Final Report: Wiehle South Stream Restoration

Smart Stream Solutions Inc.

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Draft: May 4, 2018

Revised: May 7, 2018

Final Submitted: May 9, 2018

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Executive Summary

This report represents a design project undertaken for CEE 5022 during the Spring 2018 semester at Cornell University by a project team known as Smart Streams Solutions (S^3)[1] The assignment was to redesign a proposed stream restoration project in Reston, VA known as "Wiehle South" with the goals of developing a design that is:

1) Stable under expected flow rates - thus reducing erosion and related non-point source pollutant loadings (TSS and TP primarily) in a tributary of Colvin Run, the Potomac River, and the Chesapeake Bay.
2) Representative of "Natural Stream Design" practices in an urbanized watershed, and
3) Economically viable – which the project sponsor determined the "cost threshold" to be $140/SCU

S^3 was able to achieve these goals as described below and in the subsequent chapters of this report.

Introduction

Reston began development in the 1960s as a planned community in the suburbs of D.C. As the community developed, the impervious cover of the area increased without significant or effective stormwater management to compensate. The resultant increased stormwater runoff (rate and volume) has degraded all stream reaches in this community. Wiehle South is one such reach of a stream known as Colvin Run. The Wiehle South watershed is located south of Lake Anne in Reston, Virginia, with the outlet located at latitude 38.96, longitude -77.3. The watershed spans 282 acres, or 0.44 square miles (Figure 1). Due to changes in land use from urbanization, the impervious area has increased since the development of the community and is currently at 26.6% of the total watershed (Figure 2). These land use changes have contributed to increased runoff and thereby increased stream flow rates. This has in turn led to rapidly degrading and entrenched streams, resulting in compromised channel stability, water quality, and stream biodiversity and stream-channel aesthetics. Local and state agencies have a stake in restoring such streams to reduce suspended sediments and nutrient loads entering Chesapeake Bay. It has thus become necessary to restore stream reaches in the area to contain higher flow rates without significant erosion.

A previous restoration plan for the Wiehle South stream reach was developed in 2012; however,

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it was not implemented due to high cost. It was believed that the cost of the previous design could be reduced through re-evaluation of certain conservative assumptions and practices in the design and contractor bids

Smart Stream Solutions was assigned with the task of developing a new restoration plan for the Wiehle South stream reach. This project was carried out in a series of steps corresponding to different chapters in this report. The first step of this process was to determine the return period for which the channel should be designed (Chapter 1). Once the design storm return period was selected, the associated peak discharge was estimated using the result from a range of methods (Chapter 2). This calculated discharge (80 cfs) was significantly lower than the original design storm discharge (113 cfs). Having determined the peak discharge, the team designed a new riffle cross section representative of the channel form (Chapter 3). The required substrate size for the armor layer in the channel was then calculated based on the new riffle cross section, to ensure streambed stability (Chapter 3). The team also conducted an analysis for the 100-yr and 500-yr flood events, to ensure that no significant erosion would take place in the floodplain during high intensity storms (Chapter 3). The effect of increased frequency of severe storms due to climate change and the effect of a proposed redevelopment of an existing golf course that is a large portion of the subject stream's watershed was also considered (Chapter 4). Once an appropriate downscaled riffle design was selected, the team explored modifications to designs for other structures in the channel using the same reduction factor (Chapter 5). Finally, a cost analysis was conducted for the whole project to determine whether implementation of the updated design would be economically feasible (Chapter 6).

When determining the return period for the design storm, the team consulted a range of academic and regulatory publications, covering topics ranging from ecological phenomena to structural stability. It was determined that bankfull discharges dominate the formation and transformation of channels, as most sediment transport occurs during these events. The bankfull discharge return period must thus be within natural bounds for a stable channel. Natural stable streams in the region could thus be used as a reference in determining the design storm for the stream. The team decided to design a stream to utilize the peak discharge from a 1-year storm, 24-hr storm at bankfull capacity. Please refer to <u>Chapter 1: Wiehle South Design Storm Report</u> for more details on this decision.

To estimate the peak discharge for a 1-yr, 24-hr storm, the team used information from 10 different calculation methods. The Loudoun County method, NRCS TR-55 method, the Rational Method, the Anderson Method, the Snyder Method, USGS method, ArcGIS StreamStats, the

WSSI 1-yr estimate, and regional curves for rural Maryland with and without enlargement were used to obtain peak discharge estimates. The methodology and results of each method are described in detail in Chapter 2: Wiehle South Discharge Calculation. While many of these methods yielded significantly different results, some had to be rejected for their poor applicability in Wiehle South watershed and for high-frequency storms. Once calculations had been done using all the selected methods, the most appropriate were averaged to produce the final design storm flow rate of 80 cfs. This exercise illustrate why many government agencies specify which calculation method to use (to ensure consistency), and demonstrates the uncertainty of this analysis.

Chapter 3: Wiehle South Riffle Design Analysis describes the process of designing a new riffle cross section representative of the channel form to contain the design storm, as well as calculations for armor layer substrate size. The new riffle was designed using the NRCS Cross Section Hydraulic Analyzer, to contain a flow rate of 80 cfs at bankfull and maintain stable side slopes. The new design maintains a similar structure to the original riffle from the 2012 design, but was scaled down 20% in the horizontal and 10% in the vertical dimensions to accommodate the lower design flow rates.

Reinforced armor layer substrate mix including A1 substrate with a D_{50} of 6.7 inches (this is the size from the expected supplier – which is smaller than VDOT specifications) is proposed for the streambed to prevent in-channel erosion. The flow velocity in the floodplain during the 100-yr and 500-yr storms were also analyzed in Chapter 3 to determine whether the floodplain would be subject to significant erosion during severe storms. This was done using both the NRCS cross-sectional analyzer and the 'flow slices' function in HEC-RAS. The modelled 100-yr and 500-yr storm maximum velocity in the floodplains did not exceed the maximum permissible velocity for vegetated floodplains. However, when the soil is un-vegetated (bare) during and immediately after construction permissible velocities are exceeded in the 500-yr flood event. Because this situation will only occur during the low-probability event of a 500-yr storm occurring during or immediately after construction, floodplain erosion was not deemed a cause for concern with the selected design.

Chapter 4: Wiehle South Climate and Land Use Changes deals with the effects of climate change on the storm intensity-frequency relationship in this area, and the proposed redevelopment of a golf course into housing within this project's watershed and analyses how this might affect future stability of the channel design. Climate model projections and anticipated changes in land use in the watershed were used to recalculate the near future and year 2080 discharge for the stream. Several scenarios were compared to determine which design would be appropriate to ensure channel stability. Although the frequency of bankfull events will increase to as many as 2 times per year, the floodplain is forested and preserved and erosion problems are not predicted. Therefore, we believe that the S^3 design cross section is sufficiently resilient to withstand future hydrological changes, and is thus the recommended design.

In Chapter 5: Wiehle South Concept Plan, the team used the new riffle design to determine reduction factors for other relevant structures in the original restoration plan; i.e. step pools and imbricated rock walls. Step pool widths were reduced by 20% and depths were reduced by 10% to correspond with the revised riffle sizing. The imbricated rock wall designs could not be altered in any significant way due to constraints on individual rock sizes and wall height. The drawings for final designs for riffles, step pools and imbricated rock walls, along with their dimensions, are displayed in Chapter 5.

Chapter 6: Wiehle South Cost Estimation compares the initial Wiehle South concept plan bids to the cost estimated for the revised concept plan created by S^3. Significant cost reductions include the double counting of tree removal in both the site work contractor's and tree contractor's bids (a $90,000 savings); $45,000 saved because Reston Association (the owner) agreed to retain an existing bridge instead of requiring replacement; $162,000 saved in reduction of rock costs due to the smaller cross section and elimination of a new bridge which would have been armored with Class II rip-rap; and $73,000 saved on labor and equipment from having to place less material in the smaller cross section. The only increases in cost come from the re-design process and re-permitting process with various government agencies, estimated at around $81,000. Overall, the estimated price of the Wiehle South project was reduced from $1,753,102 to $1,449,138 (a savings of approximately $370,000). This represents a total Stream Condition Unit (SCU) cost of $139/SCU (Stream Condition Unit), which is below the $140/SCU economic viability hurdle rate determined by WSSI in the current market.

Assignment #1: Wiehle South

Design Storm Report

Smart Stream Solutions, Inc.

We have each personally reviewed and approve the following report:

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Draft Submitted : February 10, 2018
Revised : February 21, 2018
Final Submitted : April 16, 2018

I. Executive Summary

This report analyzes the Wiehle South reach in Reston, Virginia to support future stream redesign. Reston began development in the 1960s as a planned community in the suburbs of D.C., as the community developed, the impervious cover of the area increased without significant or effective stormwater management to compensate. The resultant increased stormwater runoff has eroded the streams in this community and negatively affected both the stream aesthetics and water quality downstream. Due to this the community of Reston has requested that Smart Stream Solutions Inc. develop a stream remediation design proposal. This report details the selection of a design storm using current literature and site characteristics. A design storm with a return period of 1-year, and its associated 24-hour rainfall of 2.62 inches will be used for the hydrologic calculations needed to determine the appropriate channel cross-sectional size (Fairfax Co. PFM 6-89).

II. Discussion

Per the Virginia Stormwater Management Handbook, urbanization tends to decrease the time of concentration which increases peak discharge (4-36). Urbanization also reduces potential evapotranspiration (PET) and infiltration, further increases runoff. Runoff due to increased impervious cover causes higher flood frequency, increased peak flow, increased sediment load, and impaired water quality (Leopold, 1968; Paul, 2001). These impacts provide a justification for action to mitigate the effects of urbanization on the Wiehle South reach.

Wiehle South is currently classified under the Rosgen Classification System as a a class F4 or G4 stream. These streams are deeply incised in alluvial valleys, resulting in the abandonment of former floodplains (Rosgen, 1997). Sediment supply is high, due to high erodibility of the banks, leading to high deposition downstream. This F4, G4 structure leads to issues with downstream pollution, as phosphorus bound up in stream sediment will be heavily deposited downstream and carried through the Chesapeake Bay watershed. Channel reconstruction is a primary practice for remediating degraded streams (Shields, et al., 2003).

Our recommendation is to restore the Wiehle South reach and the surrounding ecosystem by reconnecting the channel to its floodplain, from an entrenched Rosgen class F4 to a B4 type stream (WSS, 2012, p. 39). This stream type is less sinuous, less entrenched, and more prone to bankfull flow, which alleviates issues related to erosion and sediment discharge by allowing frequent flow dispersion and sediment deposition across the floodplain. Various methods for creating these channels exist, including the raising of the streambed or lowering the floodplain; we recommend raising the streambed, as it leads to replacing fewer trees, less waste, and a narrower width of disturbance than does excavating the floodplain (Rolband, 2018).

Designing a B4 channel necessitates an understanding of design storm intensity, which is determined by the desired flood frequency of the stream. The following list of terms defines differences in flooding factors and measurements used:

- **Bankfull elevation:** highest stream *elevation* without spilling into the floodplain (Leopold and Wolman, 1957).
- **Bankfull discharge**: maximum *streamflow* that a channel can convey without overflowing onto the floodplain (Copeland, 2000).
- **Channel-forming discharge**: the dominant discharge used for restored channel design (USDA NRCS, 1998, 7-3). It is equivalent to bankfull discharge (Copeland, 2000), (USDA NRCS, 1998, 7-12).
- **Effective discharge**: the discharge that transports the most sediment over time, occuring at approximately 1:1 ratio of bankfull discharge (Powell, et al., 2006, p.46). Bankfull and effective discharges cannot be calculated by using a generic recurrence interval or sediment transport percentage (Powell, et al., 2006, p.46).
- **Bankfull stage:** highest elevation at the top of channel bars, identified by the change in vegetation, notably riparian grasses and shrubs (Leopold and Wolman, 1957; Leopold, 1994). Given instability of streambanks prior to restoration, this metric may be impractical or impossible to determine (USACE, 2000).
- **Return period**: represents the frequency with which the bankfull discharge occurs. There can be difficulty in estimating the return period (USDA NRCS, 1998, 7-12).

Bankfull discharge is the most important metric for our purposes in determining a design storm. It is considered to have morphological significance because it represents the breakpoint between the processes of channel formation and floodplain formation (USACE, 2000). Bankfull discharge forms and maintains the channel, carries most sediment, and is the threshold for flooding (Leopold, 1994, p. 141). It is therefore our metric for what the channel should contain, given the desired transformation of the channel to a less entrenched, more floodplain-connected state.

Channel-forming discharge is a concept based on the idea that for any given alluvial stream there exists a single discharge that dominates channel form and process (USDA NRCS, 2007). Given enough time, channel-forming discharge would produce a channel width, depth, and slope equivalent to those produced by a pre-developed state of a stream, ultimately finding equilibrium as a new, standard bankfull discharge. The channel-forming discharge is used for restored channel design, although engineers should be cautious in using historical streamflow data because of the effects of urbanization, channel modification, and hydrologic infrastructure (USDA NRCS, 1998, 7-3). With the goal of creating a stream with pre-development stability, bankfull discharge will be informed by a design storm selected for frequent flooding.

Williams (1978) concluded that since there is a wide variation in bankfull discharge with a 1.01-32 year return, it's too variable for an absolute standard (Charlton, 2008, p. 32), (Knighton, 2016). Active floodplain rivers feature a bankfull discharge with a 1-2 year return period on average (Charlton, 2008, p. 32). Leopold (1994) describes the bankfull discharge recurrence interval as a constant, occurring approximately every 1.5 years (p. 134). Using field surveys and flood frequency curves from streamgages in free-flowing rivers the estimate ranges from 1-2.5 years (p. 135). Similar recurrence intervals are 1-1.5 year (average 1.3) and between one and two years (Cinotto, 2013, p.2; Krstolic, 2007, p. 8). For rural watersheds in the Midwest, the recurrence interval of the channel forming discharge range from 0.3 to 1.4 years, as determined by an annual peaks series simple log regression but is questionable below one year (Powell, et al., 2006, p.45).

According to Rosgen (1996), North American stream gage data averaged over a 10 year period results in a bankfull discharge return interval of 1.4-1.6 years (2-4). Rosgen later claimed bankfull discharge to occur within a range of 1.05-1.8 years (2009). Urban watersheds often exhibit shorter return intervals of around 1.2 years (log pearson flood frequency analysis).

III. Analysis

Following the literature review, the target capacity of the remediated stream can be determined. Since Wiehle South is in a 282.2 acre sub-watershed with 26.6% impervious cover (WSS, 2012, p. 35), a shorter recurrence interval can be used because of the increased runoff. According to literature reviews, a minimum return interval of one year was found (Charlton 2008, Leopold 1994, Cinotto 2013, Krstolic 2007). Due to the fact that Wiehle South reach is in a forested park area, more frequent small-scale floodings do not impact neighboring area significantly. Using a smaller return period for bankfull discharge return period will correlate to a smaller and likely less costly stream. Due to these reason, a bankfull discharge return period of one year is proposed. The design storm will be based on this one year bankfull discharge, meaning that the stream will be designed to contain a one-year storm. This storm can be used to determine a channel flow rate, which can be used to determine channel geometric properties.

Table 6.19 Storm Volume in Inches of Rainfall* (122-16-PFM)

Duration of Storm

Frequency	5 Min	10 Min	15 Min	30 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
1 Yr	0.355	0.567	0.708	0.971	1.21	1.42	1.52	1.87	2.28	2.62
2 Yr	0.426	0.681	0.856	1.18	1.48	1.74	1.85	2.27	2.75	3.17
5 Yr	0.506	0.810	1.02	1.46	1.87	2.20	2.35	2.87	3.49	4.07
10 Yr	0.565	0.904	1.14	1.66	2.16	2.56	2.75	3.36	4.12	4.87
25 Yr	0.641	1.02	1.30	1.92	2.56	3.08	3.32	4.08	5.08	6.09
50 Yr	0.698	1.11	1.41	2.12	2.87	3.50	3.79	4.70	5.92	7.18
100 Yr	0.754	1.20	1.52	2.32	3.20	3.95	4.29	5.37	6.85	8.41
Max Prob								27.0		

* Storm Volumes from NOAA Atlas 14 for the Vienna Tysons Corner Station (Station ID:44-8737) except for the maximum probable storm which is from NWS Hydrometeorological Report No. 51.

Fairfax County (2011). Public Facilities Manual (PFM). Ch. 6.

Based on the data retrieved from NOAA Atlas 14 point precipitation frequency estimates, provided by Hydrometeorological Design Studies Center Precipitation Data Server (PFDS), the study area , Fairfax County, VA, has has a precipitation value of 2.62 inches for a one-year, 24-hour storm (Station ID: 44-8737; Station Name: Vienna Tysons Corner).

DEM images and NHD flowline data will be combined with GIS software, to define a watershed or watersheds of the stream. Subsequent variables of the wetlands will be defined using GIS software and following regional literatures including Fairfax County Public Facilities Manual and Virginia Stormwater Management Handbook.

Using collected data and information, the flow rate will be calculated for the 1 year storm using six methods :

1. Snyder method
2. Anderson method
3. NRCS TR-55 method
4. USGS Virginia Urban Regression equations
5. Loudoun County Regression equations
6. Rational Method
7. Wiehle Regional Design Curve

These flow rates will be compared to the original two year design storm on the WSSI planset of 113 cfs (WSS 2012), and a chosen design flow rate will be used to begin the design of physical components of this reach such as depth and width.

It is important that the design is capable of withstanding future conditions, so an evaluation of potential changes to the watershed is necessary. The most likely change to a watershed is typically redevelopment. While there may be some major redevelopment, particularly on the present golf course site in the watershed, city regulations require on site stormwater management. Due to this the impacts of any redevelopment should be negligible.

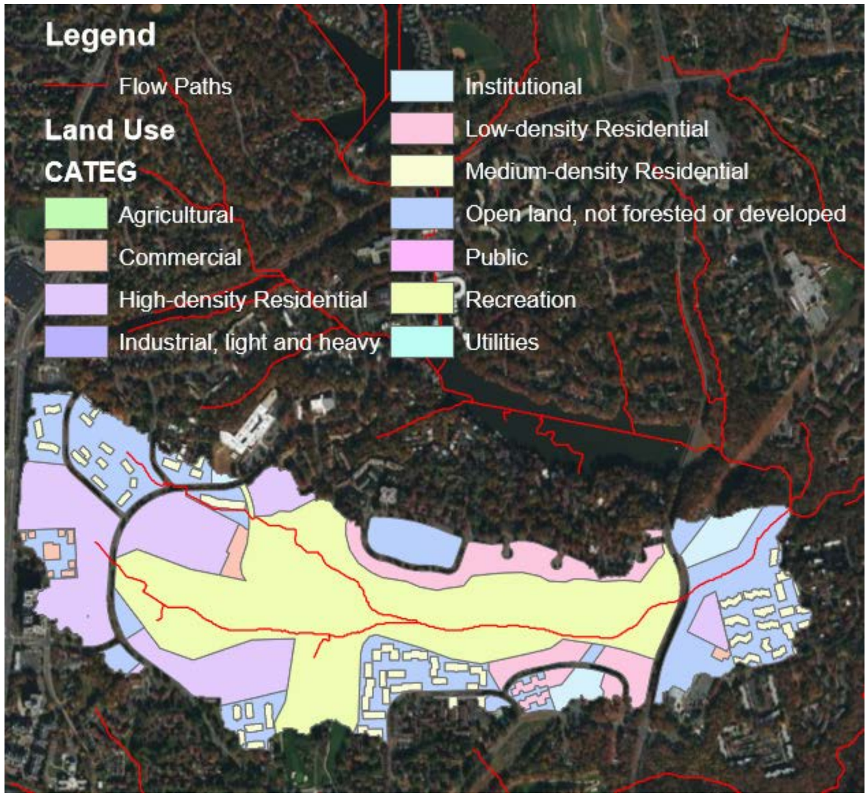
IV. Conclusion

Bankfull discharges dominate the formation and transformation of channels, as most sediment transport occurs during these events. Due to this, in order to build a stable remediated stream channel, the bankfull discharge return period must be within natural bounds. This can be done by determining the flow rate for Wiehle South as well as the physical dimensions using the bankfull discharge return period as a starting point.

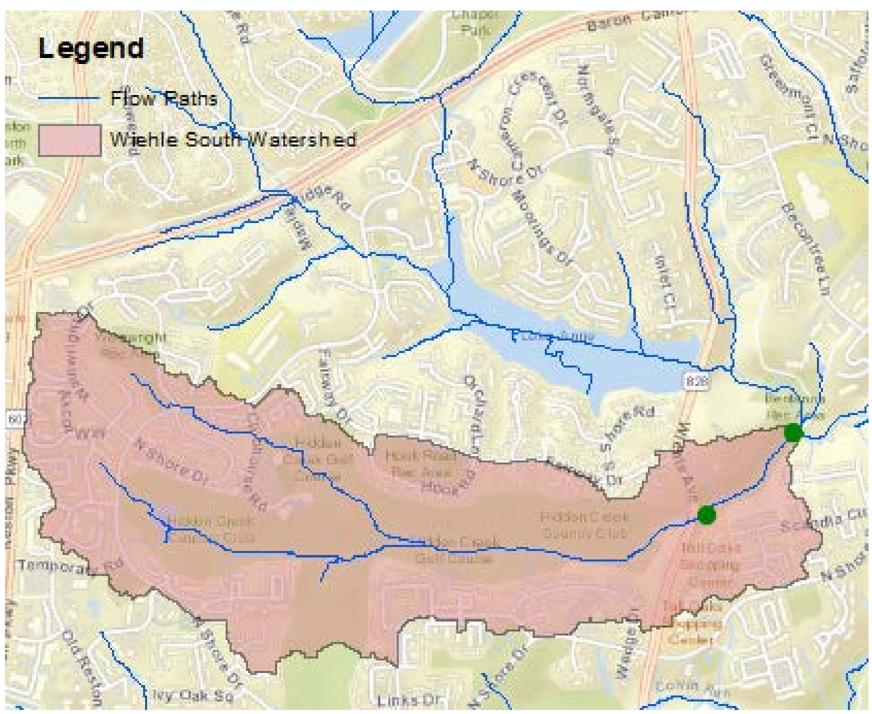
With no increased runoff into Wiehle South and remediation of the stream channel to resemble its pre-entrenched form, the design storm for the stream can be compared to the bankfull discharge of similar natural streams in the region. The current plan set cites Maryland Piedmont data for reference curves (WSS, 2012). Smart Stream Solutions will calculate potential discharge using various hydrologic flow calculation methods in an upcoming report.

Literature review suggested that the minimum return period for bankfull discharge is one year. A smaller return period would correlate with a smaller thus less expensive stream channel construction project. Since the Wiehle South reach is located in a forested park, problems that could result from annual flooding are minimal. Smart Stream Solutions recommends that Wiehle South be designed to have a bankfull discharge return period of one year. This will be accomplished by using a one-year storm as the design storm.

V. Maps



Land Use Digitized: Wiehle South Watershed



Channel Flow: Wiehle South Watershed (Subject bounded by green points)

Wiehle South Watershed in 1937[1]

Wiehle South Watershed in 1972

Wiehle South Watershed in 1990[2]

Wiehle South Watershed in 2017

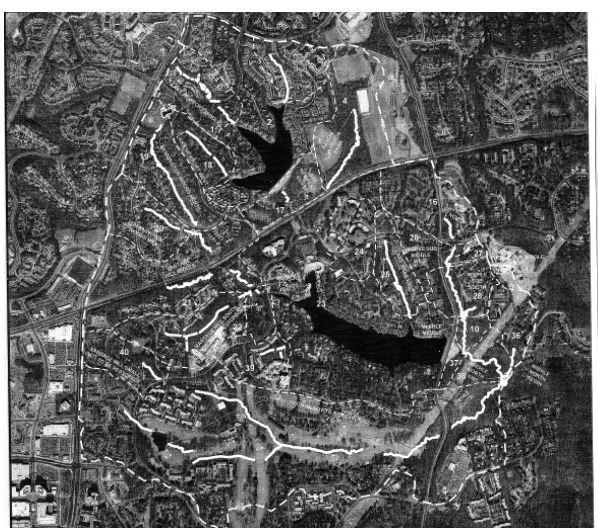


Figure 7 - Wiehle Watersheds

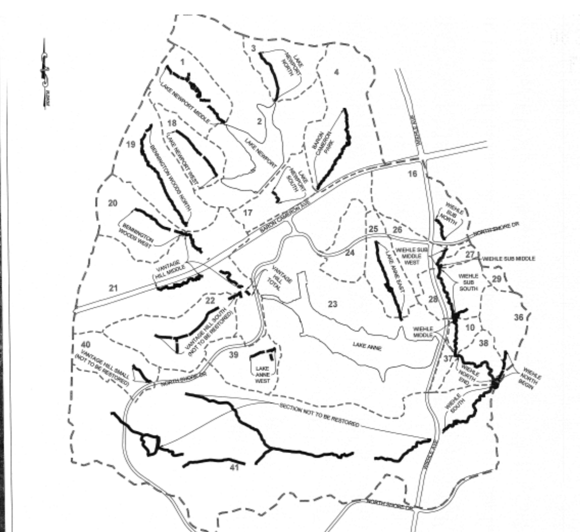


Figure 8 - Wiehle Reach Locations

WSS Plan: Watershed Maps

[1] Fairfax County historic aerials (https://www.fairfaxcounty.gov/maps/aerial-photography)

[2] Fairfax County historic aerials (https://www.fairfaxcounty.gov/maps/aerial-photography)

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Assignment #2: Wiehle South Discharge Calculation

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Draft: March 5, 2018

Revision 1: March 21, 2018

Revision 2: April 2, 2018

Revision 3: April 9, 2018

Final Submitted: April 16, 2018

I. Executive Summary

The increase in impervious cover in the Wiehle South watershed due to development has resulted in higher peak flows from precipitation events. This has created a need to restore natural streams to contain the additional flow. The purpose of this assignment is to calculate the peak discharge for a 1-year storm in the Wiehle South watershed. This discharge estimate will be used to design a new channel for the Wiehle South stream reach, since it is desired that the reach will contain the flow from a 1-year storm at bankfull height.

To improve the validity of the discharge estimate, ten different methods were used in the analysis: Loudon County, NRCS TR-55, Rational Method, Anderson, US Geological Survey-Virginia, Snyder, the StreamStats function in ArcGIS, the regional curve for rural Maryland, the regional curve for rural Maryland with an enlargement factor, and the WSSI 1-year storm estimate.

These methods yielded results with significant discrepancies among them. Thus, a range of potential values was collected and analysed to provide an upper and lower bound for the 1-yr discharge estimate. The results from many of the calculation methods had to be rejected due to poor applicability to the relevant watershed and storm frequency. It was decided that from among the methods considered, the most valid estimates were obtained from the TR-55 method, US Geological Survey method, the WSSI 1-year storm estimate, the regional curve for rural Maryland with an enlargement factor, and the regional curve without enlargement. These five values were averaged to obtain a final estimate for the 1-year storm peak flow rate. The value obtained was 80 cfs. This peak flow rate will be used in future assignments to determine the geometry of the the channel.

The 80 cfs flow rate is lower than the value used in the original 2012 construction plan (113 cfs), which suggests that a reduction in cost is possible. The use of a lower flow rate should not be a cause for concern since the area immediately adjacent to the stream consists mostly of trails rather than residential buildings. Strong conservatism is not necessary since the risk of harm to people or property is low even with a slightly more frequent overflowing of the stream.

II. Introduction

The Wiehle South watershed is located south of Lake Anne, with the outlet located at latitude 38.96, longitude -77.3. The watershed spans 282 acres, or 0.44 square miles (Figure 1). Due to changes in land use, the impervious area has increased since the development of the township and is currently 26.6% (Figure 2). The majority of the watershed is covered by hydrologic soil group B, although a large part of the upstream reach is covered by soil group C. There are significant pockets of soil group D, particularly in the reach of interest. Overall, soil group D represents about 3.5% of the total watershed area (Figure 3).

To contain the increased peak discharge due to greater impervious cover, it has become necessary to reconstruct and restore stream reaches in the area to contain higher flow rates. A previous restoration plan for the Wiehle South stream reach was developed in 2012; however, it was not implemented due to high cost. It is possible that the previous design was overly conservative in its estimate of the design storm peak flow rate (113 cfs). Thus, it is worth further exploring the design storm flow rate for the Wiehle South reach.

The first step in developing a new restoration plan for the Wiehle South stream reach was to determine the design storm for which to size the channel. The team decided to design a stream to hold the peak discharge from a 1-year storm at bankfull capacity. Please refer to 'Assignment 1: Wiehle South Design Storm Report' for more details on this decision.

The purpose of this exercise was to use 6 different methods of calculating peak discharge to obtain a valid estimate of the required channel capacity to hold the flow from a 1-year storm. The Loudon County method, NRCS TR-55 method, the Rational Method, the Anderson Method, the Snyder Method, ArcGIS StreamStats, the WSSI 1-yr estimate, and regional curves for rural Maryland with and without enlargement were used to obtain peak discharge estimates. The methodology and results of each method are described in the following sections.

While many of these methods yielded drastically different results, some had to be rejected for their poor applicability in Wiehle South watershed and for high-frequency storms. Since many hydrologic models are rooted in empirical evidence rather than a theoretical basis, they do not always generalize well to a wide range of watershed sizes and characteristics. Once calculations had been done using all the selected methods, the most appropriate were averaged to produce the final design storm flow rate.

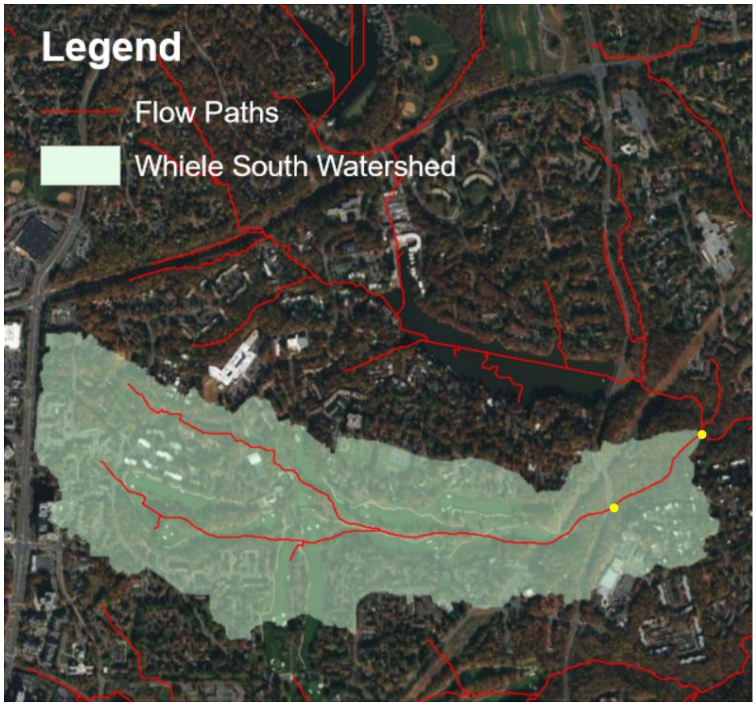


Figure 1: Wiehle South watershed with flow paths. Wiehle South, which is the reach planned to be restored, is shown bounded by points in yellow.

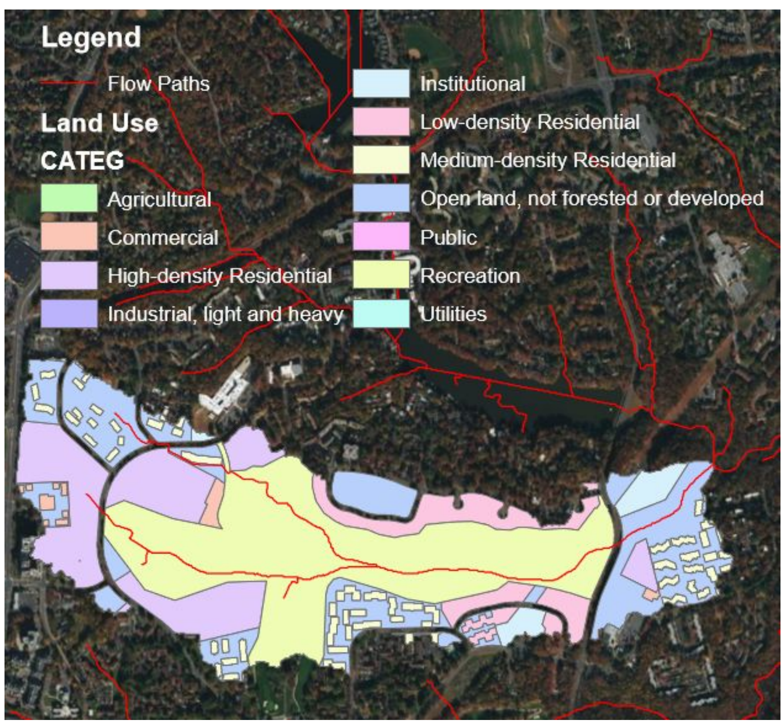


Figure 2: Land use in the Wiehle South watershed. The watershed is characterized by mostly residential and recreational use. Land use is not expected to change heavily in the coming years since the area is already developed.

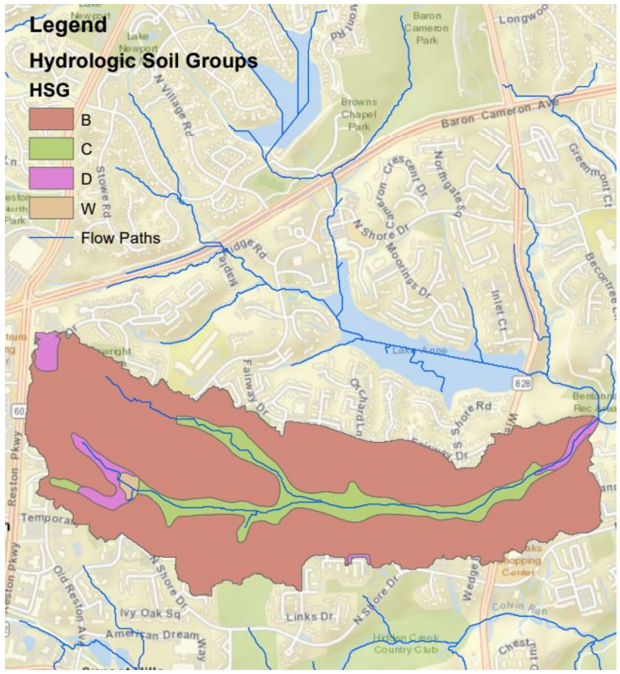


Figure 3: Hydrologic soil groups in the Wiehle South watershed.

III. Methods

1. Loudon County Method

Background:

The authors of the Loudon County method tested a wide range of factors affecting storm flows, including drainage area, percent forest cover, percent agricultural land use, percent impervious area, basin relief, and percent cover of various hydrologic soil groups. It was determined using multiple linear regression analysis for 38 gauging stations in VA and MD, that following four factors are the only statistically significant variables for the area.

1. Drainage area (A) is square miles
2. Channel Slope (SL) in ft/mile
3. Impervious Area (IA) in percentage of drainage area
4. D soil cover (Dsoil) in percentage of drainage area

Based on the regression analysis, the flow rate for a storm of a given annual exceedance probability was expressed as a function of the above four variables.

Calculations:

The storm flow for a given exceedance probability 'x' can thus be expressed as a function of these four statistically significant variables as follows:

$$Q_x = \alpha A^{\beta}(IA + 1)^{\gamma}(SL)^{\delta}(Dsoil + 1)^{\varepsilon}$$

Where empirical parameters $\alpha, \beta, \gamma, \delta,$ and ε are set based on exceedance probability 'x'.

Based on the regression analysis, the developers found the required parameters for a fixed set of storm events with exceedance probabilities of 50%, 20%, 10%, 4%, 2%, 1% and 0.2%. The most frequent storm that they analysed was the storm with 50% annual exceedance probability, i.e. the 2-year storm. The purpose of this study was to find the storm flow corresponding to a 1-year storm, which was not directly addressed in the Loudon County analysis. Thus, a curve was set up using all the return frequencies analysed in the Loudon County document, and the flow rate for the 1-yr storm was estimated from this curve. Since the extreme low-frequency storms resulted in a poor curve fit, they were eventually removed for the process of extrapolating the 1-yr storm flow rate. The 2-year storm discharge, which could be calculated directly and was thus more reliable, was used as an upper bound value in the final determination of the 1-year storm discharge.

Table 1: Calculations for 2, 5, 10, 25, 50, 100, and 500-year storms using the Loudon County method for the Wiehle South watershed (Q represented in cfs)

A (mi^2)	IA (%)	SL ft/mi	Dsoil (%)				
0.441	26.6	85.8000858	3.52				
aep	return freq	alpha	beta	gamma	delta	epsilon	Q
0.5	2	32.5	0.672	0.245	0.248	0.204	173.4291174
0.2	5	86.9	0.662	0.191	0.213	0.173	319.1711228
0.1	10	139.7	0.662	0.162	0.209	0.159	448.2372745
0.04	25	220.7	0.666	0.132	0.219	0.152	661.018
0.02	50	303.4	0.67	0.111	0.224	0.145	854.7255027
0.01	100	388.8	0.676	0.093	0.237	0.142	1083.02264
0.002	500	535.1	0.69	0.061	0.306	0.149	1820.773406

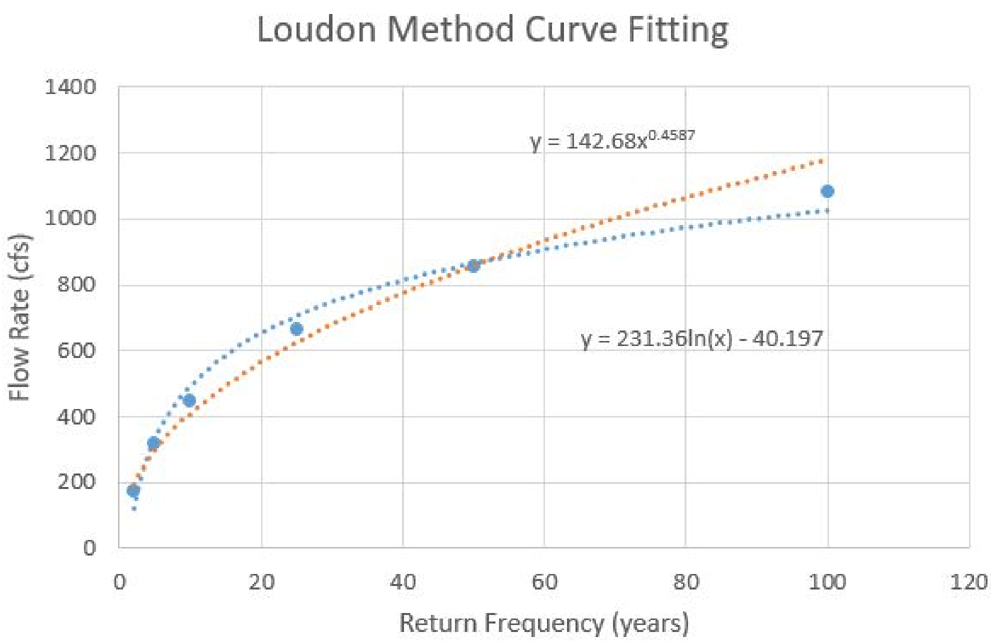


Figure 4: Curve fitting to points calculated using the Loudon Method. By fitting a function to this curve, the flow rate associated with the 1-year storm can be estimated. The logarithmic function is displayed in blue, while the power function is displayed in orange. The data points for each return frequency analyzed are represented by blue circles. Log-fitting with the inclusion of the 50 and 100-year storms yields a negative value for the 1-year storm (-40 cfs), which is invalid.

