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FOR SUSTAINABLE COMMUNITIES

FINAL DELIVERABLE

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Manchester Iowa
Mixed-Use Residential Development

Prepared for Manchester Enterprises and The City of Manchester, Iowa

IOWA
CIVIL & ENVIRONMENTAL ENGINEERING

December 11, 2020

Prepared for:
The City of Manchester and Manchester Enterprises Incorporated

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Contents

Section I: Executive Summary

Figure 1. The final designed residential subdivision

This report outlines Manchester's next opportunity for growth along the Highway 20 corridor. It includes 37 residential lots that can easily be expanded to 52 or more if adjoining property owners are interested. It also includes a commercial lot at a prime location for Highway 20 for service to area residents. In the following report, our team details the various components that went into our designs for this development. This includes lot layouts, stormwater management systems, roadways and bike paths, utility lines, storm sewers, and a proposal for an extension of the development. All the designs take into consideration Manchester's master plan, which incorporates more residential development occurring around the land surrounding our 24-acre site.

Many of the lots were configured for ranch style housing with three stall garages except for the Southern boundary lots which were seized for town housing and the southwest lot which was sized or commercial use (Figure 1). One of the main objectives was to develop as many lots as possible within the site and based on the criteria provided from the clients, 37 lots were created. An additional 25 lots optionally can be added with the future development along the northern access road and the hammer head street. There is also land that has been reserved east of the commercial lot for additional lots to be added.

As seen in Figure 1, an extension was designed to serve the area east of the church property. This section of roadway is called a hammer head street. This was done to show the client an example of how the land in that area can be used as a natural extension of our site. Our team was able to fit approximately three lots in the area, also a row of townhouses could fit well into this area. The area could be extended more if the client wants purchase land from the neighboring church, this will allow more lots, making the land more profitable. An additional 15 lots optionally can be added with the future development along the northern access road and the hammer head street. There is also land that has been reserved East of the commercial lot for additional lots to be added.

This subdivision includes storm water management features that are designed to control runoff so that the 100-year runoff from posted developed onsite conditions and provide measures to improve storm water quality and do not exceed runoff levels from the 5-year storm predeveloped environment and provide measures to improve storm water quality. A flow path analysis revealed that water from on and off site falls into 3 different regions: A (in the Northeast Corner of the Eastern parcel), B (in the lower third of the Eastern parcel), and C (on the Northwest Corner of the Southwest parcel). Based off the acreages served, location within the site, and soil type, wet detention basin designs were selected for Region A and C, and a bioswale was chosen for Region B. To get the flow safely to the stormwater management basins, overland flow and culverts were designed. Outlets and emergency spillways were placed to discharge the flow that collected within the designs off the site. The Iowa Storm Water Management Manual (ISWMM) and Iowa Statewide Urban Design and Specifications manual (SUDAS) were used to size all the designs.

All of the roadways were designed in accordance with, SUDAS standards as well as Iowa Department of Transportation standards along with the clients specifications. For our design, the lane pavement widths were designed to local standards. The curb radii for all of the intersections was designed to be 25 ft, for both the arterial and local roadways. The pavement thicknesses of the roadways were calculated using the method outlined in SUDAS in Chapter 5 Section 1. All the calculations for both the local and arterial roads are below in Appendix B. For the local road we recommend a 7-inch thickness and for the arterial roadway design we recommend an 8-inch thickness of Portland Concrete Cement (PCC), both with a 6-inch subbase of compacted gravel. The roadways were design to have two different types of jointing within the concrete. For the local roads, a basket (CD) joint, along with L-2. For the arterial streets, the roads have L-1 joints and 'C' joints.

The sanitary sewer was designed in two different sections: north and south. Both of which were design to accommodate future growth beyond the subdivision. The sanitary sewer system is a gravity-based system, which meant the cut off point for the system was based around the high point in elevations which were found. A recommendation of where this high point is located in the appendix for both the North and South Systems when looking at Figure 26. The elevations were found by using the LIDAR data of the state of Iowa.

The North end of the sanitary system will run along the main Arterial Street and branch out into the other streets. The main pipe diameter was calculated to be 8 inches based of equations from SUDAS. Due to future land usage, a proposal of oversizing the pipe to 12 inches was accepted by the client, as well as the potential of a lift station being constructed to the east. The new system ties into the existing sanitary system that runs along Bailey Drive. The South end of the sanitary system handles the Southern portion of the subdivision. It uses an 8-inch diameter PVC pipe. The Southern end flows into a lift station that is in the Southwest portion of the 24-acre property. From the lift station it goes into a force main that connects into the current sanitary line that runs along Bailey drive.

For both the North and South systems the manhole diameter sizes are to be 48 inches and is to be concrete based on SUDAS specifications. The depth of the sanitary sewer pipes varies from 12 feet to 20 feet, to accommodate basements. The location of the sanitary sewer is designed to be in the center of the right of way.

The water main was designed so that is connected in a loop. In the event of a failure, this practice makes sure that people are not without water. Our water main was also designed for the future expansion of the subdivision as it grows and extends to the east. The system will be connected to the existing water main that runs along Bailey Drive. The depth of the water main is to be is 5 feet below the ground surface, this can be seen in figure 66. The locations of the water main are to be on the South and East sections of the road. From recommendations from SUDAS, the water main was designed to have a diameter of 8-inches and be PCV pipe. The system needs to be rated for a 1500 gpm. Fire hydrants were designed to located based on SUDAS specifications, which is within 25 ft of each intersection and with a spacing of no more than 450 ft.

The Multi-use trail was designed to run along the arterial road and go into the green area to the north east and connect to the existing path that runs along Bailey Drive. The trail will be 10' wide with a 2 foot clear zone on each side. The material of the path is Portland Concrete cement and the thickness as recommended by SUDAS is 5-inches.

The cost of the new residential development in Manchester was estimated from two sources. The first source was the SUDAS specification manual. This manual outlined all the specific bid items that could be used during the development of this project. The other resource was the IowaDOT bid price lists. This was used to estimate the prices of each of the specific items. The total estimated cost of the subdivision is $2,953,913. Based on our lot layout, that comes out to and with the initial 36 lots it is estimated $82,054 per lot. This total does not include the hammer head side streets or any of the other potential future development and does not include the sale of the commercial area. The cost of the hammer head area came out to $195,000. The reason that the hammer head streets are not added into the cost is because our team wants the client to see the overall cost of the rest of the project so that they can get a sense of whether or not they wish to continue with the addition. The cost of the developing the land that runs along the north entrance road came out to be $445,000. Factoring in the number of lots that we have designed for future development, that comes out to about $47,644 per lot. When making the assumption that the commercial land sells for $200,000 the price per lot goes down to $44,418. This is important as our design has fourteen lots that can fit onto that area of land, which is roughly $47,644 per lot, but does not include storm water management. The unit costs of the lots is high and there are a couple of reasons for this. The main reason is that this development is not contiguous. This means that the current area is not connected to any existing infrastructure. This includes water, storm sewer, roads, and sanitary sewer. Due to this the prices of development are more expensive. There are many strategies for the city to make up the money including eating the cost upfronting the costs reoperate the money later and using tap on fees. As the development expands to the east the cost of development will get lower.

Our team has developed a three-part phasing plan for the construction of the new subdivision. The first step in the phasing plan is to develop the north section of the subdivision which includes the arterial street and parts of the two north-south local streets This development will stop at the high points that will have been made from the grading. The second step in the process is to develop the commercial area in the south-west portion of the development. This land can be sold for a large amount of money. This is important as it will be able to recoup some of the costs of the project. The last phase of the project is to finish the rest of the development which includes the land that is south east. Each phase of the project will require grading, water, and sanitary sewer development as well as storm water management including storm sewer. The first phase of the development will cost $1,284,586 and the second and third phases combine to cost $1,699,327. This phasing plan is flexible and can be amended to begin with the commercial lot if desired.

Section II: Organization Qualifications and Experience

Our Company

WSBK Engineering is a team of senior civil engineering students at the University of Iowa. Together, our company specializes in transportation engineering, but has experience working in water resource design and structural design and analysis.

Organization Location and Contact Information

Primary Contact:

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Organization and Design Team Description

Emily King was the External Project Manager and acted as the main point of contact for the clients. This included making sure that every party was up to speed during the different developmental phases of the project. This was done by sending out meeting invitations, agendas, and meeting minutes, as well as maintained a strong email correspondence in order to communicate additional questions or updates. Additionally, she worked alongside Jordan to delegate tasks and establish a general structure that guides the sequence of the project's development. Emily's primary focus on the design was over the stormwater management elements.

Jordan Strouse served as the Internal Project Manager for this project. The responsibilities associated with this role were the organization and facilitation of Team meetings, meetings with our faculty advisor, preparation and distribution of meeting agendas, and organization and facilitation of team assignments. As the design process proceeds, he led the design and development of the project along with Emily and assisted Emily, Travis and Brandon in their tasks.

Brandon Beaver was the Editor for the Manchester design project. The responsibilities of the editor consisted of the writing of the report and the preparations of graphics in the report. The Editor had the final editing decision when it came to any disputes and oversaw all formatting and styling of the reports. In addition to those responsibilities, the Editor assisted all other team members during the design process.

Travis Wright was the role of Technical Support for the Manchester Subdivision. The responsibilities of technical support were to help create and store documents in an organized fashion and was the go-to for drafting, computer, or electronic needs. Along with helping the team with technical issues he used his prior experience to develop grading and construction documents under the lead of the Project Managers.

Section III: Design Services

Project Scope

WSBK Engineering has provided the preliminary designs for a 24-acre residential mixed-use neighborhood for the city of Manchester and Manchester Enterprises Incorporated. The location of the neighborhood is right by the intersection of Highway 20 and Bailey Drive as seen in Figure 6. The residential neighborhood includes a range of housing options: single-family ranch (low density), condos, and multifamily homes (medium density). The final design sheets included street layouts, a bike path, water, sanitary sewer, and storm sewer lines, as well as stormwater management designs. The designs took into consideration future site developments, which are projected to happen in the surrounding parcels. A sanitary sewer lift station has been included and designed to tie into the existing sewer lines.

Project Deliverables

- Site design of the neighborhood including lot locations and multi-use trail
- Access road design and grading plan
- Design of the new bike trail connecting to existing trail
- Design of new utility network connecting to existing, including a lift station
- Design of stormwater management systems
- Phasing plan
- Cost estimate
- Master Plan of surrounding properties

Work Plan

The development of the project has been split into four phases that will structure the design process. These phases include the following: design proposal, design development, design creation, and final design. Phase one includes the development of three different general layout designs and creating our proposal. Once the proposal has been discussed with the clients, a singular design will be selected. Phase two, which includes drafting a more detailed site layout will then commence. This includes stormwater management systems, earthwork, access roads, sidewalks, and lot designs, as well as the design of the utility networks and the bike path. After that, phase three will incorporate final design details and drafts of plan sets, renderings, reports, and presentations. Lastly, phase four is producing the final design details for the clients. A timeline of the project is demonstrated in the Appendix D below.

Section IV: Constraints, Challenges, and Impacts

Constraints

The new residential development was under the influence of several constraints. The biggest constraint was that undeveloped land East of the site drained into the 24-acre development. This meant that the stormwater management designs had to be sized to accommodate large volumes from the additional land. The amount of space that the stormwater management designs need along with the 24-acre limit of the site limited how many lots could be designed. While the clients wanted to maximize the land to fit as many houses as possible, the first priority was that stormwater could be properly managed to mitigate flooding.

Another constraint was the private property to the west of our proposed site that separates the land from existing infrastructure. The design had to go through it while still making the neighborhood functional. Existing water mains and sewer systems also constrained the design, as the new additions had to be able to be attached onto the existing systems. Lastly, the time constraint of December 11th limited the amount of detail into the designs that our team was able to go into.

Challenges

There are several challenges that were addressed and remedied for design of the new development. Based on the Iowa flood maps produced by the Iowa Flood Center, Iowa Department of National Resource, and the Army Corps of Engineers, the Northwest side of the property is at risk for a 1% chance (100 year) flood Figure 22. While the rest of the property is outside the flood plains, this area had to be taken into consideration during the design to ensure that we were not designing homes at risk for flooding. To mitigate this, grading was done so that water would no longer be collecting there. Instead, the water would be routed to a wet detention basin in the Northeast parcel. One of the stormwater wet detention basins was placed in this area.

Another challenge was that the Northeast corner of the project site was under speculation that it could be part of a wetland based on aerial photography of what appeared to be water damage, paired with a map that showed wet lands existing on the surrounding offsite area (Figure 22 and 23). If that was the case, there were two options: leave the wetland untouched (which would further restrict the amount of land we can develop) or plan to develop it with the understanding that additional wetlands will need to be created elsewhere at 1.5 times the size of what was destroyed. After speaking with the clients, it was determined that since the wetland map did not specifically list the site as being part of a wet land, then we could plan our designs assuming that no wet lands existed on site.

One challenge that the client will need to deal with is that the mixed use and connected to neighborhoods infrastructure is not contiguous. This meant that all of infrastructure needed to be built and connected to the existing within the area. This infrastructure included the water main, sewer system, storm system, roads and a lift station. This was a challenge for the client as it requires more money to build the initial development.

An additional challenge was designing the storm sewer system so that all of the runoff goes to the intakes and then connects to the current storm sewer system that Manchester has in place. The contour map Figure 24 showed that there were varied elevations throughout the different parcels of land which affected how drainage for sewers was be placed within the neighborhood. The lower Southwest parcel is in a particularly low point, which required a lot of grading work.

Societal Impact within the Community

Building a new residential neighborhood can have a lot of direct impacts in Manchester. It creates housing, which helps in attracting more residents. Manchester is a smaller town with a population of a little over 5,000 residents so any increases will be noticeable and can also help boost the city's economy. Since Manchester is located in the middle of Waterloo, Cedar Rapids, and Dubuque, it is a very desirable spot for frequent commuters to settle in, and this additional housing creates more opportunities for those looking to make the move. Neighborhoods have the ability to strengthen community relationships by creating a positive environment to interact with those in your community. By designing with the livability of the neighborhood in mind, there is a real opportunity to connect the residents with each other. Bike paths can aid in this by offering activities to do in shared spaces while also promoting a healthy and active community. If a commercial zone is to be designed into this neighborhood (as proposed in our deliverables), it will help increase the economy as the location is in close proximity to Highway 20 and this is an area of the community that is currently not served by convenience stores.

Section V: Proffer of Alternative Solutions

The following layouts were presented to the clients during the project proposal meeting. The final product is a modification of these three proposals and is described later in this report under Section VI.

Alternative 1- Low-density, mixed use with commercial sector, wetland pond, weaving trail

WSBK Engineering's first option for a development is a low-density layout with a commercial sector. The neighborhood could include a mix of zero entry senior living and single-family homes. The commercial sector in the Southwest corner would help offset the cost per lot and is an ideal location for a convenience store since this site is on the outskirts of Manchester and right off Highway 20.

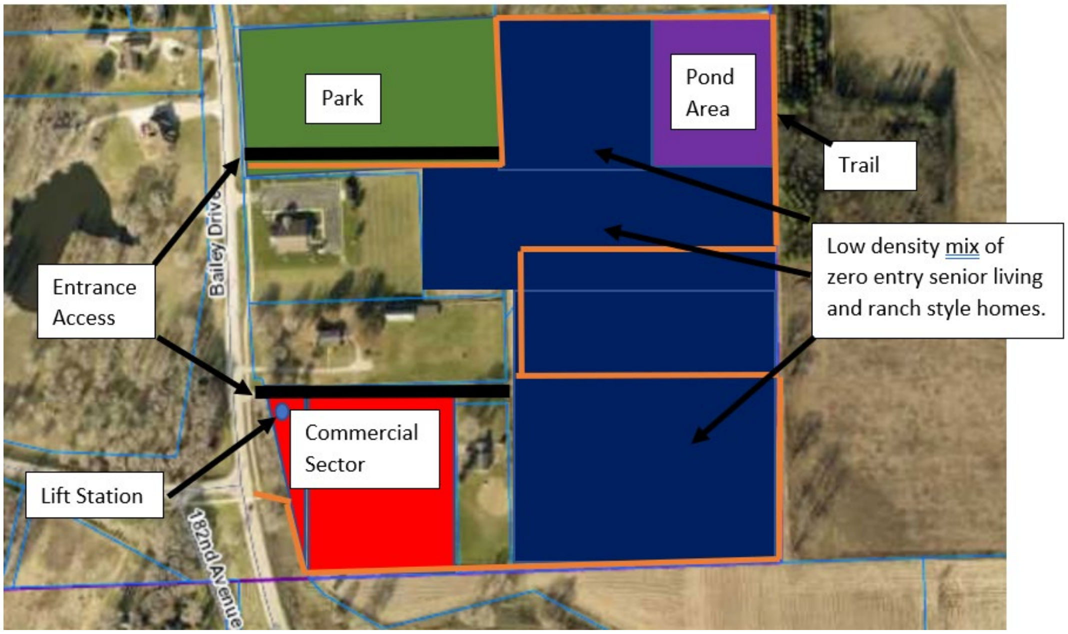


Figure 2. Alternative 1 layout

Alternative 2- Medium density, all residential, green space, border trail

The second option consists of a medium density mix that could include single-family homes, duplexes, and zero entry senior housing. The Northeast corner of the development will be left as green space if classified as a wetland. If it is not a wetland or if the client wishes to take on wetland remediation, it will be developed into more residential housing.

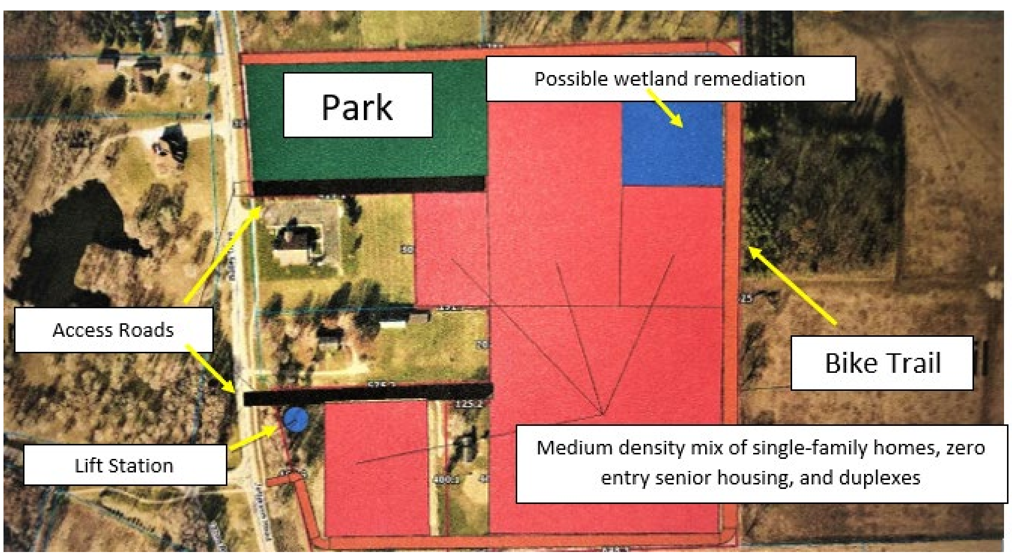


Figure 3. Alternative 2 layout

Alternative 3- High density, all residential, pond, border trail

The third option is designed to be a high-density mix with the potential for single-family homes, zero entry senior housing, condos, and multi-family housing. A wetland retention pond is to be designed on the Northeast corner of the site.

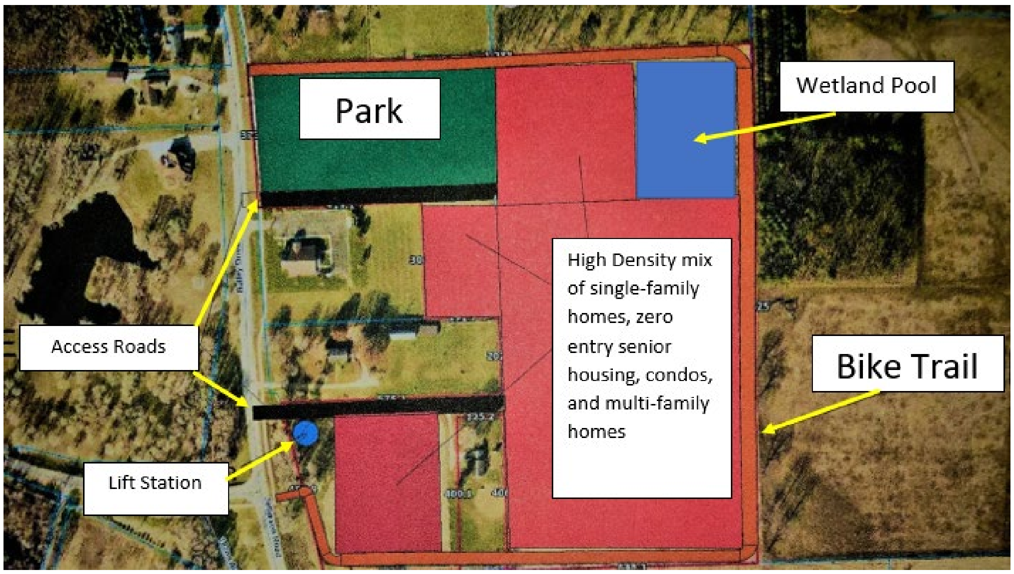


Figure 4. Alternative 3 layout

Stormwater Management Best Practice Selection

During the first round of planning, the stormwater management designs in Regions A and B were selected to be wet detention basins and the design for Region C was chosen to be bioretention cells within a dry detention basin. This was due to tables provided in the ISWMM, that used the acreage served (6.99 acres for Region A, 12.80 for Region B and 4.39 for Region C) and soil type (split between B and C for reach region) as selecting criteria. An analysis of the water tables of the site would have been the deciding factor, between whether or not A and B should be wet or dry detention basins, however this was not available during the time frame of this project. Aerial photography revealed potential wet conditions in the Northeastern corner of the stie, which suggested the water table could be closer to the surface, and the wet detention basin was ultimately chosen because it takes up more space and thus is a more conservative approach when laying lots around it.

After talking with the clients, it was decided that because Region B was in a prime spot for residential housing and because of that the clients wanted to take up as little space as possible with the stormwater management design. In lieu of placing a wet detention pond in Region B, 2 additional lots were able to get added to this zone, and the remaining remnant of space is proposed to be used to help manage storm water quality. This space was designed to be converted into a bioswale, and that the excess runoff would flow through it and into Region C. This additional runoff justified making Region C's design into a wet detention basin.

Justifications for these selections can be found in Appendix A Section 2. Area calculations and soil information can be found in Appendix A Section 3.

Section VI: Final Design Details

Stormwater management

A flow path analysis was performed in Civil3D to distinguish how many natural water catchments existed on the site. As seen below in Figure 5, 3 catchments were located (regions A, B, and C), which determined where stormwater management practices should be developed.

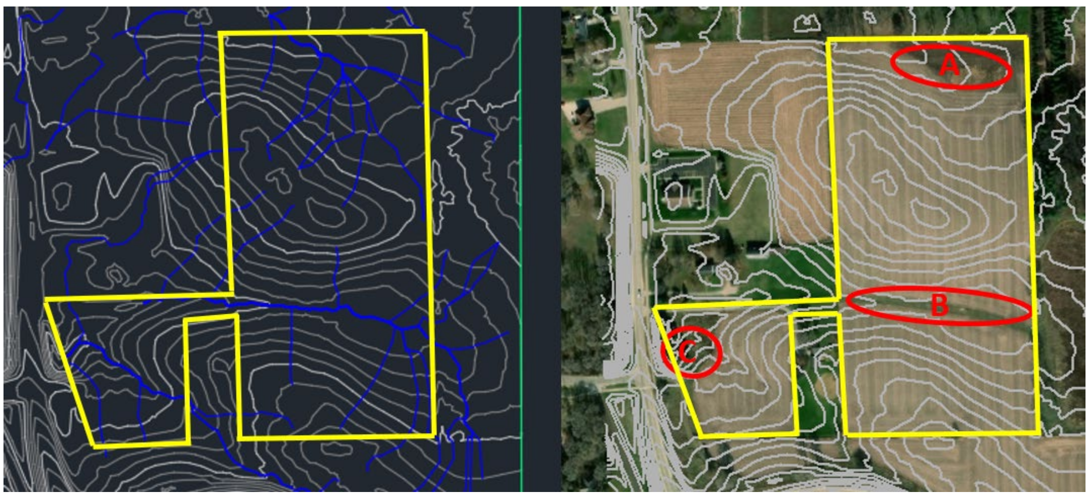


Figure 5. Stormwater management locations (regions A, B and C) based on catchments

An extended flow path analysis of the territory east of the site showed that the onsite catchments would also be collecting runoff from offsite (Figure 6). This data was accounted for when sizing the stormwater management designs, because the largest volumes of runoff came from post development onsite conditions combined with predevelopment off site conditions.

