Regional Stormwater Control for Washington Iowa

Prepared for:
City of Washington
Project Design and Management 053:084

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Spring 2014

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Executive Summary

The Regional Stormwater Control Project for Washington Iowa includes a comprehensive hydrologic assessment of the northwest watershed in Washington, and provides three alternative designs to mitigate flooding over West Main Street. The project constraints caused many challenges; most notably, the surface elevation of Sesqui Park is too high to divert stormwater from the drainage channel there by gravity, and insufficient surface area is available for the project as a result. Another hard constraint that limited storage volume is the depth to bedrock in the area. On average, the bedrock is exposed 6-7 feet below the surface elevation. Due to budget constraints, Washington does not wish to use a pump at this time. As a result, the alternatives provided use as much area as possible within the city's property lines along the natural drainage channel. The three alternatives include: a series of two in-line detention basins, channel modification, and the use of a pump to bring water to Sesqui Park for storage in a detention basin. Although Washington does not desire a pump at this time, they may want to use it for stormwater management in the future, when more land upstream is developed. The series of in-line detention basins was selected as the best recommendation for the city based off the decision making matrix found in Section 4 of this report. The storage capacity of each alternative was the largest weighted factor.

An entire field assessment was conducted before the design process began. Site visits to Washington exposed the lack of infrastructure currently managing the stormwater. The natural drainage channel running through the watershed is obstructed by a 1-foot diameter culvert, the channel is full of debris, and the outlet culvert is in pieces and covered in debris. During the site visit, it was evident Sesqui Park is on a hill. After the site visit ArcMap and the Web Soil Survey were used to analyze the surface and subsurface. The Northwest Washington Iowa watershed was delineated. Using the data obtained from the Web Soil Survey and ArcMap, the runoff analysis was conducted. A detailed explanation of the runoff analysis can be found in Section 2.6 of this report.

The runoff analysis concluded the peak discharge is 942.48 cfs for 25-year storm, 309 cfs for 5-year storm, and 131 cfs for the 2-year storm. The runoff analysis is based off the National Resources Conservation Service method. These conditions were also simulated by the Hydrologic Modeling System (HEC-HMS) software and Modified Puls method.

The in-line detention basins is the recommended alternative because it has most storage capacity. A discussion of other factors considered can be found in Section 4. Although the design is not able to handle more than 5 year storm without flooding, it will reduce flooding over West Main Street for moderate wet conditions. The channel modification is the cheapest option, but it will not reduce the flood frequency as much as the in-line detention basins do. The third option is detention basin and it located in the Sesqui Park which is the area the city wants to develop but it will be most expensive since it requires a pump.

The cost estimations were based largely on material costs such as excavation, gravel, and asphalt. The excavation costs turned out to be the most expensive because of the shear amount of earth that needed to be moved. The Gravel and asphalt were smaller costs that only applied to the in-line detention basin and the channel modification alternatives. Other costs such as labor and equipment will be finalized when the city decides on what contractor they would like to use, or what brand of equipment they normally purchase. This would apply to the Sesqui Park detention basin which requires the city to decide on a stormwater pump to buy.

The final cost for the three alternatives vary in price because of the amount of excavation done on each. So including the natural materials the total costs of each of the alternatives is $969,109.00 for the in-line detention basin, $18,869.00 for the channel modification, and $68,425.00 for the Sesqui Park detention basin.

1. Introduction and Background

Washington, Iowa is a small city located in southeastern Iowa. It spans a total area of 4.92 square miles, and is home to 7,266 residents as of the 2010 census. Within the rural city, there is medium density residential development, a high density downtown area with commercial and industrial development, and agricultural land practices in and around Washington. According to the Washington Comprehensive Plan, the city wishes to "encourage compact, contiguous, and fiscally responsible development." Additionally, the city wishes to "support and revitalize existing neighborhoods." Washington established priority growth areas in the comprehensive plan including the northwest side of the city, which currently faces stormwater management challenges. There is a proposed wellness park to be installed in this area, which is projected to have significant societal impacts for Washington. This site can be seen in Figure 1.

Figure 1: Overhead of site

While the city is somewhat flat, there are rolling hills throughout Washington. Although there are no major streams, rivers, or lakes within the city, there are natural drainage channels and areas that become inundated during wet periods. The hydrologic soil group in Washington is Group C, which has low infiltration rates, causing the ground to remain quite wet after rainfall. The runoff throughout the northwest watershed in the city caused a natural drainage channel to run through it. Sesqui Park (12.5 acres) is located on a hill adjacent from the natural drainage channel, and Sunset Park is located just south of the channels outlet. While Sunset Park is frequently utilized by Washington's residents, Sesqui Park is rarely used. The city wishes to find a way to incorporate Sesqui Park into a water management system.

To promote safe development, Washington Iowa wants to tackle its flooding issues in the northwest side of the city. The natural drainage channel is at the core of the problem. The volume of the channel is insufficient to store runoff during wet periods. Channel overtopping is experienced over West Third Street just north of the train tracks, and over West Main Street where the outlet is located. In addition to mitigating the flooding conditions, Washington wants the stormwater management system to have the capacity for the additional runoff resulting from the proposed wellness park. The Iowa Department of Natural Resources Stormwater Manual contains the design criteria for various stormwater practices.

The city administrators suggested a detention basin to manage the stormwater. Detention basins are storage areas that gather stormwater and release the stored water gradually through an uncontrolled outlet. Similar to detention basins, retention basins are storage areas where there is no outlet, or the accumulated (impounded) water is stored for a long period of time. In typical applications, detention basins are used when there is limited infiltration ability on site. Otherwise, retention basins are favorable. Basins that continuously retain water are considered wet basins and those that do not have a permanent body of water are dry basins. (Chin, 2013). Other options were explored to manage the stormwater.

Wetlands are utilized for flood mitigation, water quality improvement, aesthetic enhancements, and recreational purposes. Iowa, having been mostly wetlands in the past, is prone to flooding due to agricultural practices, development, and its flat land. The constructed wetland for Sesqui Park would be a Type 1 Wetland of the United States. Type 1 wetlands are in areas that have seasonally flooded basins or flats. They have saturated soil for periods of heavy rain, usually from the spring to fall, although during much of the growing season it is well drained. Vegetation can vary greatly, depending on the hydrology of the community. The Sesqui Park Wetland would be comprised of herbaceous growth plants, and bottom-land hardwoods. Wetlands are often very costly, but potentially a viable option for Washington.

Drainage channel modification is another practice the City of Washington wished to explore. Channel excavation would allow a greater volume to be stored in it. Sections 2.3 and 2.4 of this report explain why the two previous stormwater management practices are not viable options for this project. Section 3 describes the preliminary alternatives that are most practical for the existing conditions in northeast Washington, and section 5 describes the final design details. As found in those section, drainage channel modification will work best for the Regional Stormwater Control for Washington, Iowa.

2. Problem Statement

The City of Washington has asked us to come up with some solutions to help better manage the storm water in the northern area of the city. We have been asked and allowed to use Sesqui Park, which is a small park the city owns in the north, as the main area of our designs. We will be designing different solutions that vary in price range from a more conservative alternative to a more extensive alternative. The city also wants to stay as green as possible by keeping the natural look of the area intact. All of our solutions will be designed to blend in with the natural surroundings of Sesqui and the green waterways of the wellness park. As we go through and design these solutions there will be some constraints we need to be aware of such as the time the project will take, how much it will cost, and the requirements that need to be met as stated in the Iowa DNR. There will also be challenges we will face such as what we can and cannot change about the area or areas that we will not be able to build through simply because the land is privately owned. Other variables that will be taken into account are the societal impacts the solutions will have such as making Sesqui a place residents want to go biking or walking through, help to draw more people to want to live in the area, and preventing Main Street from being impassable during heavy rains when residents need to commute.

2.1 Design Objectives

The City of Washington desires a comprehensive approach to stormwater management in the northwest side of the city. The Regional Stormwater Control project must mitigate current flooding conditions over West Third St. and West Main St. in addition to handling the runoff resulting from the proposed Wellness Park. The city administrators are interested in utilizing Sesqui Park for a detention basin, but are opened to other alternative plans. In addition to mitigating floods, the project will preserve natural areas, aesthetically enhance the area, promote future development in the Northwest area of Washington, and provide a connection between the proposed Wellness Park and existing parks and trails.

2.2 Approaches

The alternatives for the Regional Stormwater Control for Washington, Iowa were designed following approaches suggested by the United States Environmental Protect Agency and the Iowa Department of Natural Resources (DNR) Stormwater Manual. However, the site needed to be assessed before choosing an appropriate practice for stormwater management. The City of Washington had no previous data regarding the watershed and its runoff. In order to adequately assess the site, ArcGis and the Web Soil Survey were utilized.

In order to delineate the watershed, ArcMap 10.1 was used. In a series of steps using various tools, this program was capable of delineating the watershed using a digital elevation model (DEM) layer of the area surrounding Sesqui Park and the proposed Wellness Park. The procedure for delineating the watershed is available in Appendix _.

The Web Soil Survey was used to identify soil types and the depth to bedrock. The soil type is a parameter used in calculating the runoff. Additionally, the depth the bedrock data was important for knowing how deep Sesqui Park could be excavated for storage.

In addition to evaluating the watershed and runoff, the existing natural drainage channel was evaluated using ArcMap. Using the 3D Analyst tool, cross-sectional profiles of the channel were extracted. Additionally, the cross-sections of the existing natural drainage channel helped to estimate excavation required. The cross-sections can be found in Appendix 2.

After the evaluation of the site, the Iowa DNR Stormwater Manual was consulted for design guidelines. After a meeting with city administrators, the suggested alternative designs included a wetland, a natural waterway, and a detention pond, but due to constraints an integrated approach was taken for the alternative designs. The constraints are discussed in section 2.3.

Permitting

Permit Requirements for the project:

- ❖ The Iowa Department of Natural Resources (Iowa DNR) National Pollutant Discharge Elimination System (NPDES) General Permit No. 2

To be required to have a General Permit No. 2 the project needs to fall under one of three categories:

- ❖ Construction activity that disturbs one or more acres or which is part of a larger project that disturbs one or more acres in total.
- ❖ Certain types of industrial or commercial activities.
- ❖ Many city storm sewer systems in larger communities or those near larger communities.

A project such as this one falls under the first category. In order to obtain this permit there are three forms that need to be filed. These forms will be attached in Appendix 9.

- ❖ Notice of Intent for NPDES Coverage Under General Permit
- ❖ Public Notice of Stormwater Discharge
- ❖ Notice of Discontinuation (at conclusion of the project)

Along with this permit a Stormwater Pollution Prevention Plan (SWPPP) needs to be made.

The contents of the SWPPP includes the following:

- ❖ Information of the sites existing conditions at the site of construction
- ❖ A developed site plan design looking at limiting the amount of pollution as much as possible
- ❖ Describe the ways the erosion control will be implemented along with the release of pollutants into the stormwater
- ❖ A schedule that shows what is described is actually going to be implemented and to allow the plan to be evaluated for effectiveness of pollution prevention

❖ A final stabilization and Notice of discontinuation, which means that all the activities that disturbed the soil are completed and the correct vegetative cover for unpaved areas not covered by permanent structures has been established or stabilization methods that are similar are implemented, and the discontinuation of discharge that was under the regulations of the General No. 2 Permit

2.3 Constraints

The Regional Stormwater Control for Washington, Iowa is a project with numerous soft and hard constraints. The City of Washington requested that Sesqui Park be integrated into a stormwater management system that prevents flooding over West Main Street and manages the additional runoff effects resulting from the Wellness Park plan. By investigating the area surrounding Sesqui Park and the natural drainage channel with tools including but not limited to ArcGIS and the Web Soil Survey, hard physical constraints were observed. Additionally, various soft and hard constraints were expressed by the City of Washington administrators.

Figure 2: Existing natural drainage channel

Figure 3: Existing culvert at the outlet

Figure 4: Existing culvert under west main street

Existing conditions and water networks must be considered before designing the stormwater management plan. The existing cement culvert at the outlet of the natural drainage channel is in broken pieces and almost completely blocked up with organic matter as seen in Figure 3. Additionally, the channel is filled with debris as seen in Figure 2, and the water is nearly standing upstream from the culvert on West Third Street as seen in Figure 4. Additionally, a forty-eight inch diameter storm sewer that was recently installed is likely undersized according to the city administrator. The city is not interested in replacing the storm sewer of reinforced concrete pipe; they desire a plan that manages more stormwater before it reaches the outlet. Other site features include a railroad that intersects Sesqui Park and the downstream portion of the natural drainage channel.

The features of Sesqui Park and its location result in hard and soft constraints. Sesqui Park is elevated above the natural drainage channel as seen in Figure 4. Additionally it is slightly off-line. Water could not be diverted to Sesqui Park by gravity, unless extensive excavation takes place in the park, and an underground storage reservoir is installed. Unfortunately, bedrock is present at a depth of eighty inches from the surface. This will prohibit the amount of excavation required to make water flow to Sesqui Park by gravity. Data regarding the elevations and soil in Sesqui Park can be found in Appendix _. The area surrounding Sesqui Park is not owned by the city, restricting the use of it.

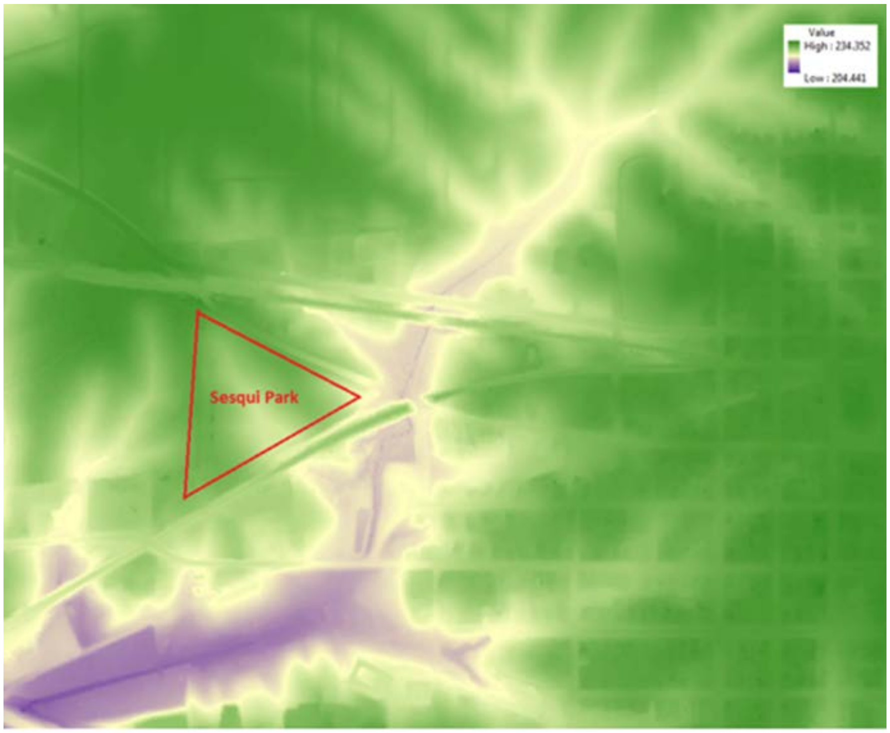

Figure 4: Surface elevation map of Sesqui Park

The combination of space and landscape constraints with budget constraints result in a major hard constraint. The city does not wish to use a pump to bring water from the natural drainage channel to Sesqui Park, because they are expensive and require extensive maintenance. Since a pump is the only way to get water to a higher elevation, Sesqui Park cannot be used to manage stormwater at this time. A hard budget was not expressed by the city administrators, but the range of $100,000 to $500,000 was mentioned. Excavation is very costly and will impact which design is chosen for the City of Washington.

Environmental impacts are another cause for constraint. Iowa’s threatened and endangered species program protects all species that are in danger of extinction or will be in the foreseeable future. If any endangered species are identified on the site of excavation and construction, the project cannot proceed as planned. An assessment of wildlife must be conducted to evaluate whether the site can be used, or a hard constraint on the location being used will be applied. The City of Washington is also concerned with flooding in downstream communities, putting a constraint on how flooding could be mitigated. The water must be managed and not simply diverted downstream. The result of these comprehensive constraints leaves few alternative design options.

2.4 Challenges

The greatest challenge of the Regional Stormwater Control for Washington, Iowa was to navigate past the project's constraints, and design three alternatives that are driven by gravity. Since Washington's administrators do not want to consider pump options, it was difficult to incorporate Sesqui Park into the design, which is elevated above the natural drainage channel. Furthermore, it was difficult to find enough space to manage the runoff volume, because most of the land within the watershed is private property. The preliminary alternatives offered in the proposal were ruled out after a full evaluation of the site and its constraints. For this reason, an integrated approach to managing stormwater was necessary.

Initially, a detention basin or constructed wetland seemed like the best options for managing water in Sesqui Park. A detention basin would store large amounts of water in addition to providing aesthetic features in the park. Similarly, a wetland would store large amounts of water and act as a bio-filtration basin to treat contaminants from the residential and agricultural runoff. After the watershed analysis, and realizing Sesqui is on a hill adjacent to the natural drainage channel, the option of a pump was explored. After speaking with city administrators, a pump was rejected and other options were explored to divert water to Sesqui Park.

An underground storage tank seemed like a viable option in Sesqui Park. An underground storage tank would utilize the land set aside for this project by the city. Additionally, aesthetic features at the land surface would beautify the lot and attract citizens to the area. Since an underground storage tank would require a great deal of excavation, the surface elevations across the park were analyzed, and the depth the bedrock was found using the Web Soil Survey. Unfortunately, the depth to bedrock was only eighty inches below the surface in Sesqui Park, prohibiting the amount of excavation required to make the water from the natural drainage channel flow by gravity to the underground storage tank. Unfortunately, the underground storage tank was not a viable alternative.

Property lines and surface elevations ruled out the option of an adequately sized wetland or detention pond to be installed within the watershed. Due to those constraints, the natural drainage channel was further assessed. Modification of the natural drainage channel was evidently the best viable option for this project, although it was challenging to determine how it could be modified within city-owned land. The cross-section extracted from the digital elevation models using ArcMap exposed more challenges to overcome. When working with channel modification, it was important to ensure the surrounding property would not be at risk of channel overflow conditions.

Some of the issues that arose during the in-line detention design were the trees, and the private property on all sides of the channel. The trees in the area posed a problem because the runoff would flood the area and erode the soil that the tree holds on to by its roots. Upon further inspection a majority of the trees in the area were already dead from flooding throughout the years. So to help create more room to hold more water the trees that were already dead could be removed and any live trees in the site will be replaced on the outside of the basin upon completion of the project. To overcome the land issue, it was decided to push the embankments to the property lines on the other sides. Since the edge of the basin cannot be less than ten feet from a property line the embankment was made wide enough to make sure the design met the city ordinance of ten feet from the property line.

2.5 Societal Impacts

The Regional Stormwater Control Project for Washington is necessary due to current flooding over West Main St. on the Northeast side of the city (just above Sesqui Park). The alternative designs must account for the additional runoff generated by the proposed wellness park. This is an opportunity for the City of Washington to invest in water resource infrastructure that will allow for further residential, commercial, and recreational development. The immense societal impacts of this project include, but are not limited to: mitigation of frequent flooding over West Main St., the reduction of flooding downstream at West Fork Crooked Creek, increased aesthetic value of the city's parks, and water management capacity for further development and economic growth.

In each of the alternative designs, a flooded West Main St. will no longer be a hindrance to citizens attempting to access Hwy. 1. In addition the traffic in and out of residential traffic will flow more easily. Alternative one suggests an in-line detention basin that will hold some of the runoff. Alternative two will be a widened channel that will allow the channel to flow naturally and protect the surrounding land from flooding by having an overflow area to catch the excess runoff. Alternative three proposes a pump that diverts the water uphill to a small detention basin in Sesqui to help mitigate the amount of runoff the channel has to handle. Alternatives one and three will also mitigate runoff to West Fork Crooked Creek, reducing the risk of floods in downstream communities. Alternative 2 will lower the channel stage for the channel and allow it to flow closer to capacity as well as reduce the risk of flooding near Sunset Park.

The detention practices proposed in alternative one and three, will allow development in Washington, and reduce the risk of impact downstream. In accordance with the City of Washington Comprehensive Plan (June 2012) 10 Principles of Future Land use and Development, the stormwater basin will sustain responsible development of private investors as well as residential development for a growing community. Landscaping the area surrounding the detention facilities could also add to the aesthetic value of city-owned property. This will promote more Iowans working in and around Washington to settle there, in turn boosting the city's economy.

