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Community Partners	City of Maquoketa

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Maquoketa Subdivision Prepared for the City of Maquoketa

CITY OF
MAQUOKETA
ONE OF A KIND

May 1st, 2023

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Section I Executive Summary

The design team is proud to present a development plan for a subdivision on the Stewart Family Farm's agricultural land. The consulting firm is based out of the University of Iowa located in Iowa City. The team is comprised of four civil and environmental engineering students in their last semester of undergraduate college. In the Spring of 2023, the City of Maquoketa, Iowa, tasked the team with developing a plan for 94 acres of agricultural farmland that sits southwest of the City of Maquoketa but would become part of city limits if the subdivision is to be built. The farmland is bounded by Swagosa Drive to the North, 184th Ave to the East, 19th Street to the South, and 174th to the West.

The design team has prepared a detailed project report, design drawings, construction and design cost estimate, and a presentation for the proposed subdivision to be developed on Stewart Family Farms. The design drawings consist of an overview of the site plan, street network layout with street classifications, trail network layout, water main network, storm sewer network, details sheets depicting cross-sections of streets, a grading plan, and a phasing plan. All infrastructure was designed to the specifications of the City of Maquoketa Code and Iowa Statewide Urban Design Specifications (SUDAS). All design sheets were made using AutoCAD Civil 3D and all stormwater calculations were done using Win-TR 55 and the Modified Rational Method.

The proposed subdivision provides Maquoketa with 71 high-end lots that range in size from 0.75 acres to 1.3 acres. These lots will offer large backyards and privacy from neighbors, while still being only a five-minute drive to downtown Maquoketa. Residents will have quick access to Highway 61, and Highway 64, and are within walking distance of the Maquoketa Country Club. A large recreational trail encompasses the whole subdivision and splits it down the middle providing residences with three different loops to use. Figure 1 shows this site plan.

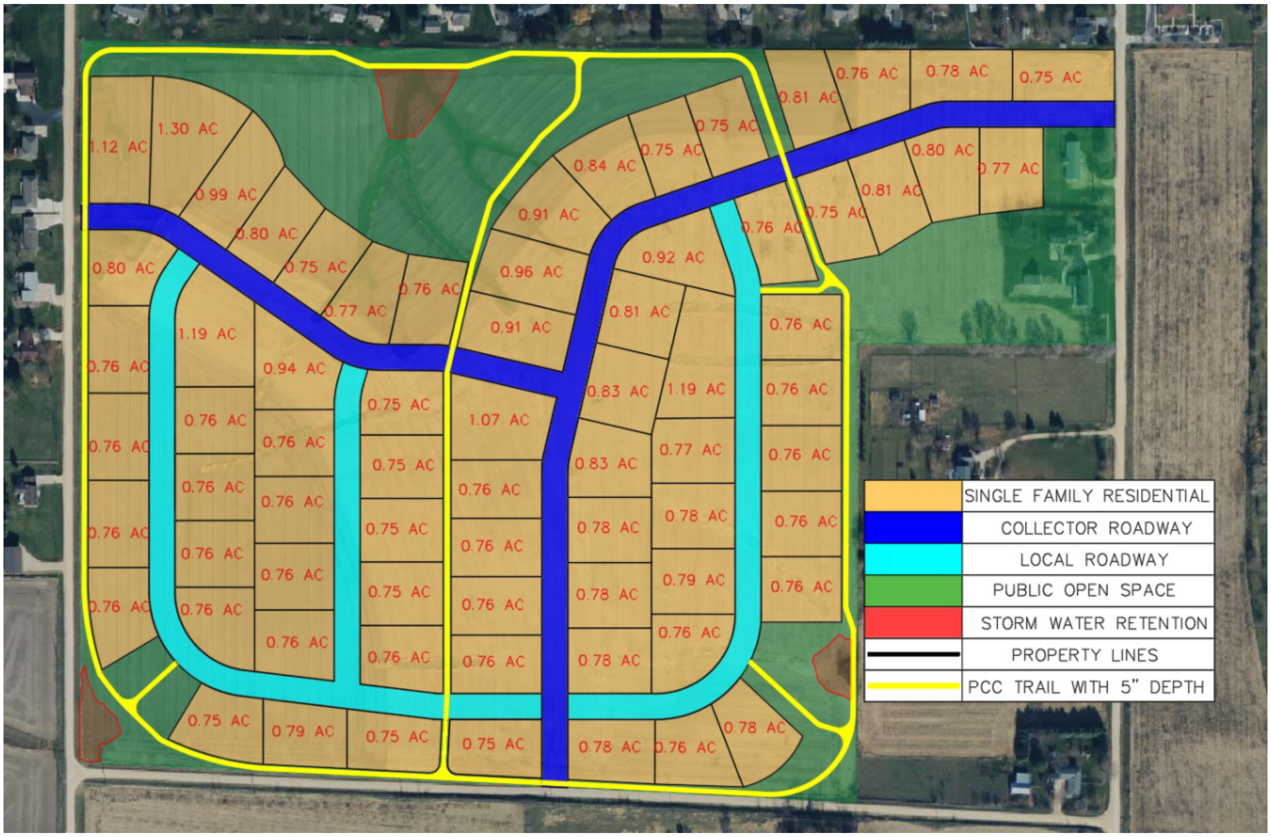


Figure 1: Proposed Subdivision

The subdivision will tie into three existing roadways: 184th Ave to the east, 19th St to the south, and 174th St to the west. All these connections will occur at the high points in their respective areas to offer the largest line of sight when trying to turn onto the existing roads. The street network for the site is shown in Figure 1 above. The subdivision will have 0.76 miles of new collector roads and 0.83 miles of local roads.

Both collector and local streets have 65' rights-of-way but offer different road layouts. Collector streets will operate with 37' of roadway width and have two lanes for through traffic and will offer on-street parking on one side of the road. A local street will have 33' of roadway with and can accommodate on-street parking on one side of the road with two lanes for through traffic. The goal with the local road was to make the pavement width slightly smaller than the collector road to deter unnecessary through traffic. Cross-sections of these two streets are shown in Figure 2 and Figure 3. Both streets will have a 2-foot PCC gutter and 6-inch wide, 6-inch-tall PCC curb on both sides of the road. A 10' space between the road and sidewalk will be used to plant trees and for snow storage. Per City of Maquoketa standard, both roads are comprised of a 3-inch hot mix asphalt (HMA) layer on top of a 10-inch base. A typical cross-section of the street is depicted in Figure 4 below.

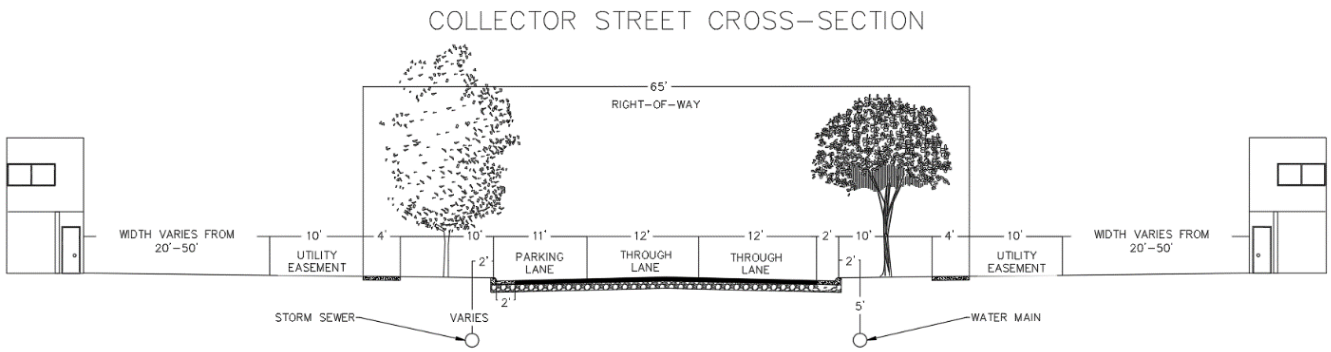


Figure 2: Typical Collector Cross-Section

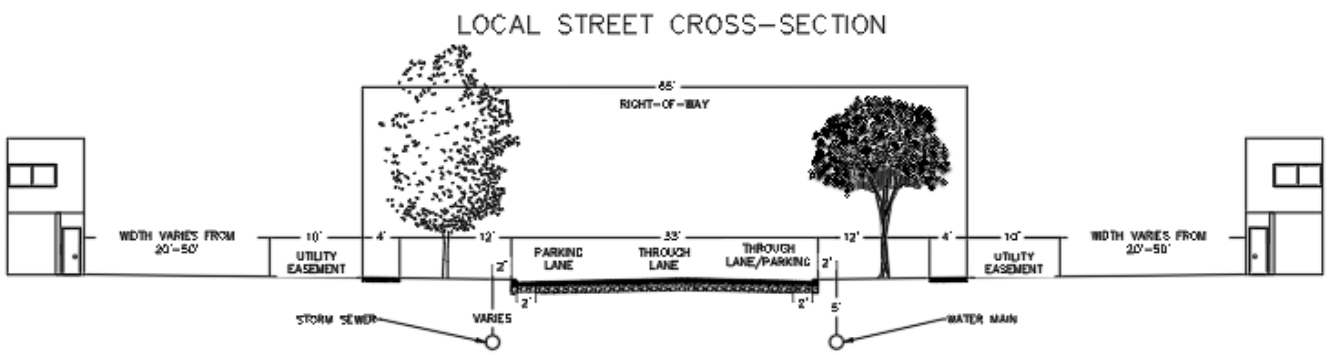


Figure 3: Typical Local Street Cross Section

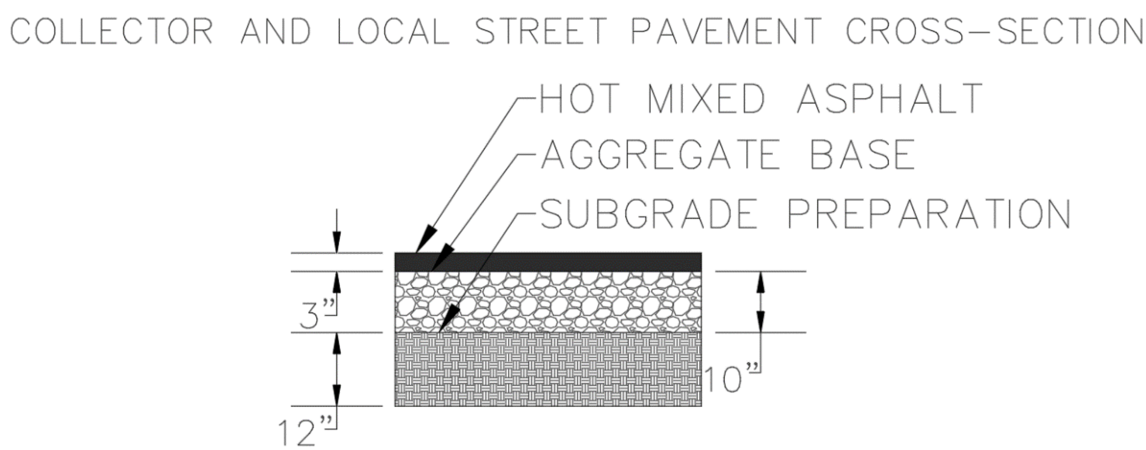


Figure 4: Collector and Local Street Pavement Cross-Section

A 1.75-mile-long recreational trail was built to offer residents a path to exercise. The full outside loop is 1.32 miles long, the west loop is 1.06 miles, and the east loop is 0.97 miles long. The goal of this design was to offer loops of different distances to allow residents to go out for a run/walk and end up back where they started without backtracking. The trail that cuts through the middle of the site offers residents quick access to the trail. The trail network can be seen in Figure 5.

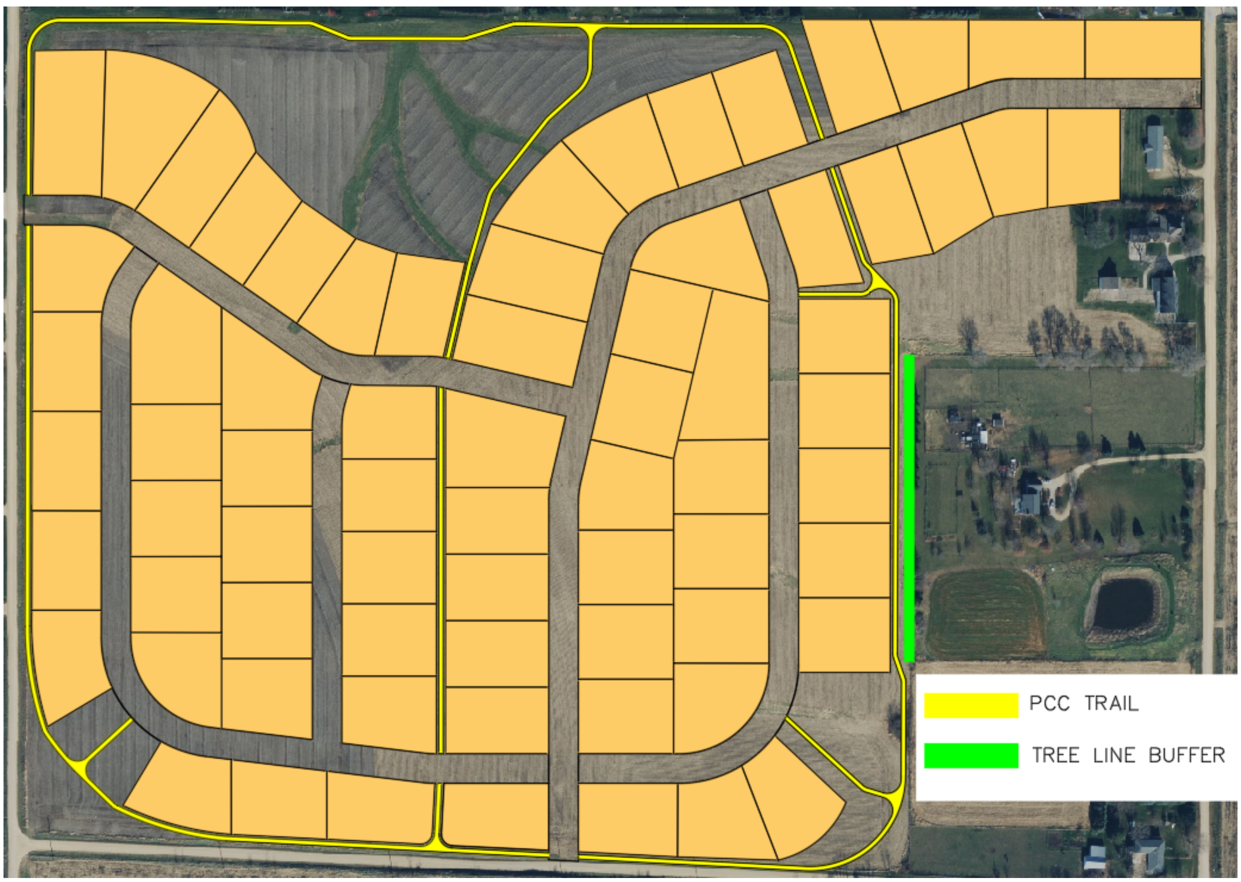


Figure 5: Trail Network

The trail has a width of 10 feet with a 2-foot buffer on each side for pedestrian safety. The trail is comprised of 5 inches of PCC on top of a compacted subbase. On the east side of the trail where it runs along an existing house, an additional 20-foot buffer was used to allow for more space between the trail users and the people that live on the existing lot. We recommend planting White Pine, a native to Iowa, as a 20-foot buffer to add additional privacy. A cross-section of the trails are shown below in Figure 6 and Figure 7.

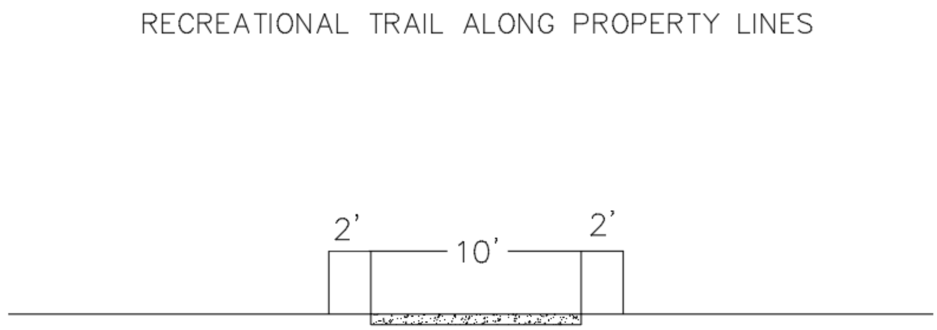


Figure 6: Typical Recreational Public Trail Cross Sections

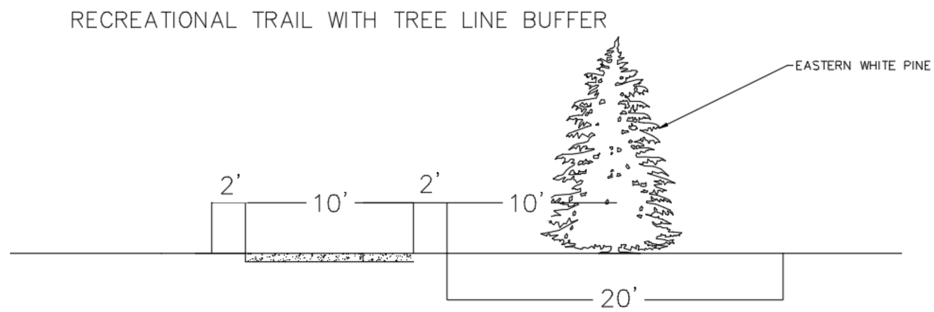


Figure 7: Typical Tree Buffer Cross-Section

The water main will connect to existing water mains along 184th St and at the end of Swagosa Dr and provide the residence of this subdivision with city water. The recommended pipe is a 10-inch PVC pipe, which is large enough to handle the flow capacity for the residents and offers better water pressure than an 8-inch pipe. Fire hydrants and water valves were placed throughout the subdivision in compliance with SUDAS standards.

