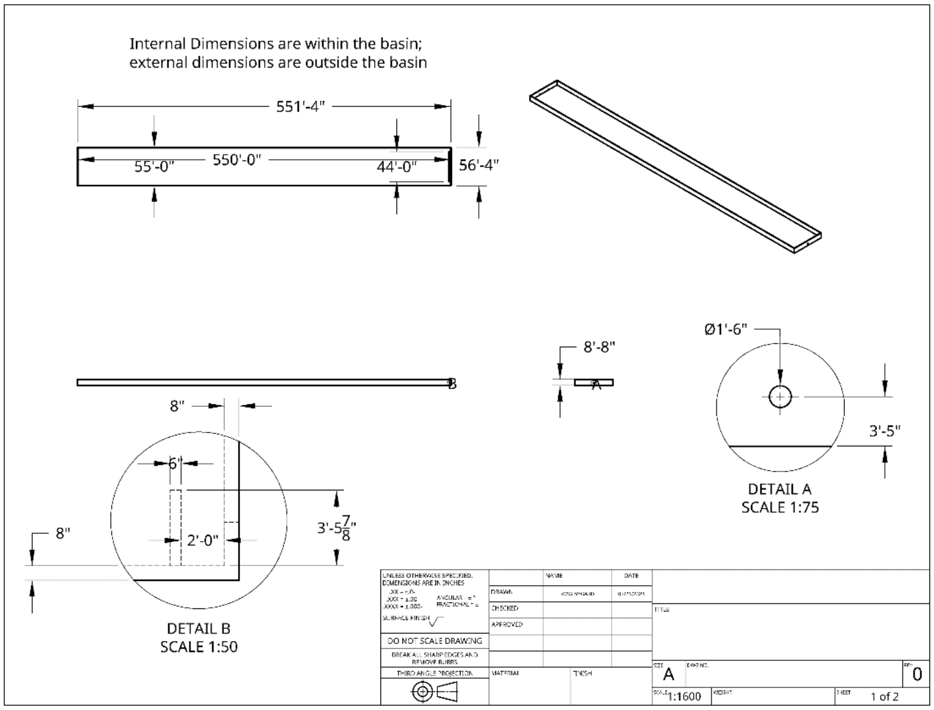


Figure 48: Page 1 of the engineering drawings for Alternative 1

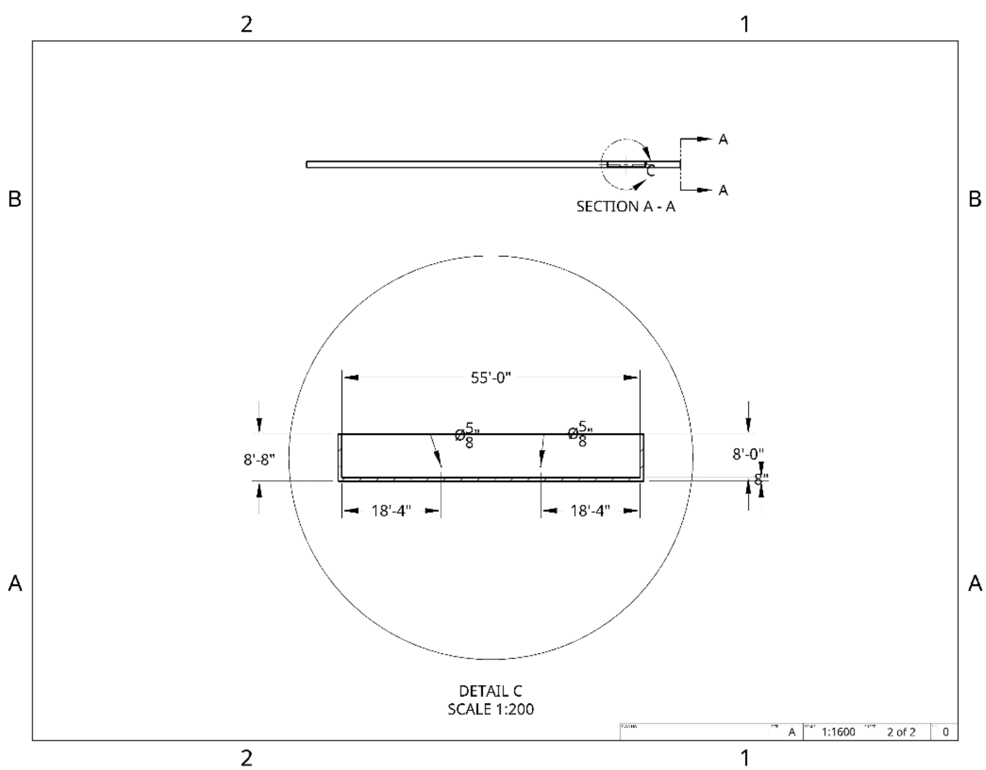


Figure 49: Page 2 of the engineering drawings for Alternative 1

E.1.4: Cost Estimates

Table 26: Alternative 1 Full Detailed Cost Estimate

Alternative 1 Full Cost Estimation Details					
No.	Line Item	Quantity	Unit	Cost/Unit	Total
Alternative 1					
1	Mobilization	-	Lump Sum	-	$48,900.00
2	Erosion and Sediment Control	-	Lump Sum	-	$24,500.00
3	Demolition	869	S.Y.	$10.24	$8,900.00
4	Excavation and Fill	6857	C.Y	$16.68	$114,400
5	Land Valuation	0.8	Acre	$100,044.40	$83,600.00
6	Grub Stump and Remove	0.8	Acre	$1,914.73	$1,600.00
7	Selective Tree Removal	9.0	Each	$388.89	$3,500.00
8	Grading	36400	S.F.	$0.08	$2,900.00
9	8" Concrete with Epoxy Coated Steel Rebar	896	C.Y.	$226.04	$202,600.00
10	6" Concrete with Rebar	1	C.Y.	$160.00	$160.00
11	18" Concrete Pipe	22	L.F.	$46.51	$1,000.00
12	Inlet Structure	1	Each	$7,000.00	$7,000.00
13	Outlet Structure	2	Each	$3,550.00	$7,100.00
14	Filter	5	Each	$1,480.00	$7,400.00
15	Fence	1226	L.F.	$29.12	$35,700.00
16	Landscaping	503	S.Y.	$25.06	$12,600.00
				Subtotal	$561,900.00
				Engineering (15%)	$84,300.00
				Permit and Insurance (5%)	$28,100.00
				Subtotal	$674,300.00
				Contingency (10%)	$67,400.00
				Total	$741,700

E.2: Alternative 2: No Baffles

Figure 50: Alternative 2 Footprint (ESRI)

Table 27: Alternative 2 PM, TN, and TP Removal Efficiencies

	Removal Efficiency	Sedimentation and Anaerobic Digestion	Filters (finer PM)	Filters (adsorption)
PM	99.1%	✓	✓	
TN	80.0%	✓	✓	
TP	90.0%	✓	✓	✓

Table 28: Alternative 2 Design Summary

ID	L (ft)	W (ft)	H (ft)	A (ft^2)	# of Baffles	Reduction Goal (80%)
A2	150	15	8	2,250	-	Yes

E.2.1: Design Parameters

*Table 29: Parameters used in Alternative 2 design. *Total basin storage includes the total available water storage, which considers 1 foot of freeboard, the volume of the wall for energy dissipation, and the filters*

Fixed Dimensions		
Orifice Diameter (2 orifices at 2 ft height):	1.25	in
Depth	8	ft
L: W Ratio	10:1	
Minimum Dimensions to Meet Storage Requirements		
Storage Required from Inflow	9041.5	cf
Surface Area	1808.3	sf
Length	134.473	ft
Width	13.4473	ft
Design Dimensions		
Length	150	ft
Width	15	ft
Surface Area	2250	sf
Wall for Energy Dissipation		
Length	0.5	ft
Width	12	ft
Height	3.5	ft
Total Basin Storage*	15,720	cf
Design Metrics		
Max stage	5.91	ft
Time to outflow peak	27	mins
Residence time	32.6	hrs
Settling removal efficiency	98.5	%

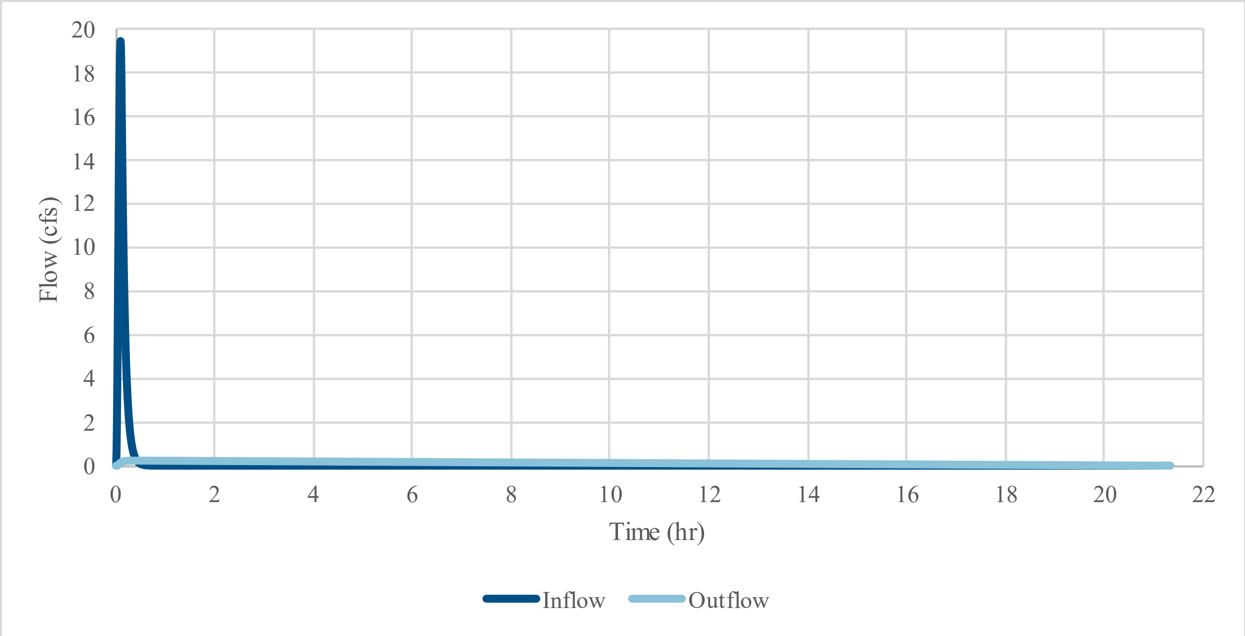


Figure 51: Inflow/outflow hydrograph in which inflow is from the design storm and outflow is out of two orifices at a height of 2 feet

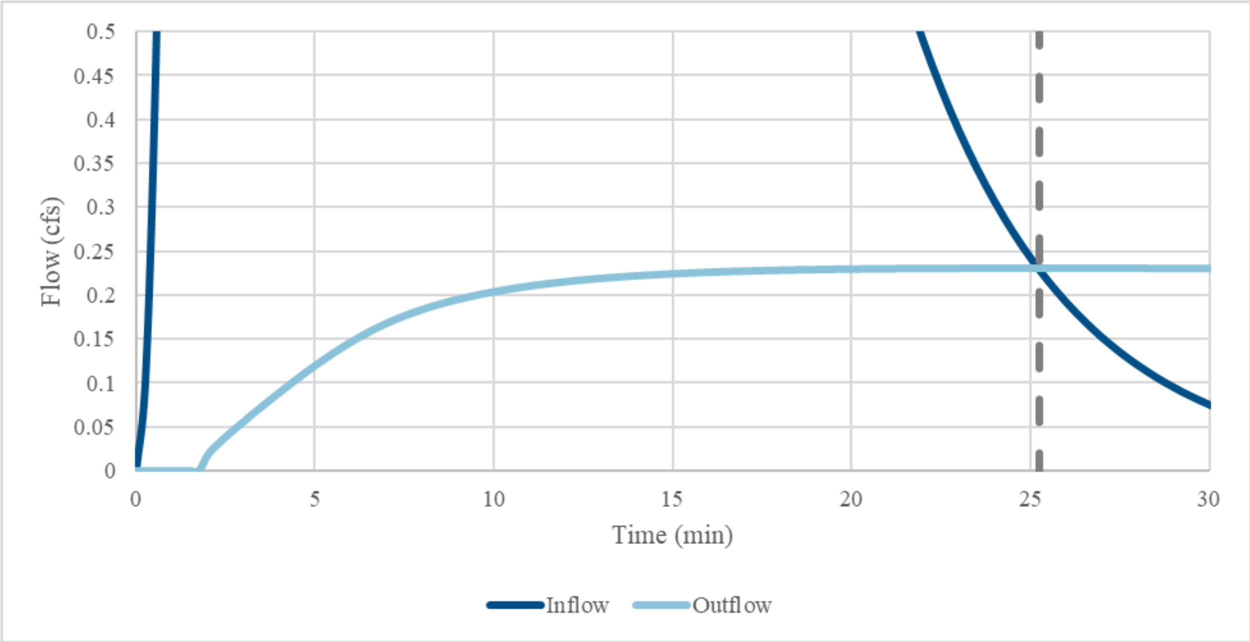


Figure 52: A zoomed in portion of the inflow/outflow hydrograph above. It shows the first 30 minutes of the storm, and up to 0.5 cfs. A dashed line was added at the time to peak outflow of 25.25 min