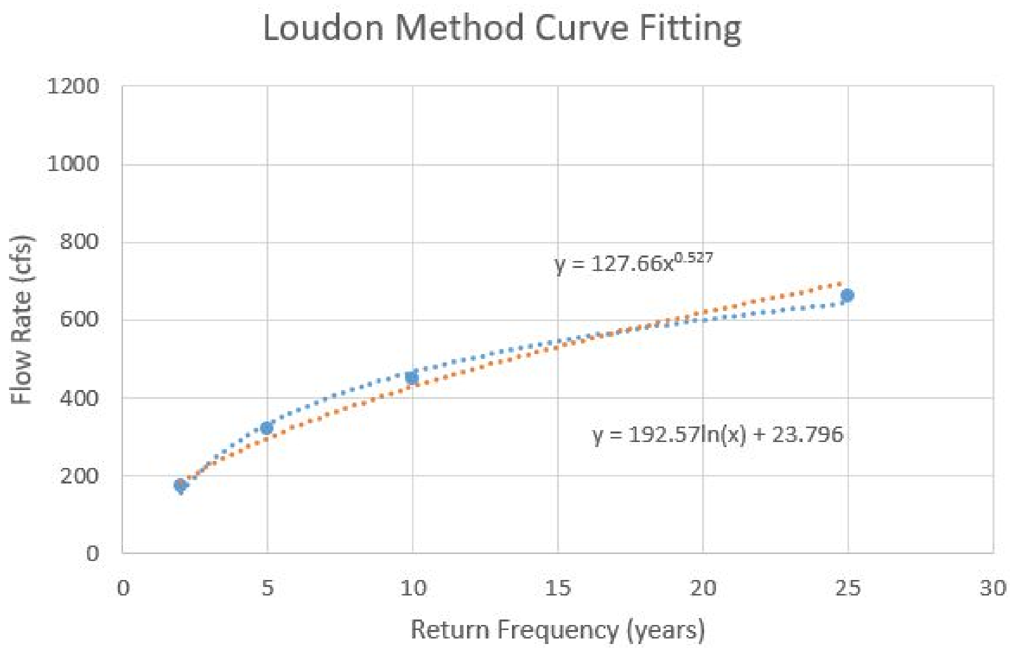


Figure 5: Curve fitting with the exclusion of 50 and 100-year storms. This logarithmic curve fit gives a value of about 24 cfs for the 1-year storm, which is low, but not physically impossible.

Determining Values for Relevant Variables:

The drainage area for the Wiehle South watershed was found using the hydrology tools in ArcGIS. This was determined to be 1.14 square kilometers, or 0.44 square miles. The percentage of the impervious area was found using land use data in ArcGIS. The channel slope was calculated from the construction plans for the Wiehle South reach. The percent of D soil cover in the watershed was also calculated using hydrologic soil groups data in ArcGIS.

Limitations:

This method was found to not be applicable to watersheds with the following characteristics:

- Significant floodplain storage
- Drainage area outside the range of 0.28 to 332 square miles
- Impervious area outside the range of 0.0 to 41.1 percent
- Channel slope outside the range of 5.96 ft/mile and 100 ft/mile
- D soil cover outside the range of 0.0 to 70.67 percent

The Wiehle South watershed fits within all of these required thresholds.

The second limitation of this method is that parameters are determined for only a fixed set of return frequencies, between once every 2 years and once every 500 years. The discharge for any other return frequency needs to be extrapolated using a curve fit.

The final and most significant limitation of this method is the low goodness of fit observed in the regression analysis. When comparing flow rate estimates from the Loudon County method with measured flow rates from gages, the fit was poor for a some stations, particularly for smaller watersheds.

Result:

The purpose of this exercise is to obtain a valid estimate of the peak discharge for a 1-year storm. The most frequent storm discharge that can be calculated using the Loudon method is the 2-year storm. Although the curve fitting method described above can provide an extrapolated estimate of the 1-year storm discharge, it involves a very high level of uncertainty. It is thus best for this exercise to use the Loudon Method for calculating the peak discharge for a 2-year storm, and treat this value as an upper-bound in our estimate of the peak discharge for a 1-year storm.

The peak discharge for a 2-years storm in the Wiehle South watershed was calculated to be 173 cfs, which is a reasonable upper bound for the 1-year storm discharge based on results from the other methods included in this analysis.

2. NRCS TR-55 Method

Background:

TR-55 was developed by the USDA NRCS as a method for analyzing hydrology in small urban watersheds. Urbanization in a watershed modifies the hydrologic regime, as an increase in impervious surface reduces infiltration and travel time of precipitation, resulting in higher peak discharges and runoff.

Limitations:

The TR-55 method calls for estimation of several parameters that are difficult to observe, particularly in the calculation of t_c (time of concentration). The method calls for separation of t_c into three components, namely sheet flow, shallow concentrated flow, and channel flow. The TR-55 manual suggests that sheet flow shouldn’t last more than 300 ft, however there is no rigorous evidence for why this value should be used. The Fairfax County Public Facilities Manual suggests that sheet flow should end at a distance of 200 ft from the start of the flow path.

The TR-55 method works best when watershed-specific observations are available. This allows for a more accurate division of flow paths into sheet flow, shallow concentrated flow, and channel flow. Past projects by Wetland Studies and Solutions, Inc. have demonstrated that the

TR-55 method works well in nearby watersheds, suggesting that it is applicable to the Wiehle South watershed.

Based on recommendations from Dr. Todd Walter, a professor in the department of Biological and Environmental Engineering at Cornell University, the Kirpich equation was also used to calculate t_c for this method. The values for t_c using the two different methods were compared.

Calculations:

TR-55 requires information on flow length, slope, roughness, channel characteristics, and rainfall in order to calculate the travel time / time of concentration of the flow in the storm event. The longest hydraulic flow path is used to determine the time of concentration, so that the model calculates peak discharge at the time when the whole watershed is contributing to the flow.

Once the longest hydraulic path has been determined, t_c is calculated as a the sum of three different travel times.

1. Sheet flow: Sheet flow is observed at the edges of a watershed, usually a few hundred feet from where a water drop first falls at the watershed boundary. In this analysis, sheet flow was set as beginning at the boundary of the watershed and lasting for 200 ft. into the longest flow path (Figure 6).

Travel time for sheet flow is calculated as follows:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}s^{0.4}}$$

Where
T_t = travel time for sheet flow (hr)
n = Manning roughness coefficient
L = flow length (ft)
P_2 = 2-yr, 24 hr rainfall (in)
s = slope of hydraulic grade line (ft/ft)

2. Shallow concentrated flow: This form of flow is observed when water from sheet flow get consolidates into shallow paths. In this analysis, shallow concentrated flow was set as the segment of the longest flow path between the 100 ft mark and the beginning of the NHD flowpath line (Figure 6). The flow lengths for sheet flow, concentrated flow, and channel flow were determined in a somewhat arbitrary manner, since the team was unable to survey the area in person. S^3 used a combination of satellite imagery, contour maps, and information from the WSSI plan set to determine reasonable lengths for the

longest flow path used to calculate the time of concentration in the TR-55 peak discharge calculation.

Travel time for shallow concentrated flow is calculated as follows:

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

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

3. Channel flow: This form is flow is set wherever data is available for channel cross-sections, or wherever NHD flowline data is set. For this analysis, channel flow was taken to begin where the NHD flowlines begin (Figure 6). The travel time for channel flow is calculated using the same equation as for shallow concentrated flow. The channel flow was calculated in one continuous channel reach because the dimensions of the channel through the golf course upstream of the Wiehle South reach are unknown. Per the suggestion of Mike Rolband, the team used the available 2’ contour map from Fairfax County in its determination of the channel dimensions for the TR-55. As a result, there is limited accuracy in the determination of the cross section flow area, wetted perimeter, and hydraulic radius used in the calculation. Were SSS able to visit the site in person and perform a channel survey, our T_c calculation would be less limited to assumptions about the present Wiehle South channel morphology, particularly the reach flowing through the Hidden Creek Golf Course.

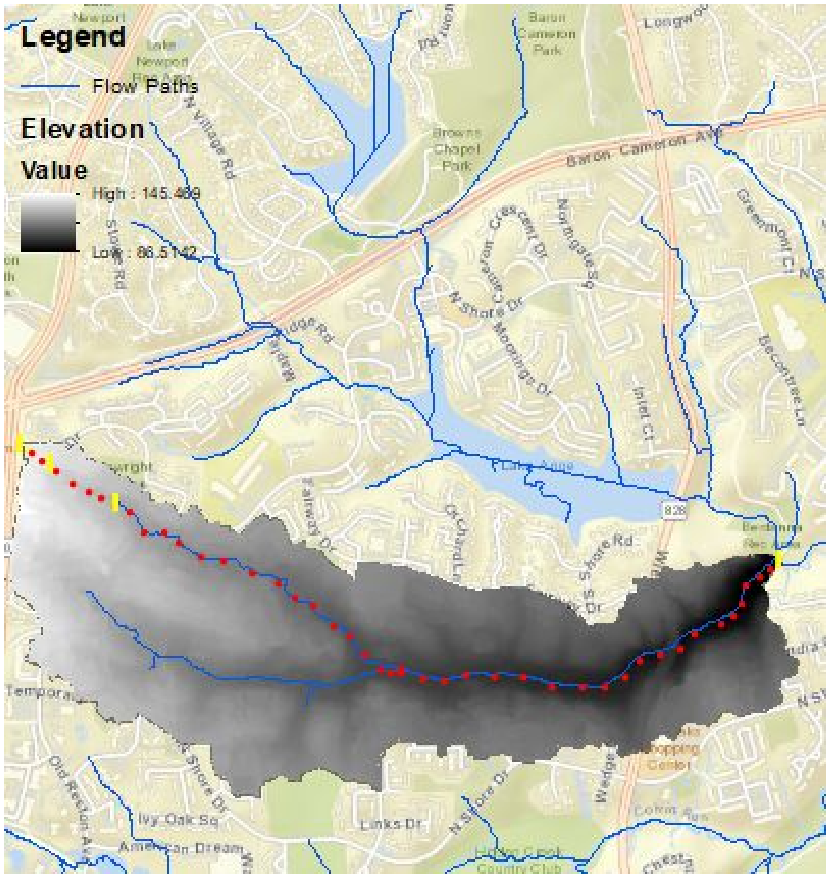


Figure 6: Representation of the longest estimated flow path. The red dots indicate the longest path for a drop of water to travel through the watershed into the outlet. The sheet flow section begins at the watershed boundary and is located between the first and second yellow markers. The shallow concentrated segment is located between the second and third yellow marker. The channel flow segment extends from the third yellow marker to the fourth yellow marker at the outlet.

For calculating travel time for both shallow concentrated flow and channel flow, velocity estimates were required. For shallow concentrated flow, velocity was estimate graphically using empirical relationships between average velocity and watercourse slope. For channel flow, velocity was estimated using manning's equation along with data from the Wiehle South Construction Plan Set:

$$V = \frac{1.49 r^{2/3} s^{1/2}}{n}$$

Where
V = average velocity (ft/s)
r = hydraulic radius (ft) [calculated as cross-sectional area/ wetted perimeter]
s = slope of hydraulic grade line (ft/ft)
n = manning's roughness coefficient

Table 2: Calculations for Tc (time of concentration) using TR-55

Values for Tc and Tt		
Sheet flow		
Manning roughness[1], n	0.1	
Flow length, L	100	ft
2 yr 24 hr rainfall[2], P_2	3.17	in
Land slope, s	0.08	ft/ft
Tt 1[3]	0.07	hr
Shallow concentrated flow		
Flow length	1800	ft
Watercourse slope (slope between max and outlet elevation)	0.03	ft/ft
Average velocity	3	ft/s
Tt 2[4]	0.18	hr
Channel flow		
Cross sec flow area[5], a	3.2	ft2
Wetted perimeter, pw	5.6	ft2
Hydraulic radius, r = a/pw	0.57	ft2
Channel slope, s (Valley Slope/Sinuosity)	0.017	ft/ft
Manning roughness[6], n	0.035	
Flow velocity[7], V	3.817	ft/s
Flow length, L	5200	ft
Tt 3	0.38	hr
TOTAL Tc	0.62	hr

[1] PFM
[2] NOAA Atlas 14
[3] TR-55 eq 3-3
[4] TR-55 figure 3-1
[5] Fairfax County 2' Contour Map
[6] VA Erosion Sediment Control Manual Ch. 5 V-65
[7] TR-55 figure 3-1

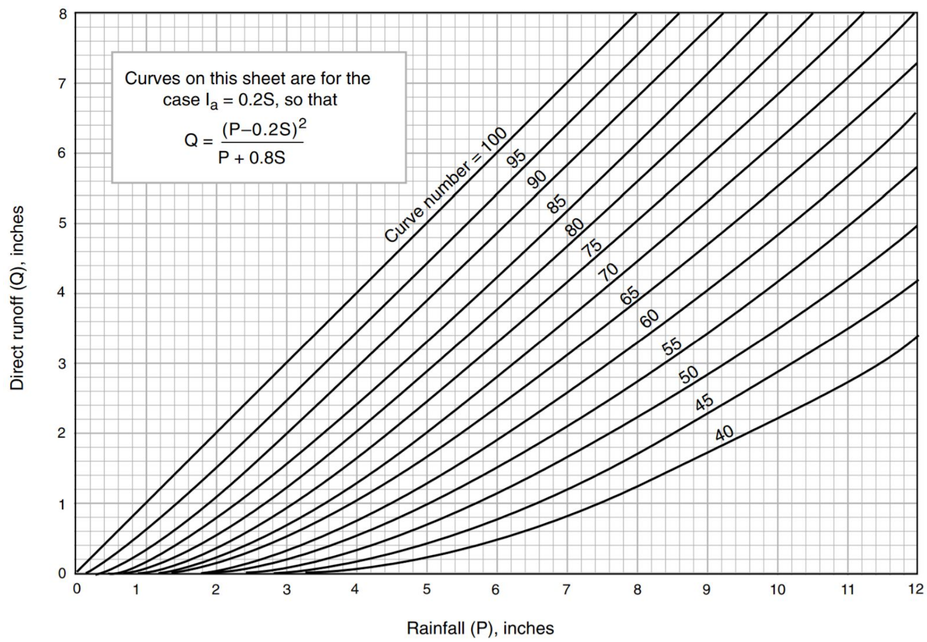


Figure 7: Direct runoff (inches) vs. Rainfall (inches) for various curve number values

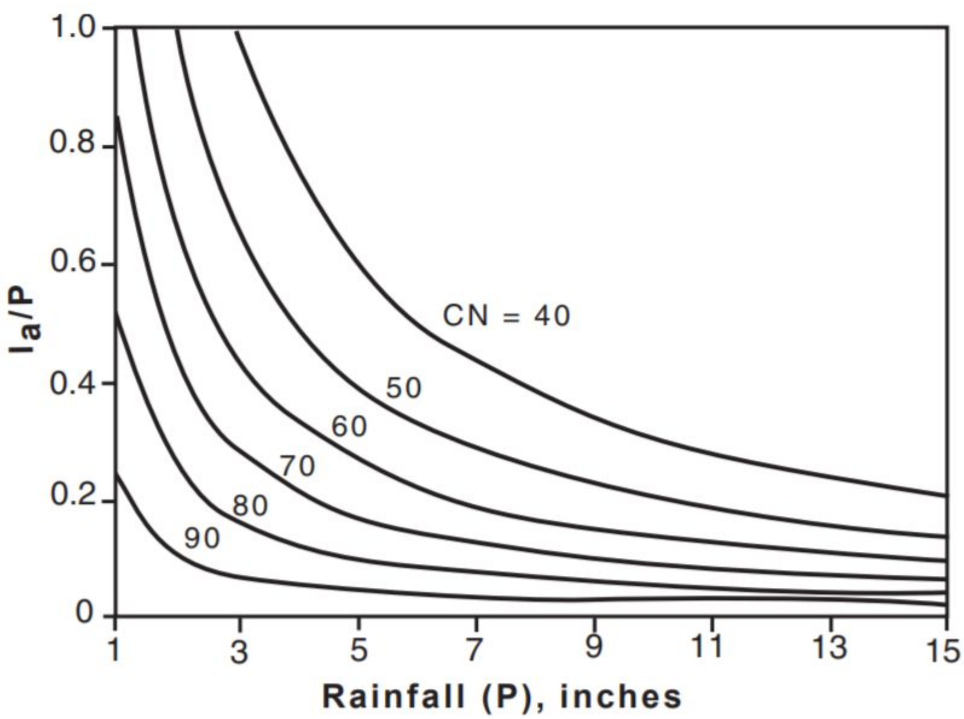


Figure 8: Variation in the ratio of initial abstraction to rainfall for various rainfall levels and curve numbers

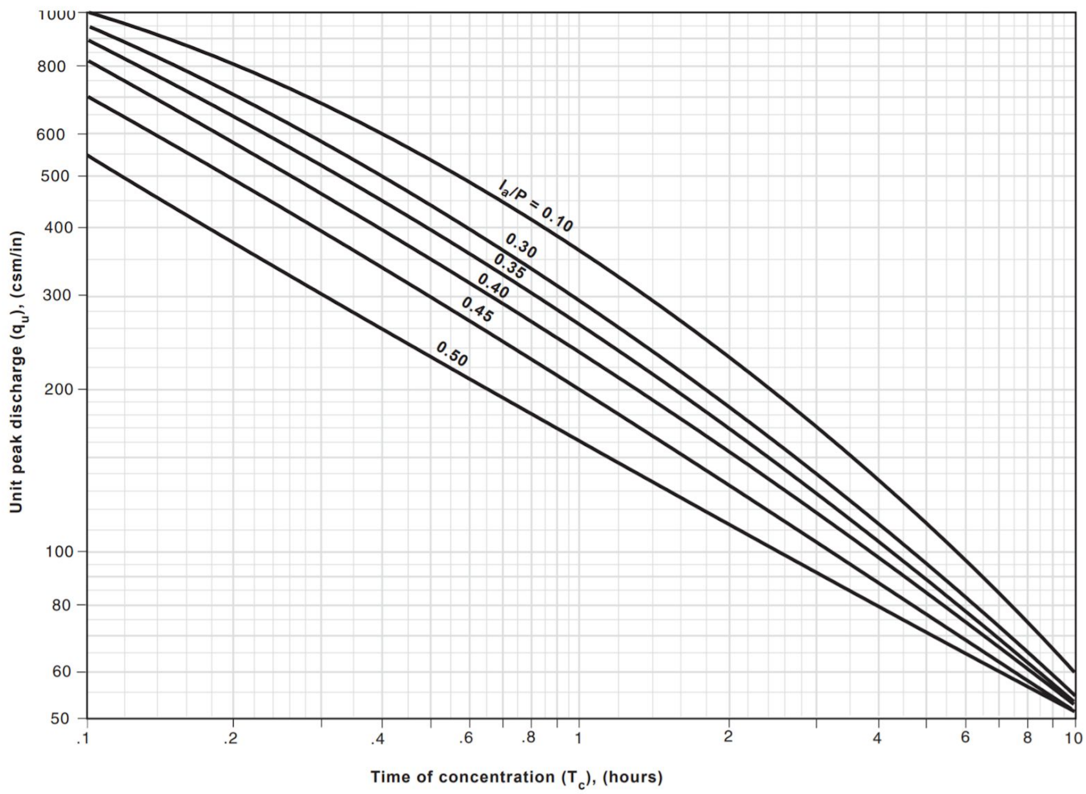


Figure 9: Unit peak discharge for various time of concentration values and ratios of initial abstraction to rainfall

The graphical TR-55 method includes charts for calculating the runoff (Table 2-1), the proportion of precipitation lost to infiltration (Figure 4-1), and peak discharge based on precipitation for the one year storm event, percent impervious area, the runoff curve number, and the drainage area (Exhibit 4-II). Reston, VA is within the NRCS type II rainfall distribution zone. The watershed area and runoff curve number were sourced from the Wetland Studies and Solutions plan. The 24 hour rainfall and Manning's roughness coefficient are per the Fairfax County PFM.

Table 3: Peak discharge calculations using the graphical TR-55 method, D-4

TR-55 Graphical Peak Discharge Method D-4	**Value**	**Unit**
Runoff curve number[8], CN	71	
Runoff[9], Q	0.6	in
Drainage area[10], Am	0.44	sq. mi
24hr rainfall[11], P	2.62	in
Time of concentration, Tc	0.66	hr
Travel time through area, Tt	0.66	hr
Pond/swamp adjustment factor, Fp	1	
Unit peak discharge[12], (qu)	370	csm/in
Ia[13]	0.81	in
Ia/P	0.31	
Peak discharge[14] (qp)=(qu)(Am)(Q)(Fp)	**97.68**	**cfs**

Analysis:

TR-55 method for Tc and runoff:

The 100 cfs peak discharge value for a one year storm produced by the TR-55 method is comparable to the 113 cfs design discharge produced by the Maryland regional reference curve with enlargement factor (WSS Plan). The more detailed, multi-variable nature of the TR-55 calculation may produce a more accurate model, but also requires more assumptions.

Based on recommendations from Dr. Todd Walter, a professor of hydrology at Cornell University, the Kirpich equation was also used to calculate T_c for application in the TR-55 method. All other aspects of calculation were consistent with the instructions in the TR-55 manual. The Kirpich time of concentration was found to be 38 minutes, which matched the time of concentration found using the TR-55 method, giving very similar final flow rate estimates. This served to further validate the results from the TR-55 method.

[8] WSS Plan
[9] TR-55 table 2-1
[10] WSS Plan
[11] FC PFM
[12] TR55 chart 4-II
[13] TR-55 table 4-1
[14] TR-55 eq 4-1

3. Rational Method

Background:

The Rational method, also known as Lloyd-Davies method, was developed in 1889 for small drainage basins in the urban area. It remains to be one of the core equations for urban stormwater design. The method assesses the urban peak runoff rate for designing small structures.

Limitations:

The Rational method has several methods to calculate the time of concentration. Kirpich and Soil Conservation Service were employed for this report. The peak urban discharge flow results differ from each other. Since literature describes the Rational method is designed for estimating peak runoff rate for a small area within "few tens of acres," there are potentials for inaccurate assessment.

Calculation:

The Rational method requires C, the runoff coefficient (tabulated based on land use), i, the rainfall intensity (in/hr), and A the watershed area (acres) to calculate peak runoff rate (cfs).

$$q_p = CiA$$

The duration of the design storm equals to the watershed's time of concentration, T_c.

Kirpich Equation (1940) defines the time of concentration as below:

$$T_c = 0.0078L^{0.77}S^{-0.385}$$

where L is length of channel from headwater to outlet (ft) and S is average watershed slope.
For Kirpich equation, the time of concentration for the study area was calculated as a sum of each time of concentration at three sections of flows - sheet flow, shallow concentrated flow, and channel flow. Detailed inputs for each section are shown below:

Table 4: Kirpich Equation time of concentration calculation inputs

Sheet Flow		
Time of Concentration, Tc =	1.2	min
Length of Channel[15], L =	200	ft
Average Watershed Slope[16], S =	0.08	ft/ft
Shallow Concentrated Flow		

[15] FC PFM
[16] Google Earth

Time of Concentration, Tc =	9.7	min
Length of Channel[17], L =	1800	ft
Average Watershed Slope[18], S =	0.03	ft/ft
Channel Flow		
Time of Concentration, Tc =	27.2	min
Length of Channel[19], L =	5200	ft
Average Watershed Slope[20], S =	0.017	ft/ft
TOTAL Time of Concentration, Tc =	38	min

Precipitation intensity values for the 1-year storm for 30 min and 60 min time of concentration were found on the NOAA Atlas 14 Point Precipitation Frequency Estimates table, and then interpolated for 38 min.

Table 5: Peak runoff using Kirpich Equation

	1-year Storm	unit
Runoff coefficient, C =	0.42	
Precipitation intensity, i =	1.31	in/hr
Watershed area, A =	282.2	acres
Peak runoff rate, q_p =	155	cfs

Soil Conservation Service Equation (1972) calculates the time of concentration as below:

$$T_c = L^{1.15}/(7700\ H^{0.38})$$

where L is length of longest flow path (ft) and H is difference in elevation between outlet and most distant ridge.

Table 6: Soil Conservation Service Equation f time of concentration calculation inputs

SCS Equation		
Length of longest flow path, L =	7200	ft
Difference in elevation, H =	147	ft
Time of Concentration, Tc =	32	min

17 FC PFM
18 Google Earth
19 FC PFM
20 Google Earth

The table below summarizes results from all equations. Last row of table uses the time of concentration from NRCS TR-55.

Table 7: Summary of results for rational method using various T_c calculation methods

	Tc (min)	Rainfall Intensity (in/hr)	1-year Storm q_P (cfs)
Kirpich	38	1.3	155
Soil Conservation Service	32	1.4	167
NRCS TR-55	37	1.3	157
		Average	160

Analysis:
In this section, peak runoff rate was evaluated using 2 different equations. Time of concentration from each equation ranges from 32 to 38 min. As a consequence, peak runoff rate ranges from 155 cfs to 167 cfs. Therefore, it can be concluded that the rational method estimates that 1-yr storm in the study area will provide 160 cfs of flow in the channel.

4. Anderson Method

Background:
This method follows an approach reported by Daniel G. Anderson, created in 1970 and based in Fairfax County, Virginia. Based on work done by Carter (1961), the Anderson method defines an imperviousness constant (K), such that Qu = KQz, where Qu is the peak flow after development and Qz is peak discharge for a completely sewered basin without impervious areas. For average storms in the D.C. area, Carter determined that 30% of rain on natural surfaces becomes direct runoff, and 75% of rain on impervious surfaces becomes direct runoff. Therefore, K = 1 + 0.015*I, where I is equal to the percentage of the basin area covered with impervious surface.

For a natural watershed, Q is a function of basin area (A) and lag time (T). Variables used in calculating runoff include:
Impervious Index (I): Percent of basin area covered with manmade impervious surfaces.
Coefficient of Imperviousness (K): Variable based on empirically derived ratio of runoff from natural and impervious surfaces, and percent impervious surfaces in an area.
Flood Frequency Ratio (R):
Length (L): Distance, in miles, along primary water course from basin mouth to basin boundary
Slope (S): Average slope, in feet per mile, of main watercourse between points 10 and 85 percent of the length L, upstream from the mouth
Length-Slope (L/√S): Ratio of basin length to square root of basin slope

Drainage class (plate 6-6): Arbitrary designation based on field inspection of drainage channels, percentage imperviousness, and influence of storm sewers, used to determine constants in Lag Time equation.
Lag Time (T): Function of Slope and Length. T = f(log L, log S).
Area Exponent (x): Constant based on overall area. X is 1 for areas greater than 200 acres but less than 1 square mile, and .82 for areas greater than 1 square mile.

Limitations:
When the Anderson method was created in 1970, Fairfax County was developing steadily and had around 450,00 residents. Today, Fairfax is home to over 1.1 million residents and is greatly more developed. The changes in land-use, as well as climate, could have dramatic effects on runoff values. Other sources of error include the use of empirically derived graphs from the 1970s, which are fitted only to as low as the 2 year storm, leaving important metrics for the 1 year storm undetermined. A further source of error may lie in assumptions made about the drainage class; we chose a class mixing sewered and natural channel, and calibrations for such channels 48 years ago may have been drastically different than today.

Calculations:
From the Fairfax County Public Facilities Manual, the Anderson equation is used in the following form:

$$Q = 230 * K * R * A^{(x)} * T^{(-0.48)}$$

$K = 1.00 + 0.015 * (I)$

x = 1 for areas greater than 200 acres but less than 1 mi²,
& x = 0.82 for areas greater than 1 mi2.
Lag Time (T) = Y (L/S1/2)(z)
R is found on plate 6-6.
Y and Z exponents are found on plate 7-6

To use Plate 7-6, the Fairfax PFM recommends the following procedure: “The top line shall be used for natural drainage basins, basins with fewer or no storm sewers. The middle line shall be used for developed drainage basins, basins where the tributaries are sewered and the main channels are natural and/or rough lined (rubble or grass). The bottom line shall be used for completely sewered and developed basins having smooth lined (concrete, brick or metal) main channels”Given that our watershed includes natural drainage areas within the golf course, as well as sewered sections of residential communities, we chose for our to use the line “Tributaries Sewered, Main Channels Natural”. Therefore, Y = 0.9 and z = 0.5, which gives us a T value of .32 hours.

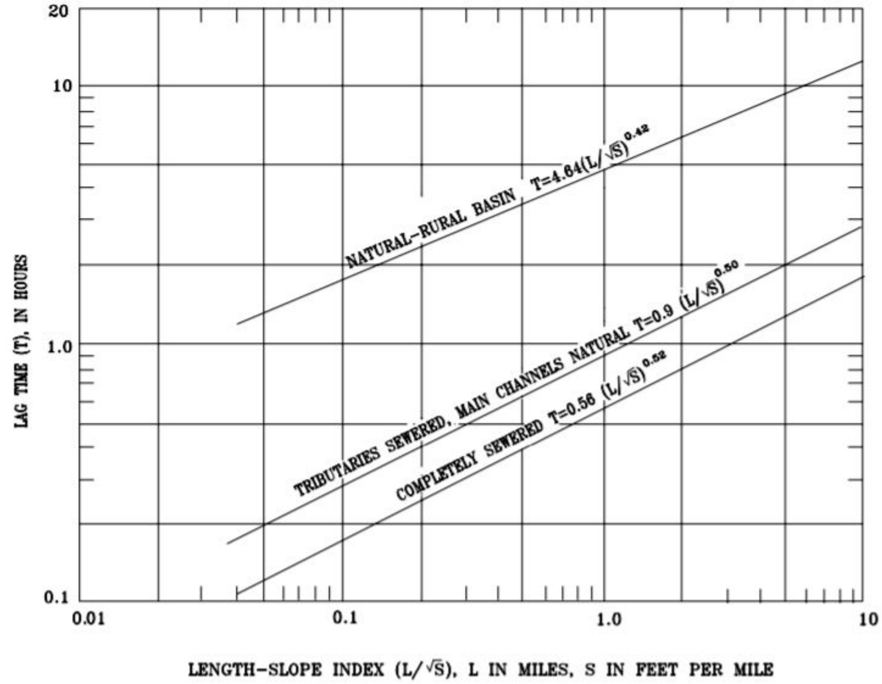


Figure 10: Plate 7-6 (Fairfax PFM Plates)

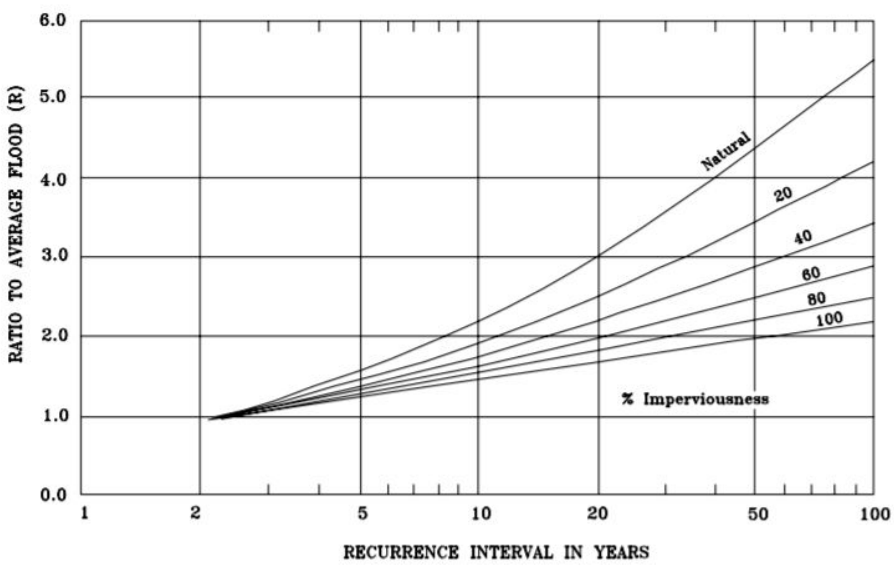


Figure 11: Plate 6-6 (Fairfax PFM Plates)

Given our percent impervious cover of approximately 26%, the above fit line seems to indicate a convergence on an R value of 1 at recurrence intervals of 2 or fewer. It is not entirely possible to accurately interpolate this line given the lack of present data points, so a different method was taken to determine the 1 year design flow. Similar to the Loudon Method of extrapolating 1 year flow from multiple return periods and associated flows, the Anderson Method was used to calculate flows across Return periods ranging from 2.3 to 100 years, and a logarithmic curve was fit.

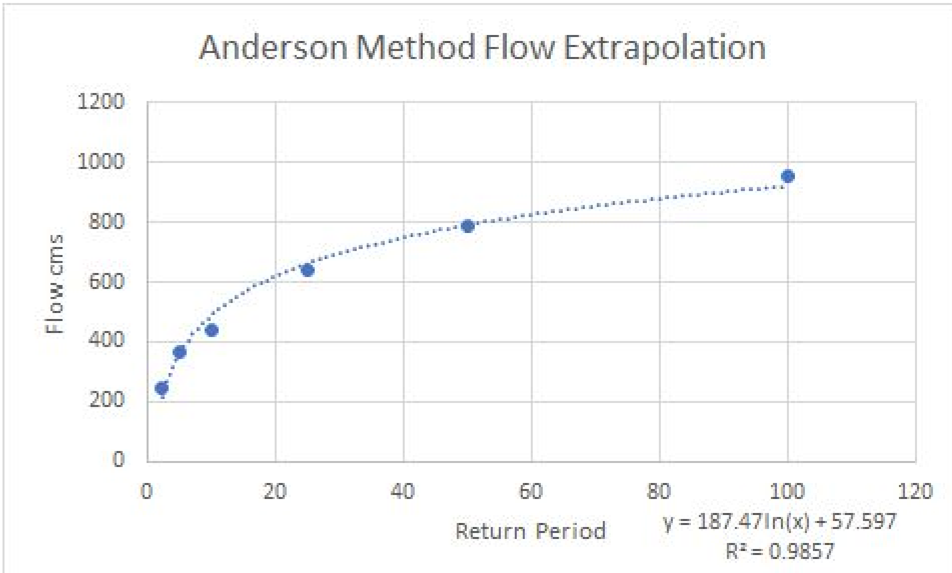


Figure 12: Anderson Method Flow Extrapolation

From this curve, a Q of 58 cubic feet per second (cfs) was found for the 1 year storm. This value of Q is lower than most values calculated, although it is equal to the value calculated by WSSI for the 1 year storm.

Analysis:
The Anderson method produced results that are close to what we would expected for this basin, but due to its speculative extrapolation and potential outdatedness, this method should not be considered for this basins design.

5. Snyder Method

Background:
The Snyder Method was developed by Franklin F. Snyder and published in a paper titled Synthetic Flood Frequency in 1958. The approach is modeled off the rational method. It utilizes the time of concentration,unit hydrograph, and considerations for storage. The following inputs were used in the calculations outlined by this method. The approach was applied to many watersheds in the Washington DC area.

Limitations:
The primary limitation for this method is scaling. The method was designed and intended for use on basins substantially larger than this project. Due to this the results from this calculation

are much less likely to be accurate. Additionally, the method is intentionally conservative. Due to the project location and cost restraints, a less conservative approach is desired.

Calculations:

Table 8: Snyder method inputs

Symbol	Description	Value	Units	Source
L	length of principal channel	0.98	mile	GIS Data
n	Manning's friction factor	0.0352		Virginia Erosion Control Manual
S	weighted slope of principal channel	1.71	%	GIS Data
SS%	percent of basin with storm sewer	68	%	Google Maps Area Tool
BR%	percent of natural drainage channels removed	14.29	%	
R	amount of rainfall for design storm	2.62	in	PFM
Cr	runoff coefficient	0.42		PFM and Google Maps Area Tool
A	area of watershed	0.44	mi^2	GIS Data

The following calculations were done, where L' is the length of an equivalent channel having the same time of concentration but with a standard slope of 1% and a standard friction factor of .1, Ct is coefficient dependent on the watershed drainage system, Tc is the time of concentration, Ir resulting runoff, and Qp is the flow rate.

Table 9: Snyder method calculations

$L' = \frac{10 * L * n}{\sqrt{S}}$	0.26	mile
$C_t = 1.7 - (1.7 - .42) * (\frac{SS\% + BR\%}{2})$	1.17	
$T_c = C_t * (L')^{.6}$	0.53	hr
$I_R = R * C_r / T_C$	2.08	in/hr
$Q_p = 500 * A * I_R$	457.73	cfs

Analysis:

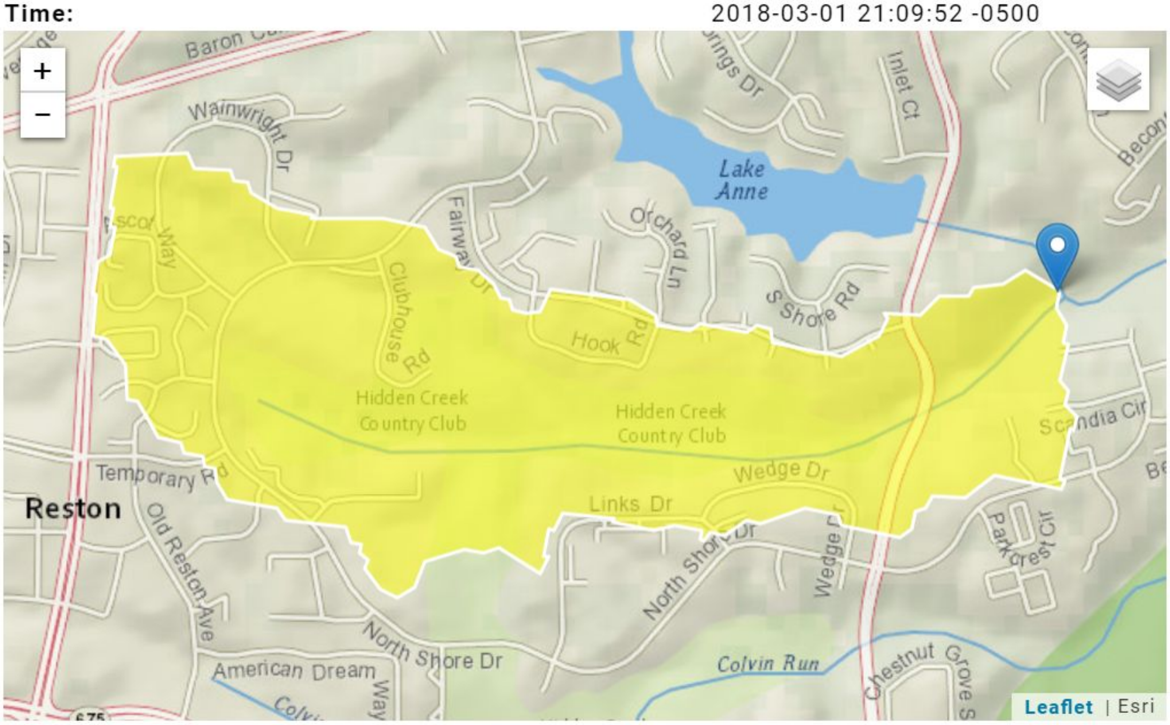
The flow rate estimated using this method is much higher than would be expected for this basin. This is due to the methods limitations that have been discussed. The snyder method should not be considered for this basins design.

6. StreamStats

An additional method of determining peak flow in a basin is to use a USGS online tool called StreamStats. StreamStats which is an integrated GIS application that is based on a combination of ArcGIS Server technology and ArcHydro Tools, as well as on Python scripts. The only input needed is a "pour point" placed onto their ESRI GUI map, and a selection of what data is to be downloaded. Below is the StreamStats Report generated using the pour point of the end of the Wiehle South reach.

StreamStats Report

Region ID:	MD
Workspace ID:	MD20180302020934859000
Clicked Point (Latitude, Longitude):	38.96335, -77.33006
Time:	2018-03-01 21:09:52 -0500



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	0.42	square miles
LIME	Percentage of area of limestone geology	0	percent
FOREST_MD	Percent forest from Maryland 2010 land-use data	3.53	percent
BSLDEM10ff	Mean basin slope computed from 10 m DEM in feet per foot	0.0713	foot per foot

Figure 13: StreamStats Report: Wiehle South

Although the watershed area offered by StreamStats differs from the design watershed by .02 square miles, it is similar enough to generally corroborate the design watershed created in ArcGIS, as well as the watershed map found on the WSSI construction planset. The report is based on a Maryland/D.C. combined Digital Elevation Model (DEM), as the Virginia DEM was, for an unknown reason, unable to calculate the watershed statistics needed.
Along with the watershed characteristics, StreamStats also generates a dataset of peak flows per return period, found in the following table and analyzed in the following graph.