After establishing a water main design, the water pressure levels at different locations throughout the neighborhood were calculated. At the highest elevation in the neighborhood, the water pressure was calculated to be between 16.1 and 19.7 psi. After referencing multiple sources, a minimum water pressure threshold for the neighborhood was set at 25 psi ("2021 INTERNATIONAL PLUMBING CODE (IPC) | ICC DIGITAL CODES"). Multiple alternatives were analyzed to address this issue, and the selected solution was to install household booster systems for homes under the 25-psi threshold. The boosters we recommend cost an estimated total of $1,800 per house.

Stormwater management for the subdivision is comprised of a storm sewer and stormwater detention ponds. The existing site drains into six different regions, and the runoff for each region had to be calculated with the pre- and post-development conditions. The result was having three storm water detention ponds to capture the excess water. A 15" concrete pipe with SW-505 intakes was used to handle flow on the streets and direct the water into the detention ponds or in ditches. Following SUDAS recommendation, the storm sewer was designed to be 2 feet from the back of the curb.

The infrastructure investment required to develop the subdivision is estimated to be $4.96 million. The cost of acquiring the land is not included in the cost estimate. The cost estimate also doesn't include a sanitary sewer system as the subdivision will be served by septic systems that will be covered by homeowners. The infrastructure cost is shown in Table 1 and all the costs for the items were taken from the Iowa DOT letting page and the Iowa Public Works Service Bureau. The construction subtotal came to a total of about $4.1 million. A 20% construction contingency was added to account for any unforeseen setbacks and problems that may arise in the future. Adding a design cost of $51,000 will bring the total to roughly $4.96 million. Taking the total cost and dividing it by the 71 lots available brings the cost of infrastructure per lot to $69,000.

Table 1 Infrastructure Cost

INFRASTRUCTURE COST ESTIMATE		
ROAD NETWORK	$	1,695,000
TRAIL NETWORK	$	437,000
WATER MAIN	$	807,000
STORM SEWER	$	857,000
SITE GRADING	$	298,000

Table 2 Infrastructure and Design Cost

INFRASTRUCTURE AND DESIGN COST ESTIMATE		
CONSTRUCTION SUBTOTAL	$	4,094,000
20% CONSTRUCTION CONTINGENCIES	$	818,800
DESIGN COST	$	51,000
TOTAL COST	$	4,964,000

Section II Organization, Qualifications, and Experience

Organization and Design Team Description

The design team is comprised of three civil engineering students and one environmental engineering student in their senior capstone design course at the University of Iowa. Kyle Patterson specializes in environmental engineering, Andrew Rohret and Jack LaDieu specialize in general civil practice, and Colin Kowalewski specializes in structural engineering.

Section III Design Services

Project Scope

The team was tasked to provide the City of Maquoketa with a single-family residential subdivision on ninety-four acres of undeveloped farmland that is bounded by Swagosa Drive, 184th Ave, 19th Street, and 174th Ave. Project scope included lot layout, street network, trail network, stormwater detention, and sanitary discharge disposal. A phasing plan will be provided with recommendations for access to sources of potable water.

Work Plan

The work schedule the Design Team followed to complete the project is summarized in Figure 8 below.

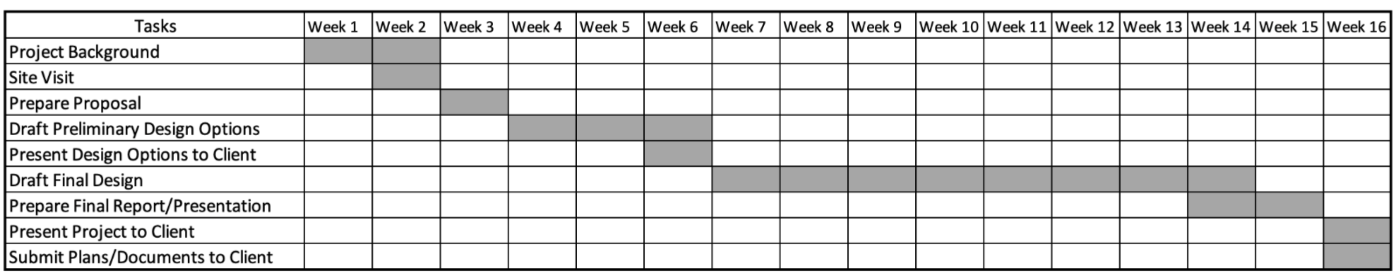


Figure 8: Project Schedule

Section IV Constraints, Challenges, and Impacts

Constraints

Grading, utilities, roadways, and trails will all cost the contractor money, and the more lots we can place on the land, the lower the cost per lot for the contractor. The existing land provided additional constraints as well.

The new subdivision will border existing houses towards the north and in the southeast. We had to make sure the additional runoff from this site didn't cause flooding issues to the existing residents. Another constraint was due to the nature of the hilly terrain; connections to existing roads could only occur in selected areas and dictate where we placed our roads. Making sure the current residents on Swagosa, 174th street and any future residents will not encounter any disturbances from runoff on the site is a priority.

Existing sanitary sewers and water main are located on the north side of the farmland and any connections to this existing infrastructure will have to be from the north and run throughout the project site.

Challenges

The hilly terrain of the existing farmland caused many challenges to building. With multiple high points and low points spread throughout the site, using a fully gravity flow sanitary sewer would be ineffective to properly drain the sanitary discharge to the sewer plant. To get the entire subdivision on a public sewer, two lift stations would have had to be installed, not a cost-effective option.

A peak site elevation of 806' is 30' higher than any other place in Maquoketa. The existing water main that will be extended into this subdivision is at about 740'. The large elevation change causes a loss in water pressure that would fall below Maquoketa standards and leave residents that live at the top of the hill with insufficient water pressure.

Societal Impact within the Community Impacts

This increase in housing lots allows for an increase in the current population of Maquoketa. The new lots will offer an attractive option for people who prefer a large lot with the convenience of city services. This may lead to current houses and nearby properties rising in value.

The project will increase the storm runoff of the current site, affecting the current runoff maintenance route. To prevent damage to the existing roadways and properties, detention ponds will be developed. Detention ponds will affect the local ecosystem near the current residential area. We have been told that there are currently drainage issues near a low point on the south side of Swagosa Dr. We have designed a detention basin that will mitigate the impacts of the new development and could potentially improve the existing drainage issues.

Section V Initial Alternative Designs

We developed multiple alternatives, each with different goals and benefits, from which to choose.

Design Alternative 1

Figure 9: Design Alternative 1

Design alternative 1 focused on aligning the roadways along the ridges of the hilly project site while trying to maximize the space for lots, with the requirement of the lots having a minimum size of 0.75 acres. With this design, the total number of lots came out to 70 lots. This design also allows for a large amount of green space that can be used for recreational use or to maintain the increase of stormwater runoff. This alternative includes a trail encircling most of the site to allow the residents to walk or bike around the neighborhood and utilize the available green space.

One of the negatives about this alternative was that the land was not fully utilized, having the fewest number of lots out of the three alternatives. Also, the collector road in the NE is very steep, meaning that a large amount of grading is needed in order to meet the maximum allowed slope of 5%.

Design Alternative 2

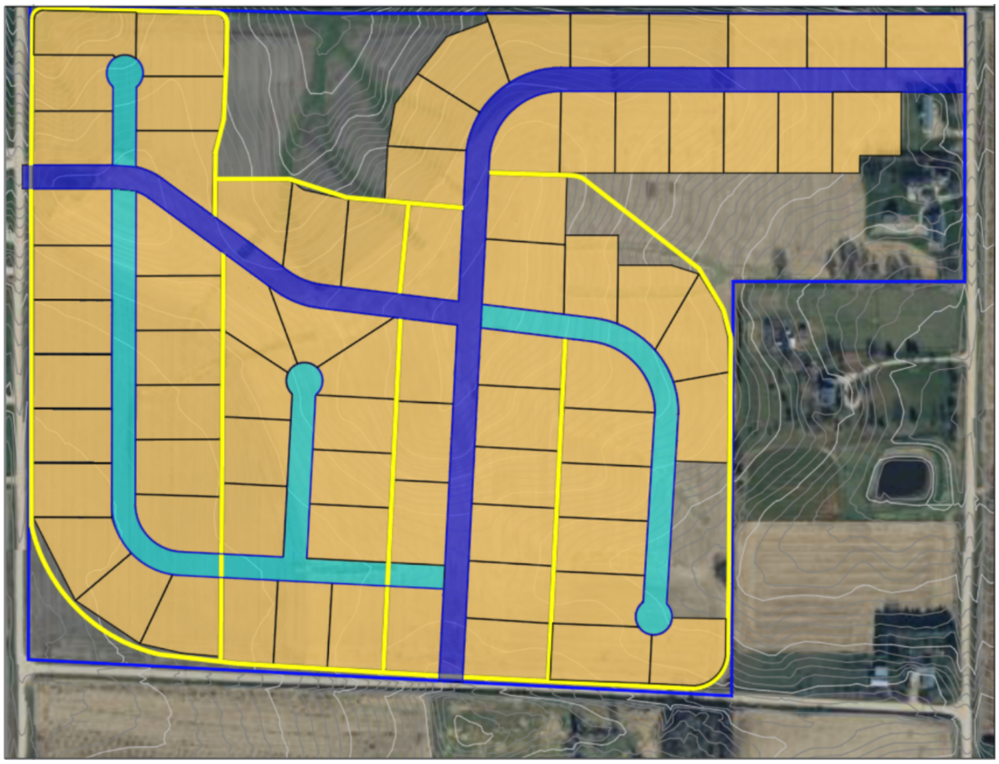


Figure 10: Design Alternative 2

Design Alternative 2 was focused on maximizing the amount of lots we could fit on the site. Seventy-eight lots ranging from 0.75 acre to 0.8 acre would be available for sale using this alternative. The use of cul-de-sacs would subdivide the development into smaller neighborhoods and would allow for a closer sense of community as well as less street traffic. Excluding the northeast section of the site, every lot has quick immediate access to the public trail that winds its way throughout the subdivision. The trail also acts as a buffer between residential lots in the subdivision and from the road on the outside of the subdivision.

Negatives to this alternative were the use of cul-de-sacs and a lack of green space. Cul-de-sacs can be expensive to maintain and adding three of them to a subdivision would not be ideal for the city. This option also means that lots are smaller and sometimes oddly shaped. There is also minimal green space on this lot design.

Design Alternative 3

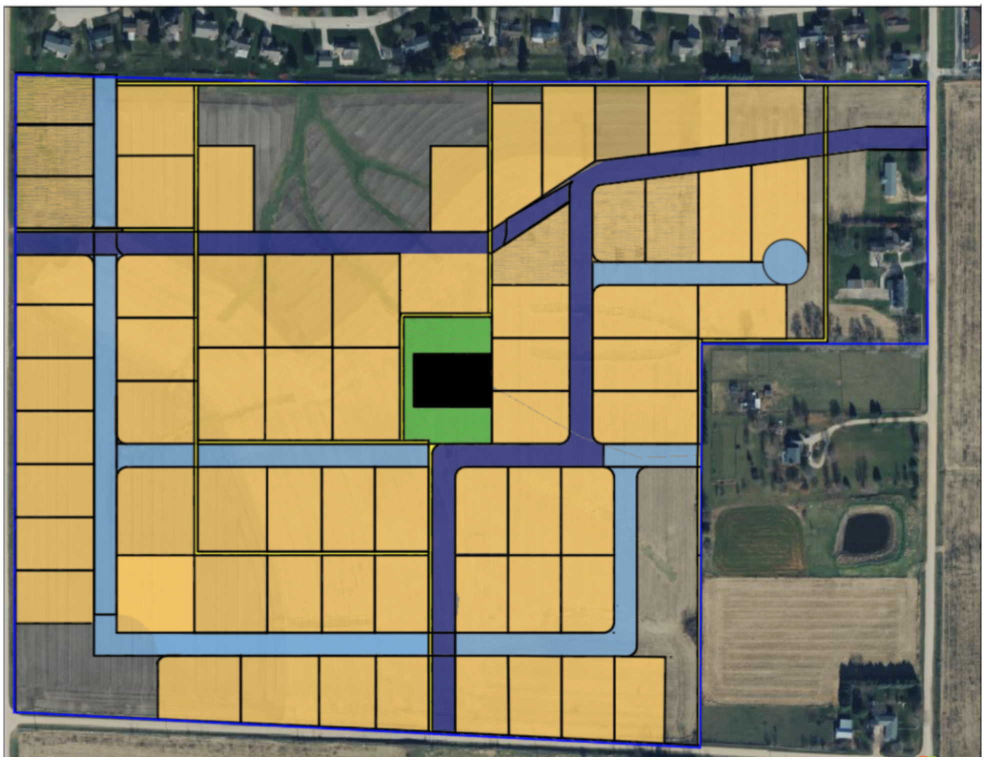


Figure 11: Design Alternative 3

This option explored areas to utilize the unique high spot in this subdivision for future municipal water storage. A water tower was considered for this, but due to the space this takes away from potential lots this was not feasible. There were no SUDAS requirements for a minimum distance from the GSR to the lots, but to allow access for necessary maintenance a green space was added around the area. This green space highlighted in a light green color on the design can also serve as a community park. Since the green space is at the highest elevation in town it could be seen as a nice overlook of the town. The design has 65 lots which is less than the other two alternatives, but this design's focus is to prepare for future developments of the expanding town. The lots range from 0.75 acres to 1 acre, per the client's request. A 10-foot trail wraps around the eastern and northern sides of the property and cuts down into the green space in the middle of the land. This gives the residents a nice trail to enjoy the scenery and easy access to the proposed park. The southwest corner, southeast corner, and northern section have additional spaces planned as easements. This was determined based off storm runoff calculations that deemed these areas as the places for stormwater storage to prevent runoff going into the resident's property.

The downside of this design is that the client loses roughly 3 lots due to the proposed green space. These lots would also presumably be the most expensive since they would be located at the highpoint of the town with a great view. The client also expressed that they wanted to keep the number of cul-de-sacs to a minimum since they are harder to maintain than a normal collector road. Another issue with this design can be seen in the southeast corner street as well as the northernmost collector road and that is the street at some points doesn't have lots on both sides. This is a negative because it does not minimize the cost of paving the road. To get the most return on investment the designers should try and have lots along the roadway on either side. The upside of this design is the future water pressure of the town, and it may alleviate future worries. The client also prefers a trail that winds through lots. This design utilizes this aspect and has multiple lots that have direct access to the trail.

Design Alternative Selection

Each option provided its unique pros and cons, but the design team felt alternative 1 was the best design. It establishes a good middle ground between large lots versus maximum lots. The flow of the street network was efficient and allows for double load the street with lots in all places not set aside for stormwater detention. The trail network has one large loop and two smaller loops within it that allow for an easy staring point and ending point for exercise.

Water Pressure Solution Alternatives

After selecting design Alternative 1, we discovered that the lots centered in the neighborhood's high point would not have adequate water pressure (Table D1 Appendix D). As a result, four solutions were investigated to address the water pressure issues caused by the high elevation of the development's large central hill: installing water pressure boosters for impacted lots; grading the top of hill; and leaving the area surrounding the highpoint undeveloped.

Booster Solution (Alt 1A)

The individual booster alternative would leave the existing design and water main intact but provides household water pressure boosters to every lot with water pressure below 25 psi at their water main connection. Sixteen lots are expected to require this booster regardless of home design, and other homes may require one if they plan to have water needs above the first floor (i.e., a 2nd floor bathroom or shower). The boosters cost approximately $900 each with an estimated additional $900 for services and installation (*Big Brand 1HP Whole House Pressure Booster*, n.d.). They have a maximum inlet pressure of 53 psi and can boost pressure up to an additional 67 psi but is capable of being adjusted to lower pressure to avoid damaging pipes from overly high pressure. Figure 12 highlights the 16 lots that are estimated to require boosters.