E.2.2: Design Method

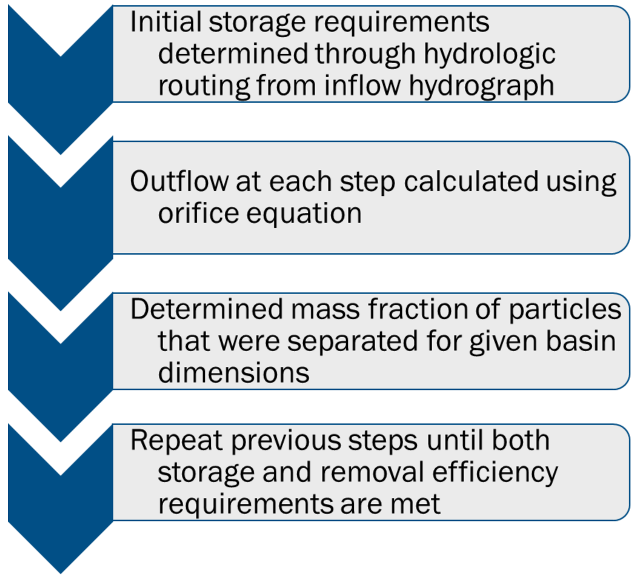


Figure 53: No Baffles Design Process

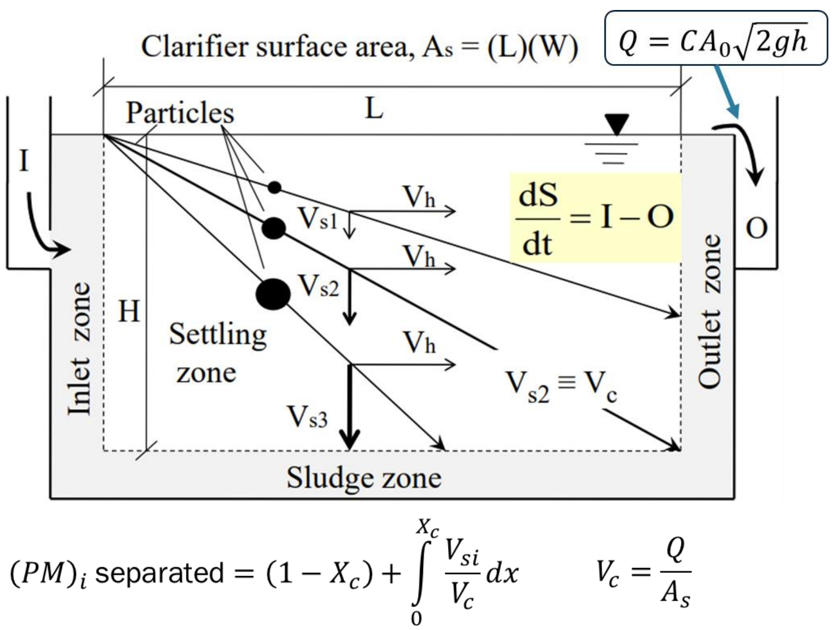


Figure 54: No Baffle Design Mechanisms

A2, and all subsequent alternatives, use the same design storm as is used in A1. Thus, the inflow data remains the same. Routing was determined using a discretized derivation of Reynold's Transport Theorem at each time step.

Equation 25

$$\Delta S_{ij} = (I_i - O_i)\Delta t_{ij}$$

Summing the change in storage for every time step yields the minimum required storage capacity for the basin, which was 9,042 cubic feet. Assuming a fixed depth of 8 feet, in which there is 1 foot of freeboard, and a 2-foot sludge zone (that is assumed full at the beginning of the design storm), an initial basin surface area is determined.

Equation 26

$$V = (H)(A_S) \qquad 9{,}042\ ft = (8\ ft)(A_S) \qquad A_S = 1{,}808\ sf$$

With a fixed 10:1 length-to-width ratio, the minimum required basin dimensions can be determined.

Equation 27

$$A_s = (L)(W) \qquad 1{,}808\ sf = (10x)(x) \qquad \text{L = 135 ft and W = 13.5 ft}$$

Then, assuming that the storm starts at a stage of 2 feet (a full sludge zone), the outflow and stage is determined for all subsequent time steps. Stage is determined using the change in storage Equation 25 plus the previous time step's stage. Outflow is calculated using the following flow through an orifice equation, Equation 28. Note, there are two orifices in which their bottom edges are 2 feet above the basin bottom (at the top of the sludge zone).

Equation 28

$$Q = CA_0\sqrt{2gh}$$

Thus, critical settling velocity can be determined for every time step.

Equation 29

$$V_c = \frac{Q}{A_s}$$

Using the site's PSD, the settling velocities for each particle diameter are calculated using an iterative process involving an initial settling velocity, a Reynolds Number, a drag coefficient, and the final settling velocity. The previous iteration's final settling velocity is plugged into the next iteration's initial settling velocity until the two values converge. This is done for every particle size, and use the following equations:

Equation 30

$$Re = \frac{\rho \cdot V \cdot L}{\mu}$$

Equation 31

$$C_D = \frac{24}{Re_d} + \frac{3}{\sqrt{Re_d}} + 0.34$$

Equation 32

$$v_t = \left[\frac{4}{3}\frac{g(\rho_s - \rho)d}{C_D \rho}\right]^{1/2}$$

Then, for each time step, the mass fraction of particles that can settle out using the following equation are weighted by the outflow volume at each time step.

$$(PM)_i \text{ separated} = (1 - X_c) + \int_0^{X_c} \frac{V_{si}}{V_c} dx$$

These are summed for every time step to get the total PM separation efficiency for that sized basin.

E.2.3: Drawings

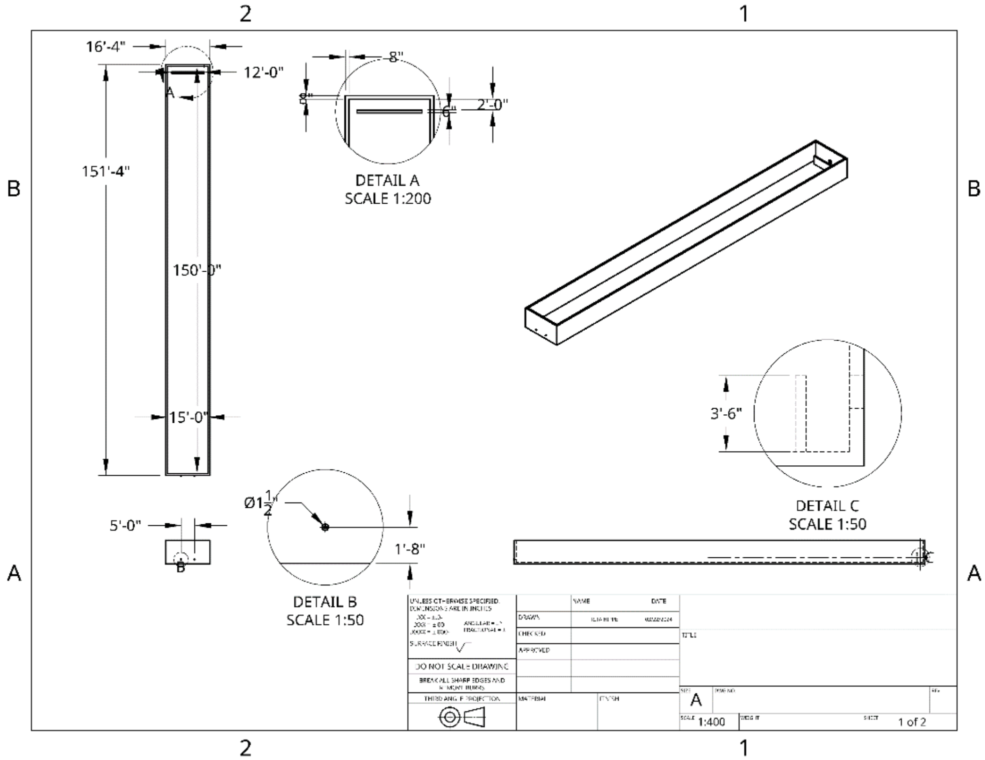


Figure 55: Page 1 of the engineering drawings for Alternative 2

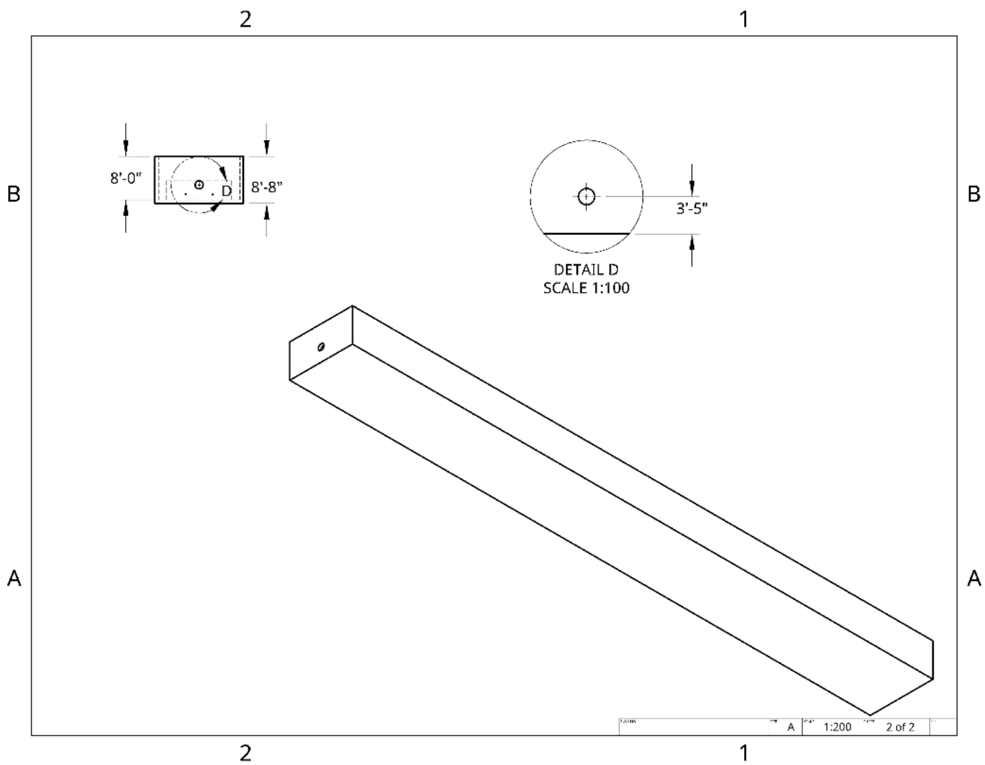


Figure 56: Page 2 of the engineering drawings for Alternative 2

E.2.4: Cost Estimates

Table 30: Alternative 2 Full Detailed Cost Estimate

Alternative 2 Full Cost Estimation Details					
No.	**Line Item**	**Quantity**	**Unit**	**Cost/Unit**	**Total**
Alternative 2					
1	Mobilization	-	Lump Sum	-	$11,000.00
2	Erosion and Sediment Control	-	Lump Sum	-	$5,500.00
3	Demolition	68	S.Y.	$10.33	$700.00
4	Excavation and Fill	805	C.Y	$18.87	$15,200
5	Land Valuation	0.09	Acre	$100,188.00	$9,200.00
6	Grub Stump and Remove	0.09	Acre	$2,178.00	$200.00
7	Selective Tree Removal	9.0	Each	$388.89	$3,500.00
8	Grading	4000	S.F.	$0.73	$2,900.00
9	8" Concrete with Epoxy Coated Steel Rebar	121	C.Y.	$329.63	$39,800.00
10	6" Concrete with Rebar	1	C.Y.	$160.00	$160.00
11	18" Concrete Pipe	22	L.F.	$46.51	$1,000.00
12	Inlet Structure	1	Each	$7,000.00	$7,000.00
13	Outlet Structure	2	Each	$3,550.00	$7,100.00
14	Filter	5	Each	$1,920.00	$9,600.00
15	Fence	346	L.F.	$29.19	$10,100.00
16	Landscaping	144	S.Y.	$25.63	$3,700.00
				Subtotal	$126,700.00
				Engineering (15%)	$19,200.00
				Permit and Insurance (5%)	$6,400.00
				Subtotal	$152,300.00
				Contingency (10%)	$15,300.00
				Total	**$167,600.00**

E.3: Alternative 3: Baffled

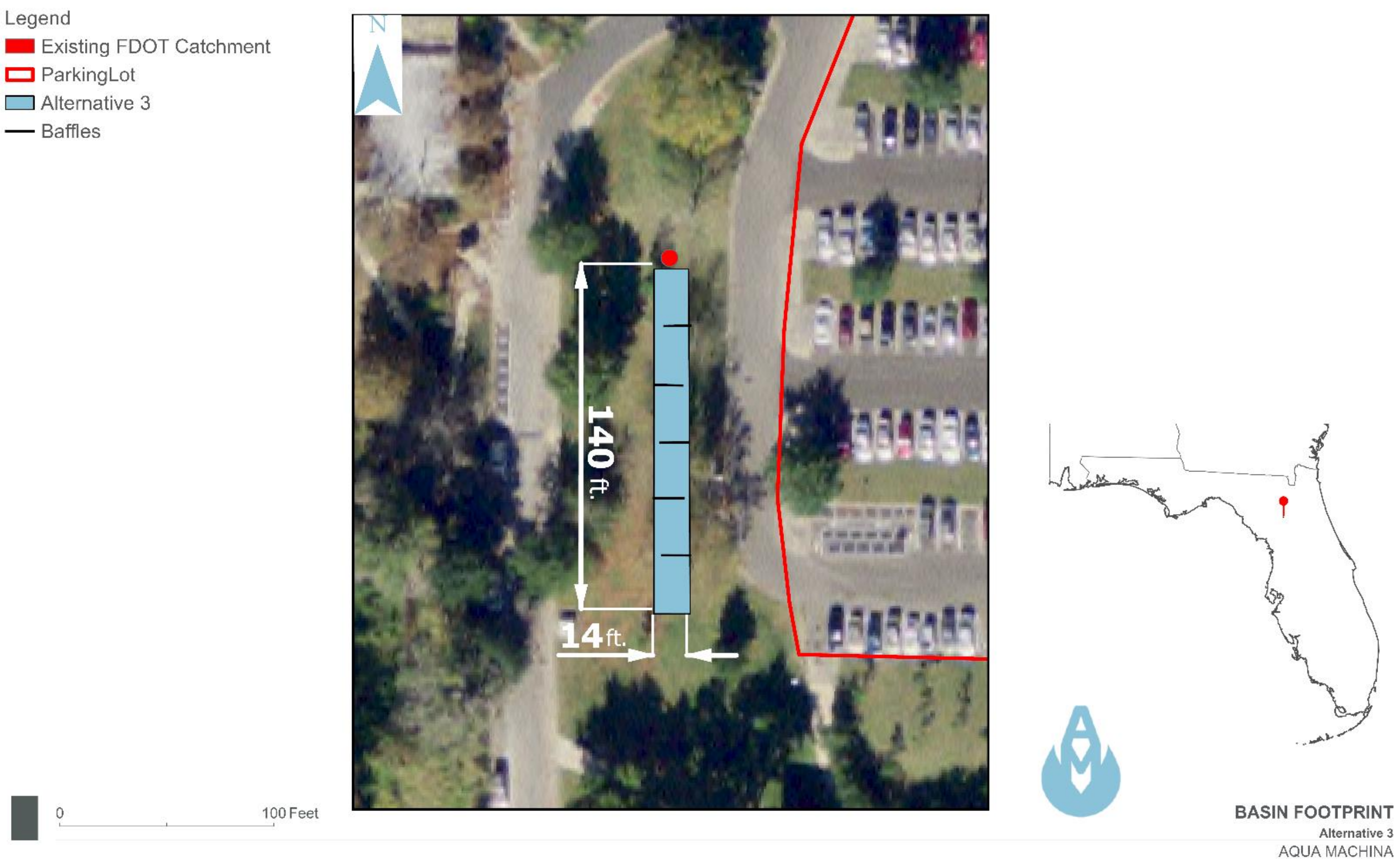


Figure 57: Alternative 3 Footprint (ESRI)