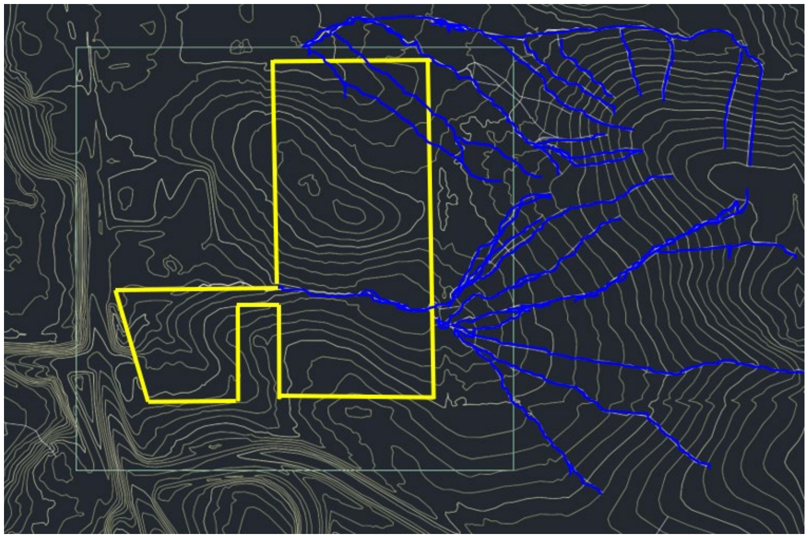


Figure 6. Flow path of offsite runoff

Storage Volume

In accordance with Manchester's stormwater codes, the stormwater designs was sized so that runoff from a 100-year storm post development did not exceed flows from a 5-year predevelopment storm. Methodology found in Chapter 2G of SUDAS were used for obtaining the channel protection volumes which were then used to size the storage and can be observed in Appendix A Section 3.

The rational method was used (based off a 24-hour 100-year design storm) to calculate the runoff values for onsite and offsite flow that would deposit into the 3 stormwater management regions under both predeveloped and post development circumstances. From those values, the runoff volume was computed to find the quantity of water that large storms would produce. The NCRS method was then used to find the peak flow rates from 5-year predevelopment and 100-year post development storms. These calculations factored in the B and C hydrologic soil types that were found within the site via Beacon's soils database. The land was classified as meadow for predeveloped land and ½ acre lots and commercial district (specifically for the Southwest parcel) for post development.

For a 5-year storm predevelopment, the onsite peak flow rates were: 1.7 cfs (Region A), 4.47 cfs (Region B), and 1.89 cfs (Region C). The onsite 100-year storm post development peak flow rates were: 12.48 cfs (Region A), 25.20 cfs (Region B), and 18.10 cfs (Region C). The 100-year storm flow rates were used as the peak inflow discharges, and the 5-year storm flow rates were used as the peak outflow discharges. The ratio between the storage volume and runoff volume was then found to be: .51 (Region A), .47 (Region B), and .55 (Region C). Factoring in the runoff volumes from the rational method calculations, this revealed the needed storage volumes that would be required to hold runoff from a 100-year post development storms to maintain 5-year predevelopment level would be: 52,436 CF (Region A), 49,542 CF (Region B), and 76,507 CF (Region C).

Water Quality Volume

The water quality volume is a measure of the amount of volume needed to store and treat runoff that comes from 90% of the storms that the site is likely to experience. These values were calculated using a standard 1.25 inches of precipitation for Iowa, and with the percentage of impervious land to be 0% for the predeveloped site and 63% for the post developed site (low density housing). The water quality volumes were found to be: 1,585 CF (Region A), 2,904 CF (Region B), and 995 CF (Region C) for predevelopment storms and 19,562 CF (Region A), 35,835 CF (Region B), and 13,530 CF (Region C) for post development storms. Calculations can be found in Appendix A Section 4.

Wet Detention Pond Design

The wet detention basins were designed to be trapezoids with a 2:1 length to width ratios. As specified in the ISWMM, the deep pool was sized to store two times the water quality volume. For Region A, the target deep pool storage was 39,122 CF and for Region C, the target storage was 27,060 CF. The deep pool is sized for a depth of 10 ft with a 3:1 side slope. 2.5 ft from the water surface level, there is a safety bench that is 5 ft wide and back pitched as an additional safety precaution. The actual volumes for the deep pools were 40,290 CF (Region A) and 28,520 CF (Region C).

The shallow zone was sized to hold the capacity of water that will prevent the 100-year storm post development runoff from over taking the 5-year predevelopment storm runoff values (52,436 CF for Region A and 126,049 CF for Region C). The shallow zone continues the deep pool's 3:1 slope for an additional foot before tapering to a 6:1 slope and at a depth of 5 ft. The actual volumes for the shallow pools were 52,544 CF (Region A) and 126,065 CF (Region C). With the inclusion of a 12 ft wide path around the perimeter for maintenance vehicles, the outside dimensions for the wet basins for Regions A and C are respectively 84 ft x 156 ft and 126 x 240 ft. In accordance with SUDAS, the top of the basin embankment is an additional 1 foot above the 100-year ponding elevation. All basin calculations can be found in Appendix A Section 5.

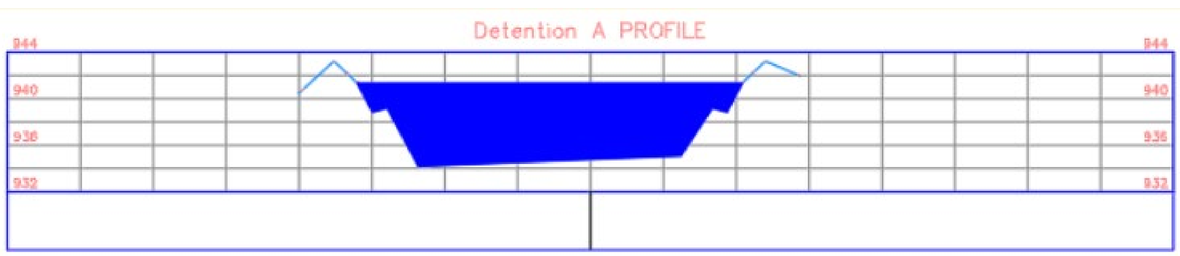


Figure 7. Profile of wet detention basin A

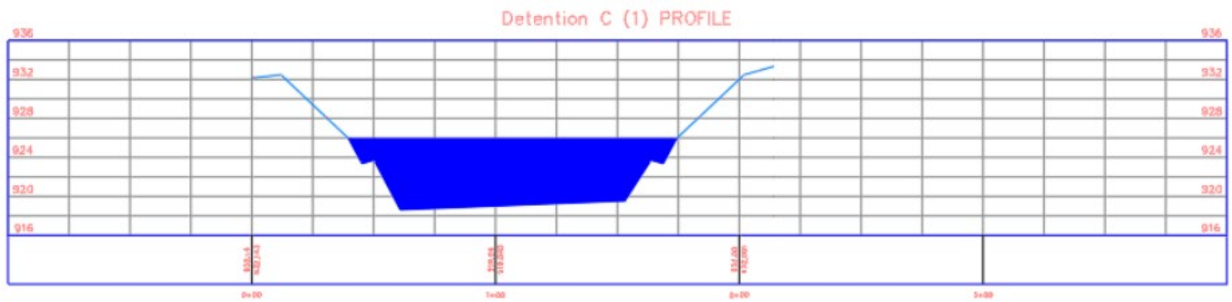


Figure 8. Profile of wet detention basin C

It is noted that since the soils in the regions where the stormwater management designs are being place fall into hydrologic group B, a clay liner should be added in order to maintain a permanent pool.

Overland Flow Routes

The overland flood routes were sized to carry the flow rate of runoff from the 100-year storm of the off-site (predevelopment) catchments combined with the runoff from the 100-year storm of the onsite (post development). The channels were designed as a trapezoid with a 4:1 side slope. The ISWMM set a minimum width parameter of 4 ft and based off the Buffalo Grass selection of turf, no more than a 4% longitudinal slope. While accounting for an additional recommended 1 ft of freeboard above the 100-year storms and making sure the velocities stayed under 5 ft/s, the designs were sized and placed in low graded points that would allow the runoff to flow through them and deposit into the stormwater management designs. All calculations can be found in Appendix A Section 6.

Due to the nominal amount of offsite run off that would affect region C, combined with the assumption that most of the southwest parcel would be impervious due to the nature of commercial zoning, an overland flow route was not designed to transport offsite runoff through the site for that parcel.

For both wet detention basins in Region A and C, emergency spillways were designed. They follow the same criteria as the overland spillways. Basin A's emergency spill was designed for the excess runoff from the basin to be carried into the offsite territory north of the site. Basin C's emergency spillway was designed to carry excess flow into the ditch along Bailey Drive.

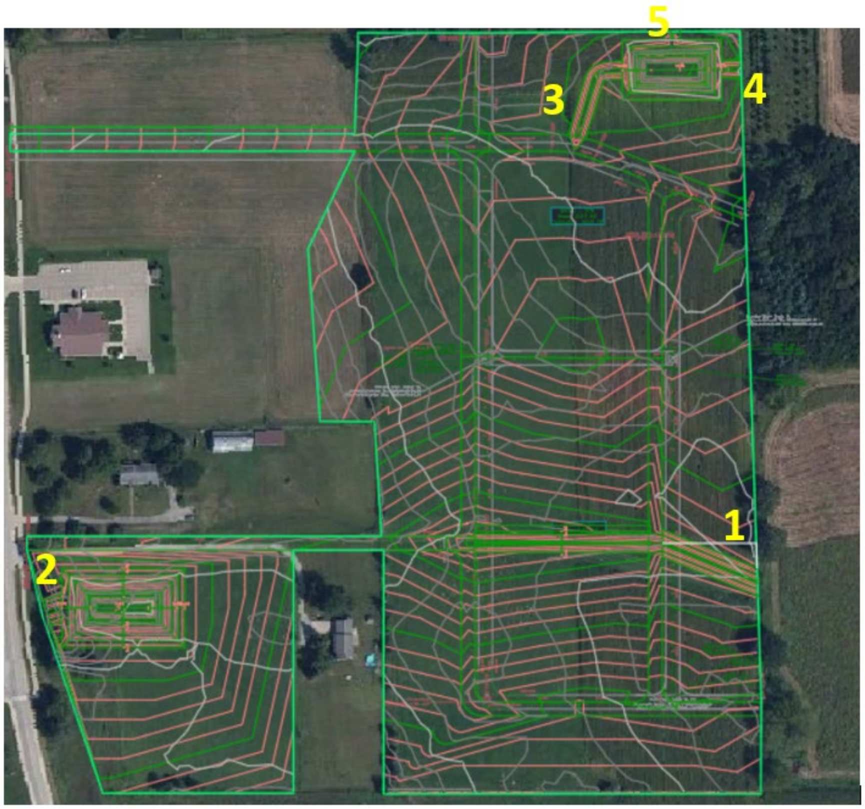


Figure 9. Map of the overland flow routes

Bioswale Design

The bioswale in Region B was sized to handle the flow of a 100-year post development onsite storm plus the 100-year predevelopment offsite storm, however it will be most effective under small storms with small velocities. This is because smaller storms are dirtier than large storms due to the high debris to water ratio that occurs during them. Due to the time constraints of this report, the bioswale was not designed to the typical criteria of a bioswale per the ISWMM and SUDAS manuals with pretreatment and subdrains. Instead, this passage was developed as an overland flow route, but with the additional recommendations to plant native species within it to slow down the flow and allow for filtration of water to occur. Nitrates are a large concern for this community, and when the bioswale is ponded with water, this creates an environment for the sediment to go anaerobic which allows bacteria to populate that will remove nitrates. A list of native plant species recommended for bioswales from the Iowa Stormwater Education Partnership can be found in Appendix A Section 7.

An additional difference between the overland flow routes and the bioswale is that the maximum velocity of the bioswale has been reduced to 1 ft/s per ISWMM standards. The flow will run through the bioswale from East to West and have a freeboard of 1 ft during 100-year storms. It is recommended that lots surrounding the bioswale have a minimum low opening that is 1 ft above the sidewalk elevation along the street to the west. This will protect the houses from potential flood damage in events that are bigger than the 100-year storm. The depth is 4 ft, the bottom width is 5 feet, and the side slope is a 4:1. The bioswale has the capacity to hold 42 cfs (the target peak flow rate is 38 cfs for the 100 year storms in Region B) while maintaining a velocity of .98 ft/s. Bioswale design calculations can also be found in Appendix A Section 7.

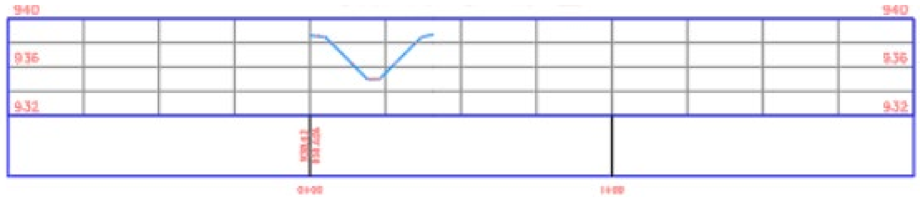


Figure 10. Profile of the bioswale

Culvert Design

There are two culverts in the development. The first culvert carries offsite flow from the eastern overland spillway underneath Travis Street into Region B's bioswale, and the second culvert carries flow from the bioswale's outlet West underneath the Southern access road where it eventually deposits into Region C's wet detention basin. Similarly to the spillways, these culverts were sized to handle 100 year storms from the pre development offsite conditions combined the post development onsite conditions. The slopes of the culverts are 3.57% (eastern) and 1% (western), and the lengths are 28 ft (eastern) and 638 ft (western). Both culverts are 30 inches in diameter, made out of Reinforced Concrete Pipe, and the inlet configuration has been mitered to conform to the slope. HY-8, a software from the Federal Highway Administration was used to simulate 100-year storms and check that the culverts were big enough to avoid roadway topping. Additional information can be found in Appendix A Section 8.

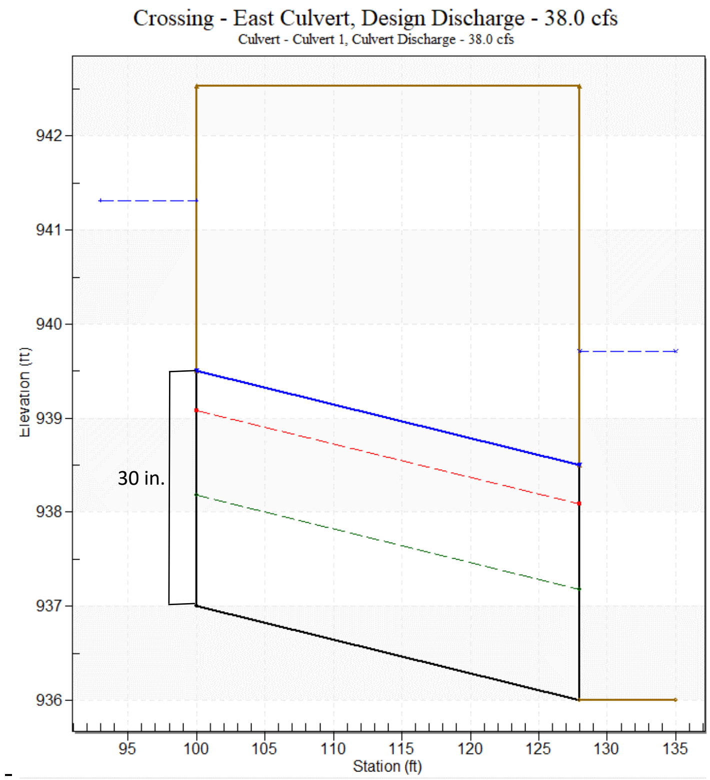


Figure 11. View of the Eastern most culvert

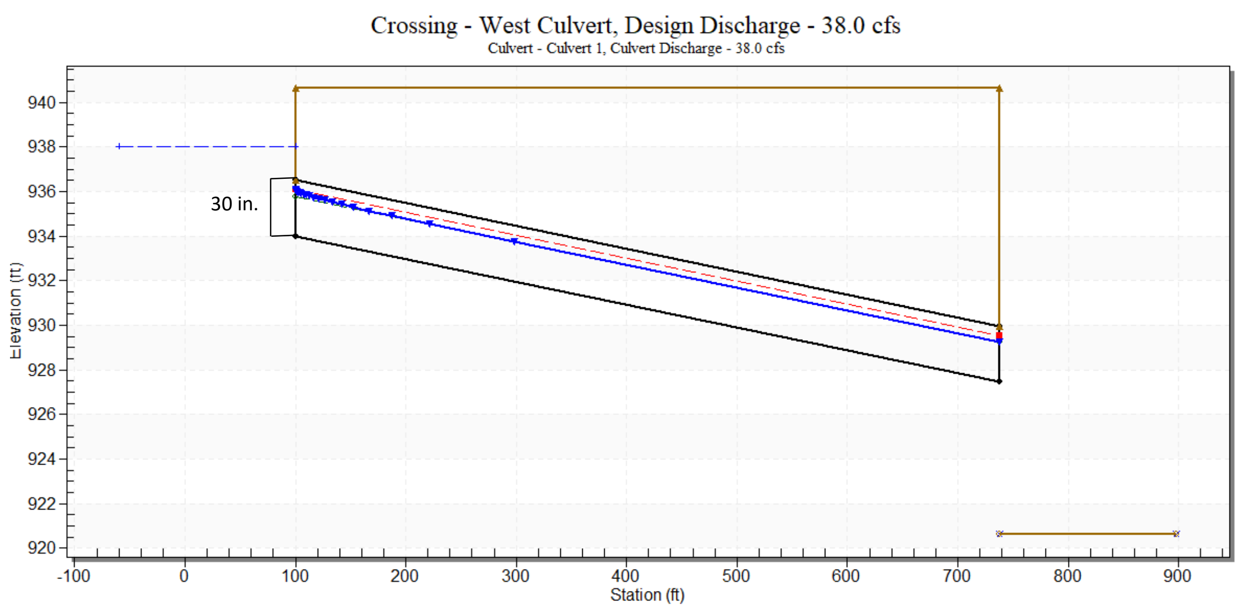


Figure 12. View of the Western most culvert

Outlet Design

3 outlets in total are to be utilized for the two wet detention ponds and the bioswale. A multistage outlet was chosen with a grate on the top to prevent objects from entering the system (Figure2

13, 14, and 15). It is advised that a slanted grate be utilized for safety purposes so that objects get caught are forced upward instead of being pinned against the opening (Figure 16).

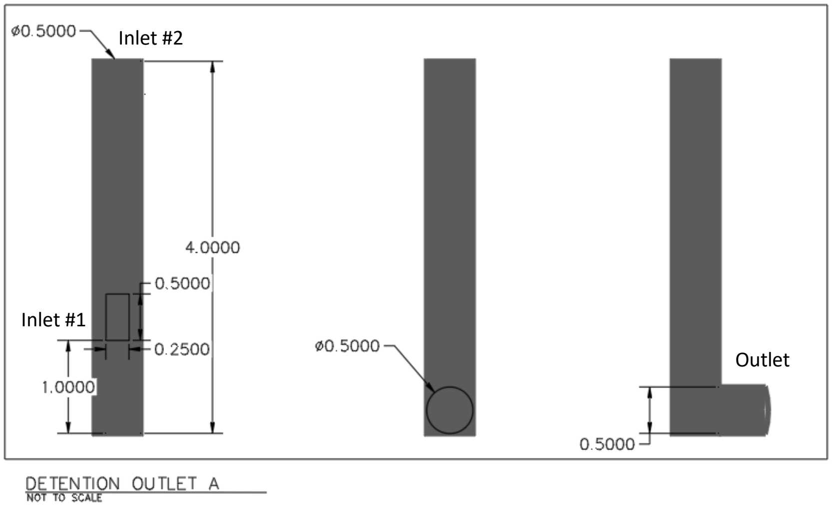


Figure 13. Design of the outlet for wet detention basin A

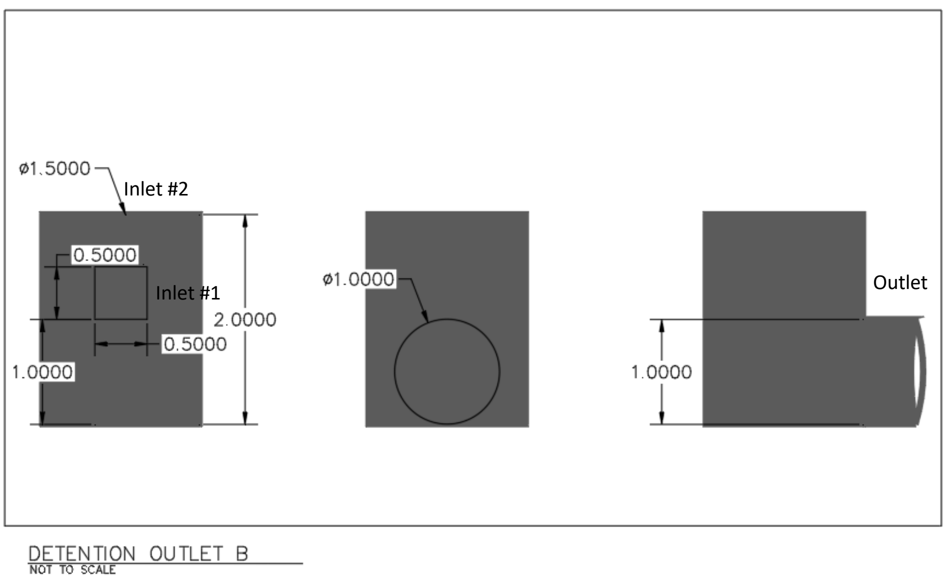


Figure 14. Design of the outlet for wet detention basin C

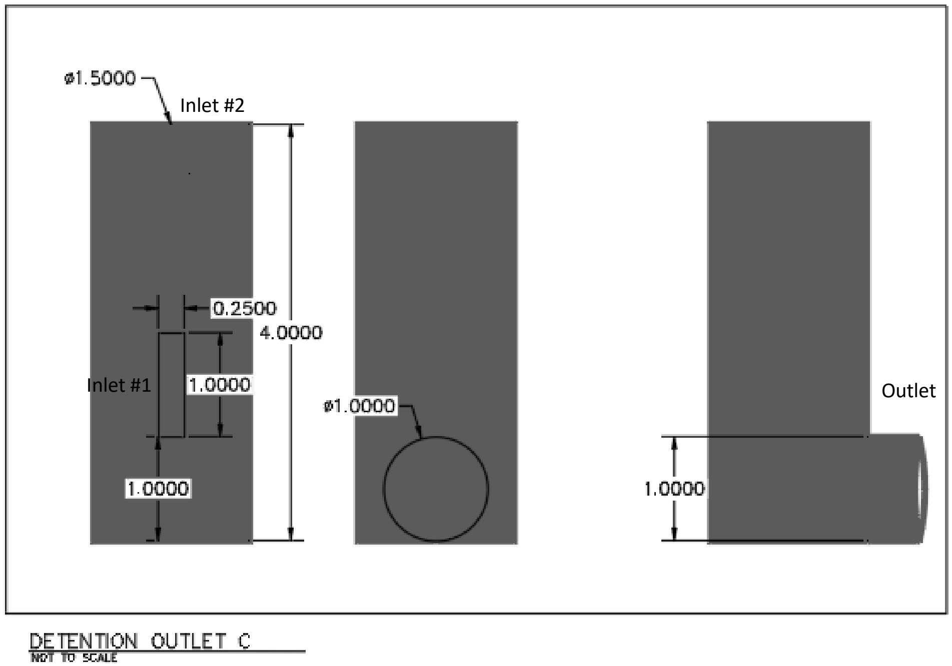


Figure 15. Design of the outlet for the bioswale

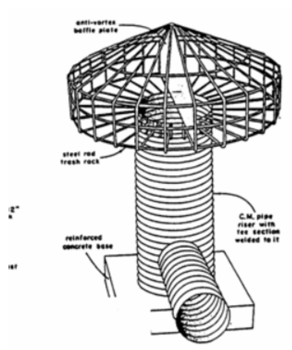


Figure 16. Example of preferred grate style on top of the outlet

The outlets are buried 1 ft into the ground and have a 3 ft head during 100-year storms. They should be placed so that rectangular inlet is level with the top of the deep pool. During larger storm events, water will be able to flow into the outlet from both the rectangular inlet and down through the circular top. The guiding parameter when sizing the outlets was that the outlet flow was not to exceed the flow rates of the 5-year predevelopment storms. Basin A's outlet has a flowrate of 7.94 cfs (target of 11.21 cfs), the outlet for the bioswale's flowrate is 2.03 cfs (target of 2.9 cfs), and Basin C's outlet has a flow rate of 7.40 cfs (target of 7.40 cfs). The rest of the design parameters can be found in Appendix A Section 9.