The Sesqui detention basin in alternative three could incorporate recreational uses, such as a nature path, bikeway, or running path. This would be a way to promote nature preservation and wellness in Washington. These suggested elements are desirable in residential areas, and could increase property values nearby. This alternative would also sustain further development in the community, making Washington a more attractive place to live.

Making Washington more aesthetically pleasing along with correcting the flooding problem will cost a great sum of money. Each of these proposed solutions will vary in cost. The cost will come from all the permits needed for the construction and the implementation of the recommended solution. These costs will also include excavation of soil, asphalt for the bike path, gravel for the street, and the extension of the culvert underneath the railroad.

The economical impacts this will have on the city of Washington, depends on what solution is chosen. If the less extensive solution is chosen then Washington will be able to make aesthetic improvements to other parts of the city. This would mean the city could clean up more of the natural aesthetics around the city. If the more extensive solution is chosen, the aesthetics of the area will be a huge focal point of the city.

With these solutions being made attractive and kept natural there are some things that need to be taken into account concerning the environment. Before construction of the chosen solution a study of the species of plant and animal life would need to be conducted to determine if there are any endangered species that would have to be protected. Changing the environment with construction could have detrimental impacts on such species.

The Iowa DNR has listed the endangered and protected species that are home to Iowa. These species are protected by law and is a priority any time land that holds wildlife and plant life is being changed. The solutions will have to be designed to allow such a species to thrive in the environment after construction. Precautions while construction is being done will have to be taken to assure the species is safe. If a species that is indigenous to the area is killed off purposely or by accident it can have extremely damaging effects for the environment. Nature is a cycle that has survived because every living organism depends on another to survive and if one is taken out of the cycle it could fall apart. Other animals or plants might not be able to survive.

This being said, this project has the potential to affect everything in the area such as nature, residents, and the local economy. To make sure the impacts are positive ones, great care must be taken when designing and implementing these solutions.

2.6 Runoff Analysis

In order to properly size all three alternatives, the volume of runoff for the Northwest Washington watershed needed to be determined. After delineating the watershed and measuring its drainage area, runoff calculations were performed. All runoff calculations were based the watershed size of 663 acres, soil type C, and an average watershed slope of 0.135%. Since the watershed is greater than 160 acres, the Rational Method is not ideal. However, the peak runoff flow rate was calculated for the entire watershed using the Rational Method for comparison purposes, and was found to be approximately 488 cfs. The calculations of the Rational Method can be seen in Table A5.1- A5.9 in the Appendix 5. The main method for performing the runoff analysis was the NRCS method, as described in the Iowa Stormwater Management Manual. The watershed was initially split into four sub-watersheds. However, since the Wellness Park development covers less than 1% (about 60 acres out of 600 acres) of the watershed, which meant that it did not significantly change the NRCS Curve Number (CN). Therefore, it was determined that the pre- and post-Wellness Park development is similar enough to be negligible, so the peak runoff for "post-development" was used. In addition, the Wellness Park conceptual design by the team of student engineers from The University of Iowa is designed in a way that retains any additional runoff on-site. The natural channel running through the Wellness Park was not disturbed and more greenways and small detention basins or swales were added. This means that the effects of the new park development will have even less of an effect than expected. When determining the CN for the park, the poor conditions of less than 50% vegetation and grass cover was assumed since there is a new YMCA and a significant amount of parking lots in the park, which built in even more of a "safety factor" for the post-development analysis. After the new development was found to not greatly impact the runoff of the watershed, the team moved forward with a runoff analysis of the entire watershed as a whole, including the park.

First, the NRCS triangular unit hydrograph was found in order to properly estimate the peak runoff, duration of rainfall excess, and time base of the unit hydrograph. This was done for the 2-, 5-, 10-, 25-, and 100-year return intervals. The Iowa Stormwater Management Manual was used to find the precipitation depth for the selected return intervals, which was then used in the triangular unit hydrograph calculations. This can be seen in Tables A5.10 - 14 in Appendix 5. These triangular hydrographs can also be found in the Appendix 5 in Figures A5.1 – A5.5. From the triangular unit hydrographs, an outflow hydrograph for a time interval of 30 minutes was found, which then helped to create the synthetic unit hydrograph (S-hydrograph). The S-hydrograph was used to find the 1-hour unit hydrograph. To find the amount of rainfall excess for the Washington, IA area, the precipitation frequency estimate for a 6-hr duration (which is typically used for design) for each recurrence interval (years) was found on the National Oceanic and Atmospheric Administration (NOAA) web database. The intensity data found can be seen in Table A5.15-A5.18 of the Appendix. The intensities were used to calculate the incremental excess rainfall, which was applied to the 1-hour unit hydrograph, and then the "design storm" was complete. The excess hyetographs and design storm direct runoff hydrographs for the 2, 5, 10, 25, and 100 year storms are shown in the Appendix in Figures A5.6-A5.10. This process produced values for the peak runoff rate and runoff volume for each return interval. The direct runoff hydrographs were then used later in final design to determine if the alternatives could handle the runoff for each of the return intervals.

The calculations, performed in Excel spreadsheets, can be seen in the Runoff Analysis section of the Appendix.

3. Preliminary Development of Alternative Solutions

A good stormwater design should develop a plan that is able to consider to treat stomwater, lower the flood frequency, river ecology, river geography, esthetics and recreation. Therefore the City of Washington then is able to develop an integrated planning and master design for the area. From the three alternatives we have suggested in the report, we recommended to use the in-line basin. Even though the in-line detention basins are not able to handle the 5-year storm due to the limited space that the City of Washington is given. However, it has highest storage capacity under the land space constrain which will reduce the strain of society problems in the local area because lower possibilities to have flood happened. In the future if the city wants to increase the capacity of the in-line basins, there is a potential area for the expanding basin in the second alternative is channel modification which is the cheapest alternative; however, this design is not able to have handle as much stormwater as the in-line detention basins design is going to have. However, by doing the channel modification the debris sediment will be less which improve the efficiency of the channel efficiency performance. Also we have put a bike trail along with the channel which increase both community recreation purpose and esthetics.The last option is having a detention basin in the Sesqui Park, which is the area that the city of Washington wants to develop. However, this design will be very expensive because it requires a pump system to route the water uphill as well as an operator to stand by and switch the pump on when it is going to flood.

Alternative 1 - In-Line Detention Basins

In line detention basin is one of the alternative that is chosen to be analysis in the report. The location of detention basin is usually in the lower elevation area because then the system will be able to use the preference of gravity. The main function of detention basin is to storage stormwater and reduce the peak discharge to achieve the goal of lowering flood disasters. The advantages of using in-line detention basin compares to the off-line detention basin is for the off-line detention basin usually a control valve at the outfall structural but the in-line detention basin do not have which makes the operation simpler. The disadvantage compare to the off-line is the in-line detention basin needs more area and it will not be able to improve the water quality.

“Detention basins are storage areas that gather stormwater and release the stored water gradually through an uncontrolled outlet. Similar to detention basins, retention basins are storage areas where there is no outlet, or the accumulated (impounded) water is stored for a long period of time. In typical applications, detention basins are used when there is limited infiltration ability on site. Otherwise, retention basins are favorable. Basins that continuously retain water are considered wet basins and those that do not have a permanent body of water are dry basins." (Chin, 2013).

The figure below shows the shapes and the location for the in-line detention basins that is suggested to be. The total surface area for the in-line detention basin is 180614 ft^2 and the total volume is 29.55 acre-ft.

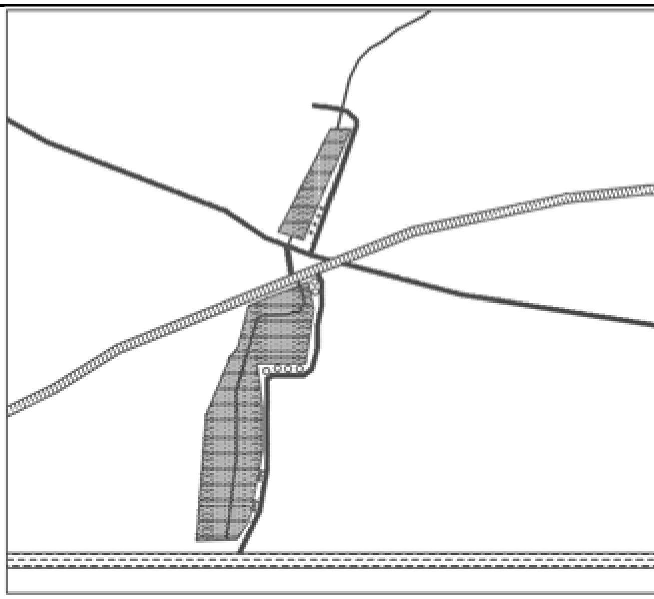

Figure 1 AutoCAD drawing for the design of in-line detention basin

Alternative 2 - Channel Modification

Alternative 2 implements a modification of the existing structures. The channel modification would be the cheapest and easiest of the three alternatives because it does not require as much excavation as the in-line detention basin alternative and does not require any expensive pumps such as the Sesqui Park detention basin alternative. The channel modification allows the channel to flow naturally as it did before modification while having a terrace like structure on either side to allow the channel to overflow and detain stormwater as it leads into the culvert running under West Main Street. As depicted in Figure 6 the flat area leading into the embankments on either side will act as a basin when the stormwater backs up during heavy rains. Also from Figure 7, it shows that the wide base allows the channel to flow and carve a natural path.

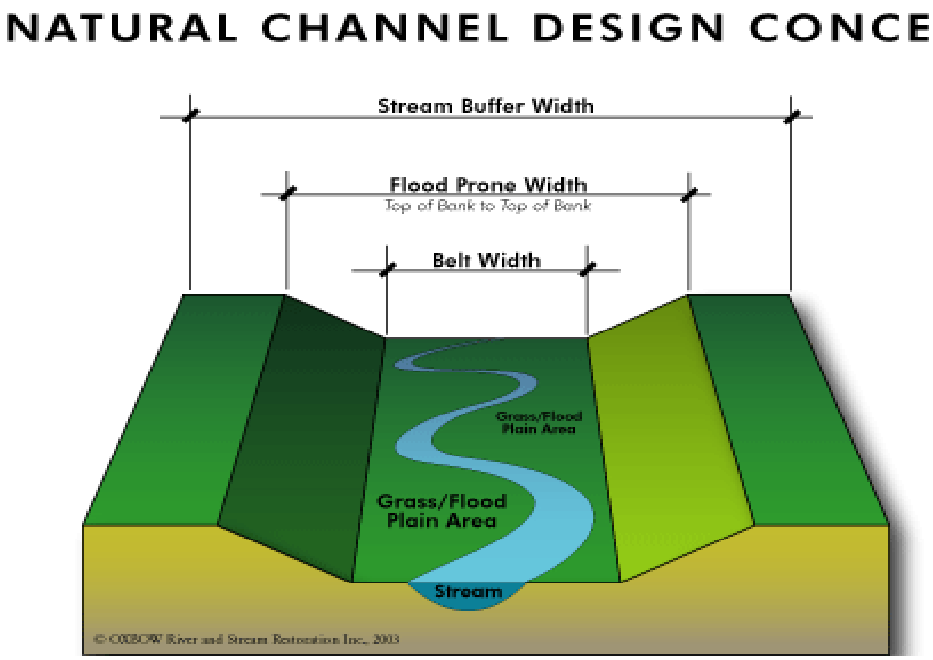

Figure 6: In-line channel modification example

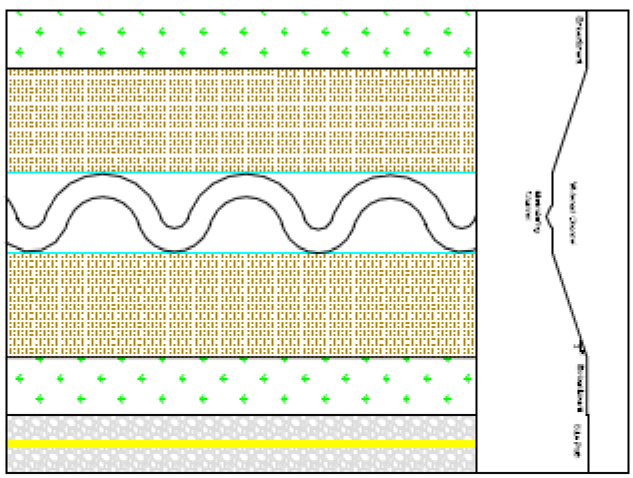

Figure 7: Overhead showing the natural channel

The site that this design would be implemented would be from West 5th Street to West Main Street. When designing the channel modification the same width was kept along the entire length of the channel. The width was 70 feet to match the narrowest point of the city's easements which are near West 5th Street.

The channel modification is not able to handle as much runoff as the city needs. This is because of the small area that is being worked with. On either side of the channel the city owns minimal amounts of land along the channel. So it is hard to handle as much runoff as the city needs without moving on to private property.

Alternative 3 – Off-line Detention Pound

The ideas for install off-line detention pound is similar to the in-line detention basin which is to hold temporarily stormwater and minimized the peak runoff; therefore to reduce the flood frequency. For the City of Washington, the main criteria in determining which type of storage basin should be used is if the amount of runoff can sustain a wet retention basin. If the amount of water flowing into the basin is not sufficient, the basin will become dry and not be aesthetically pleasing. If the water level is to be maintained, there is a need for either natural base flow or supplemental water (ISWMM, 2009).

The offline detention basin will locate at the Sesqui Park which is the area that City of Washington wants to develop. However, this location is at top of the hill; therefore, it requires a pump system to make this alternative to be feasible. In the design of detention basin it will include an inflow channel, pilot channel, micropool, emergency spillway and outlet structure. The detail drawing of off-line detention basin is shown in figure below. Beside of the advantage of alleviate the flooding frequency, off-line detention basin can also help to improve water quality up to some level.

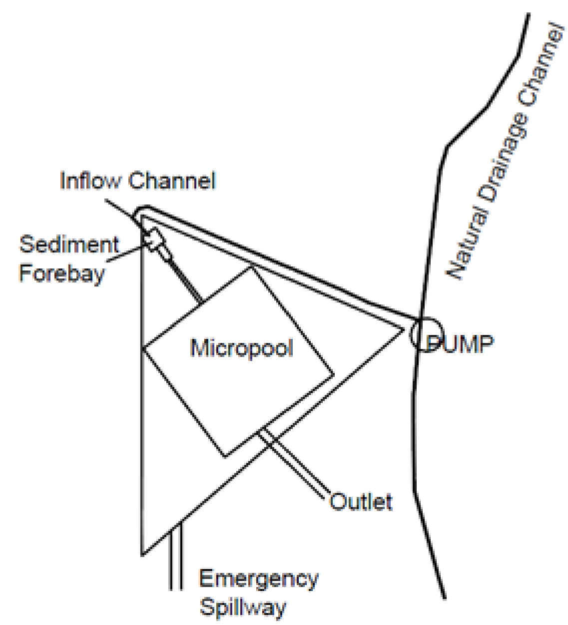

Figure 8: AutoCAD for the off-line detention basin drawing

4. Selection Process

Selection of the best alternative design is based off five factors: storage capacity, capital cost, aesthetic enhancements, environmental impacts, and expansion capabilities. The maximum amount of point attributed to each factor is based off how important that factor was in the decision-making process. Storage capacity has the greatest weight in the overall score because the City of Washington is most concerned with the mitigation of flooding over West Main Street. The in-line detention basin has the greatest capacity, followed by the detention basin in Sesqui Park, and then the channel modification alternative. The detention basin and channel modification alternatives are much cheaper than the in-line detention alternative due to extensive excavation. The aesthetic enhancements of each alternative are very similar, as well as the environmental impacts. Expansion capabilities are important to Washington administrators, due to the possibility of future development upstream. The detention basin in Sesqui Park has no expansion capability, due to property lines and bedrock. Channel modification can occur a bit further upstream, but it is still limited. In-line detention could be installed at various locations upstream, making it the most capable for expansion. The numerical distribution is found in the table below. The detention basin in Sesqui Park scored 19 out of 30 possible points, the channel modification scored 18, and the in-line detention scored 25. As a result, the in-line detention alternative is the recommended alternative.

Alternatives	Storage Capacity	Capital Cost	Aesthetic Enhancements	Environmental Impacts	Expansion Capabilities	Points Earned
Max. Points	17	5	2	1	5	30
Detention Basin in Sesqui	12	4	2	1	0	19
In-line Detention	16	1	2	1	5	25
Channel Modification	8	5	1	1	3	18

Table 1: Decision matrix

5. Final Design Details

The locations of the in-line detention basins are before street by the Sesqui Park and the area between the railroad and the culvert which is shown on the figures 9 and 10 below. The total areas of the in-line detention basins that is going to be 180614 ft^2 and 1287218 ft^3. The depth of the in-line are varies from 6 feet to 5 feet due to the elevation of the channel is changing. The total excavation soil of the detention basins is approximately 1,500,000 cubic yard. Those excavated will goes to build the embankments, besides using the excavated soil to build the embankment, there will also be some sod add into. The slope of the embankment is going to be 3.33% in slope and the top of the embankment is 10 ft wide. Foe the community recreation purpose some portion of the top of the embankment is going to be a bike trial.

Figure 9 location of the in-line detention basins before Sequi Park

Figure 10 location of in-line detention between railroad and culvert

The inflow rate and out flow rate is calculated, the results are shown in Appendix 5. Since the City of Washington provided limited spaces that are available to the stormwater management, and the undersized outlet infrastructure, the dimension of the outfall structure is a 3 feet diameter culvert. The in-line basins can handle more than 2-year storm but lower than 5-year storm event. The figure below shows the profile of the in-line detention is going to look like at the lower in-line detention basin.

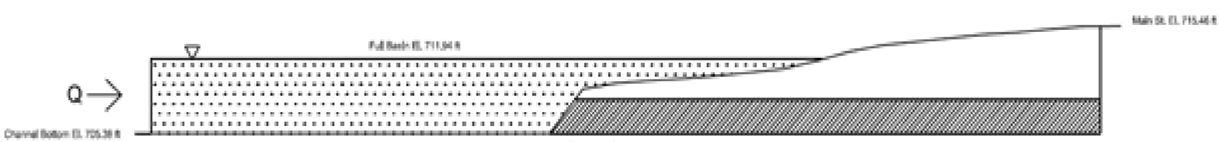

Figure 11 Profile of the lower in-line basin

6. Cost and Construction Estimates

Cost was one of the categories that helped make a decision on recommending the in-line detention basin alternative. As shown in Appendix 10, a major factor in all the cost estimating was the excavation of soil in the area. Though the cost of excavation for the in-line detention basin was greater than both the in-line channel modification and the Sesqui Park detention basin, the amount of water the alternative can hold is shown by the amount of excavation. The cost of excavation was then priced by the amount in cubic yards of soil excavated. This value was obtained in the RSMeans Heavy Construction Cost Data, 26th Annual Edition, as 0.65 cents per cubic yard which came out to be $958,667.00 for recommended alternative, $8,427.00 for the second alternative, and $68,250.00 for the third alternative.

Other costs taken into account for the in-line detention basin and the in-line channel modification where the costs of raising and redesigning the bike path that runs along the channel. The amount of asphalt used was the same in the in-line detention basin and the in-line channel modification since both required that the path be raised for the embankment. Using RSMeans Heavy Construction Cost Data, 26th Annual Edition, a unit price of $11.10 per square yard which came out to be $9835.00 with an area of 886 square yards of asphalt needed.

For the in-line detention basin and the in-line channel modification West 3rd Street is being raised to meet the height of the raised bike path to add as another embankment. West 3rd Street is made up of gravel and both the in-line detention basin and the in-line channel modification use the same amount of gravel in redesigning the street. RSMeans Heavy Construction Cost Data, 26th Annual Edition, was used to obtain a cost of gravel. The total estimated cost of gravel was $432.00 with a total volume of 52 cubic yards needed at a unit price of $8.30 per cubic yard.

For the detention basin alternative, the pump being used will be required to handle the runoff generated. For an accurate price on the pump the city will have to decide what brand or company they will want to go through to obtain the pump.

Also for all three alternatives the cost of labor was not included. To get an accurate estimate of labor and equipment costs the city will have to go through a contractor of their choice. Contractors differ in the amount they charge for certain types of jobs.

For the recommended in-line detention basin, there will be a set of steps that needs to be taken in order for the project to be completed in a timely fashion. First the permitting of the project will have to be taken care of above all else, along with a stormwater pollution prevention plan. This should take around 21 days to complete and obtain. Following the permitting comes the preparation of the project which entails putting up construction fencing and clearing the area needed of trees. This process should take 14 days. Finally heavy construction and landscaping will take place which entails all the excavation of soil, raising embankments, laying asphalt, raising the road, and laying down sod. This portion of the schedule will be the longest, lasting 210 days. Total the project will be underway for 245 days. Weather and conditions will vary the amount of time spent on certain stages of the project such as laying the asphalt or gravel if it is raining. For a more detailed schedule please see Appendix 10.