Table 31:Alternative 3 PM, TN, and TP Removal Efficiencies

	Removal Efficiency	Sedimentation and Anaerobic Digestion	Filters (finer PM)	Filters (adsorption)
PM	99.4%	✓	✓	
TN	80.2%	✓	✓	
TP	91.0%	✓	✓	✓

Table 32: Alternative 3 Design Summary

ID	L (ft)	W (ft)	H (ft)	A (ft^2)	# of Baffles	Reduction Goal (80%)
A3	140	14	8	1,690	5	Yes

E.3.1: Design Parameters

Table 33: Parameters used in Alternative 3 design

Fixed Dimensions		
Orifice Diameter (2 orifices at 2 ft height)	1.25	in
Depth	8	ft
Minimum Dimensions to Meet Storage Requirements		
L:W Ratio	10:1	
Storage Required from Inflow	9041.5	cf
Surface Area	1808.3	sf
Length	134.5	ft
Width	13.4	ft
Design Dimensions		
Length	150	ft
Width	15	ft
Surface Area	2250	sf
Wall for Energy Dissipation		
Length	0.5	ft
Width	12	ft
Height	3.5	ft
Total Basin Storage*	15,720	cf
Design Metrics		
Max Stage	5.91	ft
Time to Outflow Peak	27	mins
Residence Time	32.6	hrs
Settling Removal Efficiency	98.5	%

*Total basin storage includes the total available water storage, which does not include the 1 foot of freeboard, but considers the volume of the wall for energy dissipation, the baffles, and the filters.

Table 34: PM separation of different baffles designs in long-linear basin. This was used in part for generation of training data for the ML model (FDOT, 2021)

Basin	Total eluted PM (kg)	Total PM separation (%)	Relative difference (%)
Configuration	Long-Liner Basin (LLB)		
No baffle	638.8	63.3	--
1 baffle	610.3	64.9	2.5
3 baffles	542.4	68.8	8.7
5 baffles	**462.3**	**73.4**	**16.0**
7 baffles	415.9	76.1	20.2
9 baffles	398.1	77.1	21.8

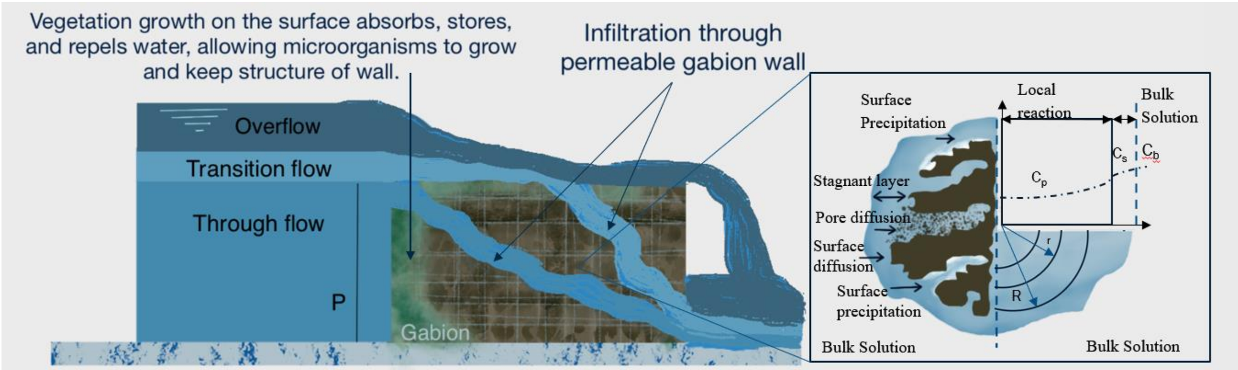


Figure 58: Function of Gabian Walls

E.3.2: Design Method

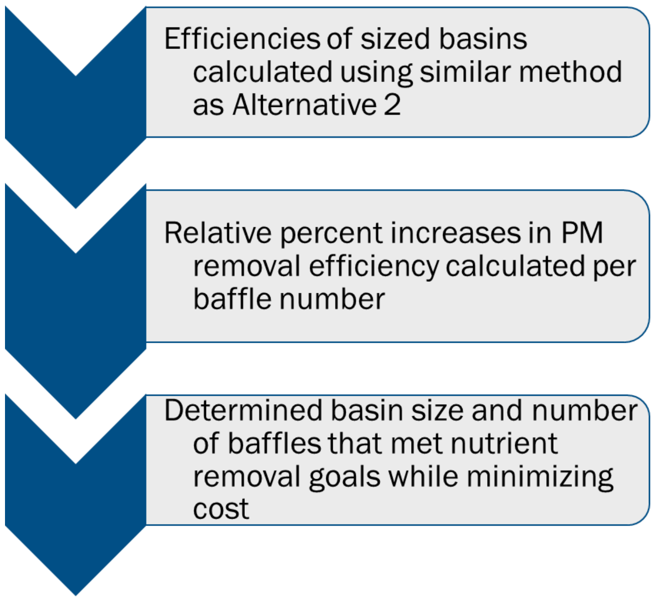


Figure 59: Baffled Design Process

A3 follows the same premise as A2, with the addition of gabion-basket baffles. The basin's initial PM separation efficiency was calculated using an identical process to that above. To find the new PM separation efficiency from the addition of baffles, the relative percent increase for each baffle number from Table 34 was compared to that of the A2, no-baffle scenario using the following equation:

Equation 33

$$RPD = \frac{|R1 - R2|}{\left(\frac{R1 + R2}{2}\right)} \times 100$$

This design used 5 transverse baffles, which research found yielded optimal hydraulic efficiency within a rectangular basin (Wilson and Venayagamoorthy, 2010).

E.3.3: Drawings

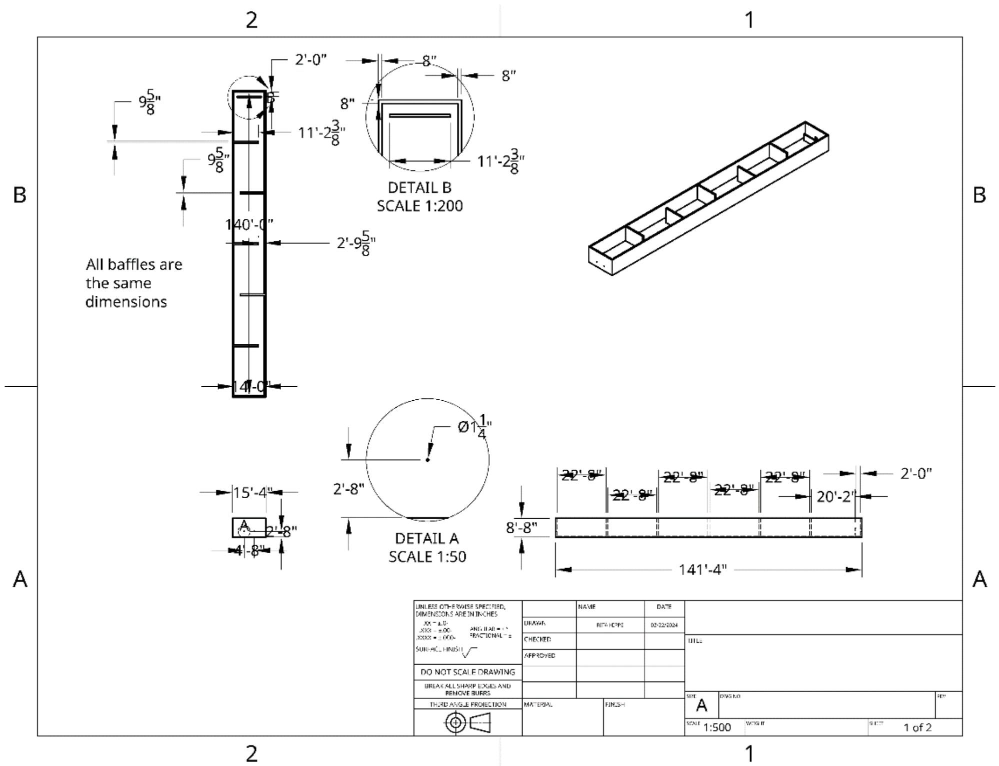


Figure 60: Page 1 of the engineering drawings for Alternative 3

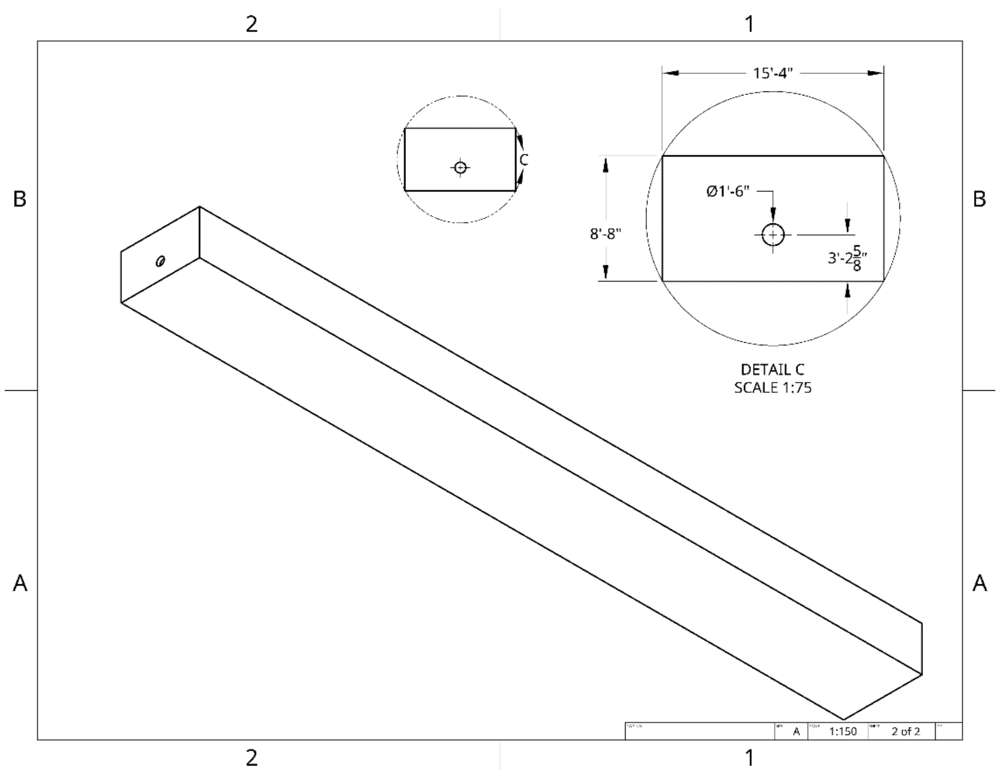


Figure 61: Page 2 of the engineering drawings for Alternative 3

E.3.4: Cost Estimates

Table 35: Alternative 3 Full Detailed Cost Estimate

Alternative 3 Full Cost Estimation Details					
No.	**Line Item**	**Quantity**	**Unit**	**Cost/Unit**	**Total**
Alternative 3					
1	Mobilization	-	Lump Sum	-	$10,500.00
2	Erosion and Sediment Control	-	Lump Sum	-	$5,300.00
4	Excavation and Fill	881	C.Y	$15.32	$13,500
5	Land Valuation	0.08	Acre	$100,000.00	$8,300.00
6	Grub Stump and Remove	0.08	Acre	$1,936.00	$160.00
7	Selective Tree Removal	9.0	Each	$388.89	$3,500.00
8	Grading	3600	S.F.	$0.81	$2,900.00
9	8" Concrete with Epoxy Coated Steel Rebar	109	C.Y.	$334.14	$36,500.00
10	6" Concrete with Rebar	1	C.Y.	$160.00	$160.00
11	18" Concrete Pipe	22	L.F.	$46.51	$1,000.00
	Baffle	358	C.F.	$5.86	$2,100.00
13	Inlet Structure	1	Each	$7,000.00	$7,000.00
14	Outlet Structure	2	Each	$3,550.00	$7,100.00
15	Filter	5	Each	$1,920.00	$9,600.00
16	Fence	324	L.F.	$29.32	$9,500.00
17	Landscaping	135	S.Y.	$25.85	$3,500.00
				Subtotal	$120,600.00
				Engineering (15%)	$18,100.00
				Permit and Insurance (5%)	$6,100.00
				Subtotal	$144,800.00
				Contingency (10%)	$14,500.00
				Total	**$159,300.00**