Roadway Design

All of the roadways were designed in accordance to SUDAS standards as well as Iowa Department of Transportation standards along with the client's wishes. For our design, the lane widths were determined by the type of street. We have one arterial road with the rest being local roads. These were determined based on the expected number of vehicles traveled. The lane widths were determined with the help of the Two-lane Urban design spreadsheet from the Iowa DOT, as well as SUDAS recommendations. For our residential neighborhood, our arterial street has lane width of 15 ft and the local street has a lane width of 12 ft. The grade of the gutter has a minimum of a .5% slope and a 2% transverse slope (based on Chapter 5C-2) for the crown of the street. Using table 5C-2.09 in SUDAS, the curb radii for all the intersections was designed to be 25 ft. The client specified that they would like rolled curbs, however the Civil3D (design software that was used) did not have that specific type of curb. Regardless, all of the calculations were done with a rolled curb in mind. A cross section of both the arterial and local road ways can be seen in Figures 17 and 18.

The pavement thicknesses of the roadway were calculated using the method outlined in SUDAS in Chapter 5 Section 1. All the calculations for both the local and arterial roads can be found in Appendix B. The calculations were determined based on the AADT(average daily traffic). Due to the road not existing yet, a traffic study was not able to be done, so AADT estimations were made based off of IOWA DOT values which can be seen in Table 42 . The ESAL's of the road were then found by using the AADT, an estimated growth factor of 2%, the percent of trucks, the truck grade, and the design life of 25 years. Using this information, tables 40 and 41 were used to determine the thickness of the road. For the local roads, the pavement thickness was determined to be 6-inches and for the arterial road, 6.5-inches. These values would preferably be based off soil testing information that could determine a CBK value, however this was not available to us. In the absence of soil information at the project site, we recommend a slightly more conservative design for the thicknesses. For the local road we recommend a 7-inch thickness and for the arterial roadway design we recommend an 8-inch thickness of Portland Concrete Cement (PCC). In addition, a 6-inch subbase of compacted gravel was requested by the clients. The roadways were design to have two different types of jointing within the concrete. For the arterial roads, the client requested a basket (CD) joint, along with O-C with L-2. For the local streets, the roads will have L-2 joints and 'C' joints.

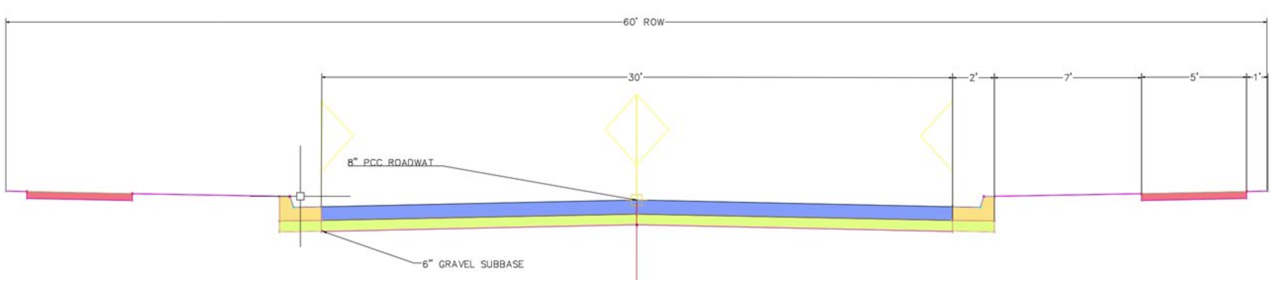


Figure 17: The Cross section of the Arterial Roadway

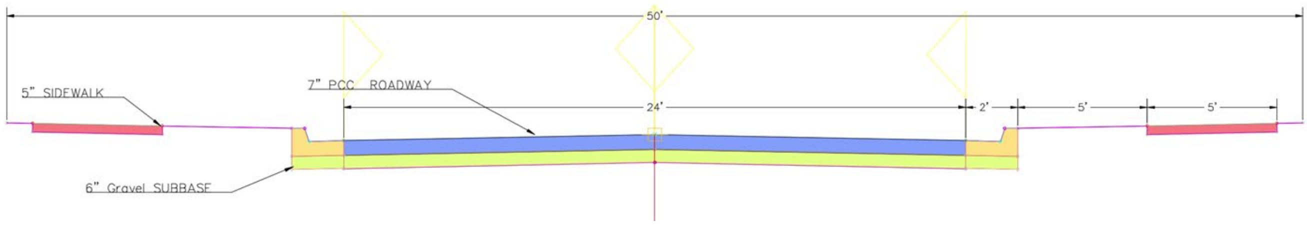


Figure 18: The Cross section of the Local Roadway

Sanitary Sewer Design

The sanitary sewer was design in two different sections: North and South. Both were designed to accommodate for future growth of the subdivision. The sanitary sewer system is a gravity-based system, which required finding the high points in the evaluation to determine where the cut off point for the system would be. A recommendation of where this high point is below for both the North and South Systems when looking at Figure 26. The elevations were found by using the LIDAR data of the state of Iowa.

To size of the sanitary sewer, the estimated population needed to be calculated based on the area of the proposed system and the number of lots that can fit into that area. Based on Table 44 it was determined that there would be 10 people/ AC. The peak flow from the Minimum Design Flow Peak Ration Chart was then determined. Once found, equation 3B-1.01 was used to calculate the Discharge (Q). Using a Manning's equation calculator, the various diameters of the proposed pipes, the slope of the proposed area, the percent full of the circular pipe which is 96% and the manning's coefficient for PVC for the North and South Areas respectively were plugged in. Once the calculator computed the discharge, this was then compared to the Discharge (Q) number, and the pipe diameter that produced the best fit the Discharge number was the pipe that was chosen.

The North end of the system will run along the main arterial street and branch out into the other streets. The main pipe diameter was calculated to be 8 inches of PVC based of equations from SUDAS. Due to future land development off site, we proposed the option to oversize the pipe to 12-inches to account for additional flows, which the clients accepted. The system will tie into the existing sanitary system that runs along Bailey Drive. The South end of the system handles the southern portion of the subdivision. The pipe size was found to be an 8-inch diameter PCV pipe. The Southern end flows into a lift station that is located in the Southwest portion of the 24-acre property. From the lift station it flows into a force main that connects into the line that runs along Bailey drive.

For both the North and South systems the manhole diameter sizes are to be 48 inches and made of concrete based on SUDAS specifications. The depth of the sanitary sewer pipes varies from 12 ft to 20 ft, this way there is plenty of room to accommodate for a basement. The location of the sanitary sewer is based off SUDAS specifications and is supposed to be in the center of the right of way.

The South Sanitary system was designed to flow to a lift station that flows to a force main that connects to the current sanitary line that runs along Bailey drive. The location of the lift station is to be on the South West part of the 24-acre property where the sanitary basin is. The peak daily flow into the lift station is 338,130 gallons per day which is equivalent to 235 gallons per minutes of waste. Our team recommends that the client install a backup generator that connects to the lift station as it will be needed in the event of a power outage. Since the lift station has a peak design flow that is less than 1500 gpm, our team recommends only using two pumps. As the residential area expands a third pump may be needed to compensate for the new lots. A record of the sanitary system calculations can be seen by looking at Sanitary Sewer appendix C below. The locations of the sanitary system can be see in Figures 20 and 21 below.

Water Main Design

The water main system is designed differently from that of the sanitary sewer. The water main was designed so that it is a pressurized system that is connected in a loop. In the event of a failure, this ensures that fewer people are without water. Our water main was also designed for the future expansion of the subdivision as it grows and extends to the East. The system will be connected to the existing water main that runs along Bailey Drive.

Based on the location of Manchester, SUDAS specifies that the depth that the water main is to be is five ft below the ground surface. This can be seen in Figure 66. The locations of the water mains are to be on the South and East sections of the road. Off of recommendations from SUDAS, the water main was designed to have a diameter of 8 inches and the material be SDR 26 PVC pipe. The location of where the new designed water network connects to the existing can be seen below in Figure 19.

The second part of the water main system considers fire hydrants and is designed and sized for fire flows. From SUDAS, the system needs to be rated for a 1500 gpm which is from Table 52. Fire hydrants were designed to located based on SUDAS specifications which is within 25 ft of each intersection and with a spacing of no more than 450 ft. The fire hydrants are placed in the area between the back of curb and the sidewalk in the right of way. The locations of the water main systems can be see in Figures 20 and 21 below.

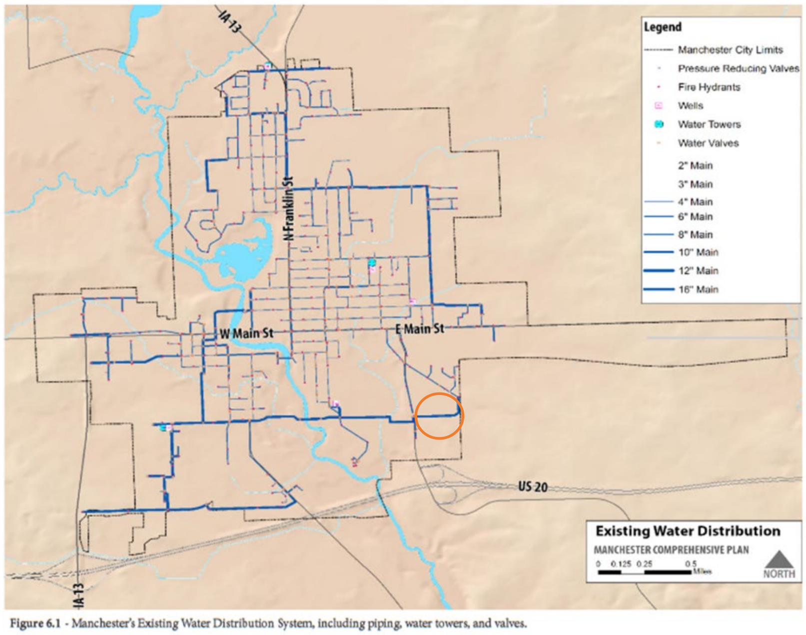


Figure 19: The Orange circle in Figure 19 shows the location of where the existing water main network and the designed network connect.

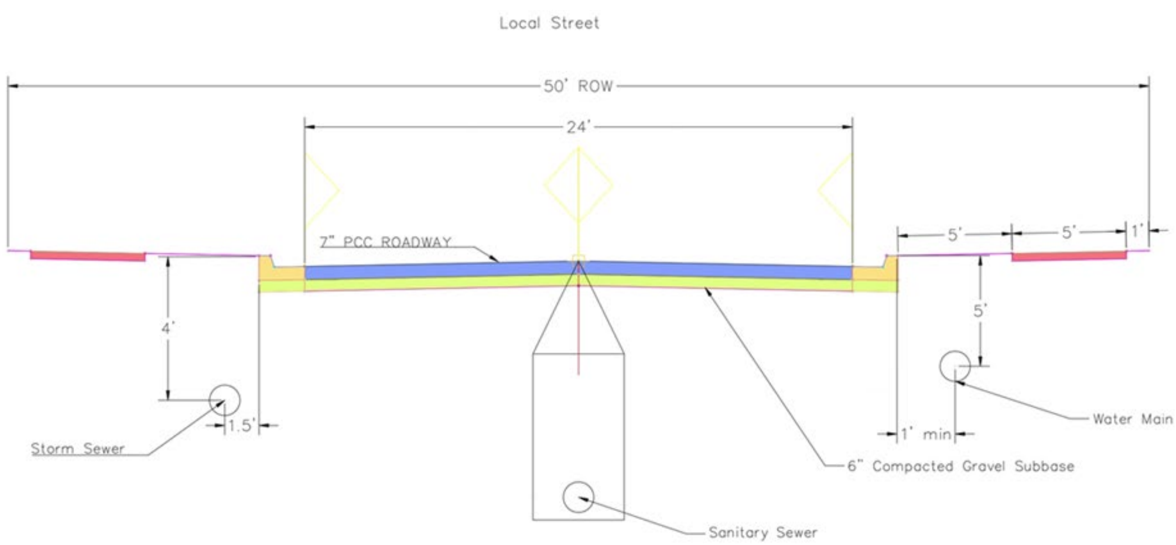


Figure 20: The Cross section of the Local Roadway including locations of utilities

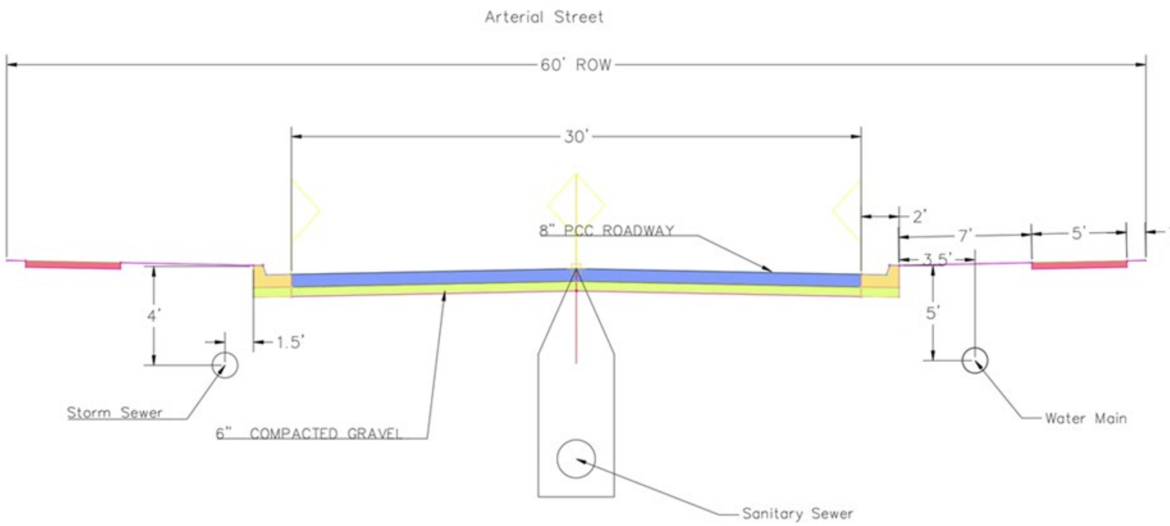


Figure 21: The Cross section of the Arterial Roadway including locations of utilities

Bike Path Design

The bike path was designed to go around the residential neighborhood and connect to the existing path that runs along Bailey Drive, however, due to money constraints the bike path was shortened to only go along the arterial road way and around the north storm water basin as well as through the bioswale area. The bike path was determined to be a Type 1 as well as a Type 3 paths as described in SUDAS Chapter 12B-2. This determines the slopes and widths. The bike path that runs along the arterial street and local street was designed to be a Type 1 path as it is directly in the right of way of the street. The rest of the path was determined to be a type 3 path because it is independent of the right of way and its primary use is for recreation and fitness.

The widths of the path were designed to be 5 ft wide on each side for a total width of 10 ft. This conforms both to the clients wishes as well as SUDAS standards. A 2 ft clearance zone was added to each side of the path for a total length of 14 ft. The material of the path is Portland Concrete Cement and the thickness as recommended by SUDAS is 5 inches. The Type 1 path uses a 2% cross slope that directs the water toward the street and the Type 3 path has a 1.5% slope. A minimum design speed of the path was designed to be 18 mph, although our team does not expect that to be reached. From the design speed, a minimum radius of 60 was used per Table 12B-2.01 in SUDAS.

Grading Design

The grading design was developed hand in hand with the stormwater management to make sure water was going to the places it was intended to do. The grading design took care to note the high points and low points in the existing lot and used those to grade appropriate new high and low points without having an excessive amount of cut or fill. Keeping the high and low points to natural was also necessary to be able to tie back in at existing grade on the surrounding borders of our site and especially into the ditch next to Bailey Drive.

In order to keep the grade at a slope that can still be mowed all the slopes were kept under a 4:1. All slopes of spillways were kept under 4% while still allowing the water to reach the correct points. The front yards of the lots were kept to 2% so water can drain but keep mowing easy and safe. Keeping all these constraints in mind to create as little earthwork as possible.

Storm Sewer Design

The storm sewer design consists of three different sections based on the water flow on the streets seen in Figure 20 below. The first section takes the flow from the high point of the arterial street and transfers it to the ditch along Bailey Drive (shown in red). The second section transfers the flow from the hammer section and the high point in the middle of the development to retention basin A in the Northeast section of the development (shown in yellow). Lastly, the third section takes the flow from the high point in the middle of the development and takes it to catchment basin C in the southwest corner of the development (seen in blue). The intakes along the streets are 2 ft by 4 ft grated intakes with an eccentric structure with a rectangular frame. The eccentric structure style of the intake will make cleaning and maintenance of the storm sewer system easier by offsetting away from the street. The piping for the system consists of reinforced concrete piping with 54 and 60 inch diameters.

Figure 22: Flow routes for the storm sewer system

The peak gutter flow rate was used to determine the size of the intakes, as well as the sizing if the pipes moving along the system. The gutter peak flow was calculated based on SUDAS for a gutter section with a uniform cross slope (Figure 21). These flow rates were based off both the longitudinal and horizontal slope of the streets and the max spread requirement for both arterial and local streets. Once the gutter flow was determined the interception capacity of the intake was designed. Comparing the gutter flow to the interception capacity determined the bypass flow which determined how the flow downstream was affected. As more flow is intercepted the size of the concrete piping increased until the final outlet. All equations and calculations used in the storm sewer design can be seen in Appendix G.

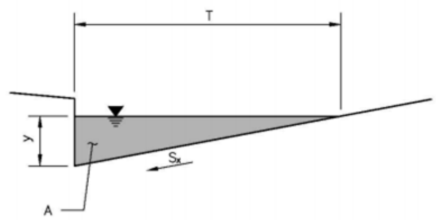


Figure 7-1. Gutter section with uniform cross slope

Figure 23: Gutter section layout from the Statewide Urban Design and Specifications manual

Lot Design

Per the client's instructions, the desired housing for the development were ranch style homes with enough space for two or three stall garages. Along the southern edge of the development closest to Highway 20, would be rows of town houses and multi-family housing. The large, southeast lot would be reserved for commercial development that would bring much needed resources to the area. The ranch style housing lots consisted of a minimum of 100' wide by 110' deep lots for all none-corner lots and 120' wide by 115' deep for corner lots. The southern town house lots were divided into three sections of roughly 0.63-acre areas. The commercial lot was estimated to be 3.6 acres. The total number of lots designed were 52 lots with 37 lots created for the current development and 15 additional lots for the future development of the northern access road and the hammer head road.

Section VII: Engineer's Cost Estimate

The cost of the new residential development in Manchester was determined from two sources. The first source was the SUDAS specification manual. This manual outlined all the specific bid items that could be used during the development of this project. The other resource was the Iowa DOT bid price lists. This was used to determine the prices of each of the specific items. The total estimated construction cost of the subdivision is $2,953,913. Based on our lot layout, that came out to an estimated $82,054 per lot based on the 32 initial lots. This total does not include the hammer head side streets, or the other future potential lots. The cost of the hammer head area came out to $192,960.00. The reason that the hammer head streets were not added into the final cost is because our team wanted to allow the clients to see the overall cost of the rest of the project so that they could get a sense of whether or not they wish to continue with the addition.

Our initial cost per lot did not include the future potential lots as to show that it is important to bring the price down. When the additional twenty-three lots were factored in, the price per lot goes down to 47,644 per lot. Assuming that the commercial land is sold for $200,000 the price per lot went down to $44,418. This information helps determine how much the client wants to buy the land and develop it for the future. Appendix H shows the cost of all the bid items in this development.

The unit costs of the lots was high due to a couple of reasons. The first is that the city can eat the cost in order to jump start the process of building the mixed-use neighborhood. Another option is that the city of Manchester could upfront the costs and as time goes on they would be able to reoperate the money. This can be done by the strategy of using tap on fees. As the area gets developed the home owners would have to pay a fee to be able to use the water, gas, electricity etc.. This also the city to get paid back over time with interest. Another way is to assess the interviewing properties and to put the homeowners on a payment schedule to pay back the city. This option is not the most popular with the citizens that live in the area. This is due to the people that live in the area may not want certain aspects of the neighborhood, but are having to pay for them anyways. As the development expands to the east the cost of development will get lower.

Section VIII: Phasing Plan

This phasing plan shows the most efficient sequencing of construction activities for the implementation of construction. This can help reduce the amount and duration of soil exposed to erosion. Another advantage of a phasing plan is that it shows the feasibility for certain parts of the neighborhood to be developed faster, which allows the clients to spread out the costs of the project and recoup some of those fees as people start to buy the developed lots. For this specific project it was broken down into three phases, however the sequence of construction can be done in any order. The client can start with the commercial lot if so inclined, the purpose of breaking down the project is to break the costs down.

Our design team has come up with a three part phasing plan. The first part of the plan is to do the North Section of the neighborhood and to stop at the high points along the north and south streets. This is a good point to stop because this where the North Sanitary system and the Storm system stops. The water system can be temporarily plugged until further expansion is done. This first phase will allow for approximately 26 lots. The next phase in the plan is to develop the Southwest commercial area. It makes the most sense to do this next, because selling this plot of land to a company will help offset the cost of the project. The last step in the phasing plan is the South Section of the development, which completes the 24-acre neighborhood development as is designed to include single family and medium density homes. Each phase of the project will require grading, water, and sanitary sewer development as well as storm water management including storm sewer. The first phase of the development will cost $1,284,586 and the second and third phases combine to cost $1,699,327.

Figure 24: Mixed-use subdivisions phasing plan.

Section IX: Appendices

Appendix A-Stormwater Management

Section 1: Preexisting maps of the site

Figure 25. Flood map pulled from Beacon

Figure 26. Wetland map pulled from the Iowa DNR's geodata database.