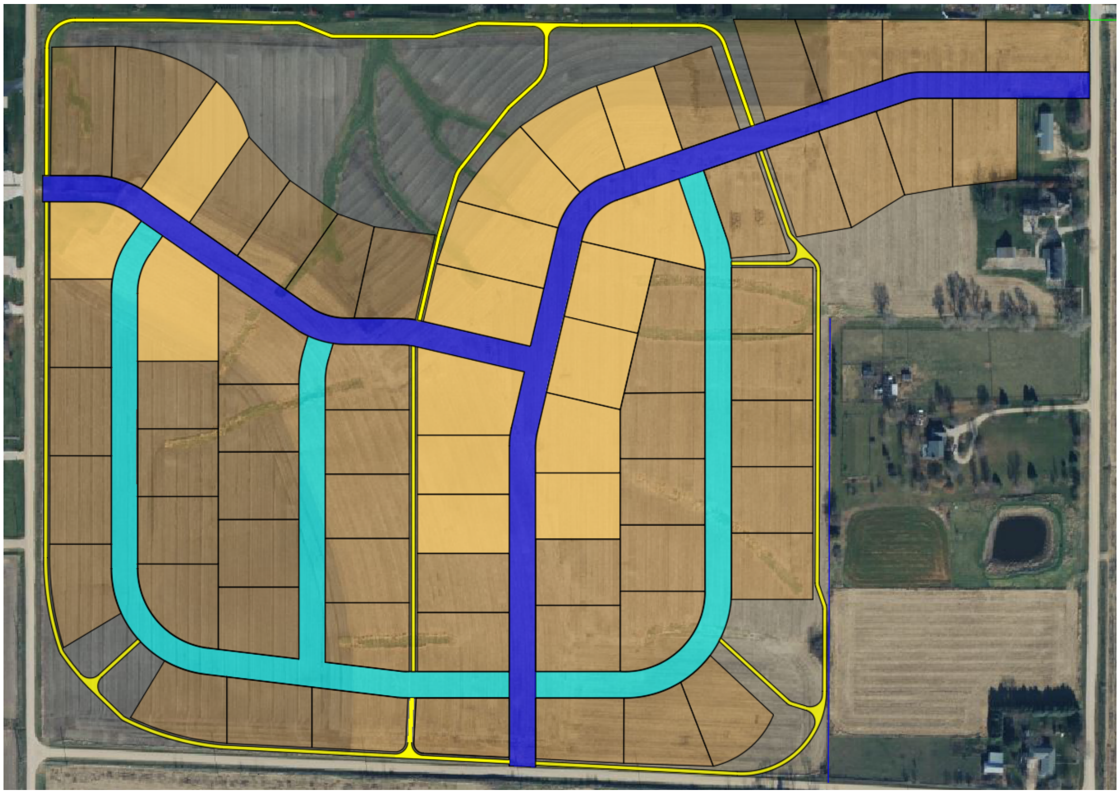


Figure 12: Individual Booster Alternative

Grading Solution (Alt 1B)

This solution involves grading the hill down 20 feet from 806’ to 786’, which will increase the psi from 16 psi to roughly 25 psi. It’s recommended to perform a boring on the land to check that soil type is appropriate for septic tanks. The total cut volume is approximately 139,000 cubic yards, which is highlighted in pink in Figure 13. At least 6 lots will still require household pressure boosters to ensure water pressure above 25 psi. Houses that develop bathrooms or other water-use appliances on a second story may need boosters outside of the yellow highlighted lots.

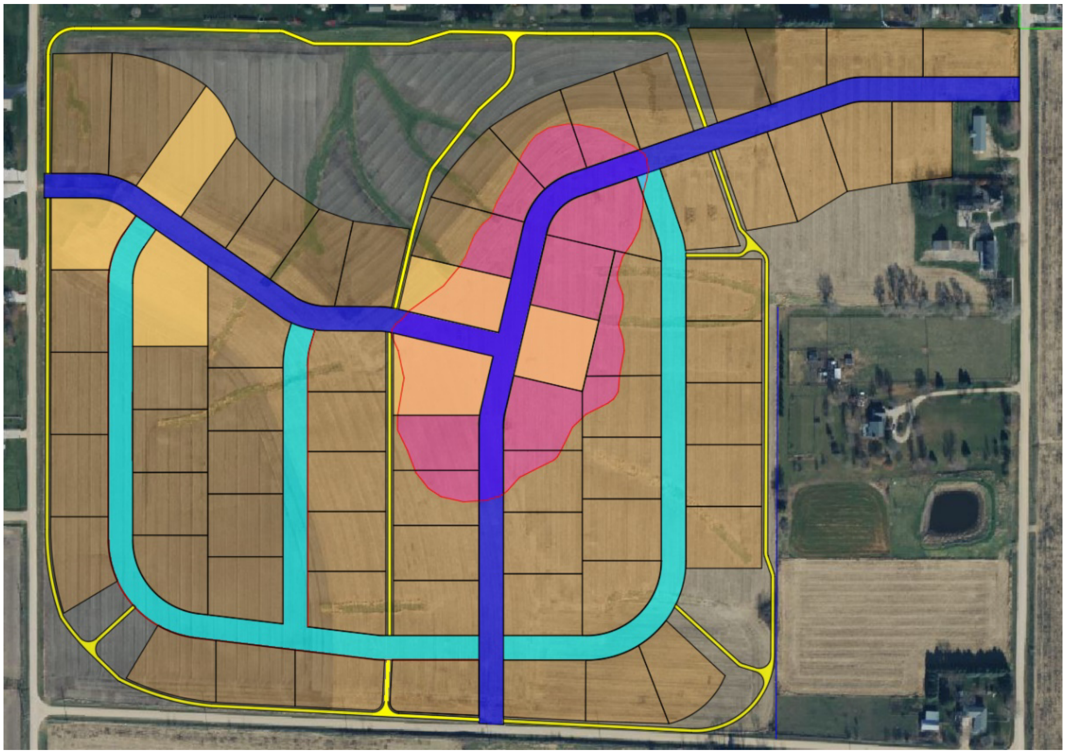


Figure 13: Grading Alternative

Another solution we considered was to not develop that section of the site. This solved the majority of the issue, but due to developing less space, 8 lots from the layout were lost. This changes the total number of lots from 72 to 64. Instead of having the main road running through the top of the hill, that road was converted into a cul-de-sac in order to add as many lots as possible. However, due to the sheer size and slope of the hill, there will be two or three houses that will have to go on individual boosters at the end of the cul-de-sac.

The benefit of this layout is that the first 8 lots entering the neighborhood on the eastern collector road are unchanged compared to the original layout. This provides design flexibility, as the developer can establish these first 8 lots either follow through with the original layout or change to this new undeveloped highpoint alternative. Since this alternative solution does not develop the top of the hill, there will be less road to pave, less piping needed for the water main and storm sewer, and more trail length.

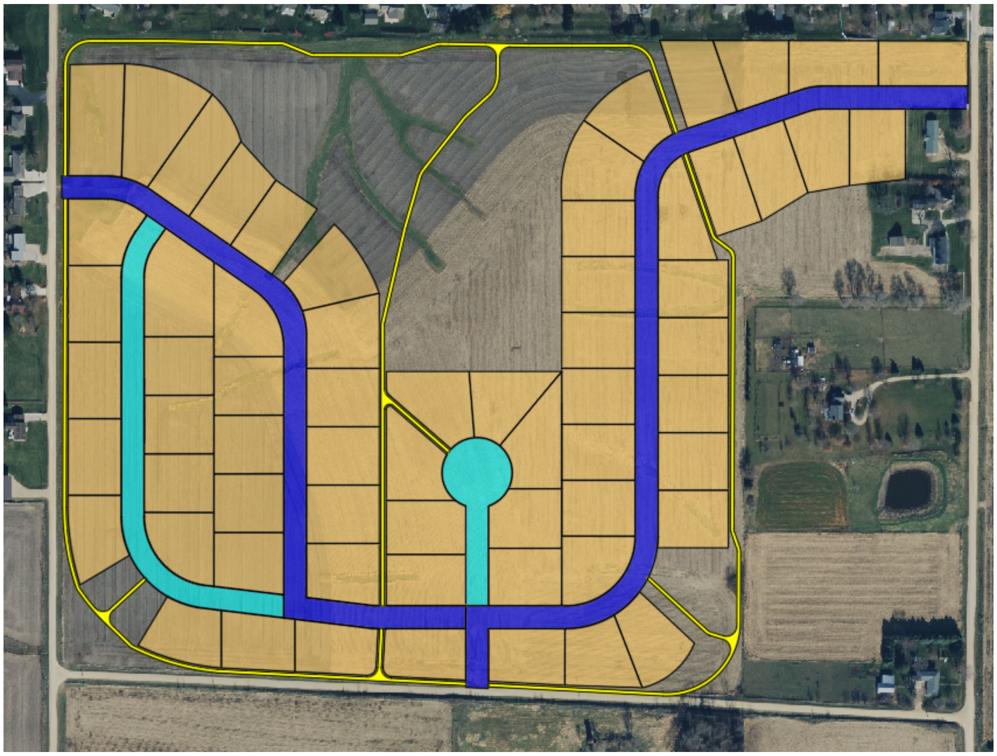


Figure 14: Undeveloped Highpoint Alternative

Well Solution (Alt 1C)

The final solution considered was to install wells for the neighborhood rather than connecting the lots to the water main, eliminating concern over water main pressure altogether. In this scenario wells would be shared between 6 lots. A well would be drilled and connected to a housing and control unit which would then pump the water to the 6 houses it serves. With 72 lots, 12 wells would be required to serve the entire development. Water main would still likely be installed in the first 8 lots of the development off of 184th street, allowing for fire hydrants in that area of the development. However, if a fire occurred outside of the Northeast lots, the fire trucks would have to fill up elsewhere.

Water Pressure Alternative Selection

The in-home booster solution is the alternative we recommend as it allows water main connections to all homes, ensures fire hydrants can be used throughout the development, avoids the removal of any lots, keeps residents on treated city drinking water, and lends itself to potential future development further south or west. As shown in Table 3, the well solution provides the least expensive solution, but we believe the value of the added benefits of staying on the water main outweigh this change in price. Additionally, the booster solution puts less of a burden on homeowners who would only have to pay for maintenance of a household booster system, rather than maintenance of a more complex private well pump and distribution system.

Table 3 Pros and Cons for Water Pressure Solution Alternatives

Features	Booster Solution (Alt 1A)	Grading Solution (Alt 1B)	Well Solution (Alt 1C)	Undeveloped Solution (Alt 2)
Water Main Cost ($)	$819,000	$819,000	$130,000	$620,000
Number of Lots	71	71	71	64
Booster Cost ($)	$28,800	$10,800	$0	$14,400
Well Drilling and Distribution Cost ($)	$0	$0	$600,000	$0
Money Lost From Undeveloped Lots ($)	$0	$0	$0	$560,000
Additional Grading Cost ($)	$0	$800,000	$0	$0
Approximate Total Cost ($)	***$847,800***	***$1,629,800***	***$730,000***	***$1,194,400***

Section VI Final Design Details

Lot Layout

This layout focuses on providing low density residential housing across much of the site with lots ranging from 0.75 to 1.30 acres. This layout, as shown in Figure 15, offers an additional 71 single-family housing units to the City of Maquoketa. If the whole subdivision was to be completed, Maquoketa would see a tax base increase of approximately $28.4 million. The tax base was estimated by taking the total number of lots (71) and multiplying it by the typical house price on a ¾ acre lot in Maquoketa ($400,000). Once the initial layout was determined, minor changes were added during the design process. One of the changes from the initial alternative was the addition of a buffer between the five houses along the east border of the subdivision to give more privacy to the current residents in that area. Another minor change was the addition of two more lots in the Southwest corner to increase the number of lots once it was determined the stormwater management would not occupy as much space as initially predicted. Finally, the lots to the northwest were increased in size to give potential residents a larger variety of lot sizes to pick from.

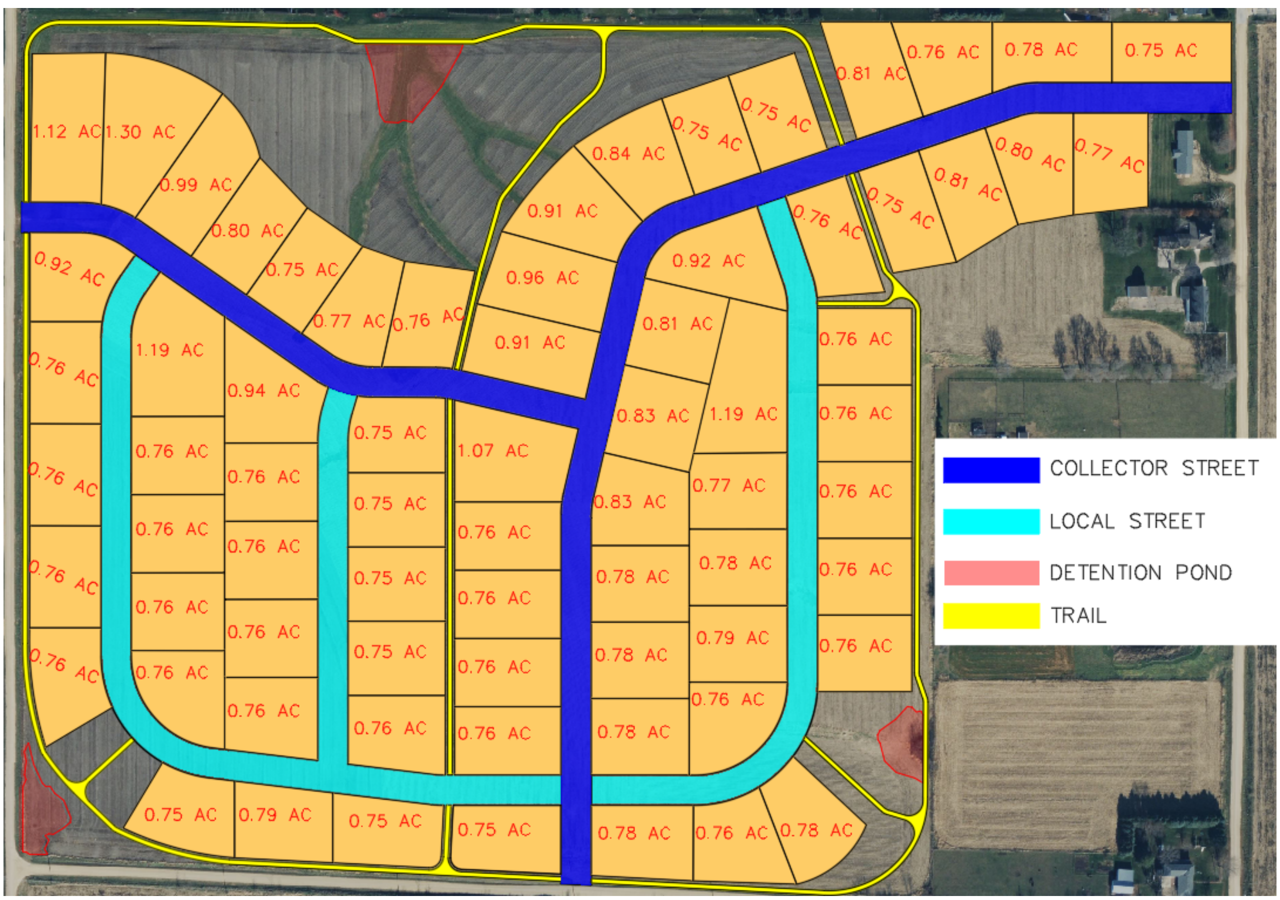


Figure 15: Final Lot Design

Table 4: Taxe Base Increase

ESTIMATED TAX BASE INCREASE	
SIGNLE-FAMILY TAX BASE INCREASE	$ 28,400,000

Street Network

The street configuration was primarily influenced by the ridges of the hills on the site. The streets follow the major ridgelines of the hill making the lots more suitable for walkout basements and backyard septic systems. The traditional grid street layout was achievable due to the drastic changes in elevation over short distances. The proposed roads would tie into existing roads in three places: 184th St to the west, 19th street to the south, and 174th to the east. All these connections will occur near the top of hills to offer the largest line of sight when trying to merge onto the existing roads.

Within this subdivision, there are two road classifications: collector and local roads. The collector roads offer a larger street width and are more suitable to handle traffic as compared to local roads. There is a collector road running through the center of the site from the connection at 184th Ave in the Northeast to 19th St in the South. There is another collector road from the connection at 174th street that connects to the other collector road in the center of the subdivision. These roads will handle all the ingress and egress of the subdivision and will see the bulk amount of traffic volume. Branching of the collector roads are local roads that serve to distribute the traffic through the subdivision and to access the individual lots. The local roads are directed toward the lower parts of the site, which helps to lead the storm water away from the lots and collector roads. Local roads will offer the lowest level of mobility and aim to be used only by the homeowners. The new subdivision will have 0.77 miles of collector roads and 0.83 miles of local roads as shown in Table 5.

The road classification, lane width, road gradient maximum and minimum, and the minimum horizontal curve radius were determined using SUDAS Chapter 5C-1 and Table 5C-1.01, which can be found in Appendix B.

Table 5: Road Classification Quantities

Road Classification	Length (Mile)
Collector	0.77
Local	0.83
Total Road Length	1.6

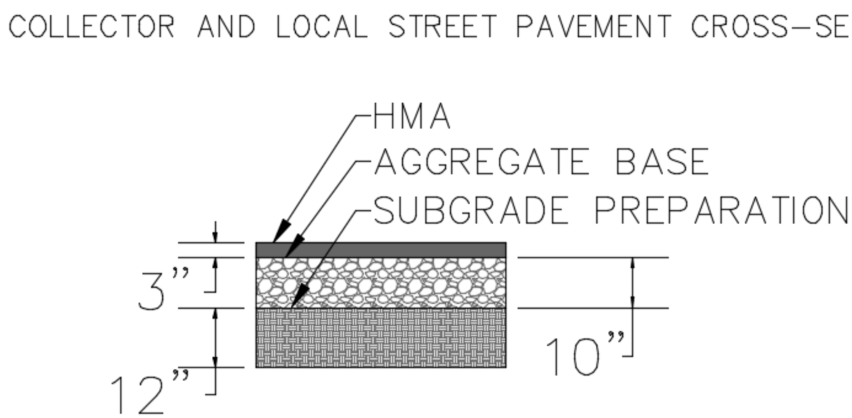


Figure 16: Collector and Local Street Pavement Cross-Section

Table 6: Pavement Thickness Design

Pavement Design			
Road Classification	Pavement Material	Pavement Thickness (in)	Base Thickness (in)
Collector	HMA	3	10
Local	HMA	3	10

The collector and local roads both have 65 feet right-of-ways. The City of Maquoketa felt the SUDAS recommended 59-foot right-of-way for a local street was too narrow and decided to have both road types have a 65-foot right-of-way for consistency. The pavement for both the roads are hot mix asphalt (HMA) with 3-inch thickness and a 10-inch subbase, per the City of Maquoketa’s standard for collector and local roads. The collector roadway consists of three lanes that are two through lanes and a parking lane. This allows for the residents of the higher traffic flow road to have room for parking outside their homes. The parking lane is 10-feet wide and the through lane is 12-feet wide, per SUDAS Table 5C-1.01: Preferred Roadway Elements recommendations (found in Appendix B Table B1). Planting of trees is recommended in the right-of-way and offer numerous benefits, including providing a traffic-calming effect when planted near the road because they appear to narrow the width of the roadway causing drivers to slow down; shading pedestrians on the sidewalk; and offering natural beauty. The local street has a width of 33 feet that offers two through lanes and one parking lane. The street width was narrowed to discourage drivers from using these roads unless they live on them. Cross-sections of the two streets are shown in Figure 17 and Figure 18 below.