Table 2: In-line detention basin

Item No.	Description	Unit	Estimated Quantity	Unit Price	Estimated Amount
1	Excavation, soil	C.Y.	1,474,871	$ 0.65	$ 958,667.00
2	Asphalt	S.Y.	886	$ 11.10	$ 9,835.00
3	Gravel	C.Y.	52	$ 8.30	$ 432.00
4	Permits	years	1	$ 175.00	$ 175.00

Total Cost = $ 969,109.00

Table 3: Channel modification

Item No.	Description	Unit	Estimated Quantity	Unit Price	Estimated Amount
1	Excavation, soil	C.Y.	12964	$ 0.65	$ 8,427.00
2	Asphalt	S.Y.	886	$ 11.10	$ 9,835.00
3	Gravel	C.Y.	52	$ 8.30	$ 432.00
4	Permits	years	1	$ 175.00	$ 175.00

Total Cost = $ 18,869.00

Table 4: Sesqui Park detention basin

Item No.	Description	Unit	Estimated Quantity	Unit Price	Estimated Amount
1	Excavation, soil	C.Y.	105000	$ 0.65	$ 68,250.00
4	Permits	years	1	$ 175.00	$ 175.00

Total Cost = $ 68,425.00 (not including the cost of a pump)

7. Conclusion

Throughout this project there were many issues that made themselves known as alternatives were researched and designed. The city had originally wanted to make use of Sesqui Park as a stormwater control facility because of its size and limited use by the residents of Washington. The first three alternatives considered in this project all used Sesqui, in one way or another. These alternatives were a wetland that would use Sesqui as not only a stormwater management facility but a nature park as well, a detention basin that would have the potential to hold the most amount of water, and a channel that would run through Sesqui to try and slow the water down so the culvert had time to take in all of the water.

Upon further inspection of Sesqui it was discovered that not only did it sit higher than the surrounding land but the bedrock sat only 80 inches from the surface of the low point of Sesqui. This meant that the only way to detain water using Sesqui would be to use a stormwater pump to carry the water to a detention basin that could only go as deep as 80 inches lower than the low point. With the runoff that this part of Washington experiences a basin of that size would not be able to handle it all but would still help mitigate the flooding.

Including the pump as an alternative two more final alternatives were designed. One being a channel modification which would protect the surrounding properties from flooding but still would not be able to hold the amount of runoff that was needed. The final alternative was the in-line detention basin, which was the alternative that was recommended. This alternative was chosen because it was able to hold the most water while still be aesthetically pleasing. The cost for this alternative was on the higher side only because of the amount of earth that had to be excavated. This alternative was able to do a majority of what the City of Washington wanted. The alternative will be able to mitigate the flooding and keep a natural feel around the area.

The flooding problem is not going to disappear with this alternative however. The amount of runoff this design can handle is in between the two year and five year storm. To assist this alternative, another facility such as a basin should be implemented further upstream in the area Washington wants to use for urban development.

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9. Appendices

<u>Appendix 1: Watershed Delineation</u>

Appendix 1 outlines the procedure for delineating the watershed in ArcMap 10.1. The project file used to execute the watershed had the projection NAD_1983_UTM_Zone_15N.

Procedure:

1. Download the 1 meter Lidar digital elevation model (DEM) block for the area covering Sesqui Park, and the watershed it is part of. This data was retrieved from the Iowa Flood Center.
2. With the drawing tool, identify the low point on the elevation layer and create a feature point at that location. Convert the feature to layer and add the layer to map.
3. Use the Flow Direction tool (Spatial Analyst) to create a raster that represents flow direction. To do this, select the DEM as the input raster.
4. Use the Flow Accumulation tool (Spatial Analyst) to create a raster that represents the flow accumulation in each cell. To do this, select the flow direction raster as the input raster.
5. Use the Snap Pour Point tool (Spatial Analyst) to create a pour point integer raster that shows the point of great flow accumulation. To do this, select the accumulation raster as the input, in addition to the feature point layer created in step 2.
6. Finally, use the Watershed tool (Spatial Analyst) to delineate the watershed for the high point of accumulation in the area. To do this, select the flow direction raster, and pour point layer. The resulting layer is a polygon feature representing the watershed upstream from the accumulation location. The area of the watershed was then measured using the 'measure' tool.

Figure 1: The watershed in northeast Washington, extends over the proposed wellness center, area of flooding, and Sesqui Park

Appendix 2: Drainage Channel Cross-Sections

Appendix 2 is comprised of the cross-sections for various locations along the natural drainage channel running through the delineated watershed seen in Figures 2a-2j. Note that Cross-Sections 4 and 5 are of the street and adjacent slope. The locations for each cross-section is found in Figure 2. The procedure for the extraction of cross-sections from ArcMap can be found in Appendix 3.

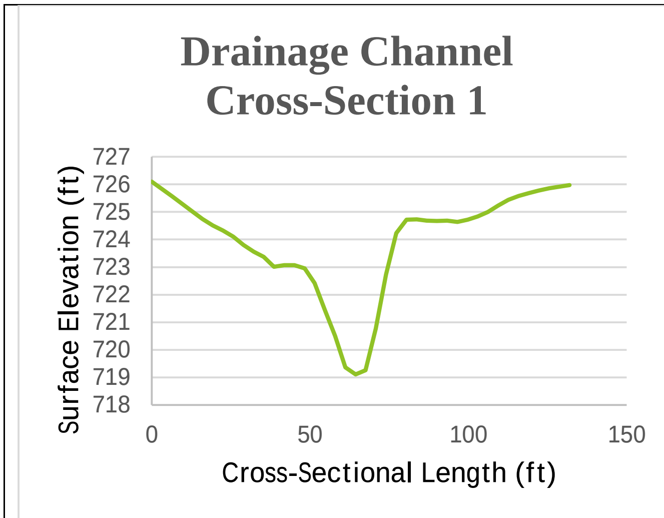


Figure 2a.

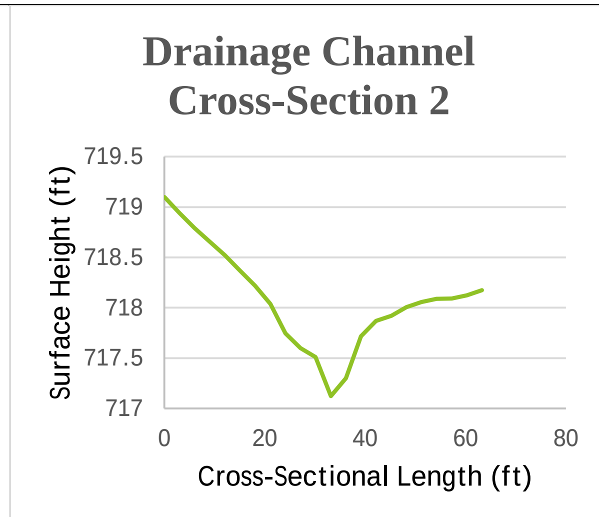


Figure 2b.

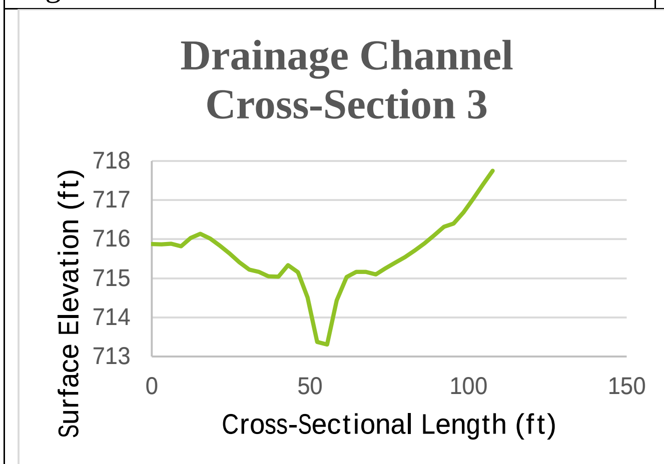


Figure 2c.

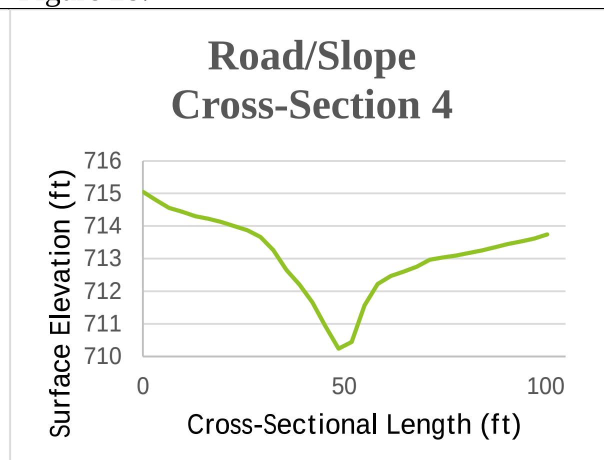


Figure 2d.

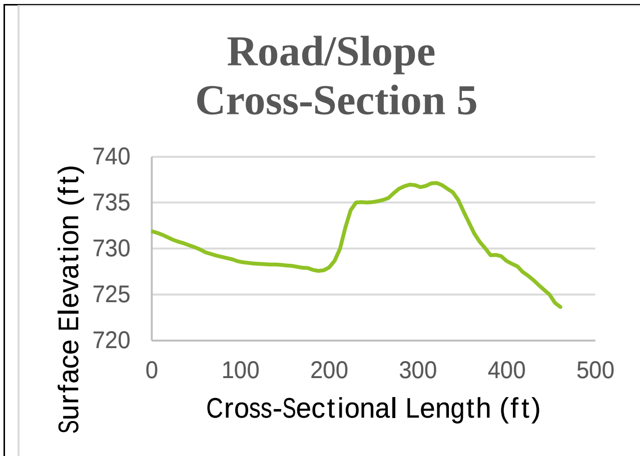


Figure 2e.

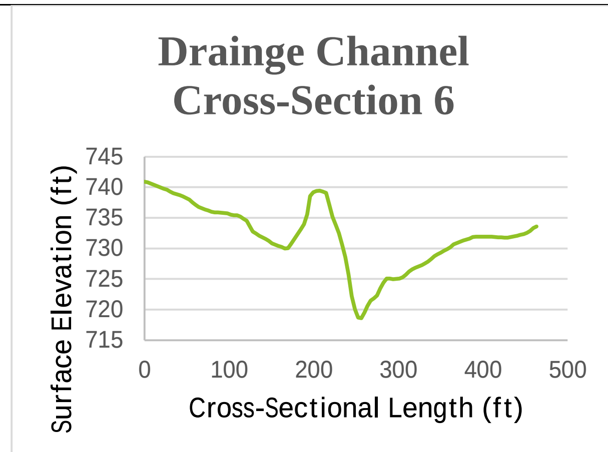


Figure 2f.

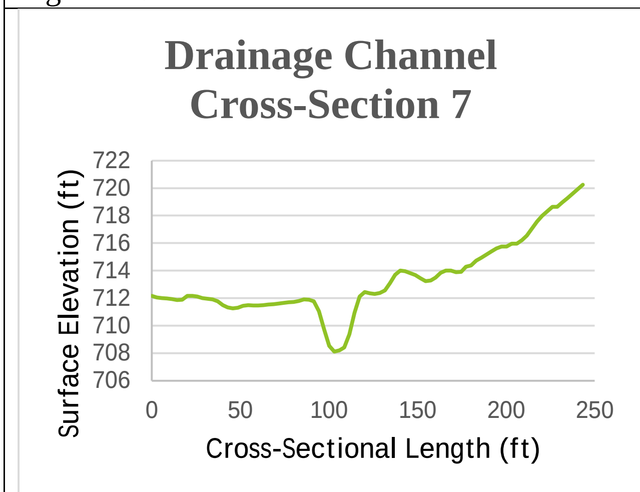


Figure 2g.

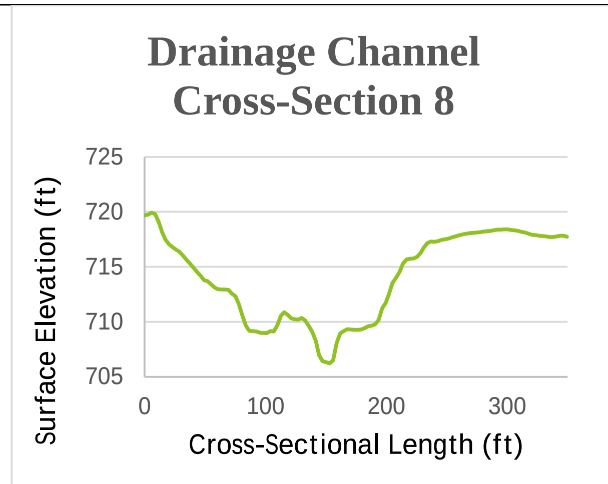


Figure 2h.

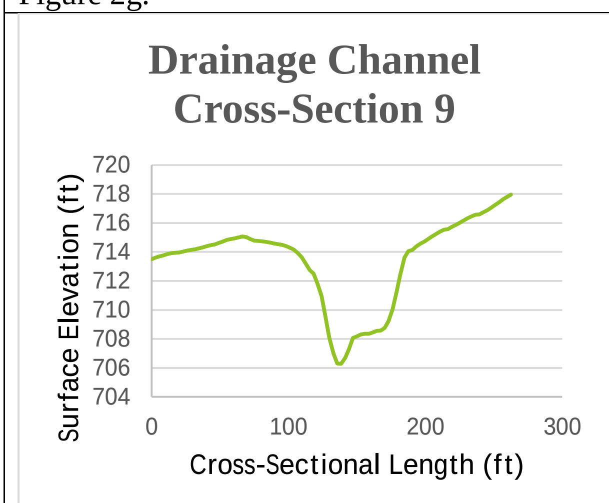


Figure 2i.

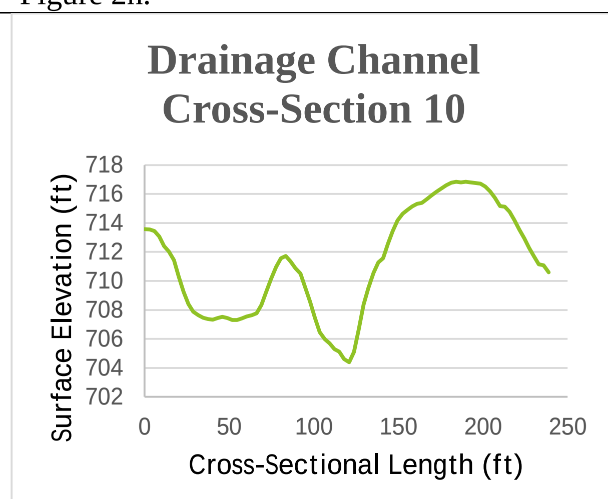


Figure 2j.

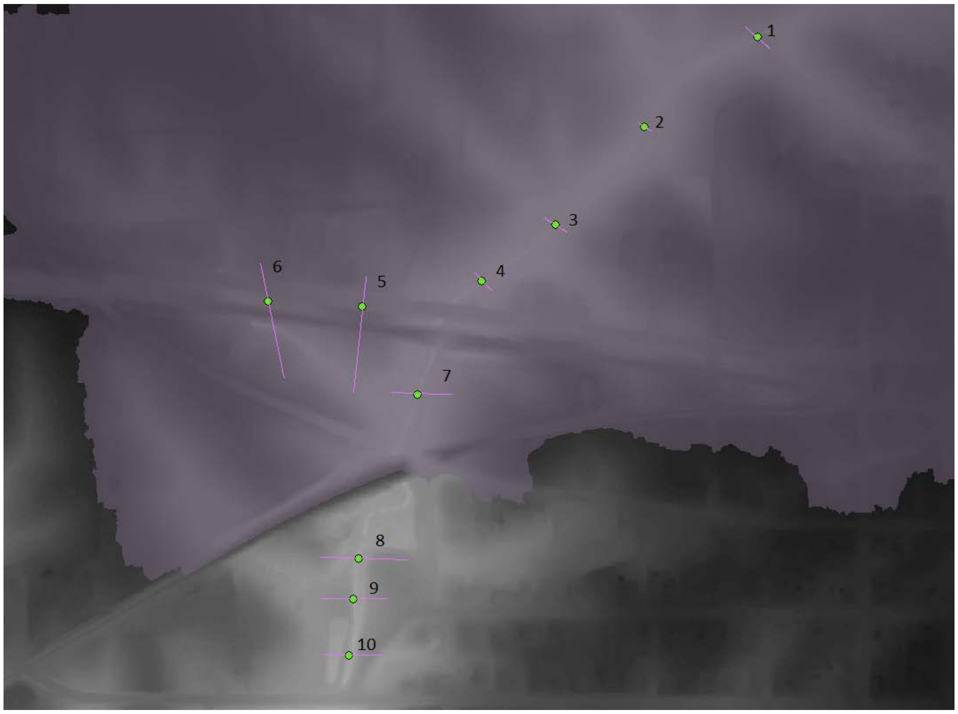


Figure 3: The location of each cross-section extracted for the runoff analysis

Appendix 3: Cross-Section Extraction

Appendix 2 outlines the procedure for extraction cross-sections from digital elevation models in ArcMap 10.1. The project file used to execute the watershed had the projection NAD_1983_UTM_Zone_15N.

Procedure:

1. Using the 3D Analyst tool, select the DEM layer to work with.
2. Select the ‘interpolate line’ button, then draw a line across the desired cross-section location. (Note: all lines were drawn from left to right, or top to bottom.)
3. After drawing the line, select the ‘profile graph’ button from the 3D Analyst toolbar.
4. A graph appears, right click on the graph, and then click export. Go to the data tab, select text for the format, then click the button that says copy.
5. Open excel and paste the text into the excel document. This gives two columns, one for the length of the line, and the other for the surface elevation.
6. Using excel, graph the data, with the length on the x-axis and elevation on the y-axis. Each of these graphs can be found in Appendix 2.

Appendix 4: Sesqui Park Cross-Sections

Appendix 4 contains the cross-sections used to evaluate the surface elevation in Sesqui Park. The procedure used to extract is found in Appendix 3.

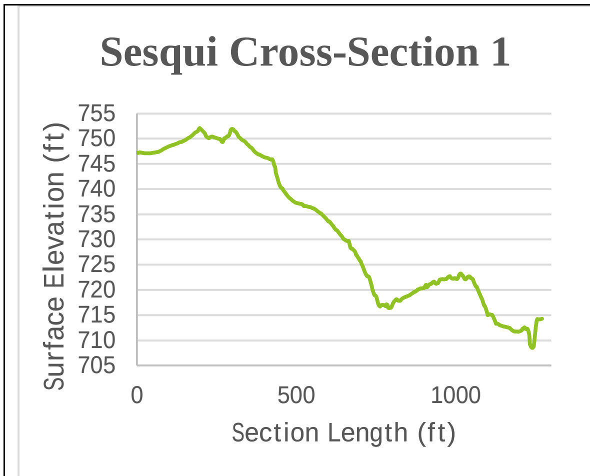


Figure 4a.

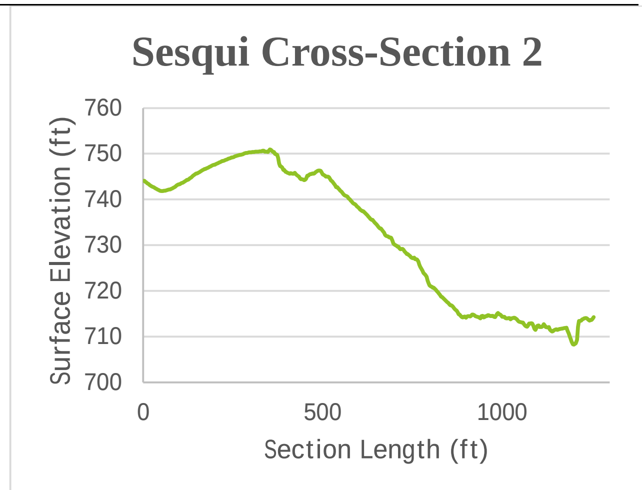


Figure 4b.

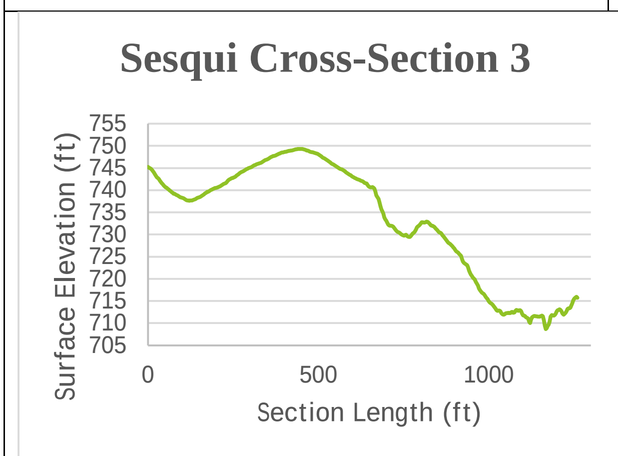


Figure 4c.