Figure 27. Contour map as reported from the Iowa DNR's geodata database

Section 2: Stormwater management Best Practice Selection

Table 1. Watershed area restrictions for best management practices from ISWMM Chapter 4

BMP	Watershed Area Served (Acres)								
	0	5	10	15	20	25	30	35	50
Infiltration Trench		★							
Infiltration Basin	★				★				
Rain Garden	★								
Bioretention Area		★							
Pervious Pavements		★							
Sand Filters	★	★ ★							
Dry Swale (enhanced w/media) Wet Swales (Wetland Channel)		★							
Vegetated Filter Strip		★							
Wet Pond			★	★					
Wet Extended Detention Basin		★	★	★					
Dry Extended Detention Basin				★	★				
Stormwater Wetlands					★				

Legend		★	
	Feasible	Marginal	Not Feasible

Table 2. Soil permeability restrictions for best management practices from ISWMM Chapter 4

BMP	Soil Type										
Infiltration trench	■	■	■	★							
Infiltration Basin	■	■	■	★							
Rain Garden	■	■	■	★							
Bioretention Area	■	■	■	■	★	★					
Pervious Pavement	■	■	■	★							
Dry Swale (enhanced w/media); Wet Swales	■	■	■	■	■	■	★	★	★		
Filter Strips	■	■	■	■	■	■	★	★	★		
Wet Pond		★	★	■	■	■	■	■	■	■	
Extended Detention Basins		★	★	■	■	■	■	■	■	■	
Stormwater Wetlands		★	★	■	■	■	■	■	■	■	
	Sand (8.3)	**Loamy Sand (2.4)**	**Sandy Loam (1.02)**	**Loam (0.52)**	**Silt Loam (0.27)**	**Sandy Clay Loam (0.17)**	**Clay Loam (0.09)**	**Silty Clay Loam (0.06)**	**Sandy Clay (0.05)**	**Silty Clay (0.04)**	**Clay (0.02)**
SOIL TYPE (MINIMUM INFILTRATION RATE, INCHES/HOUR)											

Source: Adapted from Schueler, 1987

Legend	■	★	
	Feasible	**Marginal**	**Not Feasible**

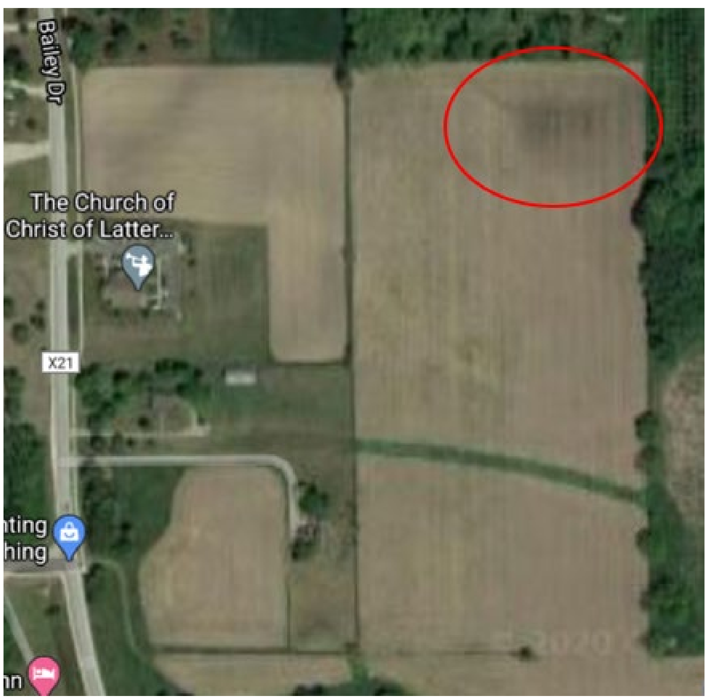


Figure 28. Arial footage of site showing potential water damage

Section 3: Storage Volume

Table 3. Size limitations of different hydraulic methods from ISWMM Chapter 3

Method	Size Limitations	Comments
Rational	≤160 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems. *Should not be used for storage design*.
NRCS	0-2000 acres	Method can be used for estimating peak flows and hydrographs for all design applications. Can be used for low-impact development hydrologic analysis.
USGS regression		Method can be used for estimating peak flows for all design applications.
Water quality		Methods used for calculating the water quality volume (WQv): (1) Simplified method, (2) NRCS CN method, (3) water quality capture volume method.

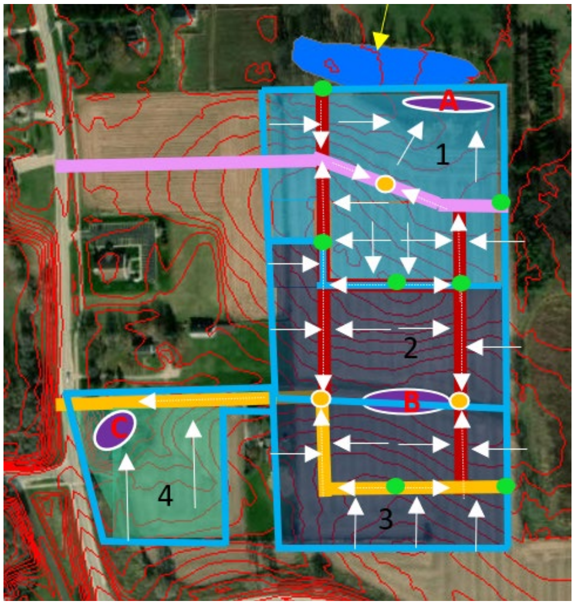


Figure 29. Onsite catchment regions and direction of grading

Table 4. On site catchment areas

Catchment	Area (acres)
1	6.99
2	6.85
3	5.95
4	4.39

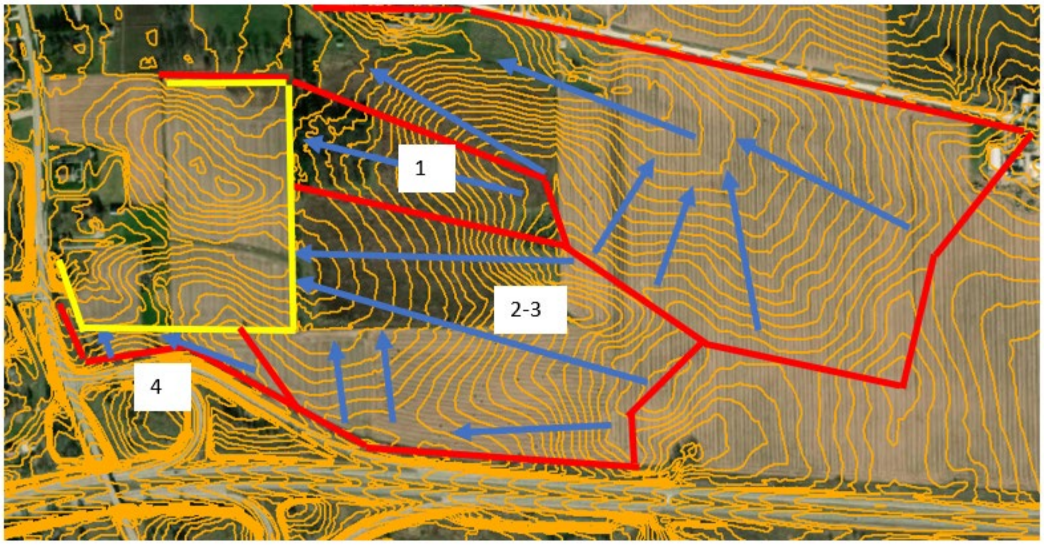


Figure 30. Offsite drainage map as it relates to the onsite catchments

Table 5. Offsite catchment areas

Catchment	Area (acres)
1	11.25
2+3	50.73
4(Private House)	1.14
4	3.34

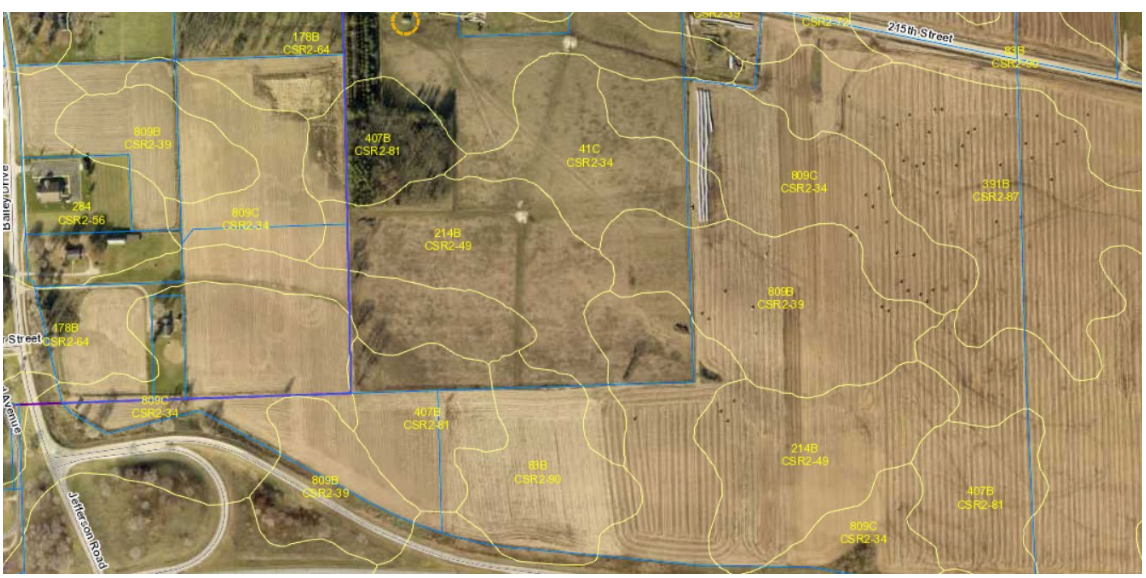


Figure 31. Soil map of site from Beacon's GIS soil survey

Table 6. Runoff coefficient values for the Rational method from Chapter 3 of the ISWMM

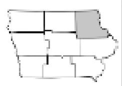
Hydrologic Soil Group	A			B			C			D		
Recurrence Interval	**5**	**10**	**100**	**5**	**10**	**100**	**5**	**10**	**100**	**5**	**10**	**100**
Land Use Or Surface Characteristics Business:												
A. Commercial Area	.75	.80	.95	.80	.85	.95	.80	.85	.95	.85	.90	.95
B. Neighborhood Area	.50	.55	.65	.55	.60	.70	.60	.65	.75	.65	.70	.80
Residential:												
A. Single Family	.25	.25	.30	.30	.35	.40	.40	.45	.50	.45	.50	.55
B. Multi-Unit (Detached)	.35	.40	.45	.40	.45	.50	.45	.50	.55	.50	.55	.65
C. Multi-Unit (Attached)	.45	.50	.55	.50	.55	.65	.55	.60	.70	.60	.65	.75
D. ½ Lot Or Larger	.20	.20	.25	.25	.25	.30	.35	.40	.45	.40	.45	.50
E. Apartments	.50	.55	.60	.55	.60	.70	.60	.65	.75	.65	.70	.80
Industrial												
A. Light Areas	.55	.60	.70	.60	.65	.75	.65	.70	.80	.70	.75	.90
B. Heavy Areas	.75	.80	.95	.80	.85	.95	.80	.85	.95	.80	.85	.95
Parks, Cemeteries Playgrounds	.10	.10	.15	.20	.20	.25	.30	.35	.40	.35	.40	.45
Schools	.30	.35	.40	.40	.45	.50	.45	.50	.55	.50	.55	.65
Railroad Yard Areas	.20	.20	.25	.30	.35	.40	.40	.45	.45	.45	.50	.55
Streets												
A. Paved	.85	.90	.95	.85	.90	.95	.85	.90	.95	.85	.90	.95
B. Gravel	.25	.25	.30	.35	.40	.45	.40	.45	.50	.40	.45	.50
Drives, Walks, & Roofs	.85	.90	.95	.85	.90	.95	.85	.90	.95	.85	.90	.95
Lawns												
A. 50%-75% Grass (Fair Condition)	.10	.10	.15	.20	.20	.25	.30	.35	.40	.30	.35	.40
B. 75% Or More Grass (Good Condition)	.05	.05	.10	.15	.15	.20	.25	.25	.30	.30	.35	.40
Undeveloped Surface[1] (By Slope)[2]												
A. Flat (0-1%)	0.04-0.09			0.07-0.12			0.11-0.16			0.15-0.20		
B. Average (2-6%)	0.09-0.14			0.12-0.17			0.16-0.21			0.20-0.25		
C. Steep	0.13-0.18			0.18-0.24			0.23-0.31			0.28-0.38		

[1]Undeveloped Surface Definition: Forest and agricultural land, open space.
[2]Source: Storm Drainage Design Manual, Erie and Niagara Counties Regional Planning Board.

Table 7. Frequency factors for Rational formula from Chapter 3 of the ISWMM

Recurrence Interval (years)	C_f
25	1.1
50	1.2
100	1.25

Table 8. Northeast Iowa rainfall depth and intensity for various return periods from Chapter 2 of the ISWMM

	Return Period															
	1 year		*2 year*		*5 year*		*10 year*		*25 year*		*50 year*		*100 year*		*500 year*	
Duration	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***
5 min	0.38	4.66	0.45	5.47	0.56	6.76	0.65	7.86	0.78	9.42	0.88	10.5	0.98	11.8	1.22	14.7
10 min	0.56	3.40	0.66	4.00	0.82	4.94	0.96	5.76	1.14	6.89	1.29	7.75	1.44	8.64	1.79	10.7
15 min	0.69	2.77	0.81	3.24	1.00	4.02	1.17	4.68	1.40	5.60	1.57	6.31	1.75	7.03	2.19	8.77
30 min	0.96	1.93	1.14	2.28	1.41	2.83	1.65	3.31	1.98	3.96	2.23	4.47	2.49	4.98	3.10	6.20
1 hr	1.25	1.25	1.47	1.47	1.85	1.85	2.17	2.17	2.64	2.64	3.01	3.01	3.39	3.39	4.34	4.34
2 hr	1.53	0.76	1.81	0.90	2.28	1.14	2.70	1.35	3.30	1.65	3.79	1.89	4.30	2.15	5.58	2.79
3 hr	1.71	0.57	2.01	0.67	2.55	0.85	3.03	1.01	3.74	1.24	4.32	1.44	4.94	1.64	6.55	2.18
6 hr	2.01	0.33	2.36	0.39	2.98	0.49	3.56	0.59	4.43	0.73	5.17	0.86	5.97	0.99	8.07	1.34
12 hr	2.32	0.19	2.69	0.22	3.38	0.28	4.02	0.33	5.02	0.41	5.86	0.48	6.79	0.56	9.25	0.77
24 hr	2.63	0.10	3.04	0.12	3.78	0.15	4.48	0.18	5.56	0.23	6.48	0.27	7.48	0.31	10.1	0.42
48 hr	3.00	0.06	3.44	0.07	4.23	0.08	4.98	0.10	6.12	0.12	7.10	0.14	8.15	0.16	10.9	0.22
3 day	3.28	0.04	3.73	0.05	4.56	0.06	5.32	0.07	6.49	0.09	7.48	0.10	8.56	0.11	11.4	0.15
4 day	3.53	0.03	4.00	0.04	4.85	0.05	5.64	0.05	6.84	0.07	7.86	0.08	8.95	0.09	11.8	0.12
7 day	4.17	0.02	4.72	0.02	5.70	0.03	6.58	0.03	7.87	0.04	8.95	0.05	10.1	0.06	13.0	0.07
10 day	4.76	0.01	5.38	0.02	6.45	0.02	7.39	0.03	8.77	0.03	9.90	0.04	11.0	0.04	14.0	0.05

D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

Table 9. Rational Method predevelopment flow rates for a 24-hour 100-year storm

	Catchment	Soil Type	C	Weighted C	Corrected C	Q (cfs)
Onsite	1	B	0.09	0.15	0.18	0.40
		C	0.23			
	2	B	0.24	0.16	0.20	0.43
		C	0.11			
	3	C	0.19	0.50	0.24	1.15
	4	B	0.19	0.95	0.20	0.27
		C	0.15			
Offsite	1	B	0.09	0.11	0.14	1.90
		C	0.15			
	2+3	B	0.09	0.11	0.13	8.23
		C	0.18			
	4(Private House)	B	0.4	0.50	0.62	0.22
		C	0.5			
	4	B	0.09	0.15	0.18	0.59
		C	0.19			

Table 10. Rational Method predevelopment flow rates for a 24-hour 100-year storm (by design region)

	Stormwater management region	Q (cfs)
Onsite flow	A	0.40
	B	1.58
	C	0.49
Offsite flow	A	1.90
	B	8.23
	C	0.81

Table 11. Rational Method post development flow rates for a 24-hour 100-year storm

	Catchment	Soil Type	C	Weighted C	Corrected C	Q (cfs)
Onsite	1	B	0.4	0.44	0.55	1.19
		C	0.5			
	2	B	0.4	0.46	0.58	1.22
		C	0.5			
	3	C	0.5	0.50	0.63	1.15
	4	B	0.95	0.95	1.19	1.61
		C	0.95			
Offsite	1	B	0.4	0.44	0.55	1.90
		C	0.5			
	2+3	B	0.4	0.42	0.52	8.23
		C	0.5			
	4(Private House)	B	0.4	0.50	0.62	0.22
		C	0.5			
	4	B	0.4	0.46	0.57	0.59
		C	0.5			

Table 12. Rational Method post development flow rates for a 24-hour 100-year storm (by design region)

	Stormwater management region	Q (cfs)
Onsite flow	A	1.19
	B	2.38
	C	1.61
Offsite flow	A	1.90
	B	8.23
	C	0.81

Table 13. NRCS runoff curve numbers for selected urban land use from ISWMM Chapter 3

Cover description		*Curve numbers for hydrologic soil group*			
Cover type and hydrologic condition	**Average impervious area**[2]	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc)[3]:					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc (excluding ROW)		98	98	98	98
Streets and roads:					
Paved: curbs and storm sewers (excluding ROW)		98	98	98	98
Paved: open ditches (including ROW)		83	89	92	93
Gravel (including ROW)		76	85	89	91
Dirt (including ROW)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1-2 inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
⅛ acre or less (town houses)	65	77	85	90	92
¼ acre	38	61	75	83	87
⅓ acre	30	57	72	81	86
½ acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]	77	86	91	94	
Idle lands (CN's are determined using cover types similar to those in Table C3-S5-3					

[1]Average runoff condition and I_a=0.2S.

[2]The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions pay be

Table 14. NRCS runoff curve numbers for other agricultural land use

Cover description		*Curve numbers for hydrologic soil group*			
Cover type	**Hydrologic condition**	**A**	**B**	**C**	**D**
Pasture, grassland, or range – continuous forage for grazing[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow – continuous grass, protected from grazing and generally mowed for hay	--	30	58	71	78
Brush – brush-weed-grass mixture with brush the major element[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[4]	48	65	73
Woods – grass combination (orchard or tree farm)[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	304	55	70	77
Farmsteads – buildings, lanes, driveways, and surrounding lots	--	59	74	82	86

[1]Average runoff condition and I_a=0.2S.
[2]Poor: <50% ground cover or heavily grazed with no mulch.
Fair: 50% to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.
[3]Poor: <50% ground cover.

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

Figure 32. Equation for potential maximum retention (inches)

Table 15. Sectional mean rainfall amount and return period (recurrence interval) in Iowa from ISWMM Chapter 3

	Duration	Return Period 1-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
Section 1 – Northwest Iowa	10-day	4.46	5.08	6.12	7.02	8.32	9.36	10.4	13.1
	7-day	3.93	4.49	5.46	6.32	7.60	8.64	9.74	12.5
	4-day	3.38	3.85	4.70	5.49	6.71	7.74	8.85	11.8
	3-day	3.16	3.60	4.41	5.17	6.36	7.38	8.50	11.5
	48-hr	2.89	3.30	4.08	4.82	5.98	6.99	8.10	11.1
	24-hr	2.51	2.92	3.67	4.39	5.50	6.46	7.50	10.3
	12-hr	2.21	2.59	3.30	3.95	4.95	5.81	6.74	9.21
	6-hr	1.95	2.30	2.91	3.47	4.32	5.04	5.81	7.84
	3-hr	1.69	1.99	2.51	2.97	3.66	4.22	4.81	6.33
	2-hr	1.53	1.80	2.27	2.68	3.26	3.74	4.23	5.45
	1-hr	1.25	1.48	1.86	2.18	2.64	3.01	3.38	4.30
	30-min	0.97	1.15	1.44	1.69	2.02	2.28	2.54	3.15
	15-min	0.69	0.82	1.03	1.20	1.44	1.62	1.81	2.24
	10-min	0.57	0.67	0.84	0.98	1.18	1.33	1.48	1.84
	5-min	0.39	0.46	0.57	0.67	0.80	0.91	1.01	1.25
Section 2 – North Central Iowa	10-day	4.78	5.45	6.58	7.56	8.99	10.1	11.3	14.3
	7-day	4.19	4.79	5.83	6.76	8.12	9.24	10.1	13.4
	4-day	3.55	4.06	4.97	5.80	7.06	8.12	9.26	12.2
	3-day	3.31	3.78	4.63	5.42	6.64	7.68	8.80	11.8
	48-hr	3.04	3.46	4.26	5.01	6.18	7.19	8.29	11.2
	24-hr	2.65	3.06	3.83	4.55	5.67	6.63	7.68	10.4
	12-hr	2.34	2.74	3.46	4.14	5.18	6.07	7.03	9.59
	6-hr	2.06	2.42	3.07	3.6	4.60	5.38	6.22	8.45
	3-hr	1.76	2.08	2.64	3.15	3.91	4.56	5.24	7.04
	2-hr	1.58	1.87	2.37	2.82	3.49	4.04	4.63	6.14
	1-hr	1.28	1.52	1.92	2.27	2.80	3.23	3.69	4.85
	30-min	0.99	1.16	1.47	1.73	2.11	2.42	2.75	3.56
	15-min	0.69	0.82	1.03	1.21	1.48	1.69	1.92	2.48
	10-min	0.57	0.67	0.84	0.99	1.21	1.39	1.57	2.03
	5-min	0.39	0.46	0.57	0.68	0.83	0.95	1.07	1.39
Section 3 – Northeast Iowa	10-day	4.76	5.38	6.45	7.39	8.77	9.90	11.0	14.0
	7-day	4.17	4.72	5.70	6.58	7.87	8.95	10.1	13.0
	4-day	3.53	4.00	4.85	5.64	6.84	7.86	8.95	11.8
	3-day	3.28	3.73	4.56	5.32	6.49	7.48	8.56	11.4
	48-hr	3.00	3.44	4.23	4.98	6.12	7.10	8.15	10.9
	24-hr	2.63	3.04	3.78	4.48	5.56	6.48	7.48	10.1
	12-hr	2.32	2.69	3.38	4.02	5.02	5.86	6.79	9.25
	6-hr	2.01	2.36	2.98	3.56	4.43	5.17	5.97	8.07
	3-hr	1.71	2.01	2.55	3.03	3.74	4.32	4.94	6.55
	2-hr	1.53	1.81	2.28	2.70	3.30	3.79	4.30	5.58
	1-hr	1.25	1.47	1.85	2.17	2.64	3.01	3.39	4.34

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$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

Figure 33. Equation for runoff

$$I_a = 0.2S$$

Figure 34. Equation for initial abstraction

Table 16. Coefficients for SCS peak discharge method from ISWM Chapter 2

I_a/P	C_0	C_1	C_2
0.10	2.55323	-0.61512	-0.16403
0.30	2.46532	-0.62257	-0.11657
0.35	2.41896	-0.61594	-0.08820
0.40	2.36409	-0.59857	-0.05621
0.45	2.29238	-0.57005	-0.02281
0.50	2.20282	-0.51599	-0.01259

$$L = \frac{l^{0.8}(S+1)^{0.7}}{1900Y^{0.5}}$$

Figure 35. Equation for lag

$$T_c = \frac{L}{0.6}$$

Figure 36. Equation for time of concentration

$$q_u = 10^{[C_0+(C_1)(\log t_c)+(C_2)(\log t_c)^2]}$$

Figure 37. Equation for unit peak discharge

Table 17. Adjustment Factor for pond and swamp areas that are spread throughout the watershed from ISWMM Chapter 2