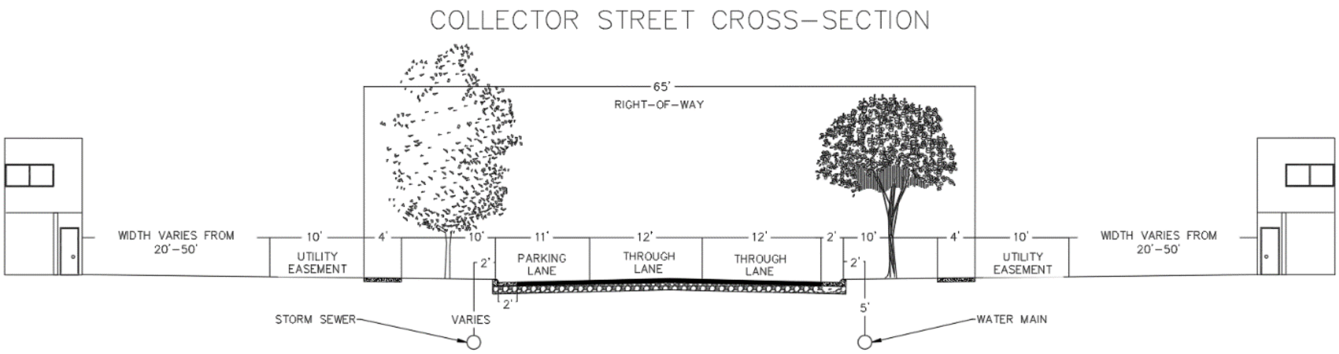


Figure 17: Typical Collector Cross-Section

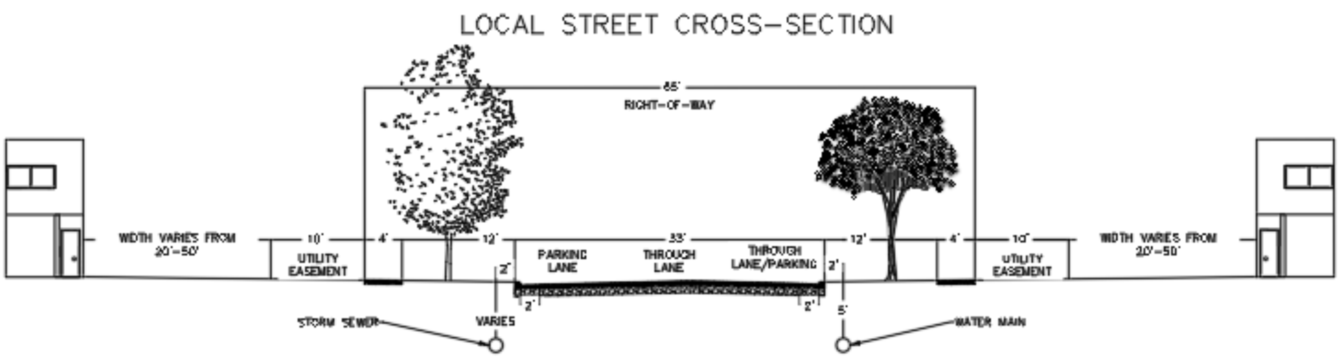


Figure 18: Typical Local Street Cross-Section

Stormwater

The stormwater management for the neighborhood is comprised of stormwater detention ponds and a stormwater sewer network. The first step in designing the detention ponds was to delineate the basins present in the neighborhood as shown in Figure 19. This was done using a topographic map in Civil 3D of the site to determine ridgelines and flow paths taken by stormwater.

Figure 19: Basins Present in the Development

Next, Win-TR55 was used to determine the curve number (CN) of the soil before and after the expected development. 85 was found to be the agricultural pre-development CN, and it was determined that the post-development CN was 80.

Then, using the CNs, the pre- and post-development time of concentrations were calculated for each of the basins shown in Table 7 by using the Modified Rational Method as recommended by the Iowa Stormwater Management Manual (ISWMM) which is found in Appendix C.

These time of concentration values were then used with the storm intensity-frequency tables in SUDAS 2B to determine the total rainfall volumetric flow pre- and post-development for the 50-year rainfall event. The difference in the total volumetric runoff pre-and post-development for each basin was then used to determine the mitigation method for this excess runoff. Basins that drained naturally to ditches were managed with overland flow routes and storm sewers which drained into the ditches. Dry-bottom detention ponds were then designed for the basins which drained into neighbors' backyards or had flow too large for ditches.

Table 7: Change in runoff flow due to development and flow reduction solution by basin

Basin Number	Basin 1	Basin 2	Basin 3	Basin 4	Basin 5	Basin 6
Pre-Development Volumetric runoff (cfs)	89.23	372.1	173.1	372.1	314.9	645.0
Post-Development Volumetric runoff (cfs)	94.81	395.4	183.9	395.4	334.6	685.3
Stormwater Management Solution	Drain to ditch	Dry bottom detention	Drain to ditch	Drain to ditch	Dry bottom detention	Dry bottom detention
Minimum detention volume to return to pre-development flow (ft^3)	N/A	4180	N/A	N/A	3510	7230

In addition to the detention basins, the development will have 7,401 feet of stormwater sewer to take in additional flow as shown in Figure 20. The proposed storm pipe will be a 15-inch class 3, concrete pipe as recommended in SUDAS 2D-1B. The concrete will prevent utility cuts through the pipe. The storm sewer will be set back 2-feet from the back of the curb as SUDAS recommended for it to be placed close to the roadway. The depth of the storm sewer will vary throughout the site, but it will maintain at least 1-foot of cover as stated in SUDAS 2D-1D. An SW-505 intake will be used in this subdivision. This is a combination intake as it has both grate intakes and a curb opening. These can intake a large amount of water, are difficult to clog, and are safe to drive over with bicycles. All intakes are to be within 500 feet of the street high point,and are spaced every 400 feet throughout the side as stated in SUDAS 2C-3G. A structure, either manhole or intake, must be at every bend in pipe. In places not near the road, we recommend using a 48" SW-401 manhole. For the sewers that discharge into the ponds, we recommend using 15" concrete aprons to outlet the stormwater into the detention ponds. A summary of all stormwater infrastructure is in Table 8.

Figure 20: Proposed Storm Sewer

Table 8: Storm Sewer Quantities

STORM SEWER QUANTITES	
LENGTH OF 12" PIPE (LF)	7401
STORM MANHOLES	34
STORM INTAKES	34
STORM OUTLET STRUCTURES	3

Water Main

The water main was designed using SUDAS Chapter 4 and City of Maquoketa Standards. The average day demand for the subdivision was calculated to be 21,600 gallons per day, 15 gpm, using SUDAS equation 4B-1.02 and Table 4b-1.01 (found in Table D1 Appendix D). The average day demand for the subdivision was calculated to be 21,600 gallons per day, or 15 gpm, using SUDAS equation 4B-1.02 and Table 4b-1.01 (found in Table D1 Appendix D). With a distance between houses of over 100 feet, the needed fire flow needs to be at least 500 gpm per SUDAS 4B-1E (found in Appendix D Table D4)m which is larger than the proposed flow in the subdivision. Calculations for the average day demand with supporting tables and equations are in Appendix D along with the fire flow demand table.

SUDAS 4B-1D requires a minimum pipe size of 8 inches in diameter, but we designed to use a 10 inch pipe that matches the existing pipe that the network is connecting to and provides with less friction losses. The pipe is to be PVC per the client's preference. The water main network along with valve and fire hydrant locations are shown in Figure 21.

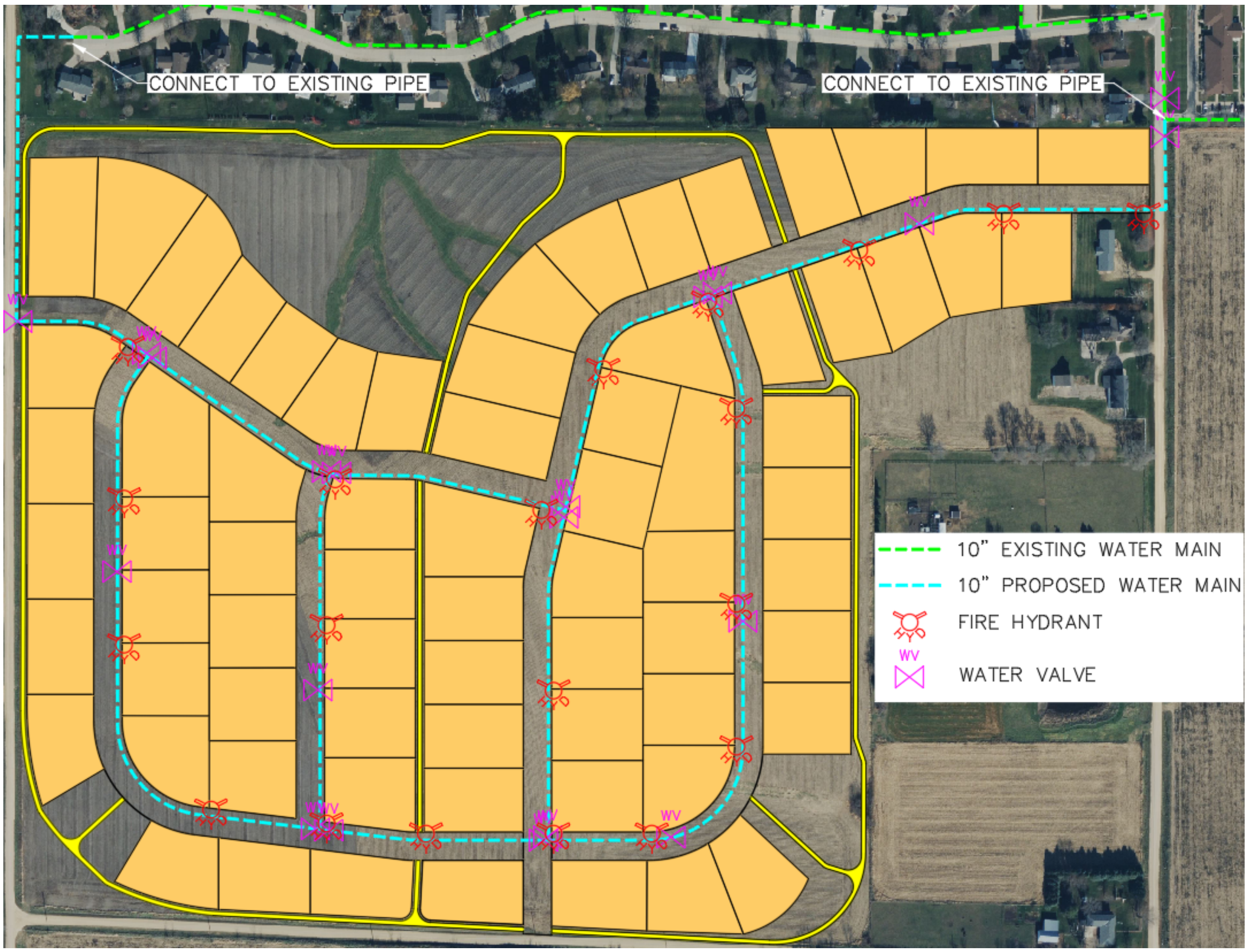


Figure 21: Watermain Network

The proposed water main network will connect to existing 10 inch infrastructure along 184th St and Swagosa Dr. This connection adds redundancy to the system as the subdivision doesn't have to rely on one pipe for water supply. Looping the water main also increases the water pressure in the network. The water main will be offset 2 feet from the back of curb and buried with 5 feet of cover ,per City of Maquoketa standards and SUDAS Figure 4C-1.01, which is located in Appendix D Table D4. Fire hydrants are spaced 25 feet from each intersection and are spaced no further than 450 feet per SUDAS 4C-1E guidelines. The fire hydrants are also placed to be near lot lines in order to minimize the chances of residents wanting to add a driveway over a hydrant. Water valves are installed at all pipe intersections, so only one unvalved pipe exists at every intersection per SUDAS 4C-1D. Valves are to be spaced no further than 800 feet apart and are not to be located on the sidewalk line or in driveways. Table 9 shows the quantities of the infrastructure needed for the water main network.

Table 9: Summary of Water Main Quantities

WATER MAIN QUANTITIES	
LENGTH OF 10" PIPE (LF)	9267
NUMBER OF VALVES	20
NUMBER OF HYDRANTS	20

Public Trail

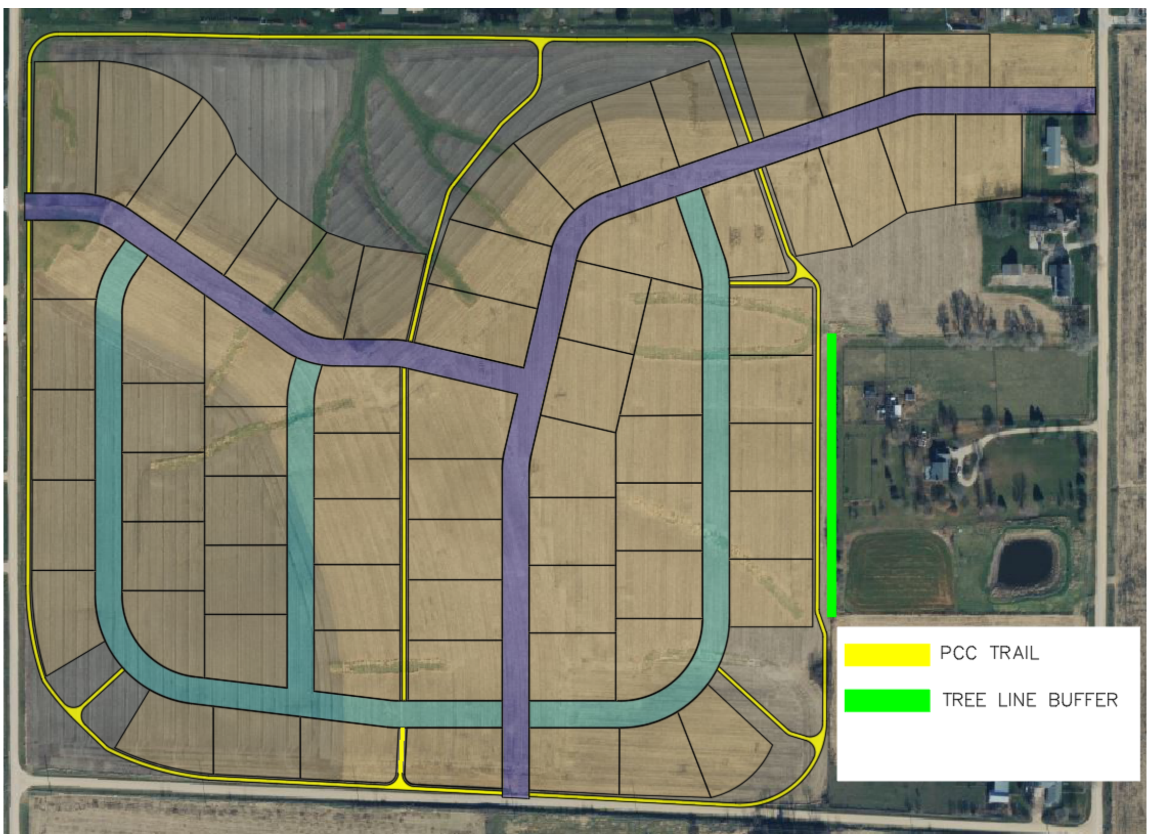


Figure 22: Recreational Trail Network

The trail system was designed to be a type 3 shared use path that's primary use is for recreation and fitness benefits. The goal of the trail is to allow the residents of the subdivision to have a two-way biking, walking, and running trail that is looped and fully contained in the subdivision. The total distance the trial covers is 1.75 miles with an outside loop distance of 1.32 miles, which can be broken into two inside loops with distances of 1.06 miles on the west and 0.97 miles on the east. The path has a width of 10’ and a clear zone of 2’ on both sides, which both fall in line with SUDAS 12B-2C guidelines and is shown in Figure 23. The trail system is designed to be 5 inches of PCC as per SUDAS 12B-2C (Figure F1-F2 in appendix F) recommendations. Along the Eastern side of the trail, there is an additional 20-foot buffer between the proposed trail and the existing houses to allow for a natural buffer of trees and shrubby to be planted, giving the current residents more privacy.

The recommended trees for the buffer are eastern white pine. This type of evergreen never loses its needles and will provide privacy year-round. The trees should be maintained so the limbs do not cross within 2-feet of the trail. The cross section of this part of the trail is shown in Figure 24. When the trail approaches one of the areas of stormwater runoff, the trail will be built on top of the berm. This will ensure that during times of high stormwater runoff, the trails will not flood or erode.