Figure 4d.

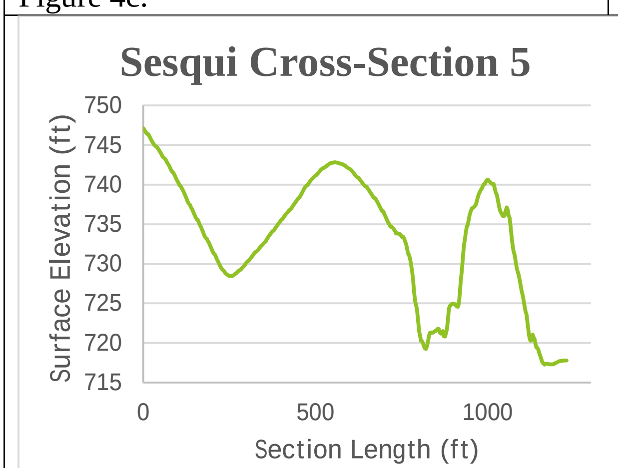


Figure 4e.

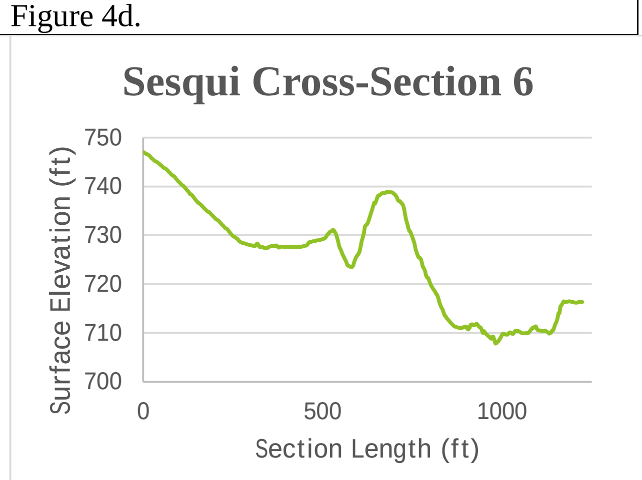
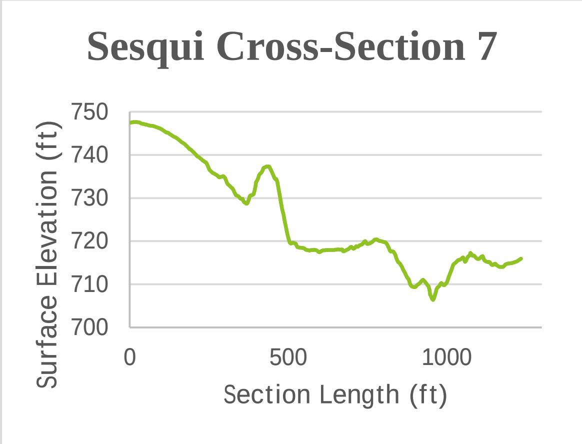
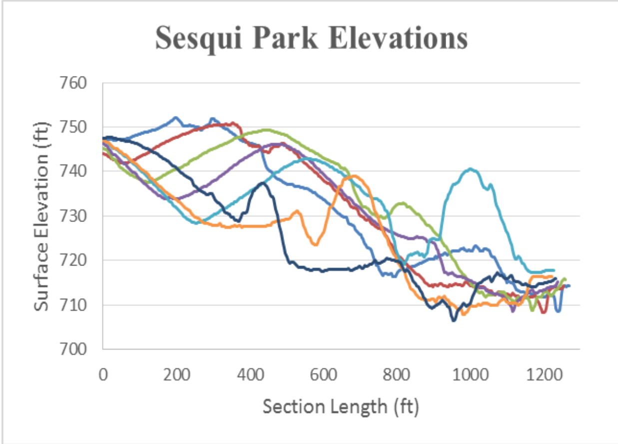


Figure 4f.

 Figure 4g.	 Figure 4h: This graph shows all the Sesqui cross-sections in one graph.

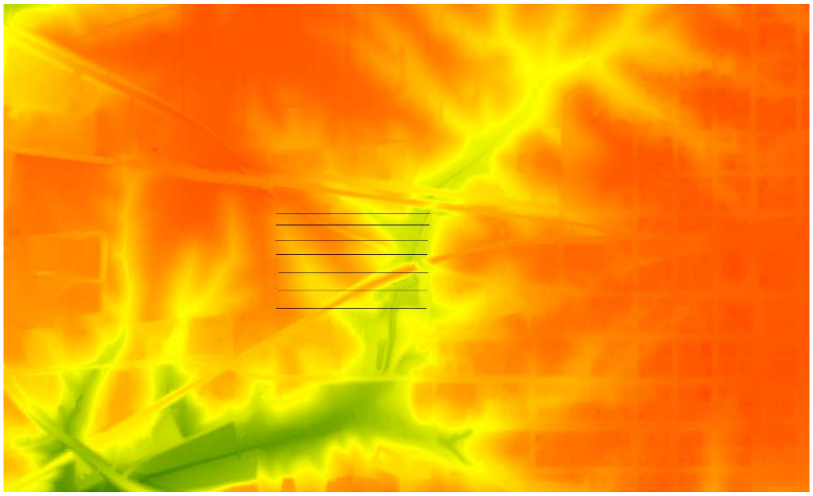


Figure 4i: This image shows the locations of the cross-sections that were extracted. Cross-sections are numbered from top to bottom and drawn from left to right

Appendix 5: Runoff Analysis

The Curve Numbers used in the runoff analysis were found in the Iowa Stormwater Management Manual and were based on the watershed make-up. The watershed classification was estimated using the percent of the entire area that each land use/cover occupies. The summary of these Curve Numbers can be seen in Tables A5.1-A5.4. The sub-watershed divisions were then used to estimate the total (cumulative) CN for the entire basin.

Table A5.1: Sub-watershed 1 Curve Number

Sub-watershed 1			
Land use	Area (ft^2)	% Area	CN
residential (1/4 acre)	8206656.6	75	83
Farm Land	1860175.5	17	79
Paved street	875376.7	8	92.5
Total	10942209		**83.08**

Table A5.2: Sub-watershed 2 Curve Number

Sub-watershed 2 (Post Park Development)			
Land use	Area (ft^2)	% Area	CN
residential (1/4 acre)	1354132.7	15	83
Farm Land	7644637.6	84.6812	79
Paved street	902.75515	0.01	92.5
Park	2787840	0.3088	87.5
Total	9027551.5		**79.628**

Table A5.3: Sub-watershed 3 Curve Number

Sub-watershed 3			
Land use	Area (ft^2)	% Area	CN
residential (1/4 acre)	89201.053	1	83
Farm Land	8830904.2	99	79
Paved street	0	0	92.5
Total	8920105.3		**79.04**

Table A5.4: Sub-watershed 3 Curve Number

Sub-watershed 4			
Land use	Area (ft^2)	% Area	CN
Open Space (fair)	283316.25	25	79
Woods (fair)	283316.25	25	73
Residential (1/4 acre)	566632.5	50	83
Total	1133265		**79.5**

The cumulative Curve Number was found to be 80.7, which was then used in the calculations for runoff volume and the triangular unit hydrograph.

Tables A5.5-A5.9 show the calculated runoff volume over the entire basin based on the cumulative precipitation found in the Iowa Stormwater Management Manual for each of the chosen return intervals (2, 5, 10, 25, 100-yr).

Table A5.5: 2-yr Runoff Volume (inches)

2-yr (total basin)		
Cumulative CN	80.71	
Cumulative S	2.39	in
Ia	0.48	in
P	3.14	in
Q	1.40	in

Table A5.6: 5-yr Runoff Volume (inches)

5-yr (total basin)		
Cumulative CN	80.71	
Cumulative S	2.39	in
Ia	0.48	in
P	4.03	in
Q	2.12	in

Table A5.7: 10-yr Runoff Volume (inches)

10-yr (total basin)		
Cumulative CN	80.71	
Cumulative S	2.39	in
Ia	0.48	in
P	4.67	in
Q	2.67	in

Table A5.8: 25-yr Runoff Volume (inches)

25-yr (total basin)		
Cumulative CN	80.71	
Cumulative S	2.39	in
Ia	0.48	in
P	5.67	in
Q	3.55	in

Table A5.9: 100-yr Runoff Volume (inches)

100-yr (total basin)		
Cumulative CN	80.71	
Cumulative S	2.39	in
Ia	0.48	in
P	7.59	in
Q	5.32	in

Tables A5.10 – A5.14 show the calculations for the triangular unit hydrographs for each return interval. The NCRS method was used to find the parameters for the unit hydrograph. Including what was calculated for the runoff volumes.

Table A5.10: 2-yr Triangular Unit Hydrograph Calculations

2-yr Triangular Unit Hydrograph		
L=	6335.3	ft
Y	0.135	%
Area=	1.077	mi2
tl= (L^0.8*(S+1.00)^0.7)/(1900*(Y)^0.5)	3.704121293	hr
tc=(tl/0.6)	6.173535488	hr
tp=(2/3)tc	4.115690326	hr
Q volume	1.402409405	inches
qp=726AQ/tc	177.6205418	cfs
tr =2(tp-tl)	0.823138065	hr
tb=8/3tp	10.9751742	hr

Table A5.11: 5-yr Triangular Unit Hydrograph Calculations

5-yr Triangular Unit Hydrograph		
L=	6335.3	ft
Y	0.135	%
Area=	1.077	mi2
tl= (L^0.8*(S+1.00)^0.7)/(1900*(Y)^0.5)	3.704121293	hr
tc=(tl/0.6)	6.173535488	hr
tp=(2/3)tc	4.115690326	hr
Q volume inches	2.123004044	inches
qp=726AQ/tc	268.8866228	cfs
tr =2(tp-tl)	0.823138065	hr
tb=8/3tp	10.9751742	hr

Table A5.12: 10-yr Triangular Unit Hydrograph Calculations

10-yr Triangular Unit Hydrograph		
L=	6335.3	ft
Y	0.135	%
Area=	1.077	mi2
tl= (L^0.8*(S+1.00)^0.7)/(1900*(Y)^0.5)	3.704121293	hr
tc=(tl/0.6)	6.173535488	hr
tp=(2/3)tc	4.115690326	hr
Q volume inches	2.669499681	inches
qp=726AQ/tc	338.1023958	cfs
tr =2(tp-tl)	0.823138065	hr
tb=8/3tp	10.9751742	hr

Table A5.13: 25-yr Triangular Unit Hydrograph Calculations

25-yr Triangular Unit Hydrograph		
L=	6335.3	ft
Y	0.135	%
Area=	1.077	mi2
tl= (L^0.8*(S+1.00)^0.7)/(1900*(Y)^0.5)	3.704121293	hr
tc=(tl/0.6)	6.173535488	hr
tp=(2/3)tc	4.115690326	hr
Q volume inches	3.554998985	inches
qp=726AQ/tc	450.2542865	cfs
tr =2(tp-tl)	0.823138065	hr
tb=8/3tp	10.9751742	hr

Table A5.14: 100-yr Triangular Unit Hydrograph Calculations

100-yr Triangular Unit Hydrograph		
L=	6335.3	ft
Y	0.135	%
Area=	1.077	mi2
tl= (L^0.8*(S+1.00)^0.7)/(1900*(Y)^0.5)	3.704121293	hr
tc=(tl/0.6)	6.173535488	hr
tp=(2/3)tc	4.115690326	hr
Q volume inches	5.322712403	inches
qp=726AQ/tc	674.1419858	cfs
tr =2(tp-tl)	0.823138065	hr
tb=8/3tp	10.9751742	hr

Graphs showing the triangular unit hydrographs for each return interval can be seen in Figures A5.1-A5.5.

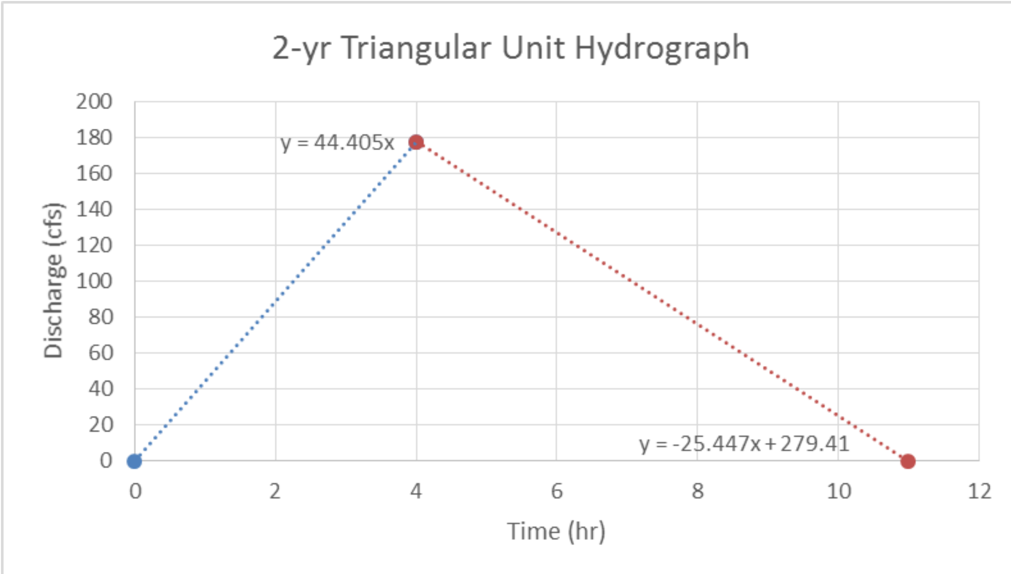


Figure A5.1: 2-yr Triangular Unit Hydrograph

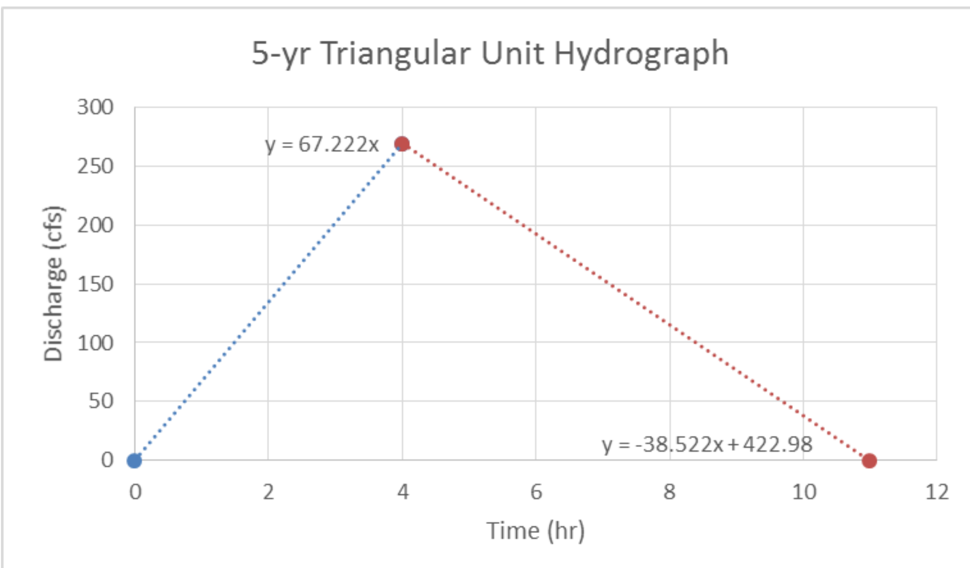


Figure A5.2: 5-yr Triangular Unit Hydrograph

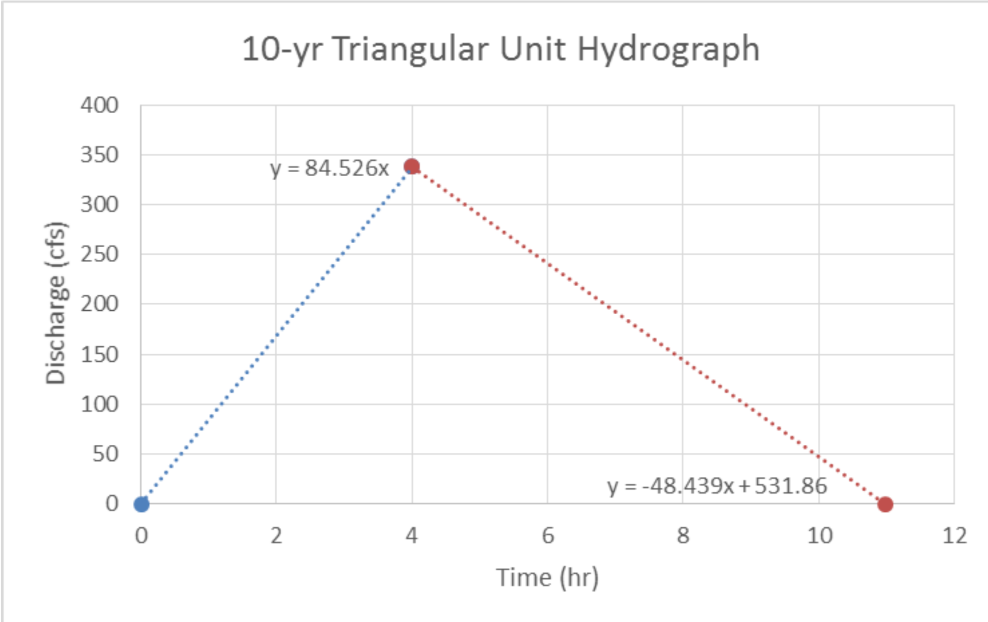


Figure A5.3: 10-yr Triangular Unit Hydrograph

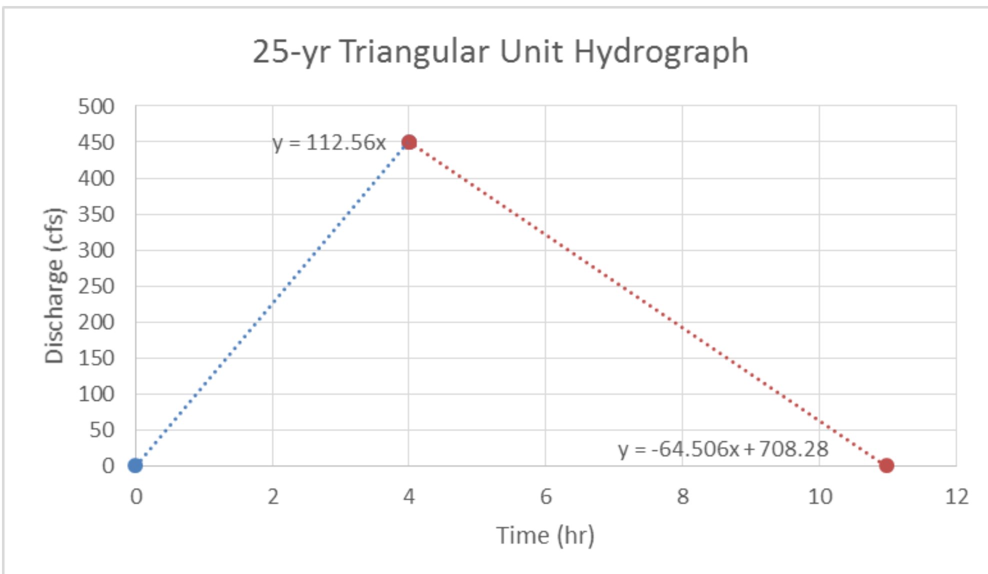


Figure A5.4: 25-yr Triangular Unit Hydrograph

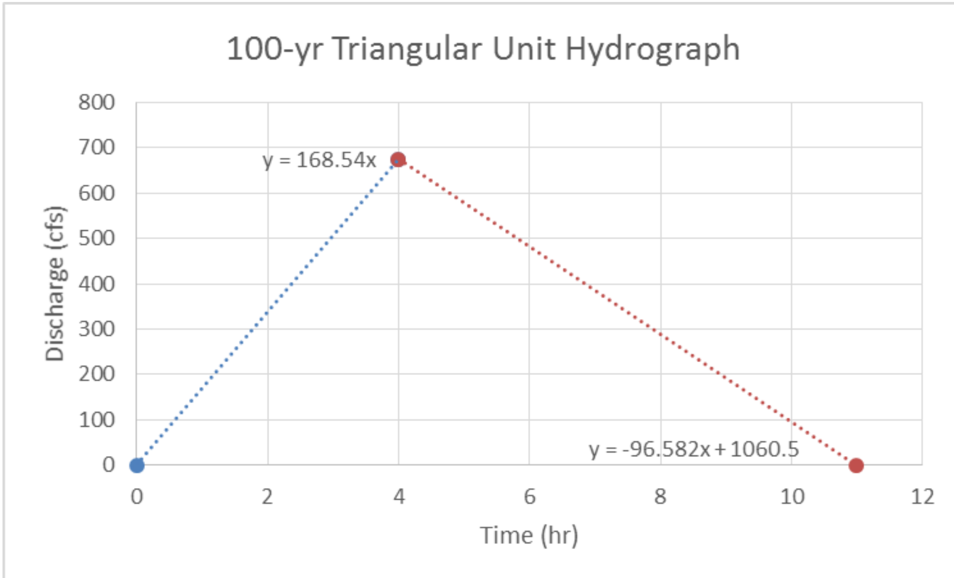


Figure A5.5: 100-yr Triangular Unit Hydrograph

Using the NRCS approach, the excess rainfall hyetographs were found. The NOAA data for frequency for the 6-hr storm were used for each return interval in the excess calculations. This data can be found in Table A5.20. Tables A5.15 - A5.19 show the calculations and Figures A5.6-A5.10 show the hyetographs for each return interval.