Table 10: StreamStats Peak Flow Statistics

Return Period	Peak Flow	Unit
1.25	78	ft^3/s
1.50	98.7	ft^3/s
2.00	126	ft^3/s
5.00	225	ft^3/s
10.00	315	ft^3/s
25.00	469	ft^3/s

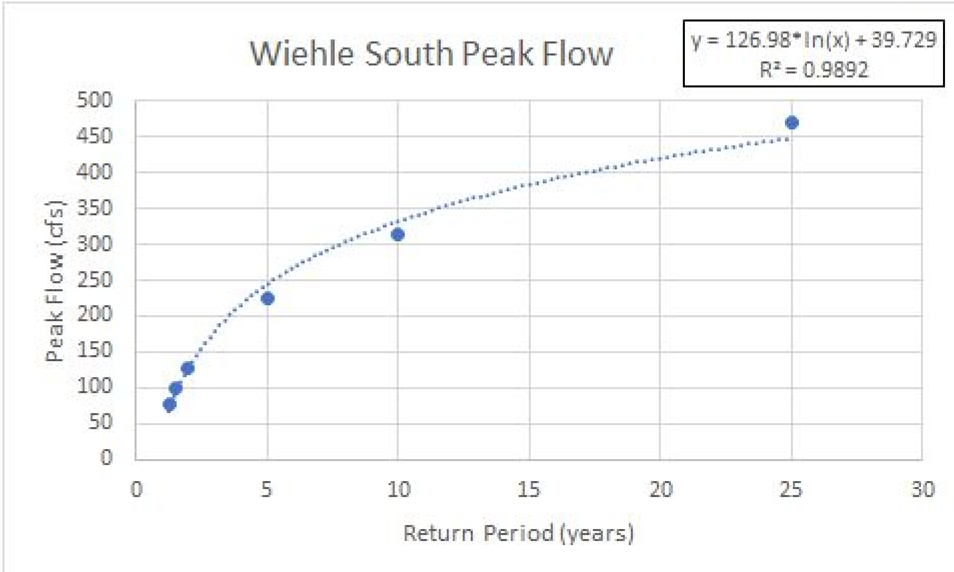


Figure 14: Streamstats Peak Flow Trendline

The data pulled from StreamStats only includes information on return periods as low as 1.25 years. The dataset was used to fit a curve to extrapolate the one year storm. The curve seems well fit, with an R value of 0.989; this equation gives us a peak flow of 40 cfs for the one year storm.

Although this analysis from StreamStats is helpful as a sanity check on our other methods, there are many reasons why StreamStats is not a tool we want to heavily rely on for our stream design. First, StreamStats is a 'black box'; we do not know where exactly it is pulling its data from and how it is making all of its calculations. The watershed map and resulting watershed characteristics appear to be coarse and approximate, and the tool for determining a poor point has a low resolution and therefore high degree of uncertainty. Further, StreamStats was unable to produce an approximate value for the 1 year storm in the Wiehle South watershed. It is unfortunate that StreamStats did not offer values for a one year storm, as fitting a logarithmic curve to a one year storm can be very inaccurate, given that the natural log of one equals zero. StreamStats' peak flow value for the two year storm is within 12% of the value determined by WSSI in their initial report, so it certainly can be a viable test for calculated values. In the case of this report, the flow values determined via StreamStats will not be considered.

7. U.S. Geological Survey - Virginia Method

Methodology:

The USGS *Methods and Equations for Estimating Peak Streamflow Per Square Mile in Virginia's Urban Basins* was published in 2014 in cooperation with the Virginia Department of Transit. The models that were developed are statistical models and equations that predict flow response in Virginia's urban streams. The two parameters of basin drainage area and basin percent urban area were used to identify and predict peak urban streamflow, which aids in the understanding of water flows and the environmental health of urban basins and their associated ecosystems. The USGS used data from 115 sites in Virginia for which urban land cover constituted at least 10 percent of the upstream basin area to yield a series of empirical equations for storms of recurrence intervals of 1-year through 500-years.

Limitations:

Some limitations of this model include:

- Specificity to the urban Virginia watersheds
- Empirical basis requires specific unit inputs
- Urban Land Cover must be between 10.01 - 95.96%
- Basin Drainage Area between .07 - 2,404 mi^2

The Wiehle South area of concern though, is within the limitations of the model though it is on the smaller drainage area side of the regression model.

Calculations:

The 1-year design storm corresponds with the 0.995 annual exceedance probability. The relevant regression equation prediction expression is:

$$Log10(0.995\ AEP\ peak\ per\ square\ mile) =$$
$$1.673 + (URBAN - 43.179) \times ((Log10(DA) - 1.412) \times -0.00637) + 0.00372 \times URBAN + -0.512 \times Log10(DA)$$

Where

- DA, basin drainage area in square miles
- AEP, annual exceedance probability
- URBAN, basin urban area in percent

This yields a flow rate of **76 ft^3/s** for Wiehle South, given a drainage area of 0.441 mi^2 and 34% urban cover.

For a typical watershed, 34% urban cover would be considered low for 26% impervious cover. This observation prompted further analysis. Urban cover was calculated from land use data in ArcGIS (Figure 2). As displayed in Figure 2, the categories of land use in the classification are

a) Commercial

b) High density Residential
c) Industrial
d) Institutional
e) Low density residential
f) Medium density residential
g) Utilities
h) Agricultural
i) Open land
j) Recreation
k) Public

Categories 'a' through 'g' were considered as contributions to urban cover, while the others were not. It is worth noting that some of these categories were not present in the Wiehle South Watershed at all. While 34% urban cover for a watershed with 26% impervious cover might seem low, Wiehle South is an unusual watershed with a golf course making up a majority of the land use (recreation). Thus, it makes sense for the percent urban cover in this watershed to be low.

To test the hypothesis that the watershed does indeed have a low urban cover for the given impervious area, the total area of each land use type in the GIS dataset was compiled. The corresponding impervious cover percentage (%IA) for each 'urban' land use category was taken from NRCS TR-55, Table 2-2a. The following values were obtained.

Table 11: Land use and impervious area for Wiehle South

Land Use Type	Area (mi^2)	%Impervious Area	Impervious Area (mi^2)
Commercial	0.004858	85%	0.00413
High Density Residential	0.075596	65%	0.04913
Institutional	0.011198	85%	0.009518
Low Density Residential	0.032351	20%	0.00647
Medium Density Residential	0.022994	30%	0.006898
Open Land	0.09677	0%	0
Recreational (mainly golf course)	0.148232	0%	0

Total	0.39200	20% (Weighted Average)	0.076154

Note that the total area is 0.39 sq miles, which is 10% lower than the true watershed area of 0.44 sq miles. The %impervious area we get using the values in Table 11 is 20%, which is slightly lower than 26% (the value listed in the 2012 planset).

The difference between the back-calculated impervious cover (20%) and the impervious cover indicated in the planset (26%) is small enough that it can be explained by error in creation of the land use dataset. The 10% difference between watershed area and sum of land use areas is also small enough to be explained by uncertainty from the land use dataset and watershed delineation. The exercise in back-calculating the impervious area thus demonstrates that the watershed has unusual characteristics with regard to land use, resulting in %urban cover being lower than expected for a given % impervious cover. It is thus concluded that the 34% urban cover is a reasonable estimate and the calculations for this section of the report were not changed.

8. Regional Curve Method and WSSI Model

The 2012 Wiehle South plan set contained regional curves based on Maryland Piedmont rural reference data, allowing flow rate to be expressed as a function of watershed size for a given region (Maryland), and level of development (rural). For Wiehle South, the expected 1-year storm flow rate was 43 cfs. This is likely to be an underestimate, since the Wiehle South watershed has 26.6% impervious cover, suggesting higher runoff rates than a fully rural watershed, the Maryland rural reference data being for a 7.8% impervious watershed.

The plan set also contained data on the relationship between runoff in urban and rural regions based on watershed size. This was developed using studies from Maryland, Texas and Vermont. Based on this relationship, an enlargement factor was derived to convert rural flow estimates to urban flow estimates. The enlargement factor used for the Wiehle South watershed was approximately 2.5. Applying the enlargement factor to the Wiehle South watershed gives an estimated flow rate of 113 cfs. This is likely to be an overestimate, since the watershed contains a golf course and large areas of open land rather than full urban cover.

There is some uncertainty in the use of the enlargement factor, since it was obtained from data collected in regions with varying topography that did not produce a perfect curve fit. The best estimate of the 1-year storm peak flow rate is likely in between the values obtained for the regional curves with and without an enlargement factor. Thus, both values were included in the final analysis.

Wetland Studies and Solutions, Inc. (WSSI) produced a curve for the 1-year storm peak flow rate using its own HEC-HMS model. The WSSI 1-year estimate was also applied to the Wiehle South watershed, resulting in a estimated discharge of 58 cfs. This value was included in the final analysis.

IV. Conclusion

Table 12: Summary of results for the 1-year storm peak discharge, for the six methods of calculation used

Method	Flow Rate (ft^3/s)
Loudoun County	24 (1-year extrapolation) 173 (2-year direct)
TR-55	100
Rational	160
Anderson	58
Snyder	457
USGS Virginia	76
Regional Curve without enlargement	43
Regional Curve with enlargement	113
WSSI 1-yr Model	58
USGS StreamStats	40

As is evident in Table 11, the ten methods yielded a wide range of results. Many of the methods could be discounted due to their lack of applicability to the Wiehle South watershed and the 1-year storms. Eventually, only five methods were considered in the final stage of analysis. The Loudon County method, Anderson method, Snyder method, Rational method, and USGS streamstats methods were not included in the determination of the final discharge value.

Since the Loudon method directly calculates the 2-yr storm as its most frequent storm, this estimate of the 1-year storm is merely an extrapolation. Rather than using the 1-year storm extrapolation, it is more useful to think of the Loudon method result for the 2-year storm (173 cfs) as a generous upper bound for the 1-year storm value. Additionally, the Loudon method is based on a regression analysis with poor goodness of fit shown for many gaging stations. Thus, the 1-year storm flow rate from the Loudon method was rejected in the final analysis.

Various issue arose with using the Snyder and Anderson methods. Both were empirically derived and tested in the Washington, D.C. area between 50-60 years ago, meaning they might be outdated for current application. Both methods seemed extremely conservative for small watersheds, likely because they were developed for larger basins. The Snyder method in particular seems to have been designed for larger watersheds, since the Wiehle South watershed is significantly smaller than the smallest basin considered in their analysis. The Anderson method uses a curve that doesn't include storms smaller than the 2.33-yr storm, making it inappropriate for calculations concerning a 1-yr storm.

The Fairfax Public Facilities Manual lists recommended methods in the table below. As shown, the Anderson method is considered acceptable above 200 acres, and our watershed area is quite close to this limit. The Rational Method is not encouraged for watersheds larger than 200 acres, thus it was discounted in the final analysis, along with the Snyder and Anderson results.

6-0801 Acceptable Hydrologies (27-89-PFM, 116-14-PFM)

Table 6.4 Acceptable Hydrologies – Applications

Name of Hydrology	200 Acres and Under	Over 200 Acres	Retention/Detention Facilities
NRCS*	X	X	X
Rational Formula	X	O	X***
Anderson Formula	O	X	O
Other**	X	X	X

X = Acceptable hydrology
O = Unacceptable hydrology
* = Recommended hydrology
** = With approval of the Director
*** = Watersheds less than 20 acres only, provided that the "C" factor for the unimproved areas does not exceed 0.15 on storm frequencies of 2 years or less.

Figure 15: Recommended Methods for Different Watershed Characteristics from Fairfax PFM

In a manner similar to the Loudon County method, the USGS StreamStats tool provided flow estimates for storms larger than the desired 1-year storm, meaning that the 1-year storm could only be extrapolated from a curve. Moreover, StreamStats is a 'black box'; the details of where exactly it is pulling its data from and how it is making all of its calculations are not known. The watershed map and resulting watershed characteristics appear to be coarse and approximate, and the tool for determining a poor point has a low resolution and therefore high degree of uncertainty. This method was thus rejected from the final analysis as well.

The methods used in the final analysis were the TR-55 method, USGS-Virginia method, the WSSI 1-yr storm estimate (obtained using HEC-HMS) and regional curves for rural Maryland with and without an enlargement factor.

The WSSI discharge model was thus included in the analysis due to its prominence in the 2012 Wiehle South plan set. Its results were noted as being consistent with the HEC-HMS model. Additionally, the regional curve for rural Maryland was used, both with and without an enlargement factor for application to urban watersheds. Regional curves are also widely used in a professional context and helped inform decision made in the original 2012 plan for Wiehle South.

Since Wiehle South is characterized by about 33% urban cover, it is neither fully rural nor fully urban. It has an intermediate level of impervious cover, implying that the runoff rate in this basin is higher than that in rural areas while being lower than that in urban areas. Thus, it makes sense to be more conservative than the regional curve, and less conservative than the unenlarged regional curve. Thus, values from both curves were included in the final analysis. The two values were also treated as upper and lower bound estimates while evaluating validity of different methods.

The TR-55 method has a low risk of being over-fit to a specific set of watersheds, however this also means it requires many physical parameters that are best determined from on-site observations. For this assignment, physical surveys of the watershed couldn't be conducted, and the parameters used for nearby watersheds were utilized in calculations. This approach has been successful for past projects in the surrounding region, which improves confidence in the results. Moreover, the time of concentration calculated using the TR-55 method was very close to the value obtained using the Kirpich equation, further validating this approach.

With the USGS method, the Wiehle South watershed is close to the lower limit of recommended watershed size. Since it was still above the lower threshold, and since calculations yielded results similar to the WSSI HEC-HMS estimate, this method was included in the final analysis.

The results from the five selected methods were averaged to obtain a peak 1-year storm flow rate of 80 cfs. This is lower than the value used in the original 2012 construction plan (113 cfs), which suggests that a reduction in cost is possible. The use of a lower flow rate should not be a cause for concern since the area immediately adjacent to the stream consists mostly of trails rather than residential buildings. Since flooding of these trails is undesirable but not physically harmful to people and property, there is less risk in opting for a lower design discharge.

V. Sources

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VI. Appendix

I. General Watershed Characteristics

Wiehle South Watershed Data			
Characteristic		Unit	Source
24 hr rainfall (Fairfax)	2.62	in	FC PFM
Area	282.2	acres	WSS Plan
Impervious	26.6	%	WSS Plan
Approx. runoff coefficient	0.42		Our estimate
Longest flow path length:	7200	ft	Google Earth
Max. diff. elevation	160	ft	Contour map
Avg watershed (differential) slope:	0.0975	ft/ft	Our GIS
Watershed Slope along longest path	0.022		Contour map
Sheet flow length:	200	ft	FC PFM
Sheet flow slope	0.08	ft/ft	Google Earth
Shallow concentrated flow length	1800	ft	Google Earth
Shallow concentrated slope	0.031	ft/ft	Google Earth
Cross sec flow area a			
Channel length (golf course)	4400	ft	Google Earth
Channel slope (gc)	0.017	ft/ft	Google Earth
Cross sec flow area a	2	ft^2	WSS Plan
Channel length (WS reach)	800	ft	WSS Plan
Channel slope (WS reach)	0.01625	ft/ft	WSS Plan
Total Channel Length	5200	ft	WSS Plan + Google Earth

Assignment #3: Wiehle South Riffle Design Analysis

Smart Stream Solutions Inc.

We have each personally reviewed and approve the following report:

Cameron Afzal		Date: 4/25/18
Cynthia Chan		Date: 4/25/18
Philip Duvall		Date: 4/25/2018
Ji Young Kim		Date: 4/25/2018
Tanvi Naidu		Date: 4/25/2018
Mike Zarecor		Date: 4/25/2018

Draft: March 21, 2018
Revision 1: April 13, 2018
Revision 2: April 18, 2018
Revision 3: April 23, 2018
Final : April 25, 2018

I. Executive Summary

Smart Stream Solutions (S^3) was hired to restore the Wiehle South stream reach in Reston, Virginia. In previous reports S^3 found that the primary cause for stream degradation was erosion due to increased runoff, which caused the channel to become entrenched and unstable. S^3 proposes that the stream cross section be changed to stably convey a flow rate of 80 cfs. S^3 proposes three different riffle designs: A, B, and C. Cross section C, which has a depth of 1.5 ft and a width of 16.5 ft, is ultimately recommended. This cross section design is smaller than previous designs which will reduce construction costs while still being stable.

While redesigning stream channels, it is important to keep in mind the effect of high intensity events on floodplain erosion. Since the new channel is designed to contain the peak discharge for a 1-yr storm, events of any higher intensity will result in the water level rising high enough to encroach into the floodplain. To ensure that high intensity events will not result in significant erosion in the floodplain, the permissible velocity for the most vulnerable soil type in the floodplain was obtained. This value was corrected for average flow depth and sinuosity. Velocity upper limits of 2.6 ft/s and 2.7 ft/s were calculated for the 100-yr storm and 500-yr storm respectively. Flow in the channel and floodplain during a 100-yr and 500-yr storm was modelled as compound open channel flow in HEC-RAS and the NRCS cross-sectional analyser.

It was found that for the 100-yr storm maximum velocity in the floodplain stayed below the 2.6 ft/s limit based on HEC_RAS. There was slight exceedance based on NRCS. For the 500-yr storm, the allowable velocity was exceeded with both analyses. The highest estimated floodplain velocity came from the NRCS spreadsheet analysis, at 3.3 ft/s. This is not deemed a cause for concern because the 2.6 and 2.7 ft/s limits assume bare soils, and the site will be vegetated at all times except during and immediately after construction. The 4 ft/s velocity limit for vegetated channels (from Fairfax Public Facilities Manual) is never exceeded for either storm according to the NRCS spreadsheet and HEC-RAS analysis. This indicates that a high-intensity storm will not result in significant floodplain erosion with the current channel design. Table 3 from Part IV of this document contains the results of both compound open channel flow models and is displayed here for clarity.

Table IV.3: Permissible velocity and the modelled maximum floodplain velocity using two different methods (HEC-RAS and NRCS Cross-sectional Analyzer) for 100-yr and 500-yr storms

Storm Frequency	Method	Permissible velocity (ft/s)	Maximum Velocity (ft/s)

100 year	HEC-RAS	2.6	2.2
	NRCS	2.6	2.7
500 year	HEC-RAS	2.7	2.9
	NRCS	2.7	3.3

Finally, it is necessary to design the channel bottom to withstand a desired level of shear stress. The 1.5 ft channel depth and 3:1 cross-sectional channel slope were the primary factors influencing in-channel shear stress**.** The S^3 calculations resulted in a shear stress factor of safety of 1.5 surpassing the 1.3 factor of safety for the WSSI design. The channel will need armor layer substrate with at least a 4.3 in D50 to withstand the erosion from a bankfull discharge event, making the A1 substrate from Cedar Mountain Quarry with a D_{50} of 6.7 in a thoroughly adequate choice for armor rock. The reinforced channel bed will function as a downscaled version of that specified in the WSSI plan.

II. Introduction

As noted in prior S^3 reports, the design storm impacts the design discharge. S^3 found that for the Wiehle South reach stream restoration design the channel should convey the 1-yr bankfull storm with a discharge of 80 cfs. When engineering stream restoration, it is critical to consider what processes have caused the degradation. Previous reports detailed how the urbanization of the Reston watersheds increased the flow rate the Wiehle South stream must convey. This increased flow has caused significant channel erosion.

Design discharge dictates channel morphology, impacting flow velocity and boundary shear stress. A deeper channel or higher flow rate result in more potential for erosion and thus need for bed stabilization, chiefly through the armor layer in the channel bed. S^3 will design a riffle to contain and withstand the effects of a 1-yr bankfull discharge event buffered by a floodplain which can appropriately contain a 100-yr and 500-yr event with flow velocities within an allowed limit to prevent erosion.

III. Riffle Cross Section Design

A. Introduction

Streams typically are composed of varying sections called pools and riffles. Pools are generally deep with a low flow velocity, while riffles are shallow with a high flow velocity. It should be noted that one of the driving factors in a stream restoration project is the cross section of a stream. One of the means by which S^3 is cutting down on costs is through a reduction in stream cross section. This section of the report will detail the design of a riffle cross section representative of the channel form.

B. Calculations

The parameters to consider in the design of a stream cross section are bankfull depth to bankfull width ratio, side slopes, allowable velocity and shear. Table 1 documents the maximum side slope values for channels based on material. Since our channel material falls under the material classification of "Loose sandy earth, sandy loan or porous clay w/ vegetative lining", a side slope of 3 at a minimum is required to keep the channel stable.

Table 1: Minimum side slopes for channels excavated in various materials
(Virginia Erosion Soil Control Handbook, 1992)

Material	Side Slope
Rock	Nearly vertical
Earth w/ stone riprap lining	2 : 1
Firm clay or earth w/ vegetative lining	2 : 1
Loose sandy earth, sandy loan or porous clay w/ vegetative lining	3 : 1
Earth w/ concrete lining extending to top of channel banks	1.5 : 1

The NRCS XSec Analyzer Version 17 Excel spreadsheet is used to inform about riffle cross section. Inputs are channel slope, proposed cross section, and Manning's n values for various parts of the channel. A few of the relevant outputs are discharge, velocity, and shear in the channel. The spreadsheet uses Manning's Equation, shown below, which is an empirical formula for open channel flow. The uniform flow from Manning's Equation approximates real stream hydraulics. A Manning's n value of 0.035 is used to approximate in channel flow.

$$V = \frac{1.49 \cdot R^{2/3} \cdot s^{1/2}}{n}$$

Where:
V = velocity (ft/s)
R = hydraulic radius (ft)
s = channel slope (dimensionless)
n = Manning's n (dimensionless)

S^3 proposes three riffle cross sections for consideration. WSSI's original riffle cross section proposal is 20 ft wide with a side slope value of 3. Cross section A aims to cut down on costs and is made to be much narrower than the original riffle cross section. The side slopes are 2:1. The width to depth ratio of the riffle should be between 11 - 33 according to the "Northern Virginia Stream Restoration Bank Wiehle South", Sheet 39: Reference Reach Data, Prepared by Wetland and Stream Solutions Inc. 4/23/17, Last revised: 1/12, 69 sheets. However, the width to depth ratio of cross section A is 9.3 and does not meet this standard of structural acceptability. It also does not meet side slope acceptability according to the Virginia Soil Erosion Control Handbook. The width to depth ratio is an indicator of the energy distribution of a channel. According to Rosgen's Applied River Morphology, a higher width to depth ratio corresponds to greater velocity gradient. Velocity gradient is the rate of change of the velocity with respect to linear length along the bed of a channel cross section. It is interesting to note that velocity gradient and average velocity have opposite trends with respect to width to depth ratio. This velocity gradient, as well as the near bank stress determine the magnitude of erosion a channel will experience.

The importance of designing a riffle with a proper width to depth ratio and side slopes is to minimize the risk of erosion and minimize the later need for either armor rock or repair of the riffle since there is a 10-year guarantee on the design. Cross section B increases the side slopes to 2.5 and aims to cut down on the cross section size while maintaining a margin of safety. Cross section C is a 20% scaling down of the stream cross section WSSI proposed. This scaled down cross section has a side slope value of 3 and keeps the channel looking natural. Accordingly, S^3

chose option C for the Wiehle South Cross Section Design. Table 2 lists each channels properties in comparison to the original WSSI design. These channels are shown in Figure 1 as an overlay for visual comparison.

Table 2: Channel cross section properties.

	Original WSSI	A narrow	B moderate slope	**C scaled down**
Bankfull Discharge (cfs)	113	80	80	**80**
Area (ft^2)	22.9	14.2	15.4	**16.1**
Bankfull Width (ft)	20	11.5	15	**16.5**
Maximum Depth (ft)	1.7	2	1.5	**1.5**
Bankfull Depth (ft)	1.1	1.2	1.0	**1.0**
Width to depth ratio (11 to 33 allowable)	17.5	9.3	14.7	**16.8**
Side Slope (ft/ft)	3	2	2.5	**3.0**
Bankfull Flow Velocity (ft/s)	6.0	5.7	5.2	**5.1**
Bankfull Flow Shear (lbs/ft^2)	1.2	1.1	0.9	**0.9**

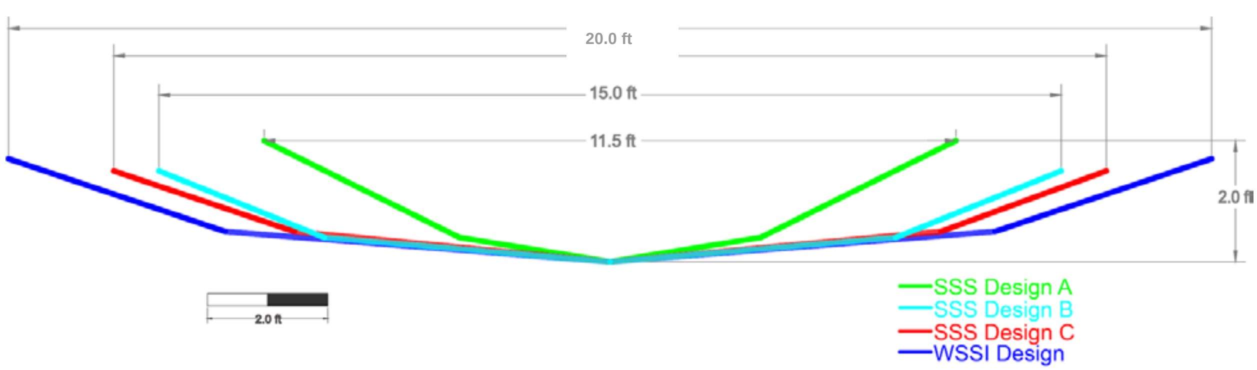


Figure 1: Proposed Channel Cross Sections

IV. Critical Flow Velocity and Shear Stress in Floodplain

A. Introduction

The Wiehle South stream reach is being redesigned to contain the flow from a 1-yr storm, meaning that for storms of greater intensity, flow will spill onto the floodplain. Since the floodplain is primarily covered in soil rather than rocky substrate, it is more vulnerable to erosion than the stream channel itself. It is important to calculate the shear experienced in the floodplain during a high intensity storm so that appropriate interventions can be made to attenuate floodplain erosion. The purpose of this section is to calculate the shear stress in the floodplain during a high intensity storm, determine if vulnerability to erosion is a concern, and if so, suggest potential solutions.

For this analysis, the 100-yr storm was chosen. Although a storm of such intensity occurs only once every 100 years on average, it can subject the floodplain to high levels of shear stress and bring about dramatic changes to the local bathymetry. This usually results in overly steep and therefore unstable banks. When banks are unstable, they are further vulnerable to erosion, setting off a positive feedback loop of bank destabilization and erosion. The potential consequences of such a situation include loss of viable floodplain habitat, loss of public land / private property, higher water turbidity, higher rates of pollutant transport, physical danger of collapse, and loss of tree cover. To avoid this wide range of undesirable consequences, it is best to design for a storm of very high intensity, which is why the 100-yr storm was chosen. Due to the increasing frequency of high intensity storms caused by climate change, analysis was also conducted for the 500-yr storm.

It was found that the Wiehle South watershed (and therefore the 100-yr and 500-yr floodplain) contains various soils of the texture silt loam and loam. Of these, silt loam is the most vulnerable to erosion. To prevent scour, this soil type requires a flow velocity below 3 ft/s. This upper limit was selected after looking up a range of sources that can be traced back to an ASCE publication from 1926 authored by Fortier and Scobey (Figure 4). Please refer to the subsection IV.B.iii 'Determining Maximum Permissible Velocity' for more details. The ASCE limiting velocities were selected based on the opinions of engineers at the Special Committee on Irrigation Hydraulics, without specific empirical evidence or theoretical basis. These velocities are currently used in many applications to determine allowable velocities in bare soils as newer peer reviewed data is not available. Research is ongoing at some research institutions to develop better tools for assessing allowable velocities to limit erosion, but currently it is our only indicator of erosion and will be used.

Fortier and Scobey use a more outdated classification system and do not list soil textures 'silt loam' and 'loam' as per the soil data and texture information from SSURGO and USDA. They instead list 'silt loam (non-colloidal)' and 'ordinary firm loam', which were assumed to be comparable to 'silt loam' and 'loam' respectively. This assumption was made because it was unfeasible to test soil samples and confirm whether or not they fit Fortier and Scobey's definition of 'colloidal'. Please refer to the subsection IV.B.iv 'Note on the Use of Limiting Velocities from the ASCE 1926 Publication' for more information on this choice. Thus, the limiting velocity for 'silt loam (non-colloidal)' from Figure 4, i.e. 3 ft/s, was taken to be the uncorrected maximum permissible velocity for the 100-yr and 500-yr discharge. When corrected for depth and sinuosity (see 'Determining Maximum Permissible Velocity' under 'Calculation'), the value of maximum permissible velocity becomes 2.6 ft/s for the 100-yr storm and 2.7 ft/s for the 500-yr storm. This is a conservative estimate since it assumes bare soils. The allowable velocity for vegetated channels is significantly higher, at 4 ft/s as per Fairfax PFM, 6-0000, Storm Drainage. Page 6-69. The floodplain is expected to be vegetated at all times except for during construction and the recovery period after construction.

The Fairfax County Public Facilities Manual Chapter 6, table 6-1006 recommends a Manning's n value of 0.035 in the channel. Plate 27-6 (27M-6) , which was deleted by 61-98 PFM, recommends a Manning's n of 0.1 for the floodplain. Assuming a Manning's roughness coefficient (n) of 0.1 in the floodplain, and 0.035 in the channel it was found based on HEC-RAS that this velocity limit would not be exceeded during a 100-yr storm. There was a slight exceedance based on NRCS. The results of both these analyses are listed in Table 3.

Table 3: Permissible velocity and the modelled maximum floodplain velocity using two different methods (HEC-RAS and NRCS Cross-sectional Analyzer) for 100-yr and 500-yr storms

Storm Frequency	Method	Permissible velocity (ft/s)	Maximum Velocity (ft/s)
100 year	HEC-RAS	2.6	2.2
	NRCS	2.6	2.7
500 year	HEC-RAS	2.7	2.9
	NRCS	2.7	3.3

For the 500-yr storm, the allowable velocity was exceeded. The highest estimated floodplain velocity came from the NRCS spreadsheet analysis, at 3.3 ft/s. This is not deemed a cause for concern because the 2.6 and 2.7 ft/s limits assume bare soils, and the site will be vegetated at all times except during and immediately after construction. The 4 ft/s velocity limit for vegetated channels is never exceeded for the 500-yr storm according to the NRCS spreadsheet and HEC-RAS analysis.

Thus, the permissible velocity limits are not expected to be exceeded with this channel design for the 100-yr and 500-yr storms in the Wiehle South watershed. This is further supported by observations by WSSI over the years, since the recent 500-yr and 100-yr events in the area have not caused noticeable erosion with permissible design velocities around 2.5 ft/s.

B. Calculations

1.) Determining Peak Discharge for 100-year and 500-year storms:

In Assignment 2, the team used a wide range of methods to determine the peak discharge associated with the 1-yr storm. Some of these methods, deemed applicable for a high intensity storm in a watershed of this size, were used in determining the peak discharge for a 100-yr storm. The methods used were the Loudoun County method, NRCS TR-55, USGS, and the WSSI estimate made using HEC-RAS. The values from these methods were averaged to obtain a final value. As an extra check, the ArcGIS streamstats method was also looked at for additional verification.

Table 4: Calculations for the 100-yr storm peak flow rate

Method	100-yr, 24-hr storm flow rate (cfs)
Loudoun	1083
TR-55	700
USGS	1047
WSSI estimate	968
Average	938
Selected	**968**

The average obtained was 938 cfs. Since the WSSI estimate of 968 cfs was based on the most complex model and was higher than this average, the higher 968 cfs value was chosen to be conservative. As an order of magnitude check, the ArcGIS streamstats method was also used, which gave a reasonably close value of 805 cfs. The 100-yr floodplain was determined to be the height that could accommodate a flow rate of 968 cfs, using the FEMA 100-yr floodplain as a starting point.

Due to the increasing frequency of higher intensity events, the design team believes it is useful to consider the effect of events more intense than the 100-yr storm. Thus, the same calculations were conducted for the 500-yr storm. Since the WSSI estimate for this storm intensity was not available, the Loudoun and USGS methods were used. The TR-55 method was not used because the HEC-RAS and NRCS spreadsheet provided different cross-sections for the 500-yr floodplain, bringing further uncertainty to the TR-55 method.

Table 5: Calculations for the 500-yr storm peak flow rate

Method	**500-yr, 24-hr storm flow rate (cfs)**
Loudoun	1821
USGS	1895.5
Average	**1858.25**

A similar analysis for the maximum floodplain velocity during 500-yr storm was conducted using both the NRCS spreadsheet and HEC-RAS, as detailed in the following subsections IV.3 and IV.4 'NRCS model' and 'HEC-RAS model'.

2.) Floodplain Delineation:

The 2012 planset for Wiehle South contains markings for the FEMA 100-yr floodplain. These markings are an approximation of the true floodplain since they do not coincide well with the true contour lines. The FEMA 100-yr floodplain was used as a starting point in determining a more accurate extent for the 100-yr floodplain. The 500-yr storm floodplain was estimated through trial and error using the NRCS spreadsheet method detailed below, with the 100-yr floodplain as a starting point.

3.) Determining Maximum Permissible Velocity:

To acquire the appropriate permissible velocity value, soil types on the study area were analyzed. Soil data for the Wiehle South watershed was obtained from the SSURGO database for Fairfax County, VA. The shapefile for the Wiehle South watershed, delineated in arcGIS, was uploaded as the 'area of interest' into the Web Soil Survey. This generated a map of the soil types in the watershed, along with descriptions and percent cover for each type. The soil textures were found from each soil series name using the USDA NRCS Website for Official Soil

Series Descriptions and Series Classification. The soil map and soil table below display this data (Figure 2, Table 6).

Figure 2: Soil types for Wiehle South watershed from the SSURGO database

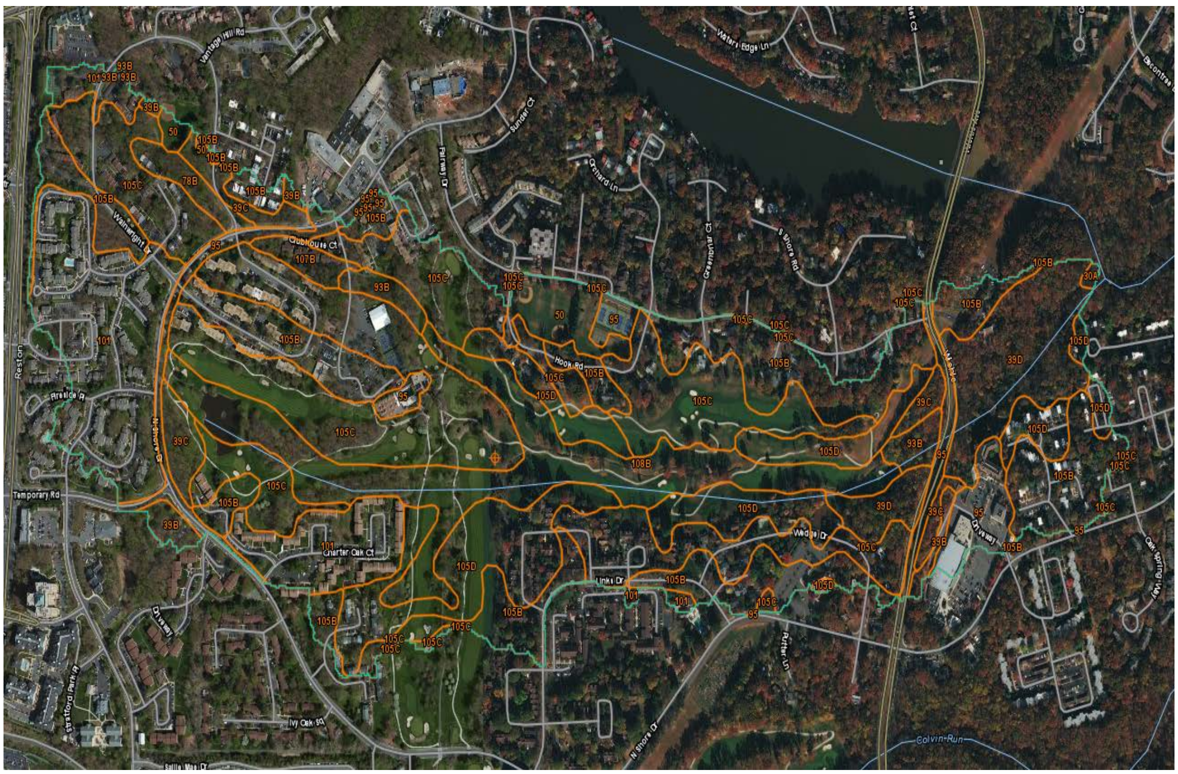


Table 6: Soil types for Wiehle South Watershed for SSURGO database

Fairfax County, Virginia (VA059)				
Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI	Top Horizon Texture
30A	Codorus and Hatboro soils, 0 to 2 percent slopes, occasionally flooded	0.2	0.10%	Silt loam, Loam

39B	Glenelg silt loam, 2 to 7 percent slopes	2.5	0.90%	Silt loam
39C	Glenelg silt loam, 7 to 15 percent slopes	6.5	2.40%	Silt loam
39D	Glenelg silt loam, 15 to 25 percent slopes	16.3	6.10%	Silt loam
50	Hattontown silt loam, 0 to 25 percent slopes	5.5	2.00%	Silt loam
78B	Meadowville loam, 2 to 7 percent slopes	2.4	0.90%	Loam
93B	Sumerduck loam, 2 to 7 percent slopes	3.3	1.20%	Loam
95	Urban land	13	4.90%	N/A
101	Urban land-Wheaton complex	33.7	12.70%	Silt loam
105B	Wheaton-Glenelg complex, 2 to 7 percent slopes	50.4	18.90%	Silt loam
105C	Wheaton-Glenelg complex, 7 to 15 percent slopes	71.9	27.00%	Silt loam
105D	Wheaton-Glenelg complex, 15 to 25 percent slopes	30.7	11.50%	Silt loam
107B	Wheaton-Meadowville complex, 2 to 7 percent slopes	1.9	0.70%	Silt loam, Loam
108B	Wheaton-Sumerduck complex, 2 to 7 percent slopes	28	10.50%	Loam
Totals for Area of Interest		**266.3**	**100.00%**	

The highest permissible velocity to avoid erosion for various soil types is also provided in the Virginia Erosion and Sediment Control Handbook in Table 5-22, Permissible Velocity for Unlined Earthen Channels (Figure 3). This table is based on a 1926 publication by ASCE, and therefore uses the outdated Fortier and Scobey soil classification rather than the soil textures currently defined in the NRCS Official Soil Classification. The soil types obtained from the SSURGO database thus had to be matched to the soil types listed in the ACSE table (Figure 3). The uppermost horizons of the soil types present in the watershed fall under the soil texture categories of 'silt loam' and 'loam'. These were matched to 'silt loam (non-colloidal)' and 'ordinary firm loam' in the ASCE table respectively, since soil samples could not be tested to confirm whether to not they fit Fortier and Scobey's definition of 'colloidal'. Thus, of the soil types present in the floodplain, the lowest scour velocity is 3.0 ft/s for silt loam. This was taken to be the uncorrected maximum permissible velocity in the floodplain for a 100-yr storm, below which no significant erosion is expected. For more details on the source of the limiting velocities, please refer to the following sub-section, IV.B.4 'Note on the Use of Limiting Velocities from the ASCE 1926 Publication'.

To improve accuracy in permissible velocity estimates, it is useful to include a correction factor based on average depth and sinuosity of the channel. The recommended correction for depth are displayed in Figure 5. The average depth in the 500-yr floodplain, excluding the channel, calculated as the cross-sectional area divided by the channel top width, is around 2.4. This was calculated using the NRCS spreadsheet, considering only the stations located in the floodplain and excluding those located in the channel. For the 100-yr floodplain, this value was 1.8.