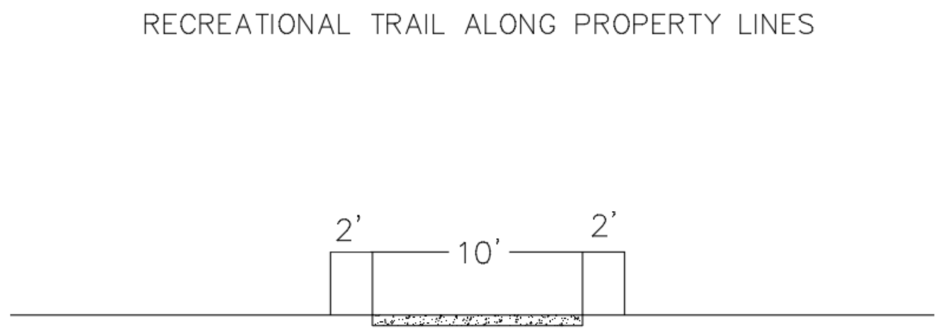


Figure 23: Typical Recreational Trail Cross-Section

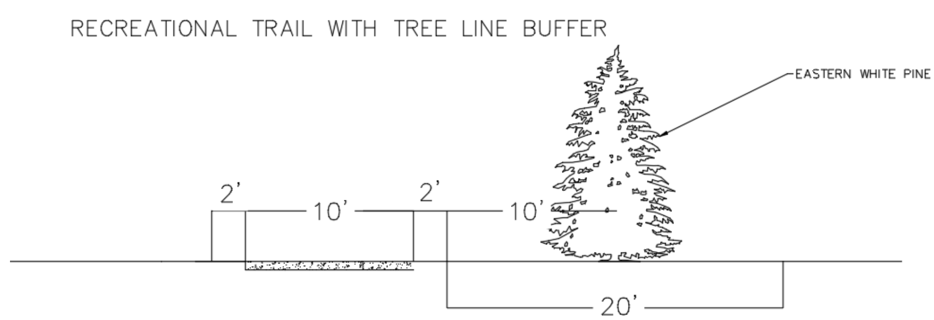


Figure 24: Typical Tree Buffer Cross-Section

Section VII Phasing

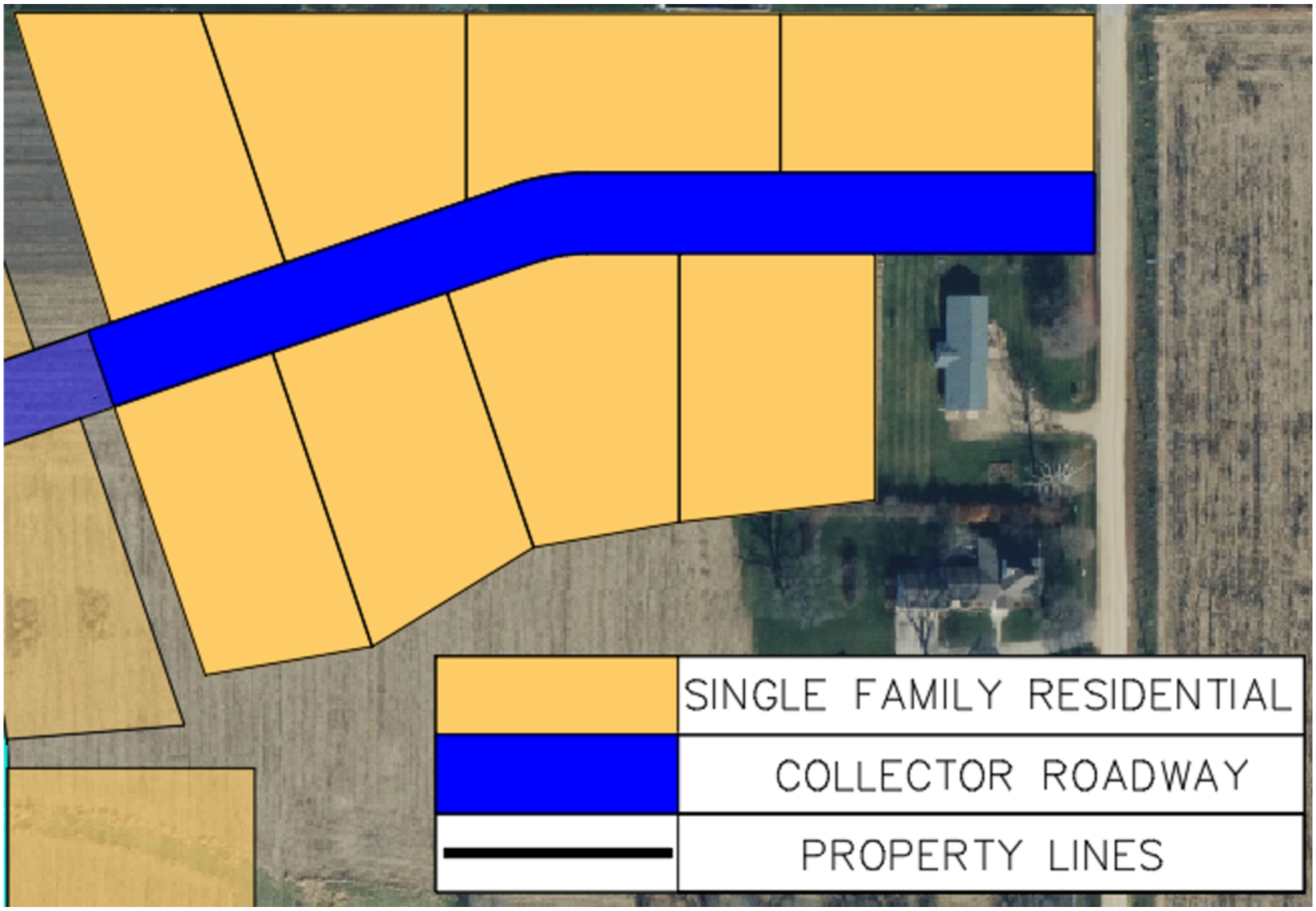


Figure 25: Phase 1 Site Design

This development can be phased if the developer wants to lower the initial investment and reduce risk while establishing if there is a market for large lots in this area. Phase 1 can be seen in Figure 25, and it accounts for the first 8 lots to the northeast. This was chosen as the first phase because these lots are the same in all design iteration which gives the city time to decide on their solution to the water pressure solution. This section is strictly cut when it comes to grading. The excess soil can be stockpiled and used to cut costs for future development.

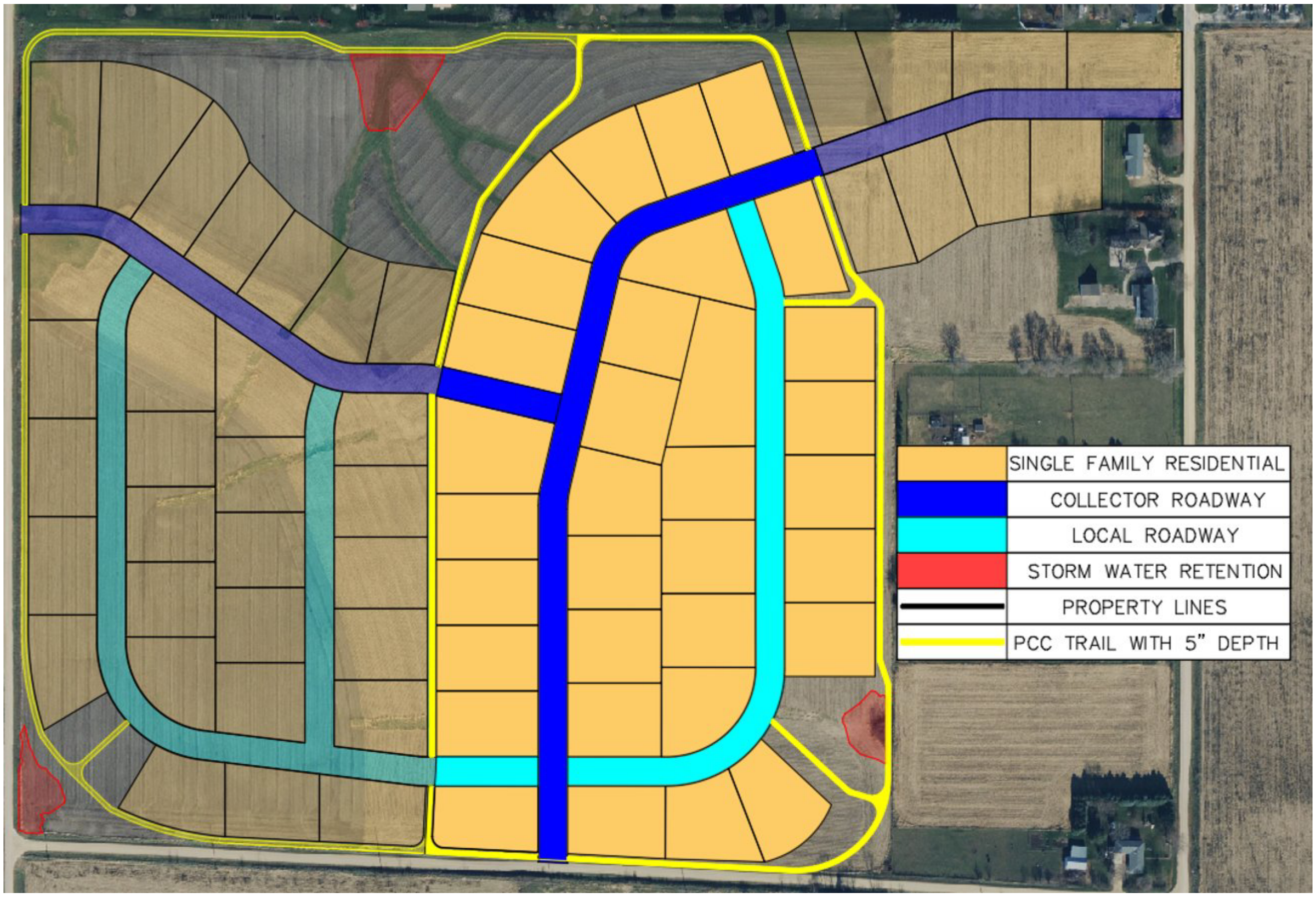


Figure 26: Phase 2 Site Design

The next phase includes 33 lots and the eastern loop of the trail. This phase includes a combination of cut and fill for the site grading. With excess soil from the development of phase one, the grading cost is significantly diminished. This phase can be seen above in Figure 26.

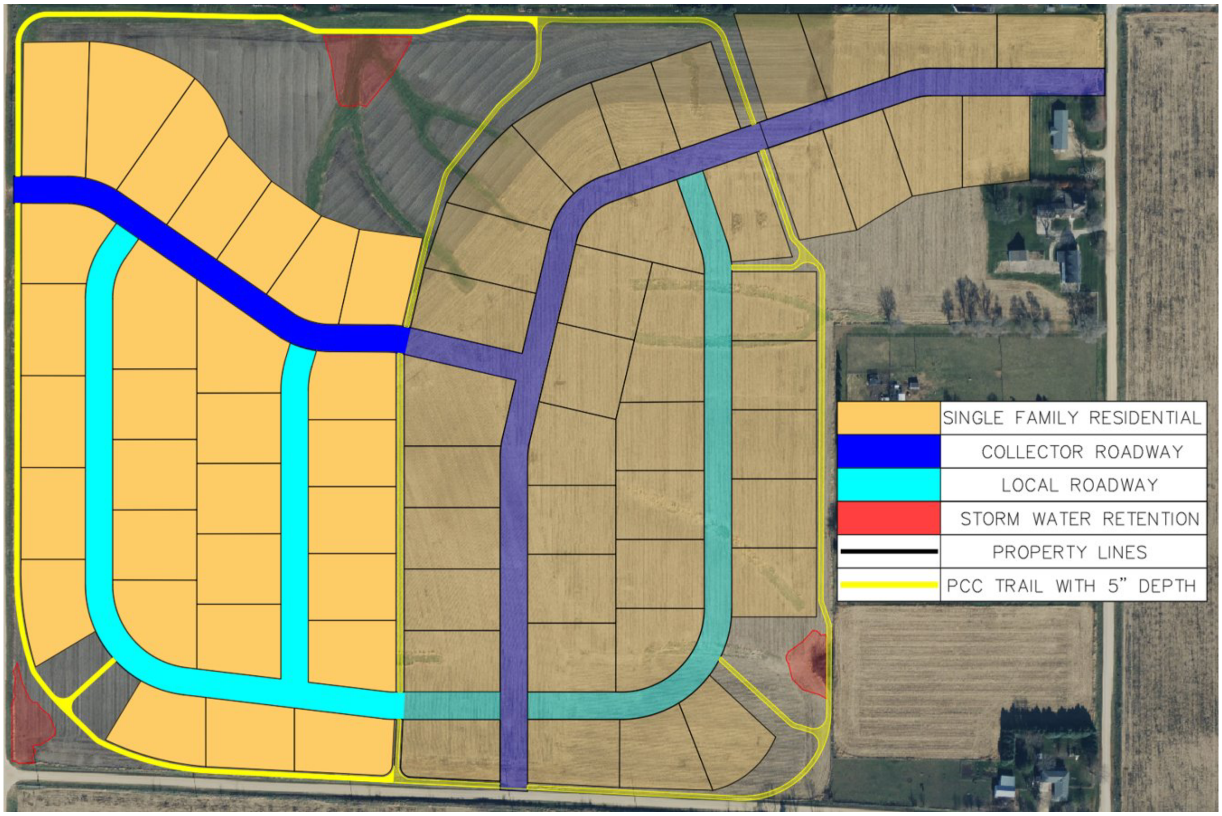


Figure 27: Phase 3 Site Design

The third phase includes the remaining 30 lots and the western loop of the trail. This section is mostly fill for the grading. Since there will be no remaining excess soil from the previous phases, soil from off site will have to be brought to the area. This will raise the cost for the grading compared to phase 2. Phase 3 can be seen in Figure 27.

Section VIII Engineer's Cost Estimate

The total estimated infrastructure cost to develop this subdivision is $4.9 million. This includes the cost of roadways, trails, storm sewers, water mains, grading, and design cost. The estimate does not include the cost to acquire the 94-acres of land. The cost estimate also doesn't include a sanitary sewer system as the subdivision will be served by septic systems that will be covered by the homeowner. The breakdown of the total cost is shown in Table 10. All unit prices were obtained from the Iowa DOT Bid Express Lettings page and the Iowa Public Works Service Bureau. For the Iowa DOT letting page, the weighted average cost of the item was the price used to estimate the cost of the specific item. There is also a 20% contingency fee added to the construction cost to account for unforeseen setbacks and problems that may arise in the future. The cost estimate follows the rounding standards by the RSMeans cost handbook.

Table 10: Cost Estimate

ENGINEER'S COST ESTIMATE

ROAD NETWORK COST ESTIMATE				
ITEM	**QUANTITY**	**UNIT**	**UNIT PRICE**	**TOTAL**
HOT MIX ASPHALT MIXTURE COMEMERCIAL MIX	4730	TON	$ 96.65	$ 457,154.50
MODIFIED SUBBASE, 10" THICK	32747	SY	$ 18.00	$ 589,440.00
CURB AND GUTTER P.C CONCRETE, 2 FT	16880	LF	$ 38.38	$ 647,854.40
			TOTAL	$ 1,694,448.90

TRAIL COST ESTIMATE				
ITEM	**QUANTITY**	**UNIT**	**UNIT PRICE**	**TOTAL**
RECREATIONAL TRAIL, PORTLAND CEMENT CONCRETE, 5 IN	10438	SY	$ 41.88	$ 437,143.44

WATER MAIN COST ESTIMATE				
ITEM	**QUANTITY**	**UNIT**	**UNIT PRICE**	**TOTAL**
WATER MAIN, TRENCHED, POLYVINYL CHROEIDE PIPE (PVC), 10 IN	9267	LF	$ 72.00	$ 667,224.00
FIRE HYDRANT ASSEMBLY, WM-201	20	EACH	$ 4,500.00	$ 90,000.00
VALVE, BUTTERFLY, DIP	20	EACH	$ 2,500.00	$ 50,000.00
			TOTAL	$ 807,224.00

STORM SEWER COST ESTIMATE				
ITEM	**QUANTITY**	**UNIT**	**UNIT PRICE**	**TOTAL**
MANHOLE, STORM SEWER, SW-401, 48 IN	4	EACH	$ 5,600.00	$ 22,400.00
INTAKE, SW-505	34	EACH	$ 5,250.00	$ 178,500.00
APRONS, CONCRETE, 15 IN. DIA.	4	EACH	$ 2,200.00	$ 8,800.00
REINFORCED CONCRETE PIPE (RCP), 2000D	7401	LF	$ 87.50	$ 647,587.50
			TOTAL	$ 857,287.50

SITE GRADING				
ITEM	**QUANTITY**	**UNIT**	**UNIT PRICE**	**TOTAL**
CLASS 10 EXCAVATION BORROW	16066	CY	$ 8.25	$ 133,000.00
EXCAVATION, CLASS 10, ROADWAY	29222	CY	$ 5.66	$ 165,000.00
			TOTAL	$ 298,000.00

INFRASTRUCTURE AND DESIGN COST ESTIMATE	
CONSTRUCTION SUBTOTAL	$ 4,094,044.86
20% CONSTRUCTION CONTINGENCY	$ 818,808.97
DESIGN COSTS	$ 51,000
TOTAL PROJECT COST	$ 4,964,000

Table 11: Phase 1 Cost

INFRASTRUCTURE COST ESTIMATE		
ROAD NETWORK	$	177,000
WATER MAIN	$	97,000
STORM SEWER	$	83,000
SITE GRADING	$	79,000
TOTAL COST	$	436,000

Table 12: Phase 2 Cost

INFRASTRUCTURE COST ESTIMATE		
ROAD NETWORK	$	707,000
TRAIL NETWORK	$	261,000
WATER MAIN	$	289,000
STORM SEWER	$	315,000
SITE GRADING	$	73,000
TOTAL COST	$	1,645,000

Table 13: Phase 3 Cost

INFRASTRUCTURE COST ESTIMATE		
ROAD NETWORK	$	809,000
TRAIL NETWROK	$	176,000
WATER MAIN	$	414,000
STORM SEWER	$	422,000
SITE GRADING	$	146,000
TOTAL COST	$	1,967,000

Section IV Appendices

Appendix A: Maquoketa Land Use Regulations

TITLE V LAND USE REGULATIONS

SUBCHAPTER 1D "R-1" RESIDENTIAL DISTRICT

5-1D-1 "R-1" DISTRICT REGULATIONS:

1. The regulations set forth in this Chapter or set forth elsewhere in this Title, when referred to in this Chapter, are the regulations in the "R-1" Residential District.