Table A5.15: 2-yr Excess Hyetograph Calculations

t (hr)	Intensity (in/hr)	Incremental P (in)	Cumulative P (in)	Ia (in)	Excess (in)	Incremental Excess (in)
0			0	0.478116311	0	
	0.17	0.17				0
1			0.17	0.478116311	0	
	0.38	0.38				0.002098411
2			0.55	0.478116311	0.002098411	
	0.55	0.55				0.126281268
3			1.1	0.478116311	0.12837968	
	0.7	0.7				0.342298528
4			1.8	0.478116311	0.470678208	
	0.4	0.4				0.250272186
5			2.2	0.478116311	0.720950394	
	0.11	0.11				0.07379809
6			2.31	0.478116311	0.794748484	

Table A5.16: 5-yr Triangular Unit Hydrograph Calculations

t (hr)	Intensity (in/hr)	Incremental P (in)	Cumulative P (in)	Ia (in)	Excess (in)	Incremental Excess (in)
0			0	0.478116311	0	
	0.19	0.19				0
1			0.19	0.478116311	0	
	0.41	0.41				0.005912772
2			0.6	0.478116311	0.005912772	
	0.72	0.72				0.21335274
3			1.32	0.478116311	0.219265512	
	0.96	0.96				0.555167821
4			2.28	0.478116311	0.774433332	
	0.52	0.52				0.369584414
5			2.8	0.478116311	1.144017746	
	0.12	0.12				0.08988579
6			2.92	0.478116311	1.233903536	

Table A5.17: 10-yr Triangular Unit Hydrograph Calculations

t (hr)	Intensity (in/hr)	Incremental P (in)	Cumulative P (in)	Ia (in)	Excess (in)	Incremental Excess (in)
0			0	0.478116311	0	
	0.2	0.2				0
1			0.2	0.478116311	0	
	0.43	0.43				0.009073341
2			0.63	0.478116311	0.009073341	
	0.82	0.82				0.271838977
3			1.45	0.478116311	0.280912318	
	1.1	1.1				0.681045207
4			2.55	0.478116311	0.961957525	
	0.7	0.7				0.526350642
5			3.25	0.478116311	1.488308168	
	0.22	0.22				0.174752829
6			3.47	0.478116311	1.663060996	

Table A5.18: 25-yr Triangular Unit Hydrograph Calculations

t (hr)	Intensity (in/hr)	Incremental P (in)	Cumulative P (in)	Ia (in)	Excess (in)	Incremental Excess (in)
0			0	0.478116311	0	
	0.13	0.13				0
1			0.13	0.478116311	0	
	0.55	0.55				0.015721339
2			0.68	0.478116311	0.015721339	
	1.11	1.11				0.449114575
3			1.79	0.478116311	0.464835914	
	1.5	1.5				1.054960858
4			3.29	0.478116311	1.519796772	
	0.86	0.86				0.704171414
5			4.15	0.478116311	2.223968186	
	0.15	0.15				0.127239324
6			4.3	0.478116311	2.35120751	

Table A5.19: 100-yr Triangular Unit Hydrograph Calculations

t (hr)	Intensity (in/hr)	Incremental P (in)	Cumulative P (in)	Ia (in)	Excess (in)	Incremental Excess (in)
0			0	0.478116311	0	
	0.37	0.37				0
1			0.37	0.478116311	0	
	0.84	0.84				0.171548342
2			1.21	0.478116311	0.171548342	
	1.49	1.49				0.89876154
3			2.7	0.478116311	1.070309882	
	1.77	1.77				1.426395511
4			4.47	0.478116311	2.496705392	
	0.96	0.96				0.842929357
5			5.43	0.478116311	3.339634749	
	0.31	0.31				0.278469885
6			5.74	0.478116311	3.618104634	

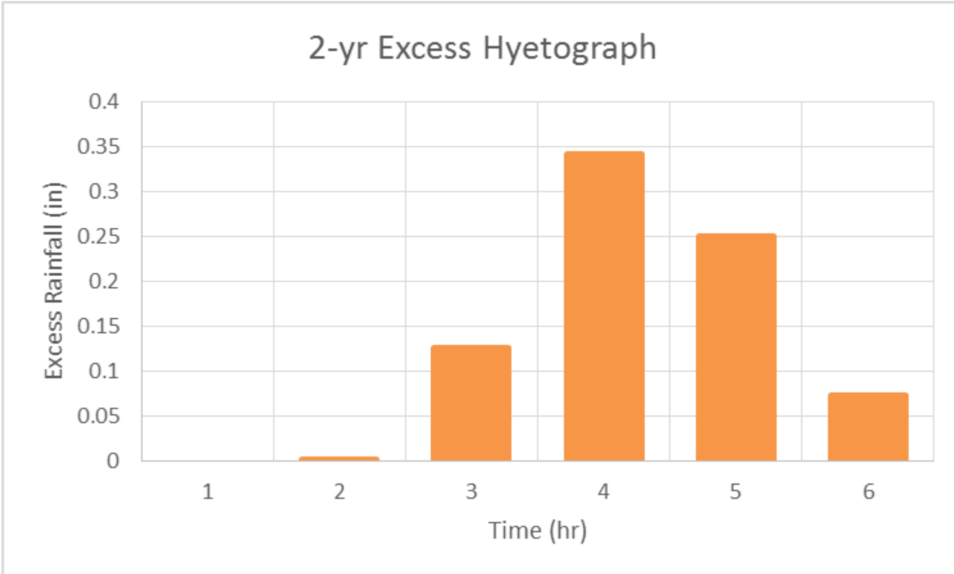


Figure A5.6: 2-yr Excess Hyetograph

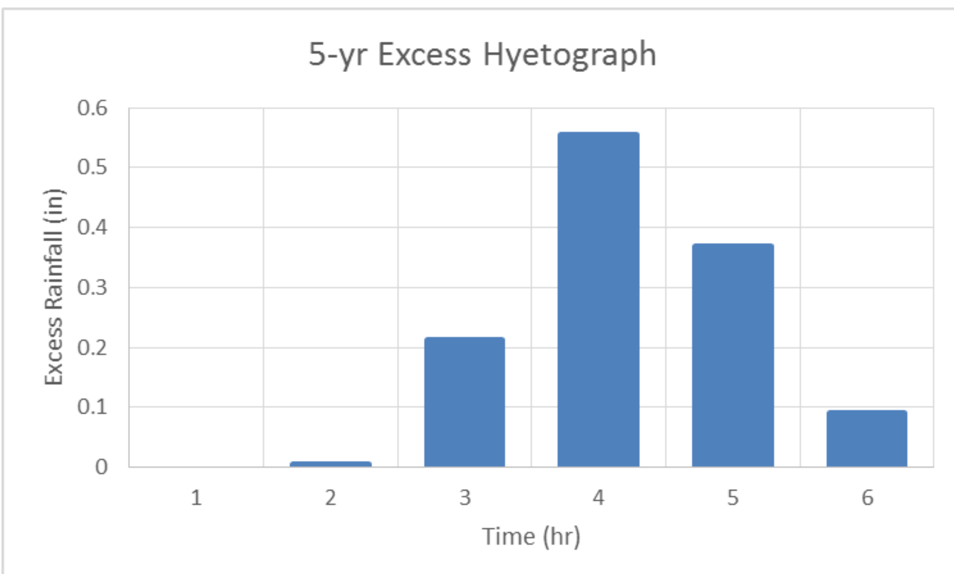


Figure A5.7: 5-yr Excess Hyetograph

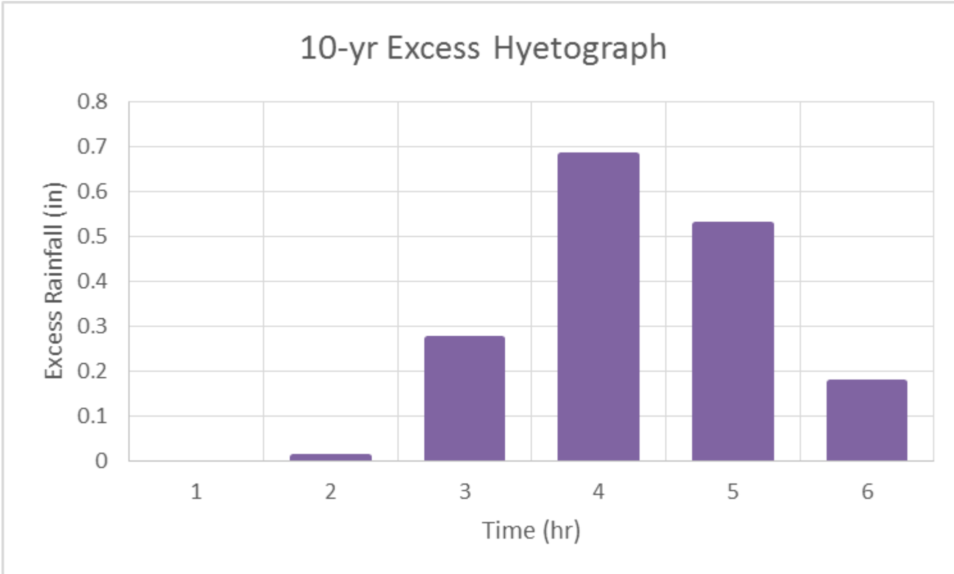


Figure A5.8: 10-yr Excess Hyetograph

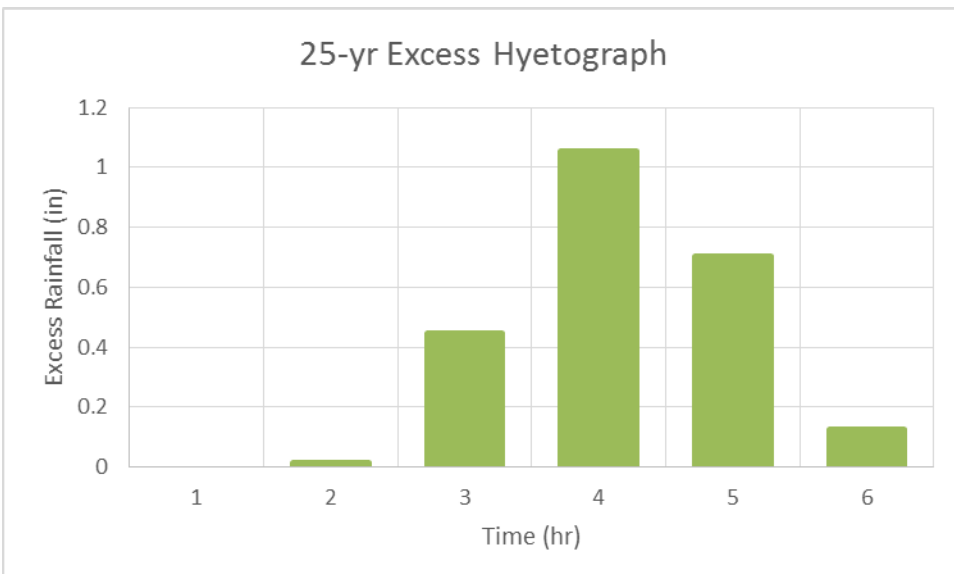


Figure A5.9: 25-yr Excess Hyetograph

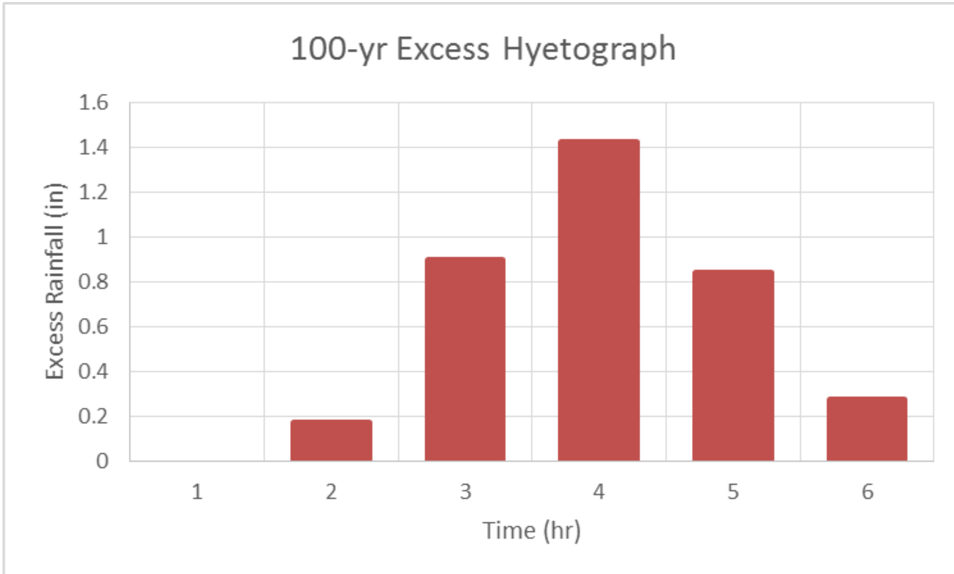


Figure A5.10: 100-yr Excess Hyetograph

After the excess hyetographs were found, the direct runoff hydrographs (DRH) were calculated and graphed. This was done using the Synthetic Unit Hydrograph Method and then applying the excess hyetograph to the S-Hydrograph produced. These calculations for the 2-yr return interval can be seen in Tables A5.23 - A5.24. The steps seen in those calculations were repeated for all chose return intervals. The DRHs can be seen in Table A5.19 and Figures A5.11 – A5.15.

Table A5.19: Calculated DRHs (cfs)

t (hr)	2-yr DRH	5-yr DRH	10-yr DRH	25-yr DRH	100-yr DRH
0	0.00	0.00	0.00	0.00	0.00
0.5	0.00	0.00	0.00	0.00	0.00
1	0.02	0.10	0.19	0.44	7.23
1.5	1.47	3.88	6.32	13.97	59.55
2	8.12	20.58	32.58	69.81	209.85
2.5	21.35	52.82	84.36	175.16	455.76
3	38.18	92.79	150.95	303.91	748.93
3.5	55.82	134.26	221.24	436.23	1053.83
4	73.47	175.73	291.53	568.56	1358.72
4.5	91.11	217.20	361.81	700.89	1663.62
5	108.72	258.52	431.80	832.48	1957.15
5.5	124.09	294.04	492.44	942.49	2179.74
6	131.27	309.25	521.41	983.47	2248.23
6.5	128.11	300.00	510.25	945.63	2166.31
7	119.29	278.62	475.78	872.63	2010.05
7.5	109.18	254.85	435.50	795.37	1835.33
8	99.06	231.08	395.22	718.36	1660.61
8.5	88.95	207.32	354.95	641.35	1485.88
9	78.84	183.55	314.67	564.34	1311.16
9.5	68.73	159.78	274.39	487.33	1136.44
10	58.62	136.02	234.11	410.32	961.72
10.5	48.50	112.25	193.83	333.31	787.00
11	38.39	88.49	153.55	256.30	612.28
11.5	28.28	64.72	113.28	179.39	437.72
12	18.21	41.10	73.25	105.50	268.15
12.5	9.05	19.79	36.94	45.60	125.63
13	2.90	5.95	12.30	11.89	39.19
13.5	0.45	0.83	2.03	1.30	6.46
14	0.00	0.00	0.00	0.00	0.00
Total DR (cfs)	1550.19	3643.52	6174.69	11396.03	26786.55
Total Volume (ft3)	2790336.10	6558337.71	11114439.37	20512858.34	48215784.97
Total Volume (ac-ft)	64.06	150.56	255.15	470.91	1106.88