For the 500-yr floodplain, the hydraulic depth corresponds to a depth-correction factor of 0.95, which gives a corrected permissible velocity of 2.85 ft/s. For the 100-yr floodplain, the correction factor is 0.9, which gives a depth-corrected velocity of 2.7 ft/s. The correction for sinuosity is displayed in Figure 5. The sinuosity for Wiehle South channel, based on the 2012 planset, is 1.1, which corresponds to a 5% reduction in permissible velocity. The floodplain, however, has a lower sinuosity than the channel and can thus be assumed to have a sinuosity of 1. This corresponds to the same 5% correction factor. When applied to the depth-corrected permissible velocity, this gives a value of 2.7 ft/s for the 500-yr floodplain, and 2.6 ft/s for the 100-yr floodplain. These were taken to be the final permissible velocity values.

The use of the 2.7 and 2.6 ft/s limits are also supported by observations made by WSSI in restored streams in the Reston area over the years. For design velocities in the range of 2.5 ft/s and below, no noticeable erosion was observed for one 500-yr event and three 100-yr events. It is worth noting that 2.7 and 2.6 ft/s are conservative estimates since they assume bare soils. The allowable velocity for vegetated channels is significantly higher, at 4 ft/s as per Fairfax PFM, 6-

0000, Storm Drainage. Page 6-69. The floodplain is expected to be vegetated at all times except for during construction and the recovery period after construction.

4.) Note on the Use of Limiting Velocities from the ASCE 1926 Publication:

The limiting velocities from the ASCE 1926 paper were determined by asking 7 professional engineers their opinions on the velocities w/out erosion for different soil types. There is thus a lack of rigorous evidence to support these values. While research is ongoing at some institutions to develop better tools to assess allowable velocities, these values are the most widely accepted estimates currently available and were therefore used to inform this analysis.

It was deemed appropriate to match 'silt loam' from the USDA Official Soil Classification (textures) to 'silt loam (non-colloidal)' from the ASCE paper. The ASCE paper categorizes soil as 'colloidal' based on whether it contains colloidally sized particles and whether the particles are dispersed or flocculated. This information cannot be gleaned purely from the SSURGO data since the texture 'silt loam' can contain a wide range of colloidally sized clay particles. Without testing soil samples from the area it is impossible to say whether the uppermost horizons of series present in the floodplain are 'colloidal' or 'non-colloidal'. The category 'silt loam (non-colloidal)' was selected because it was the closest match to 'silt loam' given the available information. This is not expected to be a problem since the estimate is already conservative due to its assumption of bare soils in a vegetated floodplain.

The team noted that many sources list limiting velocities that can be traced back to the ASCE paper. The following sources all use the same set of limiting velocities:

- Virginia Erosion and Sediment Control Handbook, Table 5-22
- 1995 PFM, page 6-50, 6-1012, Table 6.19 (Water carrying fine silts (colloidal)
- VDOT Drainage Manual, 8/98, Table 2.8.1 (Water carrying fine silts (colloidal))
- VDOT Drainage Manual, 4/02, Appendix 7D-6 (Water carrying fine silts (colloidal))

All of the above sources list 3 ft/s as the allowable velocity for silt loam (noncolloidal). Appendix 7D-2 in the VDOT Drainage Manual 4/02, however, lists a much lower permissible velocity for this soil type: 2.3 ft/s. The team believes there is reason to doubt the validity of the estimate in Appendix 7D-2. The table in this appendix also provides a loose relationship between the AASHTO (American Association of Highway and Transport Officials) classification and the Fortier and Scobey names used in the ASCE publication, suggesting that 'silt loam (noncolloidal)' should be categorised as AASHTO A-3 soil. According to the

AASHTO classification, A-3 soils are defined as 'fine sand', while the team believes that A-4: 'silty soils' is a more appropriate categorization for silt loam. Moreover, the 1995 PFM table cited above lists 'silt loam (noncolloidal)' as non-plastic A-4 soils, supporting this conclusion. Thus, VDOT Drainage Manual 4/02, Appendix 7D-2, was not considered in the analysis.

The team would like to emphasise that the ASCE limiting velocities were selected based on the opinions of engineers at the Special Committee on Irrigation Hydraulics, without specific empirical evidence or theoretical basis. These velocities are currently used in many applications to determine allowable velocities in bare soils as newer peer reviewed data is not available. Research is ongoing at some research institutions to develop better tools for assessing allowable velocities to limit erosion, but currently the most widely accepted indicator of erosion and will be used.

5.) Determining Maximum Velocity in the Floodplain During 100-yr and 500-yr storms:

The Manning's roughness coefficient (n) in the floodplain was set as 0.1 as described in the introduction. The slope of the floodplain was assumed to be comparable to that of the steepest riffle, or 1.5%. A cross section of the stream and floodplain at station 13.5 was selected from the contour map provided in the 2012 Wiehle South plan set. The contours were used to construct a representative cross-section of the stream extending to the FEMA 100-yr floodplain. The floodplain was first assumed to extend to the location of the FEMA 100-yr floodplain line, and then extended in increments of 0.5 ft height until the 968 cfs capacity was either met or exceeded. This was done using both the NRCS spreadsheet, modified to accommodate compound open channel flow, and in HEC-RAS. The 100-yr floodplain was used as a starting point to delineate the 500-yr floodplain, raising the elevation in increments of 0.5 ft until the discharge for the 500-yr storm was either met or just exceeded.

Once the floodplain height was ascertained, the NRCS and HEC-RAS models were run to determine the velocity in the floodplain associated with the 100-yr storm flow rate. The details of this analysis can be found in the following subsections 'NRCS model' and 'HEC-RAS model'. It was found that when using HEC-RAS, the maximum velocity in the floodplain stayed below or equal to 2.7 ft/s, meeting the limit set based on the most vulnerable soil type in the area. There was minor exceedance based on the NRCS method, which the team considers more simplistic than HEC-RAS. Thus we do not expect there to be noticeable erosion in the Wiehle South floodplain due to the 100-yr storm. The maximum observed velocity is also similar to the limit used in other stream restoration projects in the Reston area, increasing confidence in the validity of this conclusion.

TABLE 5-22

PERMISSIBLE VELOCITIES FOR UNLINED EARTHEN CHANNELS

Soil Types	Permissible Velocity (ft./sec.)
Fine Sand (noncolloidal)	2.5
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	3.0
Ordinary Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silt to Cobbles (noncolloidal)	5.5
Alluvial Silts (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5
Shales and Hard Pans	6.0

Source: American Society of Civil Engineers

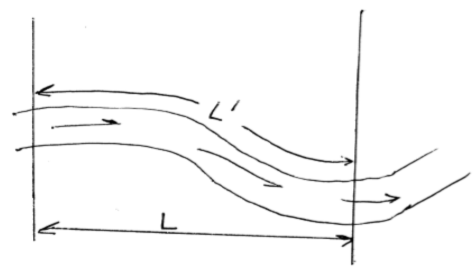
Figure 3: Permissible Velocity for each soil type

TABLE 5-23

REDUCTION IN PERMISSIBLE VELOCITY BASED ON SINUOSITY

Sinuosity*	Percent Reduction in Permissible Velocity
Slight (1.0 to 1.2)	5%
Moderate (1.2 to 1.5)	13%
Very Sinuous (1.5 and greater)	22%

* Sinuosity - degree of curvature of channel.



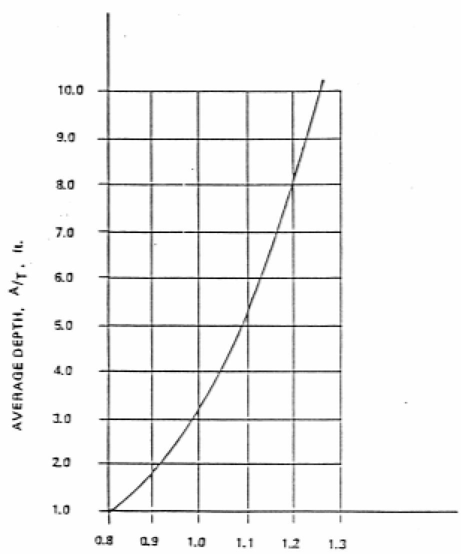
Sinuosity = L'/L

Source: Chow

Figure 4: Recommended reduction in permissible velocity based on sinuosity (Virginia Erosion and Sediment Control Handbook Ch5)

PLATE 5-30

CORRECTION FACTORS BASED FOR PERMISSIBLE VELOCITY BASED ON AVERAGE DEPTH OF FLOW



Source: Va. DSWC

Figure 5: Recommended correction for permissible velocity based on average water depth (Virginia Erosion and Sediment Control Handbook Ch5)

C. NRCS Model

The NRCS spreadsheet method was used in addition to HEC-RAS to determine floodplain velocity. It should be noted that the team considers this method more simplistic than HEC-RAS and is used more of as an order of magnitude check. Using the same NRCS spreadsheet used to design the riffle, the compound open channel flow observed in the channel and floodplain was modelled. Station elevations for the riffle were first put into the spreadsheet without any of the floodplain stations. The Manning's n value for these stations was set as 0.035 as per the Fairfax PFM.

For the compound channel flow experienced in this channel-floodplain complex, the central 'channel' part of the flow needs to be bookended by what are essentially vertical walls of water (Figure 6). This was modelled by bookending the channel stations with extremely steep slopes rising 1 ft above the FEMA 100-yr floodplain height, with a horizontal component of 0.1 ft. These steep slopes were given a Manning's n value of 0.008, the minimum value accepted by the spreadsheet. This was done because water has a very low Manning's n value.

The model for the 'channel' portion of the compound open channel flow was run. Since the height of the FEMA 100-yr floodplain was known, the flow rate associated with that height in the channel was noted. This was set as the flow rate for the central channel portion. It was observed that the velocity in this portion did not exceed the value set while designing the streambed.

To find the flow rates in the 'floodplain' component of the open channel flow, the stations within the FEMA 100-yr floodplain were used as inputs with the channel stations completely excluded (Figure 7). In other words, the left and right floodplain were modelled as though in contact, with the channel removed. A Manning's n value of 0.1 was assigned to each of these stations, based on recommendations in the Fairfax County Public Facilities Manual. The model was run and the flow rate in the 'floodplain' component was noted.

The flow rates from the 'channel' and 'floodplain' components were added up. For the first iteration, with the FEMA 100-yr floodplain height, they did not reach the desired capacity of 968 cfs. In following iterations, the floodplain height was raised in increments of 0.5 ft, until the 968 cfs threshold was either met or exceeded. This was achieved with an elevation of 303.5 ft. When the capacity was high enough to contain this flow rate, the maximum velocity in the floodplain (2.7 ft/s) slightly exceeded 2.6 ft/s. The same analysis was conducted for the 500-yr storm. The floodplain was raised until a total discharge meeting or exceeding 1858 cfs was achieved. This was achieved at a floodplain height of 305 ft. The floodplain velocity from the model output was 3.3 ft/s, which is above the limit set by the maximum permissible velocity.

Our opinion is that this is not a cause for concern for two reasons. The first is that we are looking at velocities for bare soil, while the relevant site is naturally vegetated. Based on the Fairfax PFM, allowable velocity for vegetated channels is 4 ft/s. The soil is therefore most vulnerable when it is bare during construction, and the likelihood of a 500-yr storm during and right after this window is low. Moreover, the allowable velocity is not exceeded by a large margin, suggesting that event these rare erosion events will not result in extreme erosion.

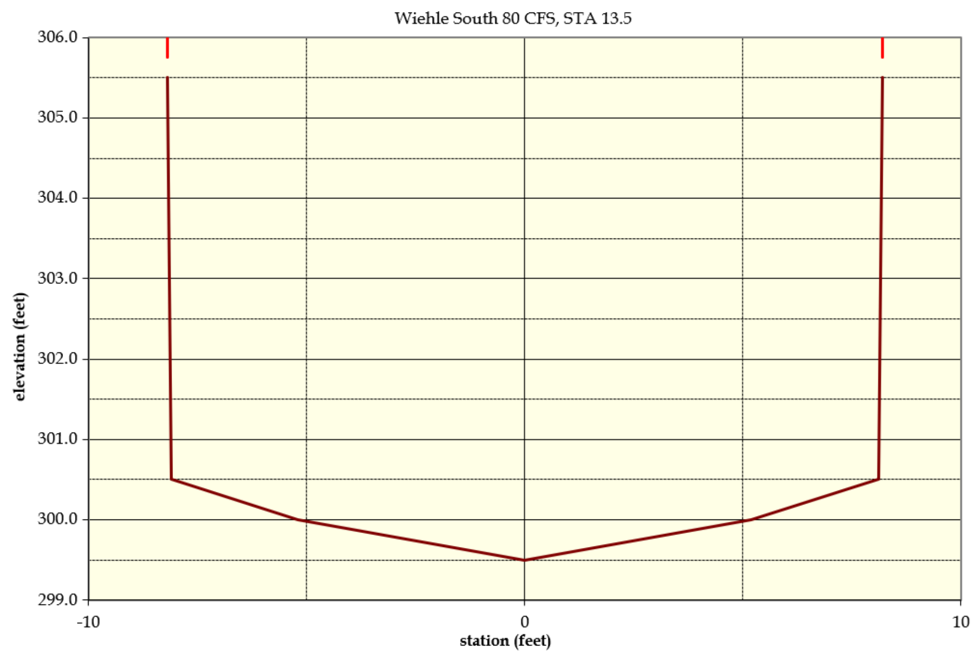


Figure 6: Modelled channel flow with almost vertical 'walls' of water rising to 500-yr floodplain height: The walls of water were assigned the lowest possible Manning's n for the NRCS spreadsheet, i.e. 0.008, while the channel had n = 0.035

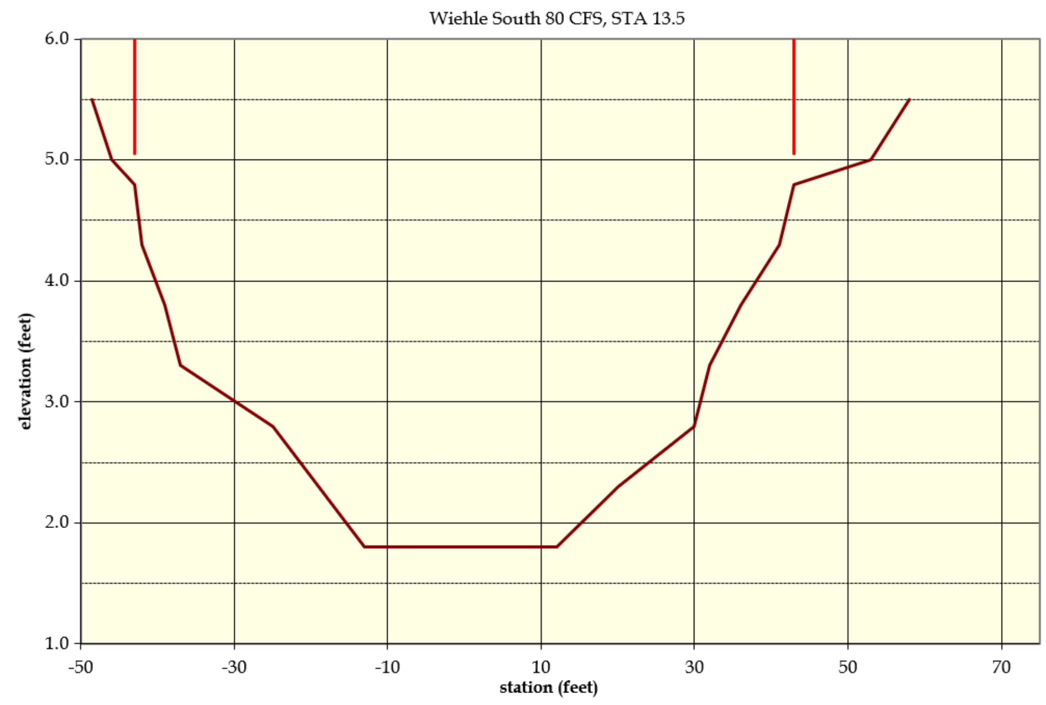


Figure 7: Cross-section of the 500-yr floodplain excluding the channel. The entire cross-section was assigned a Manning's n of 0.1.

D. HEC-RAS Model

The same analysis was conducted using HEC-RAS (Version 5.0.3, U.S. Army Corps, 2016). Using the station data for the final channel design in conjunction with station data obtained from the contour map, the compound open channel flow in the channel and floodplain were modelled in HEC-RAS. The same assumptions were made for slope and Manning's n values for the channel and floodplain.

Given the riffle slope is 0.015, an imaginary reach was drawn with 1000 ft length and 15 ft elevation difference from the start point to the end point. The cross-sectional data from a representative cross-section of the Wiehle South reach was applied to this reach. Since the maximum permissible velocity accounts for sinuosity, the modelled straight reach provided a maximum velocity that was appropriate for comparison with the maximum permissible velocity. Cross-sectional geometry and roughness values were kept the same across the section. The HEC-RAS software interpolated and replicated the cross-section geometry across 1000 ft length to generate a model for a continuous stream, and then simulated in-channel velocity, left-bank velocity, and right-bank velocity. The velocity gradient within each cross-section at each river station was analyzed.

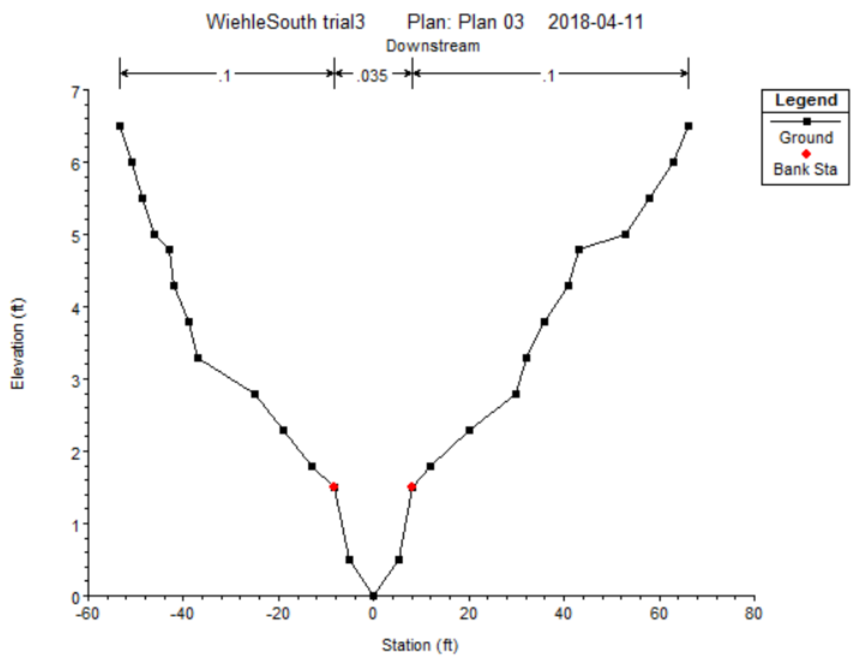


Figure 8: Cross-section geometry entered in HEC-RAS.
The ends of the constructed channel are noted with red dots.

Figure 9 below shows how the given flow of 968 cfs will be contained in the given cross-section geometry along the virtual stream section. Based on the simulation, across the 1000 ft-long stream section, the bank velocities are predicted to reach a maximum of 2.2 ft/s, as shown in Figure 8. Recalling the permissible velocity obtained from the hydrological soil map, the obtained value verifies that such flow hitting the channel would not cause damage to the floodplain. The channel geometry on Figure 7, identical to the center geometry of Figure 9, shows that the channel will not be deformed in case of 100-yr flood.

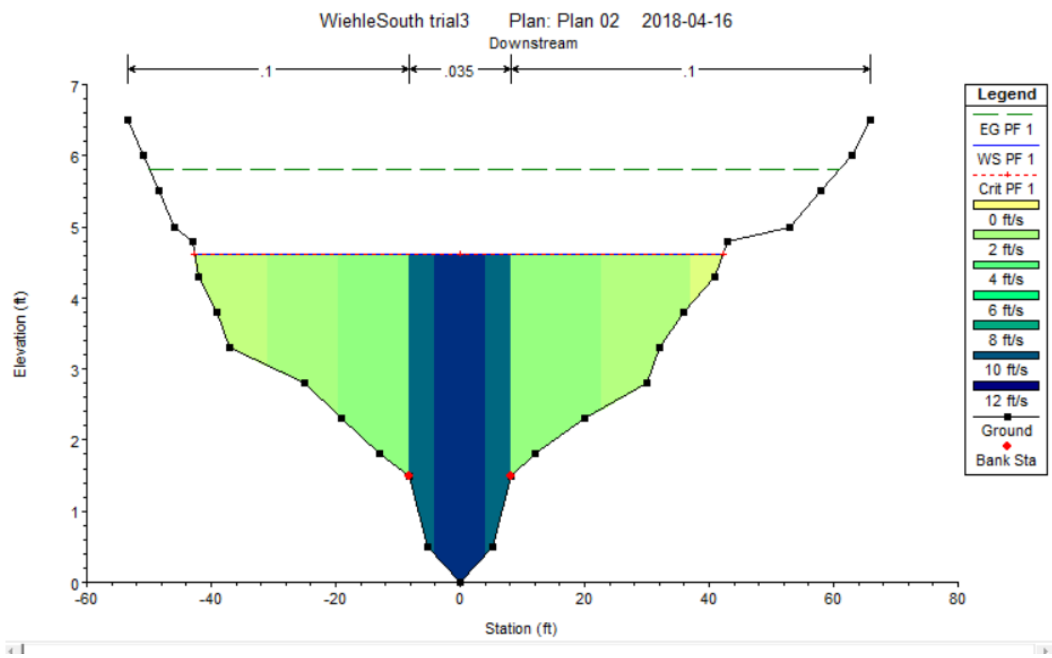


Figure 9: Predicted velocities 1) inside the channel (blue) 2) at the left-bank of the channel (yellow on the left) and 3) at the right-bank of the channel (yellow on the right)

Plan: 100yrWS WS South RS: 0 Profile: PF 1

		Element	Left OB	Channel	Right OB
E.G. Elev (ft)	5.81	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.20	Wt. n-Val.	0.100	0.035	0.100
W.S. Elev (ft)	4.61	Reach Len. (ft)			
Crit W.S. (ft)	4.61	Flow Area (sq ft)	64.51	67.49	62.64
E.G. Slope (ft/ft)	0.009248	Area (sq ft)	64.51	67.49	62.64
Q Total (cfs)	968.00	Flow (cfs)	139.59	694.40	134.01
Top Width (ft)	84.88	Top Width (ft)	34.38	16.50	34.00
Vel Total (ft/s)	4.97	Avg. Vel. (ft/s)	2.16	10.29	2.14
Max Chl Dpth (ft)	4.61	Hydr. Depth (ft)	1.88	4.09	1.84
Conv. Total (cfs)	10065.8	Conv. (cfs)	1451.6	7220.8	1393.5
Length Wtd. (ft)		Wetted Per. (ft)	34.61	16.87	34.20
Min Ch El (ft)	0.00	Shear (lb/sq ft)	1.08	2.31	1.06
Alpha	3.12	Stream Power (lb/ft s)	2.33	23.76	2.26
Frctn Loss (ft)		Cum Volume (acre-ft)			
C & E Loss (ft)		Cum SA (acres)			

Figure 10: Summary of steady flow analysis on 100-yr storm event hitting the riffle design

Plan: 100yrWS WS South RS: 0 Profile: PF 1

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)	Shear (lb/sq ft)	Power (lb/ft s)
1	LOB	-53.50	-42.19	0.01	0.05	0.49	0.00	0.11	0.28	0.06	0.02
2	LOB	-42.19	-30.88	18.06	12.69	11.44	1.87	1.12	1.42	0.64	0.91
3	LOB	-30.88	-19.56	42.15	21.03	11.34	4.35	1.86	2.00	1.07	2.15
4	LOB	-19.56	-8.25	79.37	30.74	11.34	8.20	2.72	2.58	1.56	4.04
5	Chan	-8.25	-4.13	148.61	15.53	4.29	15.35	3.76	9.57	2.09	19.99
6	Chan	-4.13	0.00	198.60	18.22	4.14	20.52	4.42	10.90	2.54	27.67
7	Chan	0.00	4.13	198.60	18.22	4.14	20.52	4.42	10.90	2.54	27.67
8	Chan	4.13	8.25	148.61	15.53	4.29	15.35	3.76	9.57	2.09	19.99
9	ROB	8.25	22.69	92.44	37.65	14.47	9.55	2.61	2.45	1.50	3.69
10	ROB	22.69	37.13	40.00	22.83	14.54	4.13	1.58	1.75	0.91	1.59
11	ROB	37.13	51.56	1.56	2.16	5.19	0.16	0.42	0.72	0.24	0.17

Figure 11: Velocity distribution across the cross-section of the stream (LOB: left of bank, Chan: channel, ROB: right of bank)

The HEC-RAS model yielded similar results to the NRCS spreadsheet, and both models suggest that for the given channel design and the existing soil cover, the floodplain will not experience scour for the flow rate associated with a 100-yr storm. Because of the slope different on the right and the left bank of the stream section, the bank velocities were not identical. The average velocity on the right and the left bank was predicted to be 2.16 and 2.14 ft/s, respectively, which are less than permissible velocity. The simulation showed that the maximum water depth will be 4.09 ft from the lowest point of the channel.

The same analysis was made on the 500-yr storm. In HEC-RAS model, it was found that the velocity on the left bank will be 2.84 ft/s, and exceed 2.7 ft/s. As described in the previous section, this is not deemed a cause for concern for two reasons. The first is that we are looking at velocities for bare soil, while the relevant site is naturally vegetated. The soil is therefore most vulnerable when it is bare during construction, and the likelihood of a 500-yr storm during and right after this window is low. Moreover, the allowable velocity is not exceeded by a large margin, suggesting that event these rare erosion events will not result in extreme erosion. Table 7 summarizes HEC-RAS simulation results for 1-yr, 100-yr, and 500-yr storm events. More information and intermediate steps can be found in Appendix section, at the end of this report.

Table 7: Floodplain characteristics for HEC-RAS estimation

	Storm Frequency			
	1 yr	**100 yr**	**500 yr**	
Storm Flow Rate	80	968	1858.25	cfs
Velocity-LOB[1]	-	2.16	2.86	ft/s
Velocity-Channel	5.02	10.29	12.71	ft/s
Velocity-ROB[2]	-	2.14	2.56	ft/s
Hydraulic Depth-LOB	-	1.88	2.84	ft
Hydraulic Depth-Channel	0.97	4.09	5.57	ft
Hydraulic Depth-ROB	-	1.84	2.39	ft
Shear-LOB	-	1.08	1.64	lb/sq ft
Shear-Channel	0.89	2.31	3.18	lb/sq ft
Shear-ROB	-	1.06	1.39	lb/sq ft

E. Discussion

The cross-sectional geometry obtained from Part 1 was employed in the NRCS Cross-sectional Analyzer spreadsheet and HEC-RAS software to simulate the flow in the floodplain, as a response to 100-yr storm. Using an iterative method with the spreadsheet, the height of floodplain was confirmed. With HEC-RAS software, it was verified that the flow in the floodplain will not exceed the permissible velocity of 2.6 ft/s. The NRCS method indicated a slight exceedance of the permissible velocity, with maximum floodplain velocity being 2.7 ft/s. Due to the conservative nature of the estimates, and the higher reliability of the HEC-RAS method, it can be concluded that the channel design will allow the floodplain to withstand 100-yr flood without significant erosion.

The analysis was also conducted for the 500-yr storm. In this case, it was found that the permissible velocity will be exceeded slightly with both analysis methods. It is important to note that we are looking at velocities for bare soil, while the relevant site is naturally vegetated.

[1] Left of Bank

[2] Right of Bank

As cited in the introduction to this section, the allowable velocity for vegetated channels is 4 ft/s. The soil is therefore most vulnerable when it is bare during construction, and the likelihood of a 500-yr storm during and right after this window is low. Moreover, the allowable velocity is not exceeded by a large margin, suggesting that event these rare erosion events will not result in extreme erosion. Overall, erosion in the the floodplain during high intensity events is not determined to be a cause for concern with the proposed design.

Table 8: Floodplain characteristics for various model assumptions

Model	Storm Frequency (yr)	Floodplain Height (ft)	Flow capacity (cfs)	Permissible Velocity (ft/s)	Modelled Velocity (ft/s)
HEC-RAS	100	303.6	968	2.6 >	2.2
	500	305.1	1858.25	2.7 <	2.9
NRCS	100	304.5	965	2.6 <	2.7
	500	305	1878	2.7 <	3.3

V. Armor Layer Substrate Size and Streambed Stabilization

A. Introduction

Natural channels evolve with banks of sediment found within the watershed. These sediments are eroded and replaced over time by sediment transport throughout the watershed. The Wiehle South reach will be redeveloped to a more stable shape, mimicking a natural evolution of the streambed. The difference between the re-developed channel and more natural channels lies in the peak flows the stream will experience and the expected sediment transport upstream of the reach.

Reston is a “fully developed urban watershed,” and thus lacks equilibrium in its sediment transport processes (WSS, 2012, p. 6). Given the development in the Colvin Run Watershed, most discharge entering the Wiehle South watershed will have originated from residential and recreational land run-off, which will contain insufficient sediment loads to replace the eroding banks of the reach. The urbanized condition of the Colvin Run watershed contributes to urban stream syndrome, which results in higher hydrological flashiness, impaired water quality, altered stream channel, and ecological impacts (Walsh, et al. 2005). Increased water temperature may also increase stream channel erosion, which must be considered for runoff originating on asphalt and other highly radiative surfaces (Hoomehr, et al. 2018). Consequently, erosion is a concern for the design of the Wiehle South stream restoration.

The EPA has identified sediment as top non-point water quality pollutant problem (Rosgen, 1994, 7-3). Channel bed and bank materials affect form and profiles of rivers, sediment transport, and resistance to hydraulic stress (Rosgen, 1994, 5-25). Variability in sediment transport depends on magnitude, duration, seasonality, source of runoff (Rosgen, 1994, 8-6). The shear strength of soil material decreases as water content increases while internal frictional resistance increases with a larger range of particle sizes (Leopold et al., 1964, p. 39).

The WSSI design reflects a desire to correct the entrenched condition of the current Wiehle South channel. WSSI designed for the Wiehle South stream to be changed from a F4 to B4 Rosgen channel type (WSSI, 2012, p. 32). B4 streams are moderately entrenched, have a moderate gradient, and a riffle dominated channel (Rosgen, 1994, 5-6). They have a very stable plan and profile, stable banks, and gravel channel material with some boulders, cobble and sand. General gravel size is 0.08 to 2.5 inches while gravel bed sediment ranges from 0.08-0.63 in in size (Rosgen, 1994, 5-6; Charlton, 2008, p. 105). Streams of the B4 type have moderate sensitivity to disturbance, moderate sediment supply, low bank erosion potential (Rosgen, 1994, 8-9).

The channel bed has distinct particles and aggregates of particles which compose its structure. The aggregates cause a drag on the flow (Leopold et al., 1964, p. 190). In a gravel channel, the armor layer has a significant impact on rates of bedload transport (Charlton, 2008, p. 103). Armoring develops during frequent low flow which entrains small particles. Removal of fine sediment leaves armor bed layer of roughly uniform particle size, protecting finer material from erosion/scour, so a larger critical threshold necessary to break the armor layer (Charlton, 2008, p. 103).

Scour of the bottom of the channel is the principal concern in stream stabilization, so that bank protection may be feasible (VESCP III - 211). To maintain stability of the stream, a substrate layer will be used to minimize erosion of the banks by resisting the shear stress of the streamflow for a majority of discharges. To accomplish stream bed stability, armor particles should function at shear stress threshold condition. The reinforced bed mix will contain a mixture of larger, stable armor particles with finer grain substrate to fill voids and aid in flow, provide habitat, and provide a sediment source (WSS, 2012, p. 6).

B. Calculations

S^3 calculated two channel alternatives using the same equations as WSSI as a basis of comparison between the designs. The first equation based on Shields (1936) calculates required D50 armor particle size using mean depth boundary shear stress results based on the riffle channel dimensions, which is based on water density and channel hydraulic radius (Rosgen, 1994, 8-4). The second equation by Rosgen (2006) relates maximum depth shear stress to the required D50, which is based on maximum channel depth and riffle slope (WSSI, 2012). This calculation is more conservative, and will be used to check the results from the Shields equation. Both Shields and Rosgen equations allow for calculating a D50 for the armor rock of the channel that would withstand the respective shear stresses, and which was used to determine a factor of safety for each method of calculation.

Calculations were run on the three proposed riffle designs, S^3 A, B, and C. Design A has the largest bankfull depth and smallest bankfull width of the three designs. Design B has a bankfull depth equivalent to design C, but a smaller width. Design C is a 20% scaled down version of WSSI's original cross section, but dimensions are slightly different to accommodate construction tolerances. As emphasized in the Table 9 below, option C is the chosen design.

Table 9: Armor Layer Calculations

		A Narrow	B Moderate Slope	**C WSSI Scaled Down**	WSSI
Shields Method	Mean Depth Boundary Shear Stress (lb/ft^2)	1.1	0.9	**0.9**	0.9
	Particle Diameter (in)	3.3	2.8	**2.7**	2.4
	Factor of Safety	2	2.3	**2.4**	2.4
Rosgen Method	Max Depth Boundary Shear Stress (lb/ft^2)	1.9	1.4	**1.4**	1.6
	Particle	5.9	4.3	**4.3**	5

	Diameter (in)				
	Factor of Safety	1.1	1.5	**1.5**	1.3
Maidment Method	Critical Shear Stress	0.017	0.014	**0.014**	0.015
	Factor of Safety	1.8	2.1	2.1	2

C. Analysis

The Rosgen Max Depth Boundary Shear Stress is the most conservative estimation of shear stress in the calculations, so its values will be used to determine particle size and factor of safety. Factor of safety is determined by a direct ratio of calculated particle size to actual particle size (A1) from the quarry. Calculations can be found in the appendix. Consulting professional engineers revealed that in practice, a factor of safety of 1.3 is a viable minimum for this design of stream.

As shown in the above table, the maximum depth boundary shear stress is greatest in channel A, resulting in a factor of safety of 1.1. Channels B and C will experience identical maximum boundary shear stress given their equivalent depths of 1.5 feet which results in the same factor of safety of 1.5. Given this shear stress, the proposed sediment size ideal for stability has a D_{50} of 4.3 inches. Based on Rosgen method factor of safety, the narrow Channel A does not meet the required specifications, as it would require too large of a armor rock D_{50}. Channels B and C both have sufficient factors of safety, although the moderately sloped Channel B was not chosen as our final design due to recommendations from section III. Channel C, with a calculated D_{50} of 4.3; using size A1 substrate from Cedar Mountain Quarry, with a D_{50} of 6.7, is thoroughly adequate. Channel C has has an appropriately high factor of safety for bankfull flow (1.5 using Rosgen method) and a stable channel slope (3:1), and is therefore the channel design elected by the S3 team.

As an order of magnitude check, the critical shear stress on the D_{30} was calculated using the Maidment method. Unlike the procedure used in the Rosgen and Shields methods, this method considers the specific weight of the armor material. If the critical shear stress on the D_{30} is found to be below 0.03 lb/ft^2, the particles are assumed to be stable. The factor of safety indicates the degree to which the maximum critical shear stress is greater than the calculated critical shear stress. This method has been used on previous WSSI plansets and is simply an additional check on the stability of our structural materials. As noted in the above table, all of the critical shear stresses in each channel are below 0.03 lb/ft^2, so D_{30} armor rock in each channel would be theoretically stable.

The empirical data graphed on the trendline in Figure 12 (below) relates critical shear stress and stream bed grain diameter, or D50. The data suggest that 110 mm (4.3 in) grain diameter to withstand such shear stress, which further validates the calculations in Table 8 (USDA NRCS). The output of the Shields shear stress calculations are therefore observable in natural rivers observed by Leopold, et al., emphasizing the fundamental relationship between particle size and shear stress, i.e. the need for a larger armor layer D50 to withstand more erosive forces.

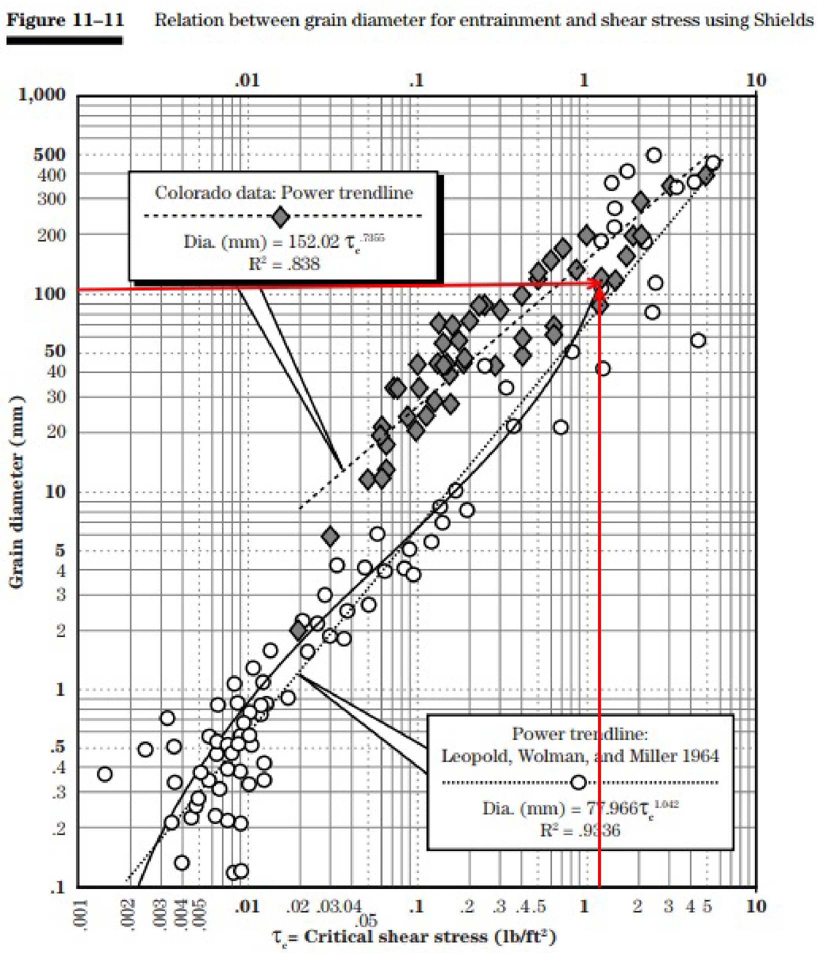


Figure 12: Relation between grain diameter for entrainment and shear stress using Shields (USDA NRCS).

The VDOT Drainage Manual chart in Figure 13 (below) relates flow depth, channel slope, and D50 riprap size. Greater flow depth and channel slope necessitate a larger D50 to protect from increased shear stress from the streamflow. The D50 for Wiehle South, with a channel slope of 0.015 feet/foot and a maximum flow depth of 1.5 ft (design A and B) at bankfull, D50 armor layer particle size should be roughly 4 inches (VDOT). The S^3 calculation of 4.3 in D50-max is similar to the value required by the State of Virginia. The VDOT chart validates the S^3 calculation for the minimum D50 armor layer particle size required to maintain stream channel stability.