E.4: Alternative 4: Baffles – AI Optimized

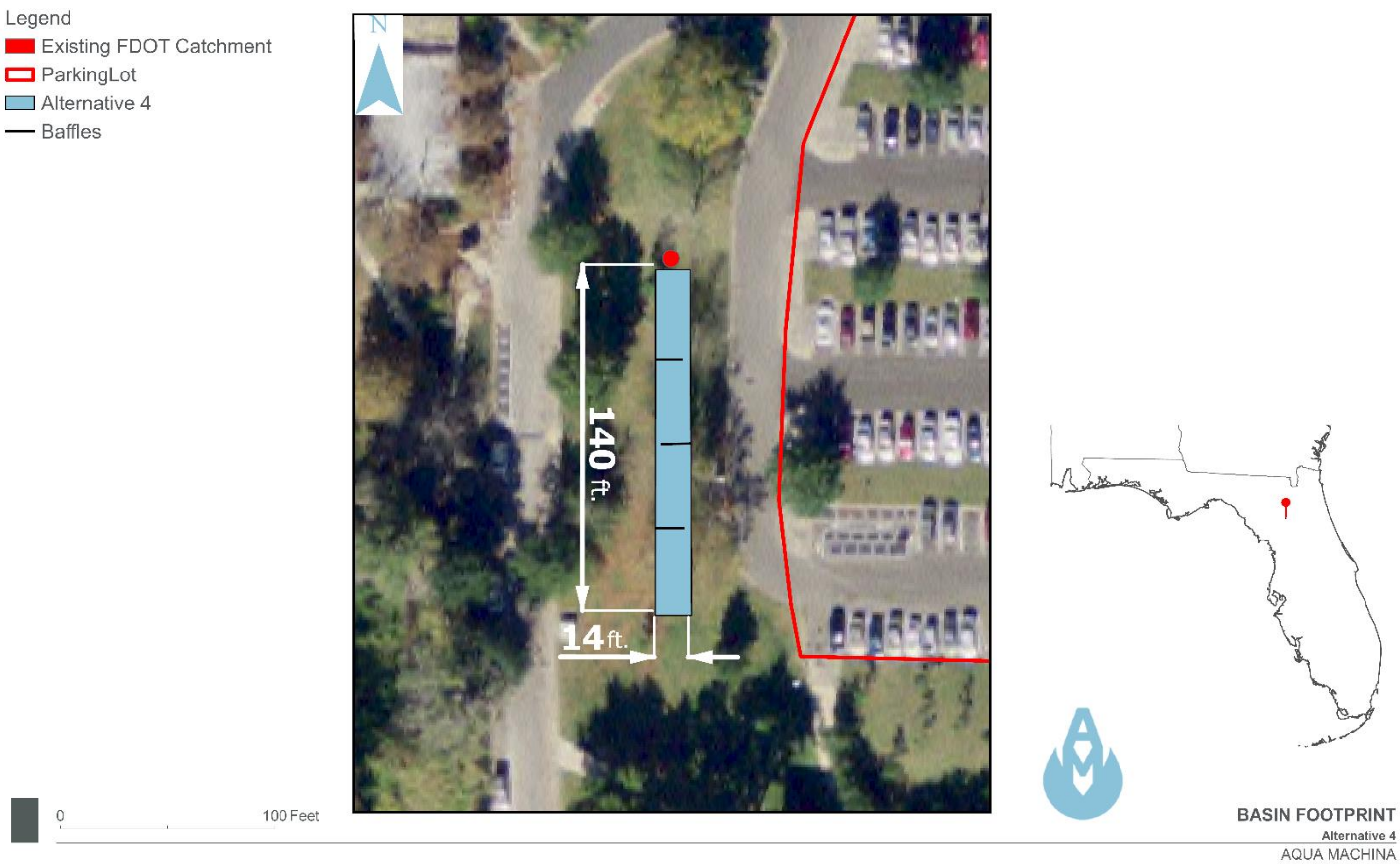


Figure 62: Alternative 4 Footprint (ESRI)

Table 36: Alternative 4 PM, TN, and TP Removal Efficiencies

	Removal Efficiency	Sedimentation and Anaerobic Digestion	Filters (finer PM)	Filters (adsorption)
PM	99.4%	✓	✓	
TN	80.2%	✓	✓	
TP	91.0%	✓	✓	✓

Table 37: Alternative 4 Design Summary

ID	L (ft)	W (ft)	H (ft)	A (ft^2)	# of Baffles	Reduction Goal (80%)
A4	140	14	8	1,690	3	Yes

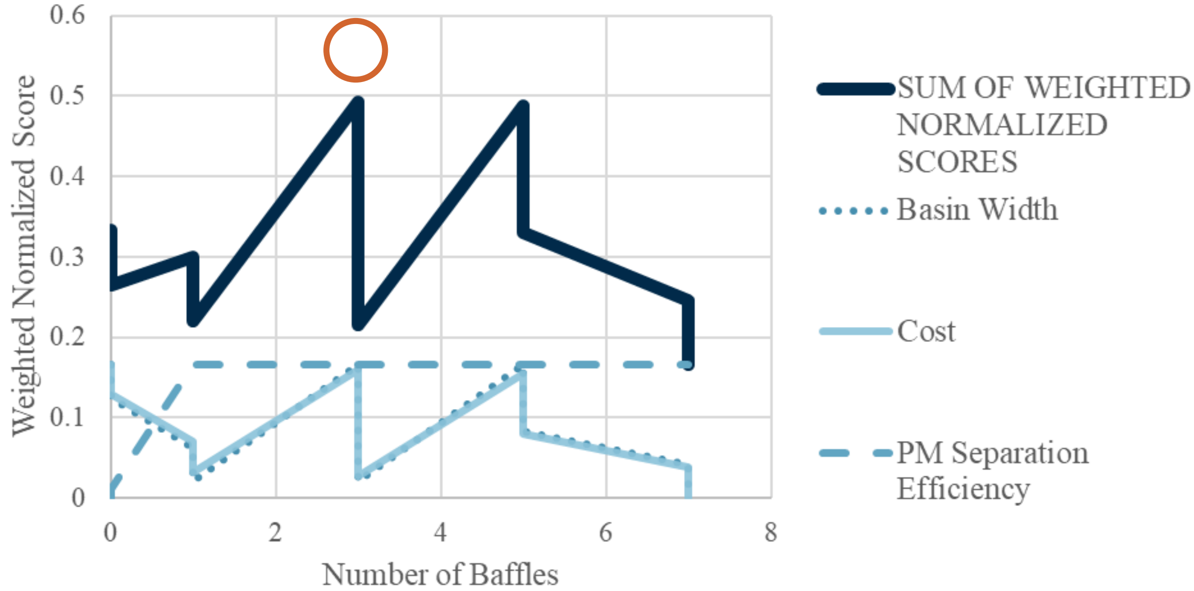


Figure 63: A summary of weighted-normalized parameters that were predicted using ML. The higher the score, the more optimal that set of parameters is.

E.4.1: Design Parameters

Table 38: Parameters used in Alternative 4 design

Fixed Dimensions		
Orifice Diameter (2 orifices at 2 ft height):	1.25	in
Depth	8	ft
L: W Ratio	10:1	
Minimum Dimensions to Meet Storage Requirements		
Storage Required from Inflow	9041.5	cf
Surface Area	1808.3	sf
Length	134.473	ft
Width	13.4473	ft
Design Dimensions		
Length	140	ft
Width	14	ft
Surface Area	1,960	sf
Wall for Energy Dissipation		
Length	0.5	ft
Width	11.2	ft
Height	3.5	ft
Total Basin Storage*	13,550	cf
Design Metrics		
Max stage	6.582	ft
Time to outflow peak	27	mins
Residence time	32.6	hrs
Settling removal efficiency	99.0	%

* Total basin storage includes the total available water storage, which does not include the 1 foot of freeboard, but considers the volume of the wall for energy dissipation, the baffles, and the filters.

E.4.2: Design Method

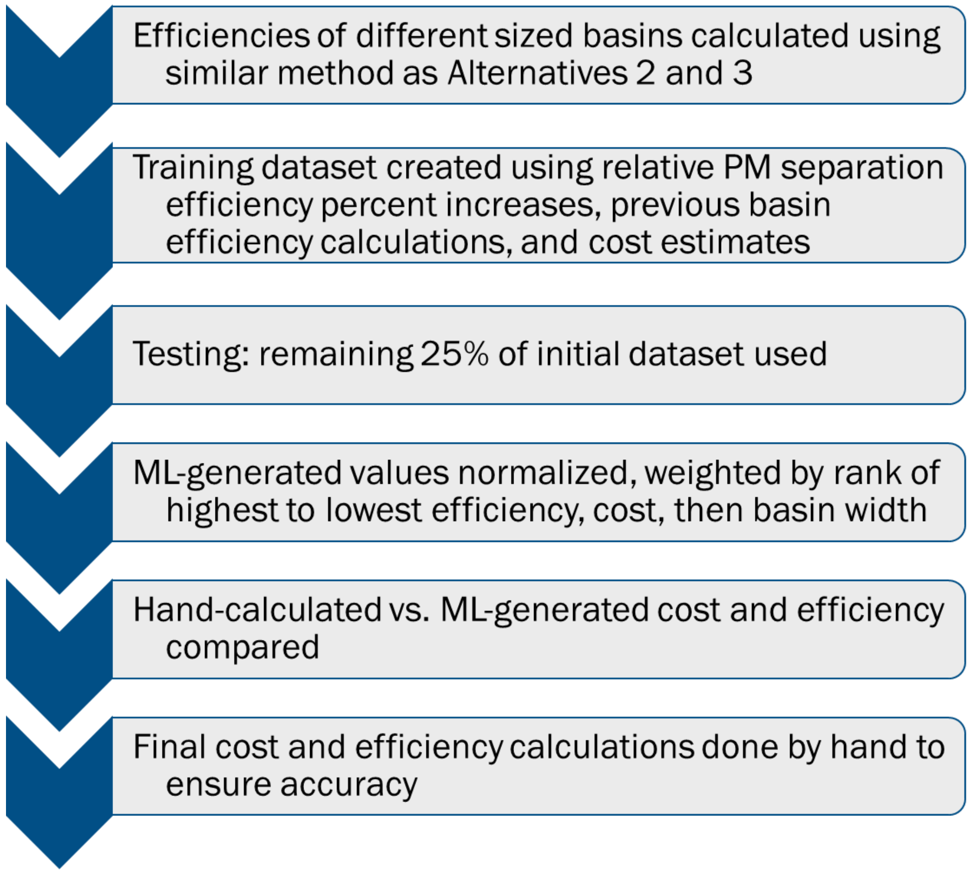


Figure 64: AI Optimized Design Process

The efficiencies of the different sized basins were calculated through the same method as Alternatives 2 and 3. This was then used to train the machine learning algorithms. A training dataset was created using relative PM separation efficiency percent increases per baffle number from Table 34, previous basin efficiency calculations, and cost estimates associated with various basin sizes. The goal of the model was to assess total cost and PM separation efficiency using basin width (assuming a fixed 10:1, length-to-width ratio) and baffle number. Only basin sizes that had the capacity to handle the storage required from the design storm were considered (140 ft by 14 ft and larger), and all other parameters (including basin depth) were held constant. In terms of machine learning, a linear regression (LN) model and an artificial neural network (ANN) were programmed. After using 75% of the initial dataset to train the models, their respective root mean squared errors (RSMEs) were compared to determine which had better predictive capability. Due to LN having a lower RSME for predicting cost, and ANN for predicting PM separation efficiency, LN and ANN were selected to model those respective characteristics. See Figure 65 for a diagram of the ANN.

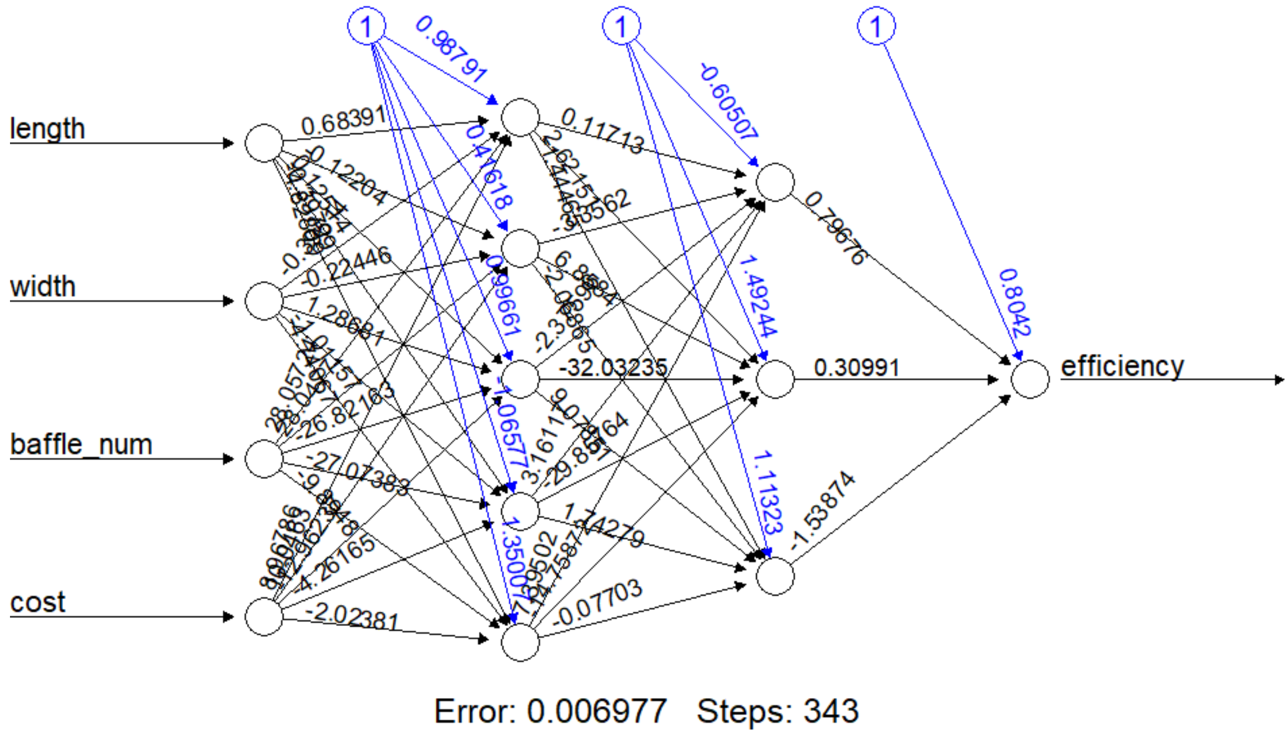


Figure 65: Diagram of the artificial neural network used to model PM separation efficiency. Black lines show how each layer is connected, and how each of those connections are weighted. The blue lines show the bias terms. The convergence of the training algorithm

The remaining 25% of the initial data set was then used to test the models. In order to compare the ML-generated basin widths, costs, and PM removal efficiencies, these values were normalized on a scale of 0 to 1, with 0 being most desirable, and then weighted by rank of highest to lowest, ranked efficiency, cost, then basin width. Efficiency was ranked highest because the lowest efficiency was just under the Federal limit for nutrient loading, making it the most important factor. Since cost is related to basin width but more familiar to clients, the factor ranks proceeded as cost, then basin width. These normalized and weighted scores were summed for each factor. The basin that yielded the most desirable outcome had dimensions of 140 feet by 14 feet, and 3 baffles. This basin's ML-predicted cost and efficiency were compared to one of the same dimensions, calculated as by hand in A3 (Table 39).