5-1D-2 USE REGULATIONS:

1. A building or premises shall be used only for the following purposes:

a. Single family dwellings.

b. Two (2) family dwellings.

c. Churches.

d. Public buildings, parks, playgrounds, community center, and recreational vehicle campsites in City Parks as designated by Council Resolution.

(Ord. 773, 1-6-92)

e. Public schools, elementary and high, and private education institutions having a curriculum the same as ordinarily given in public schools, and having no rooms regularly used for housing and sleeping rooms.

f. Home occupations.

g. Golf courses, except miniature courses or practice driving tees operated for commercial purposes.

h. Temporary buildings, the uses of which are incidental to the construction operations or sale of lots during development being conducted on the same or adjoining tract or subdivision and which shall be removed upon completion or abandonment of such construction, or upon the expiration of a period of two (2) years from the time of erection of such temporary buildings, whichever is sooner.

i. Cemetery or mausoleum on sites not less than twenty (20) acres.

j. Signs: Refer to the Subchapter 1O, Signs.

k. Accessory buildings and uses including, but not limited to, accessory private garages, swimming pools, home barbecue grills, accessory storage, and accessory off street parking and loading space.

5-1D-3 PARKING REGULATIONS:

1. Off street parking spaces shall be provided in accordance with the requirements for specific uses set forth in Subchapter 1L.

5-1D-4 HEIGHT REGULATIONS:

1. No building shall exceed two and one-half (2 1/2) stories nor shall it exceed thirty-five (35') feet except as provided in Subchapter 1K.

5-1D-5 AREA REGULATIONS:

1. Yard Regulations. Subject to the modifications set out in Subchapter 1K, the regulations are as follows:

a. Front Yard. There shall be a front yard of not less than thirty (30') feet.

b. Side Yard. There shall be a side yard on each side of a lot of not less than seven feet (7').

c. Rear Yard. There shall be a rear yard of not less than thirty feet (30').

d. Front Porch Reconstruction.

e. If a residence was constructed prior to January 1, 1964, with a front porch that does not comply with the front yard or side yard setback requirements, then the front porch may be rebuilt provided that the overall square footage of the porch is not increased and the existing nonconforming front and side yard setbacks are not decreased.

2.. Minimum Lot Area.

a. A lot occupied by a single family dwelling shall contain not less than seven thousand two hundred (7,200) square feet and shall not be less than sixty feet (60') in width.

b. A lot occupied by a two (2) family dwelling shall contain not less than nine thousand (9,000) square feet and shall not be less than seventy-five feet (75') in width.

Appendix B: Street and Lot Layout

Table B1: SUDAS Table 5C-1.01: Preferred Roadway Elements

Table 5C-1.01: Preferred Roadway Elements

Elements Related to Functional Classification

Design Element	Local		Collector		Arterial	
	Res.	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*
General						
Design level of service[1]	D	D	C/D	C/D	C/D	C/D
Lane width (single lane) (ft)[2]	10.5	12	12	12	12	12
Two-way left-turn lanes (TWLTL) (ft)	N/A	N/A	14	14	14	14
Width of new bridges (ft)[3]	See Footnote 3					
Width of bridges to remain in place (ft)[4]	----------	----------	----------	----------	----------	----------
Vertical clearance (ft)[5]	14.5	14.5	14.5	14.5	16.5	16.5
Object setback (ft)[6]	3	3	3	3	3	3
Clear zone (ft)	Refer to Table 5C-1.03, Table 5C-1.04, and 5C-1, C, 1					
Urban						
Curb offset (ft)[7]	2	2	2	3	3	3
Parking lane width (ft)	8	8	8	10	N/A	N/A
Roadway width with parking on one side[8]	26/27/31[9]	34	34	37	N/A	N/A
Roadway width without parking[10]	26	31	31	31	31	31
Raised median with left-turn lane (ft)[11]	N/A	N/A	19.5	20.5	20.5	20.5
Cul-de-sac radius (ft)	45/48[12]	45/48[12]	N/A	N/A	N/A	N/A
Rural Sections in Urban Areas						
Shoulder width (ft)						
ADT: under 400	4	4	6	6	10	10
ADT: 400 to 1,500	6	6	6	6	10	10
ADT: 1,500 to 2000	8	8	8	8	10	10
ADT: above 2,000	8	8	8	8	10	10
Foreslope (H:V)	4:1	4:1	4:1	4:1	6:1	6:1
Backslope (H:V)	4:1	4:1	4:1	4:1	4:1	4:1

Res. = Residential, C/I = Commercial/Industrial

Elements Related to Design Speed

Design Element	Design Speed, mph[13]							
	25	*30*	*35*	*40*	*45*	*50*	*55*	*60*
Stopping sight distance (ft)	155	200	250	305	360	425	495	570
Passing sight distance (ft)	900	1090	1,280	1,470	1,625	1,835	1,985	2,135
Min. horizontal curve radius (ft)[14]	198	333	510	762	1,039	926	1,190	1,500
Min. vertical curve length (ft)	50	75	105	120	135	150	165	180
Min. rate of vertical curvature, Crest (K)[15]	18	30	47	71	98	136	185	245
Min. rate of vertical curvature, Sag (K)	26	37	49	64	79	96	115	136
Minimum gradient (percent)	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Maximum gradient (percent)	5	5	5	5	5	5	5	5

Note: For federal-aid projects, documentation must be provided to explain why the preferred values are not being met. For non-federal aid projects, the designer must contact the Jurisdiction to determine what level of documentation, if any, is required prior to utilizing design values between the "Preferred" and "Acceptable" tables.

Table B2: SUDAS Table 5C-2.02: Preferred Border Area

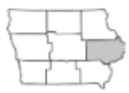
Table 5C-2.02: Preferred Border Area

Street Classification	Border Area Width (feet)
Major/minor arterial	16
Collector	14.5
Local streets	14

Appendix C: Stormwater

Stormwater Runoff Calculations

Table 2B-2.07: Section 6 - East Central Iowa
Rainfall Depth and Intensity for Various Return Periods

	**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	4.56	0.44	5.30	0.54	6.56	0.63	7.65	0.76	9.18	0.86	10.3	0.97	11.6	1.23	14.8
10 min	0.55	3.33	0.64	3.87	0.8	4.8	0.93	5.58	1.11	6.70	1.26	7.60	1.42	8.54	1.80	10.8
15 min	0.67	2.70	0.78	3.14	0.97	3.88	1.13	4.53	1.36	5.45	1.54	6.18	1.73	6.94	2.20	8.81
30 min	0.95	1.90	1.11	2.22	1.38	2.76	1.61	3.22	1.94	3.88	2.20	4.40	2.47	4.95	3.14	6.29
1 hr	1.23	1.23	1.44	1.44	1.80	1.80	2.11	2.11	2.58	2.58	2.96	2.96	3.36	3.36	4.37	4.37
2 hr	1.51	0.75	1.77	0.88	2.22	1.11	2.62	1.31	3.22	1.61	3.71	1.85	4.24	2.12	5.60	2.80
3 hr	1.68	0.56	1.96	0.65	2.47	0.82	2.93	0.97	3.63	1.21	4.22	1.40	4.85	1.61	6.50	2.16
6 hr	1.97	0.32	2.30	0.38	2.89	0.48	3.45	0.57	4.3	0.71	5.02	0.83	5.8	0.96	7.87	1.31
12 hr	2.28	0.19	2.65	0.22	3.31	0.27	3.93	0.32	4.88	0.40	5.68	0.47	6.56	0.54	8.87	0.73
24 hr	2.60	0.10	3.01	0.12	3.75	0.15	4.42	0.18	5.44	0.22	6.29	0.26	7.22	0.30	9.64	0.40
48 hr	2.98	0.06	3.43	0.07	4.22	0.08	4.93	0.10	6.01	0.12	6.90	0.14	7.86	0.16	10.3	0.21
3 day	3.28	0.04	3.72	0.05	4.51	0.06	5.24	0.07	6.32	0.08	7.22	0.10	8.19	0.11	10.7	0.14
4 day	3.53	0.03	3.98	0.04	4.78	0.04	5.50	0.05	6.58	0.06	7.49	0.07	8.46	0.08	10.9	0.11
7 day	4.17	0.02	4.67	0.02	5.53	0.03	6.29	0.03	7.39	0.04	8.30	0.04	9.25	0.05	11.6	0.06
10 day	4.75	0.01	5.30	0.02	6.24	0.02	7.04	0.02	8.20	0.03	9.12	0.03	10.0	0.04	12.4	0.05

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

A. Introduction

The time of concentration (T_c) is used in numerous equations to calculate discharge, particularly with the Rational method, WinTR-55, and WinTR-20. In most watersheds, it is necessary to add the many different time of concentrations resulting from different field conditions that runoff flows through to reach the point of investigation. Water moves through a watershed as sheet flow, shallow concentrated flow, swales, open channels, street gutters, storm sewers, or some combination of these. This section describes the many conditions and corresponding solutions that need to be considered when estimating the total time of concentration (T_c) (sum of runoff travel time).

There are also many methods utilized to estimate the time of concentration. Examples are the Kinematic Wave Method, Kirpich formula, Kerby formula, and the NRCS Velocity Method. The NRCS Velocity Method is one of the most common, is easily understood, has continuity with many computer programs, and is considered as accurate as other methods. It is for these reasons the NRCS Velocity Method is used in this manual. If there is a desire to use a different method in determining the time of concentration, the Engineer needs to be contacted for approval.

B. Definition

The time of concentration is defined as the time required for water falling on the most remote point of a drainage basin to reach the outlet where remoteness relates to time of travel rather than distance. Probably a better definition is that it is the time after the beginning of rainfall excess when all portions of the drainage basin are contributing simultaneously to flow at the outlet.

Using an appropriate value for time of concentration is very important, although it is hard sometimes to judge what the correct value is.

The time of concentration is often assumed to be the sum of two travel times (T_t). The first is the initial time required for the overland flow, and the second is the travel time in the conveyance elements (open channels, street gutters, storm sewers, etc).

C. Factors affecting time of concentration

1. **Surface roughness.** One of the most significant effects of urban development on flow velocity is a decrease in retardance to flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by urban development; the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

2. **Channel shape and flow patterns.** In small non-urban watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

3. **Slope.** Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the water management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

D. Estimating time of concentration (NRCS velocity method)

1. **Travel time.** Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

   T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing the peak discharge. But T_c can be increased as a result of:

- Ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts
- Reduction of land slope through grading
- Lengthening the flow path
- Decreasing the impervious area and/or reducing the directly connected impervious area in the catchment

Travel time (T_t) is the ratio of flow length to flow velocity:

Equation C3-S3- 1

$$T_t = \frac{L}{3600V}$$

Where:
T_t = travel time (hours)
L = flow length (ft)
V = average velocity (ft/s)
3600 = conversion factor from seconds to hours

Time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments:

Equation C3-S3- 2

$$\boldsymbol{T_c = T_{t1} + T_{t2} + T_{t3} \dots\dots T_{tm}}$$

Where:
T_c = time of concentration (hr)
T_t = travel time for a flow component m = number of flow segments

2. **Sheet flow.** Sheet flow is flow over plane surfaces (parking lots, farm fields, lawns). It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of rain drop impact; drag over the plane surface; obstacles such as litter, vegetation, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot. Table C3-S3- 1 gives Manning's n values for sheet flow for various surface conditions.

For sheet flow of less than 100 feet, use Manning's kinematic solution (Overton and Meadows, 1976) to compute T_t;

Equation C3-S3- 3

$$T_t = \frac{0.007[(n)(L)]^{0.8}}{\sqrt{P_2}S^{0.4}}$$

Where:
T_t = travel time (hours)
n = Manning's roughness coefficient (Table C3-S3- 2)

L = flow length (ft)
P_2 = the 2-year, 24-hour rainfall (inches)
S = slope of hydraulic grade line (land slope, ft/ft)

This simplified form of Manning's kinematic solution is based on the following:

- Shallow steady uniform flow
- Constant intensity of rainfall excess (that part of a rain available for runoff)
- Rainfall duration of 24 hours
- Minor effect of infiltration on travel time

Table C3-S3- 1: Roughness coefficients (Manning's n) for sheet flow

Surface Description	n[1]
Smooth surfaces (concrete, asphalt, gravel or bare soil)	0.011
Fallow (no residue	0.05
Cultivated Soils:	
Residue cover <20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods[3]:	
Light underbrush	0.40
Dense underbrush	0.80

[1]The n values are a composite of information compiled by Engman (1986).
[2]Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
[3]When selecting n, consider cover to a height of about 0.1ft. This is the only part of the plant cover that will obstruct sheet flow.

3. **Shallow concentrated flow (urban/suburban areas).** After a maximum of 100 feet, sheet flow (gutter, swales, etc.) usually becomes shallow concentrated flow. The average velocity (V) for this flow can be determined from Figure C3-S3- 1, in which average velocity is a function of watercourse slope and type of channel surface. For slopes less than 0.005 ft/ft, use equations given below for Figure C3-S3- 1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope. After determining average velocity in Figure C3-S3- 1, use Equation C3-S3- 4 to estimate travel time for the shallow concentrated flow segment.

   Figure C3-S3- 1 (average velocities for estimating travel time for shallow concentrated flow):
   Unpaved $V = 16.1345(s)0.5$
   Paved $V = 20.3282(s)0.5$

   Where:
   V= average velocity (ft/s)
   s = slope of hydraulic grade line, (watercourse slope, ft/ft)

   These two equations are based on the solution of Manning's equation (Equation C3-S3- 4) with different assumptions for *n* (Manning's roughness coefficient) and *r* (hydraulic radius, ft). For unpaved areas, *n* is 0.05 and r is 0.4; for paved areas, *n* is 0.025 and *r* is 0.2.

Tillage and vegetation surfaces can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope. After determining average velocity (V) in Figure C3-S3- 1
Figure C3-S3- 1, use Equation C3-S3- 1 to estimate travel time for the shallow concentration flow segment.

Equation C3-S9- 1

$$q_{pi} = CiA$$

Where:
q_{pi} = peak discharge (peak inflow rate for the detention basin)
C = runoff coefficient
i = rainfall intensity (in/hr) A = area of the watershed, ac

It is assumed the peak of the outflow hydrograph falls on the recession limb of the inflow hydrograph (see Figure C3-S9-3), and the rising limb of the outflow hydrograph can be approximated by a straight line. With these assumptions (Aron and Kibler, 1990):

Equation C3-S9- 2

$$S_d = q_{pi}t_d - \frac{Q_a(t_d + t_c)}{2}$$

Where:
S_d = detention volume required
Q_a = allowable peak outflow rate
t_d = design storm duration
t_c = time of concentration for the watershed

The design storm duration is that duration that maximizes the detention storage volume, S_d, for a given return period. The storm duration can be found by trial and error using local I-D-F data (or extracted from the rainfall data in Chapter 3, section 2).

Figure C3-S9- 3 provides an illustration. The rising and falling limbs of the inflow hydrograph have a duration equal to the time of concentration (T_c). An allowable target outflow is set based on pre-development conditions. The storm duration is t_d, and is varied until the storage volume (shaded area) is maximized. It is normally an iterative process done by hand or on a spreadsheet. Downstream analysis is not possible with this method, as only approximate graphical routing takes place.

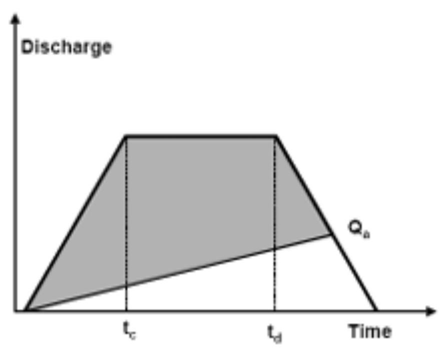


Figure C3-S9- 4: Modified Rational hydrograph definitions

F. Design example

The development drainage area, A, is 18 acres (784,080 ft^2), the runoff coefficient, C, is 0.72, and the time of

Manning's equation is:

Equation C3-S3- 4

$$V = \frac{1.49R^{\frac{2}{3}}S^{\frac{1}{2}}}{n}$$

Where:
V = average velocity (ft/s)
R = hydraulic radius (ft) and is equal to A/WP
A = cross sectional flow area (ft^2)
WP = wetted perimeter (ft)
s = slope of the hydraulic grade line (channel slope, ft/ft)
n = Manning's roughness coefficient for open channel flow (See Table C3-S3- 2)

Manning's *n* values for open channel flow can be obtained from standard textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using Equation C3-S3- 3 and Equation C3-S3- 4, T_t for the channel segment can be estimated using Equation C3-S3- 1.

5. **Reservoirs or lakes.** Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed as zero.

6. **Limitations:**
   a. Manning's kinematic solution should not be used for sheet flow longer than 100 feet. Equation C3-S3- 3 was developed for use with the four standard rainfall intensity-duration relationships.

   b. In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or non-pressure flow.

   c. The minimum T_c used in WinTR-55 is 0.1 hour.