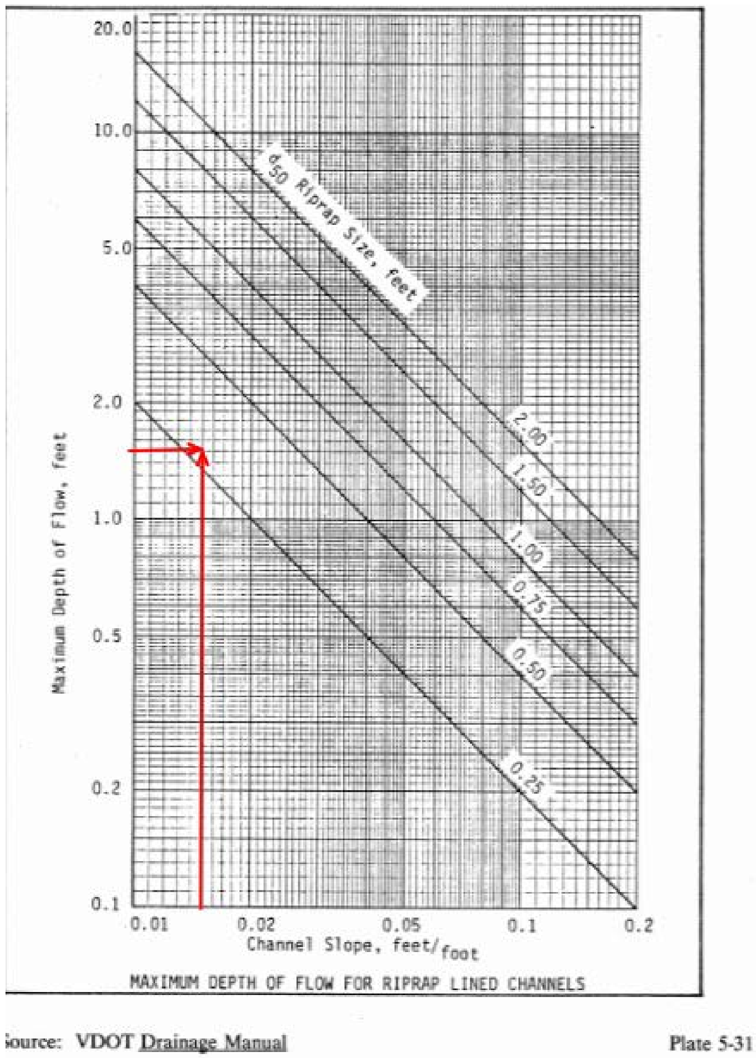


Figure 13: VDOT riprap size requirements (VDOT).

The photograph Figure 14 documents how a relatively stable channel within the Wiehle South reach has a D50 substrate size of around 6-8 in. The D50 of a naturally stable riffle is a good indicator of how large the armor layer must be under the urban runoff condition in the watershed. The riffle stability is evidenced by the lack of vertical erosion, or entrenchment, present in most other sections of the stream due to unstable degradation of the channel. This cursory field survey provides empirical justification for the role of the armor layer in providing stability in fluvial processes. The substrate size for the Wiehle South channel design should be within a similar range as the D50 shown in this photo.

Figure 14: Wiehle South site visit (S^3).

Although all of the S^3 riffle designs result in similar D50 particle size calculation results as the design in the WSSI plan, the construction cost of the S^3 designs would be less since they are both narrower, and cost is proportional to the volume of rock purchased, relocated to, and installed at the Wiehle South site. The WSSl riffle channel would contain 113 cfs discharge and be 20 ft wide, 1.7 ft deep with a 1 ft hydraulic radius. The S^3 scaled channel would contain 80 cfs discharge, be 16.5 ft wide and 1.5 ft deep, and the narrow channel would be 11.5 ft wide with 2 ft depth. For the sake of channel stability, given its 3:1 channel side slope, the S^3 team has opted for the scaled channel. The largest predictor of cost will be the stream width since that represents the area of impact during construction.

VI. Conclusion

Factors for consideration in choosing riffle channel dimensions include concerns for floodplain erosion, channel stability, and project economics. The S^3 team converged on channel design option C as the most viable alternative to the 2012 WSSI Wiehle South riffle design in these three areas.

In the occurrence of a 100-yr storm event, the riparian area along Wiehle South must contain the 968 cfs discharge within its floodplain with minimal levels of erosion. The S^3 team used HEC-RAS and the NRCS cross-sectional analyser to model the performance of riffle design C under such a flood scenario. The modeled velocity for HEC-RAS did not exceed the 2.6 ft/s permissible velocity. The NRCS method indicated a slight exceedance, giving a floodplain velocity of 2.7 ft/s. The team considers the HEC-RAS analysis more reliable than the simple NRCS spreadsheet analysis, and the latter was used as an order of magnitude check.

The analysis was also conducted for the 500-yr storm. In this case, it was found that the permissible velocity will be exceeded slightly based on both analyses. It is important to note that analysis was very conservative since it assumed bare soils. The natural vegetation in the regions outside the period of construction and regrowth will result in soils less vulnerable to erosion. For both the 100-yr storm and the 500-yr storm, the potential exceedances in allowable velocity are not deemed a cause for concern due to the conservative nature of the velocity thresholds.

The 1.5 ft channel depth and 3:1 cross-sectional channel slope were the primary factors influencing in-channel shear stress**.** The S^3 calculations resulted in a shear stress factor of safety of 1.5 surpassing the 1.3 factor of safety for the WSSI design. The channel will need armor layer substrate with at least a 4.3 in D50 to withstand the erosion from a bankfull discharge event, and can safely use Cedar Mountain Quarry's 6.7 in A1 substrate. The reinforced channel bed will function as a downscaled version of that specified in the WSSI plan.

The riffle channel design C proposed by S^3 is 3.5 ft narrower than the WSSI design since it is designed to contain a bankfull discharge of 80 cfs instead of the design discharge of 113 cfs used by WSSI. The smaller cross sectional area reduces construction costs due to a smaller area of impact and less volume of material (chiefly channel bed substrate mix) that needs to be purchased. Construction costs will be lowered by electing design C.

Finding that the design meets the aforementioned criteria for erosion control and cost minimization, the S^3 team recommends that riffle channel design C, the scaled design, be used in the restoration of the Wiehle South stream reach in Reston, VA.

VII. References

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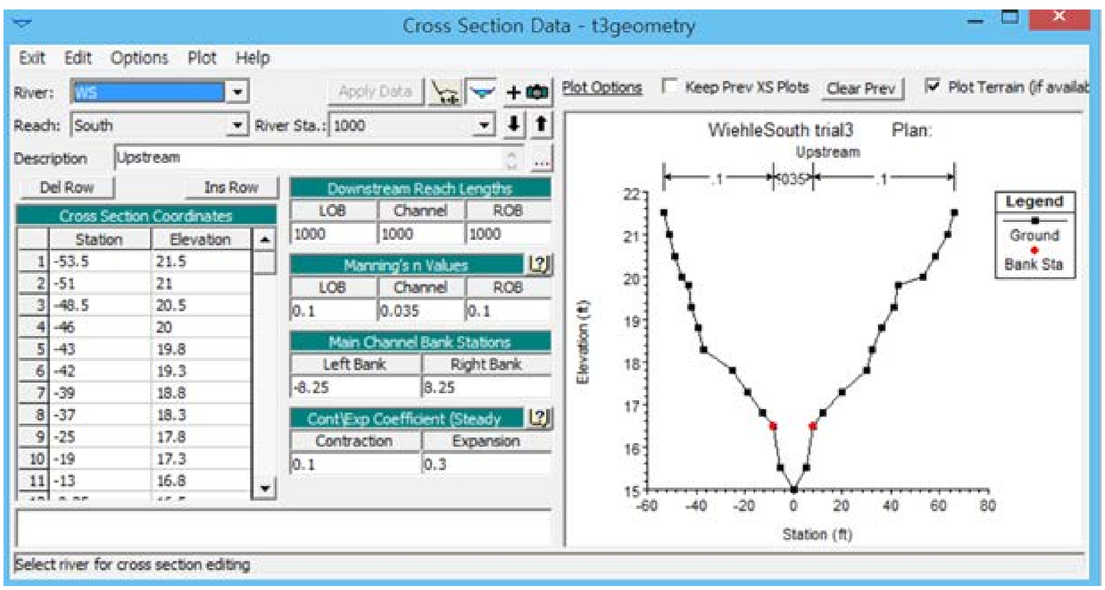
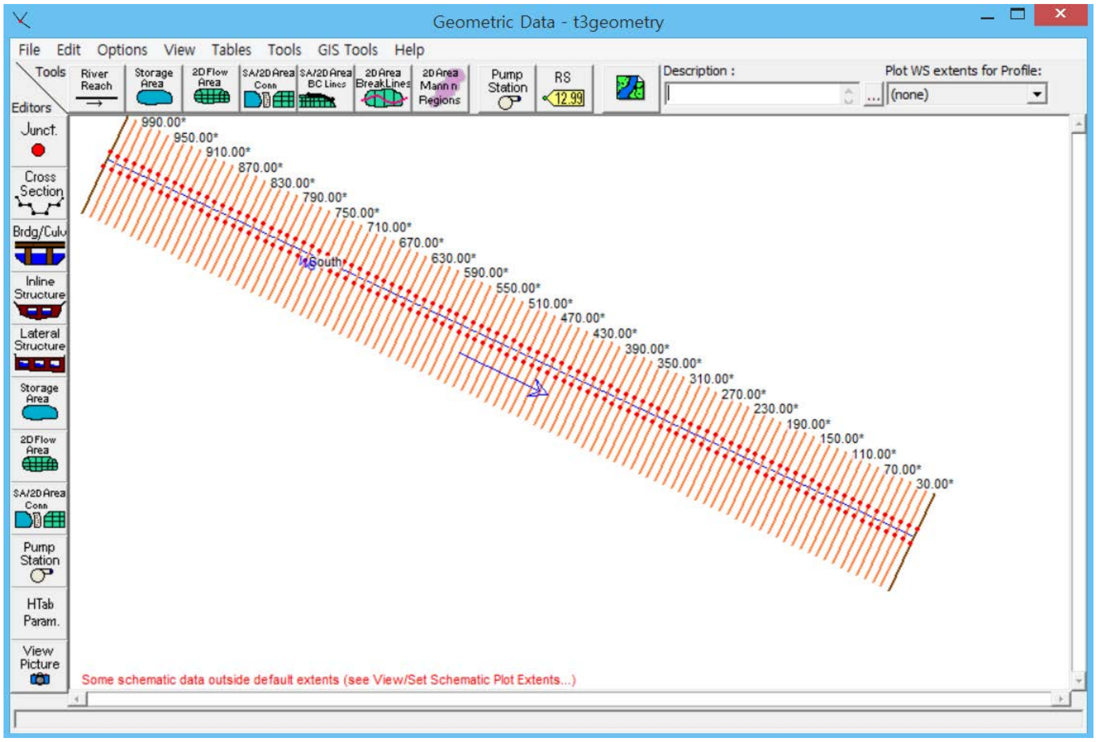
Northern Virginia Stream Restoration Bank Wiehle South, Prepared by Wetland and Stream Solutions Inc. 4/23/17, Last revised: 1/12, 69 sheets

VIII. Appendix

A. HEC-RAS

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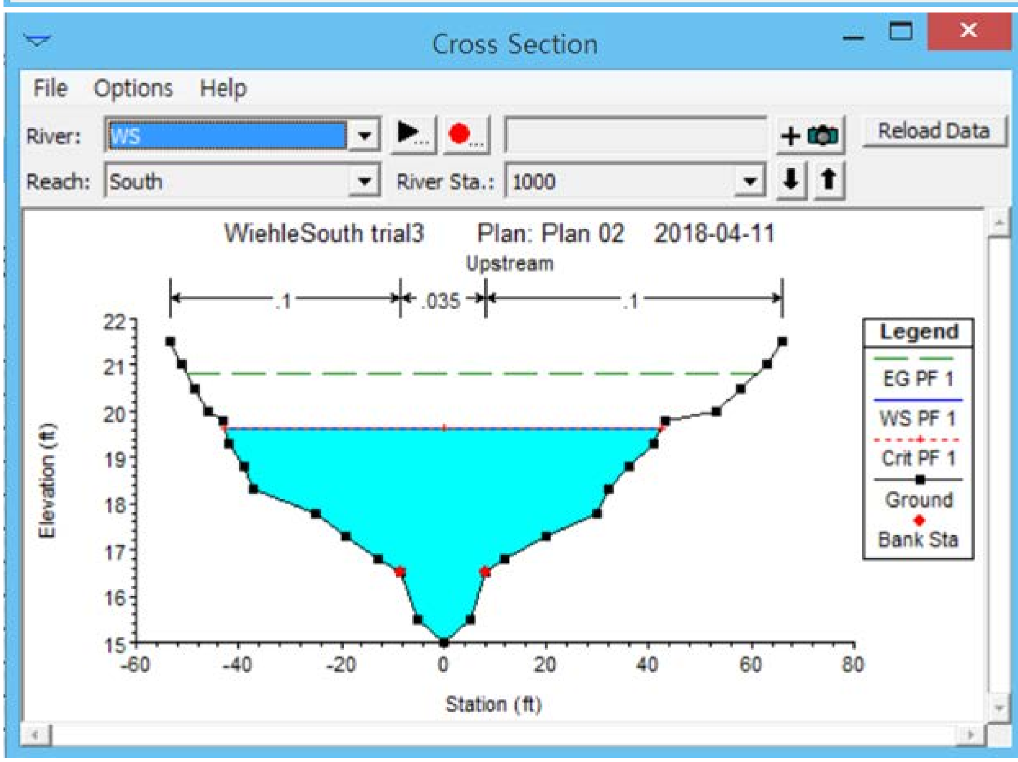
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Reach: South
River Sta.: 1000
Reload Data
WiehleSouth trial3 Plan: Plan 02 2018-04-11
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Legend
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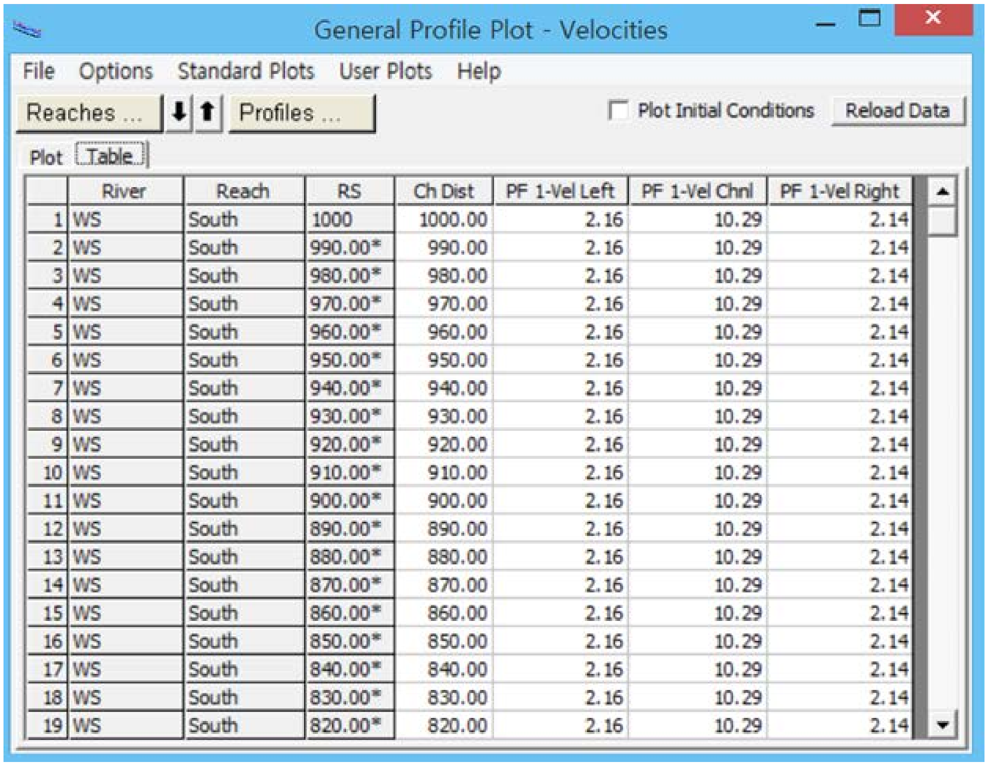
Ground
Bank Sta



General Profile Plot - Velocities

File Options Standard Plots User Plots Help

Reaches ... Profiles ... Plot Initial Conditions Reload Data

Plot Table

	River	Reach	RS	Ch Dist	PF 1-Vel Left	PF 1-Vel Chnl	PF 1-Vel Right
1	WS	South	1000	1000.00	2.16	10.29	2.14
2	WS	South	990.00*	990.00	2.16	10.29	2.14
3	WS	South	980.00*	980.00	2.16	10.29	2.14
4	WS	South	970.00*	970.00	2.16	10.29	2.14
5	WS	South	960.00*	960.00	2.16	10.29	2.14
6	WS	South	950.00*	950.00	2.16	10.29	2.14
7	WS	South	940.00*	940.00	2.16	10.29	2.14
8	WS	South	930.00*	930.00	2.16	10.29	2.14
9	WS	South	920.00*	920.00	2.16	10.29	2.14
10	WS	South	910.00*	910.00	2.16	10.29	2.14
11	WS	South	900.00*	900.00	2.16	10.29	2.14
12	WS	South	890.00*	890.00	2.16	10.29	2.14
13	WS	South	880.00*	880.00	2.16	10.29	2.14
14	WS	South	870.00*	870.00	2.16	10.29	2.14
15	WS	South	860.00*	860.00	2.16	10.29	2.14
16	WS	South	850.00*	850.00	2.16	10.29	2.14
17	WS	South	840.00*	840.00	2.16	10.29	2.14
18	WS	South	830.00*	830.00	2.16	10.29	2.14
19	WS	South	820.00*	820.00	2.16	10.29	2.14

B. Armor Layer Calculation

Step 1: Armor Size Determination for D50 Particle							
Equation 1: Mean Depth Boundary Shear Stress			Wiehle South	Wiehle South	Wiehle South	WSS Plan Set	
T0 = Yf*R*S			"80 cfs - scaled"	"80 cfs - narrow"	"80 cfs idealized"	"113 cfs"	
	Specific weight of water	**Yf**	62.4	62.4	62.4	62.4	lb/ft^3
	Hydraulic Radius (Abkf/WP)	**R**	0.97	1.15	0.99	1	ft
	Bankfull Area	Abkf	16.3	14.2	15.4	22.9	ft^2
	Wetted Perimeter (2Dbkf + Wbkf	WP	16.9	12.3	15.5	22.2	ft
	Bankfull Mean Depth (Abkf/Wbkf)	Dbkf	0.9644970414	1.154471545	0.9935483871	1.1	ft
	Bankfull Width	Wbkf	16.5	11.5	15	20	ft
	Maximum Riffle Slope	**S**	0.015	0.015	0.015	0.015	ft/ft
	Mean Depth Boundary Shear Stress	**T0**	**0.90792**	**1.0764**	**0.92664**	0.9	lb/ft^2
Equation 2: Armor Portion Particle Size							
D50 = 3.07 * (T0^1.042)							
	Boundary Shear Stress	**T0**	0.90792	1.0764	0.92664	0.9	lb/ft^2
	Particle Mass Median Diameter	**D50**	**2.776028733**	**3.314781906**	**2.835696079**	2.8	in
Equation 3: Factor of Safety							
FS = D50-actual/D50							
	Actual Mean Diameter of Quarry Armo	**D50-actual**	6.7	6.7	6.7	6.7	in
	Stable Mean Diameter of Armor Rock	**D50**	2.776028733	3.314781906	2.835696079	2.8	in
	Factor of Safety	**FS**	2.413519687	2.021249117	2.362735573	2.4	

Step 2: Conservative Estimate of Stable "Armor Rock" (Using Max Riffle Depth)							
Equation 4: Maximum Depth Boundary Shear Stress:							
t(0-max) = Yf * Dmax * S							
	Specific Weight of Water	**Yf**	62.4	62.4	62.4	62.4	lb/ft^3
	Maximum Riffle Depth	**Dmax**	1.5	2	1.5	1.7	ft
	Maximum Riffle Slope	**S**	0.015	0.015	0.015	0.015	ft/ft
	Max Depth Boundary Shear Stress	**t(0-max)**	**1.404**	**1.872**	**1.404**	1.6	Lb/ft^2
Equation 5: "Armor" Portion Particle Size							
D50 = 3.07*T(0-max)^1.042							
	Max Depth Boundary Shear Stress	**t(0-max)**	1.404	1.872	1.404	1.6	Lb/ft^2
	Rock Particle Diameter	D50-max	**4.372148474**	**5.900394715**	**4.372148474**	5	in
Equation 6: Factor of Safety							
FS = D50-actual/D50							
	Actual Mean Diameter (From Quarry)	**D50-actual**	6.7	6.7	6.7	6.7	in
	Stable Mean Diameter (From Eq. 5)	**D50**	4.372148474	5.900394715	4.372148474	5	in
	Factor of Safety	**FS**	1.532427373	1.135517253	1.532427373	1.3	

Step 3: Corroboration of Results							
Equation 7							
t* = t0/((Ys-Yf)*D30)							
	Mean Depth Boundary Shear Stress	**T0**	0.90792	1.0764	0.92664	0.9	lb/ft^2
	Specific Weight of Sediment	**Ys**	190	190	190	190	lb/ft^3
	Specific Weight of Water	**Yf**	62.4	62.4	62.4	62.4	lb/ft^3
	Diameter of > 30% of Particles	**D30**	0.5	0.5	0.5	0.47	ft
	Critical Shear Stress	t*	0.014230721	0.01687147335	0.01452413793	0.015	lb/ft^2

Assignment #4: Wiehle South Climate and Land Use Changes

Smart Stream Solutions Inc.

We have each personally reviewed and approve the following report:

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Draft: April 11, 2018

Revised: April 27, 2018

Revised: May 2, 2018

I. Executive Summary

S^3 proposed a riffle design for the Wiehle South stream reach in its third report.[1] This design was based on hydrologic models that utilize past meteorological measurements and the current land use in the watershed to determine the design flow rate. The risk involved with the assumption that historical trends will be accurate for future conditions is the reason for this report. The Wiehle South Climate and Land Use Changes report analyzes possible changes to land use and climate within the watershed, and assesses the magnitude of the impact on design discharge.

The most probable and impactful land use change would be conversion of the Hidden Creek Country Club to residences. For the scenario in which mitigation of stormwater runoff from impervious area as required by Fairfax County is implemented the design flow decreased from 80 cfs to 67 cfs. The stream channel could be 0.5 feet narrower and still contain the necessary flow. Using a scenario in which the entire area is converted to medium density housing without runoff mitigation the design flow rate increased from 80 cfs to 95 cfs. The stream channel would need to be 1.5 feet wider to convey this flow.

Climate change modeling was used to assess changes to rainfall events. The model predicted increased design storm intensity in the future. Projections through 2100 were chosen for analysis.[2] The model estimates that the 1 year storm intensity will have increased from 2.62 to 3.08 inches. This increase combined with the previously discussed land use change scenarios results in a design flow rate of 98 and 140 cfs. The stream channel would need to be 3 feet wider to convey this flow.

Impacts due to climate change are expected to include increased flow due to large storm events. The currently delineated 100 year floodplain would not be sufficient for these flows. According modeling for the year 2080 performed by S^3 the floodplain width would need to be increased by 1.5 feet.

[1] S^3 Wiehle South Riffle Design Analysis, 2018.

[2] Despite the fact that the climate model data used by S^3 projected through the year 2100, the year 2080 was selected as an arbitrary reference frame for analysis. Since there are projections before and after that date, it can be part of a predictive trendline used to scale future metrics of precipitation and temperature.

The risk of failure due to land use changes should be taken into account since the stability of the Wiehle South channel design will be guaranteed for 10 years, and the stream should be stable in perpetuity. The effects of climate change could be significant, but have large uncertainty. Somewhat more frequent bankfull events would not be problematic, as discussed in Assignment 1. Due to this S^3 recommends that the current channel design from Assignment 3, labeled “Current S^3 Design” in Section VI, Table 4, be used. If the current channel is used with urban development and climate change impacts the return period for bankfull flow events would be shorter, with potentially 15-20 bankfull causing storms per decade by the end of the century, or around 2 per year, as shown in Section IV, Figure 5.

II. Introduction

The stream design for the Wiehle South reach in Reston, Virginia proposed by S^3 is based on hydrologic models utilizing past meteorological measurements and the current land use in the watershed to determine the design flow rate. In order to evaluate risk and ensure the designed stream will be stable for at least the 10 years for which it is guaranteed, an evaluation of future changes to these parameters must be considered.

This report will outline current zoning and regulations for the Wiehle South watershed to evaluate the likelihood and magnitude of impact on design flow rate, as well as climate models which project potential changes to the design storm. To explore these prospective impacts, S^3 will calculate discharge and channel dimensions for four possible future scenarios:

1. Urban Development with Stormwater Mitigation Scenario
2. Urban Development without Stormwater Mitigation Scenario
3. Urban Development with mitigation and Climate Change Scenario for the year 2080
4. Urban Development without mitigation and Climate change Scenario for the year 2080.

III. Possible Land Use Changes

The developed land in the Wiehle South watershed is primarily residential, while the remaining land is recreational. Approximately 23% of the 282 acre watershed is the Hidden Creek Country Club, which has a large golf course. The club was sold in October 2017 to Wheelock Communities.[3] Wheelock may have additional plans for the area. An email from the previous owners to Reston residents stated that “Over the next few years, Wheelock will be working in partnership with the club members and the Reston community to explore potential changes to the property that could provide the Reston community with additional public amenities,

[3] Reston Now, 2017.

environmental benefits and new housing choices."[4] These potential changes could alter the impervious cover of the watershed, which would increase the magnitude of the design storm. This would be much more significant than any other land use change possible within the watershed, so other possible changes will be neglected.

The land is designated for private recreation according to the Fairfax County Comprehensive Plan, as shown below in Figure 1. Figure 2 shows the official zoning for the area, which is planned residential community. This zoning would likely allow for conversion to residences, if Reston approved the change.

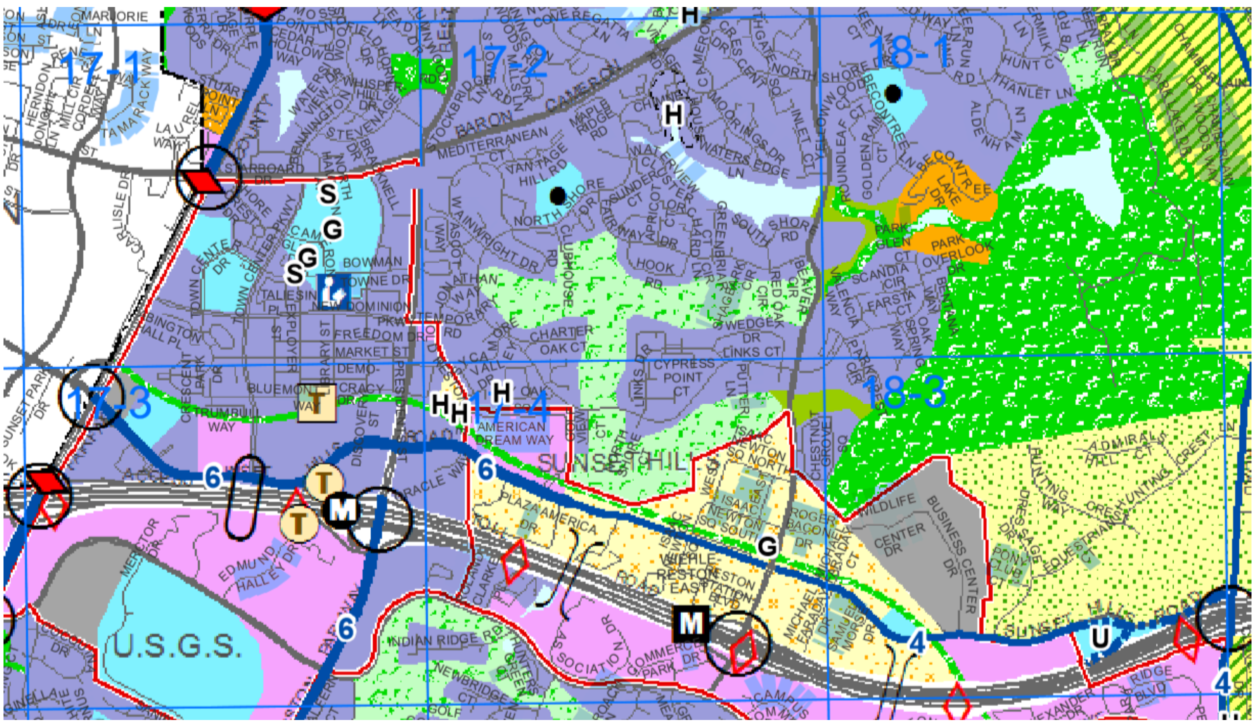

Figure 1: Section of Fairfax County Comprehensive Plan that the Wiehle South watershed is located within.[5] The Hidden Creek Country Club is marked as private recreation (light speckled green).

[4] Ibid.

[5] Fairfax County Comprehensive Plan, 2017.

Figure 2: Section of the Fairfax County Zoning Map which the Wiehle South watershed is located within.[6] The Hidden Creek Country Club is zoned as a planned residential community(light blue).

Fairfax County Municipal Code section § 124-4-4.B.3 requires that new land disturbing activities result in lower developed peak flow rates than if the land was forested.[7] Fairfax County Public Facilities Manual (PFM) Section 6-0203.4A(1) states that:

> It shall be presumed that no adverse impact and an improvement will occur if onsite detention is provided as follows and the outfall is discharging into a defined channel or man made drainage facility: In order to compensate for the increase in runoff volume, the 1-year, 2-year and 10-year post-development peak rates of runoff from the development site shall be reduced below the respective peak rates of runoff for the site in good forested condition (e.g., for NRCS method, a cover type of "woods" and a hydrologic condition of "good") in accordance with the requirements of Chapter 124 of the County Code.[8]

Consequently development within Fairfax County should not impact the flow rates in county streams. The effects of redevelopment on the Hidden Creek golf course site with proper compliance with this regulation are gauged in the Urban Development with Stormwater Mitigation Scenario.

If the local government allows for an exception to this rule, impacts could be significant. Additionally, heavy runoff during construction is possible if the stormwater management plan

[6] Fairfax County Zoning, 2018.

[7] Fairfax County Municipal Code, 2017.

[8] Fairfax County Public Facilities Manual, 2011.

best management practices are improperly implemented, or are overwhelmed by a large storm. Such runoff could result in discharge the channel was not designed for, in addition to sediment and other waterbound pollution. This case is explored in the Urban Development without Stormwater Mitigation Scenario.

The impact analysis section will consider a scenario in which the entire golf course is converted to typical residential without runoff mitigation. The impervious area percentage for medium density residential is 30%.[9] If the 0.1 square miles of golf course were converted it would result in an increase of impervious area to roughly 36% from its current 26.6%.[10] Since these changes are purely speculative, their accuracy is limited.

IV. Climate Change

Predicted changes in global climate are primarily based on simulated responses to increased and accumulated greenhouse gas emissions.[11] Climate projections from NASA Earth Exchange Global Daily Downscaled Projections (NEX-GDDP) provide data downscaled to the Reston area encompassing the Wiehle South watershed.[12] The General Circulation Models (GCMs) produce projections of daily precipitation, daily maximum temperature, and daily minimum temperature for a general geographic area.

S^3 selected the NOAA GFDL-ESM2G model with projections through the year 2100. S^3 used data from two climate scenarios: Representative Concentration Pathway (RCP) 4.5, a moderate-to-optimistic prediction of greenhouse gas emissions, and RCP 8.5, which represents a starker emissions scenario and thus more intense global warming.[13] Data from the two RCPs diverge mainly in terms of projected temperature.

Given their large scale, there are limitations in the predictive utility of the GCM models. Their resolution is 0.25 degrees (~25 km x 25 km).[14] They are unable to predict storms which occur on a sub-grid scale, such as summer convective rainfall.[15] Since deep convection contributes to

[9] USDA NRCS, 1986.

[10] S^3 Wiehle South Report 2, 2018.

[11] Green, 2016.

[12] OpenNEX / Planet OS, 2015. Climate scenarios used were from the NEX-GDDP dataset, prepared by the Climate Analytics Group and NASA Ames Research Center using the NASA Earth Exchange, and distributed by the NASA Center for Climate Simulation (NCCS). See Appendix D for R code.

[13] NASA, 2014.

[14] NASA, 2015.

[15] Benestad, 2016.

heavy precipitation events, overall rainfall and storm intensity are likely underestimated.[16] While amounts are not accurate, more general trends may prove useful in calculating design discharge for a stream restoration project given the immediate impact of climate dynamics on local hydrology.

A. Precipitation

The S[3] team established the 1 year storm as the bankfull discharge event for which the Wiehle South reach should be designed.[17] The rainfall intensity influences dimensions such as channel capacity and the ability of the channel and floodplain to withstand certain flow velocity and shear stress. Storm intensity particularly affects small channel-source flood flows, and smaller, more urban watersheds have a greater propensity for flashiness.[18] Since Wiehle South is a relatively small watershed with a natural (not reinforced) channel, it could prove sensitive to changes in rainfall patterns.

Figure 3 (below) suggests an overall no significant changes in total daily precipitation over the next 80 years. Despite potential constancy in the overall precipitation, the distribution of the storm events may be variable.

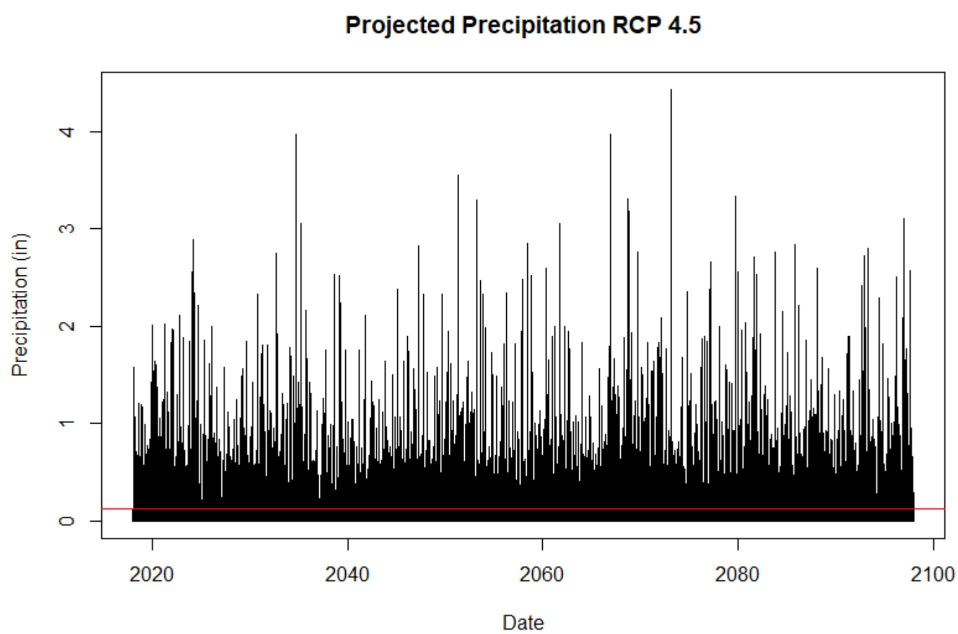

Figure 3: Daily precipitation projection over 80 year period.

[16] Prein, et al., 2015.
[17] S[3] Wiehle South Report 1, 2018.
[18] Hewlett & Bosch, 1983; Baker, et al. 2004.

Figure 4 (below) illustrates an increase in 1 year storm intensity, evident in the upward linear regression trendline (in red). The analysis involved calculating the maximum precipitation for each of the 80 years projected in the climate model. The 2 inch rainfall as the baseline storm is an underestimate from the current 1 year, 24 hour precipitation of 2.62 inches per the Fairfax PFM, suggesting the limitation of the GCM data accuracy and the probable exclusion of rainfall from the predictions.[19] Regardless, the 1 year storm intensity trend matches regional observations of a 27% increase in heavy precipitation events in the southeast United States from 1958 to 2012.[20] S[3] will use the GCM precipitation projections not to forecast changes in actual precipitation but instead calculate the change in magnitude of storm intensity using statistical analysis.

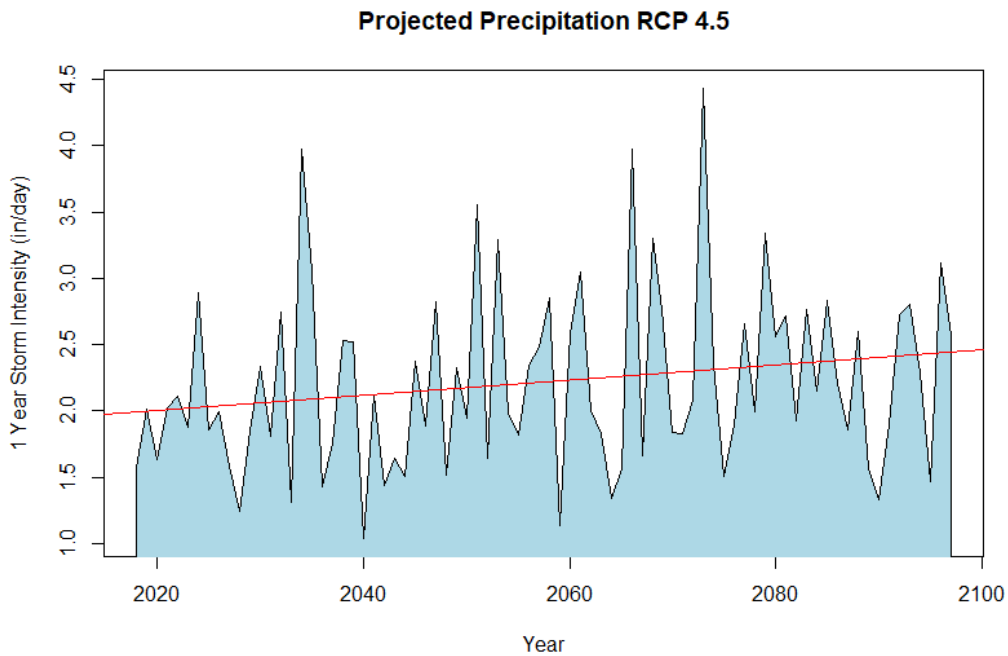

Figure 4: Bankfull storm projections over 80 year period.

Over the 8 decades of precipitation simulated in the NOAA GFDL-ESM2G climate model for the Reston area, there is an increase in the relative frequency of storms which would cause bankfull events, or 24 hour rainfall of more than 2.62 inches. More rainfall intensity has implications for the Wiehle South stream design. The projected increase in flood frequency is of a greater magnitude than the increase in 1 year storm intensity.

[19] Fairfax County Public Facilities Manual, 2011.

[20] NCA, 2014.

Using linear regression for the climate projections, S³ estimates that the 1 year storm will increase to 3.08 inches and the 100 year storm will increase to 10.24 inches. The floodplain of the restored stream must withstand certain flow velocities during the 100 year storm, delineated in Report 3 as 3.3 feet/second for the contemporary 100 year flood and modeled using climate projection data in Section V.[21]

Greater bankfull capacity will be needed if the channel is designed on climate change time horizon. The 1 year storm discharge drives channel capacity design, a relationship further elaborated in Section VI. Figure 5 (below) shows the bankfull frequency over an 80 year period, summed for each decade, with a linear regression trendline (in red).

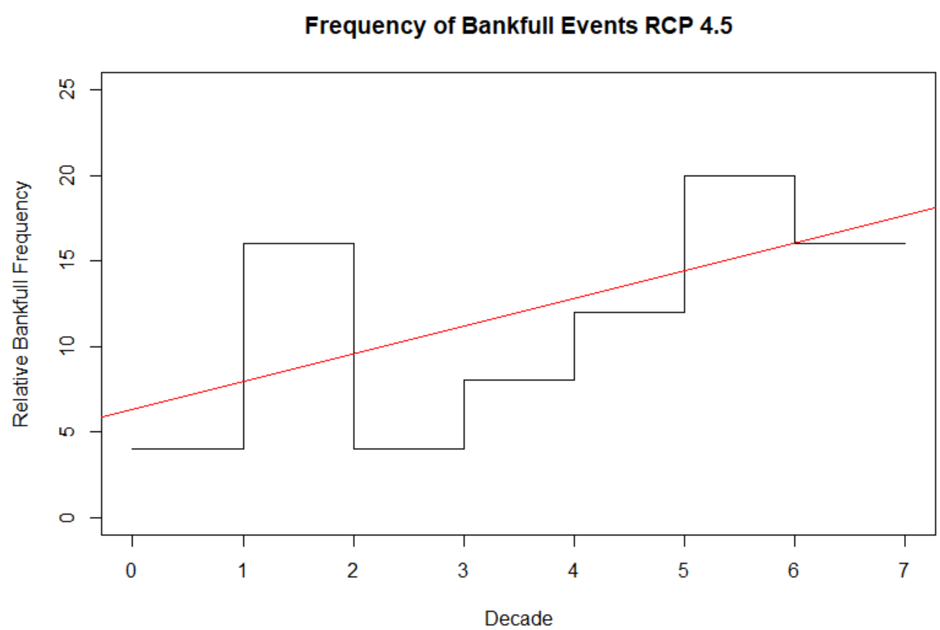

Figure 5: Relative bankfull frequency over 80 years based on frequency of 1 year storm at a present baseline of 2.62 inches.[22]

B. Temperature

The most evident use for the GCM climate data is to predict changes in temperature. Since temperature shifts occur on a large climactic scale, the temperature projections can be downscaled for application to a particular location. Figures 6 and 7 (below) demonstrate a

[21] S³ Wiehle South Riffle Design Analysis Report, 2018.