Table 39: Hand Calculated and ML-Generated Cost Estimates to Determine Relative Percent Difference

	Cost	Efficiency (%)
Calculated	$78,776.17	99.9
ML-Generated	$78,458.36	99.9
Relative Percent Difference (%)	0.40%	0%

It should be noted that the ML models' predictive capabilities would increase with a more-robust dataset, perhaps using computational fluid dynamics (CFD) modeling to capture more efficient data. In this design, the ML-generated data is used to inform what basin size and baffle number is *most likely* to have the most-optimal results, but both final cost and efficiency calculations were done by hand in this alternative to ensure accurate assessment of the chosen basin size.

A link to the GitHub repository where the files related to the generation of this model are stored is included below. The file "BaffleMLData.csv" contains the training/testing data for this model, and the file "MLPredictedData.csv" contains the ML-generated dataset that was analyzed above.

https://github.com/catboymothman/AquaMachina

Below are resources we used while developing this model.

https://datascienceplus.com/fitting-neural-network-in-r/

E.4.3: Drawings

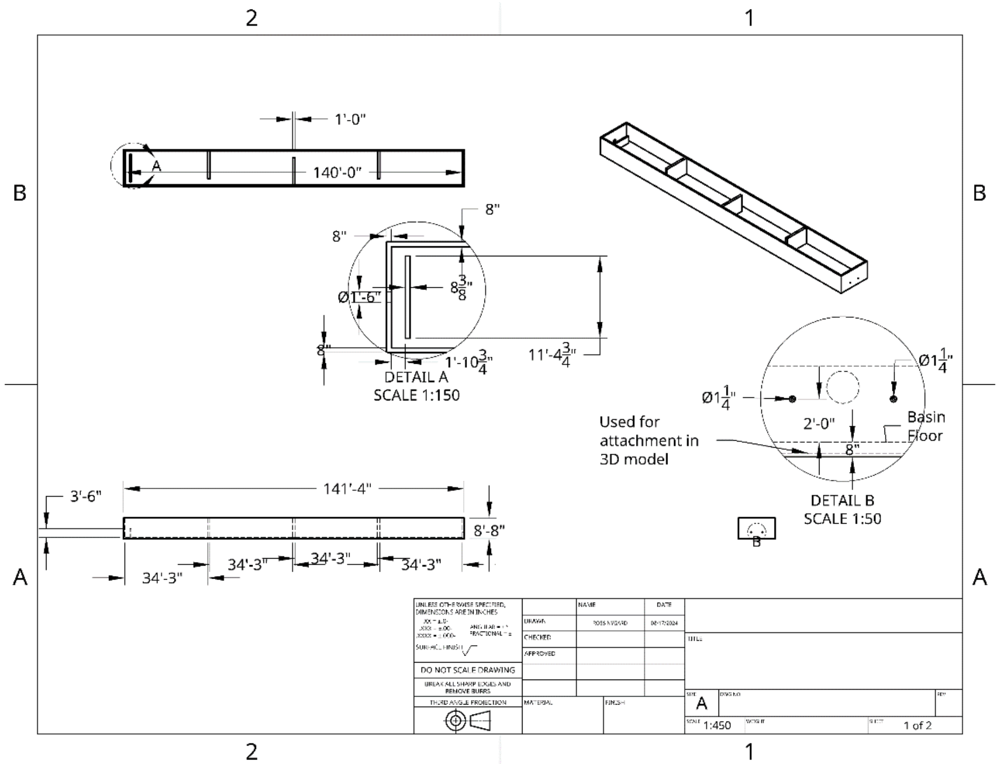


Figure 66: Page 1 of the engineering drawings for Alternative 4

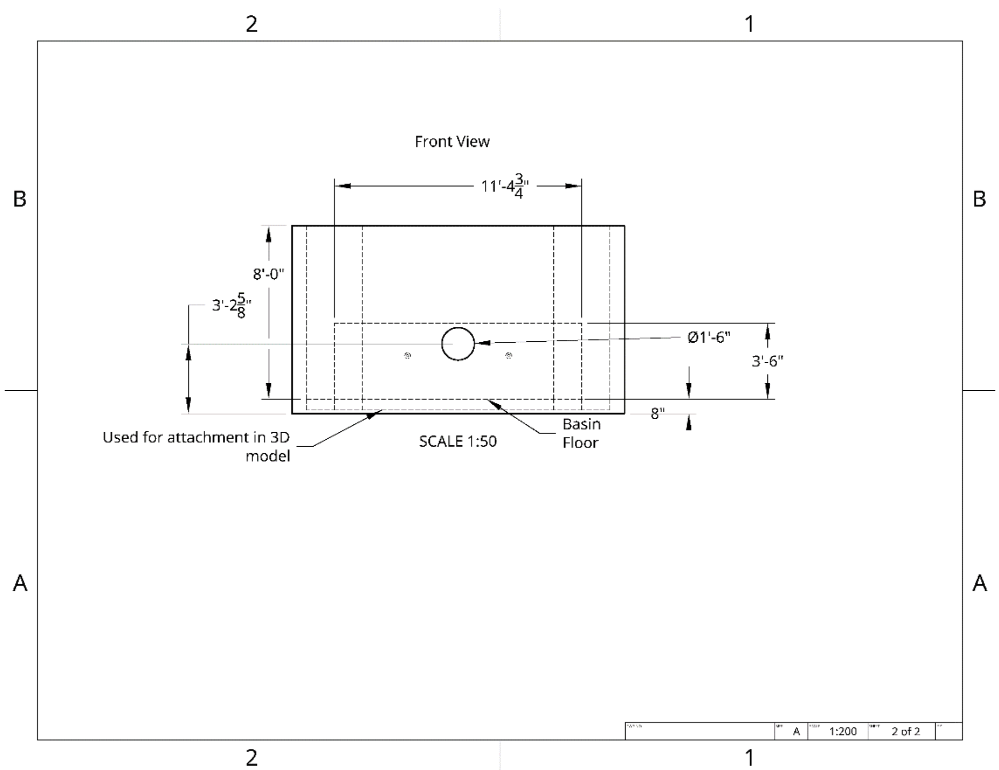


Figure 67: Page 2 of the engineering drawings for Alternative 4

E.4.4: Cost Estimates

Table 40: Alternative 4 Full Detailed Cost Estimate

Alternative 4 Full Cost Estimation Details					
No.	**Line Item**	**Quantity**	**Unit**	**Cost/Unit**	**Total**
Alternative 4					
1	Mobilization	-	Lump Sum	-	$10,400.00
2	Erosion and Sediment Control	-	Lump Sum	-	$5,200.00
4	Excavation and Fill	881	C.Y	$15.32	$13,500
5	Land Valuation	0.08	Acre	$100,000.00	$8,300.00
6	Grub Stump and Remove	0.08	Acre	$1,936.00	$160.00
7	Selective Tree Removal	9.0	Each	$388.89	$3,500.00
8	Grading	3600	S.F.	$0.81	$2,900.00
9	8" Concrete with Epoxy Coated Steel Rebar	109	C.Y.	$334.14	$36,500.00
10	6" Concrete with Rebar	1	C.Y.	$160.00	$160.00
11	18" Concrete Pipe	22	L.F.	$46.51	$1,000.00
	Baffle	215	C.F.	$6.05	$1,300.00
13	Inlet Structure	1	Each	$7,000.00	$7,000.00
14	Outlet Structure	2	Each	$3,550.00	$7,100.00
15	Filter	5	Each	$1,920.00	$9,600.00
16	Fence	324	L.F.	$29.32	$9,500.00
17	Landscaping	135	S.Y.	$25.85	$3,500.00
				Subtotal	$119,700.00
				Engineering (15%)	$18,000.00
				Permit and Insurance (5%)	$6,000.00
				Subtotal	$143,700.00
				Contingency (10%)	$14,400.00
				Total	**$158,100.00**

E.4.5: Improvements

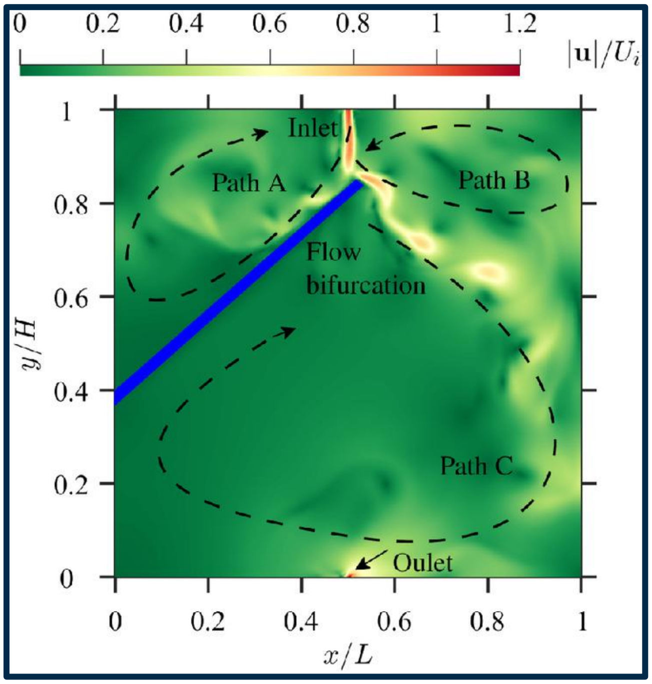


Figure 68: Possible ML Improvements

To enhance the predictive capabilities of machine learning (ML) models, leveraging a more robust dataset by incorporating Computational Fluid Dynamics (CFD) modeling can provide a deeper understanding of flow dynamics and aid in optimizing baffle placement within the system. Parameters such as baffle spacing, and angle can be systematically varied and analyzed using CFD simulations to identify the configurations that maximize flow control and sediment removal efficiency. By iteratively refining these design aspects based on CFD results and integrating them into the ML framework, more accurate predictions and optimized stormwater management strategies can be developed.

Appendix F: Comparison of Alternatives

F.1: Operation & Maintenance

Table 41: Detailed Annual Operation and Maintenance Cost Estimates

Annual Operation and Maintenance Costs		
Alternative 1	Clean out clarifier	$105,145
	Landscaping	$4,185.00
	Fence maintenance	$942.69
	Overall Cost	$120,172.48
Alternative 2	Clean out clarifier	$7,821
	Landscaping	$4,185.00
	Fence maintenance	$266.05
	Overall Cost	$12,271.73
Alternative 3	Clean out clarifier	$6,813
	Landscaping	$3,825.00
	Fence maintenance	$232.21
	Overall Cost	$10,869.90
Alternative 4	Clean out clarifier	$11,067
	Landscaping	$3,825.00
	Fence maintenance	$232.21
	Overall Cost	$10,869.90

The annual operation and maintenance cost for each alternative is based on three major components: landscaping, fence repairs, and annual sludge removal. The sludge removal cost estimates were based on the RS Means cost for a vacuum truck. The volume of sludge per alternative was then used to determine the number of trucks required to remove the sludge generated annually. The landscaping cost was based on the hourly wages for the UF's landscaping staff listed on their website. Finally, the fencing repair cost were determined from a study from the United States Corps of Engineers and was distributed over the lifetime of the fence (US Army Corps of Engineers, 1999).

The cost for the replacement of the gabion basket substrate, CRC, was not included in the annual operation and maintenance cost for Alternatives 3 and 4. CRC has a high capacity for phosphorous adsorption. Due to the high adsorption capacity, the lifetime of CRC is longer than the basin itself making its annual maintenance costs negligible.

Table 42: Summarized Cost Estimation

Cost		
Alternative 1	Construction	$658,000
	Land Valuation	$84,000
	Annual O&M	$120,000
	Overall Cost	**$862,000**
Alternative 2	Construction	$159,000
	Land Valuation	$9,000
	Annual O&M	$12,000
	Overall Cost	**$180,000**
Alternative 3	Construction	$151,000
	Land Valuation	$8,000
	Annual O&M	$11,000
	Overall Cost	**$170,000**
Alternative 4	Construction	$150,000
	Land Valuation	$8,000
	Annual O&M	$11,000
	Overall Cost	**$169,000**

F.2: Decision Matrix

The decision matrix shown in Table 43 was used as the primary tool in evaluating the alternatives for selection. The evaluation criteria were each assigned weighting factors, listed under each column label and within the *Weight* row. The performance of each alternative within each evaluation criteria category is scored from 1 to 4, with 1 representing the best, or most desirable, score.

Table 43: Evaluation of Alternatives based on Peak Reduction, Community Impact, Space, Nutrient Loading, and Cost.