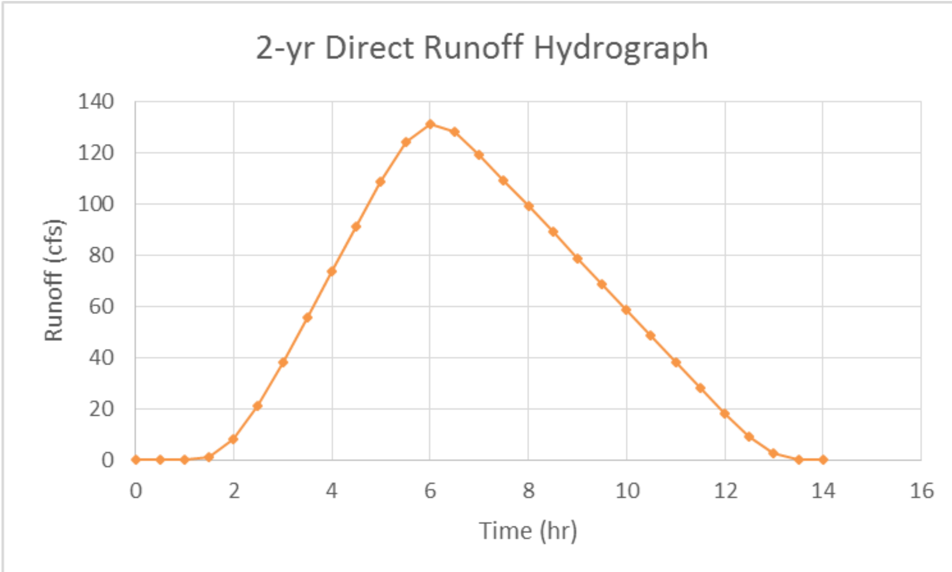


Figure A5.11: 2-yr Excess Hyetograph

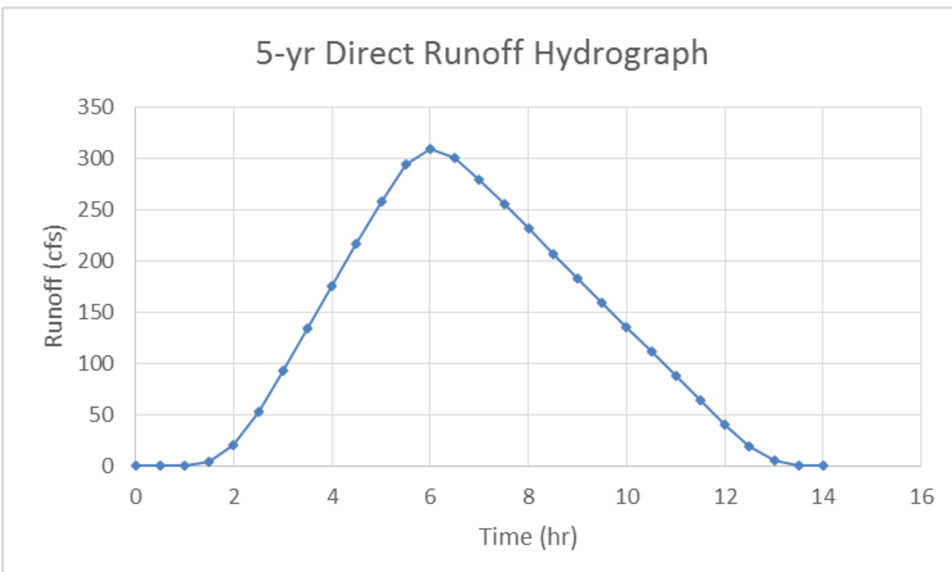


Figure A5.12: 5-yr Excess Hyetograph

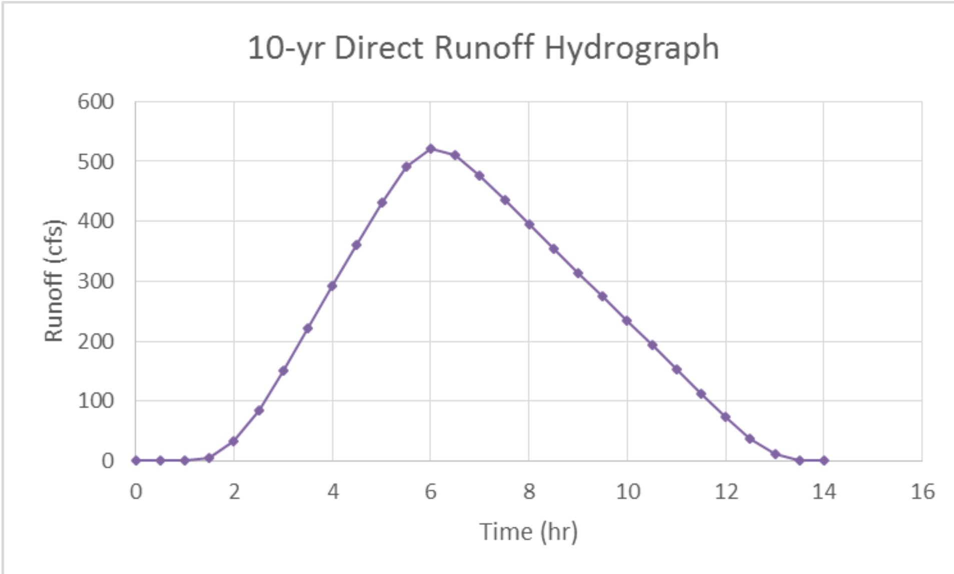


Figure A5.13: 10-yr Excess Hyetograph

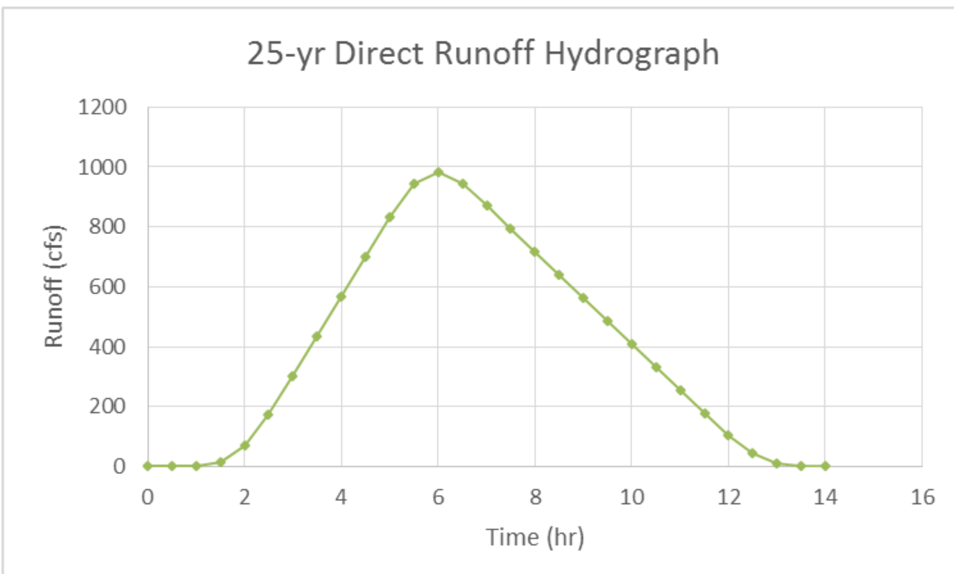


Figure A5.14: 100-yr Excess Hyetograph

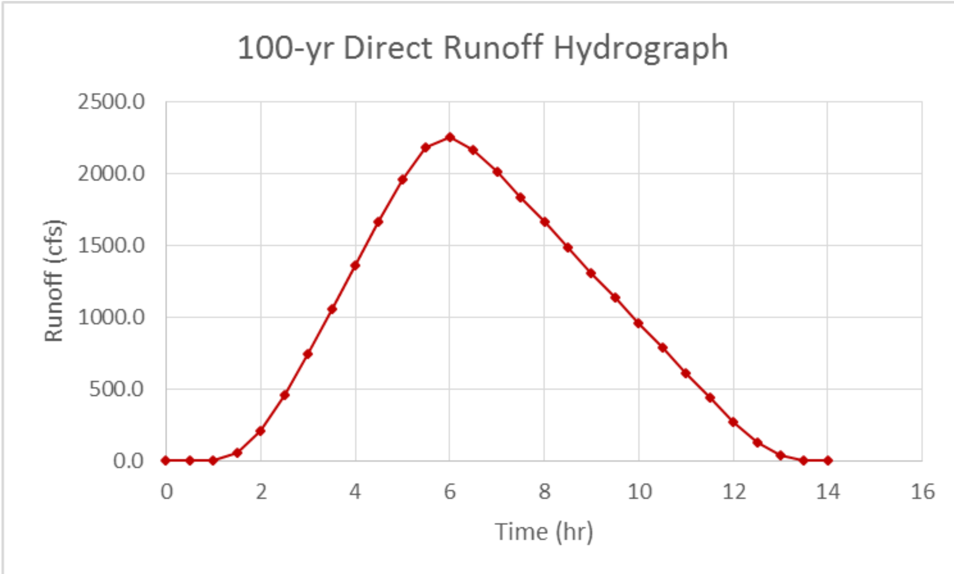


Figure A5.15: 100-yr Excess Hyetograph

After the DRHs were calculated, the Modified Puls Method was used to determine the outflow hydrographs for the given storms for Alternative 1, the in-line detention basins. The 2-yr storm was calculated first to see if the design of the basins could handle the required runoff of the smallest, most frequent storm. The analysis calculations can be seen in Table A5.21 and Figures A5.16 and Figure A5.17 show the method used to find the outflow hydrograph and the total storage required, which is the area between the inflow and outflow hydrographs (see Tables A5.21 and A5.22 for the calculated required storage. This process was repeated for the 5-yr storm and the calculations and results for this storm can be seen in Table A5.22 and Figure A5.18.

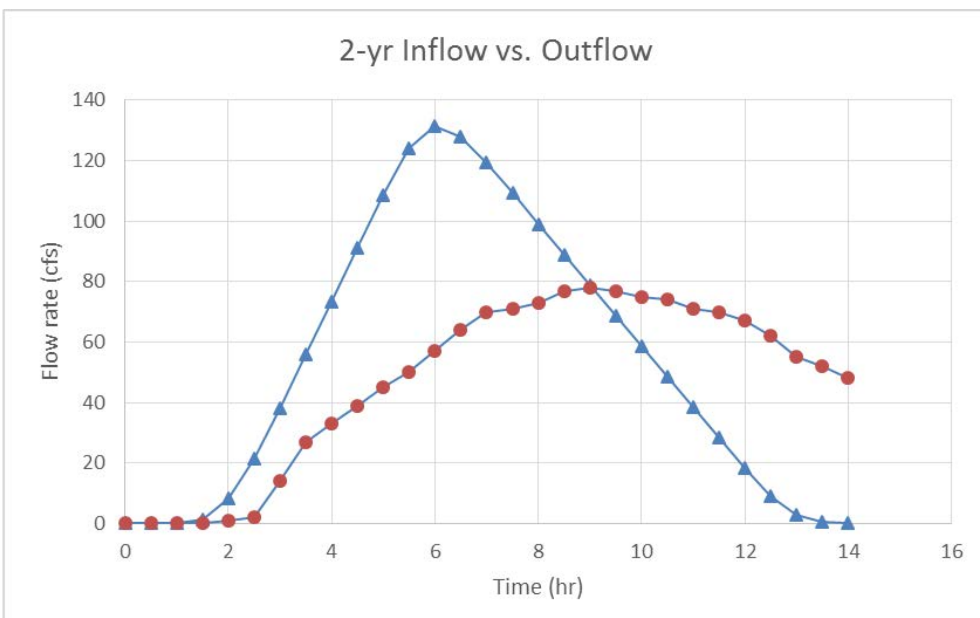


Figure A5.17: 2-yr Inflow vs. Outflow Relationship

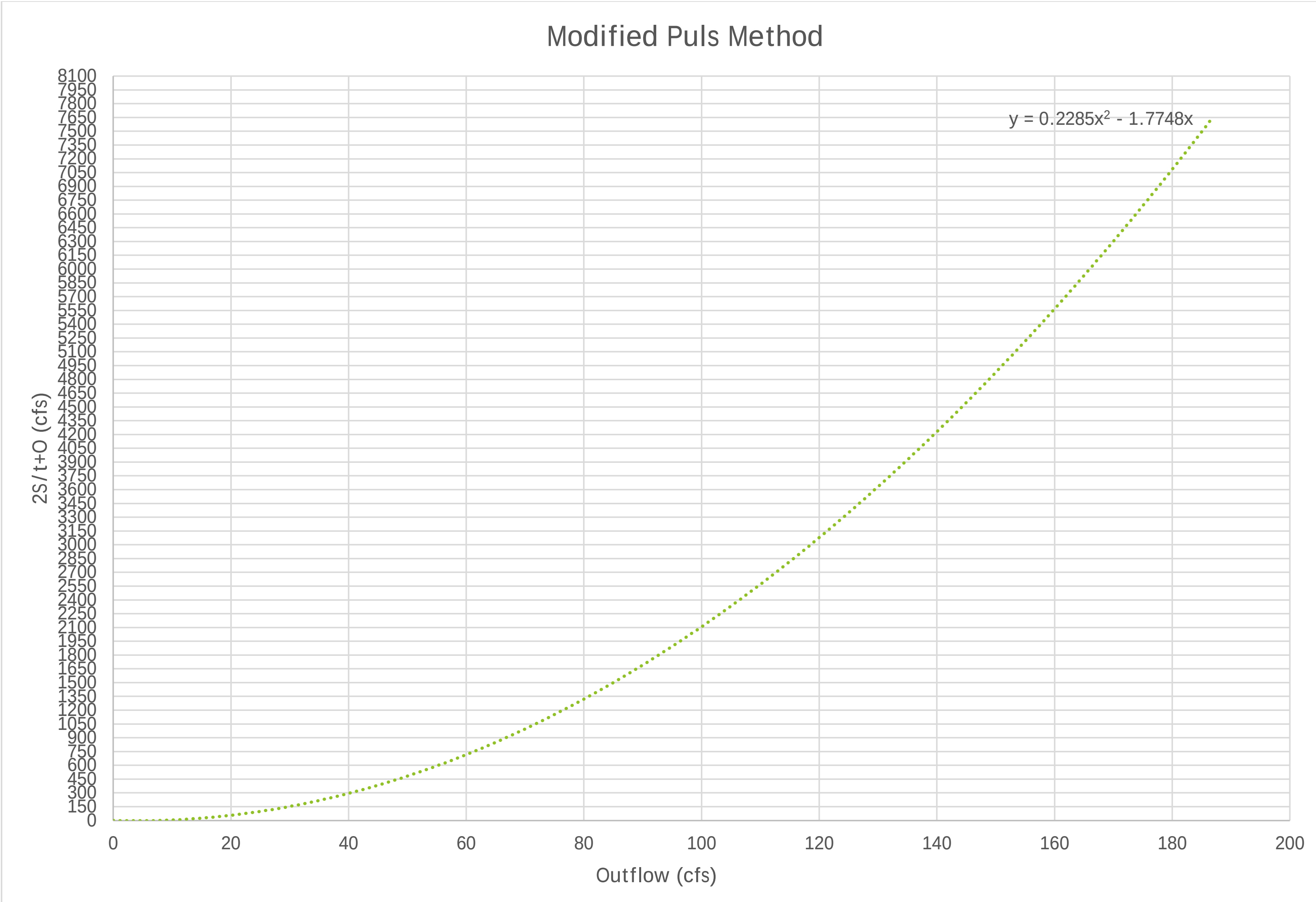


Figure A5.16: Outflow vs. (2S/t +O) Relationship used to estimate O (cfs)

Table A5.20: PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.374** (0.327-0.434)	**0.437** (0.382-0.507)	**0.542** (0.472-0.631)	**0.634** (0.548-0.740)	**0.764** (0.637-0.925)	**0.869** (0.705-1.06)	**0.977** (0.761-1.23)	**1.09** (0.809-1.40)	**1.25** (0.884-1.64)	**1.37** (0.941-1.83)
10-min	**0.548** (0.479-0.635)	**0.639** (0.559-0.742)	**0.794** (0.691-0.924)	**0.928** (0.802-1.08)	**1.12** (0.933-1.35)	**1.27** (1.03-1.56)	**1.43** (1.11-1.79)	**1.6** (1.19-2.05)	**1.82** (1.29-2.41)	**2** (1.38-2.68)
15-min	**0.668** (0.585-0.775)	**0.78** (0.681-0.905)	**0.968** (0.843-1.13)	**1.13** (0.978-1.32)	**1.36** (1.14-1.65)	**1.55** (1.26-1.90)	**1.75** (1.36-2.19)	**1.95** (1.45-2.50)	**2.23** (1.58-2.94)	**2.44** (1.68-3.26)
30-min	**0.925** (0.810-1.07)	**1.09** (0.949-1.26)	**1.36** (1.18-1.58)	**1.59** (1.38-1.86)	**1.93** (1.60-2.33)	**2.19** (1.78-2.69)	**2.47** (1.92-3.10)	**2.76** (2.05-3.55)	**3.16** (2.24-4.17)	**3.47** (2.38-4.63)
60-min	**1.2** (1.05-1.39)	**1.41** (1.23-1.64)	**1.77** (1.54-2.06)	**2.08** (1.80-2.43)	**2.54** (2.13-3.09)	**2.92** (2.37-3.59)	**3.31** (2.59-4.17)	**3.73** (2.77-4.81)	**4.32** (3.07-5.71)	**4.78** (3.29-6.39)
2-hr	**1.48** (1.30-1.70)	**1.73** (1.52-2.00)	**2.18** (1.91-2.52)	**2.58** (2.24-2.99)	**3.16** (2.66-3.83)	**3.65** (2.98-4.46)	**4.16** (3.27-5.21)	**4.71** (3.52-6.04)	**5.48** (3.92-7.21)	**6.09** (4.22-8.09)
3-hr	**1.66** (1.46-1.90)	**1.94** (1.71-2.23)	**2.44** (2.14-2.82)	**2.9** (2.53-3.35)	**3.58** (3.02-4.32)	**4.15** (3.40-5.06)	**4.75** (3.75-5.93)	**5.4** (4.06-6.91)	**6.32** (4.54-8.30)	**7.07** (4.91-9.34)
6-hr	**1.97** (1.75-2.25)	**2.31** (2.04-2.64)	**2.92** (2.57-3.34)	**3.47** (3.04-3.98)	**4.3** (3.65-5.17)	**4.99** (4.12-6.06)	**5.74** (4.55-7.12)	**6.54** (4.95-8.32)	**7.69** (5.56-10.0)	**8.61** (6.02-11.3)
12-hr	**2.3** (2.04-2.60)	**2.7** (2.40-3.06)	**3.4** (3.01-3.87)	**4.04** (3.55-4.61)	**4.99** (4.26-5.95)	**5.78** (4.79-6.96)	**6.63** (5.28-8.16)	**7.53** (5.73-9.51)	**8.81** (6.41-11.4)	**9.84** (6.93-12.8)
24-hr	**2.66** (2.38-2.99)	**3.1** (2.77-3.49)	**3.87** (3.45-4.38)	**4.57** (4.04-5.18)	**5.6** (4.80-6.62)	**6.45** (5.38-7.71)	**7.35** (5.89-8.99)	**8.32** (6.37-10.4)	**9.69** (7.09-12.5)	**10.8** (7.64-14.0)
2-day	**3.1** (2.79-3.46)	**3.54** (3.18-3.96)	**4.32** (3.87-4.85)	**5.03** (4.47-5.67)	**6.09** (5.25-7.15)	**6.97** (5.84-8.27)	**7.91** (6.38-9.61)	**8.92** (6.87-11.1)	**10.4** (7.64-13.2)	**11.5** (8.22-14.8)
3-day	**3.42** (3.08-3.81)	**3.85** (3.47-4.29)	**4.62** (4.15-5.17)	**5.33** (4.75-5.98)	**6.38** (5.53-7.47)	**7.27** (6.12-8.59)	**8.22** (6.66-9.94)	**9.24** (7.15-11.5)	**10.7** (7.92-13.6)	**11.9** (8.50-15.2)
4-day	**3.68** (3.32-4.08)	**4.12** (3.71-4.58)	**4.9** (4.40-5.46)	**5.6** (5.00-6.27)	**6.66** (5.78-7.75)	**7.54** (6.36-8.88)	**8.48** (6.88-10.2)	**9.49** (7.36-11.7)	**10.9** (8.11-13.8)	**12.1** (8.68-15.5)
7-day	**4.32** (3.92-4.78)	**4.83** (4.37-5.34)	**5.69** (5.13-6.31)	**6.44** (5.77-7.17)	**7.52** (6.52-8.65)	**8.39** (7.09-9.77)	**9.29** (7.56-11.1)	**10.2** (7.96-12.5)	**11.5** (8.60-14.5)	**12.6** (9.09-16.0)
10-day	**4.9** (4.46-5.40)	**5.48** (4.98-6.04)	**6.44** (5.83-7.12)	**7.26** (6.52-8.05)	**8.39** (7.29-9.59)	**9.29** (7.86-10.8)	**10.2** (8.31-12.1)	**11.1** (8.67-13.5)	**12.4** (9.26-15.5)	**13.4** (9.70-17.0)

20-day	**6.64** (6.07-7.26)	**7.42** (6.77-8.12)	**8.69** (7.90-9.53)	**9.73** (8.79-10.7)	**11.2** (9.71-12.6)	**12.2** (10.4-14.0)	**13.3** (10.9-15.6)	**14.4** (11.3-17.4)	**15.9** (11.9-19.6)	**16.9** (12.4-21.3)
30-day	**8.13** (7.45-8.86)	**9.09** (8.32-9.91)	**10.6** (9.70-11.6)	**11.9** (10.8-13.0)	**13.6** (11.8-15.3)	**14.9** (12.7-16.9)	**16.1** (13.2-18.8)	**17.4** (13.6-20.8)	**18.9** (14.3-23.3)	**20.1** (14.8-25.2)
45-day	**10** (9.23-10.9)	**11.3** (10.3-12.2)	**13.2** (12.1-14.3)	**14.7** (13.4-16.1)	**16.7** (14.6-18.7)	**18.2** (15.6-20.7)	**19.7** (16.2-22.8)	**21** (16.5-25.0)	**22.8** (17.2-27.8)	**24** (17.6-30.0)
60-day	**11.7** (10.8-12.6)	**13.2** (12.1-14.2)	**15.4** (14.1-16.7)	**17.2** (15.7-18.7)	**19.5** (17.0-21.6)	**21.1** (18.0-23.8)	**22.7** (18.7-26.2)	**24.1** (19.0-28.6)	**25.9** (19.5-31.5)	**27** (20.0-33.7)

1 Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Table A5.21: 2-yr Modified Puls Method Calculations

Stage (ft)	Storage (ft3)	O (cfs)	2S/delta_t + O	Time (hr)	Inflow (cfs)	In +In+1	2S/delta_t - O	2S/delta_t + O	O (cfs)	Delta_S = I-O (cfs)
		0	0							
0.5	830	24.06642	24.98864696		DRH					
1	4150	34.03506	38.64617537	0	0	0.00	0.00		0	0
1.5	300830	41.68427	375.939826	0.5	0	0.02	0.00	0.00	0	0
2	400830	48.13285	493.4995161	1	0.023294988	1.50	0.02	0.02	0.0001	0
2.5	500830	53.81416	610.2919395	1.5	1.471764895	9.59	1.42	1.52	0.05	1
3	600830	58.95046	726.5393494	2	8.12205627	29.47	8.81	11.01	1.1	7
3.5	700830	63.67377	842.3737748	2.5	21.35062329	59.53	34.28	38.28	2	19
4	800830	68.07013	957.8812396	3	38.17677547	94.00	65.81	93.81	14	24
4.5	900830	72.19927	1073.121496	3.5	55.82217868	129.29	105.81	159.81	27	29
5	1000830	76.10472	1188.138051	4	73.4675819	164.58	169.10	235.10	33	40
5.5	1100830	79.8193	1302.963745	4.5	91.11298512	199.83	255.68	333.68	39	52
6	1200830	83.36854	1417.624096	5	108.7217459	232.81	365.52	455.52	45	64
6.5	1300830	86.77273	1532.139395	5.5	124.0887386	255.36	498.33	598.33	50	74
				6	**131.2732625**	259.38	639.69	753.69	57	74
total	1300830	ft3		6.5	128.109848	247.40	771.07	899.07	64	64
	29.86294766	acre-ft		7	119.2869258	228.46	878.47	1018.47	70	49
				7.5	109.1750172	208.24	964.93	1106.93	71	38
				8	99.06303488	188.01	1027.17	1173.17	73	26
				8.5	88.95105254	167.79	1061.18	1215.18	77	12
				9	78.8390702	147.57	1072.97	1228.97	78	1
				9.5	68.72708787	127.34	1066.54	1220.54	77	
				10	58.61510553	107.12	1043.88	1193.88	75	
				10.5	48.50312319	86.89	1003.00	1151.00	74	
				11	38.39114086	66.67	947.89	1089.89	71	
				11.5	28.27969047	46.49	874.56	1014.56	70	
				12	18.21360195	27.26	787.06	921.06	67	
				12.5	9.050473584	11.95	690.32	814.32	62	576 cfs
				13	2.899764441	3.35	592.27	702.27	55	1036630.541 ft23
				13.5	0.450777181	0.45	491.62	595.62	52	**23.79776265** ac-ft
				14	0	0.00	396.07	492.07	48	