Percentage of pond and swamp area	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

$$q_p = q_u A_m Q F_p$$

Figure 38. Equation for peak discharge

Table 18. NRCS peak flow rates for a predevelopment 24-hour 5-year storm

	Catchment	Soil	Area (acres)	Cn	Weighted Cn	S (in)	Q (in)	Ia/P	C0	C1	C2	Tc (hr)	qu (ft^3/s/mi^2/in)	qp (cfs)
Onsite	1	B	4.14	58	63	5.80	0.82	0.3	2.51	-0.62	-0.14	2.2	190.93	1.70
		C	2.85	71										
	2	B	2.65	58	66	5.16	0.96	0.3	2.55	-0.62	-0.16	3.1	161.80	1.66
		C	4.21	71										
	3	C	5.95	71	71	4.08	1.25	0.2	2.55	-0.62	-0.16	1.8	243.28	2.82
	4	B	1.09	58	68	4.76	1.05	0.3	2.55	-0.61	-0.16	1.6	261.00	1.89
		C	3.29	71										
Offsite	1	B	7.15	54	60.20	6.61	0.67	0.35	2.51	-0.62	-0.14	4.95	102.76	1.20
		C	4.11	71										
	2+3	B	41.38	58	60.40	6.56	0.68	0.35	2.51	-0.62	-0.14	10.33	54.61	2.92
		C	9.35	71										
	4(Private House)	B	0.03	68	78.72	2.70	1.77	0.14	2.51	-0.62	-0.16	0.57	445.76	1.41
		C	1.12	79										
	4	B	1.43	58	65.42	5.29	0.93	0.28	2.51	-0.62	-0.16	4.56	107.78	0.52
		C	1.91	71										

Table 19. NRCS peak flow rates for a predevelopment 24-hour 5-year storm (by design region)

	Stormwater management region	qp (cfs)
Onsite	A	1.70
	B	4.47
	C	1.89
Offsite	A	1.20
	B	2.92
	C	1.93

Table 20. NRCS peak flow rates for a post development 24-hour 100-year storm

	Catchment	Soil	Area (acres)	Cn	Weighted Cn	S (in)	Q (in)	Ia/P	C0	C1	C2	Tc (hr)	qu (ft^3/s/mi^2/in)	qp (cfs)
Onsite	1	B	4.14	72	76	3.22	4.65	0.1	2.55	-0.62	0.16	1.9	245.80	12.48
		C	2.85	81										
	2	B	2.65	72	78	2.90	4.86	0.1	2.55	-0.62	-0.16	2.3	205.94	10.71
		C	4.21	81										
	3	C	5.95	81	81	2.35	5.25	0.1	2.55	-0.62	-0.16	1.3	296.71	14.49
	4	B	1.09	92	94	0.70	6.71	0.02	2.59	-0.61	-0.18	1.0	394.00	18.10
		C	3.29	94										
Offsite	1	B	7.15	72	75	3.28	4.61	0.09	2.56	-0.62	-0.17	4.95	112.27	9.09
		C	4.11	81										
	2+3	B	41.38	72	74	3.58	4.43	0.10	2.55	-0.62	-0.16	7.27	79.68	27.95
		C	9.35	81										
	4(Private House)	B	0.03	68	79	2.70	4.99	0.07	2.57	-0.61	-0.17	0.57	511.84	4.57
		C	1.12	79										
	4	B	1.43	72	77	2.96	4.82	0.08	2.56	-0.61	-0.17	3.30	156.99	3.94
		C	1.91	81										

Table 21. NRCS peak flow rates for a post development 24-hour 100-year storm (by design region)

	Stormwater management region	qp (cfs)
Onsite	A	12.48
	B	25.20
	C	18.10
Offsite	A	9.09
	B	27.95
	C	8.51

Table 22. The runoff ratio between predevelopment 5-year storms and post development 100-year storms

Region	Q0/Qi
A	0.14
B	0.18
C	0.1

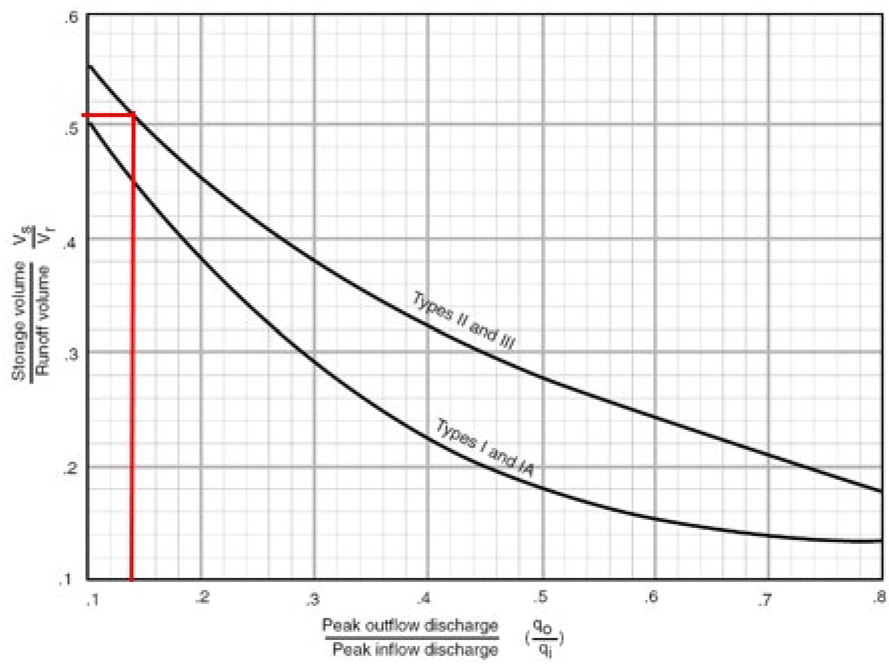


Figure 39. Approximate detention basin routing for Region A

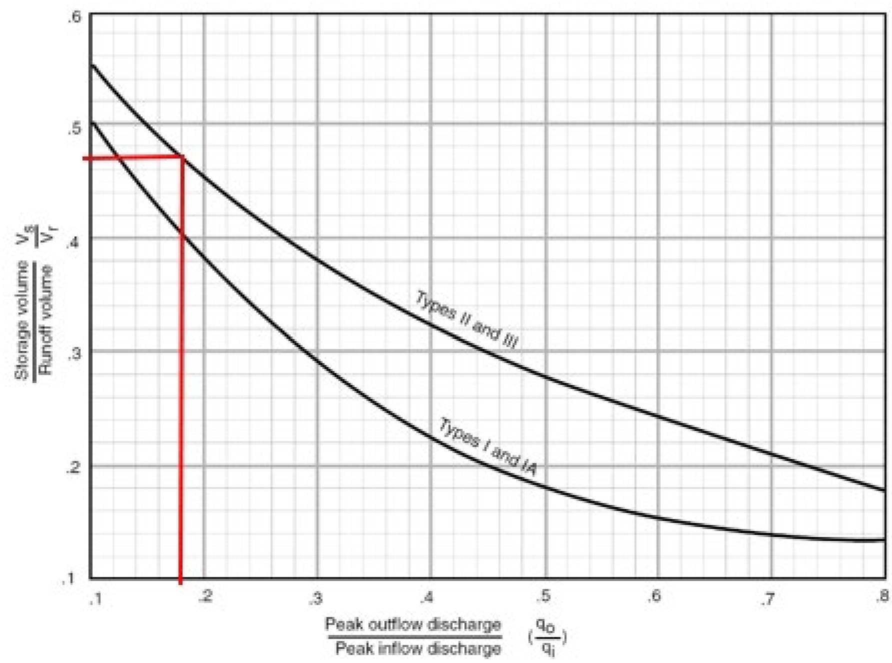


Figure 40. Approximate detention basin routing for Region B

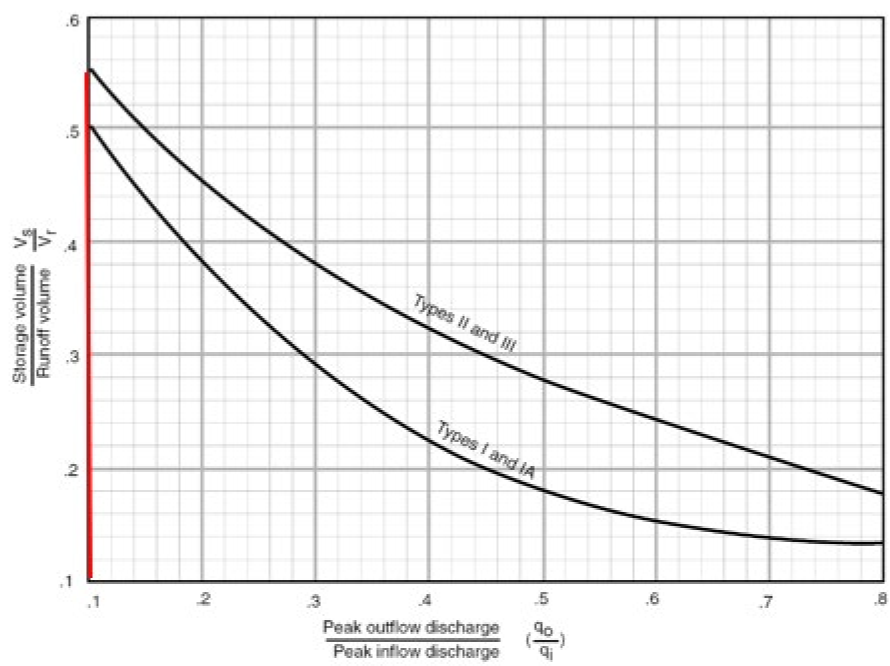


Figure 41. Approximate detention basin routing for Region C

Figure 42. Storage volumes to keep run off from post development 100-year storms at predevelopment 5-year storm levels

Region	Vs/Vr	Vr (CF)	Vs (CF)
A	0.51	102,816	52,436
B	0.47	105,408	49,542
C	0.55	139,104	76,507

Section 4: Water Quality Volume

Table 23. Total impervious coverage from the Journal of the American Planning Association, Vol. 62 No. 2.

Surface Coverage Type	Average Approximate Site Coverage, % High Density Residential (3–7 units/acre)	Multifamily (7–30 units/acre)	Commercial
1. Streets	16	11	03
2. Sidewalks	03	05	04
3. Parking/driveways	06	15	53
4. Roofs	15	17	26
5. Lawns/landscaping	54	19	13
6. Open space	n/a	34	n/a
Total impervious surface (1–4)	40	48	86
Road-related impervious surface (1–3)	25	31	60
(Road-related as a percentage of total impervious coverage)	(63%)	(65%)	(70%)

$$Rv = 0.05 + 0.009(I)$$

Figure 43. Equation for runoff volume

$$WQv = P\ A\ R_v$$

Figure 44. Equation for water quality volume

Table 24. Water quality volumes for predevelopment

Catchment	Area (acre)	P (inches)	I (%)	Rv	WQv (CF)
1	6.99	1.25	0.00	0.05	1585
2	6.85	1.25	0.00	0.05	1554
3	5.95	1.25	0.00	0.05	1350
4	4.39	1.25	0.00	0.05	995

Table 25. Water quality volumes for predevelopment (by region)

Region	WQv (CF)
A	1585
B	2904
C	995

Table 26. Water quality volumes for post development

Catchment	Area (acres)	P (inches)	I (%)	Rv	WQv (CF)
1	6.99	1.25	63	0.62	19561
2	6.85	1.25	63	0.62	19178
3	5.95	1.25	63	0.62	16658
4	4.39	1.25	70	0.68	13530

Table 27. Water quality volumes for post development (by region)

Region	WQv (CF)
A	19561
B	35835
C	13530

Section 5: Wet Detention Pond Design

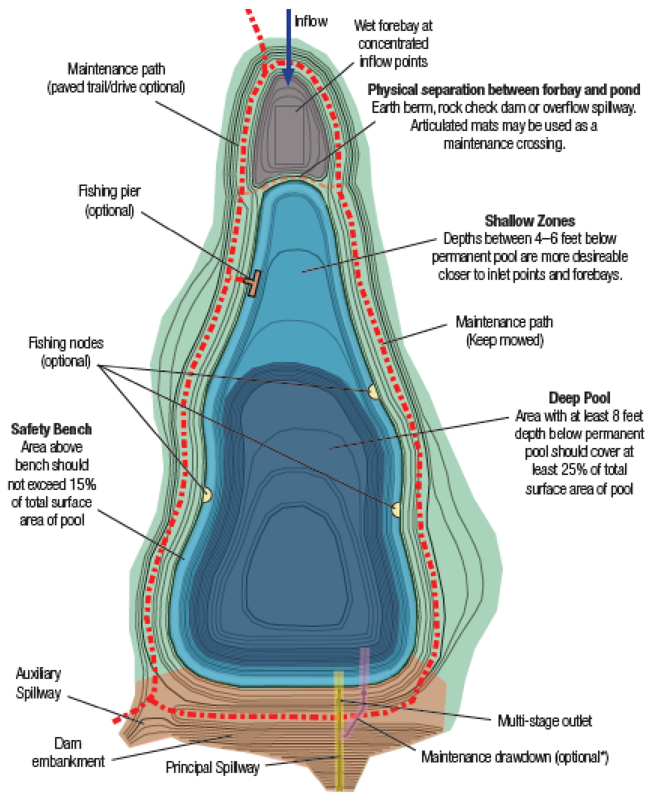


Figure 45. Typical features of a wet detention basin from ISWMM Chapter 9

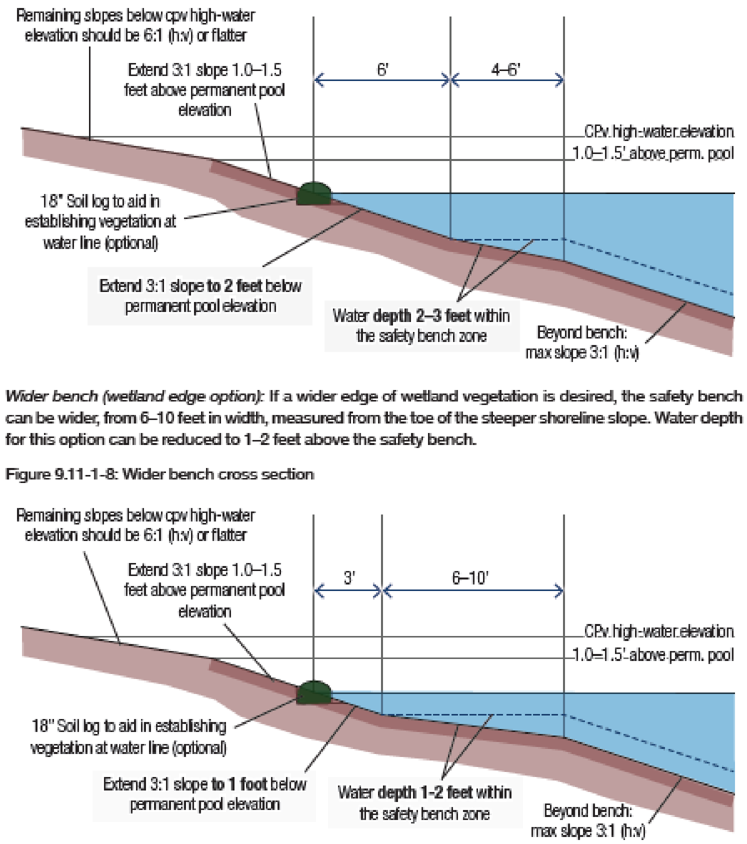


Figure 46. Safety bench detail from ISWMM Chapter 9

Table 28. Wet detention pool dimensions for basins A and C

	Requirements		Deep Pool (3:1)					Shallow Zone (3:1)				Shallow Zone 2 (6:1)				Shallow Zone	l dimensions		Final dimension		
Basin	Storage (CF)	2x WQv (CF)	Top Width (ft)	Bottom Width (ft)	Length (ft)	Depth (ft)	Vol (actual) (CF)	Top Width (ft)	Bottom Width (ft)	Length (ft)	Depth (ft)	Top Width (ft)	Bottom Width (ft)	Length (ft)	Depth (ft)	Vol (actual) (ft)	Width (ft)	Length (ft)	Pathway (ft)	Total Width (ft)	Total Length (ft)
A	52436	39122	53	23	106	10	40280	56	53	112	1	72	57	144	5	52544	72	144	12	84	156
C	126049	27060	46	16	92	10	28520	49	46	98	1	114	99	228	5	126065	114	228	12	126	240

Section 6: Overland Flow Routes

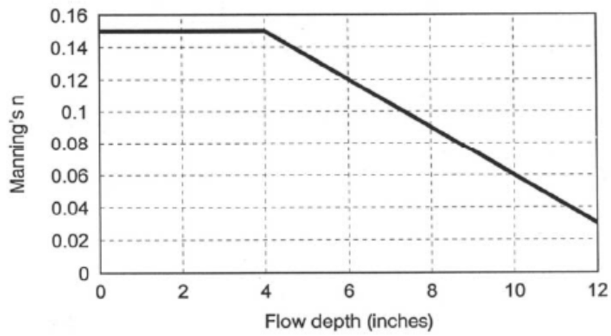


Figure 47. Variable Manning's n with flow depth from ISWMM Chapter 9

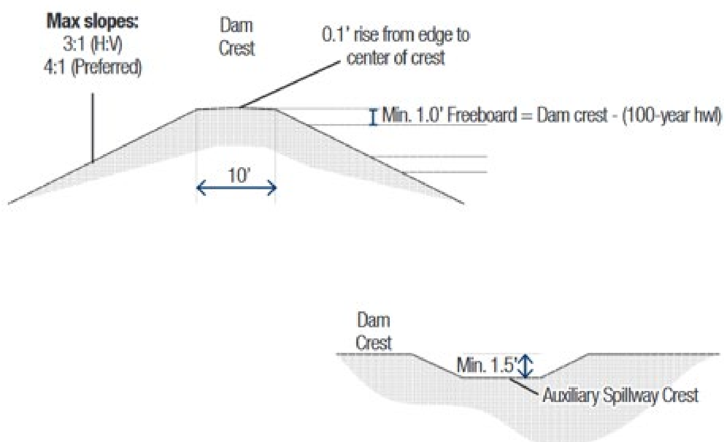


Figure 48. Spillway parameters from ISWMM Chapter 9

Cover	Slope Range (percent)	Permissible Velocity on…	
		Erosion Resistant Soils (fps)	*Easily Eroded Soils (fps)*
Bermudagrass	0 to 5	8	6
	5 to 10	7	5
	Over 10	6	4
Buffalograss Kentucky bluegrass Smooth brome Blue grama	0 to 5	7	5
	5 to 10	6	4
	Over 10	5	3
Grass mixture	0 to 5	5	4
	5 to 10	4	3
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	0 to 5	3.5	2.5
Common lespedeza[2] Sudangrass	0 to 5[3]	3.5	2.5

[1] Use velocities of 5 fps only where good covers and proper maintenance can be obtained.
[2] Annuals, used on mild slopes or as temporarily protection until permanent covers are established.
[3] Use on slopes steeper than 5% is not recommended.

Figure 49. Permissible velocities for channels from SUDAS Chapter 2

$$Q = AV = \frac{1.486}{n}\left(AR^{2/3}\right)\left(s^{1/2}\right)$$

Figure 50: Manning's equation

Table 29. Spillway design calculations

Spillway Design (4:1 Side Slope)										
Spillway	**Longitudinal Slope**	**Yn (ft)**	**Depth (ft)**	**Bottom Width (ft)**	**Top Width (ft)**	**A (CF)**	**P (ft)**	**R (ft)**	**Q (cfs)**	**V (ft/s)**
1	3.6%	2	3	4	16	20	20	1	40	2
2	0.5%	4	5	6	26	64	39	2	67	1
3	0.4%	2	3	7	19	26	23	1	19	1
4	0.5%	2	3	6	18	24	22	1	19	1
5	2.0%	2	3	4	16	20	20	1	30	1

Section 7: Bioswale Design

Table 30. Bioswale design calculations

Bioswale Design (4:1 Side Slope)										
	Longitudinal Slope	**Yn (ft)**	**Depth (ft)**	**Bottom Width (ft)**	**Top Width (ft)**	**A (CF)**	**P (ft)**	**R (ft)**	**Q (cfs)**	**V (ft/s)**
Bioswale	0.7%	3	4	5	21	39	30	1	40	1

Table 31. List of native plant species for bioswales from the Iowa Stormwater Education Partnership (1/3)

Recommended Plants for Iowa Rain Gardens, Biocells, and Bioswales		BMP Use			Sun Exposure			Soil Moisture				Location in BMP			Height				Blooming Period							Comments
Common Name	*Botanical Name*	Rain garden	Biocell	Bioswale	Full sun	Part shade	Shade	Wet	Moist	Mesic	Dry	Bottom	Sides	Top of Berm	6"-12"	12"-24"	24"-36"	>36"	April	May	June	July	August	September	October	
Native Forbs (Flowers)																										
Anemone, Canada	*Anemone canadensis*	x	x	x	x	x			x	x	x	x	x	x	x					x	x					Will spread
Aster, Lance Leaf	*Symphyotrichum lanceolatum*		x	x	x	x	x		x	x		x				x	x	x				x	x	x		
Aster, New England	*Symphyotrichum novae-angliae*		x	x	x	x				x		x				x	x						x	x	x	
Aster, Silky	*Symphyotrichum sericeum*	x	x	x	x	x				x	x	x	x	x	x	x							x	x		
Aster, Smooth Blue	*Symphyotrichum laeve*	x	x		x					x		x		x		x							x	x	x	
Aster, Upland White	*Solidago ptarmicoides*		x	x	x	x				x		x				x						x	x	x		
Beardtongue, Foxglove	*Penstemon digitalis*		x	x	x	x			x	x		x					x				x	x				
Beardtongue, Large Flowered	*Penstemon grandiflora*	x	x	x	x	x						x				x	x			x	x					
Bergamot, Wild (bee-balm)	*Dalea candida*		x	x	x	x				x	x	x					x	x			x	x	x			
Blazingstar, Prairie	*Liatris pycnostachya*	x	x	x	x					x	x	x		x		x						x	x	x		
Blazingstar, Rough (Button)	*Liatris aspera*	x	x	x	x					x	x	x				x						x	x	x		
Bluebells, Virginia	*Mertensia Virginica*	x	x			x	x		x	x		x				x			x	x						
Catchfly, Royal	*Silene Digia*	x	x	x	x	x				x	x						x	x				x	x			Can be a more sensitive plant
Clover, Purple Prairie	*Dalea purpurea*	x	x	x	x					x	x	x	x	x		x					x	x	x			
Clover, White Prairie	*Dalea candida*	x	x	x	x					x	x	x	x			x					x	x				
Columbine, American	*Aquilegia canadensis*	x	x	x	x	x	x		x	x		x	x	x		x				x	x					Will spread
Coneflower, Gray-headed Prairie	*Ratibida pinnata*		x	x	x	x			x	x		x					x				x	x	x	x		
Coneflower, Orange	*Rudbeckia fulgida*	x	x	x	x	x			x	x		x					x				x	x	x	x		
Coneflower, Pale Purple	*Echinacea, pallida*	x	x	x	x					x	x	x				x	x					x	x	x		
Coneflower, Purple	*Echinacea purpurea*	x	x	x	x	x			x	x		x	x	x			x	x				x	x	x		
Coreopsis	*Coreopsis palmata*	x	x	x	x	x			x	x		x				x	x				x	x	x			
Coreopsis, Lance Leaf	*Coreopsis lanceolata*	x	x	x	x				x	x		x	x	x		x					x	x	x			
Culvers Root	*Veronicastrum virginicum*	x	x	x	x	x			x	x		x					x	x				x	x			Tall but does not flop
False Sunflower (Ox-eye)	*Heliopsis helianthoides*		x	x	x	x			x	x	x	x						x				x	x	x		
Gentian, Bottle	*Gentiana andrewsii*	x	x	x	x	x			x	x		x	x	x		x							x	x		
Gentian, Cream	*Gentiana flavida*	x	x	x	x	x			x	x		x	x	x		x							x	x		
Geranium, Wild	*Geranium maculatum*	x	x	x		x	x		x	x		x	x	x		x				x	x					Will spread
Ginger, Wild	*Ascarum canadense*	x	x				x		x	x		x	x	x	x				x	x						Will spread; very unique flower
Golden Alexander	*Zizia aurea*		x	x	x	x			x	x		x					x	x			x	x				
Goldenrod, Showy	*Solidago speciosa*			x	x					x	x	x	x					x					x	x	x	
Goldenrod, Stiff	*Solidago rigida*			x	x					x	x	x	x					x					x	x	x	
Harebell	*Campanula Rotundifolia*	x	x	x		x	x			x	x	x	x	x	x						x	x	x	x		
Hyssop, Anise	*Agastache foeniculum*		x	x	x				x	x	x	x	x	x		x	x				x	x	x			Will spread
Indigo, White Wild	*Baptisia alba*	x	x	x	x	x			x	x	x	x						x			x	x				Bush-like, tall for a rain garden
Indigo, Wild Blue	*Baptisia australis*	x	x	x	x	x				x		x						x		x	x					Bush-like, tall for a rain garden
Iris, Blue Flag	*Iris virginica shrevei*	x	x	x	x			x	x				x	x			x			x	x					Prefers moist conditions
Ironweed	*Vernonia fasciculata*		x	x	x	x		x	x	x		x						x				x	x	x		
Jacobs Ladder	*Polemonium reptans*	x	x	x	x	x	x		x	x			x			x	x	x		x	x					Will spread
Leadplant	*Amorpha canescens*	x	x	x	x					x	x	x	x				x				x	x				
Lobelia, Great Blue	*Lobelia siphilitica*	x	x	x	x	x		x	x	x		x	x	x			x	x				x	x	x	x	Will spread
Loosestrife, Winged	*Lythrum alatum*				x			x	x	x		x	x			x	x	x			x	x	x	x		
Loosestrife, Prairie	*Lysimachia quadriflora*	x	x	x	x			x	x			x				x	x					x	x			Prefers moist conditions
Milkvetch, Canada	*Astragalus canadensis*		x	x	x	x			x	x		x				x	x	x			x	x	x			
Milkweed, Butterfly	*Asclepias tuberosa*	x	x	x	x	x				x	x	x		x							x	x	x	x		
Milkweed, Common	*Asclepias syriaca*		x	x	x	x		x	x	x		x						x			x	x	x			
Mint, Virginia Mountain	*Pycnanthemum virginianum*	x	x	x	x	x			x	x	x	x	x	x		x					x	x	x			
Monkey Flower	*Mimulus ringens*	x	x	x	x	x		x	x			x	x			x					x	x	x			
Obedient Plant	*Physpstegia virginiana*				x	x			x	x		x		x			x				x	x	x	x		
Onion, Nodding	*Allium cernuum*	x	x	x	x	x				x		x	x	x	x	x						x	x			
Onion, Wild	*Allium textile*	x	x		x					x	x	x	x	x	x					x	x					