	Peak Reduction	Sustainability	Community Impact	Space	Nutrient Loading	Cost	Weighted Score	Overall Rank
Weight	0.0667	0.0667	0.1333	0.2000	0.2667	0.2667	-	-
A1	1	4	4	4	3	4	3.40	4
A2	2	2	3	2	1	2	1.87	3
A3	2	1	2	1	1	1	1.20	2
A4	2	1	1	1	1	1	1.07	1

Cost and nutrient loading were weighted highest and equally, as those characteristics set the design requirements for the project and load reduction is the goal of the Florida 2024 CWA. Area was ranked the second highest due to the desire of the client to avoid disruption of the existing infrastructure, structures, and activities of the site. Community impact was rated next highest, as it aligned with the client's values. Peak reduction and sustainability were ranked equally and the lowest, as these aspects identify secondary and indirect benefits. Peak reduction was included as a recognition of the additional benefits provided by the implementation of a stormwater basin on reducing peak flows and times to peak within stormwater management systems.

Peak reduction scores were assigned as a ranking of the reduction of each alternative. A1 had the lowest peak outflow, while the other three designs had approximately the same peak outflow. Thus, A1 was given a rank of 1, and the others were given a rank of 2 for peak reduction.

A3 and A4 were given a rank of 1 for sustainability due to their smaller size in comparison to A1 and A2, which reduced the amount of excavation needed (thus reducing the need for fossil fuels) and the amount of disturbance to the carbon stored in the soil. A1 was given a rank of 4 due to the alternative significant difference in sustainability regarding size, which considers excavation and increased planting along the basin perimeters.

Community impact scores were developed to quantitatively capture the relevance of the alternative to the current goals and ideals of the University and surrounding community. A score of 1 indicates that the design clearly aligns with the current goals and ideals of the University and community and will provide them with other benefits (outside of serving its primary purpose) which bring attention to said goals or ideals. A score of 2 indicates that the project aligns with the current goals and ideals of the community and will interact with the University and community positively (outside of serving its primary purpose). A score of 3 indicates that the alternative aligns with the goals and ideals of the University but does not provide significant additional benefits to the University and/or community. A score of 4 indicates that the alternative does not necessarily align with the goals and ideals of the University.

Footprint area ranges were used to determine the scores for the improvement area criteria. A score of 1 includes areas less than 2,000 square feet. A score of 2 falls within 2,000 and 8,000 square feet. A score of 3 falls within 8,000 and 14,000 square feet. A score of 4 was given to areas greater than 14,000 square feet.

Alternatives 2, 3, and 4 scored a 1 for the nutrient loading criteria, as each was designed to meet the 80% reduction criteria. However, Alternative 1 scored a 3 because the basin was able to achieve the required 80% removal efficiency of PM and TN but not TP.

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Appendix G: Extensibility

G.1: Funding Opportunities

Table 44: Funding Opportunities

Entity	Funds Type	Applicable Project Types/Description
EPA	Sewer Overflow and Stormwater Reuse Municipal Grant	Planning, design, and construction of treatment works to manage, reduce, treat, or recapture stormwater or subsurface drainage water.
EPA	Clean Water State Revolving Fund Loan	Low-interest loans to upgrade pollution prevention projects
FDEP	Funding for Water Quality Improvement	Stormwater sources of nutrients in water bodies not attaining nutrient standards and/or with a basin management plan
FDEP	State Water Quality Assistance Grants	Water quality projects targeting specific pollutant sources
FDEP	Innovative Technology for Harmful Algal Blooms	Projects that evaluate and implement innovative solutions to combat algal blooms and nutrient enrichment

G.2: Extensibility Study

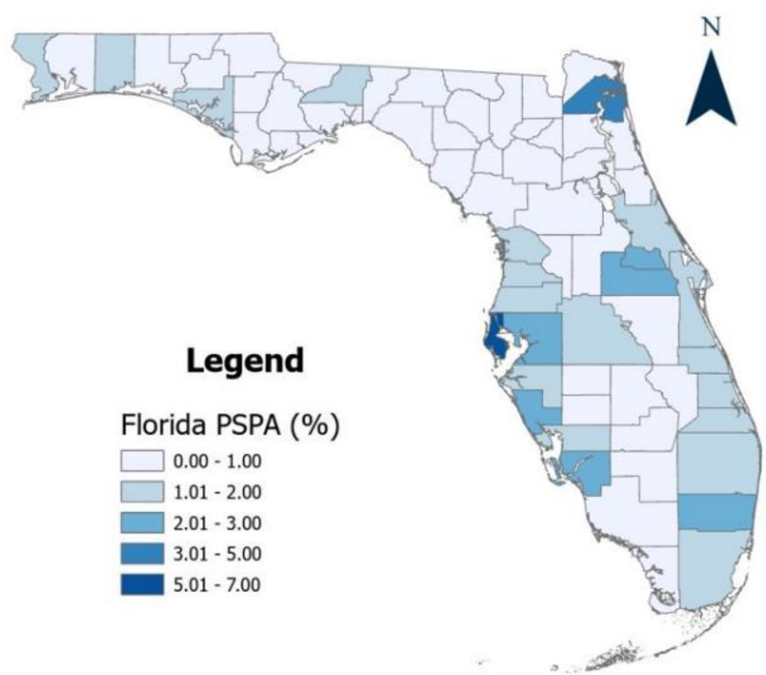

Figure 69: Estimated Paved Surface Area for Parking Across Florida (ESRI; Falcone & Nott, 2019) Falcone and Nott, 2019)

Table 45: Estimated Paved Surface for Parking Across Nine Counties in Florida (ESRI; Falcone and Nott, 2019)

County	Percentage
Alachua	0.85%
Broward	2.37%
Duval	3.13%
Hillsborough	2.89%
Lee	2.72%
Orange	2.92%
Pinellas	6.65%
Sarasota	2.26%
Seminole	2.74%

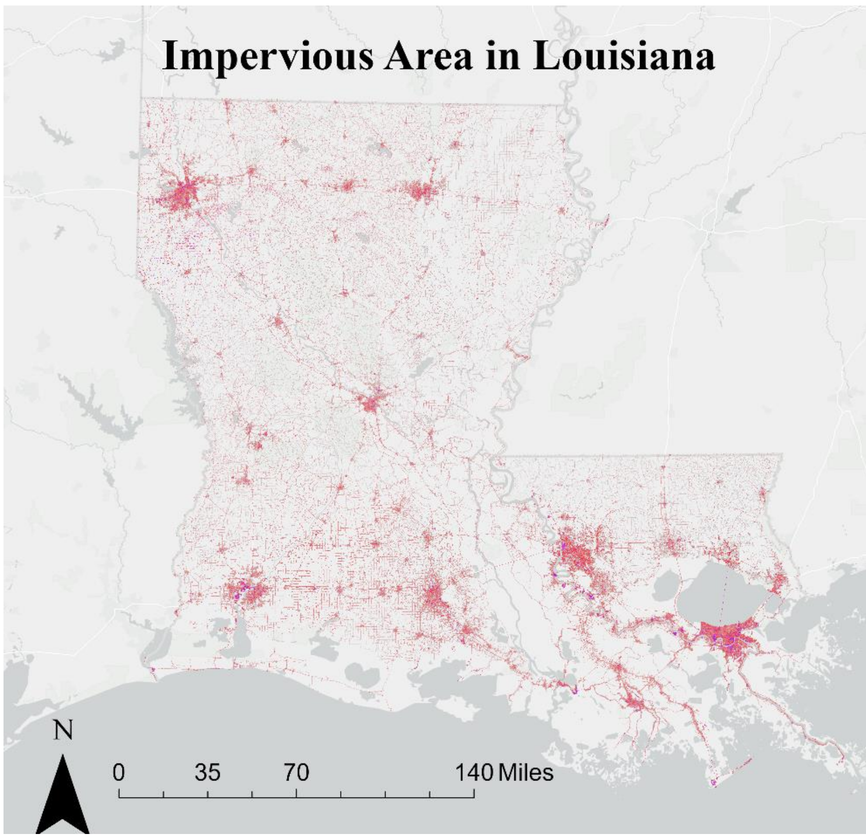

Figure 70: Impervious Area in Louisiana (LDTD, 2016)

G.2.1: Alternative XA1 Design Details

G.2.1.1: Buoyancy Calculations and Concrete Pilings

Due to the high-water table in New Orleans, the buoyancy force was calculated to prevent the uplift of the buried treatment basin. The buoyancy force was calculated using the density or water, total volume of the basin, and gravity which was calculated to be 9,875,000 lb ft/s^2. Refer to Equation 34 for the buoyant force equation and Table 46 for a summary the buoyant force calculation.

Equation 34

$$F_b = \rho_{fluid} * V_{tank\ submerged} * g$$

Table 46: Buoyant Force Calculation

Buoyant Force		
ρ (fluid)	64.2	lb/ft^3
volume of tank	15680	ft^3
g	9.81	ft/s^2
Buoyant Force	9,875,295	lb ft/s^2

The total weight of the full basin was calculated to be 549,000 lb which is not enough to counter act the buoyant force. Two alternatives were considered including making the bottom of the basin thick, adding additional weight with additional concrete and the use of friction piles. The amount of excess concrete necessary to counteract the buoyant force was calculated. It was determined that approximately 72,000 cubic feet of concrete would need to be added to the base of the basin. Due to the high cost of concrete this alternative is not considered to be feasible. Alternative 2 evaluated the use of friction piles to counter act the buoyant force. The length of the friction piles was determined using NOLA standards (City of New Orleans, 2022). Due to the size of the basin, it was determined that the length of the piles would need to be 40 feet (City of New Orleans, 2022). Meyerhof's method was used to determine the diameter of the piles which requires the critical embedment ratio to be between 16 and 18 for sandy soil (Das, B.M., 2007). Calculation using the critical embedment ratio determined the diameter of the piles should be 2 feet. The piles were spaced out along the centerline of the basin spaced 12 feet apart. A total of 14 piles will be required to span the centerline of the basin. The friction piles along with the friction forces along the basin concrete walls will counteract the buoyant force ensuring the basin remains in its designed location. Alternative 2 is a more cost-effective solution than Alternative 1 and was chosen to be used for the design of basins in areas with high water tables.

G.2.1.2: Pump Sizing

To calculate the average time between storms, historical 15-minute precipitation data from NOAA was examined to determine the average time between storms (NOAA National Center for Environmental Information, 2001). The data was downloaded as a CSV file, and then modified to look at the date and time in a format Excel can analyze. Missing collection times were removed from the data set. From there, the time difference between each reading was measured. Because a 15-minute collection frequency was used from the NOAA, the AVERAGEIFS Excel function was used to average all time differences greater than 15 minutes. Additionally, the AVERAGEIFS checked if the month was September, October, or November to determine if the data was for New Orleans' dry season or not. This method was done for both measuring methods provided in the NOAA dataset: QGAG and QPCP. The results of this process are shown below in Table 47.

Table 47: Mean Time Between Storms for New Orleans, Louisiana using different methods of measuring precipitation

	Mean Time Between Storms	
Measuring Method	QPCP	QGAG
Wet Season	1.26 Days	1.15 Days
Dry Season	1.51 Days	1.33 Days
Year-Round	1.31 Days	1.19 Days

Thus, at maximum capacity, the basin must be able to empty within 1.15 days (t). The maximum basin storage capacity (V) is 13,550 cubic feet, which includes total volume subtracting 1 foot of freeboard, and the volume of the energy dissipation wall and 3 baffles. This provides the pump flow (Q) required to empty the full basin before the next anticipated storm.

Equation 35

$$Q = \frac{V}{t} = \frac{13{,}550\ cf}{1.15\ days} = \ 8.18\ cfm$$

To account for storms that may occur faster than the 1.15-day average time between storms, the pump will be sized to handle a flow of 10-cfm. To determine the cross-sectional area (A) of the 466-ft pipe that connects treated basin effluent to the discharge point at the Mississippi River, a velocity (v) of 7 fps is assumed, which is high enough to also aid in pipe scouring.

Equation 36

$$Q = vA \qquad A = \frac{Q}{v} = \frac{10\ cfm}{7\ fps} = 0.0238\ ft^2$$

This provides a means of calculating the pipe diameter (D).

Equation 37: Cross-Sectional Area of a Pipe

$$A = \frac{\pi}{4}D^2 \quad D = \sqrt{\frac{A}{\frac{\pi}{4}}} = \sqrt{\frac{0.0238}{\frac{\pi}{4}}} = 0.522\ in$$

The next commercially available pipe size is 1-inch, however, the frictional head losses associated with a 1-inch diameter pipe over a long distance were significantly higher than that of a 2-inch diameter pipe. As such, the 466-ft discharge pipe was 2 inches in diameter. PVC was selected as the pipe material to further minimize frictional losses.

Major losses were calculated using the Darcy-Weisbach equation using the above parameters, in addition to a PVC pipe roughness (e) of 0.00084 inches and kinematic viscosity (v) of 1.06E-5 ft^2/s (at 70° F).

Equation 38: Darcy-Weisbach Equation

$$h_L = \frac{fLv^2}{2Dg} = 57\ ft$$

The grade elevation above the basin outlets is 25 feet above sea level (New Orleans Topographic Map) the top of the vault is 8 feet below grade to handle car weight above it, (ASCE, 1992) and the outlet orifices would be 2 feet from the bottom of the 8-ft tall vault with a 0.5-ft thick covering. Thus, the elevation of the basin outlets would be 10.5 feet above sea level. The proposed discharge point is 5 feet above sea level (New Orleans Topographic Map), alleviating head loss by 5.5 feet.

Minor losses came from one standard 90° bend and branch wye fitting and were calculated using Equation 39.

Equation 39: Minor Losses

$$h_{LM_i} = K_i \frac{v^2}{2g} = (0.75\ 90°\ bend + 0.30\ wye)\frac{v^2}{2g}\ 0.95\ ft$$

The summation of these losses results in a total head loss of 52.5 feet. A .5-hp pump with a variable frequency drive (VFD) was selected to meet these requirements.