Table A5.22: 5-yr Modified Puls Method Calculations

Stage (ft)	Storage (ft3)	O (cfs)	2S/delta_t + O	Time (hr)	Inflow (cfs)	In +In+1	2S/delta_t - O	2S/delta_t + O	O (cfs)	Delta_S = I-O (cfs)
		0	0							
0.5	830	24.066425	24.98864696		DRH					
1	4150	34.035064	38.64617537	0	0.00	0.00	0.00		0	0
1.5	300830	41.68427	375.939826	0.5	0.00	0.10	0.00	0.00	0	0
2	400830	48.132849	493.4995161	1	0.10	3.98	0.06	0.10	0.02	0
2.5	500830	53.814162	610.2919395	1.5	3.88	24.47	2.04	4.04	1	3
3	600830	58.950461	726.5393494	2	20.58	73.41	22.51	26.51	2	19
3.5	700830	63.673775	842.3737748	2.5	52.82	145.61	47.92	95.92	24	29
4	800830	68.070129	957.8812396	3	92.79	227.04	127.53	193.53	33	60
4.5	900830	72.199274	1073.121496	3.5	134.26	309.99	270.57	354.57	42	92
5	1000830	76.104717	1188.138051	4	175.73	392.94	480.56	580.56	50	126
5.5	1100830	79.819301	1302.963745	4.5	217.20	475.72	745.50	873.50	64	153
6	1200830	83.368541	1417.624096	5	258.52	552.56	1069.22	1221.22	76	183
6.5	1300830	86.772728	1532.139395	5.5	294.04	603.29	1451.78	1621.78	85	209
				6	**309.25**	609.25	1869.07	2055.07	93	216
total storage	1300830	ft3		6.5	300.00	578.62	2268.32	2478.32	105	195
	29.86294766	acre-ft		7	278.62	533.46	2612.94	2846.94	117	162
				7.5	254.85	485.93	2904.40	3146.40	121	134
				8	231.08	438.40	3146.33	3390.33	122	109
				8.5	207.32	390.87	3324.73	3584.73	130	77
				9	183.55	343.33	3451.60	3715.60	132	52
				9.5	159.78	295.80	3526.93	3794.93	134	26
				10	136.02	248.27	3550.74	3822.74	136	0
				10.5	112.25	200.74	3529.01	3799.01	135	
				11	88.49	153.21	3461.74	3729.74	134	
				11.5	64.72	105.82	3356.95	3614.95	129	1843 cfs
				12	41.10	60.89	3216.77	3462.77	123	3318078.721 ft23
				12.5	19.79	25.74	3035.65	3277.65	121	**76.17260608** acre-ft
				13	5.95	6.78	2825.39	3061.39	118	
				13.5	0.83	0.83	2600.17	2832.17	116	
				14	0.00	0.00	2379.00	2601.00	111	

Table A5.23: Synthetic Unit Hydrograph time lags

t (hr)	Q (cfs)	Lag 1	Lag 2	Lag 3	Lag 4	Lag 5	Lag 6	Lag 7	Lag 8	Lag 9	Lag 10	Lag 11	Lag 12	Lag 13	Lag 14	Lag 15	Lag 16	Lag 17	Lag 18	Lag 19	Lag 20	Lag 21	Lag 22	Lag 23	Lag 24	Lag 25	Lag 26	Lag 27	Lag 28	Lag 29	Lag 30	Lag 31	Lag 32
0	0.0																																
0.5	22.2	0.0																															
1	44.4	22.2	0.0																														
1.5	66.6	44.4	22.2	0.0																													
2	88.8	66.6	44.4	22.2	0.0																												
2.5	111.0	88.8	66.6	44.4	22.2	0.0																											
3	133.2	111.0	88.8	66.6	44.4	22.2	0.0																										
3.5	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0																									
4	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0																								
4.5	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0																							
5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0																						
5.5	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0																					
6	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0																				
6.5	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0																			
7	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0																		
7.5	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0																	
8	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0																
8.5	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0															
9	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0														
9.5	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0													
10	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0												
10.5	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0											
10.89	0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0										
		0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0									
			0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0								
				0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0							
					0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0						
						0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0					
							0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0				
								0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0			
									0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0		
										0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0	
											0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2	0.0
												0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4	22.2
													0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6	44.4
														0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8	66.6
															0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0	88.8
																0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2	111.0
																	0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4	133.2
																		0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6	155.4
																			0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9	177.6
																				0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2	164.9
																					0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5	152.2
																						0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7	139.5
																							0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0	126.7
																								0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3	114.0
																									0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6	101.3
																										0.0	12.2	24.9	37.7	50.4	63.1	75.8	88.6
																											0.0	12.2	24.9	37.7	50.4	63.1	75.8
																												0.0	12.2	24.9	37.7	50.4	63.1
																													0.0	12.2	24.9	37.7	50.4
																														0.0	12.2	24.9	37.7
																															0.0	12.2	24.9
																																0.0	12.2
																																	0.0

Table A5.24: Direct Runoff Calculation from S-Hydrograph (2-yr Return Interval)

t (hr)	S-hyd	Lag by 1-hr	S-Hyd	divide by 2 (1 hr UH)	UH x 0	0.002	0.126	0.342	0.250	0.074	DRH	
0	0.0		0.0	0.0	0.0						0.0	
0.5	22.2		22.2	11.1	0.0	0.0					0.0	
1	66.6	0.0	66.6	33.3	0.0	0.0	0.0				0.0	
1.5	133.2	22.2	111.0	55.5	0.0	0.1	1.4	0.0			1.5	
2	222.0	66.6	155.4	77.7	0.0	0.1	4.2	3.8	0.0		8.1	
2.5	333.0	133.2	199.8	99.9	0.0	0.2	7.0	11.4	2.8	0.0	21.4	
3	466.3	222.0	244.2	122.1	0.0	0.2	9.8	19.0	8.3	0.8	38.2	
3.5	621.7	333.0	288.6	144.3	0.0	0.3	12.6	26.6	13.9	2.5	55.8	
4	799.3	466.3	333.0	166.5	0.0	0.3	15.4	34.2	19.4	4.1	73.5	
4.5	964.2	621.7	342.5	171.3	0.0	0.3	18.2	41.8	25.0	5.7	91.1	
5	1116.4	799.3	317.1	158.5	0.0	0.4	21.0	49.4	30.6	7.4	108.7	
5.5	1255.8	964.2	291.6	145.8	0.0	0.3	21.6	57.0	36.1	9.0	124.1	
6	1382.5	1116.4	266.2	133.1	0.0	0.3	20.0	58.6	41.7	10.7	131.3	
6.5	1496.5	1255.8	240.7	120.4	0.0	0.3	18.4	54.3	42.9	12.3	128.1	
7	1597.8	1382.5	215.3	107.6	0.0	0.3	16.8	49.9	39.7	12.6	119.3	
7.5	1686.4	1496.5	189.8	94.9	0.0	0.2	15.2	45.6	36.5	11.7	109.2	
8	1762.2	1597.8	164.4	82.2	0.0	0.2	13.6	41.2	33.3	10.8	99.1	
8.5	1825.3	1686.4	138.9	69.5	0.0	0.2	12.0	36.8	30.1	9.8	89.0	
9	1875.7	1762.2	113.5	56.7	0.0	0.1	10.4	32.5	26.9	8.9	78.8	
9.5	1913.4	1825.3	88.1	44.0	0.0	0.1	8.8	28.1	23.8	7.9	68.7	
10	1938.3	1875.7	62.6	31.3	0.0	0.1	7.2	23.8	20.6	7.0	58.6	
10.5	1950.5	1913.4	37.2	18.6	0.0	0.1	5.6	19.4	17.4	6.1	48.5	
11	1950.5	1938.3	12.2	6.1	0.0	0.0	4.0	15.1	14.2	5.1	38.4	
11.5	1950.5	1950.5	0.0	0.0	0.0	0.0	2.3	10.7	11.0	4.2	28.3	
12						0.0	0.8	6.4	7.8	3.2	18.2	
12.5							0.0	2.1	4.6	2.3	9.1	
13								0.0	1.5	1.4	2.9	
13.5									0.0	0.5	0.5	
14										0.0	0.0	
											1550.2	
										DR Volume	2790336.1	ft3
											64.1	acre-ft

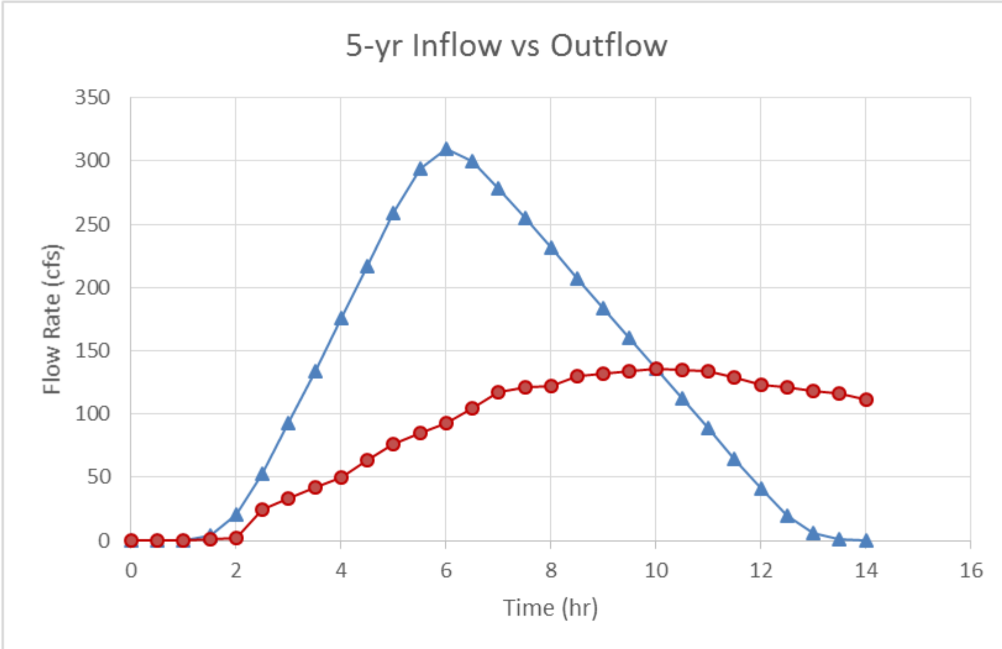


Figure A5.18: 5-yr Inflow vs. Outflow Relationship

The Rational Method was also used to double-check the NRCS Method, and was found to be fairly consistent for the 25-yr storm. The calculations and results for the Rational Method can be seen in Table A5.25.

Table A5.25: Rational Method Results

Land Use	**% of total area**	**Runoff Coefficient**	**Weighted Runoff Coefficient**
Neighborhood	20	0.66	0.132
Undeveloped (ex-farmland)	75	0.12485	0.0936375
Park	1	0.33	0.0033
Roads	4	0.935	0.0374
	100		0.2663375
Q (100 year)=CiA			
	(1-hr Tc) 25 yr		
C	**I (in/hr)**	**A (acre)**	**Q (cfs)**
0.2663375	2.66	689.24	**488.29741961**

Appendix 6: HEC-HMS and HEC-RAS Modeling

In order to make sure the runoff calculations we performed were correct, we used the Hydrologic Modeling System (HEC-HMS) software to support our analysis. From the results of HEC-HMS, the peak discharge for the 25-year storm was 943.5 cfs, and time to peak that the software provided was 5 hours. Comparing these two values to our “hand-calculated” values is critical in the analysis. The NRCS Method calculations and the HEC-HMS calculations are very close to each other and have a less than 1% variance of the peak discharge. The time to peak was only 0.5 hours off from the NRCS Method. These differences can be caused by the different methods and the use of varying parameters.

We also used the River Analysis System (HEC-RAS) to simulate our water surface profiles. However, there is a difference in the results from the software and the hand calculations. This is because we made an assumption that the outlet of the river is sufficient enough that it allows the water to continue to go through the channel. However, in reality, the size of the culvert at the outlet of the basin is likely undersized, and so since it was not modeled in the software, the simulation does not show the flooding that results from the culvert. In order to add the existing culvert in the HEC-RAS, we would need to have more information such as elevations and length of culvert and the size of the pipe it connects to, but the City of Washington was not able to provide these specifications. Therefore, certain assumptions were made and the outlet was just considered a circular orifice. Also, we were not able to do survey the site by ourselves because we do not have the equipment to do so, and would require consultants to do so. In final design, it would be necessary to survey the area to determine the culvert specifications. However, it is still beneficial to see the results from the HEC-RAS because if the City of Washington wants to upgrade the culvert that is exists to allow water to pass through the channel more freely, then the HEC-RAS will be the simulation of river and its water profiles after a culvert upgrade. However, a culvert upgrade is not ideal, since the main goal of the project is to retain and handle the water on-site rather than move it downstream quickly.

For the Hydrologic Modeling System (HEC-HMS) software simulation, certain parameters were used and can be seen in Table A6.1. The results can be seen in Figures A6.1 and A6.1.

Table A6.1: HEC-HMS parameters

Parameters for the HEC-HMS	
Area of Drainage Basin (mi^2)	1.077
Design Storm	6 Hour storm
F index (in/hr)	0.14
Initial loss (in)	0
Lag time (min)	1 hour
SCS Curve number method	

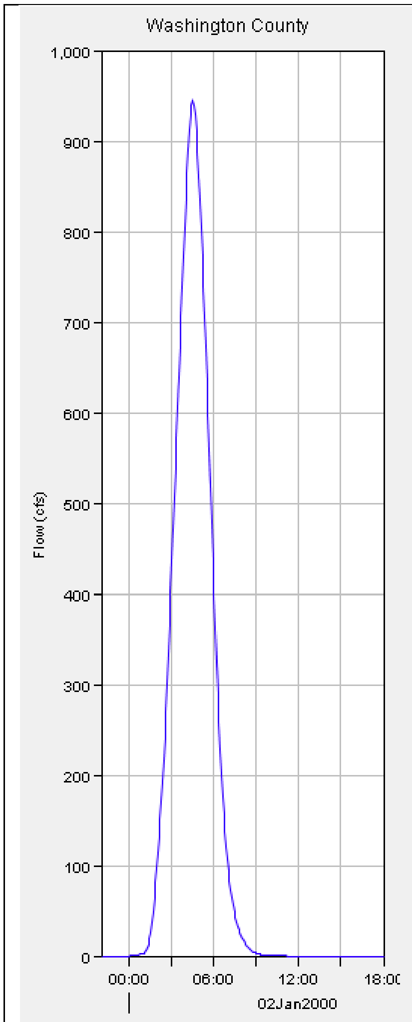


Figure 1 Direct Runoff

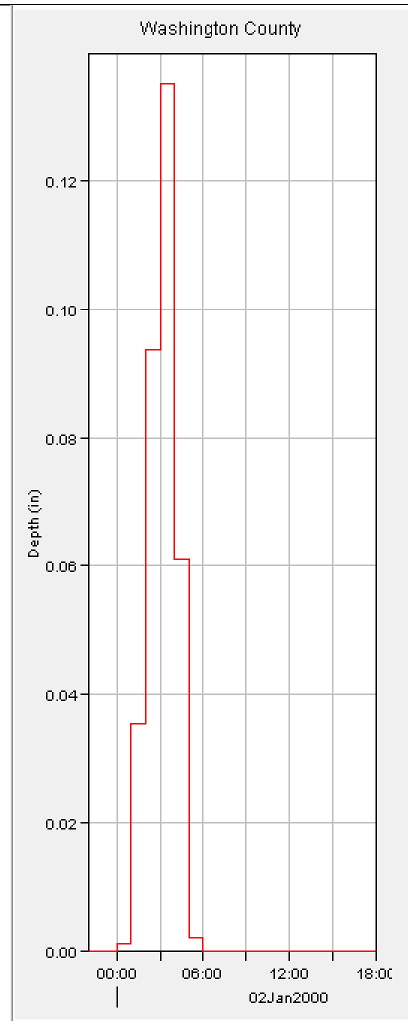


Figure 2 Excess Precipitation

Figure A6.1 and Figure A6.2: The DRH and Excess Precipitation Hydrograph from the HEC-HMS simulation of the 25-yr storm

For the River Analysis System (HEC-RAS) software simulation, the parameters used can be seen in Table A6.2.

Table A6.2: HEC-RAS parameters

Manning's n for main channel	0.040
Manning's n for overbank area	0.030
Peak flow rate (cfs)	942.4878

The reason for choosing the Manning's n for the main channel as 0.040 is because from the observations made during site investigations, we saw an irregular natural channel without debris sediments. Also we assume that the debris will be removed regularly through maintenance. For the overbank area, we use a Manning's n of 0.030 because the floodplains are essentially pasture areas with short grass.

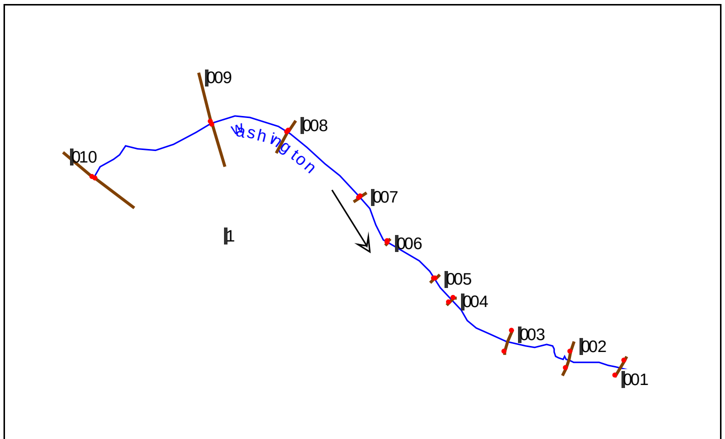


Figure 3 Cross sections along the river

Figure A6.3: HEC-RAS cross sections

Figure A6.3 shows the cross section we have by using ArcMap to gain the elevations. The reason for choosing these specific cross sections is because by using multiple cross sections at even intervals throughout the channel, we can represent the channel best in terms of average channel shape, slope, and elevation.

FiguresA6.4 and A6.7 show the cross sections located at the in-line detention basin area. This show the difference between pre- and post- in-line detention basin development and installation.