Table 32. List of native plant species for bioswales from the Iowa Stormwater Education Partnership (2/3)

Recommended Plants for Iowa Rain Gardens, Biocells, and Bioswales		BMP Use			Sun Exposure			Soil Moisture				Location in BMP			Height				Blooming Period							Comments
Common Name	*Botanical Name*	Rain garden	Biocell	Bioswale	Full sun	Part shade	Shade	Wet	Moist	Mesic	Dry	Bottom	Sides	Top of Berm	6"-12"	12"-24"	24"-36"	>36"	April	May	June	July	August	September	October	
Petunia, Wild	*Ruellia humillis*	x	x	x	x					x	x		x	x		x	x				x	x	x			Spreads; grows well along the edge
Phlox, Prairie	*Phlox pilosa*	x	x	x	x	x			x	x		x	x	x		x			x	x	x					
Phlox, Woodland	*Phlox divaricata*	x	x	x		x	x		x	x		x	x	x	x	x			x	x						
Prairie Smoke	*Geum triflorum*	x	x		x					x	x	x	x	x	x				x	x						
Primrose, Common Evening	*Oenothera biennis*			x	x	x				x	x										x	x	x	x		
Rattlesnake Master	*Eryngium yuccifolium*			x	x				x	x	x	x				x	x	x				x	x			
Rue, Purple Meadow	*Thalictrum dasycarpum*		x	x	x	x			x	x		x				x	x	x			x	x				
Shooting Star	*Dodecatheon Pulchellum*	x	x		x	x		x	x	x		x		x		x				x	x	x				
Sneezeweed	*Helenium autumnale*		x	x	x			x	x			x				x	x	x					x	x	x	
Solomon's Seal	*Polygonatum multiflorum variegatum*	x				x		x	x	x		x	x	x		x			x	x						
Spiderwort, Ohio	*Tradescantia ohioensis*	x	x	x	x	x			x	x		x	x			x				x	x	x				Will spread; may bloom again in fall
Spiderwort, Prairie (Bracted)	*Tradescantia bracteata*	x	x	x	x				x	x		x	x			x				x	x	x				
Susan, Black-eyed	*Helenium autumnale*	x	x	x	x					x	x	x				x	x				x	x	x	x		Early sucessional
Susan, Brown-eyed	*Rudbeckia triloba*	x	x	x	x	x				x	x	x				x	x	x			x	x	x	x		Early successional
Sweet William, Wild	*Phlox Maculata*	x	x	x	x	x			x	x		x					x	x			x	x				
Trefoil, Showy Tick	*Desmodium canadense*		x	x	x					x	x	x	x			x	x	x				x	x	x		
Turtlehead	*Chelone glabra*	x	x	x	x			x	x	x		x				x	x	x					x	x		
Vervain, Blue	*Verbena hastata*	x	x	x	x	x		x	x	x		x		x		x						x	x	x		
Vervain, Hoary	*Verbena stricta*		x	x	x					x	x	x	x	x		x	x					x	x	x		
Native Grasses, Wildrye, Sedges and Ferns																										
Big Bluestem	*Andropagon gerardii*			x	x				x	x	x	x	x	x				x								> 48" tall; spreads
Blue Grama	*Bouteloua gracillis*	x	x	x	x				x	x	x	x	x		x											Can use for turfgrass
Fern, Cinnamon	*Osmundastrum cinnamomeum*	x	x			x	x	x	x			x	x			x	x	x								
Fern, Lady	*Athyrium Filix-femina*	x	x			x	x		x	x		x	x			x	x	x								
Fern, Maidenhair	*Adiantum*	x					x		x			x		x	x											
Fern, Ostrich	*Matteuccia struthiopteris*	x				x	x	x	x			x					x	x								
Grass, Bottlebrush	*Elymus Hystrix*	x	x			x	x			x		x						x								
Indian Grass	*Sorghastrum*			x	x	x			x	x		x						x								
Little Bluestem	*Schizachyrium scoparium*	x	x	x	x	x				x	x	x		x		x										
Prairie Dropseed	*Sporobolus heterolepis*	x	x	x	x					x	x	x	x	x		x	x									
Sedge, Bicknell	*Carex bicknellii*	x	x		x	x				x		x	x	x	x											
Sedge, Bur Reed	*Carex sparganioides*	x	x			x	x		x			x	x	x		x	x									Shade conditions
Sedge, Brown Fox	*Carex vulpinoidea*	x	x	x	x	x		x	x	x		x	x				x									
Sedge, Common Fox	*Carex stipata*	x	x	x	x	x	x	x	x			x	x				x									
Sedge, Common Wood	*Carex blanda*	x					x	x	x			x	x	x	x											Shade conditions
Sedge, Eastern Woodland	*Carex blanda*	x	x			x		x	x			x			x											
Sedge, Wooly	*Carex pellita*	x			x	x		x	x			x			x	x	x									Could potentially be used as turf
Sideoats Grama	*Bouteloua curtipendula*	x	x	x	x					x	x	x	x	x		x										
Switch Grass	*Panicum virgatum*	x	x	x	x	x				x	x	x	x	x				x								Some species can be too tall and are floppy
Wildrye, Canada	*Elymus canadensis*		x	x	x	x						x				x	x	x								
Horticultural Cultivar Flowers and Ferns																										
Astilbe, Chinese	*Astilbe chinensis 'Pumila'*	x	x			x	x		x	x		x		x	x	x						x	x			
Balloon Flower	*Platycodon*	x	x		x	x			x	x		x		x		x						x	x			
Beebalm Hybrids	*Monarda*	x	x		x	x			x	x		x		x		x					x	x	x			
Columbine Hybrids	*Aquilegia hybrids*	x	x	x					x	x						x										
Coneflower, Purple	*Echinacea purpurea*	x	x	x	x	x			x	x	x	x		x		x	x					x	x	x		

Table 33. List of native plant species for bioswales from the Iowa Stormwater Education Partnership (3/3)

Recommended Plants for Iowa Rain Gardens, Biocells, and Bioswales		BMP Use			Sun Exposure			Soil Moisture				Location in BMP			Height				Blooming Period							Comments
Common Name	Botanical Name	Rain garden	Biocell	Bioswale	Full sun	Part shade	Shade	Wet	Moist	Mesic	Dry	Bottom	Sides	Top of Berm	6"-12"	12"-24"	24"-36"	>36"	April	May	June	July	August	September	October	
Coralbells Hybrids	*Heuchera*	x	x		x	x	x						x	x								x	x	x		
Coreopsis	*Coreopsideae*	x	x		x	x				x		x		x		x						x	x	x		Deadheading will extend bloom period
Hosta Hybrids	*Hosta*	x	x				x		x			x		x	x	x	x	x				x	x			
Iris, Siberian	*Iris sibirica*	x	x		x	x			x	x		x		x		x	x			x	x					
Lilly, Stella De Oro Day	*Hemerocallis* 'Stella de Oro'	x	x		x	x			x	x		x		x		x					x	x	x	x		
Sedum, Autumn Joy	*Sedum*	x	x		x				x	x	x	x		x		x	x						x	x	x	
Spike Gay Feather	*Liatris apicata*	x	x	x	x	x			x	x			x	x		x						x	x			
Yarrow, Paprika, Terracotta	*Achillea hybrids*	x	x		x											x	x					x	x			
Ornamental Grasses																										
Fountain Grass	*Pennisetum alopecuroides*	x	x		x					x	x	x					x									
Karl Foerster	*Calamagrostis acutifora cultivars*	x	x		x							x					x	x								
Switchgrass	*Panicum virgatum*	x	x	x	x	x				x	x	x	x	x			x	x								Shorter growing cultivars are available

Section 8: Culvert Design

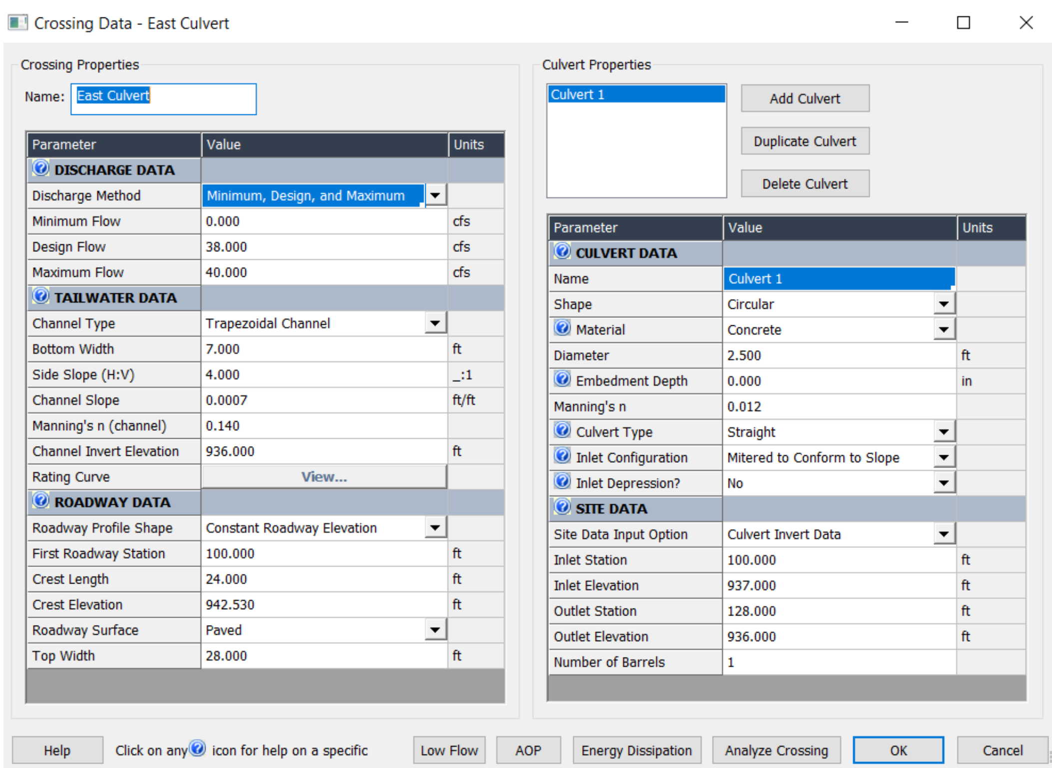
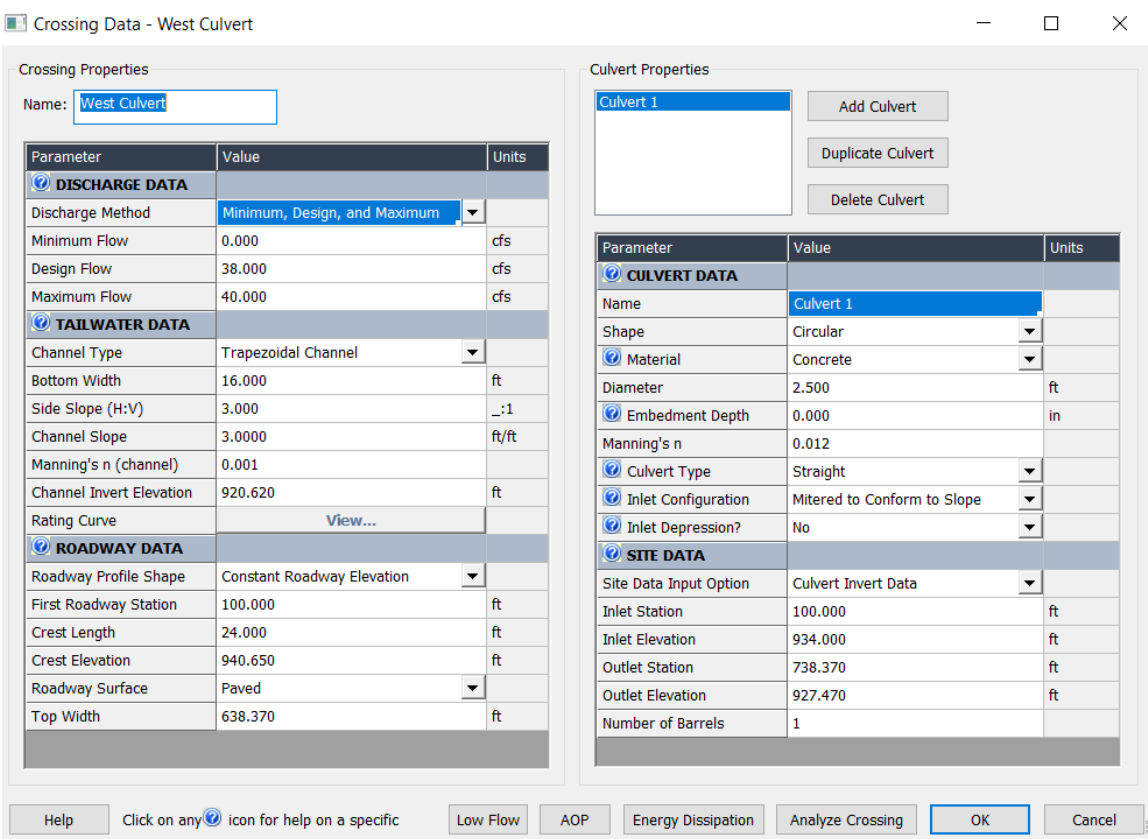


Figure 51. Crossing Data from Eastern Culvert

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
937.00	0.00	0.00	0.00	1
937.98	4.00	4.00	0.00	1
938.42	8.00	8.00	0.00	1
938.77	12.00	12.00	0.00	1
939.08	16.00	16.00	0.00	1
939.41	20.00	20.00	0.00	1
939.85	24.00	24.00	0.00	1
940.23	28.00	28.00	0.00	1
940.73	32.00	32.00	0.00	1
941.63	38.00	38.00	0.00	1
941.96	40.00	40.00	0.00	1
942.53	43.31	43.31	0.00	Overtopping

Figure 52. Elevation and discharge data from Eastern Culvert


Crossing Data - West Culvert

Crossing Properties

Name: West Culvert

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	0.000	cfs
Design Flow	38.000	cfs
Maximum Flow	40.000	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	16.000	ft
Side Slope (H:V)	3.000	_:1
Channel Slope	3.0000	ft/ft
Manning's n (channel)	0.001	
Channel Invert Elevation	920.620	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	100.000	ft
Crest Length	24.000	ft
Crest Elevation	940.650	ft
Roadway Surface	Paved	
Top Width	638.370	ft

Culvert Properties

Culvert 1

Add Culvert

Duplicate Culvert

Delete Culvert

Parameter	Value	Units
CULVERT DATA		
Name	Culvert 1	
Shape	Circular	
Material	Concrete	
Diameter	2.500	ft
Embedment Depth	0.000	in
Manning's n	0.012	
Culvert Type	Straight	
Inlet Configuration	Mitered to Conform to Slope	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	100.000	ft
Inlet Elevation	934.000	ft
Outlet Station	738.370	ft
Outlet Elevation	927.470	ft
Number of Barrels	1	

Help | Click on any icon for help on a specific | Low Flow | AOP | Energy Dissipation | Analyze Crossing | OK | Cancel

Figure 53. Crossing data from Western culvert

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
934.00	0.00	0.00	0.00	1
934.94	4.00	4.00	0.00	1
935.37	8.00	8.00	0.00	1
935.72	12.00	12.00	0.00	1
936.04	16.00	16.00	0.00	1
936.37	20.00	20.00	0.00	1
936.74	24.00	24.00	0.00	1
937.18	28.00	28.00	0.00	1
937.69	32.00	32.00	0.00	1
938.58	38.00	38.00	0.00	1
938.91	40.00	40.00	0.00	1
940.65	49.56	49.56	0.00	Overtopping

Figure 54. Elevation and discharge data from Western culvert

Section 9: Outlet Design

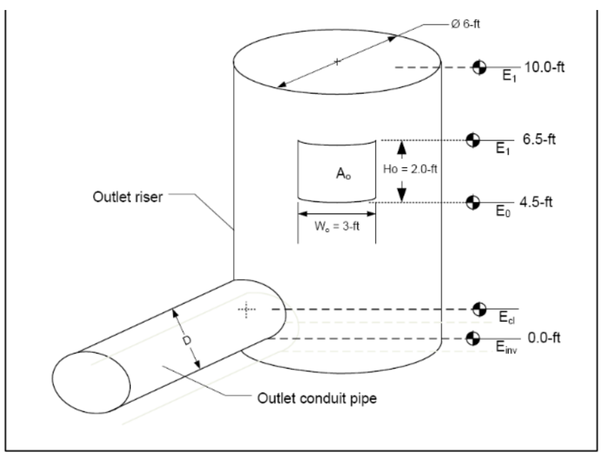


Figure C3-S12- 18: Example two-stage outlet riser

The riser has an inside diameter of 6 feet, and the top of the riser will function as a broad-crested weir of length L = 6.0π ft = 18.84 feet. The orifice has an area of 6 ft^2, and the stage-discharge relationship can be described as follows:

$$Q = 0 \qquad h \le 4.5ft$$
$$Q = C_w L(2gh)^{3/2} = (3.1)(3)[2g(h-4.5)]^{3/2} = 9.3[2g(h-4.5)]^{3/2} \qquad 4.5ft < h \le 6.5ft$$
$$Q = CA_o(2gh)^{0.5} = (0.6)(3)(2)[2g(h-4.5)]^{0.5} = 3.6[2g(h-4.5)]^{0.5} \qquad 6.5ft < h \le 10ft$$
$$Q = 18.0\pi(h-10)^{1.5} + 3.6[2g(h-4.5)]^{0.5} \qquad 10ft < h$$

Figure 55. Equations used from design example in ISWMM Chapter 3

Table 34. Weir coefficient values from ISWMM Chapter 3

Head (h)[1] (feet)	Weir Crest Breadth (b) (feet)										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.64	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.27	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[1]Measured at least 2.5h upstream of the weir

$$A = \frac{Q}{C\sqrt{2gh}}$$

Figure 56. Equation used to determine area of outlet pipe

$$\mathbf{d = (4A / 3.14)^{0.5}}$$

Figure 57. Equation used to determine diameter of outlet pipe

Table 35. Outlet structure design parameters

Parameter	Outlet A	Outlet B	Outlet C
Head (ft)	3.00	3.00	3.00
Breadth (ft)	0.50	0.50	0.50
Diameter (ft)	0.50	1.50	1.50
Area (ft^2)	0.20	1.77	1.77
Length (ft)	4.00	2.00	4.00
Inlet Width (ft)	0.25	0.50	0.25
Inlet Height (ft)	0.50	0.50	1.00
Cw	3.32	3.32	3.32
E0	0.00	0.00	0.00

Table 36. Outlet structure discharge pipe parameters

Outlet	Target Q (cfs)	Actual Q (cfs)	Outlet Area (ft^2)	Diameter (ft)
A	11.21	7.94	0.67	1.00
B	2.90	2.03	0.17	0.50
C	7.40	7.40	0.77	1.00

Appendix B-Roadway Design

Table 37. The Curb Radii based upon each of the specific roadway classifications

Table 5C-2.09: Curb Return Radii Based Upon Roadway Classification

Roadway Classification	Arterial	Collector	Local - Commercial/ Industrial	Local - Residential
Arterial	Special*	Special*	30'	30'
Collector	Special*	30'	30'	25'
Local - Commercial/Industrial	30'	30'	25'	25'
Local - Residential	30'	30'	25'	25'

*Special design required. Use turning templates.

Table 38. Calculations done to find the ESALs based on the SUDAS equations for an arterial road

Two Lane Roadway, PCC Arterial Road		
AADT	2000	
Trucks, Type A	4%	
Annual Growth Rate	2%	
Design period, years	25	
Base year Design AADT(From Table 5F-1.07)	4000	
Growth Factor (from Table 5f-1.11)	32	
2000*32	128000	ESALs
Base year Design AADT(From Table 5F-1.07) rounding up to an ESAL of 1,000,000 the pavment thickness is to be 6.5 inches with a 12 inch subbase and a CBR value of 3		

Table 39. Calculations done to find the ESALs based on the SUDAS equations for a local road

Two Lane Roadway, PCC Local Road		
AADT	1000	
Trucks, Type A	7%	
Annual Growth Rate	2%	
Design period, years	25	
Base year Design AADT(From Table 5F-1.07)	2000	
Growth Factor (from Table 5f-1.11)	32	
2000*32	64000	ESALs
Base year Design AADT(From Table 5F-1.07) rounding up to an ESAL of 300,000 the pavment thickness is to be 6 inches with a 12 inch subbase and a CBR value of 3		

Table 40. From Chapter 5F in the SUDAS manual shows the recommended thickness for rigid pavements for arterial roads, that was used to determine the road for the subdivision

Table 5F-1.15: Recommended Thickness for Rigid Pavement - *Arterial Roads*

CBR	3						5					
ESAL/ Subbase	Natural	4" Granular	6" Granular	8" Granular	10" Granular	12" Granular	Natural	4" Granular	6" Granular	8" Granular	10" Granular	12" Granular
1,000,000	7.5	7	7	7	7	6.5	7.5	6.5	6.5	6.5	6.5	6.5
1,500,000	8	7.5	7.5	7.5	7.5	7	8	7	7	7	7	7
2,000,000	8	8	7.5	7.5	7.5	7.5	8	7.5	7.5	7.5	7.5	7.5
3,000,000	8.5	8	8	8	8	8	8.5	8	8	8	8	8
4,000,000	9	8	8	8	8	8	8.5	8	8	8	8	8
5,000,000	9	8.5	8.5	8.5	8	8	9	8	8	8	8	8
7,500,000	10	9	9	9	9	9	9.5	8.5	8.5	8.5	8.5	8.5
10,000,000	10	9.5	9.5	9.5	9	9	10	9	9	9	9	9
12,500,000	10.5	9.5	9.5	9.5	9.5	9.5	10.5	9.5	9.5	9.5	9.5	9
15,000,000	11	10	10	10	10	10	10.5	9.5	9.5	9.5	9.5	9.5
17,500,000	11	10	10	10	10	10	11	10	10	10	10	10
20,000,000	11.5	10.5	10.5	10.5	10.5	10.5	11	10	10	10	10	10

Table 41. From Chapter 5F in the SUDAS manual shows the recommended thickness for rigid pavements for local roads, that was used to determine the road for the subdivision

Table 5F-1.13: Recommended Thickness for Rigid Pavement - *Local Roads*

CBR	3						5					
ESAL/ Subbase	Natural	4" Granular	6" Granular	8" Granular	10" Granular	12" Granular	Natural	4" Granular	6" Granular	8" Granular	10" Granular	12" Granular
300,000	7*	6*	6*	6*	6*	6*	7*	6*	6*	6*	6*	6*
500,000	7*	6*	6*	6*	6*	6*	7*	6*	6*	6*	6*	6*
750,000	7*	6	6	6*	6*	6*	7*	6*	6*	6*	6*	6*
1,000,000	7	6	6	6	6	6	7	6	6*	6*	6*	6*
1,500,000	7.5	6.5	6.5	6.5	6.5	6.5	7.5	6.5	6	6	6	6
2,000,000	8	7	7	7	7	7	7.5	6.5	6.5	6.5	6.5	6.5
3,000,000	8	7.5	7.5	7.5	7.5	7.5	8	7	7	7	7	7

* Represents the minimum thickness based on established policies of local jurisdictions; the calculated value is less.