Two float switches are attached to the basin wall containing the exit orifices. One is installed at a basin height of two feet to signal the VFD to decrease the pump speed, and the other is installed at a height of six feet to increase pump speed to prevent overflow. Much like the addition of a second orifice for redundancy, there is a second pump identical to that mentioned above for the same purpose.

G.2.1.3: Maintenance

The maintenance of the XA1 basin design is similar to the A4 design with some additional challenges. An annual inspection is required, and the frequency of maintenance is at the discretion of the inspector. The process of maintenance involves the removal of sediment accumulated in the 2-foot sludge zone. As the basin is proposed to be placed underground, maintenance equipment must fit within the manholes and operable from above ground. We recommend the use of a high-pressure water jet and vacuum truck combination. This would involve using the water jet to complete multiple passes towards the downstream side of the basin, where the vacuum truck will be placed to extract the sediment. This process is best implemented when there is little to no flow or standing water within the system, which is possible during New Orleans' dry season of Fall. Additionally, all inlets and outlets must be blocked prior to beginning the maintenance process to prevent sediment clogging.

G.2.1.4: Cost Estimates

Table 48: Alternative XA1 Cost Estimate

No.	Line Item	Quantity	Unit	Cost/Unit	Total
Alternative XA1 Full Cost Estimation Details					
Alternative XA1					
1	Mobilization	-	Lump Sum	-	$25,000.00
2	Erosion and Sediment Control	-	Lump Sum	-	$12,000.00
3	Demolition	14,014	SF	1.47	$20,600
4	Excavation and Fill	3,652	C.Y	$4.38	$16,000
5	Easement[1]	0.80	Acre	$427,500.00	$35,300.00
6	Grading	3600	S.F.	$0.81	$2,900.00
7	8" Concrete with Epoxy Coated Steel Rebar	16830	C.Y.	$2.17	$36,500.00
8	6" Concrete with Rebar	43	C.Y.	$160.00	$160.00
9	Pile and Pile Caps	14	Each		$67,500.00
10	2" PVC Pipe	11797	L.F.	$1.70	$20,000.00
11	Baffle	215	C.F.	$6.05	$1,300.00
12	Inlet Structure	1	Each	$7,000.00	$7,000.00
13	Outlet Structure	2	Each	$3,550.00	$7,100.00
14	Filter	6	Each	$1,600.00	$9,600.00
15	Pump	2	Each	$290.00	$580.00
16	Replacement of Pavement	14,014	SF	$3.03	$42,400.00
				Subtotal	$303,900.00
				Engineering (15%)	$46,000.00
				Permit and Insurance (5%)	$15,000.00
				Subtotal	$364,900.00
				Contingency (10%)	$36,500.00
				Total	**$401,400.00**

[1]The easement estimate is based on 95% of the Fair Value Market price of land in New Orleans (Landsearch, 2024), (Natural Resources Conservation Service, 2023)

G.2.2: Alternative XA2 Design Details

G.2.2.1: Overview

A permeable pavement system consists of a permeable surface course underlain by a storage bed placed on uncompacted subgrade to facilitate stormwater infiltration (Water Environment Federation, 2014).

The permeable pavement system will consist of a porous surface layer, an aggregate base, and a subbase designed to facilitate infiltration and storage of stormwater. The design ensures durability and functionality while supporting environmental goals. This sustainable approach aligns with the city's efforts to mitigate flooding and improve water quality. (M.C.S., Ord. No. 28353, § 1, 5-21-20, eff. 1-1-21).

To ensure regulatory compliance, the design adheres to the City of New Orleans' Building Code and Comprehensive Zoning Ordinance (CZO) (City of New Orleans, 2024). It requires a 60% reduction in Total Suspended Solids (TSS) from new developments and mandates the retention of the initial 1.25 inches of stormwater from each rainfall event. The pavement system is engineered to maintain an infiltration rate of 200 inches per hour (City of New Orleans, Municipal Code § Sec 26-15.121.7.a.3). Furthermore, the design is in accordance with the Modified Consent Decree, the LPDES MS4 Permit, and the Sewer and Water Board of New Orleans Green Infrastructure Plan. The permeable pavement will cover 50% of the parking lot, totaling 64,700 square feet.

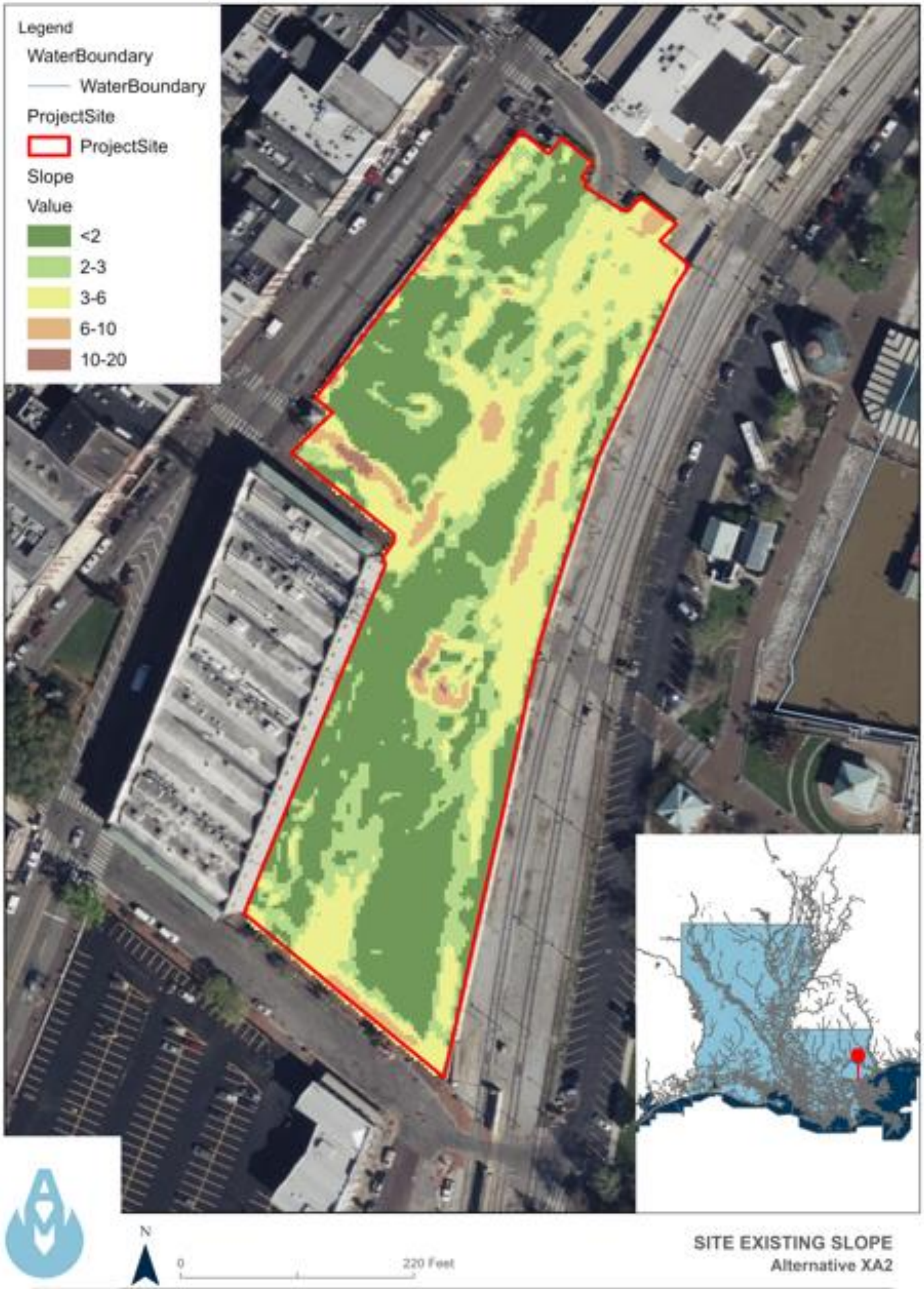

Figure 71: New Orleans Site Existing Slope

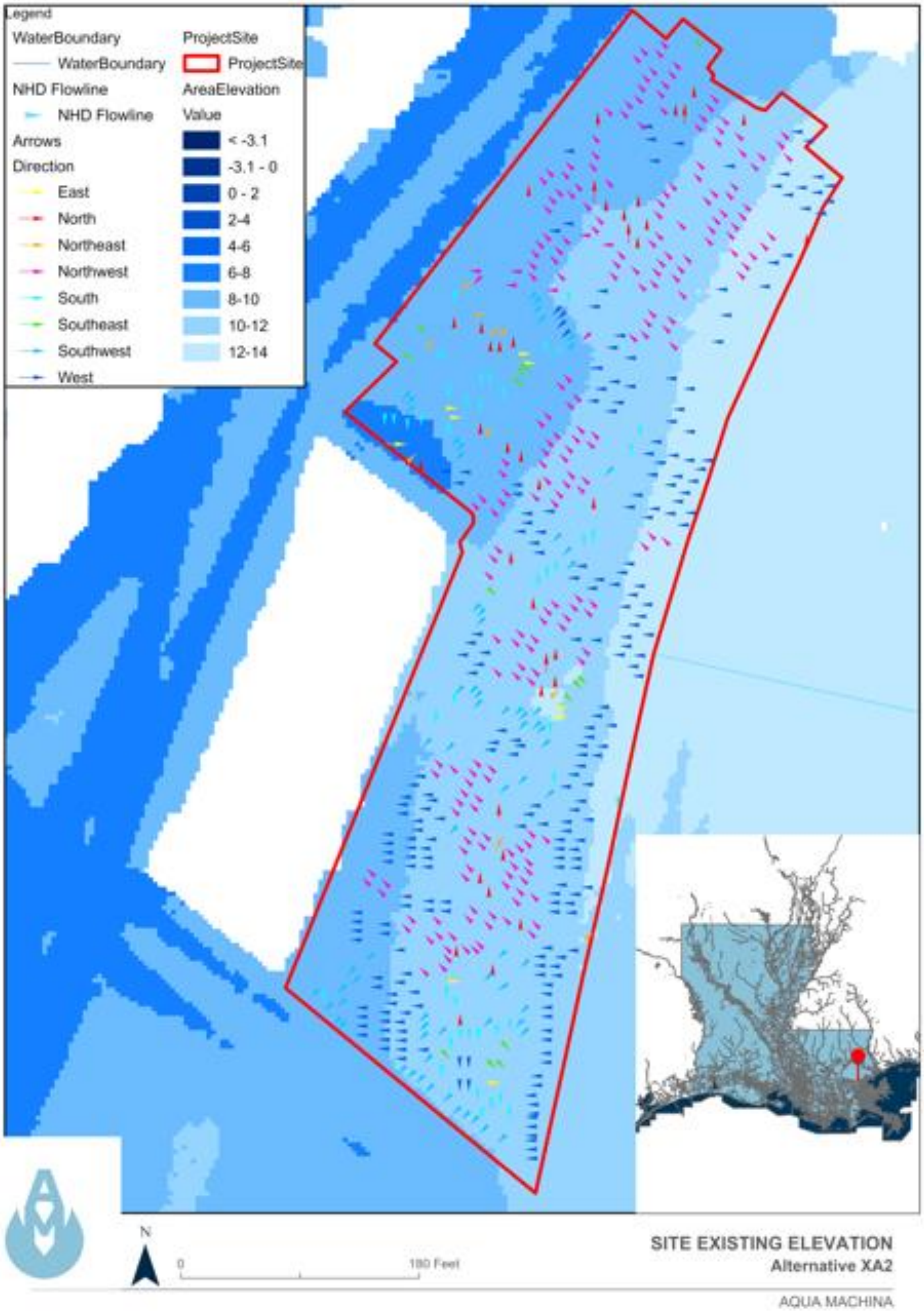

Figure 72: Site existing elevation and direction of water flow through parking lot

G.2.2.2: Hydrological Design

The existing drainage system in the parking lot will be retained to support the overall stormwater management plan. Notably, water in this area flows away from the river towards the street, but the current drainage system likely has an outlet near the Mississippi River. To enhance stormwater management, portions of the impervious surface will be removed and replaced with green infrastructure (GI) elements, such as permeable pavement. This reduction in impervious surface will decrease the volume of stormwater runoff entering the drainage system, mitigating potential adverse downstream effects. With less water entering the system, downstream areas are less likely to experience flooding or overburdening during heavy rainfall events.

G.2.2.2.1: Requirements

- Infiltration Rate: Designed to handle a rainfall intensity of 9.24 inches per hour
- Storage Capacity: Capable of storing runoff from a 10-year storm event
- Stormwater Code: Retaining or detaining and filtering the initial 1.25 inches of stormwater from each rainfall event

G.2.2.2.2: Calculations

Volume of Water to be retained:

$$1.25 \text{ inches} = 1.25 / 12 \text{ feet} = 0.1042 \text{ feet}$$

$$\text{Volume} = \text{Area} \times \text{Depth} = 129{,}000 \text{ sq ft} \times 0.1042 \text{ ft} = 13{,}441.8 \text{ cubic feet}$$

Additional Storage for intense rainfall:

Maximum rainfall retained (9.24 inches) for extreme events

$$9.24 \text{ inches} = 9.24 \text{ in} / 12 \text{ feet} = 0.77 \text{ feet}$$

$$\text{Volume} = \text{Area} \times \text{Depth} = 129{,}000 \text{ sq ft} \times 0.77 \text{ ft} = 99{,}330 \text{ cubic feet}$$

Goal Storage Capacity:

Minimum Requirement = 13,4375.5 cubic feet (to meet the 1.25 inches retention requirement)

Optimal Capacity = 99,330 cubic feet (to handle extreme rainfall events)

G.2.2.3: Pavement Design

As per the New Orleans Stormwater Ordinance, the maximum contributing drainage area to permeable pavement surface area ratio is 4:1 unless otherwise approved (City of New Orleans, Municipal Code § Sec 26-15.121.7.a.2). At least 90% of the area draining to permeable pavement shall be impervious, not including the pervious pavement (Williams, et al., 2018). Our proposed permeable pavement will take up 50% of the area with 64,700 square feet of pavement. The exact area in square feet of pavement for each section is labeled on Figure 73 below.