[22] The bankfull frequency was scaled based on projections for the first two decades since the GCM precipitation projections are an underestimate for reasons mentioned on page 6 of this report. A linear regression line was then applied based on the frequency of scaled bankfull events per decade for the projections through 2100.

upward trends in daily maximum temperature, especially in the RCP 8.5 model, given the linear regression trendline (in red).

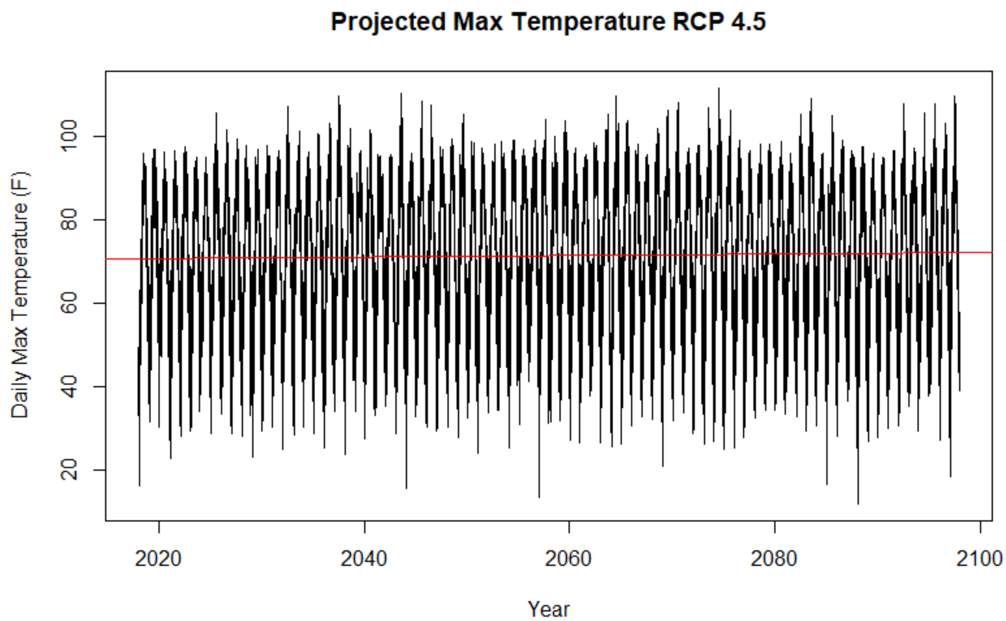

Figure 6: Projected daily maximum temperature under RCP 4.5 emissions scenario.

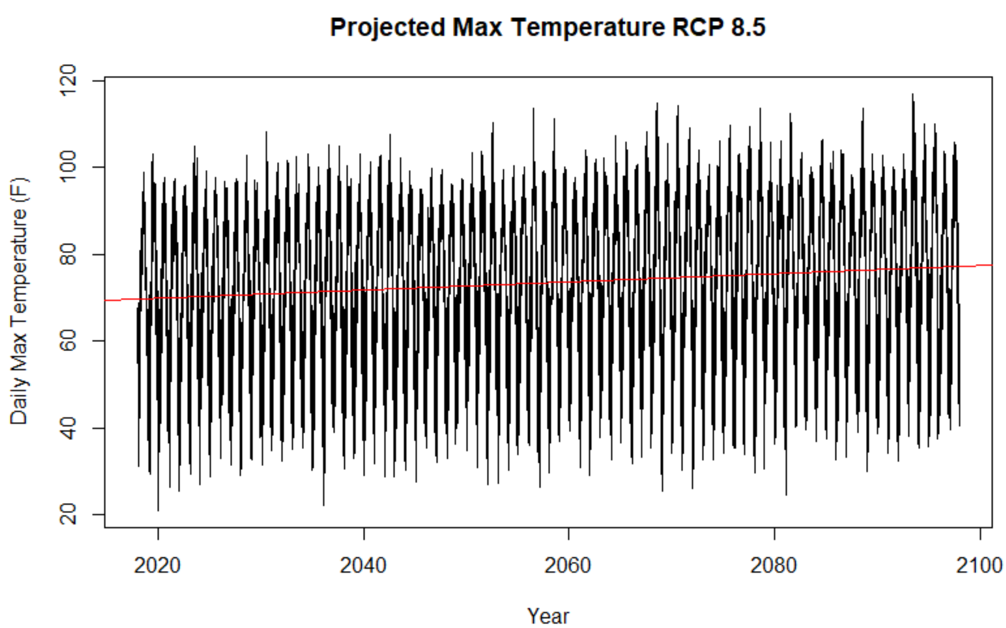

Figure 7: Projected daily maximum temperature under RCP 8.5 emissions scenario.

Higher surface temperatures correspond with an increase in potential evapotranspiration (PET) and a decrease in runoff and soil water content.[23] Changes in the magnitude of precipitation and PET will impact streamflow.[24] The increase in temperature may result in lower baseflow in the stream, especially since total precipitation is not predicted to rise in tandem with the intensity of maximum temperature increases.

Urbanization and climate change both contribute to the urban stream syndrome, defined by a "flashier hydrograph, elevated concentrations of nutrients and contaminants, altered channel morphology, and reduced biotic richness."[25] In addition to the already-present effects of urban development in the Wiehle South watershed, rising temperatures may have implications for local ecology. Some riparian species are sensitive to water temperature change, which is already exacerbated by warmer urban runoff.[26] An increase in precipitation is correlated with sediment and other pollution to water bodies.[27]

V. Impact Analysis

The hydrological impacts of urbanization and climate change are interlinked. Land development clearly impacts the stability of watercourses. As noted in the first S^3 report, impervious land cover of urban areas increases storm runoff, flood frequency, and water pollution.[28] Climate change in the Wiehle South watershed will likely cause higher rainfall intensity and an increase in frequency of what are currently considered bankfull events. Urbanization and wetland losses in the Mid-Atlantic U.S. results in the region's streams becoming more vulnerable to more frequent and intense storms.[29]

Fairfax County has regulations requiring stormwater runoff mitigation from new development within its limits, which include the Wiehle South watershed.[30] Even if regulations are followed, temporary impacts from the prospective redevelopment of the Hidden Creek golf course site may cause an increase in contamination. The EPA states that construction causes the highest

[23] Lakshmi, et al., 2003; Green, 2016
[24] Ficklin, et al., 2016.
[25] Ibid.
[26] Walsh, et al., 2005.
[27] Ibid.
[28] S^3 Wiehle South Design Storm Report, 2018.
[29] Rogers & McCarty, 2000.
[30] Reston Association, 2017.

loading of total suspended sediment of any land use classification.[31] Such sediment would be considered a pollutant.[32]

The analysis in this report is constrained by the unknowable nature of future developments in both the built and climactic realms. While the downscaled GCM climate models have much uncertainty in regards to precipitation, for the purposes of watershed engineering it seems valid to operate under the assumption of an altered intensity regime. There may be unforeseen feedback loops in hydrologic cycle due to climate change. The year 2080 was chosen as an arbitrary time horizon towards the limit of an 80 year range of projected data.

Redesigning the channel for a longer time horizon would require consideration of urbanization (without appropriate stormwater management) and climate change. S^3 used the TR-55 in-channel flow calculation method to examine the impact of future watershed changes on the Wiehle South stream reach.[33] Consideration for flooding into the floodplain is not urgent because the Wiehle South stream flows through a forested park area.[34]

A. Urban Development with Stormwater Mitigation Scenario

This scenario considers development of the Hidden Creek Country Club that meets current Fairfax County regulations. These regulations require that the redeveloped areas include runoff mitigation resulting in a curve number equal to or lower than a forest in good condition. Table 1 describes the scenario parameters. The runoff curve number is lower than the current value of 71.[35] Even though impervious cover is higher due to assumed development, there is less runoff because excess stormwater is managed on site per the PFM regulations.

Impervious cover	32%
Curve number	69
1 year, 24 hour rainfall	2.62 inches
2 year, 24 hour rainfall	3.17 inches

31 EPA, 1999.

32 Reston Association, 2017.

33 USDA NRCS, 1986.

34 S^3 Wiehle South Design Storm Report, 2018.

35 WSSI, 2012.

100 year, 24 hour rainfall[36]	8.3 inches

Table 1: Urban Development with Mitigation Scenario Parameters

The TR-55 flow calculation for Wiehle South design discharge in the scenario was 84 cfs, a 16% decrease from 100 cfs flow rate S^3 calculated for Wiehle South in Report 2.[37] The S^3 design discharge average of 80 cfs can be scaled to **67 cfs** for the urbanization *with* mitigation scenario.

B. Urban Development without Stormwater Mitigation Scenario

Only a change in the curve number was used in the Urban Development without Stormwater Mitigation Scenario detailed in Table 2. The curve number increase affected rainfall runoff in the watershed. While a waiver releasing prospective developers from Fairfax County stormwater regulations is exceedingly unlikely, this scenario provides a basis of comparison for associated effects of possible urbanization within the watershed.

Impervious cover	31.5%
Curve number	73
1 year, 24 hour rainfall	2.62 inches
2 year, 24 hour rainfall	3.17 inches
100 year, 24 hour rainfall	8.3 inches

Table 2: Urban Development without Mitigation Scenario Parameters

The TR-55 flow calculation for Wiehle South design discharge in the scenario was 119 cfs, a 19% increase from 100 cfs flow rate S^3 calculated for Wiehle South in Report 2. The S^3 design discharge average of 80 cfs can be scaled to **95 cfs** for the urbanization *without* mitigation scenario.

C. Urban Development *with* Mitigation and Climate Change Scenario (2080)

Impervious cover	31.5%

[36] NOAA Atlas 14, 2017.

[37] See Appendix B for flow calculations.

Curve number	70
1 year, 24 hour rainfall[38]	3.08 inches
2 year, 24 hour rainfall[39]	3.72 inches
100 year, 24 hour rainfall[40]	10.24 inches

Table 3: Urban Development with Mitigation and Climate Change Scenario Parameters

The TR-55 flow calculation for Wiehle South design discharge in the Urban Development and Climate Change Scenario scenario was 123 cfs, a 23% increase from 100 cfs flow rate S^3 calculated in Report 2.[41] Curving the S^3 design discharge average of 80 cfs to **98 cfs**, compared to the 113 cfs WSSI calculated using reference curves for their 2012 design of the Wiehle South reach.[42]

Floodplain flow velocity and erosion resistance is dictated by the discharged produced by runoff from the 100 year storm. The TR-55 flow calculation for the 100 year storm discharge in the Climate Change Scenario was 1464 cfs, a 51% increase from the 968 cfs used by S^3 in Report 3 and 49% increase from the TR-55 calculation of 982 cfs for the contemporary 100 year, 24 hour rainfall intensity.[43]

D. Urban Development *without* Mitigation and Climate Change Scenario (2080)

S^3 used NEX-GDDP GCM precipitation trend projected to year 2080 to estimate change in rainfall for TR-55 flow rate calculations in the Wiehle South reach. The same curve number from the Urban Development without Stormwater Mitigation Scenario was used, as further development beyond that scenario's time horizon was expected to be limited. Since mitigation would be likely in the case of urban development, this combined development and climate change scenario can be considered a worst case or relatively extreme projection of hydrological

[38] An approximately 17.5% increase from current 2.62 inches using linear regression trend slope of aggregated 1 year storm GCM projection.

[39] An approximately 17.4% increase from current 3.17 inches using linear regression trend slope of aggregated 2 year storm GCM projection.

[40] An approximately 23.4% increase from current 8.3 inches using linear regression trend slope of aggregated maximum storms each decade (2020 to 2080) in GCM projection.

[41] S^3 Wiehle South Design Storm Discharge Calculation Report, 2018.

[42] Wetland Studies and Solutions, Inc. (WSS), 2012.

[43] S^3 Wiehle South Riffle Design Analysis Report, 2018.

conditions in the Wiehle South watershed. Table 3 outlines the conditions for the year 2080 scenario.

Impervious cover	31.5%
Curve number	73
1 year, 24 hour rainfall[44]	3.08 inches
2 year, 24 hour rainfall[45]	3.72 inches
100 year, 24 hour rainfall[46]	10.24 inches

Table 4: Urban Development without Mitigation and Climate Change Scenario Parameters

TR-55 flow calculation for Wiehle South design discharge in the Urban Development and Climate Change Scenario scenario was 174 cfs, a 74% increase from 100 cfs flow rate S^3 calculated in Report 2.[47] Curving the S^3 design discharge average of 80 cfs to **140 cfs**, compared to the 113 cfs WSSI calculated using reference curves for their 2012 design of the Wiehle South reach.[48]

Floodplain flow velocity and erosion resistance is dictated by the discharged produced by runoff from the 100 year storm. The TR-55 flow calculation for the 100 year storm discharge in the Climate Change Scenario was 1500 cfs, a 55% increase from the 968 cfs used by S^3 in Report 3 and 50% increase from the TR-55 calculation of 1012 cfs for the contemporary 100 year, 24 hour rainfall intensity.[49]

VI. Alternative Riffle Designs

The S^3 team created three alternative typical riffle cross sections that would be able to convey the flows predicted in Section V. Table 4 shows the geometric properties of these designs, which

[44] An approximately 17.5% increase from current 2.62 inches using linear regression trend slope of aggregated 1 year storm GCM projection.
[45] An approximately 17.4% increase from current 3.17 inches using linear regression trend slope of aggregated 2 year storm GCM projection.
[46] An approximately 23.4% increase from current 8.3 inches using linear regression trend slope of aggregated maximum storms each decade (2020 to 2080) in GCM projection.
[47] S^3 Wiehle South Design Storm Discharge Calculation Report, 2018.
[48] Wetland Studies and Solutions, Inc. (WSS), 2012.
[49] S^3 Wiehle South Riffle Design Analysis Report, 2018.

are compared with the original 2012 WSSI design and the S³ design proposed in Report 3. Due to the predicted combined effects of stormwater mitigation from future development in the Wiehle South watershed and increased storm runoff from climate change, S³ recommends maintaining the current S³ design as the elected channel design.

	Original WSSI	**Current S³ Design**	**Urban Developmen t *with Stormwater* Mitigation Design**	**Urban Development *without Stormwater* Mitigation Design**	**Urban Development *with Stormwater Mitigation* and Climate Change Design**	**Urban Development *without Stormwater Mitigation* and Climate Change Design**
Bankfull Discharge (cfs)	113	80	70	98	98	152
Bankfull Area (ft^2)	22.9	16.1	14.6	18.5	18.5	25.1
Bankfull Width (ft)	20	16.5	16	17.5	17.5	19.5
Maximum Depth (ft)	1.7	1.5	1.4	1.6	1.6	1.9
Bankfull Depth (ft)	1.1	1.0	0.9	1.1	1.1	1.3
Width to depth ratio (11 to 33 is allowable)	17.5	16.9	17.8	17.5	17.5	15.1
Side Slope	3.0	3.0	3.1	3.125	3.125	3.0
Bankfull Flow Velocity (ft/s)	5.97	5.05	4.81	5.31	5.31	6.05
Bankfull Flow Shear (lbs/ft^2)	1.188	.895	.834	.967	.967	1.174

Table 5: Wiehle South riffle channel design comparisons

The riffle designs taking into account the hydrological effects the prospective Urban Development without Mitigation and Climate Change Scenario are larger than the proposed S^3 design, while the Urban Development with *Stormwater* Mitigation Design is smaller. The most likely urban development change would be with runoff mitigation. This would allow for a 9% smaller cross-sectional area. The alternative designs would need to contain more discharge from increased runoff intensity within an acceptable flow velocity to prevent erosion to the stream channel. The cross-sectional area of the Urban Development without *Stormwater* Mitigation Design, Urban Development with Stormwater Mitigation and Climate Change, and Urban Development with Stormwater Mitigation and Climate Change channel designs are 14%, 14% and 56% larger, respectively, than the Current S^3 design.

VII. Riffle Design Performance

The S³ team used HEC-RAS to evaluate the flow performance of the Urban Development *without Stormwater Mitigation* and Climate Change scenario, the "worst case" scenario explored in this report.[50] S³ calculated flow outputs for both a bankfull event and a 100 year storm which extends into the floodplain, as described in Section V.-C. of this report. The HEC-RAS model informed the evaluation of riffle channel design performance in regards to floodplain flow velocity and in-channel shear stress as indicators of potential for erosion, explained for the current Wiehle South design in sections IV. and V. of the S³ third report.[51] The channel dimensions were scaled larger than the channels analyzed in the third report and the same Manning's n values were used: 0.035 for in-channel and 0.1 for floodplain. The same channel slope was used, 1.5% for a 100 foot representative stream reach.

A. 1 Year Storm: 140 cfs

As shown in the cross-section below, 140 cfs is a bankfull event for the Urban Development *without Stormwater Mitigation* and Climate Change channel design. The stream channel should be able to contain a 1 year storm of 140 cfs. The channel dimensions are 19.5 feet wide and 1.9 feet deep.

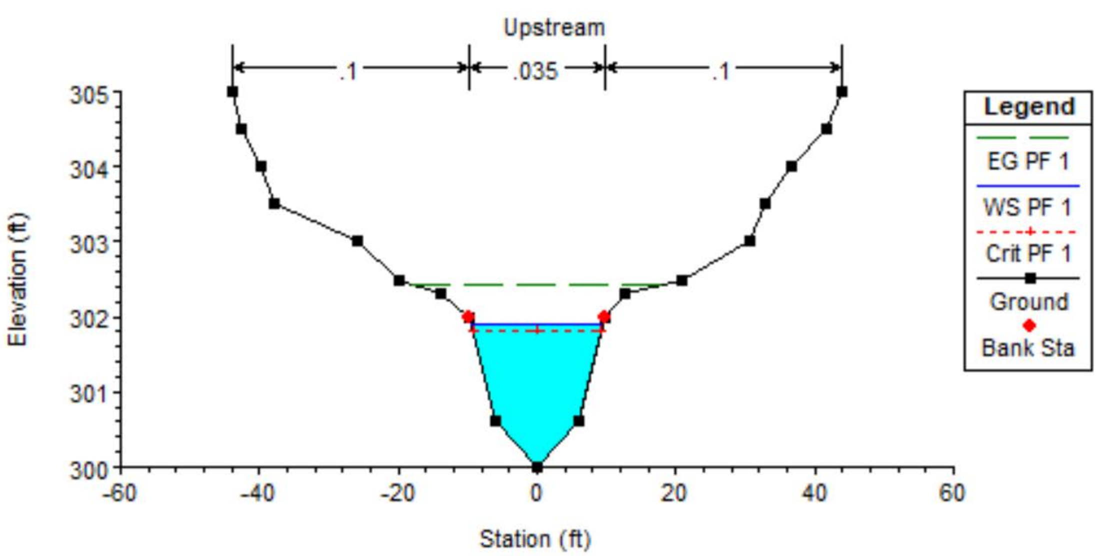

Figure 8: HEC-RAS cross section for 140 cfs flow

Table 5 (below) summarizes the flow output data from the HEC-RAS model. The shear stress value of 1.14 lb/ft² is similar to the 0.9 lb/ft² and 1.4 lb/ft² from the Shields and Rosgen methods,

[50] Version 5.0.3, U.S. Army Corps, 2016

[51] S³ Wiehle South Riffle Design Analysis Report, 2018.

respectively, calculated in Table 9 in Section V of Report 3.[52] This suggests that the Urban Development and Climate Change riffle channel design is adequate to contain the amplified flow without concerns for erosion and flooding.

	Flow (cfs)	Hydraulic Depth (feet)	Velocity (feet/sec)	Shear Stress (lb/ft^2)
Channel	140	1.25	5.92	1.14

Table 6: HEC-RAS 140 cfs output summary

B. 100 Year Storm: 1500 cfs

The S[3] team used HEC-RAS to analyze concerns for floodplain erosion explored in the S[3] Riffle Design Analysis report in Section IV.[53] The 100 year storm is the storm by which the floodplain is delineated. The stream channel and floodplain combined should be able to contain a 100 year storm of 1500 cfs. The Wiehle South watershed soil type means an upper flow velocity in the floodplain of 3 feet/second is required to prevent erosion. The floodplain width of 87.5 feet and 5 feet deep.

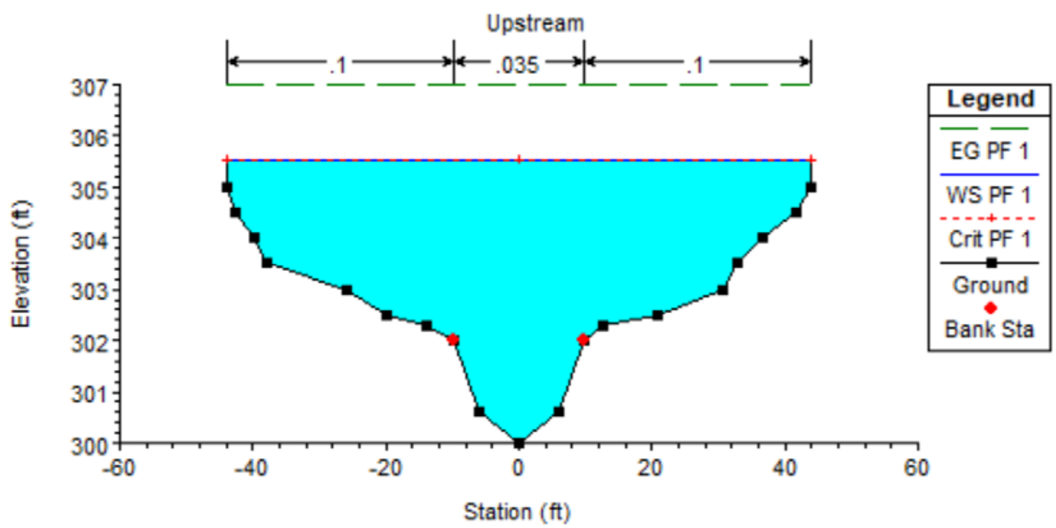

Figure 9: HEC-RAS cross section for 1500 cfs flow

The HEC-RAS outputs for the Urban Development *without Stormwater Mitigation* and Climate Change scenario channel design are summarized in Table 6 (below). The floodplain velocity approaches the 3 feet/sec upper limit explained in the third Wiehle South Report by S[3]. The velocity is still below the limit, so S[3] does not currently expect erosion in the floodplain. This velocity and the associated erosion should be re-evaluated periodically, as climate models are

[52] S[3] Wiehle South Riffle Design Analysis Report, 2018.

[53] Ibid.

subject to substantial uncertainty. The floodplain cross-sectional width would be 1.5 feet greater due to increased flood prone elevation resulting from the higher flow rates. Such a consideration extends beyond the scope of stream restoration and into floodplain delineation: the 100 year floodplain as designated by FEMA and constrained by land-ownership/easements in Reston would would need to be modified to contain larger 100 year flood magnitudes (see Figure 10).[54]

	Flow (cfs)	Hydraulic Depth (feet)	Velocity (feet/sec)	Shear Stress (lb/ft^2)
Channel	1076	4.8	11.4	2.7
Floodplains (x 2)	215 (x 2)	2.4	2.5	1.4

Table 7: HEC-RAS 1500 cfs output summary

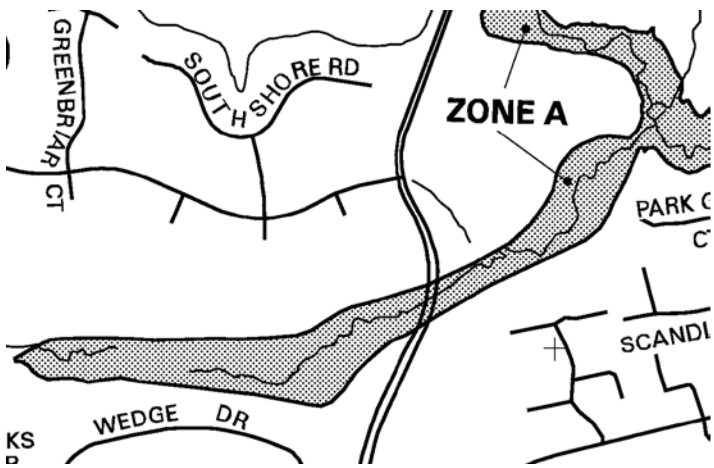

Figure 10: Current FEMA 100 year floodplain delineation for Wiehle South reach

VIII. Conclusion

The results from this analysis indicate that land use and climate changes could have a significant impact on the Wiehle South stream. S^3 predicts that future watershed conditions will necessitate a stream channel that could convey a discharge larger than the current baseline if engineering on a long-term time horizon. Enlarging the design would result in a more costly construction project.

The most probable and impactful land use change would be conversion of the Hidden Creek Country Club to residences and thus further urban development within the Wiehle South

[54] FEMA, 2010.

watershed. When S^3 envisioned a scenario in which golf course area is developed *with* runoff mitigation per Fairfax County regulations, the Wiehle South design discharge decreased to 67 cfs. The channel could be 0.5 feet narrower and 0.1 feet shallower at the maximum depth to convey this flow, resulting in a less costly construction project. Conversely, under scenario in which the entire golf course area is converted to medium density housing *without* runoff mitigation the design discharge increased from 80 cfs to 95 cfs. The channel would need to be 1.5 feet wider and 0.1 feet deeper at the maximum depth than the current S^3 design to convey this flow.

Climate change modeling was used to access changes to rainfall events. The model predicted increasing intensity for the design storm using climate projections through the year 2080. The model estimates that the one year storm intensity will increase from 2.62 to 3.08 inches. Increased rainfall runoff along with the aforementioned land use changes with proper stormwater mitigation resulted in a design discharge of 98 cfs. The channel designed for that scenario was 1 foot wider and 0.1 feet deeper at its maximum depth than the riffle proposed by S^3 in Report 3. Climate change and development without mitigation increased the discharge to 140 cfs. The channel would need to be 3 feet wider and 0.4 feet deeper at the maximum depth than the current S^3 design to convey this flow.

Impacts due to climate change are expected to include increased flow due to large storm events. The currently delineated 100 year floodplain would not be sufficient for these flows. According to year modeling for the year 2080 performed by S^3 the floodplain width would need to be increased by 1.5 ft.

The risk of failure due to land use changes should be taken into account since the stability of the Wiehle South channel design will be guaranteed for 10 years, and the stream should be stable in perpetuity. The effects of climate change could be significant, but have large uncertainty. There will likely be increased storm intensity in the future due to climate change. The projected increase in frequency of bankfull events to 2 per year would not be problematic, as discussed in Assignment 1. Many of the hydrological effects of increased runoff would be offset by development in the watershed which will incorporate stormwater mitigation. Due to this S^3 recommends that the current channel design as proposed in Assignment 3 be used for the Wiehle South stream restoration.

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IX. Appendix

A. Curve Number Calculations

Subwatershed	Land Use	HSG	Acres	Curve Numbers	Impervious Acres
40	Building	B	1.36	98	1.36
40	Forest	B	10.38	55	0
40	Open Space	B	2.6	61	0
40	Paved Parking Lot	B	1.32	98	1.32
40	Paved Road	B	1.56	98	1.56
40	Building	D	0.31	98	0.31
40	Forest	D	1.46	77	0
40	Openspace	D	0.94	80	0
40	Paved Parking Lot	D	0.14	98	0.14
40	Paved Road	D	0.02	98	0.02
41	Building	B	24.38	98	24.38
41	Forest	B	51.22	55	0
41	Open Space	B	43.7	61	0
41	Forest	B	64	55	19.2
41	Paved Parking Lot	B	29.14	98	29.14
41	Paved Road	B	9.81	98	9.81
41	Building	C	0.25	98	0.25
41	Forest	C	7.46	70	0
41	Open Space	C	22.69	74	0
41	Paved Parking Lot	C	0.18	98	0.18
41	Paved Road	C	0.24	98	0.24
41	Building	D	0.15	98	0.15
41	Forest	D	3.65	77	0
41	Opens Space	D	2.93	80	0
41	Paved Parking Lot	D	0.57	98	0.57
41	Paved Road	D	0.37	98	0.37
41	Pond	W	1.33	98	0
	Total/Weigfhted Avg:		**282.2**	69.5	89
				% Impervious:	32%

Wiehle South curve numbers and impervious cover with land use changes that follow current codes.

Subwatershed	Land Use	HSG	Acres	Curve Numbers	Impervious Acres
40	Building	B	1.36	98	1.36
40	Forest	B	10.38	55	0
40	Open Space	B	2.6	61	0
40	Paved Parking Lot	B	1.32	98	1.32
40	Paved Road	B	1.56	98	1.56
40	Building	D	0.31	98	0.31
40	Forest	D	1.46	77	0
40	Openspace	D	0.94	80	0
40	Paved Parking Lot	D	0.14	98	0.14
40	Paved Road	D	0.02	98	0.02
41	Building	B	24.38	98	24.38
41	Forest	B	51.22	55	0
41	Open Space	B	43.7	61	0
41	Residential Development	B	64	72	19.2
41	Paved Parking Lot	B	29.14	98	29.14
41	Paved Road	B	9.81	98	9.81
41	Building	C	0.25	98	0.25
41	Forest	C	7.46	70	0
41	Open Space	C	22.69	74	0
41	Paved Parking Lot	C	0.18	98	0.18
41	Paved Road	C	0.24	98	0.24
41	Building	D	0.15	98	0.15
41	Forest	D	3.65	77	0
41	Opens Space	D	2.93	80	0
41	Paved Parking Lot	D	0.57	98	0.57
41	Paved Road	D	0.37	98	0.37
41	Pond	W	1.33	98	0
	Total/Weigfhted Avg:		**282.16**	73.346222	89
				% Impervious:	32%

Wiehle South curve numbers and impervious cover with land use changes without runoff mitigation

B. TR-55 Flow Rate Calculations

Urban Development with Stormwater Mitigation Scenario

TR-55 Graphical Peak Discharge Method D-4	*Value*	*Unit*	*Source*
Runoff curve number CN	69.5		WSS Plan
Runoff Q	0.5	in	TR-55 table 2-1
Drainage area Am	0.44	mi^2	WSS Plan

24hr rainfall P	2.62	in	FC PFM
Time of concentration Tc	0.62	hr	
Travel time through area Tt	0.62	hr	
Pond/swamp adjustment factor Fp	1		
Unit peak discharge (qu)=	380	csm/in	TR55 chart 4-II
Ia	0.875	in	TR-55 table 4-1
Ia/P	0.3339694656		
Peak discharge (qp)=(qu)(Am)(Q)(Fp)	**83.6**	**ft^3/s**	TR-55 eq 4-1
Values for Tc and Tt			
<u>Sheet flow</u>			
Manning roughness n	0.1		
Flow length L	100	ft	
2 yr 24hr rainfall P2	3.17	in	
Land slope s	0.08	ft/ft	TR 55 eq 3-3
Tt 1	0.0681290627	hr	(0.007(nL)^0.8)/((P)^0.5)(s^0.4)
<u>Shallow concentrated flow</u>			
Flow length	1900	ft	
Watercourse slope (slope between max and outlet elev)	0.03166666667	ft/ft	
Average velocity	3	ft/s	TR55 figure 3-1
Tt 2	0.1759259259	hr	(L)/(3600(V))
<u>Channel flow</u>			
Cross sec flow area a	3.2	ft2	Contour map
Wetted perimeter pw	5.6	ft2	
Hydraulic radius r=a/pw	0.57	ft2	

Channel slope s (Valley Slope/Sinuosity)	0.017	ft/ft	
Manning roughness n	0.035		VA Erosion Sediment Control Manual Ch. 5 V-65
V = [(1.49r^(2/3)s^(1/2)]/n	3.817291617	ft/s	
Flow length L	5200	ft	[see TR55 3-1]
Tt 3	0.3783951003	hr	
TOTAL Tc	0.6224500889	hr	
	37.34700534	mins	

Urban Development without Stormwater Mitigation Scenario

TR-55 Graphical Peak Discharge Method D-4	*Value*	*Unit*	*Source*
Runoff curve number CN	73.5		WSS Plan
Runoff Q	0.65	in	TR-55 table 2-1
Drainage area Am	0.44	mi^2	WSS Plan
24hr rainfall P	2.62	in	FC PFM
Time of concentration Tc	0.66	hr	
Travel time through area Tt	0.66	hr	
Pond/swamp adjustment factor Fp	1		
Unit peak discharge (qu)=	415	csm/in	TR55 chart 4-II
Ia	0.7215	in	TR-55 table 4-1
Ia/P	0.2753816794		
Peak discharge (qp)=(qu)(Am)(Q)(Fp)	**118.69**	**ft^3/s**	TR-55 eq 4-1
Values for Tc and Tt			
Sheet flow			
Manning roughness n	0.1		

Flow length L	100	ft	
2 yr 24hr rainfall P2	3.17	in	
Land slope s	0.08	ft/ft	TR 55 eq 3-3
Tt 1	0.0681290627	hr	(0.007(nL)^0.8)/((P)^0.5)(s^0.4)
<u>Shallow concentrated flow</u>			
Flow length	1900	ft	
Watercourse slope (slope between max and outlet elev)	0.03166666667	ft/ft	
Average velocity	3	ft/s	TR55 figure 3-1
Tt 2	0.1759259259	hr	(L)/(3600(V))
<u>Channel flow</u>			
Cross sec flow area a	3.2	ft2	Contour map
Wetted perimeter pw	5.6	ft2	
Hydraulic radius r=a/pw	0.57	ft2	
Channel slope s (Valley Slope/Sinuosity)	0.017	ft/ft	
Manning roughness n	0.035		VA Erosion Sediment Control Manual Ch. 5 V-65
V = [(1.49r^(2/3)s^(1/2)]/n	3.817291617	ft/s	
Flow length L	5200	ft	[see TR55 3-1]
Tt 3	0.3783951003	hr	
TOTAL Tc	0.6224500889	hr	
	37.34700534	mins	

Urban Development with Stormwater Mitigation and Climate Change (2080) Scenario

TR-55 Graphical Peak Discharge Method D-4	*Value*	*Unit*	*Source*
Runoff curve number CN	69.5		WSS Plan
Runoff Q	0.68	in	TR-55 table 2-1
Drainage area Am	0.44	mi^2	WSS Plan
24hr rainfall P	3.08	in	FC PFM
Time of concentration Tc	0.62	hr	
Travel time through area Tt	0.62	hr	
Pond/swamp adjustment factor Fp	1		
Unit peak discharge (qu)=	410	csm/in	TR55 chart 4-II
Ia	0.88	in	TR-55 table 4-1
Ia/P	0.2857142857		
Peak discharge (qp)=(qu)(Am)(Q)(Fp)	**122.672**	**ft^3/s**	TR-55 eq 4-1
Values for Tc and Tt			
Sheet flow			
Manning roughness n	0.1		
Flow length L	100	ft	
2 yr 24hr rainfall P2	3.72	in	
Land slope s	0.08	ft/ft	TR 55 eq 3-3
Tt 1	0.06289130048	hr	(0.007(nL)^0.8)/((P)^0.5)(s^0.4)
Shallow concentrated flow			
Flow length	1900	ft	
Watercourse slope (slope between max and outlet elev)	0.03166666667	ft/ft	
Average velocity	3	ft/s	TR55 figure 3-1
Tt 2	0.1759259259	hr	(L)/(3600(V))

Channel flow			
Cross sec flow area a	3.2	ft2	Contour map
Wetted perimeter pw	5.6	ft2	
Hydraulic radius r=a/pw	0.57	ft2	
Channel slope s (Valley Slope/Sinuosity)	0.017	ft/ft	
Manning roughness n	0.035		VA Erosion Sediment Control Manual Ch. 5 V-65
V = [(1.49r^(2/3)s^(1/2)]/n	3.817291617	ft/s	
Flow length L	5200	ft	[see TR55 3-1]
Tt 3	0.3783951003	hr	
TOTAL Tc	0.6172123267	hr	
	37.0327396	mins	

Urban Development without Stormwater Mitigation and Climate Change (2080) Scenario

TR-55 Graphical Peak Discharge Method D-4	*Value*	*Unit*	*Source*
Runoff curve number CN	73.5		WSS Plan
Runoff Q	0.9	in	TR-55 table 2-1
Drainage area Am	0.44	mi^2	WSS Plan
24hr rainfall P	3.08	in	FC PFM
Time of concentration Tc	0.62	hr	
Travel time through area Tt	0.62	hr	
Pond/swamp adjustment factor Fp	1		
Unit peak discharge (qu)=	440	csm/in	TR55 chart 4-II
Ia	0.7215	in	TR-55 table 4-1
Ia/P	0.2342532468		
Peak discharge (qp)=(qu)(Am)(Q)(Fp)	**174.24**	**ft^3/s**	TR-55 eq 4-1

Values for Tc and Tt			
Sheet flow			
Manning roughness n	0.1		
Flow length L	100	ft	
2 yr 24hr rainfall P2	3.72	in	
Land slope s	0.08	ft/ft	TR 55 eq 3-3
Tt 1	0.06289130048	hr	(0.007(nL)^0.8)/((P)^0.5)(s^0.4)
Shallow concentrated flow			
Flow length	1900	ft	
Watercourse slope (slope between max and outlet elev)	0.03166666667	ft/ft	
Average velocity	3	ft/s	TR55 figure 3-1
Tt 2	0.1759259259	hr	(L)/(3600(V))
Channel flow			
Cross sec flow area a	3.2	ft2	Contour map
Wetted perimeter pw	5.6	ft2	
Hydraulic radius r=a/pw	0.57	ft2	
Channel slope s (Valley Slope/Sinuosity)	0.017	ft/ft	
Manning roughness n	0.035		VA Erosion Sediment Control Manual Ch. 5 V-65
V = [(1.49r^(2/3)s^(1/2)]/n	3.817291617	ft/s	
Flow length L	5200	ft	[see TR55 3-1]
Tt 3	0.3783951003	hr	
TOTAL Tc	0.6172123267	hr	

	37.0327396	mins	

100 Year Storm (2080 Projection with Stormwater Mitigation)

TR-55 Graphical Peak Discharge Method D-4	*Value*	*Unit*	*Source*
Runoff curve number CN	69.5		WSS Plan
Runoff Q	6.4	in	TR-55 table 2-1
Drainage area Am	0.44	mi^2	WSS Plan
24hr rainfall P	10.24	in	FC PFM
Time of concentration Tc	0.59	hr	
Travel time through area Tt	0.59	hr	
Pond/swamp adjustment factor Fp	1		
Unit peak discharge (qu)=	520	csm/in	TR55 chart 4-II
Ia	0.88	in	TR-55 table 4-1
Ia/P	0.0859375		
Peak discharge (qp)=(qu)(Am)(Q)(Fp)	**1464.32**	**ft^3/s**	TR-55 eq 4-1

100 Year Storm (2080 Projection without Stormwater Mitigation)

TR-55 Graphical Peak Discharge Method D-4	*Value*	*Unit*	*Source*
Runoff curve number CN	73.5		WSS Plan
Runoff Q	6.5	in	TR-55 table 2-1
Drainage area Am	0.44	mi^2	WSS Plan
24hr rainfall P	10.24	in	FC PFM
Time of concentration Tc	0.59	hr	
Travel time through area Tt	0.59	hr	
Pond/swamp adjustment factor Fp	1		
Unit peak discharge (qu)=	525	csm/in	TR55 chart 4-II
Ia	0.7215	in	TR-55 table 4-1

Ia/P	0.07045898438		
Peak discharge (qp)=(qu)(Am)(Q)(Fp)	**1501.5**	**ft^3/s**	TR-55 eq 4-1
Values for Tc and Tt			
Sheet flow			
Manning roughness n	0.1		
Flow length L	100	ft	
2 yr 24hr rainfall P2	10.24	in	
Land slope s	0.08	ft/ft	TR 55 eq 3-3
Tt 1	0.03790635861	hr	(0.007(nL)^0.8)/((P)^0.5)(s^0.4)
Shallow concentrated flow			
Flow length	1900	ft	
Watercourse slope (slope between max and outlet elev)	0.03166666667	ft/ft	
Average velocity	3	ft/s	TR55 figure 3-1
Tt 2	0.1759259259	hr	(L)/(3600(V))
Channel flow			
Cross sec flow area a	3.2	ft2	Contour map
Wetted perimeter pw	5.6	ft2	
Hydraulic radius r=a/pw	0.57	ft2	
Channel slope s (Valley Slope/Sinuosity)	0.017	ft/ft	
Manning roughness n	0.035		VA Erosion Sediment Control Manual Ch. 5 V-65
V = [(1.49r^(2/3)s^(1/2)]/n	3.817291617	ft/s	
Flow length L	5200	ft	[see TR55 3-1]

Tt 3	0.3783951003	hr	
TOTAL Tc	0.5922273848	hr	
	35.53364309	mins	

C. Channel Cross-Section Analysis

Mannings Equation Analysis

version 17, Sep 2016

Wiehle South 80 CFS, STA 13.5

cross-section data: preview

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

calculate rating table

unit system: en

auto scale: yes

station	elevation	n-value
-8	1.4	0.035
-5.25	0.5	
0	0	
5.25	0.5	
8	1.4	

hydraulic properties for given water surface elevation:

w.s. elev	flow area	wetted P	hydr. radius	top width	hydr. depth	n value	darcy-weis. f	conveyance	discharge	velocity	shear

analyze single condition (w.s.elev or discharge)

profile slope= 0.01500

bank stations: left -8.0 right 8.0

hydraulic properties rating table:

w.s. elev	flow area	wetted P	hydr. radius	top width	hydr. depth	n value	darcy-weis. f	conveyance	discharge	velocity	shear
1.40	14.6	16.3	0.89	16.0	0.91	0.035	0.1485	571.9	70.0	4.81	0.834
1.00	8.64	13.8	0.63	13.6	0.64	0.035	0.1669	268.9	32.9	3.81	0.588
0.50	2.63	10.5	0.25	10.5	0.25	0.035	0.2272	44.1	5.40	2.06	0.233
0.00	0.00	0.0	0.00	0.0	0.00	0.035	0.1669	0.00	0.00	0.00	0.000

Cross section design calculations for urban development with mitigation scenario

Mannings Equation Analysis

version 17, Sep 2016

Wiehle South 80 CFS, STA 13.5

cross-section data: preview

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

calculate rating table

unit system: en

auto scale: yes

station	elevation	n-value
-43	4.8	0.1
-42	4.3	
-39	3.8	
-37	3.3	
-25	2.8	
-19	2.3	
-13	1.8	
-9	1.6	0.035
-5.25	0.5	
0	0	
5.25	0.5	
9	1.6	
12	1.8	0.1
20	2.3	
30	2.8	
32	3.3	
36	3.8	
41	4.3	
43	4.8	

hydraulic properties for given water surface elevation:

w.s. elev	flow area	wetted P	hydr. radius	top width	hydr. depth	n value	darcy-weis. f	conveyance	discharge	velocity	shear
1.60	**18.3**	**18.4**	**1.00**	**18.0**	**1.02**	**0.035**	**0.1431**	**775.2**	**94.9**	**5.19**	**0.933**

analyze single condition (w.s.elev or discharge)

profile slope= 0.01500

bank stations: left -9.0 right 9.0

hydraulic properties rating table:

w.s. elev	flow area	wetted P	hydr. radius	top width	hydr. depth	n value	darcy-weis. f	conveyance	discharge	velocity	shear
4.80	210.9	86.9	2.43	86.0	2.45	0.089	0.2097	11515	1410	6.69	2.272
4.50	185.3	84.9	2.18	84.2	2.20	0.088	0.1992	9844	1206	6.51	2.042
4.00	144.4	78.8	1.83	78.2	1.85	0.087	0.1796	7403	906.7	6.28	1.715
3.50	107.1	72.0	1.49	71.4	1.50	0.086	0.1543	5342	654.2	6.11	1.394
3.00	73.7	61.1	1.21	60.6	1.22	0.083	0.1265	3651	447.1	6.07	1.129
2.50	47.0	45.8	1.03	45.4	1.04	0.077	0.1063	2347	287.4	6.11	0.961
2.00	28.2	31.0	0.91	30.6	0.92	0.063	0.0998	1364	167.0	5.93	0.851
1.50	16.5	17.7	0.94	17.3	0.95	0.035	0.1461	672.0	82.3	4.98	0.877
1.00	8.73	14.1	0.62	13.9	0.63	0.035	0.1677	269.1	33.0	3.78	0.579
0.50	2.63	10.5	0.25	10.5	0.25	0.035	0.2272	44.1	5.40	2.06	0.233
0.00	0.0	0.0	0.00	0.0	0.00	0.035	0.1677	0.00	0.00	0.00	0.000

Cross section design calculations for urban development without mitigation scenario.