Table 42. From Chapter 5F in the SUDAS manual shows the Average annual Traffic growth rate based on the Design period

Table 5F-1.07: Base Year Design ESALs for Two Lane *Rigid Pavement*

% Trucks	Truck Mix Type	Two-Way Base Year AADT							
		1,000	2,000	3,000	4,000	5,000	10,000	15,000	20,000
1	A	1,000	2,000	3,000	4,000	5,000	10,000	14,500	19,500
	B	1,500	3,500	5,000	6,500	8,000	16,500	24,500	32,500
	C	2,500	5,000	7,000	9,500	12,000	24,000	35,500	47,500
2	A	2,000	4,000	6,000	8,000	10,000	19,500	29,500	39,000
	B	3,500	6,500	10,000	13,000	16,500	32,500	49,000	65,500
	C	5,000	9,500	14,500	19,000	24,000	47,500	71,500	95,000
3	A	3,000	6,000	9,000	11,500	14,500	29,500	44,000	58,500
	B	5,000	10,000	14,500	19,500	24,500	49,000	73,500	98,000
	C	7,000	14,500	21,500	28,500	35,500	71,500	107,000	142,500
4	A	4,000	8,000	11,500	15,500	19,500	39,000	58,500	78,000
	B	6,500	13,000	19,500	26,000	32,500	65,500	98,000	130,500
	C	9,500	19,000	28,500	38,000	47,500	95,000	142,500	190,000
5	A	5,000	10,000	14,500	19,500	24,500	49,000	73,000	97,500
	B	8,000	16,500	24,500	32,500	41,000	81,500	122,500	163,500
	C	12,000	24,000	35,500	47,500	59,500	119,000	178,000	237,500
6	A	6,000	11,500	17,500	23,500	29,500	58,500	88,000	117,000
	B	10,000	19,500	29,500	39,000	49,000	98,000	147,000	196,000
	C	14,500	28,500	43,000	57,000	71,500	142,500	214,000	285,000
7	A	7,000	13,500	20,500	27,500	34,000	68,500	102,500	136,500
	B	11,500	23,000	34,500	45,500	57,000	114,500	171,500	228,500
	C	16,500	33,500	50,000	66,500	83,000	166,500	249,500	332,500
8	A	8,000	15,500	23,500	31,000	39,000	78,000	117,000	156,000
	B	13,000	26,000	39,000	52,500	65,500	130,500	196,000	261,500
	C	19,000	38,000	57,000	76,000	95,000	190,000	285,000	380,000
9	A	9,000	17,500	26,500	35,000	44,000	88,000	132,000	175,500
	B	14,500	29,500	44,000	59,000	73,500	147,000	220,500	294,000
	C	21,500	43,000	64,000	85,500	107,000	214,000	321,000	427,500
10	A	10,000	19,500	29,500	39,000	49,000	97,500	146,500	195,000
	B	16,500	32,500	49,000	65,500	81,500	163,500	245,000	326,500
	C	24,000	47,500	71,500	95,000	119,000	237,500	356,500	475,000
12	A	11,500	23,500	35,000	47,000	58,500	117,000	175,500	234,000
	B	19,500	39,000	59,000	78,500	98,000	196,000	294,000	392,000
	C	28,500	57,000	85,500	114,000	142,500	285,000	427,500	570,500
14	A	13,500	27,500	41,000	54,500	68,500	136,500	205,000	273,500
	B	23,000	45,500	68,500	91,500	114,500	228,500	343,000	457,500
	C	33,500	66,500	100,000	133,000	166,500	332,500	499,000	665,500
16	A	15,500	31,000	47,000	62,500	78,000	156,000	234,000	312,500
	B	26,000	52,500	78,500	104,500	130,500	261,500	392,000	522,500
	C	38,000	76,000	114,000	152,000	190,000	380,000	570,500	760,500
18	A	17,500	35,000	52,500	70,500	88,000	175,500	263,500	351,500
	B	29,500	59,000	88,000	117,500	147,000	294,000	441,000	588,000
	C	43,000	85,500	128,500	171,000	214,000	427,500	641,500	855,500
20	A	19,500	39,000	58,500	78,000	97,500	195,000	293,000	390,500
	B	32,500	65,500	98,000	130,500	163,500	326,500	490,000	653,500
	C	47,500	95,000	142,500	190,000	237,500	475,000	713,000	950,500

Assumes two lane roadway with 50/50 directional split of base year AADT

Table 43. From Chapter 5F in the SUDAS manual shows the ESAls, for different types of trucks

Table 5F-1.11: Growth Factor

Design Period Years (n)	Average Annual Traffic Growth Rate, Percent					
	No Growth	1%	2%	3%	4%	5%
1	1.0	1.0	1.0	1.0	1.0	1.0
2	2.0	2.0	2.0	2.0	2.0	2.1
3	3.0	3.0	3.1	3.1	3.1	3.2
4	4.0	4.1	4.1	4.2	4.2	4.3
5	5.0	5.1	5.2	5.3	5.4	5.5
6	6.0	6.2	6.3	6.5	6.6	6.8
7	7.0	7.2	7.4	7.7	7.9	8.1
8	8.0	8.3	8.6	8.9	9.2	9.5
9	9.0	9.4	9.8	10.2	10.6	11.0
10	10.0	10.5	10.9	11.5	12.0	12.6
11	11.0	11.6	12.2	12.8	13.5	14.2
12	12.0	12.7	13.4	14.2	15.0	15.9
13	13.0	13.8	14.7	15.6	16.6	17.7
14	14.0	14.9	16.0	17.1	18.3	19.6
15	15.0	16.1	17.3	18.6	20.0	21.6
16	16.0	17.3	18.6	20.2	21.8	23.7
17	17.0	18.4	20.0	21.8	23.7	25.8
18	18.0	19.6	21.4	23.4	25.6	28.1
19	19.0	20.8	22.8	25.1	27.7	30.5
20	20.0	22.0	24.3	26.9	29.8	33.1
21	21.0	23.2	25.8	28.7	32.0	35.7
22	22.0	24.5	27.3	30.5	34.2	38.5
23	23.0	25.7	28.8	32.5	36.6	41.4
24	24.0	27.0	30.4	34.4	39.1	44.5
25	25.0	28.2	32.0	36.5	41.6	47.7
26	26.0	29.5	33.7	38.6	44.3	51.1
27	27.0	30.8	35.3	40.7	47.1	54.7
28	28.0	32.1	37.1	42.9	50.0	58.4
29	29.0	33.5	38.8	45.2	53.0	62.3
30	30.0	34.8	40.6	47.6	56.1	66.4
31	31.0	36.1	42.4	50.0	59.3	70.8
32	32.0	37.5	44.2	52.5	62.7	75.3
33	33.0	38.9	46.1	55.1	66.2	80.1
34	34.0	40.3	48.0	57.7	69.9	85.1
35	35.0	41.7	50.0	60.5	73.7	90.3
36	36.0	43.1	52.0	63.3	77.6	95.8
37	37.0	44.5	54.0	66.2	81.7	101.6
38	38.0	46.0	56.1	69.2	86.0	107.7
39	39.0	47.4	58.2	72.2	90.4	114.1
40	40.0	48.9	60.4	75.4	95.0	120.8
41	41.0	50.4	62.6	78.7	99.8	127.8
42	42.0	51.9	64.9	82.0	104.8	135.2
43	43.0	53.4	67.2	85.5	110.0	143.0
44	44.0	54.9	69.5	89.0	115.4	151.1
45	45.0	56.5	71.9	92.7	121.0	159.7
46	46.0	58.0	74.3	96.5	126.9	168.7
47	47.0	59.6	76.8	100.4	132.9	178.1
48	48.0	61.2	79.4	104.4	139.3	188.0
49	49.0	62.8	81.9	108.5	145.8	198.4
50	50.0	64.5	84.6	112.8	152.7	209.3

Growth Factor = $\frac{[(1+r)^n]-1}{r}$ for values of n > 0

Appendix C-Sanitary Sewer Design

For residential development, the flows can be predicted using the following densities:

1. **Discharge (Q) Average Daily Flow (minimum):**

   Area x Area Density x Flow Rate = Average Daily Flow **Equation 3B-1.01**

   Number of Units x Unit Density x Flow Rate = Average Daily Flow **Equation 3B-1.02**

2. **Discharge (Q) Peak Sewer Flow (minimum):** Average daily flow times ratio of peak to average daily flow (See Figure 3B-1.01 for ratio). NOTE: Population values shown in Figure 3B-1.01 are based on the area that discharges into the sewer.

3. **Design Density and Rate:** See Table 3B-1.01.

Figure 58. The Discharge (Q) equations from SUDAS used to find the sizes of the pipes for the sanitary sewer

Table 44. The minimum values for the land use type, area density, unit density and the rate used to find the sizes

Table 3B-1.01: Minimum Values

Land Use	Area Density	Unit Density	Rate
Low Density (Single Family) Residential	10 people / AC	3 people / unit	100 gpcd*
Medium Density (Multi-Family) Residential	15 people / AC 6.0 people / duplex	3 people / unit	100 gpcd*
High Density (Multi-Family) Residential	30 people / AC	2.5 people / unit	100 gpcd*
Office and Institutional	5,000 gpd / AC (IDNR)	Special Design Density	N/A
Commercial and Light Industrial	5,000 gpd/AC (IDNR)	Special Design Density	N/A
Industrial	10,000 gpd/AC (IDNR)	Special Design Density	N/A

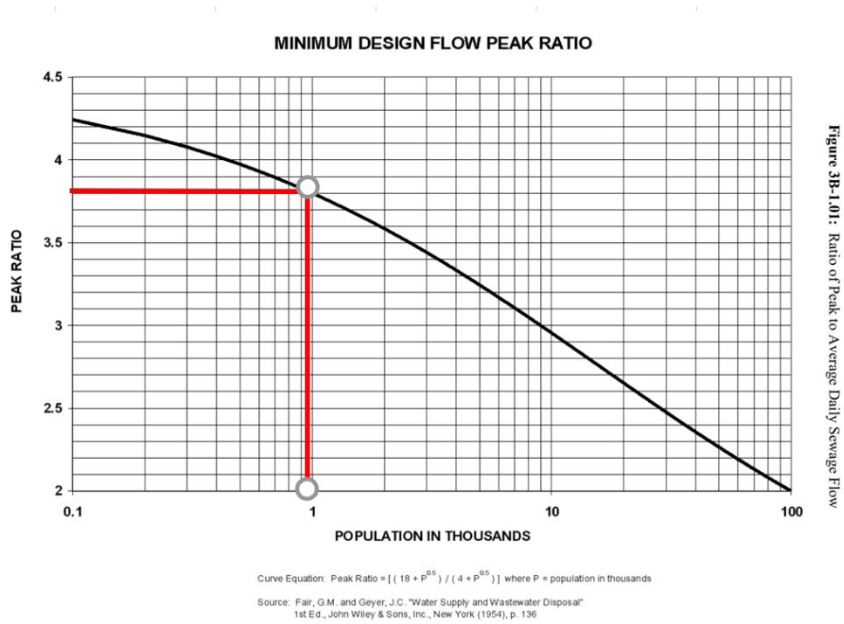


Figure 59. The minimum design flow peak ratio from the chart

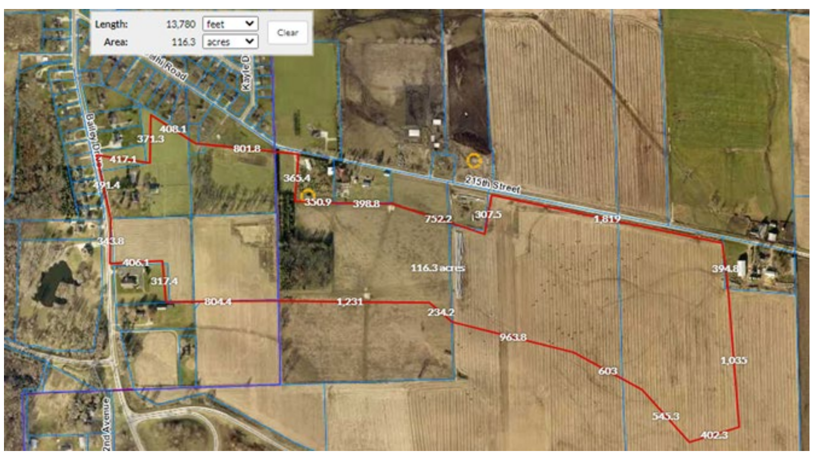


Figure 60. The area boundary of the North sanitary sewer system for the full expansion

Table 45. The calculations used to find the Peak Flow Factor for the full North section

To find out the Peak Flow:	
total acres(AC)	120
lot sizes(AC)	0.45
total lots	266.6666667
people per lot	3.76
total pop	1002.666667
Peak flow(from design flow peak ratio)	3.8

Table 46. The calculations used to find the discharge flow in cfs for the full North section

Residential Predicted flows(Q) SUDAS 3B-1.01	
Area AC	120
Area Density (People/AC)	10
Flow rategcpd (Table 3B-1.01)	100
Averag Daily Flow gallons/day	120000
Discharge (Q) gallons/day	456000
Discharge (Q) (cfs)	0.7055232

Set units: m mm ft in [Hide this line]

Inputs

Pipe diameter, d_0	12	in	X
Manning roughness, n	0.012		X
Pressure slope (possibly ? equal to pipe slope), S_0	.01	rise/run	X
Percent of (or ratio to) full depth (100% or 1 if flowing full)	0.96	fraction	X

Results

Flow, Q	4.1348	cfs	X
Velocity, v	5.3363	ft/sec	X
Velocity head, h_v	0.4426	ft H2O	X
Flow area	0.7749	ft^2	X
Wetted perimeter	2.7388	ft	X
Hydraulic radius	0.2829	ft	X
Top width, T	0.3919	ft	X
Froude number, F	0.67		X
Shear stress (tractive force), tau	0.1766	psf	X

Figure 61. The calculated measurements that were found from the Manning's equation calculator for a 12 in diameter pipe

Figure 62. The boundary of the North sanitary sewer system used to check to see if the pipe size needs to change

Table 47. The calculations used to find the discharge flow in cfs for the second test of the North section

Residential Predicted flows(Q) SUDAS 3B-1.01	
Area (AC)	80
Area Density (People/ AC)	10
Flow rate:gcpd (Table 3B-1.01)	100
Averag Daily Flow (gal/day)	80000
Discharge (Q) Peak Sewer Flow(gal/day)	304000
Discharge (Q) (cfs)	0.470349

Figure 63. The boundary of the North sanitary sewer system used to check to see if the pipe size needs to change

Table 48. The calculations used to find the discharge flow in cfs for the North section for the third section

Residential Predicted flows(Q) SUDAS 3B-1.01	
Area (AC)	75
Area Density (People/ AC)	10
Flow rate:gcpd (Table 3B-1.01)	100
Averag Daily Flow (gal/day)	75000
Discharge (Q) Peak Sewer Flow(gal/day)	292500
Discharge (Q) (cfs)	0.452556

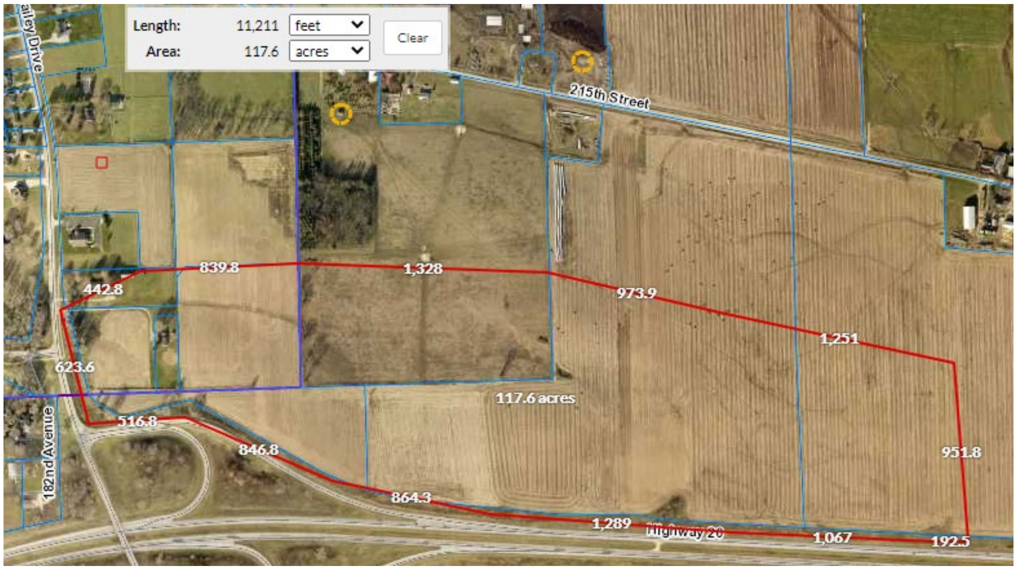


Figure 64. The boundary of the South sanitary sewer system including additional area for expansion

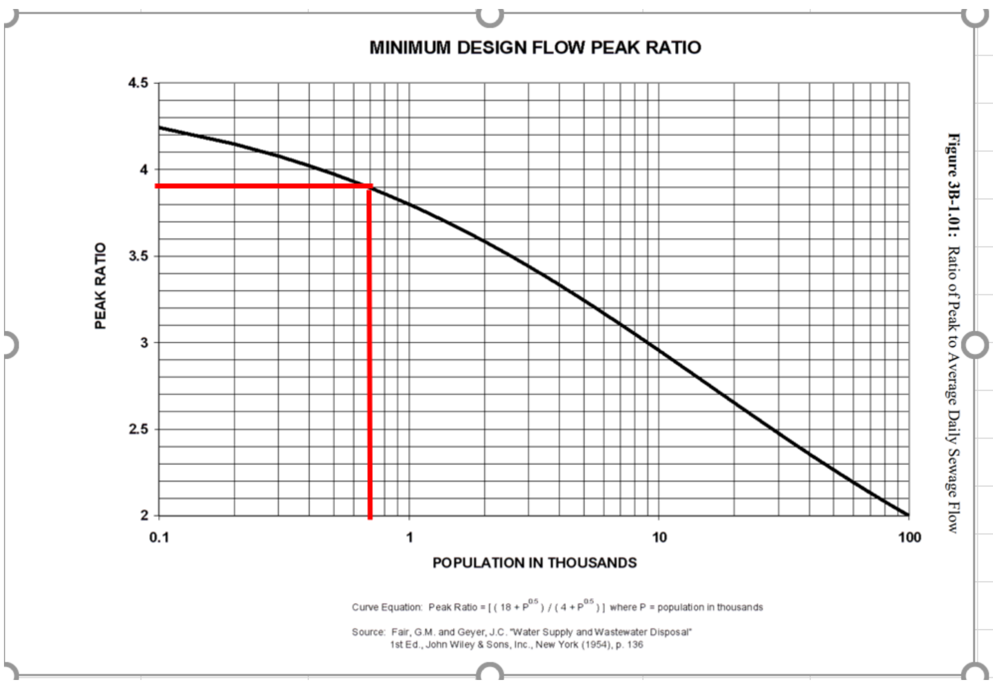


Figure 65. The calculations to find the Peak flow Factor for the full South system

To find out the Peak Flow:	
total acres(AC)	86.7
lot sizes(AC)	0.45
total lots	192.6667
people	3.76
total pop	724.4267
peak flow from min design flow Peak ration	3.9

Table 49. The calculations used to find the discharge flow in cfs for the full South system section

Residential Predicted flows(Q) SUDAS 3B-1.01	
Area (AC)	86.7
Area Density (People/ AC)	10
Flow rate:gcpd (Table 3B-1.01)	100
Averag Daily Flow (gal/day)	86700
Discharge (Q) Peak Sewer Flow(gal/day)	338130
Discharge (Q) (cfs)	0.523155

Set units: m mm ft in [Hide this line]

Inputs

Pipe diameter, d_0	8	in	X
Manning roughness, n	0.012		X
Pressure slope (possibly ? equal to pipe slope), S_0	.01	rise/run	X
Percent of (or ratio to) full depth (100% or 1 if flowing full)	0.96	fraction	X

Results

Flow, Q	1.4024	cfs	X
Velocity, v	4.0724	ft/sec	X
Velocity head, h_v	0.2577	ft H2O	X
Flow area	0.3444	ft^2	X
Wetted perimeter	1.8259	ft	X
Hydraulic radius	0.1886	ft	X
Top width, T	0.2613	ft	X
Froude number, F	0.63		X
Shear stress (tractive force), tau	0.1177	psf	X

Figure 66. The calculated measurements found from the Manning's equation calculator for a 8 in diameter pipe

Figure 67. The boundary of the second set of calculations of the South sanitary sewer

Table 50. The calculations used to find the discharge flow in cfs for the South system section

Residential Predicted flows(Q) SUDAS 3B-1.01	
Area (AC)	68
Area Density (People/ AC)	10
Flow rate:gcpd (Table 3B-1.01)	100
Averag Daily Flow (gal/day)	68000
Discharge (Q) Peak Sewer Flow(gal/day)	265200
Discharge (Q) (cfs)	0.410317

Figure 68. The boundary of the last set of calculations for the South sanitary sewer system

Table 51. The calculations used to find the discharge flow in cfs for the South system section

Residential Predicted flows(Q) SUDAS 3B-1.01	
Area (AC)	66
Area Density (People/ AC)	10
Flow rate:gcpd (Table 3B-1.01)	100
Averag Daily Flow (gal/day)	66000
Discharge (Q) Peak Sewer Flow(gal/day)	248160
Discharge (Q) (cfs)	0.383953

Appendix D-Water Main Design

Table 52. The needed fire flows for one and two family from Chapter 4B in the SUDAS Design Manual

Distance Between Buildings	Needed Fire Flow
Over 100’	500 gpm
31’ to 100’	750 gpm
11’ to 30’	1,000 gpm
10’ or less	1,500 gpm