Common Information for Tc and Storage Calculations		
Variable	Value	Notes/References
2-year 24-hr Rainfall Depth (in)	3.01	ISWMM C3-S3-D-2
Sewer Diameter (ft)	1.25	
Sewer Wetted Perimeter (ft)	3.926990817	ISWMM C3-S3-D-4
Sewer Cross-Sectional Area (square ft)	1.22718463	
Sewer Hydraulic Radius (ft)	0.3125	
Pre-Development Curve Number	85	WinTR-55
Post-Development Curve Number	80	WinTR-55
50-year 24 hour Rainfall Intensity (in/hr)	0.26	SUDAS Table 2B-2.07
Design Storm Time (h)	24	ISWMM C3-S9-A-2

Northwest Basin (Basin 1) Time of Concentration Calculations		
Variable	Value	Notes/References
Total Non-Sewer Length (ft)	604.21	
Overland Flow Slope (ft)	0.11	
Paved Length (ft)	0	
Unpaved roughness coefficient	0.24	Table C3-S3-1 [ISWMM]
Paved roughness coefficient	0.011	Table C3-S3-1 [ISWMM]
Sheet Flow Length (ft)	100	
Sheet Flow Travel Time (hr)	**0.01052863**	Equation C3-S3-3 [ISWMM]
Shallow Concentrated Unpaved Flow Length (ft)	604.21	
Shallow Concentrated Unpaved Flow Velocity (ft/s)	5.15	Figure C3-S3-1 [ISWMM]
Shallow Concentrated Unpaved Flow Travel Time (hr)	**0.032589536**	Equation C3-S3-1 [ISWMM]
Shallow Concentrated Paved Flow Length (ft)	0	
Shallow Concentrated Paved Flow Velocity (ft/s)	5.2	Figure C3-S3-1 [ISWMM]
Shallow Concentrated Paved Flow Travel Time (hr)	**0**	Equation C3-S3-1 [ISWMM]
Sewer High Point (ft)	783.86	
Sewer Low Point (ft)	739.73	
Sewer Flow Length (ft)	880.62	
Sewer Slope (ft/ft)	0.050112421	
Sewer Flow Velocity (ft/s)	13.96367147	Equation C3-S3-4 [ISWMM]
Sewer Flow Travel Time (hr)	**0**	Equation C3-S3-1 [ISWMM]
Total Time of Concentration (hr)	**0.043118166**	Equation C3-S3-2 [ISWMM]
Total Time of Concentration (min)	**2.587089972**	
Basin Area (acres)	4.29	
Peak Discharge (cfs) (Pre-Development)	89.232	C3-S9-1
Peak Discharge (cfs) (Post-Development)	94.809	C3-S9-1

North Basin (Basin 2) Time of Concentration Calculations		
Variable	Value	Notes/References
Total Non-Sewer Length (ft)	322.03	
Overland Flow Slope (ft)	0.068565041	
Paved Length (ft)	322.03	
Unpaved roughness coefficient	0.24	Table C3-S3-1 [ISWMM]
Paved roughness coefficient	0.011	Table C3-S3-1 [ISWMM]
Sheet Flow Length (ft)	100	
Sheet Flow Travel Time (hr)	**0.012720022**	Equation C3-S3-3 [ISWMM]
Shallow Concentrated Unpaved Flow Length (ft)	0	
Shallow Concentrated Unpaved Flow Velocity (ft/s)	NA	Figure C3-S3-1 [ISWMM]
Shallow Concentrated Unpaved Flow Travel Time (hr)	**0**	Equation C3-S3-1 [ISWMM]
Shallow Concentrated Paved Flow Length (ft)	322.03	
Shallow Concentrated Paved Flow Velocity (ft/s)	5.2	Figure C3-S3-1 [ISWMM]
Shallow Concentrated Paved Flow Travel Time (hr)	**0.017202457**	Equation C3-S3-1 [ISWMM]
Sewer High Point (ft)	783.86	
Sewer Low Point (ft)	739.73	
Sewer Flow Length (ft)	880.62	
Sewer Slope (ft/ft)	0.050112421	
Sewer Flow Velocity (ft/s)	13.96367147	Equation C3-S3-4 [ISWMM]
Sewer Flow Travel Time (hr)	**0.017518077**	Equation C3-S3-1 [ISWMM]
Total Time of Concentration (hr)	**0.047440555**	Equation C3-S3-2 [ISWMM]
Total Time of Concentration (min)	**2.846433323**	
Basin Area (acres)	17.89	
Peak Discharge (cfs) (Pre-Development)	372.112	C3-S9-1
Peak Discharge (cfs) (Post-Development)	395.369	C3-S9-1
Required Detention Storage (cubic ft)	**4176.881738**	C3-S9-2

Northeast Basin (Basin 3) Time of Concentration Calculations		
Variable	Value	Notes/References
Total Non-Sewer Length (ft)	372.33	
Overland Flow Slope (ft)	0.025326995	
Paved Length (ft)	372.33	
Unpaved roughness coefficient	0.24	Table C3-S3-1 [ISWMM]
Paved roughness coefficient	0.011	Table C3-S3-1 [ISWMM]
Sheet Flow Length (ft)	100	
Sheet Flow Travel Time (hr)	**0.018945038**	Equation C3-S3-3 [ISWMM]
Shallow Concentrated Unpaved Flow Length (ft)	0	
Shallow Concentrated Unpaved Flow Velocity (ft/s)	NA	Figure C3-S3-1 [ISWMM]
Shallow Concentrated Unpaved Flow Travel Time (hr)	**0**	Equation C3-S3-1 [ISWMM]
Shallow Concentrated Paved Flow Length (ft)	372.33	
Shallow Concentrated Paved Flow Velocity (ft/s)	3.37	Figure C3-S3-1 [ISWMM]
Shallow Concentrated Paved Flow Travel Time (hr)	**0.030689911**	Equation C3-S3-1 [ISWMM]
Sewer High Point (ft)	796.62	
Sewer Low Point (ft)	728	
Sewer Flow Length (ft)	1337.14	
Sewer Slope (ft/ft)	0.051318486	
Sewer Flow Velocity (ft/s)	14.13070557	Equation C3-S3-4 [ISWMM]
Sewer Flow Travel Time (hr)	**0.026285154**	Equation C3-S3-1 [ISWMM]
Total Time of Concentration (hr)	**0.075920103**	Equation C3-S3-2 [ISWMM]
Total Time of Concentration (min)	**4.555206189**	
Basin Area (acres)	8.32	
Peak Discharge (cfs) (Pre-Development)	173.056	C3-S9-1
Peak Discharge (cfs) (Post-Development)	183.872	C3-S9-1

East Basin (Basin 4) Time of Concentration Calculations		
Variable	Value	Notes/References
Total Non-Sewer Length (ft)	457	
Overland change in elevation (ft)	34.66	
Overland Flow Slope (ft)	0.075842451	
Paved Length (ft)	54	
Unpaved roughness coefficient	0.24	
Paved roughness coefficient	0.011	Table C3-S3-1 [ISWMM]
Sheet Flow Length (ft)	100	
Sheet Flow Travel Time (hr)	**0.008866798**	Equation C3-S3-3 [ISWMM]
Shallow Concentrated Unpaved Flow Length (ft)	303	
Shallow Concentrated Unpaved Flow Velocity (ft/s)	4.3	Figure C3-S3-1 [ISWMM]
Shallow Concentrated Unpaved Flow Travel Time (hr)	**0.019573643**	Equation C3-S3-1 [ISWMM]
Shallow Concentrated Paved Flow Length (ft)	54	
Shallow Concentrated Paved Flow Velocity (ft/s)	5.8	
Shallow Concentrated Paved Flow Travel Time (hr)	**0.002586207**	
Sewer High Point (ft)	771.34	
Sewer Low Point (ft)	754.63	
Sewer Flow Length (ft)	236.55	
Sewer Slope (ft/ft)	0.070640457	
Sewer Flow Velocity (ft/s)	16.57882242	Equation C3-S3-4 [ISWMM]
Sewer Flow Travel Time (hr)	**0.00396339**	Equation C3-S3-1 [ISWMM]
Total Time of Concentration (hr)	**0.034990038**	Equation C3-S3-2 [ISWMM]
Total Time of Concentration (min)	**2.099402306**	
Basin Area (acres)	17.89	
Peak Discharge (cfs) (Pre-Development)	372.112	C3-S9-1
Peak Discharge (cfs) (Post-Development)	395.369	C3-S9-1

Southeast Basin (Basin 5) Time of Concentration Calculations

Variable	Value	Notes/References
Total Non-Sewer Length (ft)	792.63	
Overland change in elevation (ft)	57.15	
Overland Flow Slope (ft)	0.072101737	
Paved Length (ft)	68.42	
Unpaved roughness coefficient	0.24	Table C3-S3-1 [ISWMM]
Paved roughness coefficient	0.011	Table C3-S3-1 [ISWMM]
Sheet Flow Length (ft)	100	
Sheet Flow Travel Time (hr)	**0.146826854**	Equation C3-S3-3 [ISWMM]
Shallow Concentrated Unpaved Flow Length (ft)	624.21	
Shallow Concentrated Unpaved Flow Velocity (ft/s)	4.25	Figure C3-S3-1 [ISWMM]
Shallow Concentrated Unpaved Flow Travel Time (hr)	**0.040798039**	Equation C3-S3-1 [ISWMM]
Shallow Concentrated Paved Flow Length (ft)	68.42	
Shallow Concentrated Paved Flow Velocity (ft/s)	5.7	Figure C3-S3-1 [ISWMM]
Shallow Concentrated Paved Flow Travel Time (hr)	**0.003334308**	Equation C3-S3-1 [ISWMM]
Sewer High Point (ft)	763.55	
Sewer Low Point (ft)	743.96	
Sewer Flow Length (ft)	543.88	
Sewer Slope (ft/ft)	0.036018975	
Sewer Flow Velocity (ft/s)	11.83838895	Equation C3-S3-4 [ISWMM]
Sewer Flow Travel Time (hr)	**0.012761684**	Equation C3-S3-1 [ISWMM]
Total Time of Concentration (hr)	**0.203720885**	Equation C3-S3-2 [ISWMM]
Total Time of Concentration (min)	**12.22325312**	
Basin Area (acres)	15.14	
Peak Discharge (cfs) (Pre-Development)	314.912	C3-S9-1
Peak Discharge (cfs) (Post-Development)	334.594	C3-S9-1
Required Detention Storage (cubic ft)	**3508.678107**	C3-S9-2

Southwest Basin (Basin 6) Time of Concentration Calculations		
Variable	Value	Notes/References
Total Non-Sewer Length (ft)	414.58	
Overland Flow Slope (ft)	0.03265972	
Paved Length (ft)	414.58	
Unpaved roughness coefficient	0.24	Table C3-S3-1 [ISWMM]
Paved roughness coefficient	0.011	Table C3-S3-1 [ISWMM]
Sheet Flow Length (ft)	100	
Sheet Flow Travel Time (hr)	**0.017112914**	Equation C3-S3-3 [ISWMM]
Shallow Concentrated Unpaved Flow Length (ft)	0	
Shallow Concentrated Unpaved Flow Velocity (ft/s)	NA	Figure C3-S3-1 [ISWMM]
Shallow Concentrated Unpaved Flow Travel Time (hr)	**0**	Equation C3-S3-1 [ISWMM]
Shallow Concentrated Paved Flow Length (ft)	414.58	
Shallow Concentrated Paved Flow Velocity (ft/s)	3.6	Figure C3-S3-1 [ISWMM]
Shallow Concentrated Paved Flow Travel Time (hr)	**0.031989198**	Equation C3-S3-1 [ISWMM]
Sewer High Point (ft)	788.25	
Sewer Low Point (ft)	736	
Sewer Flow Length (ft)	1599.83	
Sewer Slope (ft/ft)	0.03265972	
Sewer Flow Velocity (ft/s)	11.27283526	Equation C3-S3-4 [ISWMM]
Sewer Flow Travel Time (hr)	**0.039421957**	Equation C3-S3-1 [ISWMM]
Total Time of Concentration (hr)	**0.088524068**	Equation C3-S3-2 [ISWMM]
Total Time of Concentration (min)	**5.311444062**	
Basin Area (acres)	31.01	
Peak Discharge (cfs) (Pre-Development)	645.008	C3-S9-1
Peak Discharge (cfs) (Post-Development)	685.321	C3-S9-1
Required Detention Storage (cubic ft)	**7226.006299**	C3-S9-2

Appendix D: Water Main

EQ D1:

$$\frac{p_1}{\gamma} + \alpha_1 \frac{\overline{V}_1^2}{2g} + z_1 + h_p = \frac{p_2}{\gamma} + \alpha_2 \frac{\overline{V}_2^2}{2g} + z_2 + h_t + h_L$$

EQ D2:

$$h_f = f \frac{L}{D} \frac{V^2}{2g}$$

EQ D3:

$$\mathrm{Re} = \frac{\rho V D}{\mu} = \frac{V D}{\nu}$$

Knowns		Sources/Notes
Connection Point (ft)	729.06	Civil 3D Topographic Map
High Point (ft)	806.00	Civil 3D Topographic Map
Elevation Change (ft)	76.94	
Expected Pipe Length (ft)	1707.01	Civil 3D Topographic Map
Slope (ft/ft)	0.04507	
Pressure at connection point (psi)	53	Jen Schwoob - Alliance Water
Pressure at connection point (lbs./sq. ft)	7632	
Pipe Diameter (ft)	0.83333	Jen Schwoob - Alliance Water

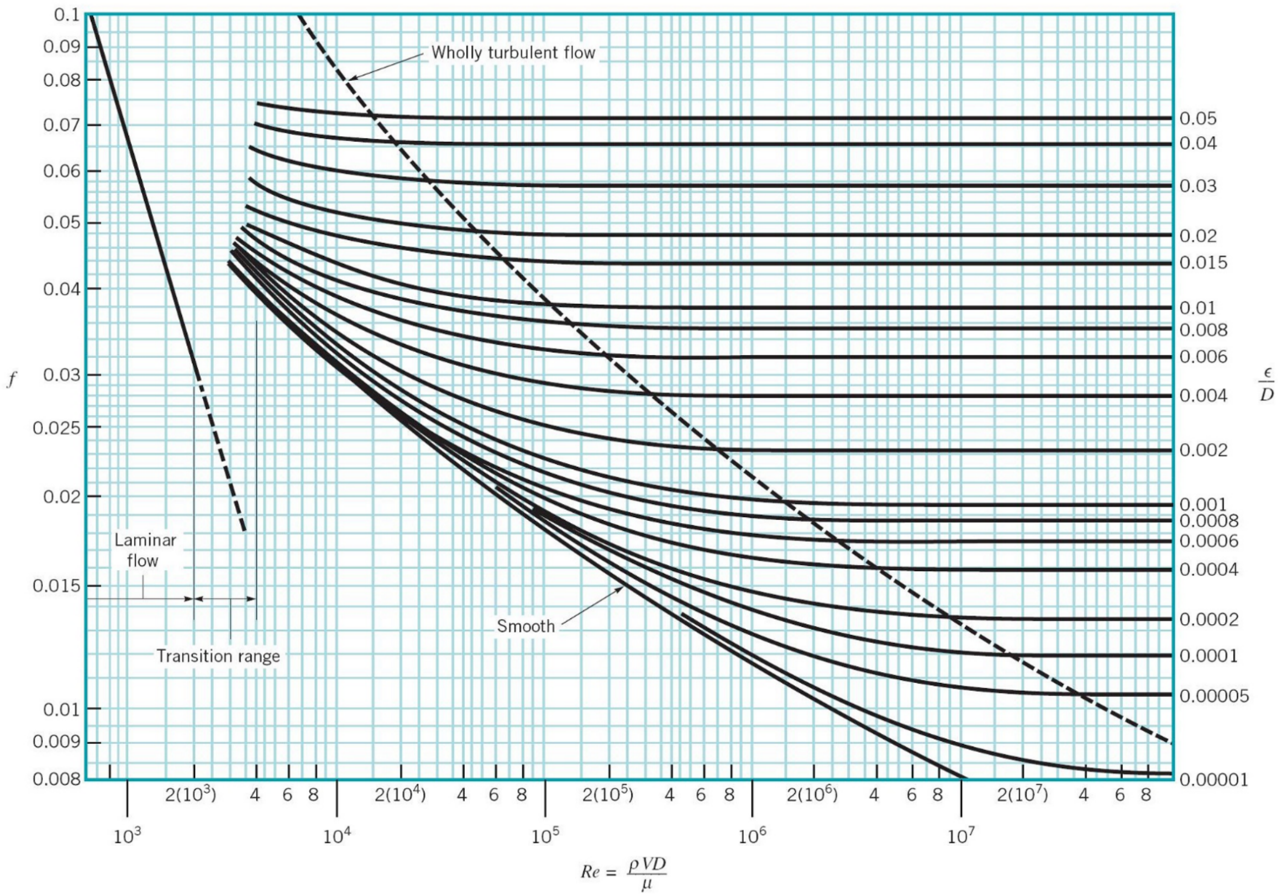
Assumptions		Sources/Notes
v1 = v2 = u		Pipe diameter is same, conservation of energy
hp = ht = 0	0	No pumps or turbines
PVC pipes (Є = 0.000075 in = 6.25*10^-6 ft)	0.00000625	(Engineering Toolbox, 2019a)
Water temperature = 20 C		
Kinematic viscosity = 1.08 * 10^-5 ft^2/s	0.0000108	(Engineering Toolbox, 2019b)
g = 32.2 ft/s^2	32.2	
Velocity (ft/s)	2.5	(SUDAS minimum to prevent buildup)
Specific Gravity (γ) lb.*ft^-3	62.4	(Engineering Toolbox, 2003)
alpha 1,2 = 1	1	Hydraulics & Hydrology Lecture 3 Slide 9. George Constantinescu

Table D1

Calculated/Determined Values		Sources/Reasoning
Reynold's number (Re)	1.9290E+05	(EQ3 with kinematic viscosity assumption)
Є/D	0.000008	Roughness (Є) divided by pipe diameter
Friction Factor (f)	0.016	Figure D1
hf (ft)	3.18	(EQ2 with applicable assumptions)
Pressure at Peak Elevation (psf)	2632.46	(EQ1 with listed assumptions)
Pressure at Peak Elevation (psi)	18.281	(EQ1 with listed assumptions)
Change in Pressure (psi)	34.71899554	
Required pump head (ft) to maintain 53 psi at high point	80.12075894	(EQ1 with listed assumptions except p2 = 53 and hp is unknown instead of 0)
Best Case Scenario Change in pressure (psi)	-33.34066667	(EQ1 with listed assumptions except no losses)
Best Case Pressure at Peak Elevation (psi)	19.659333	
Worst Case Pressure at Peak Elevation (psi)	16.114338	

Factors Not Considered
Losses from pipe connections
Losses from valves
Losses from tie-ins for each house
Losses from neighborhood water draw (Assuming 100 GPD/household)

Figure D1: friction factor



Number of Units x Unit Density x Rate = Average Daily Demand **Equation 4B-1.02**

Table D2: Average Daily Demand Equation

Land Use	Area Density	Unit Density	Rate
Low Density (Single Family) Residential	10 people/AC	3.0 people/unit	100 gpcd
Medium Density (Multi-Family) Residential	15 people/AC	3.0 people/unit 6.0 people/duplex	100 gpcd
High Density (Multi-Family) Residential	30 people/AC	2.5 people/unit	100 gpcd
Office and Institutional	Special Design Density[1]		
Commercial	Special Design Density[1]		
Industrial	Special Design Density[1]		

Table D3: Land use Densities

Number of Units	Unit Density (people/unit)	Rate (gpcd)	Average Day Demand (gpd)
72	3	100	21600

Table D4: Average Daily Demand for Site.