Mannings Equation Analysis

version 17, Sep 2016

Wiehle South 80 CFS, STA 13.5

cross-section data: preview

station	elevation	n-value
-8.75	1.6	0.035
-5	0.4	
0	0	
5	0.4	
8.75	1.6	

w.s. elev	flow area	wetted P	hydr. radius	top width	hydr. depth	n value	darcy-weis. f	conveyance	discharge	velocity	shear

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

calculate rating table

unit system: en

auto scale: yes

hydraulic properties for given water surface elevation:

w.s. elev	flow area	wetted P	hydr. radius	top width	hydr. depth	n value	darcy-weis. f	conveyance	discharge	velocity	shear

analyze single condition (w.s.elev or discharge)

profile slope= 0.01500

bank stations: left -8.8 right 8.8

hydraulic properties rating table:

w.s. elev	flow area	wetted P	hydr. radius	top width	hydr. depth	n value	darcy-weis. f	conveyance	discharge	velocity	shear
1.60	18.5	17.9	1.03	17.5	1.06	0.035	0.1414	802.7	98.3	5.31	0.967
1.50	16.8	17.3	0.97	16.9	0.99	0.035	0.1442	699.5	85.7	5.11	0.911
1.00	9.13	14.0	0.65	13.8	0.66	0.035	0.1647	291.7	35.7	3.91	0.611
0.50	3.03	10.7	0.28	10.6	0.29	0.035	0.2175	55.6	6.80	2.24	0.265
0.00	0.0	0.0	0.00	0.0	0.00	0.035	0.1647	0.00	0.00	0.00	0.000

Cross Section design calculations for urban development with mitigation and climate change scenario.

Mannings Equation Analysis

version 17, Sep 2016

Wiehle South 80 CFS, STA 13.5

cross-section data: preview

station	elevation	n-value
-9.75	2	0.035
-5.25	0.5	
0	0	
5.25	0.5	
9.75	2	

w.s. elev	flow area	wetted P	hydr. radius	top width	hydr. depth	n value	darcy-weis. f	conveyance	discharge	velocity	shear

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

calculate rating table

unit system: en

auto scale: yes

hydraulic properties for given water surface elevation:

w.s. elev	flow area	wetted P	hydr. radius	top width	hydr. depth	n value	darcy-weis. f	conveyance	discharge	velocity	shear

analyze single condition (w.s.elev or discharge)

profile slope= 0.01500

bank stations: left -9.8 right 9.8

hydraulic properties rating table:

w.s. elev	flow area	wetted P	hydr. radius	top width	hydr. depth	n value	darcy-weis. f	conveyance	discharge	velocity	shear
2.00	25.1	20.0	1.25	19.5	1.29	0.035	0.1325	1241	151.9	6.05	1.174
1.50	16.1	16.9	0.96	16.5	0.98	0.035	0.1451	664.3	81.4	5.05	0.895
1.00	8.63	13.7	0.63	13.5	0.64	0.035	0.1668	268.9	32.9	3.82	0.589
0.50	2.63	10.5	0.25	10.5	0.25	0.035	0.2272	44.1	5.40	2.06	0.233
0.00	0.0	0.0	0.00	0.0	0.00	0.035	0.1668	0.00	0.00	0.00	0.000

Cross Section design calculations for urban development without mitigation and climate change scenario.

D. Climate Model Interpretation R Code

```r
# Goal: Future projections of precipitation (and temperature) to influence stream discharge and ultimately stream restoration design.
# 1 year storm used for channel design. 100 year storm used for floodplain analysis.


# Discharge (now). 1 year = 80 cfs. 100 year = 968 cfs.
# Flow in cfs 80*(24*60*60)*0.0283 = 195609.6 m^3/day
# Area in km^2 1.14121*1000000 = 1141210 m^2
# Bankfull discharge depth 195609.6/1141210 = 0.1714054


# Flow in cfs 968*(24*60*60)*0.0283 = 2366876 m^3/day
# Area in km^2 1.14121*1000000 = 1141210 m^2
# 100 yr discharge depth = 2.074006


# Precip (now). 1 year, 24 hr = 2.62 in. 100 year, 24 hr = 8.3 in
# Precip (mm/day). 1 year: 66.548. 100 year: 210.82


# NASA NEX GDDP Data
# RCP 4.5


# Temp : kelvin  (1.8*(K - 273.15) + 32) = F
# Precip : KG/M^2*S P*86400 = mm/day


noaa(
  datasetid = "PRECIP_HLY",
  locationid = "ZIP:28801",
  datatypeid = "HPCP",
  limit = 5,
  token = "YOUR_TOKEN"
)




Pout2 <-
  ncdc(
    datasetid = 'GHCND',
    stationid = 'GHCND:US1VAFX0001',
    datatypeid = 'PRCP',
    startdate = '2017-01-01',
    enddate = '2017-12-31',
    limit = 365,
    token = "ykyiJzUNRbUtLjCmbxZEKjOljMGUtToH"
  )
P2017 <- Pout2$data
```

```
plot(P2017$value)
summary(P2017$value)
sum(P2017$value)


Pout1 <-
  ncdc(
    datasetid = 'GHCND',
    stationid = 'GHCND:USC00448737',
    datatypeid = 'PRCP',
    startdate = '1925-04-01',
    enddate = '2018-03-31',
    limit = 29037,
    token = "ykyiJzUNRbUtLjCmbxZEKjOljMGUtToH"
  )


Tout1 <-
  ncdc(
    datasetid = 'GHCND',
    stationid = 'GHCND:USC00448737',
    datatypeid = 'TMAX',
    startdate = '1925-04-01',
    enddate = '2018-03-31',
    limit = 29037,
    token = "ykyiJzUNRbUtLjCmbxZEKjOljMGUtToH"
  )


USC00448737


monitors <- c("US1VAFX0001", "USC00448737")
obs <- meteo_pull_monitors(monitors)
obs_covr <- meteo_coverage(obs)


load(EcoHydRology)


restonNEX_GDDPprojections$Prmm = 0
restonNEX_GDDPprojections$Tminc = 0
restonNEX_GDDPprojections$Tmaxc = 0


#restonNEX_GDDPprojections <- read.csv("restonNEX-GDDPprojections.csv")


#P = "pr"


# for (i in 1:nrow(restonNEX_GDDPprojections))
```

```
# {
# if (restonNEX_GDDPprojections$Variable[i] == "pr")
#   {
#   restonNEX_GDDPprojections$Prmm[i] = restonNEX_GDDPprojections$Value[i]*86400
#   }
# else if (restonNEX_GDDPprojections$Variable[i] == "tasmin")
#   {
#   restonNEX_GDDPprojections$Tminc[i] = ((1.8*(restonNEX_GDDPprojections$Value[i]-273.15))+32)
#   restonNEX_GDDPprojections$Tmaxc[i] = 0
#   restonNEX_GDDPprojections$Prmm[i] = 0
#   }
# else (restonNEX_GDDPprojections$Variable[i] == "tasmax")
#   {
#     restonNEX_GDDPprojections$Tmaxc[i] = ((1.8*(restonNEX_GDDPprojections$Value[i]-273.15))+32)
#     restonNEX_GDDPprojections$Tminc[i] = 0
#     restonNEX_GDDPprojections$Prmm[i] = 0
#   }
# }


restonNEX_GDDPprecip <- read.csv("wiehleprecipprojections.csv")
restonNEX_GDDPprecip$Date <-
  as.Date(restonNEX_GDDPprecip$Date, format = "%m/%d/%Y")


restonNEX_GDDPprecip$Prmm = 0
restonNEX_GDDPprecip$BFPlus = 0
for (i in 1:nrow(restonNEX_GDDPprecip))
{
  restonNEX_GDDPprecip$Prmm[i] = restonNEX_GDDPprecip$Value[i] * 86400

  if (restonNEX_GDDPprecip$Prmm[i] > 66.548)
  {
    restonNEX_GDDPprecip$BFPlus[i] = 1
  }

}
summary(restonNEX_GDDPprecip$Prmm)
sum(restonNEX_GDDPprecip$Prmm)


restonNEX_GDDPprecip$BFPlus <-
  as.numeric(as.character(restonNEX_GDDPprecip$BFPlus))


sum(restonNEX_GDDPprecip$BFPlus)
plot(
  restonNEX_GDDPprecip$Date,
  restonNEX_GDDPprecip$BFPlus,
  type = "h",
  main = "Frequency of Bankfull Events RCP 4.5",
  xlab = "Year",
```

```
  ylab = "Occurence"
)


D <- ts(restonNEX_GDDPprecip$BFPlus, frequency = 3650)
Dec = 1:8
Dsum <- aggregate(D, FUN = sum)
plot(
  Dsum,
  xlab = "Decade",
  ylab = "Relative Bankfull Frequency",
  axes = FALSE,
  frame.plot = TRUE,
  main = "Frequency of Bankfull Events RCP 4.5"
)
Axis(side = 1, labels = TRUE)
polygon(c(Dec, rev(Dec)),
        c(as.vector(Dsum), rep(0.001, length(Dsum))), col = "grey")
abline(lm(formula = Dsum ~ Dec), col = "red")


Prin <- (0.0393701 * restonNEX_GDDPprecip$Prmm)
summary(Prin)
plot(
  restonNEX_GDDPprecip$Date,
  Prin,
  type = "l",
  xlab = "Date",
  ylab = "Precipitation (in)",
  main = "Projected Precipitation RCP 4.5"
)
abline(lm(formula = Prin ~ restonNEX_GDDPprecip$Date), col = "red")
plot(
  restonNEX_GDDPprecip$Date[(79 * 365):(80 * 365)],
  Prin[(79 * 365):(80 * 365)],
  type = "l",
  xlab = "Date",
  ylab = "Precipitation (in)",
  main = "Projected Precipitation RCP 4.5"
)
lines(restonNEX_GDDPprecip$Date[1:365],
      Prin[1:365],
      col = "red",
      type = "l")


lm(restonNEX_GDDPprecip$Prmm ~ restonNEX_GDDPprecip$Date)


restonNEX_GDDPprecip$Year <-
  format(as.Date(restonNEX_GDDPprecip$Date, format = "%d/%m/%Y"), "%Y")
```

```r
Y <- ts(restonNEX_GDDPprecip$Prmm, frequency = 365)
Yr = 2018:2097
Ymax <- aggregate(Y, FUN = max)
Ymaxin <- Ymax * 0.0393701
summary(Ymaxin)


Y2 <- ts(restonNEX_GDDPprecip$Prmm, frequency = (365 * 2))
Yr = 2018:2097
Y2max <- aggregate(Y2, FUN = max)
Y2maxin <- Y2max * 0.0393701
summary(Y2maxin)
# Use rainfall intensity for flow calcs


plot(
  Yr,
  Ymaxin,
  type = "l",
  xlab = "Year",
  ylab = "1 Year Storm Intensity (in/day)",
  main = "Projected Precipitation RCP 4.5"
)
polygon(c(Yr, rev(Yr)),
        c(as.vector(Ymaxin), rep(0.001, length(Ymaxin))), col = "light blue")
abline(lm(formula = Ymaxin ~ Yr), col = "red")
plot(lm(Y ~ restonNEX_GDDPprecip$Date))
# R squared / R value


#Min temp c 4.5
Tmin <- read.csv("Tmin.csv")
Tmin$Date <- as.Date(Tmin$Date, format = "%m/%d/%Y")
Tmin$Tmin <- ((1.8 * (Tmin$Value - 273.15)) + 32)


#Max temp c 4.5
Tmax <- read.csv("Tmax.csv")
Tmax$Date <- as.Date(Tmax$Date, format = "%m/%d/%Y")
Tmax$Tmax <- ((1.8 * (Tmax$Value - 273.15)) + 32)


summary(Tmax$Tmax)
plot(
  Tmax$Date,
  Tmax$Tmax,
  type = "l",
  xlab = "Year",
  ylab = "Daily Max Temperature (F)",
  main = "Projected Max Temperature RCP 4.5"
)
```

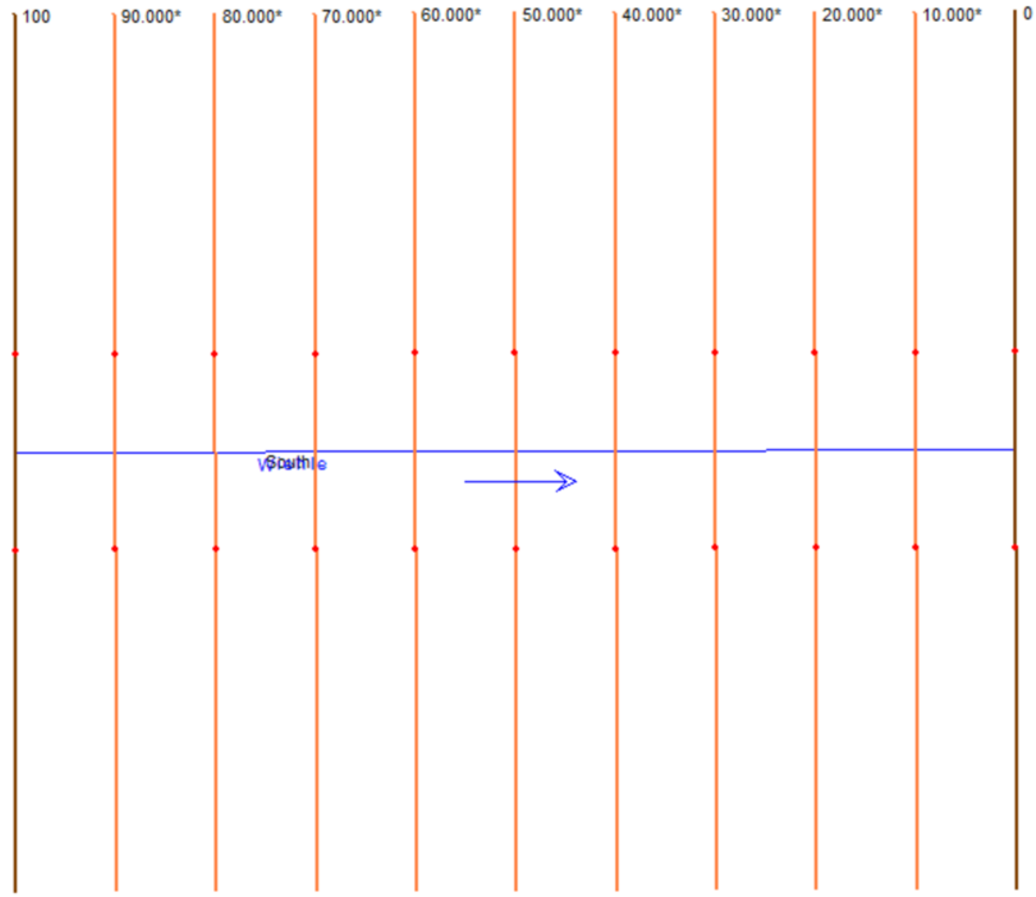
```
abline(lm(formula = Tmax$Tmax ~ Tmax$Date), col = "red")


plot(
  Tmax8$Date[(60 * 365):(61 * 365)],
  Tmax8$Tmax[(60 * 365):(61 * 365)],
  type = "l",
  xlab = "Year",
  ylab = "Daily Max Temperature (F)",
  main = "Projected Max Temperature RCP 4.5"
)[(79 * 365):(80 * 365)]
lines(Tmax8$Date[(5 * 365):(6 * 365)], Tmax8$Tmax[(5 * 365):(6 * 365)], col =
        "grey", type = "l")


# VSA45 <- Lumped_VSA_model(restonNEX_GDDPprecip$Date, restonNEX_GDDPprecip$Prmm, Tmax$Tmax, Tmin$Tmin, Depth = NULL,
SATper = NULL, AWCper = NULL,
#                 percentImpervious = 0, no_wet_class = 10, Tp = 5, latitudeDegrees = 38.96, albedo = 0.23,
#                 StartCond = "avg", PETin = NULL, AWC = Depth * AWCper, SAT = Depth * SATper, SW1 = NULL,
#                 BF1 = 1, PETcap = 5, rec_coef = 0.1, Se_min = 78, C1 = 3.1, Ia_coef = 0.05,
#                 PreviousOutput = NULL, runoff_breakdown = RunoffBreakdown(Tp, HrPrcDelay = (Tp/2 - 4)))


# VSA85 <- Lumped_VSA_model(dateSeries = restonNEX_GDDPprecip$Date,      P = restonNEX_GDDPprecip$Prmm,
percentImpervious = 27,
#                  Tmax = Tmax8$TmaxC, Tmin = Tmin8$TminC, latitudeDegrees=38.96, Tp = 5.8, Depth = 2010,
#                  SATper = 0.27, AWCper = 0.13, StartCond = "avg")


summary(VSA85$modeled_flow)
plot(
  VSA85$Date,
  VSA85$modeled_flow,
  type = "l",
  ylab = "Flow Depth (mm/day)",
  xlab = "Date",
  main = "Modeled Flow"
)
abline(lm(formula = VSA85$modeled_flow ~ VSA85$Date), col = "red")


plot(
  VSA85$Date,
  VSA85$SoilWater,
  type = "l",
  ylab = "Soil Water Content (mm/day)",
  xlab = "Date",
  main = "Modeled Soil Water Content"
)
abline(lm(formula = VSA85$SoilWater ~ VSA85$Date), col = "red")
```

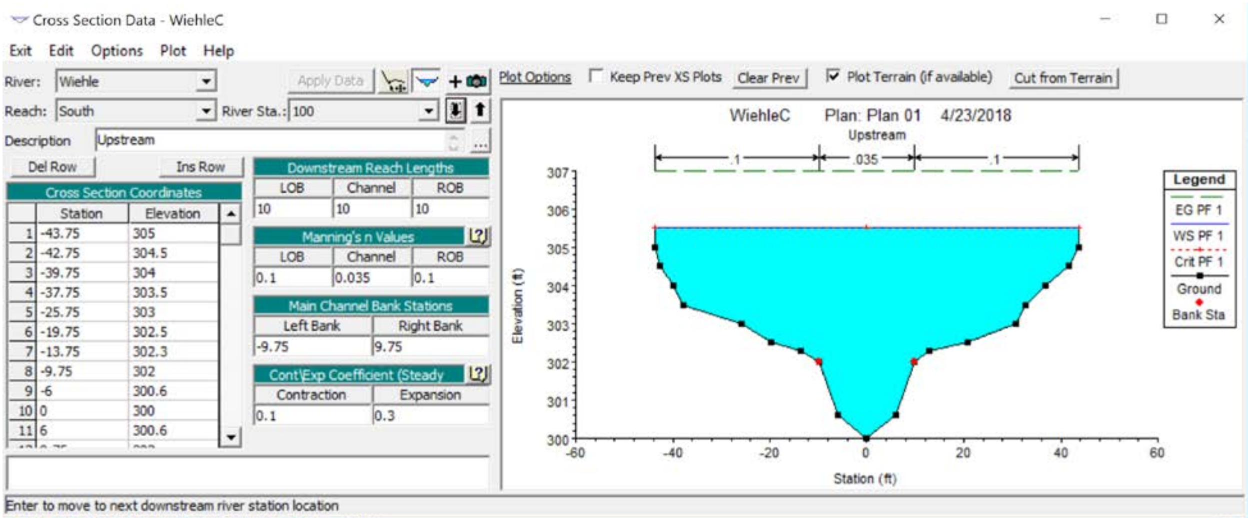
```r
plot(
  VSA85$Date,
  VSA85$Se,
  type = "l",
  ylab = "Groundwater Storage (mm/day)",
  xlab = "Date",
  main = "Modeled Groundwater Storage"
)
abline(lm(formula = VSA85$Se ~ VSA85$Date), col = "red")


plot(
  VSA85$Date,
  VSA85$impervRunoff,
  type = "l",
  ylab = "Runoff (mm/day)",
  xlab = "Date",
  main = "Impervious Runoff"
)
abline(lm(formula = VSA85$impervRunoff ~ VSA85$Date), col = "red")


# Convert to cfs
VSA85$modeled_flowCFS <-
  (((VSA85$modeled_flow / 1000) * 1141210) / (24 * 60 * 60)) * 35.3147
summary(VSA85$modeled_flowCFS)
plot(VSA85$Date, VSA85$modeled_flowCFS, type = "l")




# NASA NEX GDDP Data
# RCP 8.5


restonNEX_GDDPprecip8 <- read.csv("wiehleprecipprojections85.csv")
restonNEX_GDDPprecip8$Date <-
  as.Date(restonNEX_GDDPprecip8$Date, format = "%m/%d/%Y")


for (i in 1:nrow(restonNEX_GDDPprecip8))
{
  restonNEX_GDDPprecip8$Prmm[i] = restonNEX_GDDPprecip8$Value[i] * 86400

  if (restonNEX_GDDPprecip8$Prmm[i] > 66.548)
  {
    restonNEX_GDDPprecip8$BFPlus[i] = 1
  }
}


restonNEX_GDDPprecip$BFPlus <-
```

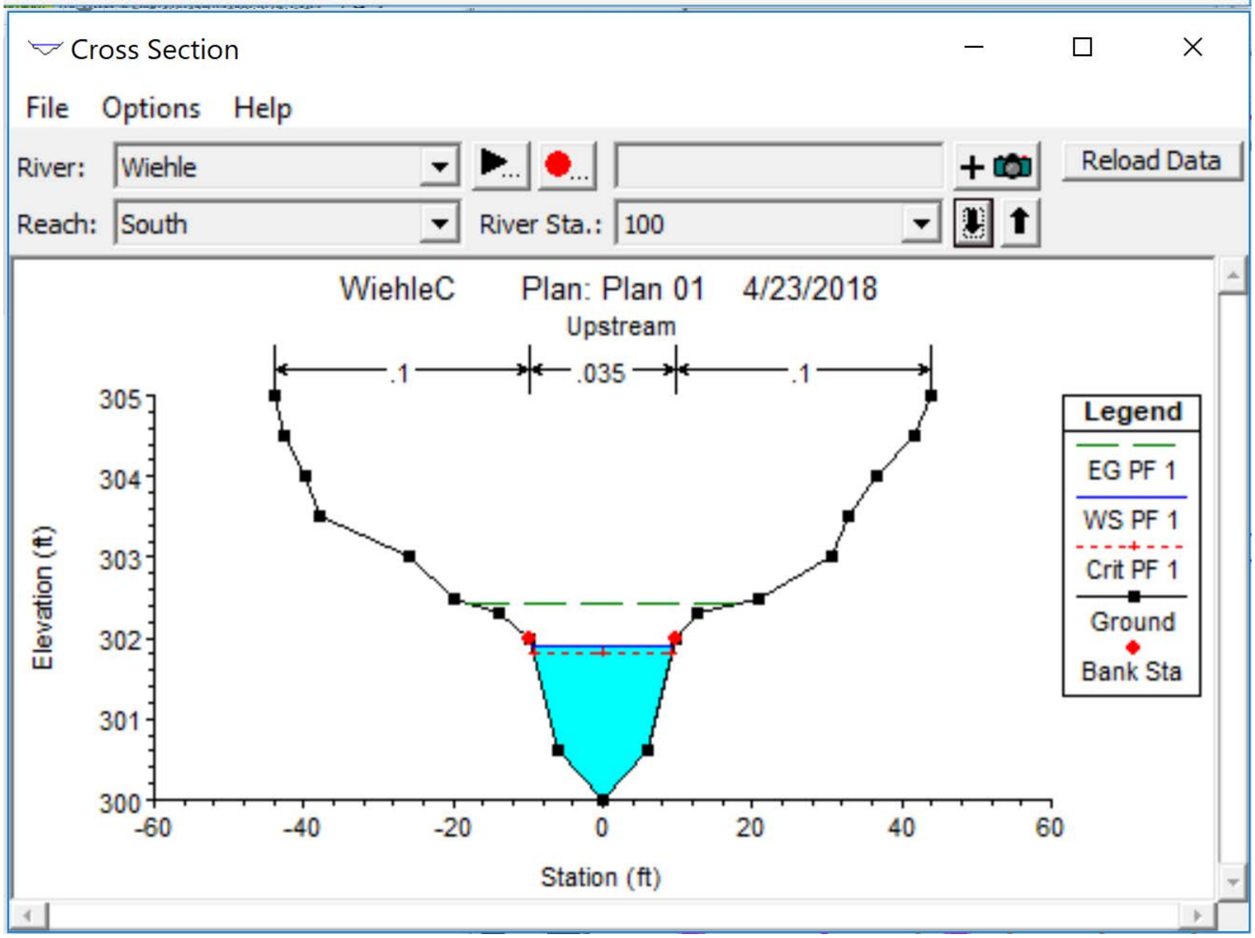
```r
  as.numeric(as.character(restonNEX_GDDPprecip$BFPlus))


sum(restonNEX_GDDPprecip$BFPlus)
plot(restonNEX_GDDPprecip$Date,
     restonNEX_GDDPprecip$BFPlus,
     type = "h")


summary(restonNEX_GDDPprecip8$Prmm)
plot(restonNEX_GDDPprecip8$Date,
     restonNEX_GDDPprecip8$Prmm,
     type = "l")
lm(restonNEX_GDDPprecip8$Prmm ~ restonNEX_GDDPprecip8$Date)
abline(lm(formula = restonNEX_GDDPprecip8$Prmm ~ restonNEX_GDDPprecip8$Date),
       col = "red")


restonNEX_GDDPprecip8$Year <-
  format(as.Date(restonNEX_GDDPprecip8$Date, format = "%d/%m/%Y"), "%Y")


Y8 <- ts(restonNEX_GDDPprecip8$Prmm, frequency = 365)
Yr8 = 2018:2097
Ymax8 <- aggregate(Y, FUN = max)
Ymaxin8 <- Ymax8 * 0.0393701
summary(Ymaxin8)
# Use rainfall intensity for flow calcs


plot(
  Yr8,
  Ymaxin8,
  type = "l",
  xlab = "Year",
  ylab = "1 Year Storm Intensity (in/day)",
  main = "Projected Precipitation RCP 8.5"
)
abline(lm(formula = Ymaxin8 ~ Yr8), col = "red")
plot(lm(Y8 ~ restonNEX_GDDPprecip8$Date))
# R squared / R value


#Min temp c 8.5
Tmin8 <- read.csv("Tmin8.csv")
Tmin8$Date <- as.Date(Tmin8$Date, format = "%m/%d/%Y")
Tmin8$Tmin <- ((1.8 * (Tmin8$Value - 273.15)) + 32)
Tmin8$TminC <- (Tmin8$Value - 273.15)


#Max temp c 8.5
Tmax8 <- read.csv("Tmax8.csv")
```

```r
Tmax8$Date <- as.Date(Tmax8$Date, format = "%m/%d/%Y")
Tmax8$Tmax <- ((1.8 * (Tmax8$Value - 273.15)) + 32)
Tmax8$TmaxC <- (Tmax8$Value - 273.15)


summary(Tmax8$Tmax)
plot(
  Tmax8$Date,
  Tmax8$Tmax,
  type = "l",
  xlab = "Year",
  ylab = "Daily Max Temperature (F)",
  main = "Projected Max Temperature RCP 8.5"
)
abline(lm(formula = Tmax8$Tmax ~ Tmax8$Date), col = "red")


#VSA85 <- Lumped_VSA_model(dateSeries = restonNEX_GDDPprecip8$Date,      P = restonNEX_GDDPprecip8$Prmm,
percentImpervious = 27,
#                          Tmax = Tmax8$Tmax, Tmin = Tmin8$Tmin, latitudeDegrees=38.96, Tp = 5.8, Depth = 2010,
#                          SATper = 0.27, AWCper = 0.13, StartCond = "avg")


#summary(VSA85$modeled_flow)
#plot(VSA85$Date, VSA85$modeled_flow, type="l")
#abline(lm(formula = VSA85$modeled_flow ~ VSA85$Date), col="red")
# Convert to cfs
```

E. HEC-RAS Analysis




Cross Section Data - WiehleC
Exit Edit Options Plot Help
River: Wiehle
Reach: South
River Sta.: 100
Description Upstream
Apply Data
Plot Options
Keep Prev XS Plots
Clear Prev
Plot Terrain (if available)
Cut from Terrain
Del Row
Ins Row
Cross Section Coordinates
Station Elevation
1 -43.75 305
2 -42.75 304.5
3 -39.75 304
4 -37.75 303.5
5 -25.75 303
6 -19.75 302.5
7 -13.75 302.3
8 -9.75 302
9 -6 300.6
10 0 300
11 6 300.6
Downstream Reach Lengths
LOB Channel ROB
10 10 10
Manning's n Values
LOB Channel ROB
0.1 0.035 0.1
Main Channel Bank Stations
Left Bank Right Bank
-9.75 9.75
Cont\Exp Coefficient (Steady
Contraction Expansion
0.1 0.3
WiehleC Plan: Plan 01 4/23/2018
Upstream
.1 .035 .1
Elevation (ft)
Station (ft)
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Bank Sta
Enter to move to next downstream river station location


Cross Section
File Options Help
River: Wiehle
Reach: South
River Sta.: 100
Reload Data
WiehleC Plan: Plan 01 4/23/2018
Upstream
.1 .035 .1
Elevation (ft)
Station (ft)
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Bank Sta

Cross Section Output

File Type Options Help

River: Wiehle Profile: PF 1

Reach South RS: 100 Plan: C2

Plan: C3 Wiehle South RS: 100 Profile: PF 1

E.G. Elev (ft)	302.44	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.54	Wt. n-Val.		0.035	
W.S. Elev (ft)	301.90	Reach Len. (ft)	10.00	10.00	10.00
Crit W.S. (ft)	301.83	Flow Area (sq ft)		23.67	
E.G. Slope (ft/ft)	0.014973	Area (sq ft)		23.67	
Q Total (cfs)	140.00	Flow (cfs)		140.00	
Top Width (ft)	18.95	Top Width (ft)		18.95	
Vel Total (ft/s)	5.92	Avg. Vel. (ft/s)		5.92	
Max Chl Dpth (ft)	1.90	Hydr. Depth (ft)		1.25	
Conv. Total (cfs)	1144.1	Conv. (cfs)		1144.1	
Length Wtd. (ft)	10.00	Wetted Per. (ft)		19.48	
Min Ch El (ft)	300.00	Shear (lb/sq ft)		1.14	
Alpha	1.00	Stream Power (lb/ft s)		6.72	
Frctn Loss (ft)	0.15	Cum Volume (acre-ft)		0.05	
C & E Loss (ft)	0.00	Cum SA (acres)		0.04	

Flow Distribution Output

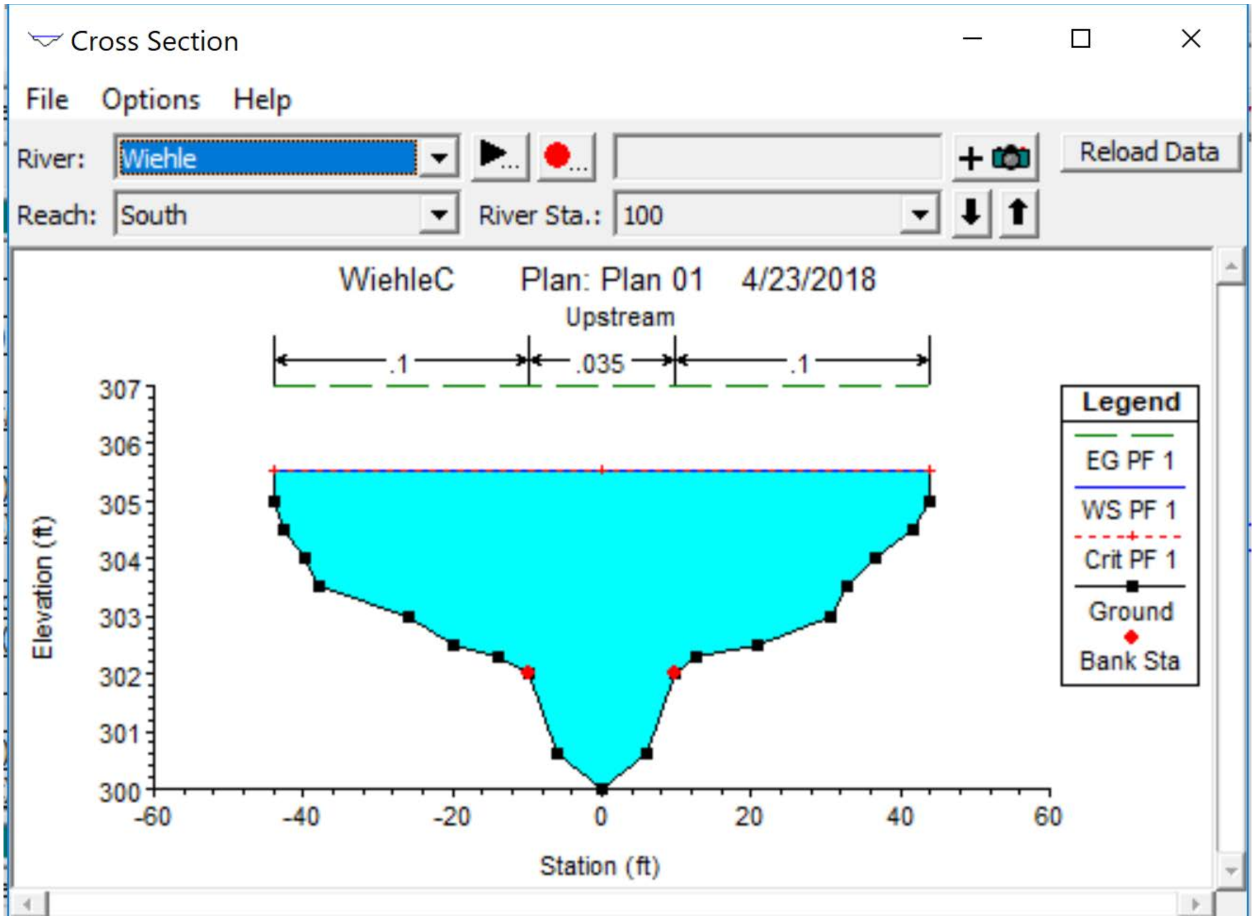
File Type Options Help

River: Wiehle Profile: PF 1

Reach South RS: 100 Plan: C2

Plan: C3 Wiehle South RS: 100 Profile: PF 1

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)	Shear (lb/sq ft)	Power (lb/ft s)
1	Chan	-9.75	9.75	140.00	23.67	19.48	100.00	1.25	5.92	1.14	6.72


Cross Section
File Options Help
River: Wiehle
Reach: South
River Sta.: 100
Reload Data
WiehleC Plan: Plan 01 4/23/2018
Upstream
.1
.035
.1
Elevation (ft)
Station (ft)
307
306
305
304
303
302
301
300
-60
-40
-20
0
20
40
60
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Bank Sta

Cross Section Output

File Type Options Help

River: Wiehle — Profile: PF 1

Reach: South — RS: 100 — Plan: C

Plan: C Wiehle South RS: 100 Profile: PF 1

		Element	Left OB	Channel	Right OB
E.G. Elev (ft)	307.00	Wt. n-Val.	0.100	0.035	0.100
Vel Head (ft)	1.49	Reach Len. (ft)	100.00	100.00	100.00
W.S. Elev (ft)	305.51	Flow Area (sq ft)	84.00	94.18	82.10
Crit W.S. (ft)	305.51	Area (sq ft)	84.00	94.18	82.10
E.G. Slope (ft/ft)	0.009221	Flow (cfs)	215.73	1076.39	207.88
Q Total (cfs)	1500.00	Top Width (ft)	34.00	19.50	34.00
Top Width (ft)	87.50	Avg. Vel. (ft/s)	2.57	11.43	2.53
Vel Total (ft/s)	5.76	Hydr. Depth (ft)	2.47	4.83	2.41
Max Chl Dpth (ft)	5.51	Conv. (cfs)	2246.6	11209.6	2164.9
Conv. Total (cfs)	15621.0	Wetted Per. (ft)	34.78	20.07	34.72
Length Wtd. (ft)	100.00	Shear (lb/sq ft)	1.39	2.70	1.36
Min Ch El (ft)	300.00	Stream Power (lb/ft s)	3.57	30.88	3.45
Alpha	2.88	Cum Volume (acre-ft)	0.19	0.22	0.19
Frctn Loss (ft)	0.92	Cum SA (acres)	0.08	0.04	0.08
C & E Loss (ft)	0.00				

Errors, Warnings and Notes

Warning:	The energy equation could not be balanced within the specified number of iterations. The program used critical depth for the water surface and continued on with the calculations.
Warning:	The cross-section end points had to be extended vertically for the computed water surface.
Warning:	The cross section had to be extended vertically during the critical depth calculations.
Warning:	The energy loss was greater than 1.0 ft (0.3 m). between the current and previous cross section. This may indicate the need for additional cross sections.