Figure 4C-1.01: Minimum Depth of Cover for Water Main Installation

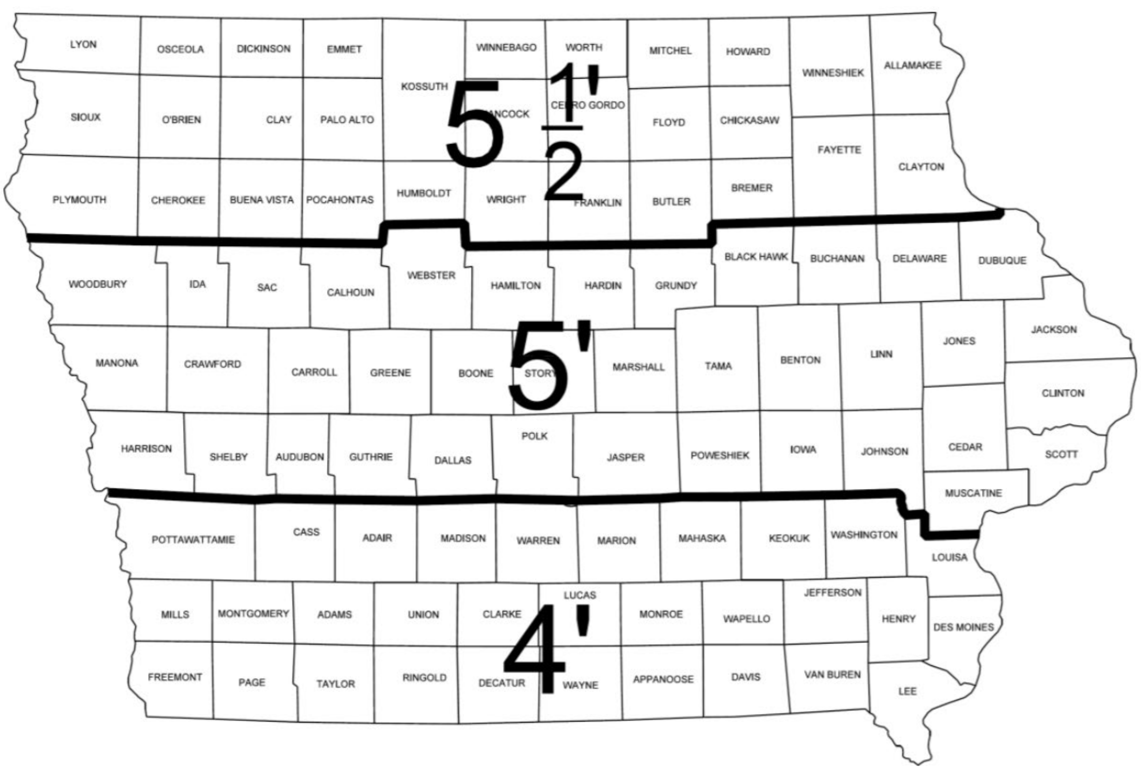


Figure 69. The minimum depth of cover for water Main installation from the SUDAS design manual

Appendix E-Grading

Appendix F-Bike Path Design

Table 53. The minimum design speeds and horizontal alignments for the bike path

Table 12B-2.01: Minimum Design Speed and Horizontal Alignment

Terrain	Design Speed (mph)	Minimum Radius[1] (Horizontal Curve) (feet)
Grades less than 2%	18	60
Grades less than or equal to 5%	25	115
Grades 6% and more	30	166

[1] Based on 20 degree maximum lean angle

Appendix G-Storm Sewer Design

$$Q = \frac{1.8}{n} A R^{2/3} S_o^{1/2} = \frac{0.56}{n} S_x^{5/3} S_o^{1/2} T^{8/3}$$

Q = calculated flow rate for the half-street (cfs)
n = Manning's roughness coefficient (0.016 for asphalt street with concrete gutter, 0.013 for concrete street and gutter)
R = hydraulic radius of wetted cross section = A/P (ft)
A = cross-sectional area (ft^2)
P = wetted perimeter of cross section (ft)
S_x = street cross slope (ft/ft)
S_o = longitudinal slope (ft/ft)
T = top width of flow spread (ft).

Figure 67. Equation for the gutter flow rate

$$A = \frac{S_x T^2}{2}$$

Figure 71. Equations for the cross-sectional area of the gutter

The flow depth can be found using:

$$y = TS_x$$

Where:

y = flow depth at the gutter flowline (ft).

Figure 72. Equation of the flow depth of the gutter

$$E_0 = 1 - \left(1 - \frac{W}{T}\right)^{2.67}$$

where:

E_0 = ratio of frontal flow to total gutter flow
W = width of depressed gutter or grate, ft
T = total spread of water, ft

Figure 73: Ratio of frontal flow to total gutter flow

$$V = \left(\frac{1.11}{n}\right)(S_L)^{0.5}(S_x)^{0.67}T_A{}^{0.67}$$

where:

V = Velocity in a triangular channel (gutter), ft/s
T_A = Average width of flow (spread) between intakes, ft
n = Manning's coefficient (see Table 2C-2.01)
S_x = Cross slope of pavement, ft/ft
S_L = Longitudinal slope of pavement, ft/ft

Figure 74. Equation of the gutter velocity

$$R_f = 1 - 0.09(V - V_0) \quad \text{(see note below)}$$

where:

V = velocity of flow in the gutter, ft/s
V_0 = gutter velocity where splash over first occurs, ft/s

Figure 75. Ratio of frontal flow to intercepted velocity

$$R_S = \frac{1}{1+\left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)} \quad \text{(see note below)}$$

where:

V = velocity of flow in the gutter, ft/s
L = length of the grate, ft
S_x = cross slope, ft/ft

Figure 76. Ratio of side flow intercepted to total side flow

The efficiency (E) of a grate is expressed as:

$$E = R_f E_0 + R_s(1 - E_0)$$

Figure 77. Equation of the efficiency of the grate intake

$$Q_i = EQ_t = Q_t[R_f E_0 + R_s(1 - E_0)]$$

Figure 78. Equation of the interception capacity for the grated intake

Table 54. Arterial street flow calculations

Arterial (15')

Location	SL	T (ft)	Sx	y (ft)	n	Gutter Flow Q (cfs)	E0	V (ft/s)	Rf	Rs	E	Qi (cfs)
Running West to East	0.017	9	0.02	0.18	0.016	2.35679888	0.4888	2.8672	0.877	0.2004	0.5311	1.2516
Running East to West to intersection	0.008	9	0.02	0.18	0.016	1.61675069	0.4888	1.9669	0.958	0.3305	0.6372	1.0303
Running East to West to low point	0.015	9	0.02	0.18	0.016	2.21382706	0.4888	2.6933	0.8926	0.219	0.5483	1.2138
Running West to East to low point	0.021	9	0.02	0.18	0.016	2.6194355	0.4888	3.1867	0.8482	0.1716	0.5023	1.3158
Running West to East to inersection	0.016	9	0.02	0.18	0.016	2.28643076	0.4888	2.7816	0.8847	0.2092	0.5394	1.2333

Table 55. Local street calculations

Local (12')

Location	SL	T (ft)	Sx	y (ft)	n	Q (cfs)	E0	V (ft/s)	Rf	Rs	E	Qi (cfs)	Qb (cfs)
Running from the North boundary to int	0.04	14	0.02	0.28	0.016	11.7441826	0.3374	5.9133	1	0.0637	0.3796	4.4585	7.2857
Rinnung S to N from hammer inetsection	0.02	14	0.02	0.28	0.016	8.30439115	0.3374	4.1813	0.7587	0.1127	0.3307	2.746	5.5584
Running N to South to South entry Int.	0.023	14	0.02	0.28	0.016	8.90546737	0.3374	4.484	0.7314	0.1007	0.3135	2.7922	6.1133
Running S to N from S road to S entry Int.	0.027	14	0.02	0.28	0.016	9.64883058	0.3374	4.8583	0.6978	0.0884	0.294	2.8367	6.8121
Running W to E along S entry	0.006	14	0.02	0.28	0.016	4.54850236	0.3374	2.2902	0.9289	0.273	0.4943	2.2482	2.3003
Runn W to E to Souther road HP to S curve	0.005	14	0.02	0.28	0.016	4.15219557	0.3374	2.0907	0.9468	0.3067	0.5227	2.1703	1.9819
Running E to W from S HP to S intersection	0.012	14	0.02	0.28	0.016	6.43255372	0.3374	3.2388	0.8435	0.1675	0.3956	2.5446	3.888
Running W to E from W boundary to S inter.	0.007	14	0.02	0.28	0.016	4.91294406	0.3374	2.4737	0.9124	0.2463	0.471	2.3142	2.5988
Running S to N from S inter to LP (West road)	0.028	14	0.02	0.28	0.016	9.82588811	0.3374	4.9474	0.6897	0.0858	0.2896	2.8452	6.9807
Running N to S from W road HP to LP	0.02	14	0.02	0.28	0.016	8.30439115	0.3374	4.1813	0.7587	0.1127	0.3307	2.746	5.5584
Running S to N from W road HP to W inter.	0.022	14	0.02	0.28	0.016	8.70971891	0.3374	4.3854	0.7403	0.1044	0.319	2.7782	5.9316

Appendix H-Cost Estimate

Table 56. The total cost estimate of the project

Reference	Item Code	Bid Item Description	Items to be Specified on Plans or in Contract Documents (I)	Incidental or Included Items (J)	Total	Unit	Unit Price in Dollars	Total Cost(Dollars)
EARTHWORK								
2010, 1.08, B	2010-108-B-0	Clearing and Grubbing		Removal and disposal of all materials and placement of backfill in area where roots have been removed.	2	AC	$ 400.00	$ 800.00
2010, 1.08, E	2010-108-E-0	Excavation, Class 10	Class 10	a. Site preparation for, and the construction of, embankment, fills, shoulder backfill, and backfill behind curbs. b. Overhaul. c. Finishing the soil surface, including roadways, shoulders, behind curbs, side ditches, slopes, and borrow pits. d. Repair or replacement of any fences that have been unnecessarily damaged or removed.	67228	CY	$ 5.57	$ 374,460.00
2010, 1.08, D	2010, 1.08, D	Strip and stock pile			24200	CY	$ 3.00	$ 72,600.00
2010, 1.08, I	2010-108-I-0	Subbase, gravel,6"		Furnishing, placing, compacting, and trimming to the proper grade.	13308	SY	$ 4.00	$ 53,232.00

Section 4010 - Sanitary Sewers								
4010, 1.08, A, 1	4010-108-A-1	Sanitary Sewer Gravity Main, Trenched, PVC, 12",SDR 26		Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	1300	LF	$ 56.00	$ 72,800.00
4010, 1.08, A, 1	4010-108-A-1	Sanitary Sewer Gravity Main, Trenched, PVC, 8",SDR26		Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	3093	LF	$ 47.00	$ 145,371.00
		Lift Station		Motors, pumps, pipes	1	LS	$ 250,000.00	$ 250,000.00
		Force Main 8" Trenched		Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; tracer system; testing; disinfection; and polyethylene wrap for ductile iron pipe and for fittings.	500	LF	$ 50.00	$ 25,000.00

6010, 1.08, A	6010-108-A-0	Manhole,Sanitary, 48"		Excavation; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; base; structural concrete; reinforcing steel; precast units (if used); concrete fillets; pipe connections; infiltration barriers (sanitary sewer manholes only); castings; and adjustment rings.	20	EA	$ 5,000.00	$ 100,000.00
Section 4020 - Storm Sewers								
4020, 1.08, A, 1	4020-108-A-1	Storm Sewer, Trenched, Concrete, 12 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	1279.59	LF	$ 38.50	$ 49,265.00
4020, 1.08, A, 1	4020-108-A-1	Storm Sewer, Trenched, Concrete, 15 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe	829.79	LF	$ 30.00	$ 24,894.00

				joints; pipe connections; testing; and inspection.				
4020, 1.08, A, 1	4020-108-A-1	Storm Sewer, Trenched, Concrete, 21 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	159	LF	$ 55.00	$ 8,745.00
4020, 1.08, A, 1	4020-108-A-1	Storm Sewer, Trenched, Concrete, 24 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	252.12	LF	$ 50.00	$ 12,606.00
4020, 1.08, A, 1	4020-108-A-1	Storm Sewer, Trenched, Concrete, 27 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	227.52	LF	$ 76.75	$ 17,463.00
4020, 1.08, A, 1	4020-	Storm Sewer, Trenched,	Class 3	Trench excavation; dewatering; furnishing and	51.63	LF	$ 64.50	$ 3,331.00

	108 -A-1	Concrete, 30 inch		installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.				
4020, 1.08, A, 1	402 0- 108 -A-1	Storm Sewer, Trenched, Concrete, 36 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	40 0.9 9	LF	$ 80.00	$ 32,08 0.00
4020, 1.08, A, 1	402 0- 108 -A-1	Storm Sewer, Trenched, Concrete, 42 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	51 0.6 8	LF	$ 150.0 0	$ 76,60 2.00
4020, 1.08, A, 1	402 0- 108 -A-1	Storm Sewer, Trenched, Concrete, 48 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe	16 6.3 5	LF	$ 135.0 0	$ 22,45 8.00

				joints; pipe connections; testing; and inspection.				
4020, 1.08, A, 1	402 0-108 -A-1	Storm Sewer, Trenched, Concrete, 54 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	75 2.7 2	LF	$ 220.0 0	$ 165,5 99.00
4020, 1.08, A, 1	402 0-108 -A-1	Storm Sewer, Trenched, Concrete, 60 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	12 2.2 9	LF	$ 340.0 0	$ 41,57 9.00
4020, 1.08, A, 1	402 0-108 -A-1	Intake, SW-502, 48 in		Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	53	EA	$ 2,600. 00	$ 137,8 00.00
Section 4030 - Pipe Culverts								

4030, 1.08, A, 1	4030-108-A-1	Pipe Culvert, Trenched, Type III, 30"	Type III, 30" Diameter, 638 feet	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; connectors; testing; and inspection.	666	LF	$32.00	$ 21,312.00
DIVISION 5 - WATER MAINS AND APPURTENANCES								
5010, 1.08, A, 1	5010-108-A-1	Water Main, Trenched, PVC, 8"		Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; tracer system; testing; disinfection; and polyethylene wrap for ductile iron pipe and for fittings.	4525	LF	$ 50.00	$ 226,250.00
5010, 1.08, C, 1	5010-108-C-1	Fitting, PVC TEE,8"		Specify whether measurement of fittings will be made by count or by weight.	10	EA	$ 56.00	$ 560.00
5010, 1.08, C, 1	5010-108-C-1	Fitting, PVC Cross,8"		Specify whether measurement of fittings will be made by count or by weight.	2	EA	$ 45.00	$ 90.00
Section 5020 - Valves, Fire Hydrants, and Appurtenances								

5020, 1.08, A	5020-108-A-0	Valve, Gate, 8"		All components attached to the valve or required for its complete installation, including underground or above ground operator, square valve operation nut, valve box and cover, valve box extension, and valve stem extension.	18	EA	$ 750.00	$ 13,500.00
5020, 1.08, C	5020-108-C-0	Fire Hydrant Assembly		The fire hydrant, barrel extensions sufficient to achieve proper bury depth of anchoring pipe and height of fire hydrant above finished grade, and components to connect the fire hydrant to the water main, including anchoring pipe, fittings, thrust blocks, pea gravel or porous backfill material, and fire hydrant gate valve and appurtenances, except tapping valve assembly if used.	9	EA	$ 1,500.00	$ 13,500.00
Section 7010 - Portland Cement Concrete Pavement								
7010, 1.08, A	7010-108-A-0	Pavement, PCC, 7"		Final trimming of subgrade or subbase, integral curb, bars and reinforcement, joints and sealing, surface curing and	10371	SY	$ 55.00	$ 570,405.00

				pavement protection, safety fencing, concrete for rigid headers, boxouts for fixtures, and pavement smoothness testing.				
7010, 1.08, A	701 0- 108 -A-0	Pavement, PCC, 8"		Final trimming of subgrade or subbase, integral curb, bars and reinforcement, joints and sealing, surface curing and pavement protection, safety fencing, concrete for rigid headers, boxouts for fixtures, and pavement smoothness testing.	43 27	SY	$ 62.00	$ 268,2 74.00
Section 7030 - Shared Use Paths				#NAME?				
7030, 1.08, C	703 0- 108 -C-0	Shared Use Path, PCC, 6"		Subgrade preparation, jointing, sampling, slope and smoothness testing and correction, and testing.	19 75	SY	$ 50.00	$ 98,75 0.00
DIVISION 9 - SITE WORK AND LANDSCAPING								
9010,1.08, A	901 0- 108 -A-0	Conventiona l Seeding, Seeding, Fertilizing, and Mulching		Removal of rock and other debris from the area; repairing rills and washes; preparing the seedbed; furnishing and placing seed, including any treatment required;	24	AC	$ 1,000. 00	$ 24,00 0.00

				furnishing and placing fertilizer and mulch; and furnishing water and other care during the care period, unless these items are bid separately.				
Section 9010 - Seeding								
9010,1.08, A	9010-108-A-0	Conventional Seeding, Seeding, Fertilizing, and Mulching	Switch Grass		50601	SF	$0.02	$ 1,013.00
9030, 1.08, A	9030-108-A-0	Conventional Seeding, Seeding, Fertilizing, and Mulching	Buffalo Grass		4976	SF	$0.37	$ 1,842.00
Section 9030 - Plant Material and Planting								
9030, 1.08, A	9030-108-A-0	Plants, Native Species	Native species for bioswales as recommended from the Iowa Education Partnership list	Delivery, excavation, installation, watering, placing backfill material, mulching, wrapping, staking or guying, herbicide, maintenance during the establishment period, and replacements.	3965	EA	$ 0.30	$ 1,190.00
Section 9040 - Erosion and Sediment Control								

9040, 1.08, L, 1. c	9040-108-L-1	Outlet Structure, 1.5 ft diameter	Reinforced Concrete Pavement, 4 ft long. Includes grate	Specify the use of anti-seep collars.	2	EA	$755.00	$ 1,510.00
9040, 1.08, L, 1. c	9040-108-L-1	Outlet Structure, 0.5 ft diameter	Reinforced Concrete Pavement, 4 ft long. Includes grate	Specify the use of anti-seep collars.	1	EA	$738.00	$ 738.00
9040, 1.08, U	9040-108-U-0	Flow Transition Mat	Bentonite Clay Liner	Anchoring devices.	12429	SF	$1.15	$ 14,294.00
Section 11,020 - Mobilization								
11,020 1.08, A	11,020-108-A	Mobilization		The movement of personnel, equipment, and supplies to the project site; the establishment of offices, buildings, and other facilities necessary for the project; and bonding, permits, and other expenses incurred prior to construction.	1	LS	$ 10,000.00	$ 10,000.00

Table 57. The cost estimate of the hammer head portion of the design

Reference	Item Code	Bid Item Description	Items to be Specified on Plans or in Contract Documents (I)	Incidental or Included Items (J)	Total	Unit	Unit Price in Dollars	Total Cost(Dollars)
EARTHWORK								
2010, 1.08, B	2010 - 108-B-0	Clearing and Grubbing		Removal and disposal of all materials and placement of backfill in area where roots have been removed.	0	AC	$ 400.00	$ -
2010, 1.08, E	2010 - 108-E-0	Excavation, Class 10	Class 10	a. Site preparation for, and the construction of, embankment, fills, shoulder backfill, and backfill behind curbs. b. Overhaul. c. Finishing the soil surface, including roadways, shoulders, behind curbs, side ditches, slopes, and borrow pits. d. Repair or replacement of any fences that have been unnecessarily damaged or removed.	4500	CY	$ 5.57	$ 25,065.00
2010, 1.08, D	2010 , 1.08, D	Strip and stock pile			1600	CY	$ 3.00	$ 4,800.00
2010, 1.08, I	2010 - 108-I-0	Subbase, gravel,6"		Furnishing, placing, compacting, and trimming to the proper grade.	1390	SY	$ 4.00	$ 5,560.00

Section 4010 - Sanitary Sewers								
4010, 1.08, A, 1	4010 - 108- A-1	Sanitary Sewer Gravity Main, Trenched, PVC, 8",SDR26		Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	37 2	LF	$ 47.00	$ 17,484. 00
		Lift Station		Motors, pumps, pipes		LS		
Section 4020 - Storm Sewers								
4020, 1.08, A, 1	4020 - 108- A-1	Storm Sewer, Trenched, Concrete, 12 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	10 1.2 8	LF	$ 38.50	$ 3,899.2 8
4020, 1.08, A, 1	4020 - 108- A-1	Storm Sewer, Trenched, Concrete, 18 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	10 9.4 3	LF	$ 39.00	$ 4,267.7 7

4020, 1.08, A, 1	4020 - 108- A-1	Storm Sewer, Trenched, Concrete, 27 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	59. 03	LF	$ 76.75	$ 4,530.5 5
4020, 1.08, A, 1	4020 - 108- A-1	Storm Sewer, Trenched, Concrete, 30 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	12 4.9 3	LF	$ 64.50	$ 8,057.9 9
4020, 1.08, A, 1	4020 - 108- A-1	Storm Sewer, Trenched, Concrete, 36 inch	Class 3	Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	34. 01	LF	$ 80.00	$ 2,720.8 0
4020, 1.08, A, 1	4020 - 108- A-1	Intake, SW- 502, 48 in		Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	8	EA	$ 2,600. 00	$ 20,800. 00

4020, 1.08, A, 1	4020 - 108- A-1	Manhole, SW-401, 48 in		Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; joint wrapping; wyes and other fittings; pipe joints; pipe connections; testing; and inspection.	8	EA	$ 2,600. 00	$ 20,800. 00
DIVISION 5 - WATER MAINS AND APPURTENANCES								
5010, 1.08, A, 1	5010 - 108- A-1	Water Main, Trenched, PVC, 8"		Trench excavation; dewatering; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; tracer system; testing; disinfection; and polyethylene wrap for ductile iron pipe and for fittings.	53 0	LF	$ 50.00	$ 26,500. 00
5010, 1.08, C, 1	5010 - 108- C-1	Fitting, PVC TEE,8"		Specify whether measurement of fittings will be made by count or by weight.	1	EA	$ 56.00	$ 56.00
5010, 1.08, C, 1	5010 - 108- C-1	Fitting, PVC Cross,8"		Specify whether measurement of fittings will be made by count or by weight.	1	EA	$ 45.00	$ 45.00
Section 5020 - Valves, Fire Hydrants, and Appurtenances								
5020, 1.08, A	5020 - 108- A-0	Valve, Gate, 8"		All components attached to the valve or required for its complete installation, including underground or above ground operator, square valve	4	EA	$ 750.00	$ 3,000.0 0

				operation nut, valve box and cover, valve box extension, and valve stem extension.				
5020, 1.08, C	5020 - 108- C-0	Fire Hydrant Assembly		The fire hydrant, barrel extensions sufficient to achieve proper bury depth of anchoring pipe and height of fire hydrant above finished grade, and components to connect the fire hydrant to the water main, including anchoring pipe, fittings, thrust blocks, pea gravel or porous backfill material, and fire hydrant gate valve and appurtenances, except tapping valve assembly if used.	2	EA	$ 1,500. 00	$ 3,000.0 0
Section 6010 - Structures for Sanitary and Storm Sewers								
6010, 1.08, A	6010 - 108- A-0	Manhole,Sanitary, 48"		Excavation; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; base; structural concrete; reinforcing steel; precast units (if used); concrete fillets; pipe connections; infiltration barriers (sanitary sewer manholes only); castings; and adjustment rings.	3	EA	$ 5,000. 00	$ 15,000. 00

6010, 1.08, A	6010 - 108- A-0	Manhole,Storm, 48"		Excavation; furnishing and installing pipe; furnishing, placing, and compacting bedding and backfill material; base; structural concrete; reinforcing steel; precast units (if used); concrete fillets; pipe connections; infiltration barriers (sanitary sewer manholes only); castings; and adjustment rings.	8	EA	$ 5,000.00	$ 40,000.00

Appendix I -Citations

1) (2020, April) How Much Do Civil Engineers Make in 2020 (including Starting Salary). [Online]. Available: https://www.owlguru.com/career/civil-engineers/salary/.[2020 September3rd]

2) Deltek Clarity. *Architecture & Engineering Industry Study 36th Annual Comprehensive Report, 201*