The permeable pavement in the parking lot will be installed slightly below the surrounding surfaces, creating a subtle gradient that allows water to naturally flow towards and collect in these areas. This design ensures that stormwater runoff from the impervious sections of the lot is directed towards the permeable pavement, where it can infiltrate the ground rather than overwhelming the existing drainage system. By strategically lowering the permeable pavement, the system effectively captures and manages runoff, reducing the risk of flooding and promoting better water management on-site.

Figure 73: Permeable Pavement Map with Area Calculations

G.2.2.3.1: Design Specifications

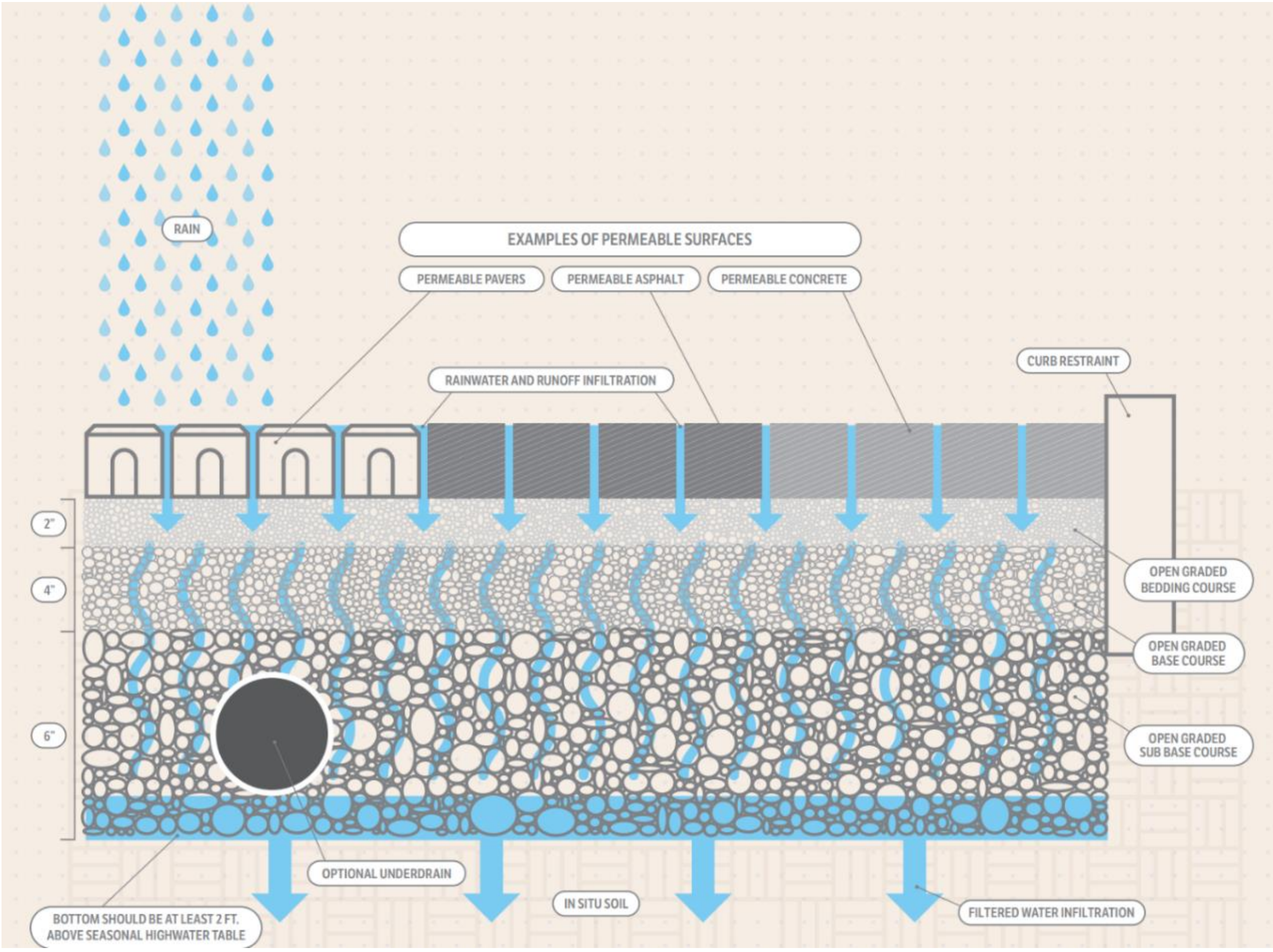

Figure 74: Cross section of example pervious pavement (FDEP)

- Surface Layer: Permeable interlocking pavers with a thickness of 4.5 inches
  - Each permeable paver is 11.75x13.70x4.5 inches and weighs approximately 44 pounds.
  - Design will require 4,823 individual permeable pavers to cover every parking stall
  - quarter-inch joint width between each paver
- Aggregate Base/Choker Course: Clean, uniformly graded aggregate with a thickness of 6 inches
- M200 Woven Monofilament geotextile made from high-tenacity monofilament polypropylene yarns with a flow rate of 145 gallons per minute per square foot (ASTM D4491)
- Subbase/Reservoir Course: Compacted subgrade with a thickness of 24 inches (City of New Orleans, Municipal Code § Sec 26-15.121.7.a.4)

G.2.2.3.2: Calculations

Sum area of permeable pavement:

64,700 sq feet = (64,700 x 144) sq inches = 9,316,800 sq inches

Paver surface area:

11.75 in x 13.7 in = 161 square inches

Number of pavers:

9,316,800 square inches of pavement/ 161 square inches per paver = 57,869 pavers

G.2.2.4: Construction

Construction of this system involves excavation to the required subgrade depth, installation of geotextile fabric, preparation of the base material, and the placement of pavers with sand or pea gravel filling the gaps (Bean, E. et al., 2023).

Climate	
Storage capacity (ft^3)	40,682.88
Cost	
Construction ($)	5,287,147
Maintenance ($/year)	55,922
Water quality	
Pathogen reduction (%)	87.91
Nutrient reduction (%)	78.15
Adsorbing pollutants (%)	87.91

Figure 75: Estimated results of permeable pavement design based on the New Orleans – Climate Resilient City Tool (Brolsma, 2022)

G.2.2.5: Maintenance

Maintenance of the proposed permeable pavement system would include annual vacuum sweeping to prevent clogging of the pavement surface, as well as regular inspection and necessary repairs to ensure that the system remains effective, which includes weeding to prevent large root systems from damaging subsurface structural components season (Bean, E. et al., 2023).

G.2.2.6: Environmental and Economic Benefits

The environmental and economic benefits of the design are significant, including reduced surface runoff, improved water quality, lower surface temperatures by reducing the Heat Island Effect, and long-term savings in stormwater management costs through earned credits under voluntary standards. The economic benefits of permeable pavement systems include more cost-effectively preventing and reducing urban flooding, thus reducing financial losses (Tota-Maharaj et al., 2024).

Table 49: Permeable pavement pollutant removals

Surface Type	Total Suspended Solids	Metals	Nutrients
Porous asphalt	94–99%	76–97%	42–43%
Pervious concrete	91%	75–92%	N/A
PICP	67–81%	13–88%	34–72%

Sources: Barrett et al., 2006; Bean et al., 2007b; Clausen & Gilbert, 2006; Rushton, 2001; UNHSC, 2007; Van Seters, 2007

G.2.2.7: Cost Estimates

G.2.2.7.1: Construction Cost Estimation

Table 50: XA2 Cost Estimation

Extensibility Alternative 2 Full Cost Estimation Details					
No.	**Line Item**	**Quantity**	**Unit**	**Cost/Unit**	**Total**
XA2					
1	Mobilization	-	Lump Sum	-	$156,565.35
2	Erosion and Sediment Control	-	Lump Sum	-	$78,282.68
3	Excavation and Fill	20,043.2890	C.Y	$12.84	$257,441.56
4	Land Valuation	2,366.4654	Acre	$124.75	$295,212.50
5	Demolition	128,594.5669	SF	$1.22	$156,885.37
6	Subgrade	4,762.7617	SY	$12.42	$59,171.93
7	Geotextile Liner	2.9521	SY	$1,942.92	$5,735.75
8	Pavers	64,700.0000	SF	$9.70	$627,590.00
5	Repaving Asphalt	63,894.5669	SF	$2.53	$161,747.91
10	Additional Labor	32.0000	HR	$58.39	$1,868.47
				Subtotal	$1,800,501.53
				Engineering (15%)	$270,075.23
				Permit and Insurance	$90,025.08
				Subtotal	$2,160,601.84
				Contingency (10%)	$216,060.18
				Total	$2,376,662.02

G.2.2.7.2: Operation and Maintenance

Table 51: Annual Maintenance and Inspection Costs for XA2. General Repairs assumes 10% of permeable paver area needs replacement.

Extensibility Alternative 2 Annual Operation and Maintenance Costs	
Inspection and sweeping	$408.04
General Repairs	$62,788.38
Overall Costs	$63,196.42

Appendix H: Education Program

The clarifying basin located on the UF campus serves as an invaluable hands-on learning opportunity for students. Courses within disciplines such as Soil and Water Sciences, Environmental Science, and Environmental Engineering benefit from field trips to the basin. By directly engaging with the basin, students can apply course content to real-world scenarios, thereby deepening their understanding of stormwater management principles and practices. This practical application enriches the learning experience and equips students with valuable skills and knowledge for addressing environmental challenges. The clarifying basin located in New Orleans creates the opportunity to be near Tulane. Multiple courses in the Earth and Environmental Sciences major directly relate to water resources and urban planning of the New Orleans area.

Table 52: Relevant Courses that would Benefit from Site Visitation (UF, 2024b)

Relevant Courses	Description	Distance to Clarifier	Time
Soil and Water Conservation (SWS 4233)	This course delves into the issues surrounding our two most valuable and most mistreated resources: soil and water. Topics discussed include soil/water resources, water conservation, water usage, water footprint, virtual water, dams... water resources management, stormwater management, ... community based and policy-based solutions.	0.2 miles	5 Min Walking
Water Resource Sustainability (SWS 4235)	The quantitative effects of human impacts on hydrologic ecosystems (aquifers, watersheds, coastal zones, lakes, and wetlands) Case studies illustrate detrimental effects of unsustainable resource utilization and beneficial management strategies	0.2 miles	5 Min Walking
Stormwater Control Systems (ENV 4411)	Chemical, physical, biological, and hydraulic aspects of rainfall runoff and control through unit operations and processes (UOPs). Stormwater physical and chemical loads. Interactions between hydrologic processes, water chemistry, sediment transport, infrastructure materials and UOPs for treatment and reuse.	0.3 miles	8 Min Walking
Environmental Hydrology 1 (ENV 4501)	Surface and atmospheric hydrology. Hydrologic processes controlling streamflow events. Practical application to stormwater management	0.3 miles	8 Min Walking
Preview of Urban and Regional Planning	An overview of the comprehensive planning process designed for undergraduates who may be considering a career in urban and regional planning or who may be	0.4 miles	8 Min Walking

(URP 4000)	pursuing studies where some knowledge of the planning process is desirable.		
Pathways to Urban Sustainability (EENS 3730)	Common environmental impacts of urbanization and approaches to minimize them, drawing on case studies from the Greater New Orleans Region and elsewhere.	5.3 miles	18 Min Driving
Groundwater Hydrology (EENS 4300)	Occurrence of water in the near-surface environment. Topics include saturated and unsaturated flow in aquifers, aquifer characterization, well hydraulics, and groundwater chemistry.	5.3 miles	18 Min Driving
Remote Sensing for Environmental Analysis (EENS 4380)	Remote sensing data are now used in geology, hydrology, meteorology, environmental sciences, geography, urban planning, anthropology, civil engineering, and environmental monitoring. Students will use the multispectral, hyperspectral, thermal, Radar, and LiDAR data for watersheds, wetlands, water quality, coastal changes, vegetation analysis, mineral resources, land use and land cover changes.	5.3 miles	18 Min Driving

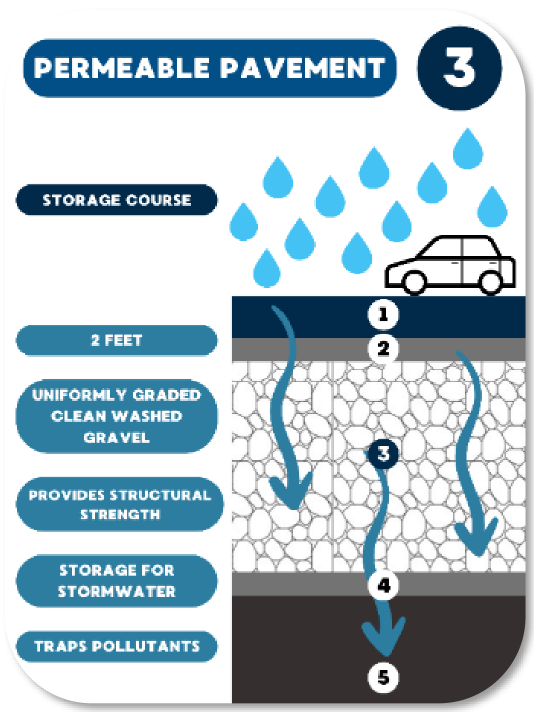

Figure 76: Permeable Pavement Signage describing the benefits and parameters of the Storage Course layer in Extensibility Alternative 2

Note: Signage enhances public understanding of project significance and fosters engagement with city's sustainability efforts.

Strategic placement of informational signage at the project locations, particularly targeting students, professors, and visitors parking in the Reitz Union Lot or tourists visiting New Orleans' French Quarter, serves as a vital educational tool. These signs aim to educate individuals about the basins or permeable pavement's purpose, principles, and its significant role in local stormwater management and ecosystem health. By providing clear and accessible information, the signage enhances awareness and understanding among the community and visitors, fostering a sense of environmental stewardship, and promoting sustainable practices in a high traffic tourist destination and university campus.