Flow Distribution Output

File Type Options Help

River: Wiehle Profile: PF 1

Reach South RS: 100 Plan: C

Plan: C Wiehle South RS: 100 Profile: PF 1

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)	Shear (lb/sq ft)	Power (lb/ft s)
1	LOB		-9.75	215.73	84.00	34.78	14.38	2.47	2.57	1.39	3.57
2	Chan	-9.75	9.75	1076.39	94.18	20.07	71.76	4.83	11.43	2.70	30.88
3	ROB	9.75		207.88	82.10	34.72	13.86	2.41	2.53	1.36	3.45

Errors, Warnings and Notes

Warning:	The energy equation could not be balanced within the specified number of iterations. The program used critical depth for the water surface and continued on with the calculations.
Warning:	The cross-section end points had to be extended vertically for the computed water surface.
Warning:	The cross section had to be extended vertically during the critical depth calculations.
Warning:	The energy loss was greater than 1.0 ft (0.3 m). between the current and previous cross section. This may indicate the need for additional cross sections.

Assignment #5: Wiehle South Concept Plan

Smart Stream Solutions Inc.

We have each personally reviewed and approve the following report:

Cameron Afzal		Date: 5/2/18
Cynthia Chan		Date: 5/2/18
Phillip Duvall		Date: 5/2/18
Ji Young Kim		Date: 5/2/2018
Tanvi Naidu		Date: 5/2/2018
Mike Zarecor		Date: 5/2/2018

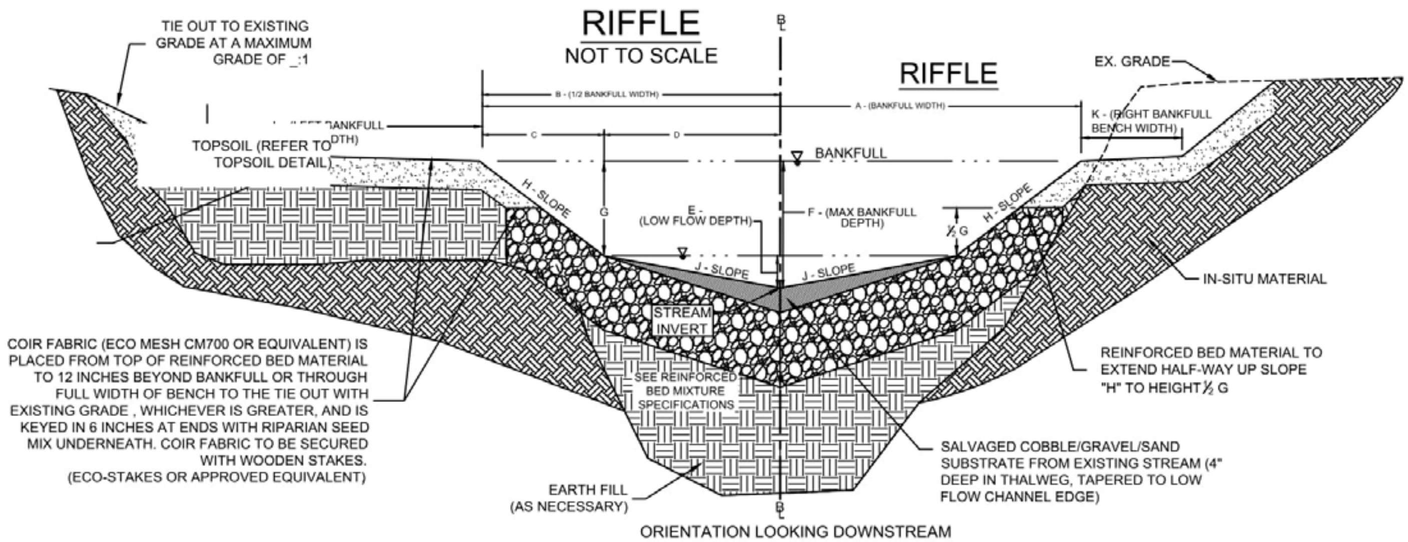
Draft: April 25, 2018

Revised: May 2, 2018

I. Design Element Details

The design of key elements such as riffles, step pools, and imbricated rock walls inform the cost estimation and future redesign Wiehle South reach stream restoration project. To accommodate the 80 cfs lower peak discharge, the team designed a new riffle with a reduced depth and width. This design was iteratively analyzed and redesigned to ensure that erosion in the channel and floodplain would be minimized. The final design, detailed in Assignment 3, involved a 20% reduction in riffle width, and a 10% reduction in riffle depth.

The team applied this modification to other key structures in the Wiehle South plan by reducing all horizontal elements by 20%, and all vertical elements by 10%. This modification was applied to step pools and low flow pools. The dimensions of the imbricated rock walls could not be reduced in a significant way, however, these structures were important in the cost analysis and are thus included in this report. Diagrams for the riffle, step pools, and low flow pools, and imbricated rock walls are displayed below, along with updated dimension.



SECTION LOCATION			RIFFLE CROSS-SECTION DIMENSIONS[2]											
REACH ID	PER BASELINE STATION[1]		CROSS-SECTION PARAMETER (FT)											
	FROM	TO	A	B	C	D	E	F[1]	G	H[3]	J[3]	K[4]	L[4]	NOTES
WIEHLE SOUTH	10+00.00	22+24.00	16.5	8.25	3.0	5.25	0.5	1.5	1	3.0:1	10.2:1	*	*	* VARIES - SEE GRADING PLAN

RIFFLE NOTES:

[1] REFER TO THE GRADING PLAN AND LONGITUDINAL PROFILE SHEETS FOR PLACEMENT OF ROCK STRUCTURES WITHIN THE RIFFLE SECTIONS SPECIFIED ABOVE.

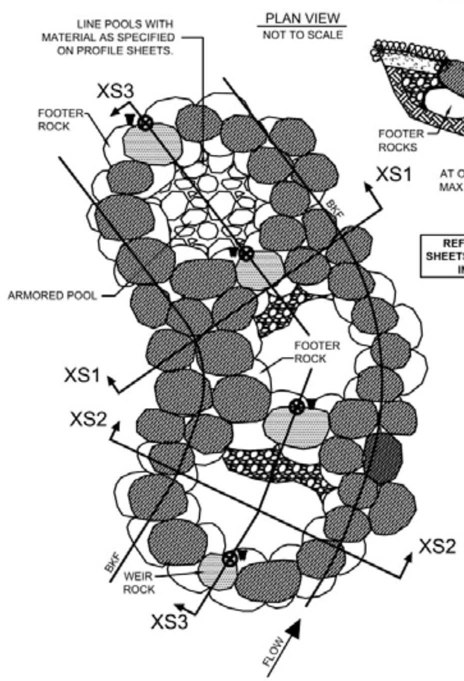
[2] THE "STREAM CROSS SECTION SUMMARY" IS PROVIDED ON THE GRADING AND LONGITUDINAL PROFILE SHEETS. THIS SUMMARY SPECIFIES THE TYPE OF CROSS-SECTIONS AND STRUCTURES THAT SHALL BE CONSTRUCTED ALONG THE PROFILE.

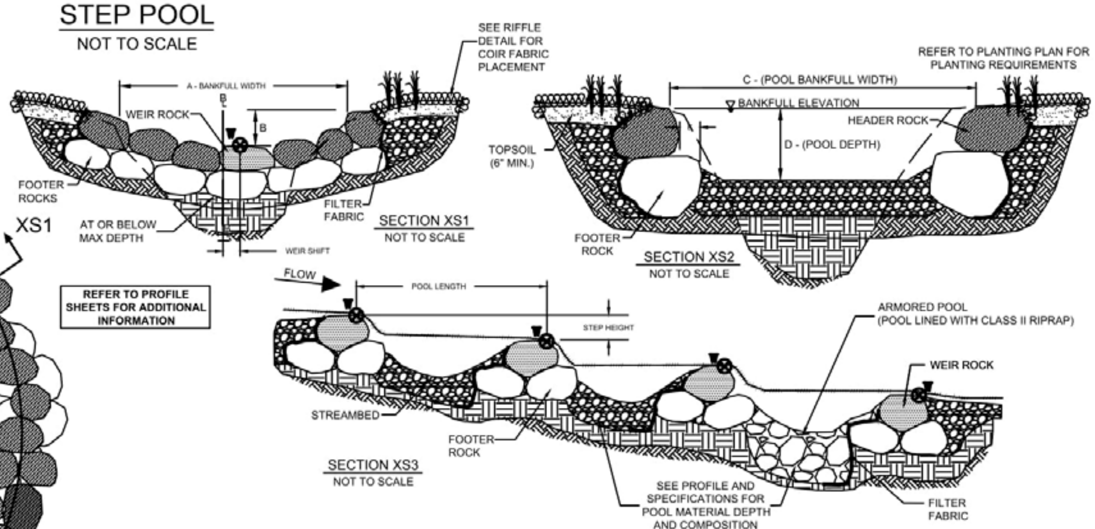
[3] J IS THE SLOPE OF THE SALVAGED SUBSTRATE. VARIABLES "H" & "J" ARE SLOPES EXPRESSED AS HORIZONTAL:VERTICAL (H:V).

[4] TYPICAL VALUES GIVEN IN TABLE ABOVE. VARIABLES MAY BE ADJUSTED TO MINIMIZE DISTURBANCE TO TREES **BUT** THE SPECIFIED SUM OF K AND L MUST BE MET OR EXCEEDED. IF BENCH WIDTHS ARE NOT SPECIFIED OVERBANK AREAS SHOULD BE GRADED WITH A 2% CROSS SLOPE EXTENDING AS FAR AS POSSIBLE WHILE STILL ALLOWING 3:1 (MAX.) TIE OUT SLOPES.

Exhibit 1: Riffle

STEP POOL
NOT TO SCALE





STRUCTURE LOCATION			STEP POOL DIMENSIONS						
REACH ID	PER BASELINE STATION		PARAMETER (FT)					NUMBER OF POOLS	MAX. WEIR SHIFT
	FROM	TO	A	B	C	D	E		
WIEHLE SOUTH	10+03.0	10+87.0	16.5	1.5	14.6	2.7	0.8	3	1.0
	11+30.0	12+11.0	16.5	1.5	15.5	3.2	0.8	3	1.0
	12+60.0	13+45.0	16.5	1.5	16.5	3.2	0.8	3	1.0
	14+00.0	14+99.0	16.5	1.5	16.5	3.2	0.8	3	1.0
	15+85.0	16+41.0	16.5	1.5	15.0	3.2	0.8	2	1.0
	16+80.0	17+33.0	16.5	1.5	17.9	3.2	0.8	2	1.0
	17+50.0	18+13.0	16.5	1.5	16.5	3.2	0.8	2	1.0
	18+50.0	18+75.0	16.5	1.5	17.0	3.2	0.8	1	1.0
	19+10.0	19+99.0	16.5	1.5	18.4	3.2	0.8	3	1.0
	20+90.0	21+18.0	16.5	1.5	16.5	3.2	0.8	1	1.0
	21+50.0	22+24.0	16.5	1.5	16.5	3.5	0.8	3	1.0

STEP POOL NOTES:

1. REFER TO "STRUCTURE NOTES" FOR ADDITIONAL INFORMATION CONCERNING ALL STRUCTURES.
2. SEE GRADING PLAN AND/OR PROFILE FOR WEIR POINT LOCATIONS AND ELEVATIONS.
3. ALL WEIR POINTS ARE LOCATED ON THE DOWNSTREAM EDGE OF THE WEIR ROCK.
4. THE WEIR HEADER ROCK MAY BE CENTERED ON THE BASELINE OR SHIFTED TO EITHER SIDE BY NO MORE THAN THE MAX WEIR SHIFT SPECIFIED IN THIS DETAIL. CENTER WEIR ROCKS SHOULD BE SHIFTED TO FORM AN IRREGULAR FLOW PATTERN FROM POOL TO POOL.

DESIGN DROP HEIGHT (FT)	TOLERANCE (FT)	COMMENT
0.0 - 0.3	± 0.1	NO POOL REINFORCEMENT REQUIRED UNLESS OTHERWISE SPECIFIED
0.3 - 0.7		POOL LINED WITH REINFORCED BED MIX
> 0.7		ARMORED POOL (LINED WITH CLASS II RIPRAP)

Exhibit 2: Step Pool

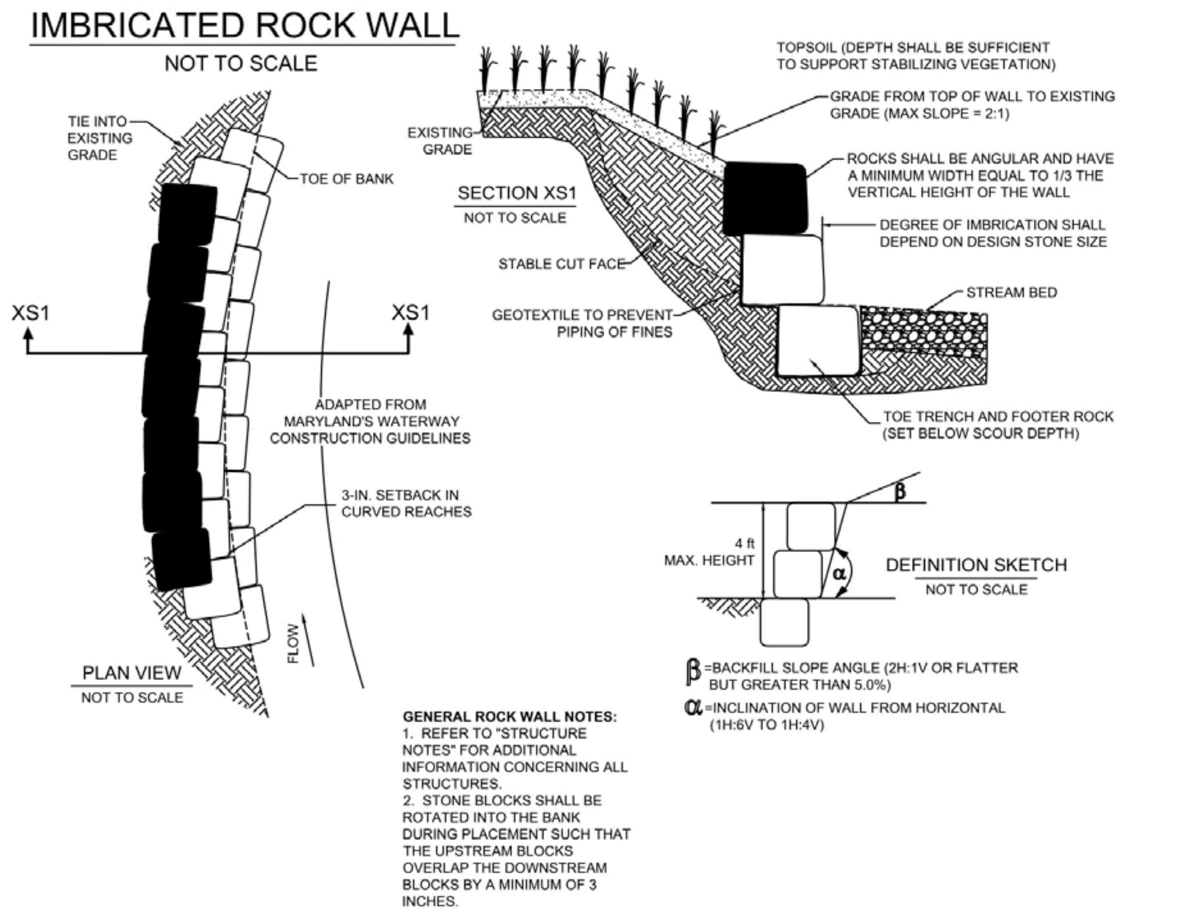


Exhibit 3: Imbricated Rock Wall

III. References

Northern Virginia Stream Restoration Bank Wiehle South, Sheet 39: Reference Reach Data, Prepared by Wetland and Stream Solutions Inc. 4/23/17, Last revised: 1/12, 69 sheets

Assignment #6: Wiehle South Cost Estimation

Smart Stream Solutions Inc.

We have each personally reviewed and approve the following report:

Cameron Afzal		Date:	5/9/2018
Cynthia Chan		Date:	5/9/18
Philip Duvall		Date:	5/9/2018
Ji Young Kim		Date:	5/9/2018
Tanvi Naidu	Tanvi Naidu	Date:	5/9/2018
Mike Zarecor	Michael Zarecor	Date:	5/9/2018

Draft: April 26, 2018

Revised: May 2, 2018

Final Submitted: May 9, 2018

I. Executive Summary

Smart Stream Solutions (S^3) was hired to restore the Wiehle South stream reach in Reston, Virginia. In previous reports S^3 came up with a new riffle design which is smaller than the previous designs, which will reduce construction costs while still being stable. In this report, the team revised a WSSI budget proposal made in 2016. Most of the revisions were made to reflect the change in the riffle design, which became smaller with a 1.5 ft channel depth and a 3:1 cross-sectional channel slope. A formal budget proposal letter was written and submitted along with this document.

As Part IV, Figure 5 below presents, S^3 was able to conserve project execution expenses by decreasing major budget on labor and rock.

Line Item	Initial Price	Final Price	Price Reduction
Tree Clearing	$90,000	$0	-$90,000
Labor & Equipment	$365,600	$292,480	-$73,120
Bridge	$45,000	$0	-$45,000
Rock	$308,974	$146,477	-$162,497
Fees & Re-Design	$0	$81,250	$81,250
Total Cost	$1,753,102	$1,449,138	-$303,964
Price per SCU	$168	$139	-$29

Part IV, Figure 5 - Final Price Reduction

The decreased dimensions of the riffle and step pools led to less stone and fewer person hours of labor. Additionally, extra $90,000 dollars was saved due to double counting of tree removal, and $45,000 was saved due to Restona Homeowners Association, RA's decision to accept the existing bridge. As a consequence, the new estimate for Wiehle South restoration is $1,449,138 total and $139/SCU, which is below the targeted budget of $140/SCU.

II. Introduction

As noted in prior S^3 reports, the design storm impacts the design discharge. S^3 found that for the Wiehle South reach stream restoration design the channel should convey the 1-yr bankfull storm with a discharge of 80 cfs. When engineering stream restoration, it is critical to consider what processes have caused the degradation. Previous reports detailed how the urbanization of the Reston watersheds increased the flow rate the Wiehle South stream must convey. This increased flow has caused significant channel erosion because design discharge dictates channel morphology, impacting flow velocity and boundary shear stress. Therefore, S^3 designed a new riffle with a shallower depth and smaller width-to-depth ratio, to contain and withstand the effects of a 1-yr bankfull discharge event buffered by a floodplain which can appropriately accommodate a 100-yr and 500-yr event with flow velocities within an allowed limit to prevent erosion.

In the process of updating the cost for the decreased tonnage, reverse engineering was done based on multiple documents, including "Northern Virginia Stream Restoration Bank Wiehle South", Sheet 7: Construction Sequence and Construction Details, Prepared by Wetland and Stream Solutions Inc. 4/23/17, Last revised: 1/12, 69 sheets. WSSI also provided two past appraisal Excel spreadsheet documents made by Total Development Solutions LLC in 2016: "Total Development Solutions LLC Proposal," prepared by Susanna Headly from Total Development Solutions LLC. 6/23/2017, Excel spreadsheet file, TDS Proposal in short, and "Exhibit 2: Costs to Rebuild Remaining Reaches", prepared by Susanna Headly from Total Development Solutions LLC. 4/21/2016, Excel spreadsheet file, Exhibit 2 in short.

These documents provided baseline prices on services including construction, monitoring, and maintenance. Administrative and MBI category includes activities on project management, permit renewals, accounting, IRT coordination, PM, Sales, and commissions to Friends of Reston, Reston Association and The Peterson Companies. The estimated contingency budget is also included in the document. Expenses on purchasing materials, at which this report focuses the most, are included in construction category.

Our main objectives of this assignment are to decrease the Price per Stream Condition Unit ($/SCU). For Wiehle South, it was once proposed to be $168/SCU. Other previously bids on constructed stream section projects were commissioned at the average of $145/SCU. However, a leading professional, Michael Rolband in WSSI directed attention that past projects were done high uncertainty in return on investment (ROI), which depends on the selling price of SCUs. S^3 aims to keep the Price per Stream Condition below $140/SCU so that we can offer a viable option to WSSI and be commissioned by WSSI.

III. Methods

A. Updating previous documents

Previous cost estimations for Wiehle South were generated in 2016 and were worked out over two or more independent spreadsheets tabulating costs. In reviewing these prior estimates as a reference for the updated cost estimate, a glaring error was found in the cost tabulation. The TDS proposal tabulated the majority of projects expenses related to site work, including a $90,000 estimate for “Clearing,” shown below:

14	**Clearing**	Clearing	1.00	ls	$82,500.00	$82,500.00
15		Cut and Split Logs	1.00	ls	$7,500.00	$7,500.00
16					**Subtotal**	**$90,000.00**

Figure 1 - Clearing Costs on TDS Proposal

The TDS Proposal, which had a grand total of $745,202.85, was then included as a “Site Work” expense in the document Exhibit 2. Exhibit 2 tabulated an expanded scope of costs, including construction, monitoring, maintenance, administrative work, and more. The goal of this Exhibit 2 is to give the most accurate prediction of project costs for the entire timeline of the project, ultimately estimating the most essential metric for project feasibility - the price per stream conditioning unit ($/SCU). The price per SCU for Wiehle South was significantly higher than other stream reaches which put it out of consideration for building - in the S^3 team’s opinion, it was suspiciously high.

Upon examining Exhibit 2, it was discovered that the $90,000 estimate for “clearing” was counted on both Exhibit 2 and the TDS Proposal as independent charges. This means that the project is anticipating spending $90,000 more than it actually will. Below are shown a few line items in Exhibit 2 in its construction category, including the previously mentioned $745,203 from the TDS proposal, and the $90,000 clearing charge, which is apparently being counted twice.Fixing this error alone saves WSSI $90,000 and decreases the price per SCU from $168 to $158.

Task	Average $/lf	Wiehle South
A. Construction		
1. Construction - Tree Clearing	Bid	$90,000
- Grubbing	Bid	$13,500
2. Construction-Site Work	Bid	$745,203
3. Construction-Rock	Bid	$308,974

Figure 2 - Exhibit 2's double-counted "clearing" charge

An additional decrease in price came with the removal of the "Bridge" line-item. A decade ago when Reston was first considering this project, the bridge at the confluence of Wiehle North and Wiehle South was planned to be replaced. Recently, however, the Reston Homeowners Association (RA) has agreed to leave the bridge in place, leading to two major decreases in cost. One flat-rate decrease is in the price of the bridge - initially priced at $45,000, and now $0. The second cost cut is a significant decrease in Class II rock. This type of rock is used solely around sewer crossings and under bridges. Given that no sewer lines cross the Wiehle South reach, and that the bridge is being completely removed, no Class II rock is needed. The initial estimated cost of Class II rock was $82,698, and is now $0.

B. Re-assessing Person-Hours

According to the TDS Proposal, human labor and equipment rentals compose approximately half of all "site-work" related costs, coming in at an estimated $365,600. These charges account for hourly contracts for foremen, laborers, and equipment operators, as well as rental hours of Bobcats, 'Track Trucks,' excavators, and water pumps. Given the nature of the construction workflow, a section of stream can only be restored as quickly as it can be dug out, the material hauled in, and rocks placed across the section of the stream. This, according to a leading professional in the field, would imply that any percent reduction in the width of a stream's cross-section would lead to a comparable reduction in person-hours of labor and rentals.

The updated riffle and step-pool designs outlined in S^3's Concept Plan Report are scaled 80% of the original Wiehle South design width. Applying this same scaling factor to the human labor and equipment rental costs by 80% leads to estimated labor and rental reduction from $365,600 to $292,480, and a total decrease of the TDS proposal from $745,202 to $670,986.

There is a significant increase in cost for person-hours when considering the engineering re-design and associated fees. WSSI budgeted $50 per linear foot for estimating the cost of this re-designing, which leads to an estimated $61,250 in engineering work alone. Additionally, $5,000 was budgeted for obtaining a permit from the Army Corps, $10,000 for obtaining a permit through Fairfax, and $5,000 for working with the Design Review Board for review, RA meetings, and other administrative work. Overall, the cost of re-designing and re-submitting an new engineering concept plan is budgeted at $81,250. This is approximately an $8/SCU charge.

C. Decreasing Rock Tonnage

The second largest price on the Exhibit 2 cost estimation sheet is the price for rock. Exhibit 2 lists three different classes of rock priced - reinforced bed, structural rock, and Class II rock. Reinforced bed mix is a combination of A1 riprap, gravel, sand, and topsoil, and serves as the foundational streambed in riffles and step pools. Structural rock is typically placed surrounding step pools, x-vanes, and other structures that require extra fortification. Class II rock generally is only used surrounding gas and sewer pipeline crossings and under bridges.

As the width of the riffle and step pools decreased by 20% and the height by 10%, the rocks and reinforced bed must also decrease in volume - the question is how much. The approach taken by the S^3 team was to estimate an average volume of rock needed to create a standard step pool and standard riffle, and then extrapolate that across the length of the design reach.

Step pools were estimated to be perfect cylinders, with reinforced bed making up the circular base of the cylinder and structure rock forming the sidewalls of the cylinder. Using geometry and the dimensions found in S^3's Concept Plan, the volume of the structure rock and the reinforced bed was determined for a single step pool. This price was then multiplied by the number of pools along the reach.

The riffle was analyzed to determine the area of reinforced bed placed in a standard cross-section. This value was then multiplied by the total length of riffle to be constructed according to the WSSI plan set. No structural rock is assumed to be used in these standard riffles. Further notes on assumptions made and calculations can be found in the appendix.

Step Pool Reinforced Bed			
R =	Step Pool Radius	7.25	ft
D =	Reinforced Bed Depth	1.50	ft
V =	Reinforced Bed Volume	248	ft^3
W =	Reinforced Bed Weight	19	ton
P =	Reinforced Bed Price	$ 706	/pool

Step Pool Structure Rock			
D_out =	Step Pool Rock Outer Diameter	21.50	ft
D_in =	Step Pool Rock Inner Diameter	16.50	ft
V =	Structure Rock Volume	701	ft^3
W =	Structure Rock Weight	66	ton
P =	Structure Rock Price	$ 3,541	/pool

Riffle Reinforced Bed			
X =	Riffle cross section area	25	ft^2
L =	Riffle length within reach	506	ft
V =	Reinforced Bed Volume	12650	ft^3
W =	Reinforced Bed Weight	948.75	ton
P =	Reinforced Bed Price	$ 36,053	

Combined Weights / Pricing		
Step Pool Reinforced Bed Weight	483	ton
Step Pool Structure Rock Weight	1705	ton
Step Pool Total Price	$ 110,425	
Riffle Total Price	$ 36,053	
Total Rock Price	$ 146,477	

Figure 3 - Riffle and reinforced bed calculations

Comparing between the 2016 Exhibit 2 values and the newly estimated values for the volume and price of rocks reveals interesting observations. As seen below, the amount of structure rock has been reduced by over 50%. This is due to the conservative nature of the previous plan set, in which all step pools with terrain above a certain grade were fortified with massive amounts of structure rock as their base instead of the reinforced bed. Leading engineers at WSSI have determined through experience that this sort of reinforcement is overly conservative and overly costly, so newer designs show step pools with reinforced bed mix - which is significantly less expensive and leads to fewer tons of rock per step pool.

Estimated Rock Quantities

Wiehle South - Original

Delivered Rock Costs

Type	Amount	Unit Cost	Total	$/lf
Reinforced Bed	1700	$38	$64,600	$53
Structure Rock	2994	$54	$161,676	$132
Class II	2506	$33	$82,698	$68
		Total	$308,974	$252

Wiehle South - Redesign

Estimated Rock Costs

Type	Amount	Unit Cost	Total	$/lf
Reinforced Bed	1432	$38	$54,407	$44
Structure Rock	1705	$54	$92,070	$75
Class II	0	$33	$0	$0
		Total	$146,477	$120

Figure 4 - Comparative rock volumes and prices, 2016 and present

The reinforced bed is reduced by 16% overall. The design team has determined that this value is due to a 30% decrease in the reinforced bed due to riffle downsizing, and additional reinforced bed being added to step pools.

Even with conservative estimates made in the updated cost profile, the new amount of rock to be used is less than the previous values, with a price drop of over $160,000. This reduction can be primarily contributed to the removal of the conservative rock armoring of step pools, as structure rock has the highest unit cost. The reduction in price per SCU based on rock material is $16.

IV. Results

Given the three-part reductions in cost from various sources, Wiehle South has potential to be a productive and affordable stream restoration project. The reductions in the following table show the dramatic improvement in affordability granted by re-envisioning and re-designing the WSSI Wiehle South project.

Line Item	Initial Price	Final Price	Price Reduction
Tree Clearing	$90,000	$0	-$90,000
Labor & Equipment	$365,600	$292,480	-$73,120
Bridge	$45,000	$0	-$45,000
Rock	$308,974	$146,477	-$162,497
Fees & Re-Design	$0	$81,250	$81,250
Total Cost	$1,753,102	$1,449,138	-$303,964
Price per SCU	$168	$139	-$29

Figure 5 - Final Price Reduction

The new Wiehle South project has an estimated cost of $1,449,138. The reach counts towards 10,408 stream condition units, so the price can be seen as $139 per SCU. This is a $29 per SCU reduction from the original concept design.

V. Conclusion

A cost estimation was completed for the re-designed Wiehle South reach by S3 which shows improvements in affordability. Costs were largely reduced in the two most expensive project areas - labor and rock. The re-designed concept plan which decreased the dimensions of the riffle and step pools led to less stone and fewer person hours of labor, resulting in hundreds of thousands in savings. The double-counting of tree removal in the original cost estimation saved an extra $90,000.

Coming in at $1,449,138 total and $139/SCU, this reach is below the budgeted $140/SCU. Recent selling prices at Mitigation Banks are approaching $200/SCU (less 5% catastrophic event fund--for a net of $190/SCU) making the newly re-designed Wiehle South reach a potential candidate for development.

VI. Appendix

A.1 Cost Estimation Comparison - Original v. Redesign

	2017	Old	New
	Length >	1,225	1,225
	SCU's >	10,408	10,408
Task	**Average $/lf**	**Wiehle South**	**Wiehle South**
A. Construction			
1. Construction - Tree Clearing	Bid	$90,000	$0
- Grubbing	Bid	$13,500	$0
2. Construction-Site Work	Bid	$745,203	$670,986
3. Construction-Rock	Bid	$308,974	$146,477
4. Construction-Planting [1]	Bid	$56,350	$56,350
5. Construction - Bridge[2]		$45,000	$0
Construction Oversight	$67	$81,839	$81,839
Survey Stakeout	$22	$27,018	$27,018
As-Built Survey	$16	$19,347	$19,347
B. Monitoring (10-yr)	$48	$58,800	$58,800
C. Maintenance (10-yr)	$29	$35,525	$35,525
D. Administrative and MBI			
1. Project Management	$11	$13,881	$13,881
2. Permit Renewals	$10	$12,250	$12,250
3. Accounting/IRT Coordination/PM/Sales[3]		$62,448	$62,448
4. Friends of Reston[4]		$17,456	$17,456
5. Reston Association[5]		$9,188	$9,188
6. The Peterson Companies[6]		$46,836	$46,836
7. Redesign Costs	$50	$0	$61,250
E. Fees			
1. Army Corps		$0	$5,000
2. Fairfax		$0	$10,000
3. Design Review Board		$0	$5,000
E. Contingency[7]		$104,080	$104,080
F. Reimbursables	$4	$5,408	$5,408
	TOTAL w Est Rock Quantities	$1,753,102	$1,449,138
	$/lf	$1,431	$1,183
	$/SCU	**$168**	**$139**

A.2 "Site Work" Cost Estimation:

		Modified Pricing 4/26/2018			
	DESCRIPTION OF WORK	***QTY***		***UNIT***	***TOTAL***
Mobilization	Mobilization/ General Conditions	1	ls	$7,500.00	$7,500.00
	TV Sanitary (before and after)	2	ea	$2,500.00	$5,000.00
				Subtotal	**$12,500.00**
Clearing	Clearing	1.00	ls	$82,500.00	$82,500.00
	Cut and Split Logs	1.00	ls	$7,500.00	$7,500.00
				Subtotal	**$90,000.00**
Erosion Controls	**Materials**				
	Construction Entrance	1	ea	$4,500.00	$4,500.00
	Tree Protection	8000	lf	$1.95	$15,600.00
	Super Silt Fence	1000	lf	$5.90	$5,900.00
	Silt Fence As needed	1	ls	$3,500.00	$3,500.00
	Check Dams	9	ea	$250.00	$2,250.00
	Temporary Seeding	6500	sy	$0.40	$2,600.00
	Pedestrian Gate	4	ea	$550.00	$2,200.00
	E and S Maintenance	1	ls	$7,500.00	$7,500.00
				Subtotal	**$44,050.00**
Access Road CR	**Install Deck Mats Construction**				
	Install Deck Matting	1650	lf	$10.00	$16,500.00
	Remove Deck Matting	1650	lf	$10.00	$16,500.00
	Traffic Control	1	ls	$3,750.00	$3,750.00
	Filter Cloth for Deck Matting	3000	sy	$1.00	$3,000.00
	VDOT #1 for Stream Crossings	400	tons	$20.00	$8,000.00
	36"Stream Crossings	270	lf	$100.00	$27,000.00
				Subtotal	**$74,750.00**
Labor and Equip	**Channel Grading, Install Rock Structures, Reinforced Bedding and Stabilize:**				
	Foreman	640	hrs	$56.00	$35,840.00
	Labor	1280	hrs	$28.00	$35,840.00
	Operators	1920	hrs	$35.00	$67,200.00
	Track Bobcat	640	hrs	$30.00	$19,200.00
	CD 110 Track Truck	640	hrs	$60.00	$38,400.00
	JD 200 Excavators	1280	hrs	$70.00	$89,600.00
	Pumps Dewatering	13	wks	$500.00	$6,400.00
				Subtotal	**$292,480.00**

A.3 "Site Work"Cost Estimation Continued

Labor and Equip	**Channel Grading, Install Rock Structures, Reinforced Bedding and Stabilize:**				
	Foreman	640	hrs	$56.00	$35,840.00
	Labor	1280	hrs	$28.00	$35,840.00
	Operators	1920	hrs	$35.00	$67,200.00
	Track Bobcat	640	hrs	$30.00	$19,200.00
	CD 110 Track Truck	640	hrs	$60.00	$38,400.00
	JD 200 Excavators	1280	hrs	$70.00	$89,600.00
	Pumps Dewatering	13	wks	$500.00	$6,400.00
				Subtotal	**$292,480.00**
Materials	**Purchase and Supply following Materials:**				
	CM-700	3600	sy	$2.75	$9,900.00
	Eco Stakes	30	boxes	$135.00	$4,050.00
	Reinforced Bed Material in channel (to be purchased and supplied by WSSI)	1700	tns	$0.00	$0.00
	Filter Fabric	6000	sy	$0.60	$3,600.00
	Wetlands Seed	330	lbs	$15.00	$4,950.00
	Turfgrass	200	lbs	$2.50	$500.00
	In-stream Seed	100	lbs	$2.50	$250.00
	Fertilizer 10-10-10	7600	lbs	$0.40	$3,040.00
	Structure Rock (to be purchased and supp	5500	tns	$0.00	$0.00
	Offsite Disposal (Allowance)	2800	cy	$40.00	$112,000.00
				Subtotal	**$138,290.00**
Trails	8' Asphalt Trail Repair	300	lf	$30.00	$9,000.00
				Subtotal	**$9,000.00**
				Total	**$661,070.00**
P&P Bond ADD	Performance Bond (1.5% of total bid)	1	ls	$9,916.05	**$9,916.05**
				Total	**$670,986.05**

A.4 Rock Quantity & Structure Price Calculations

Step Pool Reinforced Bed			
R =	Step Pool Radius	7.25	ft
D =	Reinforced Bed Depth	1.50	ft
V =	Reinforced Bed Volume	248	ft^3
W =	Reinforced Bed Weight	19	ton
P =	Reinforced Bed Price	$ 706	/pool

Step Pool Structure Rock			
D_out =	Step Pool Rock Outer Diameter	21.50	ft
D_in =	Step Pool Rock Inner Diameter	16.50	ft
V =	Structure Rock Volume	701	ft^3
W =	Structure Rock Weight	66	ton
P =	Structure Rock Price	$ 3,541	/pool

Riffle Reinforced Bed			
X =	Riffle cross section area	25	ft^2
L =	Riffle length within reach	506	ft
V =	Reinforced Bed Volume	12650	ft^3
W =	Reinforced Bed Weight	948.75	ton
P =	Reinforced Bed Price	$ 36,053	

Combined Weights / Pricing		
Step Pool Reinforced Bed Weight	483	ton
Step Pool Structure Rock Weight	1705	ton
Step Pool Total Price	$ 110,425	
Riffle Total Price	$ 36,053	
Total Rock Price	$ 146,477	

Notes on Calculations:

Step Pool Reinforced Bed:

- Radius = (Bankfull width / 2) - 1 [see planset page 7]
- Reinforced Bed Weight = 150 lbs / cf
- Multiply $/pool by # of pools (26) to get total

Step Pool Structure Rock:

- Assuming 0% void space
- Weight = 187 lbs/cf (S.G. = 3)

Riffle Reinforced Bed:

- Weight calculation takes void space of reinforced bed into account

A.5 Rock Pricing

Estimated Rock Quantities

Wiehle South - Original

Delivered Rock Costs

Type	*Amount*	*Unit Cost*	*Total*	*$/lf*
Reinforced Bed	1700	$38	$64,600	$53
Structure Rock	2994	$54	$161,676	$132
Class II	2506	$33	$82,698	$68
		Total	$308,974	$252

Wiehle South - Redesign

Estimated Rock Costs

Type	*Amount*	*Unit Cost*	*Total*	*$/lf*
Reinforced Bed	1432	$38	$54,407	$44
Structure Rock	1705	$54	$92,070	$75
Class II	0	$33	$0	$0
		Total	$146,477	$120

VII. References

Headly, Susanna. "Exhibit 2 Costs to Build Remaining Reaches". 6/23/2016. Microsoft Excel File.

Headly, Susanna. "Total Development Solutions L.L.C Proposal". 4/21/2016. Microsoft Excel File.

Northern Virginia Stream Restoration Bank Wiehle South, Prepared by Wetland and Stream Solutions Inc. 4/23/17, Last revised: 1/12, 69 sheets.