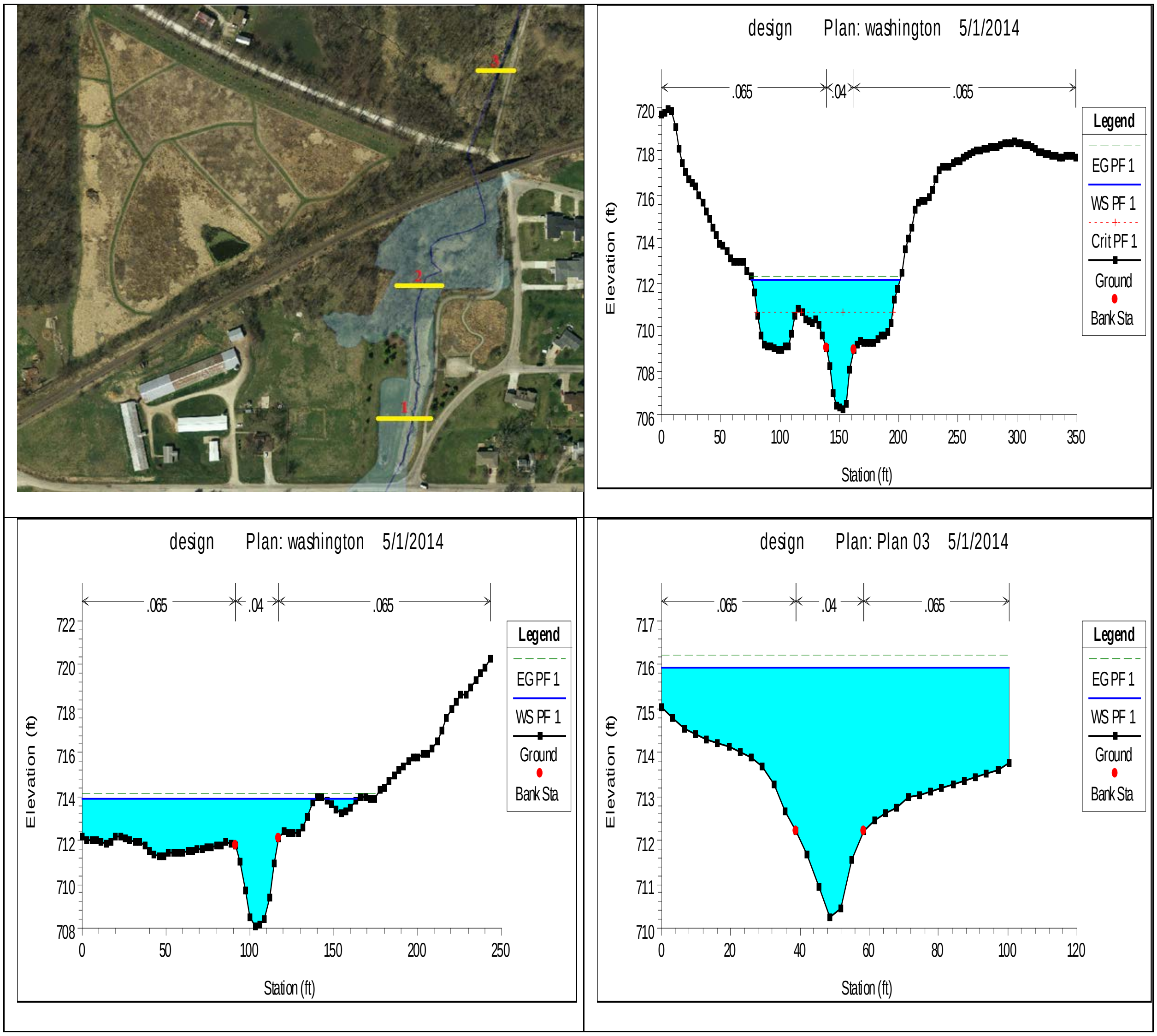


Figure A6.4: Pre-development (before in-line detention basin design)

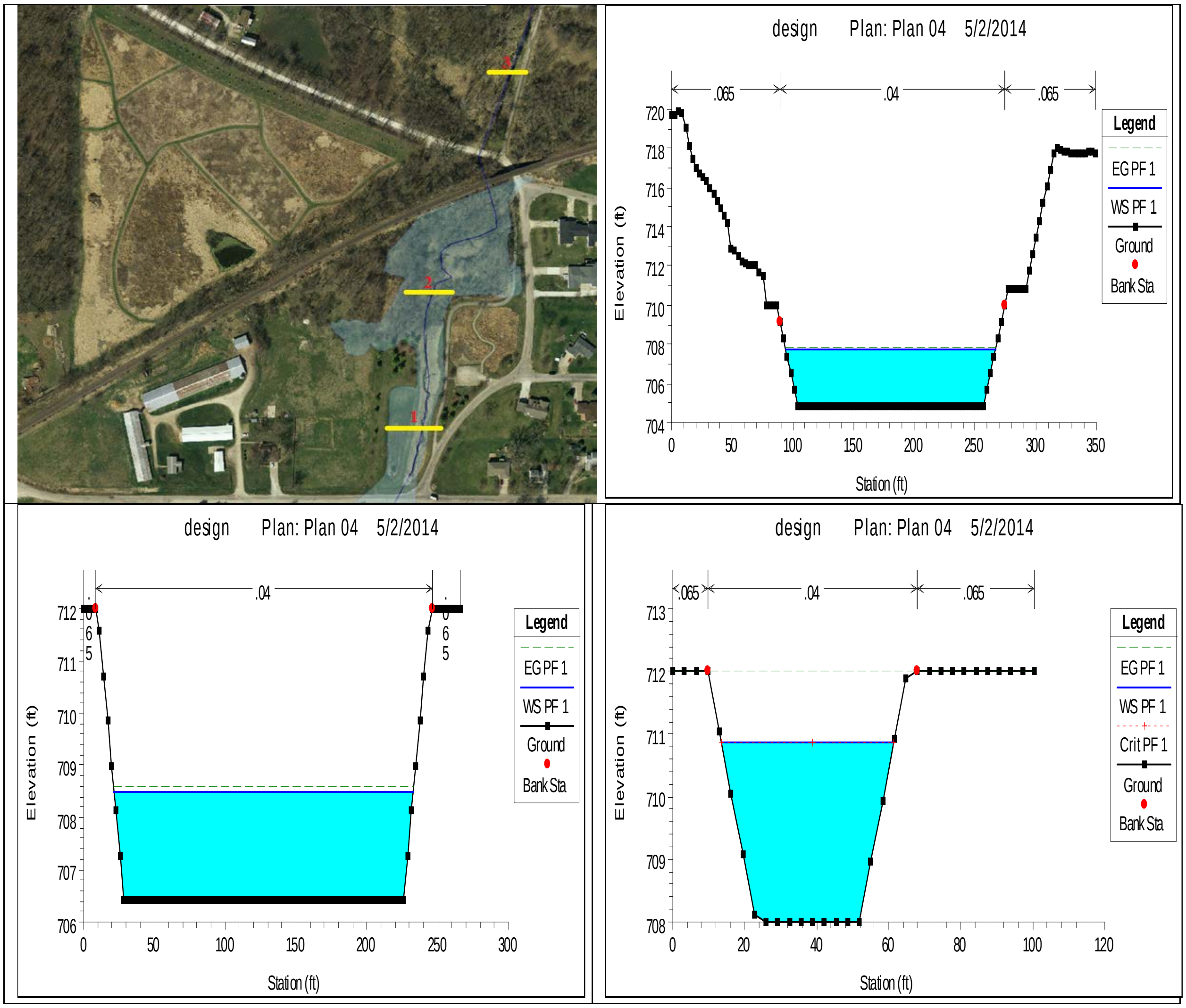


Figure A6.5: Post-development (after the in-line detention basins are installed)

Appendix 7: Site description and AutoCAD rendering

The site was analyzed using a variety of methods, but the Web Soil Survey (USDA) was useful in finding the soil information needed in design. Figure A7.1 shows the results of the online survey of the Washington, IA site.

Tables — AASHTO Group Classification (Surface) — Summary By Map Unit

Summary by Map Unit — Washington County, Iowa (IA183)

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
76C2	Ladoga silt loam, 5 to 9 percent slopes, moderately eroded	A-4	18.9	2.9%
76D2	Ladoga silt loam, 9 to 14 percent slopes, moderately eroded	A-4	10.3	1.6%
87B	Colo-Zook silty clay loams, 0 to 3 percent slopes	A-7	40.7	6.3%
122	Sperry silt loam, 0 to 2 percent slopes	A-6	4.8	0.7%
179D2	Gara loam, 9 to 14 percent slopes, moderately eroded	A-6	1.9	0.3%
222D2	Clarinda silty clay loam, 9 to 14 percent slopes, moderately eroded	A-7	10.6	1.6%
223D2	Rinda silty clay loam, 9 to 14 percent slopes, moderately eroded	A-7	1.8	0.3%
279	Taintor silty clay loam, 0 to 2 percent slopes	A-7	219.4	34.0%
280	Mahaska silty clay loam, 0 to 2 percent slopes	A-7-6	112.4	17.4%
281B	Otley silty clay loam, 2 to 5 percent slopes	A-7	19.8	3.1%
281C	Otley silty clay loam, 5 to 9 percent slopes	A-7	13.3	2.1%
281C2	Otley silty clay loam, 5 to 9 percent slopes, moderately eroded	A-7	5.7	0.9%
281D2	Otley silty clay loam, 9 to 14 percent slopes, moderately eroded	A-7	3.3	0.5%
570B	Nira silty clay loam, 2 to 5 percent slopes	A-7	115.2	17.8%
570C2	Nira silty clay loam, 5 to 9 percent slopes, moderately eroded	A-7	16.7	2.6%
571B	Hedrick silt loam, 2 to 5 percent slopes	A-4	17.5	2.7%
571C2	Hedrick silt loam, 5 to 9 percent slopes, moderately eroded	A-4	6.9	1.1%
779	Kalona silty clay loam, 0 to 2 percent slopes	A-7	26.2	4.1%
Totals for Area of Interest			**645.5**	**100.0%**

Figure A7.1: Web Soil Survey Results

The Wellness Park engineers provided our team with a rendering of their proposed park design, and we synthesized it into our site to show its location in relations to the stormwater management site. This drawing can be seen in Figure A7.2.

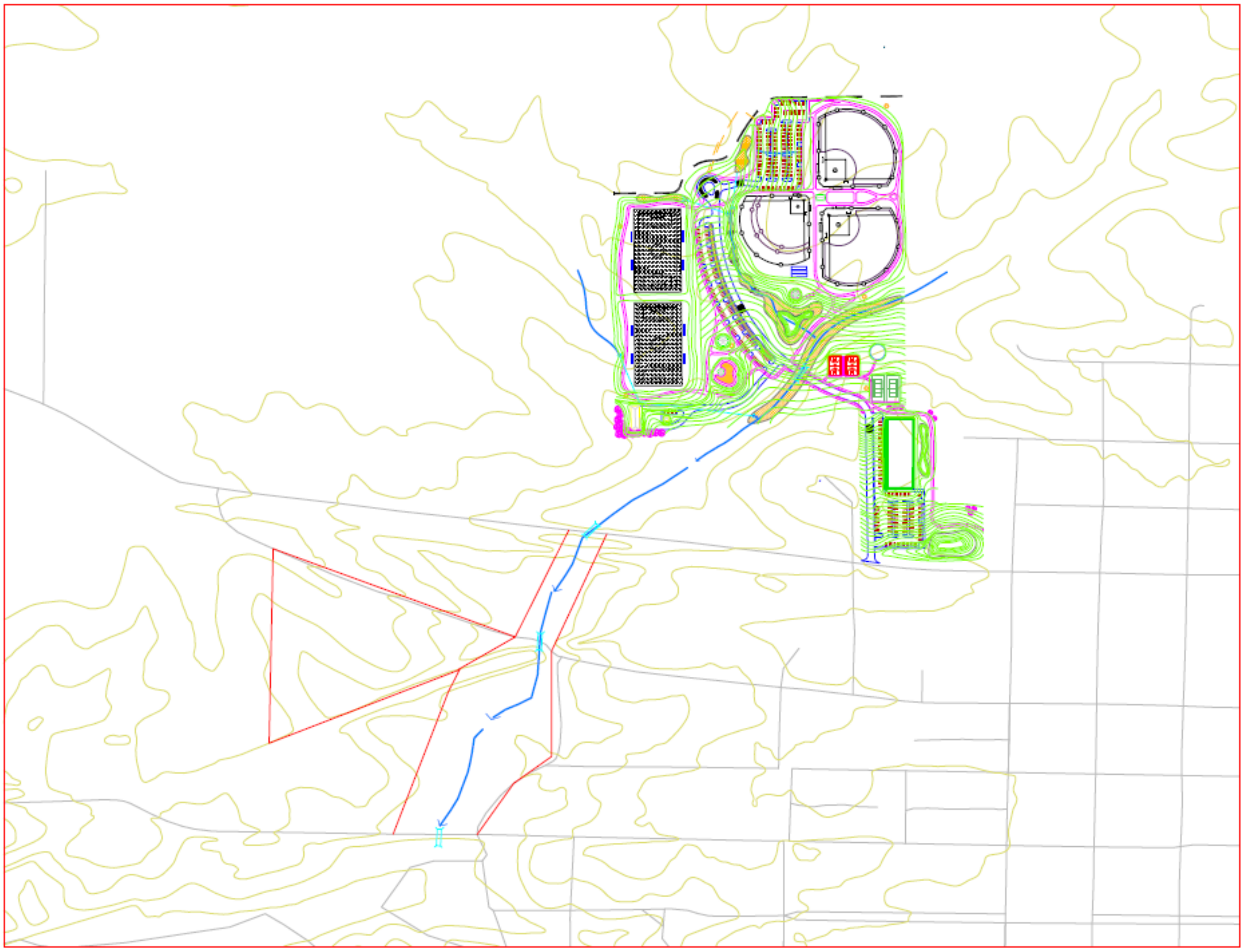


Figure A7.2: Wellness Park and Sesqui Park

Appendix 8: Grantt chart

Grantt Chart is one of the easiest way to show the schedule of the project. For this project, we have provided an estimate Grantt chart for the in-line detention basins to be predict the overall project duration. The Grantt chart is shown in Figure A8.1.

ACTIVITY	Start	DURATION	Finish
Legalities	M 5/5/14	21 days	M 5/26/14
Permitting	M 5/5/15		M 5/26/14
SWPPP	M 5/5/16		M 5/26/14
Preparation	M 5/26/17		M 6/10/14
Construction Fence	M 5/26/14		M 5/26/14
Silt Fence	M 5/26/14		M 5/26/14
Construction Signs	M 5/26/14		M 5/26/14
Tree Clearing	T 5/27/14		T 6/10/14
Path Removal	T 5/27/14		T 6/10/14
Heavy Construction	W 6/11/14		TH 5/24/14
Excavation	W 6/11/14		W 6/26/14
Culvert Extension	TH 6/27/14		TH 7/3/14
Construct Embankment	TH 7/3/14		TH 7/17/14
Site Grading	TH 7/17/14		TH 7/24/14
Light construction	F 7/25/14		TH 7/31/14
Lay Asphalt Path	F 7/25/14		TH 7/31/14
Sign and Fence Removal	TH 7/31/14		TH 7/31/14
Landscaping	F 8/1/14		M 8/11/14
Laying Sod	F 8/1/14		W 8/6/14
Planting Trees/Shrubs	W 8/6/14		M 8/11/14
As-Built Surveys	T 8/12/14		F 8/22/14

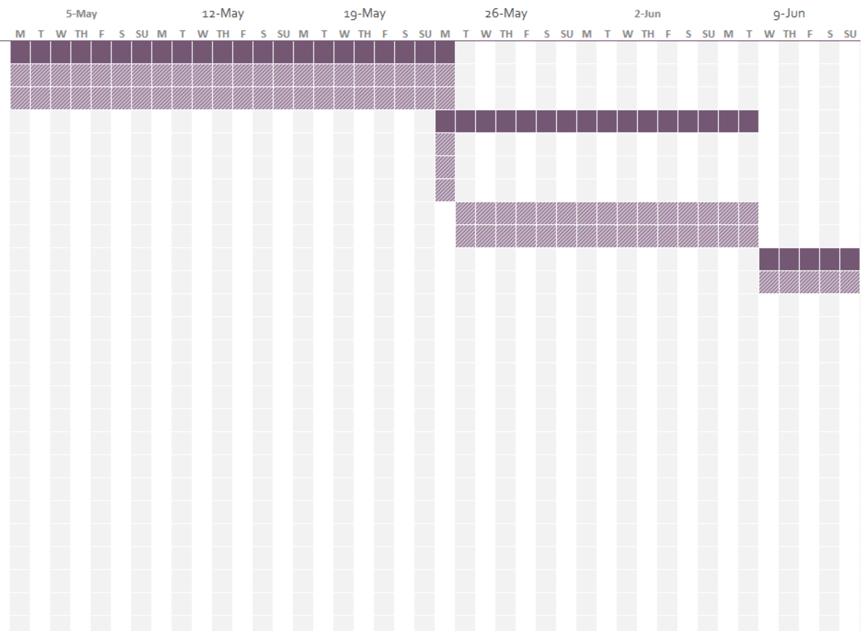


Figure A8.1: Grantt chart for the in-line detention basin

Appendix 9: General Permit No. 2

According to the Federal Clean Water Act regulations that our project needs to have a National Pollutant Discharge Elimination System (NPDES) permit. The Figure A9.1 to A 9.4 shows the part of the forms that is need to be fill to apply the General Permit No. 2.

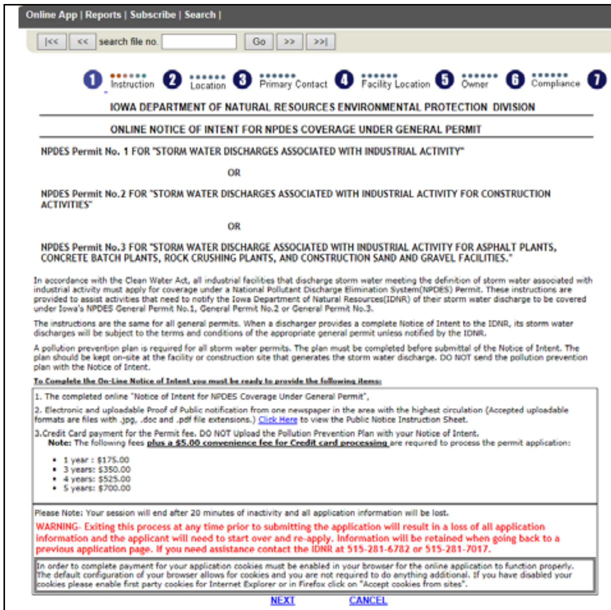


Figure A9.1

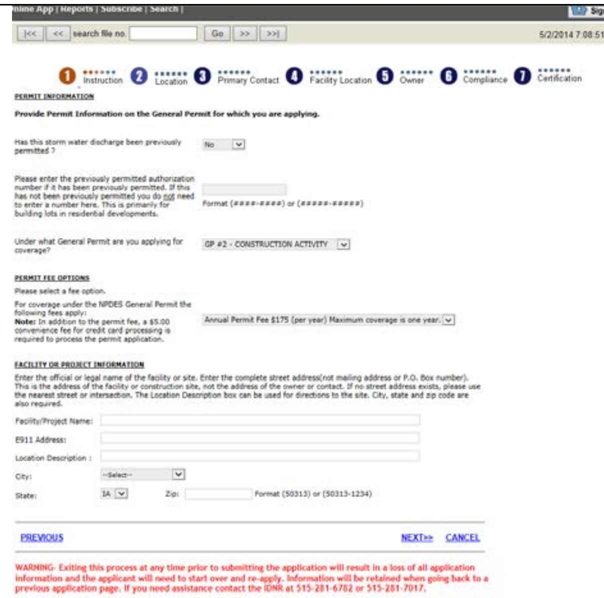


Figure A9.2

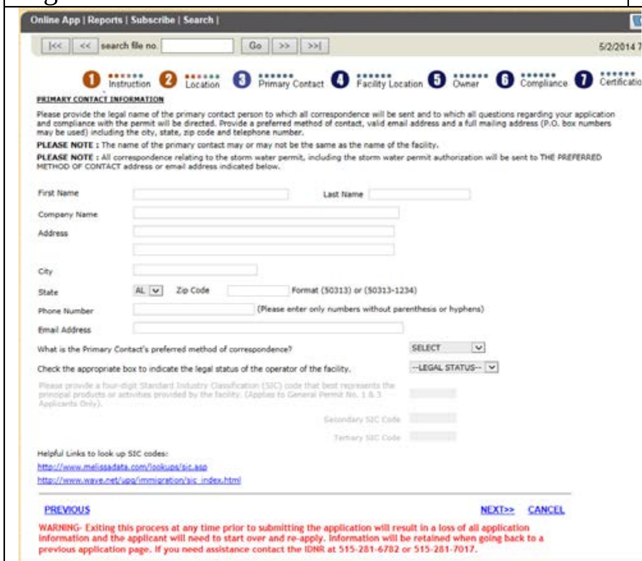


Figure A9.3

Figure A9.4

Appendix 10: Cost estimation calculation

Rec: In-line Detention Basin					
Item No. [1]	Description [2]	Unit [3]	Estimated Quantity [4]	Unit Price [5]	Estimated Amount [6]
1	Excavation, soil	C.Y.	1474871	0.65	958667
2	Asphalt	S.Y.	886	11.1	9835
3	Gravel	C.Y.	52	8.3	432

Table A10.1: Cost estimation for in-line detention basin

In-line Channel Widening					
Item No. [1]	Description [2]	Unit [3]	Estimated Quantity [4]	Unit Price [5]	Estimated Amount [6]
1	Excavation, soil	C.Y.	12964	0.65	8427
2	Asphalt	S.Y.	886	11.1	9835
3	Gravel	C.Y.	52	8.3	432

Table A10.2: Cost estimation for channel modification

Sesqui Detention Basin					
Item No. [1]	Description [2]	Unit [3]	Estimated Quantity [4]	Unit Price [5]	Estimated Amount [6]
1	Excavation, soil	C.Y.	105000	0.65	68250

❖ **The pump cost is not included**

Table A10.3: Cost estimation for detention basin