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

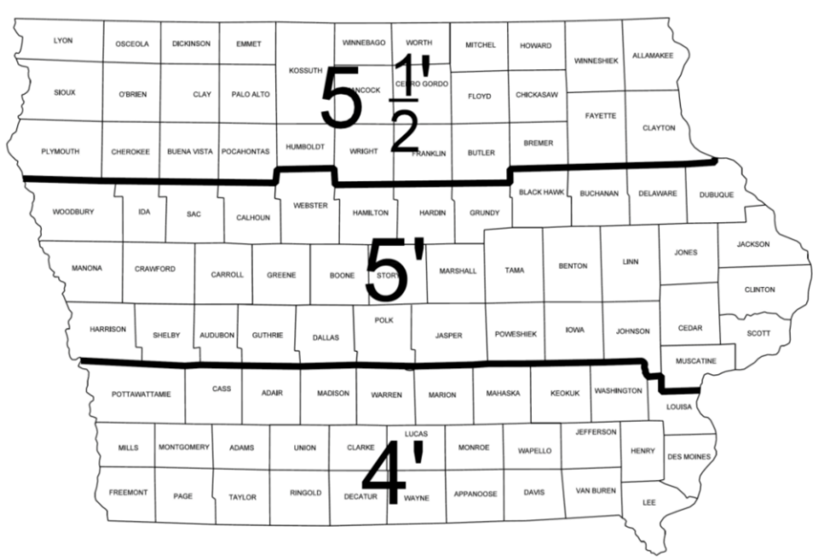


Figure D2: Minimum Depth Cover

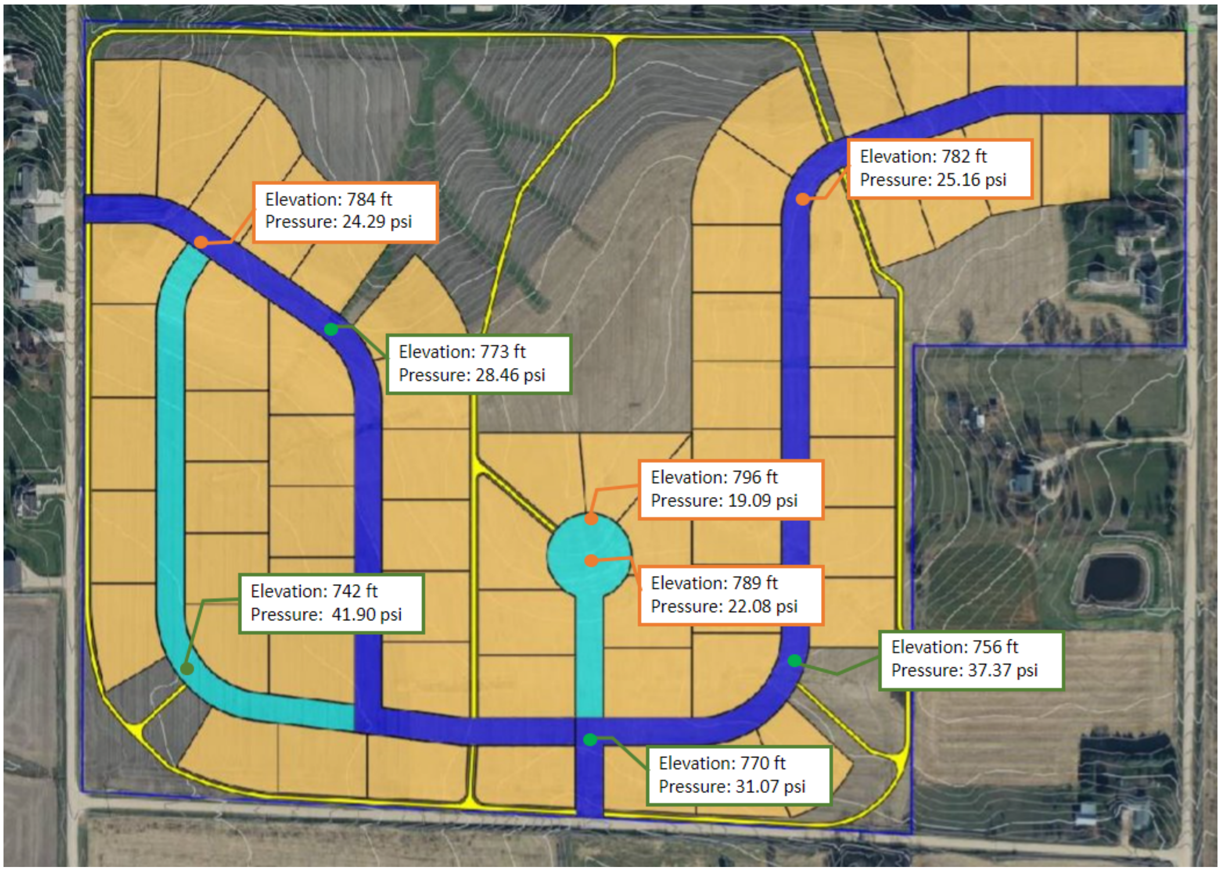

Elevation: 784 ft
Pressure: 24.29 psi
Elevation: 782 ft
Pressure: 25.16 psi
Elevation: 773 ft
Pressure: 28.46 psi
Elevation: 796 ft
Pressure: 19.09 psi
Elevation: 789 ft
Pressure: 22.08 psi
Elevation: 742 ft
Pressure: 41.90 psi
Elevation: 756 ft
Pressure: 37.37 psi
Elevation: 770 ft
Pressure: 31.07 psi

Figure 3C-1.01: Flow for Circular Pipe Flowing Full (Based on Manning's Equation n=0.013)

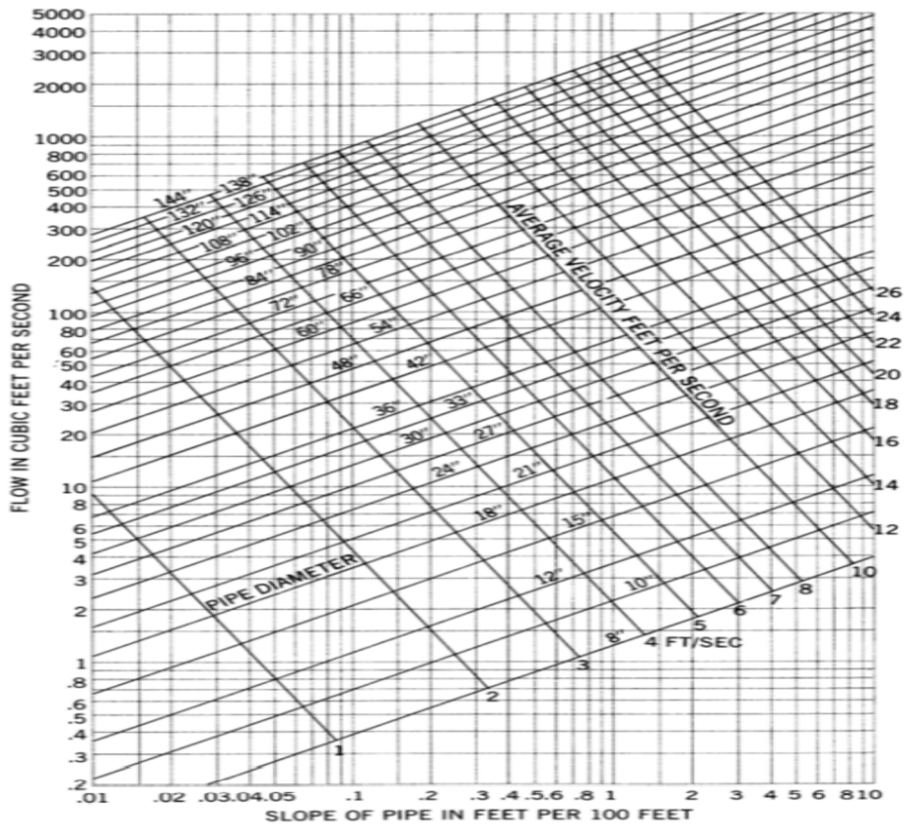


Figure E3: Flow for Circular Pipe Flowing Full

D. Valves

1. As a minimum, valves should be located at intersections, such that only one unvalved pipe exists at the intersection. Valves should be equally spaced, if possible, with spacing no more than 800 feet in residential areas and no more than 400 feet in high density residential, commercial, and industrial areas. (See Figures 4C-1.02 through 4C-1.03 for valve locations at intersections).

2. Valves should not be located in the sidewalk line or in driveways.

3. All valves should be installed with valve boxes. Use slide type valve boxes in paved areas and screw type in all other areas. A screw type valve box that is located in an area to be paved should be changed to a slide type valve box as a part of the paving program.

4. No valves (except blowoff valves) should be placed at the end of a dead-end main unless required by a Jurisdiction. A valve should be installed between the existing main and new main when the main is extended. Intermediate valve locations between the end of a dead-end main and last valved street intersection may be required by the Jurisdiction to provide required valve spacing.

5. A tapping sleeve and valve should be used when making a perpendicular connection to an existing main.

6. If the project area has high water pressure, usually exceeding 100 psi, it may be appropriate to install system pressure relief valves as opposed to individual building controls. The potential for using a system pressure reducing valve is limited by the interconnected nature of a distribution system. Check with the Jurisdiction to determine the potential need for use of pressure reducing valves.

E. Fire Hydrants

1. Hydrants should comply with AWWA C502. The connecting pipe between the supply main and the hydrants should be a minimum of 6 inches in diameter and be independently valved. Fire hydrants should not be installed on water mains that do not provide a minimum pressure.

2. Hydrant drains should not be connected to or located within 10 feet of sanitary sewers.

3. Locations of fire hydrants are governed by the rules and regulations of the Iowa DNR and the local Jurisdiction and by the following principles. Satisfy each principle in the order they are listed. See Figures 4C-1.02 through 4C-1.03 for typical hydrant locations.

   a. Locate fire hydrants within 25 feet of each street intersection, measured from an end of a street paving return.

      Locate fire hydrants outside street paving returns. Avoid conflicts with storm sewers, intakes, and sidewalks. Whenever possible, locate fire hydrants at the high point of the intersection.

   b. Locate fire hydrants between street intersections to provide spacings of no more than 450 feet in single family residential districts and no more than 300 feet in all other districts. Coverage radii for structures as noted below should be checked when determining hydrant placement.

      Vary spacings slightly to place fire hydrants on extensions of property lines. When hydrants are required between intersections, they should be located at the high point of the main for air release or at a significant low point for flushing on the downhill side of an in-line valve.

      When street curvature or grid patterns places a proposed protected structure at an unusual distance from the fire hydrant, the coverage radius should not exceed 300 feet in single family residential districts and 150 feet in all other districts. The Jurisdiction's fire marshall may have additional private fire protection requirements.

   c. On cul-de-sac streets, hydrants should be located at the intersection of the cul-de-sac street and cross-street and the end of the cul-de-sac.

      1) For cul-de-sacs between 300 feet and 500 feet in length, an additional hydrant should be located at the mid-block.

      2) For cul-de-sacs greater than 500 feet in length, hydrants should be placed at near equal spacings, but not exceeding the spacings described above.

   d. Hydrants must be located to provide the required fire flows. ISO evaluates fire hydrant locations within 1,000 feet of the test location, measured along the streets as fire hose can be laid, to evaluate the availability of water for fire protection. Hydrant capacity is credited as shown in the following table:

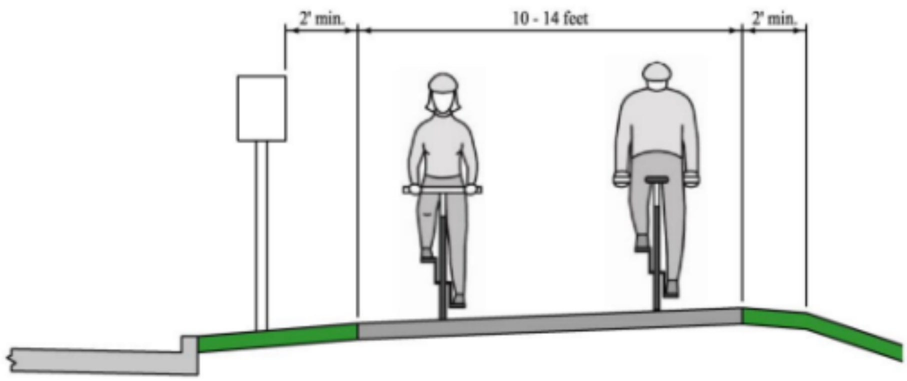
Hydrant Location	Credited Capacity
Within 300' of location	1,000 gpm
Within 301' to 600' of location	670 gpm
Within 601' to 1,000' of location	250 gpm

   e. Locate fire hydrants to maintain a 3 foot clear space around the circumference of the fire hydrant to create unobstructed access for the fire department.

Appendix E: Greenspace and Trail

Figure F1: SUDAS Two-Way Shared Use Path

Figure 12B-2.01: Typical Cross-Section of Two-Way Shared Use Path on Independent Right-of-Way



Source: Adapted from *AASHTO Bike Guide* Exhibit 5.1

Figure F2: Sudas 12B-12C

C. Shared Use Path Design Elements

The following considerations should be used as a guide when designing shared use paths.

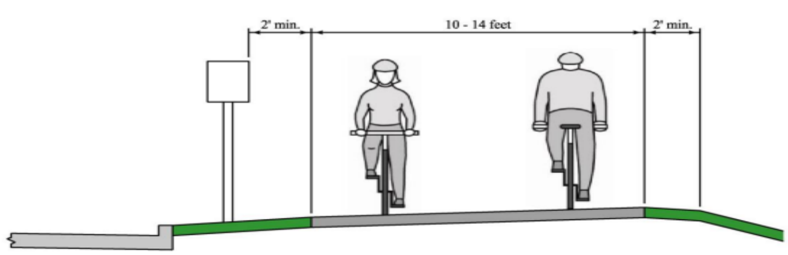
1. **Width:** A bicyclist requires a minimum of 4 feet and a preferred 5 feet of essential operating space based upon their profile. The typical path width is 10 feet to accommodate two-way traffic. Consider wider paths (11 to 14 feet) when at minimum one of the following is anticipated:
   - User volume exceeding 300 users within the peak hour.
   - Curves where more operating space should be provided.
   - Large maintenance vehicles.
   - There is a need for a bicyclist to pass another path user while maintaining sufficient space for another user approaching from the opposing direction. 11 feet is the minimum width for three lanes of traffic.

   Path width can be reduced to 8 feet where the following conditions prevail:
   - Bicycle traffic is expected to be low.
   - Pedestrian use is generally not expected.
   - Horizontal and vertical alignments provide well-designed passing and resting opportunities.
   - The path will not be regularly subjected to maintenance vehicle loading conditions.
   - A physical constraint exists for a short duration such as a utility structure, fence, etc.

   Path widths between 8 and 5 feet should be avoided; paths less than 5 feet do not meet ADA requirements.

   If segregation of pedestrians and bicycle traffic is desirable, a minimum 15 foot width should be provided. This includes 10 feet for two-way bicycle traffic and 5 feet for two-way pedestrian traffic. (AASHTO 5.2.1).

Figure 12B-2.01: Typical Cross-Section of Two-Way Shared Use Path on Independent Right-of-Way



Source: Adapted from *AASHTO Bike Guide* Exhibit 5.1

2. **Minimum Surface Thickness:** For Iowa DOT projects, contact the Pavement Design Section in the Design Bureau for a pavement determination. For local agency projects administered through Iowa DOT, Iowa DOT will accept the thickness design as determined by the engineer.

Appendix F: References

References

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